Montana Department
of Environmental Quality

CIRCULAR DEQ-8

MONTANA STANDARDS FOR

SUBDIVISION STORM WATER DRAINAGE

~~2017~~ 2023 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~~ 2017 Edition.

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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~~ subject to review under the Sanitation in Subdivisions Act. 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 an authority (other than the Department)~~ an 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 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.

1.3 DEVIATIONS FROM STANDARDS

The terms shall, must, may not, and require indicate mandatory items, and applicants must obtain approval to deviate from these mandatory requirements. Other items, 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.

Deviations from the requirements of this circular may be granted pursuant to ARM 17.36.601. A request for a deviation must include adequate justification. "Engineering judgment" or "professional opinion" without supporting data is not adequate justification. The justification must address each of the items included in ARM 17.36.601. Additionally, justification must address potential adverse impacts, such as flooding and erosion, to neighboring properties.

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

1.4 DEFINITIONS

In addition to the definitions included in ARM 17.36.101, the following definitions are used in this Circular.

1.2.1. Building Location Area – an area identified on a lot layout where construction of impervious area may occur.

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

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

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

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

1.2.6. **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."

1.2.7. **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.

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

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

1.2.10. **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.

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

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

1.2.13. **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.

1.2.14. **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.

1.2.15. **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.

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

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

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

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

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

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

1.2.22. **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.

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

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

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

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

1.2.27. **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.

1.2.28. **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.

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

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

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

1.2.32. **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."

1.2.33. **Storm Water Facility** means those structures that temporarily hold or convey water as part of storm water management; including retention and detention ponds, infiltration facilities, drainage ditches and storm sewer systems. For the purposes of evaluating setbacks, and depiction on the lot layout, storm water facility does not include building gutters, downspouts, and landscaping.

1.2.34. **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.).

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

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

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

1.2.38. **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~~ an engineering report, ~~and four copies of~~ drawings, specifications, and the operation and maintenance plan as presented in this Chapter.

2.2 REPORT

A copy of a storm water drainage design report must be included with each application and must include the ~~following:~~ information included in this section.

2.2.1 SIMPLIFIED PLAN REPORT:

For applications that meet the requirements of Section 3.2 for a simplified plan the following information must be included in the engineering report:

A. The name of the subdivision;

B. Qualifying criteria 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; and
   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 existing and proposed storm water drainage facilities;

F. Calculations supporting facility size and design per subchapter 3.2.

G. A description of the routing of the 100-year peak flow through the proposed retention pond.

2.2.2 STANDARD PLAN REPORT:

For standard plans, the following information must be included in the engineering report:

A. The name of the subdivision;

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

B. 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.;

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

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

E. ~~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

F. Calculations supporting facility size and design.

G. Calculations showing peak flow, at the time of concentration from all offsite basins impacting the site for the following storm events: (1) 2-year storm event; (2) 10-year storm event; and (3) 100-year storm event.

H. Figures supporting design assumptions such as profile sections, on-site and off-site basins, time of concentration, and contour maps if required in this circular or requested by the reviewing authority.

2.3 DRAWINGS

All applications must show existing and proposed storm water facilities on the lot layout documents as required in ARM 17.36.104.

~~Four sets of storm~~ Applications that require storm_water drainage design by a professional engineer, per ARM 17.36.310, must submit drawings ~~must be submitted and must include the following:~~ as presented in this section, unless design details and the professional engineer's seal and signature can legibly be shown on the lot layout.

At least one copy of the storm drainage design drawings must be submitted with the application. Prior to final approval, three copies of the final design drawings must be submitted.

2.3.1 GENERAL LAYOUT:

A site plan submitted under this section must include the following information where applicable.

A. ~~The~~ name of the subdivision;

B. ~~A~~ north arrow and scale;

C. ~~The name and affiliation of the person who prepared the plan;~~ professional engineer's seal and signature;

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, wells, drainfields, and utilities;

H. Existing building and driveway locations,

I. Proposed building and driveway locations or proposed building location area and numerical quantity of impervious area in square feet;

J. Locations, sizes, and design details of existing and proposed storm water ~~structures~~ facilities;

K. Locations of drainage ways;

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

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

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

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

2.3.2 DETAILED PLANS:

Detailed plans for proposed stormwater facilities must be submitted when required by this circular or requested by the reviewing authority.

~~2.4~~ ~~CONSTRUCTION DOCUMENTS~~ TECHNICAL SPECIFICATIONS

When technical specifications cannot be clearly identified on the design drawings or lot layout, ~~Four sets~~ at least one copy of complete, detailed, technical specifications ~~may be required for the construction of complex storm water drainage facilities.~~ must be submitted with the application. Prior to final approval, three copies of the final design drawings must be submitted.

2.5 OPERATION AND MAINTENANCE PLANS

~~Four sets of~~ an operation and maintenance plan must be submitted. ~~and must include:~~ If the information required in this section can legibly be included on the lot layout, separate operation and maintenance plans may not be required. Operation and maintenance plans must include the following:

A. Procedures for the long-term operation and maintenance of facilities; and

B. Designation of the party responsible for the overall management and implementation of the operation and maintenance. The reviewing authority may require the applicant to create a homeowner's association or other legal entity that will be responsible for maintenance of storm drainage structures and that will have authority to charge; and

C. Easement information as necessary to ensure continued access to storm water drainage facilities meeting the requirements of ARM 17.36.122.

~~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.~~

~~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.~~

2.6 COMPUTER MODELS

Computer models may be used for storm water designs. When using computer models the following information must be provided with the application:

A. Computations and assumptions for the model, such as layout;
B. hydraulic and hydrologic methods used;
C. input parameters;
D. a detailed summary of model outputs within the design report

If computer modeling software is used for complex drainage designs such as storm sewer networks, storm inlet capture/bypass calculations, multiple sub-basins computations, detention facilities with multiple discharge structures, or routing; the following information must be provided when requested by the reviewing authority.

E. schematic diagrams used for all routing; and
F. inflow-outflow hydrographs.

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.~~ Each storm water design must be submitted as either a Simplified Plan or Standard Plan as presented in this Chapter.

All storm water designs must include an initial storm water facility as required by Section 3.4.

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 B and D.

A Simplified Plan may be used only if all of the following criteria are satisfied for every lot in an application. Additional information must be provided if requested by the reviewing authority.

A. The total number of lots subject to review is five or fewer;

~~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.

D. The increase in storm water runoff will be retained on the lot where it is generated.

E. Roadway or driveway construction will not cross off-site drainages with a contributing basin area of more than 5 acres.

~~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.~~

The minimum retention pond volume must be 250 cubic feet per 1,000 square feet of impervious area. The minimum retention pond size using this methodology is 750 cubic feet. If a lesser volume of storage is proposed, the retention volume must be calculated using the spreadsheet provided in Appendix B.

Retention facilities for Simplified Plan submissions must be designed in accordance with Chapter 5 of this circular.

Conveyance structures should be minimal for simplified plans. Roadway or driveway construction for simplified plans must include a minimum 12-inch diameter culvert where roadside ditches or drainages are crossed. Culverts should be installed at a minimum 2% grade and must not be less than the local county road standards minimum grade. Sizing calculations for culvert crossing must be provided if requested by the reviewing authority.

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 design for a Standard Plans are provided in Appendices C and E.

Standard Plans must address storm water drainage peak flow and volume ~~in accordance with Appendix B~~. Standard Plans must demonstrate that the post-development runoff flowrate from the proposed subdivision will not ~~allow storm water to do any of the following:~~ exceed the pre-development runoff flowrate during the 2-year storm event. For subdivision exceeding the pre-developed flowrate, the increased volume of runoff must be either detained, retained, or infiltrated in accordance with Chapters 5, 6, or 7.

~~A. Exceed the pre-development runoff to an adjoining property during the 2-year storm event; or~~

~~B. Overtop roadways or driveways during a 10-year storm event; and~~

~~C. Inundate any buildings or drain fields during a 100 year storm event. This may be demonstrated through either narrative descriptions or calculations.~~

Storm water runoff and conveyance for standard plans must demonstrate compliance with the requirements of Chapter 4.

Standard plans must not inundate buildings or drainfields during a 100-year storm event. This may be demonstrated narratively. When requested by the reviewing authority supporting calculations must be provided.

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. Landscaping (existing or proposed), or the initial abstraction in the SCS Method, may not be used to satisfy the initial storm water facility volume.

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)

For standard plans only, best management practices (BMPs) designed in accordance with the guidance provided in the most recent version of the Montana Post Construction Storm Water BMP Design Manual are allowed to capture the initial storm water facility volume. The BMPs must be designed specifically for the treatment and removal of 80 percent total suspended solids (TSS) from storm water runoff, which include infiltration basins, bioretention, biofiltration swales, extended detention basins, wet detention basins, and proprietary treatment devices. Permeable pavement systems and dispersion BMPs are not allowed as the initial storm water facility.

~~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 certificate of subdivision approval under the Sanitation in Subdivisions Act, the pre-development runoff must be calculated based on natural/undeveloped conditions. ~~For rewrite applications under ARM 17.36.112, the pre-development runoff may be calculated based on the approved developed conditions.~~ Predeveloped conditions may include state and county highways and roads existing prior to the date of the application.

For parcels with an existing certificate of subdivision approval, the pre-development runoff must be based on the previously approved use of the parcel.

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 proposed impervious area is known, the ~~proposed~~ post-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.~~

When the extent and location of post-development impervious area is not known, the applicant must identify a building location area, a stormwater facility construction area, and a numerical estimate of the proposed impervious area on the lot layout and storm water design drawings. The estimated impervious area should be conservative and must be consistent with similar developments in the area of the project. The stormwater facility construction area must be located separate from the building location area and where runoff will naturally occur. The stormwater facility construction area must meet all applicable setbacks in ARM 17.36.323.

For lots with a certificate of subdivision approval, relocating storm water facilities outside of an approved stormwater facility area or impervious area outside of a building location area may be done with a revised lot layout as outlined in ARM 17.36.112 if the conditions in that rule are met.

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;

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

~~An IDF curve at the time of concentration; or~~

B. Chapter 9, Appendix B-2022 of the Montana Department of Transportation Hydraulics Manual https://www.mdt.mt.gov/other/webdata/external/hydraulics/manuals/Chapter-09-Hydrology.pdf; or,

C. Other sources approved by the reviewing authority.

When using rainfall information to calculate peak runoff rates, the rainfall intensity (i) must be determined at the time of concentration of the drainage basin using an Intensity-Duration-Frequency (IDF) curve applicable to the location of the development. The Montana Department of Transportation Hydrology Manual 2022 – Chapter 9 Appendix B has tabulated rainfall depths at varying storm durations for selected locations across the state that may be used to develop an IDF curve.

The time of concentration must be determined per Section 3.7.5. The minimum time of concentration is 5 minutes.

3.7 ACCEPTABLE METHODS

Storm water volume and flow rates must be computed ~~in accordance with Appendix B~~ using the hydraulic and hydrologic methods presented in this section. Other methods may be used upon approval by the reviewing authority.

3.7.1 RATIONAL METHOD

The Rational Method may be used for calculating peak flow rate and volume 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 for a storm with a duration equal to the time of concentration (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.

When using the rational method to determine the peak runoff rate from a drainage area, the rainfall intensity (i) must be determined at the time of concentration using an Intensity-Duration-Frequency (IDF) curve per Section 3.6.

The time of concentration must be determined using the methodology presented in Section 3.7.5. The minimum time of concentration is 5 minutes.

The rational method may also be used to estimate the volume of runoff from a drainage area for design of retention facilities. When using the rational method to estimate the volume of runoff, the following equation must be used:

$$V = C * d * A$$

Where: V = volume of runoff (ft^3)
C = runoff coefficient per Section 3.7.1.A (unitless)
d = rainfall depth for 24-hour storm event (inches)
A = Area (acres)

The example spreadsheets in Appendices B and C use the Rational Method to calculate volume and flow.

3.7.2 MODIFIED RATIONAL METHOD

The Modified Rational Method uses the peak flow rate (Q) calculated using the Rational Method equation and the time of concentration to create a synthetic hydrograph that estimates the runoff from a drainage area for a given storm duration (d). This methodology can be used to develop a runoff hydrograph to calculate the required storage volume of a detention facility.

Figure 1 presents example hydrographs for storm durations equal to, and greater than the time of concentration. The total volume of runoff is calculated by determining the area of the trapezoidal hydrograph for storm durations greater than the time of concentration, or the area of the triangular hydrograph in the case of the storm duration equal to the time of concentration.

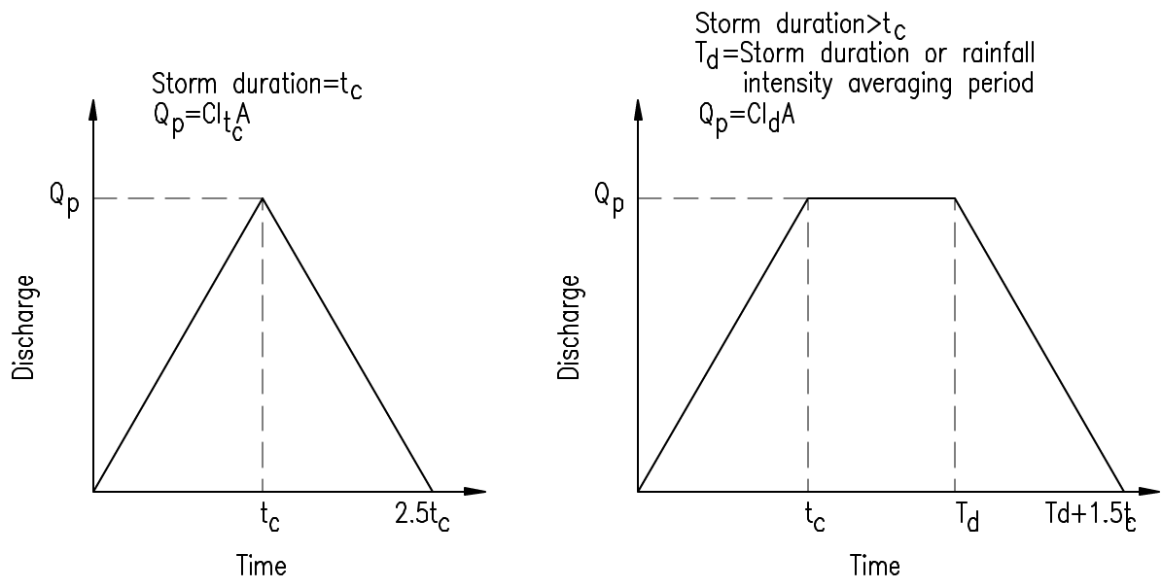


Figure 1: Modified Rational Method Inflow Hydrographs

The modified rational method must use the critical storm duration to calculate the storage volume required for a detention facility. The critical storm duration is the storm duration that generates the greatest storage volume required. To determine the critical storm duration, multiple storm durations equal to and greater than the time of concentration must be analyzed. The storm durations should be selected to correspond with intensity-duration-frequency curves available in the project area. Interpolation of rainfall intensities may be necessary.

The required detention facility storage must be determined by calculating the area between the inflow hydrograph generated using the modified rational equation and the outflow hydrograph. The outflow hydrograph must be determined using the stage-discharge relationship of the outlet structure or estimated using a straight line starting at Q = 0 and t = 0 as shown in Figure 2. The duration of the receding limb of the inflow and outflow hydrographs must be 1.5 times the time of concentration.

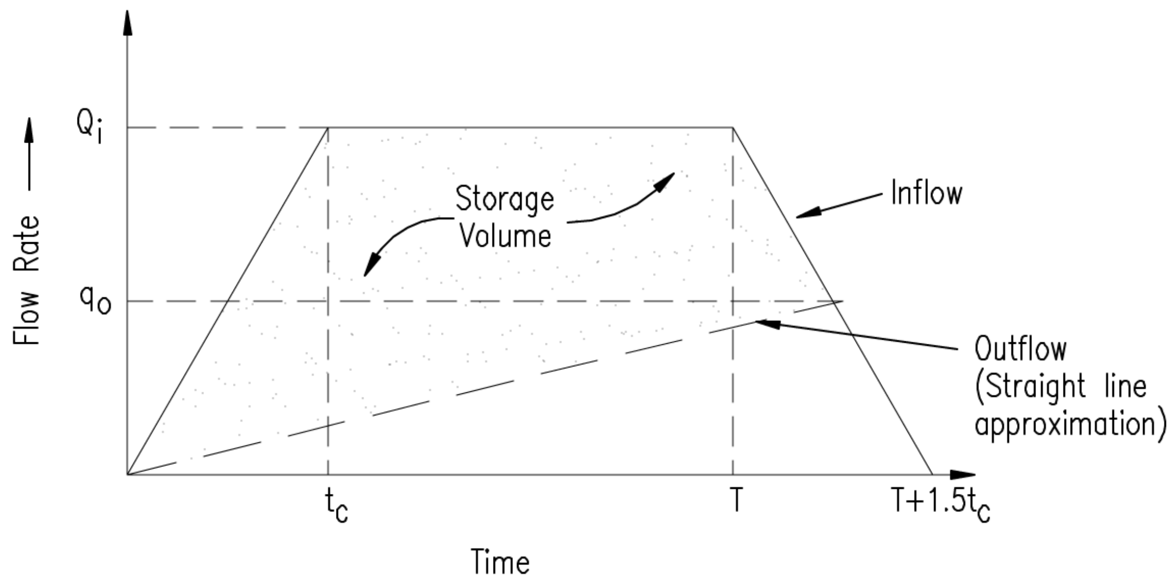


Figure 2: Modified Rational Method Inflow Hydrograph, Straight-line Outflow Hydrograph, and Required Storage Volume shown between the curves

Calculations must show the critical storm duration results in the largest area between the inflow and outflow hydrographs. When the critical duration storm event is longer than the time of concentration, calculations must show a minimum of one event longer and one shorter than the critical duration storm.

The storage volume between the inflow and outflow hydrographs for storm durations equal to or greater than the time of concentration, as shown in Figure 2, can be calculated with the following equation:

$$V = 60 * [Qi * \left(T + \frac{tc}{4}\right) - \frac{qo}{2} * \left(T + \frac{3}{2}tc\right)]$$

Where: V = storage volume (ft^3)
Qi = runoff flowrate at storm duration T (cfs)
qo = pre development peak runoff (cfs)
T = storm duration (min)
tc = time of concentration (min)

An example of the modified rational method discussed in this section is included in Appendix F.

3.7.3 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. TR-55 may not be used for drainage areas larger than 3 square miles.

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 depth (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 time of concentration must be determined using the methodology presented in Section 3.7.5. The minimum time of concentration is 5 minutes.

B. The hydrologic group as provided by the NRCS Soil Survey for each soil type, vegetation/land use, and slope of the site must be known.

C. The soil type curve number must be computed using the values shown in Table 1, as a weighted average of the site conditions. Typical curve number values may also be obtained from the TR-55 Bulletin.

D. The initial abstractions may not exceed more than 50% of the total precipitation ($I_a/P < 0.50$). If $I_a/P > 0.5$ then a value of 0.5 must be used to determine the unit peak discharge factor.

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

Table 1. SCS Runoff Curve Number Table

	CNs for Hydrologic Soil Group			
Cover Type and Hydrologic Condition	**A**	**B**	**C**	**D**
Open Space (lawns, parks, golf courses, cemeteries, landscaping, etc.) 1/				
Fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Good condition: grass cover on >75% of the area	39	61	74	80

Impervious Areas				
Open water bodies: lakes, wetlands, ponds, etc.	100	100	100	100
Paved parking lots, roofs, driveways, etc. (excluding right of way)	98	98	98	98
Streets and Roads				
Paved; open ditches (including right of way)	83	89	92	93
Gravel (including right of way)	76	85	89	91
Dirt (including right of way)	72	82	87	89
Pasture, Grassland, or Range – Continuous Forage for Grazing				
Fair condition: ground cover 50% to 75% and not heavily grazed	49	69	79	84
Good condition: ground cover >75% and lightly or only occasionally grazed	39	61	74	80
Cultivated Agricultural Lands				
Row Crops (good), e.g., corn, sugar beets, soy beans	64	75	82	85
Small Grain (good), e.g., wheat, barley, flax	60	72	80	84
Meadow (continuous grass, protected from grazing, and generally mowed for hay)	30	58	71	78
Brush (brush-weed-grass mixture, with brush the major element)				
Fair: 50% to 75% ground cover	35	56	70	77
Good >75% ground cover	30[2/]	48	65	73
Woods				
Fair: woods are grazed but not burned, and some forest litter covers the soil	36	60	73	79
Good: woods are protected from grazing, and litter and brush adequately cover the soil	30[2/]	55	70	77
Herbaceous (mixture of grass, weeds, and low-growing brush, with brush the minor element) 3/				
Fair: 30% to 70% ground cover		71	81	89
Good: >70% ground cover		62	74	85
Sagebrush With Grass Understory 3/				
Fair: 30% to 70% ground cover		51	63	70
Good: >70% ground cover		35	47	55

F. 1/ CN's shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

G. 2/ Actual curve number is less than 30; use CN = 30 for runoff computations.

H. 3/ Curve numbers have not been developed for Group A soils.

3.7.4 STORAGE-INDICATION ROUTING

Storage-indication routing may be used to calculate detention or infiltration facility volumes.

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)

Inflow values in this method must be based on an inflow hydrograph developed using one of the methods listed in this chapter (modified rational method or SCS Curve Number (TR-55)).

Outflow values in this method must be based on a stage-discharge relationship for detention facility outlet structures and soil absorption rates as described in Chapter 6 and 7.

The storage indication routing method requires establishing a time step at which runoff storage in the facility will be calculated. The time step used in this method is dependent on the basin characteristics. Generally, the shorter the time step, the more accurate. The time step used in the storage-indication routing method may not be greater than the time of concentration.

3.7.5 TIME OF CONCENTRATION

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.).

Time of concentration may be calculated using the NRCS TR-55 methodology presented in this section. A spreadsheet using the TR-55 formulas, is available on the Department's website that may be submitted for time of concentration calculations.

Other accepted methodologies for calculating time of concentration may be allowed. If time of concentration was determined using a methodology other than that presented in this section, the engineering report must include a description of the procedure and its applicability to the application.

The minimum time of concentration that may be used in storm water calculations is 5 minutes.

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. Since development includes the construction of impervious areas such as buildings and gravel or pavement driveways, the post-development time of concentration is typically shorter than the pre-development time of concentration.

For applications that require time of concentration calculations, a figure must be included with the engineering report showing the flow path(s) used to determine the pre-development and post development time of concentration.

3.7.5.1 SHEET FLOW

Sheet flow is rainfall runoff over flat surfaces at a relatively uniform depth, typically in the upper regions of a drainage basin. Sheet flow generally occurs over relatively short distances. Sheet flow length must be limited to a maximum of 300 feet, as most sheet flow becomes shallow concentrated flow at greater flow lengths.

Time of concentration for sheet flow must 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)

3.7.5.2 SHALLOW FLOW

After a maximum of 300-feet, sheet flow usually becomes shallow flow. Shallow flow consists of shallow rivulets of concentrated runoff following sheet flow. The length of shallow concentrated flow of runoff is dependent on the basin configuration. Time of concentration for shallow flow must 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

The velocity used to calculate the travel time for shallow concentrated flow is dependent on the slope and surface cover of the watercourse. The velocity must be obtained from TR-55 or estimated using the equations below:

Unpaved: $V = 16.1345 * \sqrt{s}$

Paved: $V = 20.2382 * \sqrt{s}$

Where: V = average velocity, ft/s
S = slope of hydraulic grade line (watercourse slope), ft/ft

These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, ft). For unpaved areas, n is 0.05 and r is 0.4; for paved areas, n is 0.025 and r is 0.2.

3.7.5.3 CONCENTRATED FLOW

Concentrated or channel flow time must be calculated using accepted hydraulic equations for the ditch, stream, culvert or structure used to convey the runoff.

~~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.~~

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.~~

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.~~

~~PEAK FLOW~~

SIMPLIFIED PLAN

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

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.~~

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.~~

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~~ drainfields/soil absorption systems during a 100-year storm event.

Conveyance structures should be included to prevent overtopping driveways for single family living units.

Conveyance structures must be designed to meet the following during the 10-year storm event:

A. Flow depth at culvert crossings must not over-top roadways
B. Flow depth at culvert crossing must not over-top driveways that provide access to a commercial unit or three or more single family living units
C. Flow depth at culvert crossings over the driveway must not exceed a flow depth of 2-inches for less than three single family living units
D. Cross-pans or valley pans at roadway intersections must not exceed a flow depth of 2-inches.
E. Flow depth and spread in roadside ditches or curb/gutter/inlets must maintain a 10-foot-wide lane for emergency access, 5-foot either side of the crown or median.
F. parking lots must be designed such that a maximum flow depth of 2-inches is not exceeded

Flow rates must be calculated in accordance with Chapter 3 at the time of concentration.

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, plan view, and the slope or profile 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.~~ Storm sewer pipe network designs must include inlet and storm drainpipe capacity calculations. Designs must address potential surcharging. Energy losses at inlets, transitions, and manholes should be considered. Hydraulic grade lines must be provided when requested by the reviewing authority.

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 1.

Table 1. Minimum Grades to Ensure 3 fps for Full Flow (ft/ft)

Pipe Size (in)	Q Full Flow (cfs)	Grade (ft/ft)
12	2.36	0.0037
15	3.68	0.0028
18	5.30	0.0022
21	7.22	0.0018
24	9.43	0.0015
27	11.93	0.0013
30	14.73	0.0011
33	17.82	0.00097
36	21.21	0.00086

Applications utilizing storm sewer networks must provide detail drawings for each run of pipe showing profiles of proposed storm sewer conveyance structures 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.

~~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. Culverts should be at least 12-inch diameter. Culverts less than 12-inch in diameter may be used for driveways and pathways; and must not be greater than 20 feet in length and include provisions describing the need for increased maintenance.

Culvert capacity and velocity calculations must be provided. Type of control assumed in the calculations (inlet, barrel, and/or outlet controls) must be documented. The Federal Highway Administration, *"Hydraulic Design of Highway Culverts*," Hydraulic Design Series No. 5 FHWA-NHI-01-026, Washington, DC, April 2012, may be used as a reference.

Calculations including culvert inverts, roadway elevations, and headwater and tailwater elevations for both the 10-year and 100-year storm events may be required for more complex designs when required by the reviewing authority.

Adequate protection from erosion must be provided at culvert outlet structures when the outlet velocity exceeds 10 feet per second.

The following design details for culverts must be shown on the lot layout or plans.

- diameter
- slope
- length

Additional design details may be required by the reviewing authority for complex designs. Details include, but not limited to, invert and roadway elevations, profile views, inlet and outlet structures, and erosion protection.

~~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.~~

~~RETENTION AND DETENTION FACILITIES~~

5. RETENTION 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.~~

~~**RETENTION FACILITIES**~~

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

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 minimum design storm for retention facilities must be the 2-year 24-hour storm for standard plans and the 100-yaer 24-hour storm for simplified plans. The capacity of a retention facility may be used to satisfy the minimum volume requirement for an Initial Storm Water Facility~~, as described~~ provided in Chapter 3.

If infiltration is used to reduce the required storage volume, the retention facility must be sized in accordance with the requirements of Chapter 7.

Retention storm water facilities must be constructed at locations where the increased runoff from impervious areas will naturally accumulate or where runoff can be directed.

If roadside ditches or swales are proposed to serve as retention facilities, the design must include check dams or other methods to ensure required volume is retained. The size and spacing of check dams must be provided to show that the required storage will be available. Check dams must not hinder the ability of the roadside ditch to convey the 10-year rainfall event without overtopping the roadway or driveway.

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.

Retention facilities must be designed to:

A. Include side slopes no steeper than 3 H to 1 V and must be stabilized;
B. Be constructed such that the bottom of the facility is one foot above the seasonally high groundwater or bedrock layer;
C. A maximum depth of four feet, unless signage is provided, and side slopes are constructed no steeper than 4 H to 1 V or fencing is included;

If the reviewing authority has reason to believe that groundwater at the proposed retention facility will be within two feet of the bottom of the pond, groundwater monitoring may be required. Monitoring must be performed in accordance with the procedure presented in Department Circular DEQ-4.

Retention facilities must include considerations for routing 100-year peak flows without damaging adjacent or down-gradient buildings. An emergency overflow should be included and must be provided when required by the reviewing authority. Emergency overflow structures must be designed with a stabilized transition from the retention facility to down-gradient swales.

The following design details for proposed retention facilities must be shown on the lot layout or plans.

- volume,
- depth,
- top and bottom dimensions,
- side slopes

Additional design details may be required by the reviewing authority for complex designs. Details include, but not limited to, contour maps, basin cross sections, inlet structures, or emergency overflow structure details.

6. DETENTION FACILITIES

6.1 GENERAL

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 calculated at the time of concentration.

Detention storm water facilities must be constructed at locations where the increased runoff will naturally accumulate or where runoff can be directed.

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 21 and Figure 22 in Appendix O.~~ Outflows from a detention pond are controlled by an outlet structure such as drop inlets, pipes, weirs, orifice plates.

~~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.~~

The following design details for proposed detention facilities must be shown on the lot layout or plans.

- volume,
- depth,
- top and bottom dimensions,
- side slopes
- outlet structure dimensions and elevation or distance above pond bottom,

Additional design details may be required by the reviewing authority for complex designs. Details may include, but not limited to, contour maps, basin cross sections, inlet and outlet structure plan, profile, and section views; and emergency overflow structure details.

6.2 DESIGN

Detention facilities must be designed to provide:

A. Side slopes no steeper than 3 H to 1 V and must be stabilized;
B. A minimum of one foot separation from the bottom of the facility to the seasonally high groundwater or bedrock layer with the exception described in this section;
C. A maximum depth of four feet, unless signage is provided, and side slopes are constructed no steeper than 4 H to 1 V or fencing is included;

If the reviewing authority has reason to believe that groundwater at the proposed retention facility will be within two feet of the bottom of the pond, groundwater monitoring may be required. Monitoring must be performed in accordance with the procedure presented in Department Circular DEQ-4.

If the seasonal high groundwater level is above the bottom of the pond, the facility must be designed in accordance with the wet detention basin procedure included in the Montana Post-Construction Storm Water BMP Design Guidance Manual.

Detention facilities must include considerations for routing 100-year peak flows without damaging adjacent or down-gradient buildings. An emergency overflow should be included and must be provided when required by the reviewing authority. Emergency overflow structures should be designed with a stabilized transition from the retention facility to down-gradient swales.

6.3 CALCULATIONS

The storage volume, above the outlet structure invert, must be determined by limiting the peak discharge release rate to the predevelopment flow rate during the 2-year event. Calculations must be performed using the Storage Indication Method, Modified Rational Method in Section 3.7, or another method approved by the Department. The volume below the invert of the outlet structure must meet the requirements of Section 3.4.

The following calculations must be provided for each detention facility:

A. Stage-storage relationship for the detention facility;
B. Outlet structure, weir, or orifice equations, references, coefficients and assumptions used;
C. Emergency outlet sizing calculations for the 100-year storm event;
D. Inflow and outflow hydrographs when requested by the reviewing authority;
E. Reservoir routing calculations when requested by the reviewing authority;

7. INFILTRATION FACILITIES

7.1 GENERAL

~~Infiltration facilities must be sized in accordance with Chapter~~ 3. Infiltration facilities include subsurface features such as chambers, drainage sumps, french drains, boulder pits, catch basins, dry wells, and surface features such as ~~lawns and infiltration trenches~~ basins and 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 23 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.

Infiltration facilities may not be used in Simplified Plans. Infiltration facilities must be sized to maintain pre-development runoff volume in accordance with Section 3.3.

Infiltration facilities must be constructed at locations where the increased runoff will naturally accumulate, or where runoff can be directed.

7.2 DESIGNS

Infiltration facilities must be sized based on soil textures obtained in a test pit dug in accordance with Section 7.4. Infiltration rates used in design must be based on those rates presented in Appendix A and the soil texture encountered at the depth of the facility.

Infiltration facilities may not be constructed where soil textures are clay loam, silty clay loam, or finer.

The infiltration facility capacity ~~may also~~ must 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:

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

A. Be constructed two feet above the seasonal high groundwater level or bedrock layer;

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

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

D. Be sized to drain within 48 hours; and

E. Include a pre-treatment facility, designed in accordance with Chapter ~~7~~ 8, where sediment, trash, debris, or organic materials are likely to impact the operation or maintenance of the infiltration facility.

7.3 CALCULATIONS

The required storage volume of infiltration facilities must be calculated as presented in this section.

The minimum storage volume of infiltration facilities must be the difference between the pre- and post-development runoff volume from the 2-year 24-hour storm event. The footprint of the infiltration facility must be based on the infiltration rate in Appendix A and the required maximum storage time per Section 7.2.D.

All infiltration facilities must include consideration for larger storm events. Excess runoff at the infiltration facilities must be directed in a way to prevent inundating buildings and drainfields during the 100-year storm event. This requirement may be narratively addressed. Calculations must be provided when requested by the reviewing authority.

7.4 TEST PIT REQUIREMENTS

At least one test pit is required per 5,000 square feet of basin infiltration surface. The test pit must be located within 25 feet of the proposed infiltration facility. The test pit location must be shown on the lot layout. Test pits more than 25 feet from a proposed infiltration facility, or a reduced number of test pits, may be allowed if test pits have been completed for at least 25 percent of the proposed infiltration facilities and soils and depth to seasonal high groundwater can be shown to be consistent across the project area.

Test pit logs must include:

A. Depth of pit;

B. Soil descriptions including NRCS textural class for each soil horizon in observed in the pit;

C. Depth to seasonally high groundwater or other limiting layer;

D. Evidence of mottling and/or redoximorphic features;

E. Estimated coarse fragment/gravel percentages; and

F. Depth of at least 2 feet below infiltration facility base.

8. PRE-TREATMENT FACILITIES

8.1 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 must be provided for infiltration facilities and are recommended for detention and retention facilities. 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.

8.2 DESIGN

8.2.1 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.

8.2.2 VEGETATED SWALES

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

8.2.3 SCREENS

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

8.2.4 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.

8.2.5 PROPRIETARY SPINNERS/SWIRL CHAMBERS/CENTRIFUGES

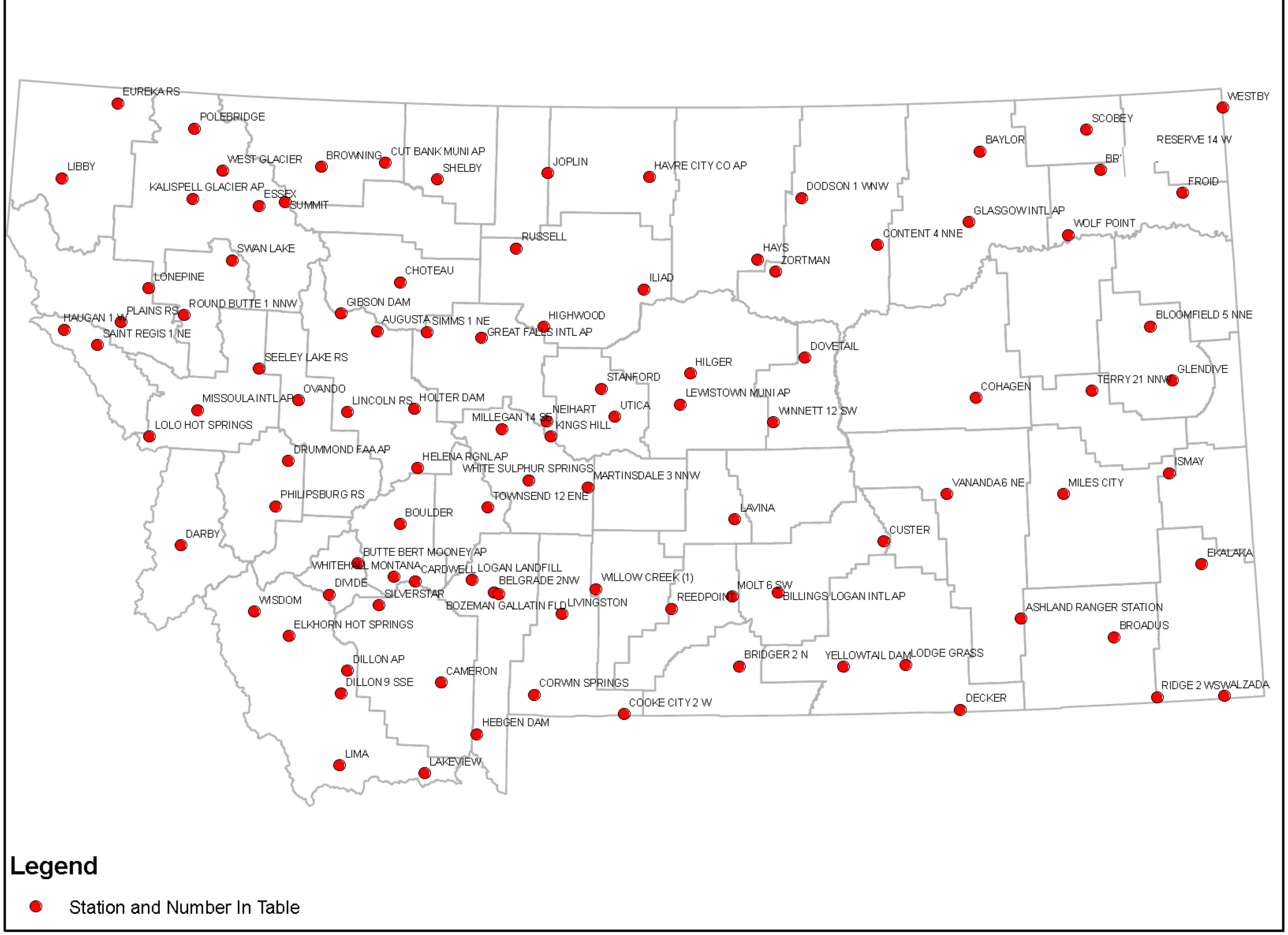
Proprietary spinners/swirl chambers/centrifuges cause storm water to move in a circular motion that enhances the settling of sediments, removes particulates, oils/greases, floatable sands, and debris. These must be installed in accordance with manufacturer specifications.

8.2.6 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.

~~RAINFALL DATA~~

102 Stations Listed In DEQ 8



~~Table 2. 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~~

~~ACCEPTABLE HYDROLOGIC MODELS AND TIME OF CONCENTRATION~~

~~METHODS~~

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:~~

$$\cancel{Q = C * i * A}$$

~~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:~~

~~B. 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.~~

~~C. 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.~~

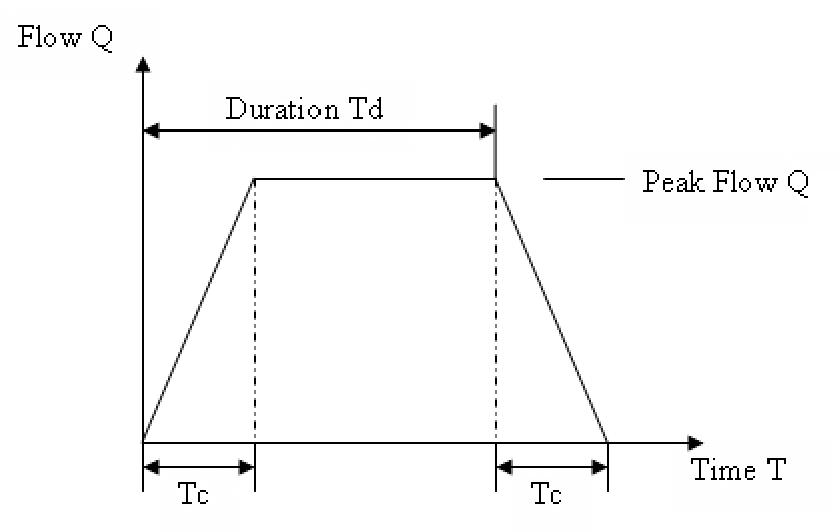
~~The Modified Rational Method is used to calculate volume of storm water runoff using the flow (Q) calculated using the Rational Method equation.~~

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

$$\cancel{V = T_D * Q}$$

~~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.~~

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:~~

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

~~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:~~

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

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

K. ~~The minimum time of concentration is 5 minutes.~~

L. ~~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.~~

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

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

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

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:~~

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

~~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)~~

~~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:~~

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

~~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:~~

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

~~Where: T_t = travel time (hr),~~
~~L = flow length (ft, max of 300 ft)~~
~~V = velocity, ft/s (see Figure 20)~~

~~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 A - INFILTRATION TESTING PROCEDURES

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

A. Design Infiltration Rate in A.1; or
B. Encased Falling Head Test in A.2.

A.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 3. Infiltration Rates

Texture	Infiltration rate (inches per hour)
Gravel, gravelly sand, or very coarse sand (c)	~~2.6~~ 4.0
Loamy sand, coarse sand (d)	~~1.05~~ 1.6
Medium sand, sandy loam	~~0.9~~ 1.4
Fine sandy loam, loam	~~0.7~~ 1.1
Very fine sand, sandy clay loam, silt loam	~~0.7~~ 1.1
Clay loam, silty clay loam	~~0.07~~ 0.1
Sandy clay	~~0.07~~ 0.1
Clays, silts, silty clays (e)	0.0

A.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.

A.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.

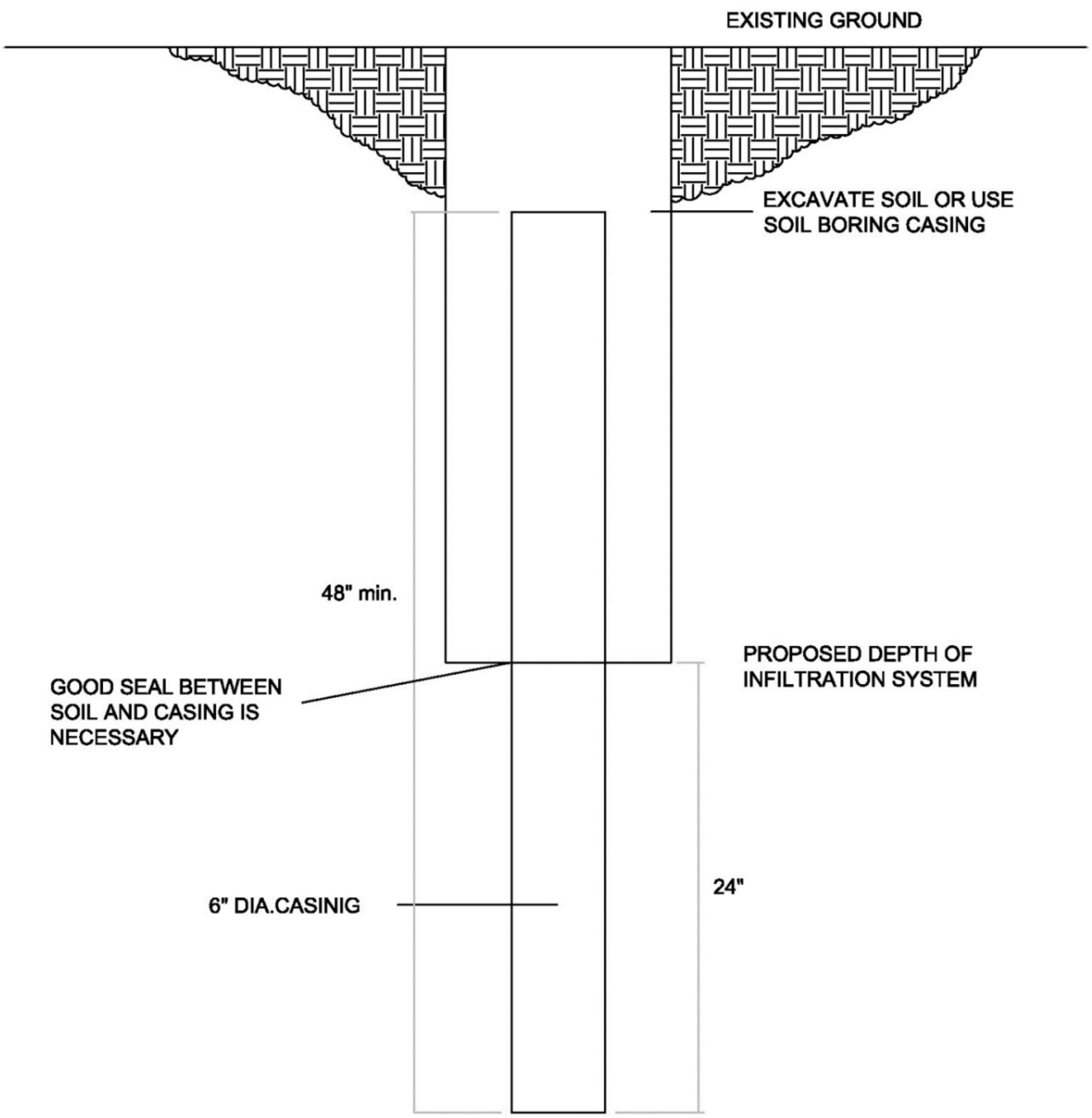


Figure 2. Encased Falling Head Schematic

~~DETENTION OUTLET STRUCTURE EQUATIONS~~

~~A.4 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)~~

~~A.5 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)~~

~~CONVEYANCE STRUCTURE EQUATIONS~~

~~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 (ft²)~~
~~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 (ft²)~~

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

~~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)~~

~~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 4. 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 B - 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
100-year, 24-hour, i		inches	
Total Area/Lot Size		acres =	0 ft^2

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

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 C - 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 ft²

Initial Stormwater Facility Volume (0.5" x Impervious Area): 0 ft³

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	ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Gravel Area	0 acres	ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Lawn/Landscaping	0 acres	ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Unimproved Area	0 acres	0 ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Total	0 acres	0 ft²	Q_{Total}= **0.000** ft³/sec	V_{Total}= **0.000** ft³	Q_{Total}= **0.000** ft³/sec	Q_{Total}= **0.000** ft³/sec	V_{Total}= **0.000** ft³

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	ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Gravel Area	0 acres	ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Lawn/Landscaping	0 acres	ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Unimproved Area	0 acres	0 ft²	Q= 0.000 ft³/sec	V= 0.000 ft³	Q= 0.000 ft³/sec	Q= 0.000 ft³/sec	V= 0.000 ft³
Total	0 acres	0 ft²	Q_{Total}= **0.000** ft³/sec	V_{Total}= **0.000** ft³	Q_{Total}= **0.000** ft³/sec	Q_{Total}= **0.000** ft³/sec	V_{Total}= **0.000** ft³
Runoff Flow/Volume Change			ΔQ= **0.000** ft³/sec	ΔV= **0.000** ft³	ΔQ= **0.000** ft³/sec	ΔQ= **0.000** ft³/sec	ΔV= **0.000** ft³

Required Minimum Facility Volume: **0** ft³

= input field

INITIAL STORM WATER FACILITY EXAMPLE

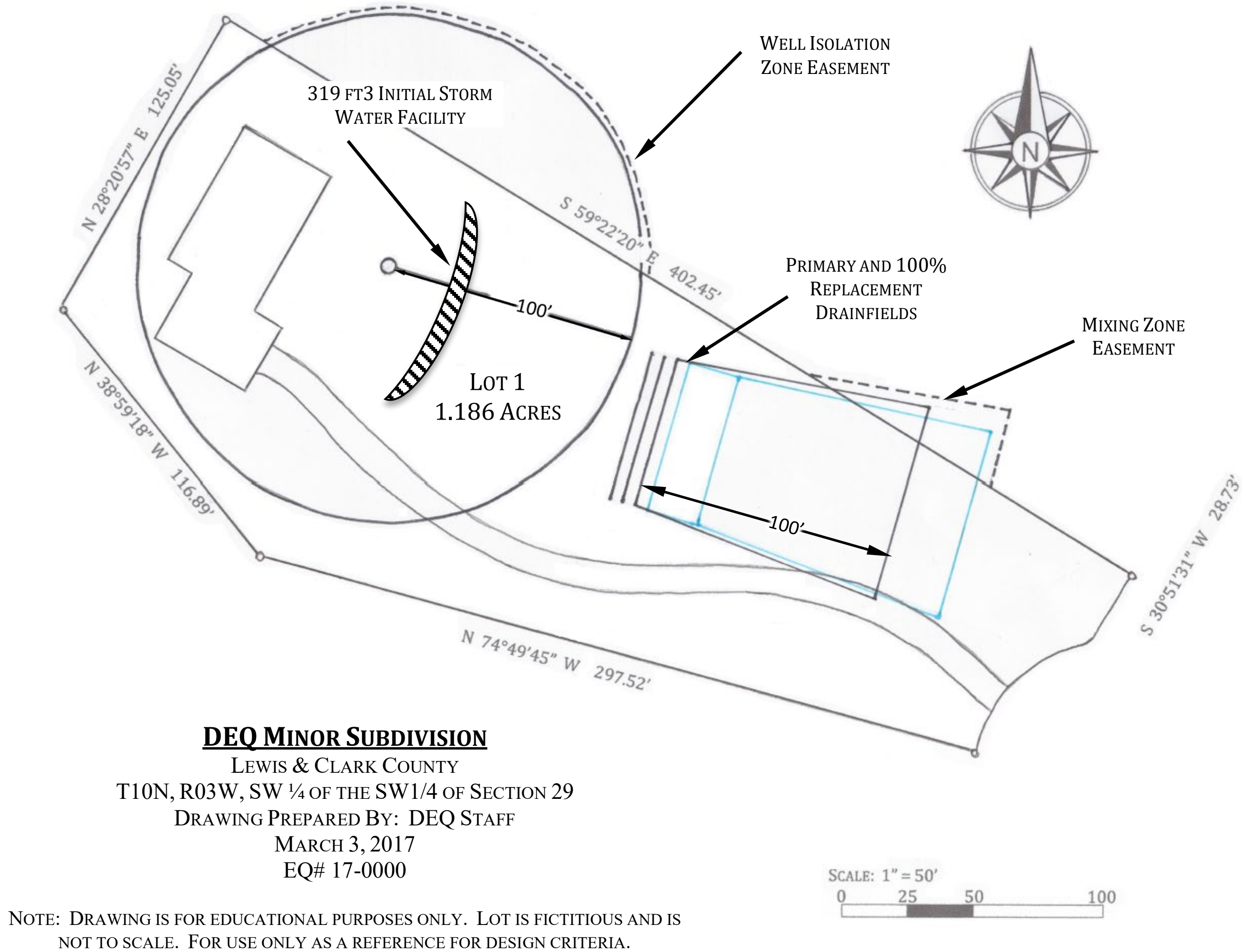


Figure 3. Initial Storm Water Facility Lot Layout

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 D - SIMPLIFIED PLAN EXAMPLE

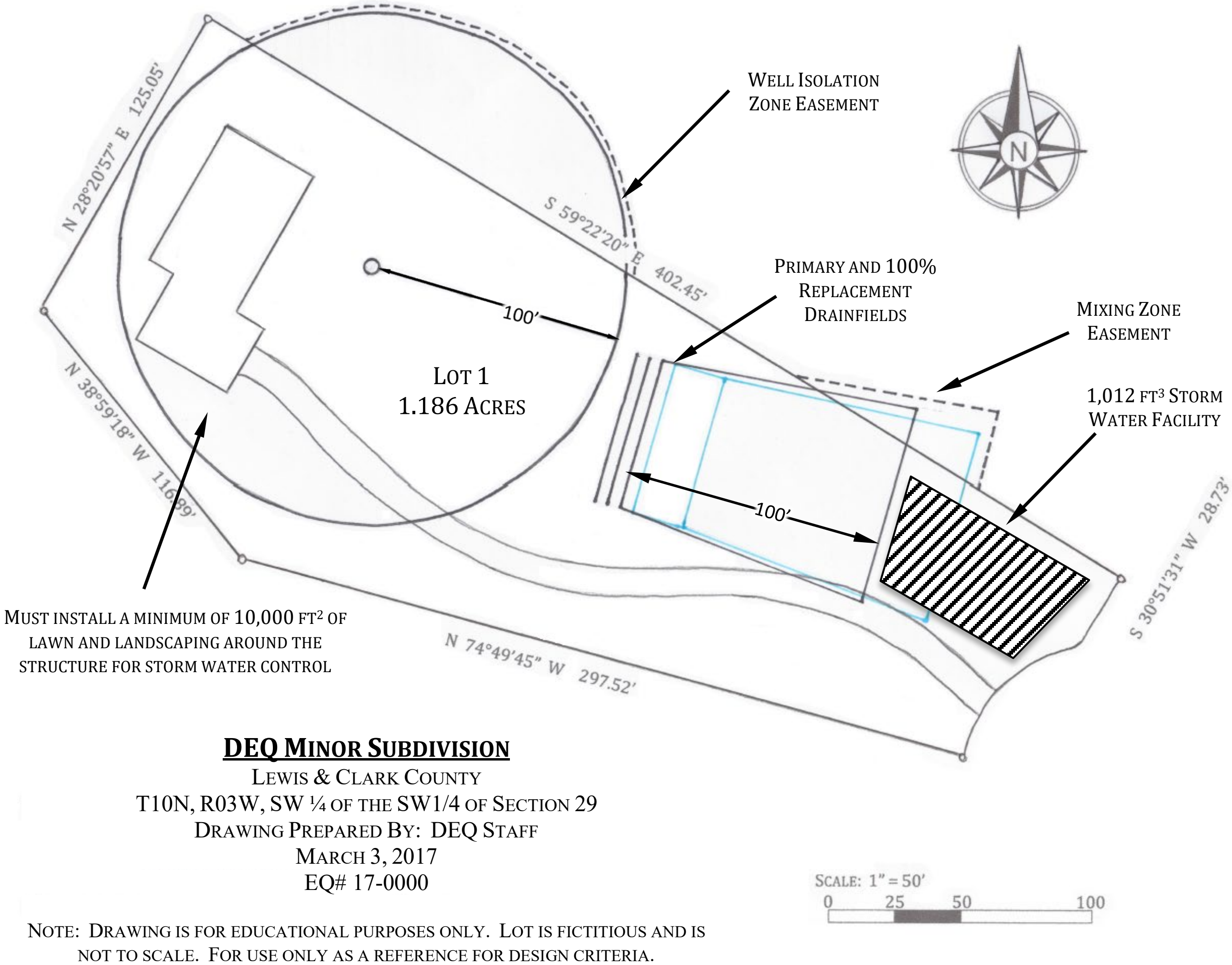


Figure 4. Simplified Plan Lot Layout

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 application is for one lot
- 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.

Second, determine the total impervious area on the lot to calculate the minimum facility volume required to retain and/or infiltrate the first 0.5 inches of rainfall. Landscaping and undeveloped areas are not included in this facility sizing, so the total impervious area is the house/roof, patio, and gravel driveway.

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

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

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

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.

The minimum retention facility size can be determined using the rational method (example spreadsheet provided in Appendix B) and the 100-year rainfall ~~intensity~~ depth for Helena of

~~3.04~~ 2.44 inches (~~provided in Appendix A~~ Montana Department of Transportation Hydraulics Manual Appendix B-2017: Helena Airport COOP: 244055).

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

$$Q = Cf * C_w * i * A$$

The correction factor for the 100-year return period per Section 3.7.1. is Cf = 1.25. Using the rational coefficients in ~~Appendix 0~~ Section 3.7.1, the pre-development coefficient of runoff is $C_{w\text{-pre}} = 0.2$. The pre-development runoff volume is:

$$Q_{pre-development} = 1.25 * 0.2 * 2.44\ inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 2{,}626.16\ ft^3$$

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:

$$Q_{post-development} = 1.25 * 0.277 * 2.44 inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 3{,}637.74\ ft^3$$

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

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

$$V_{minimum} = 3{,}637.74\ ft^3 - 2{,}626.16\ ft^3 = 1{,}011.58 ft^3$$

The **Simplified Plan Facility** must provide a minimum volume of ~~**1,009 ft^3**~~ **1,012 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 E - STANDARD PLAN – RETENTION FACILITY EXAMPLE

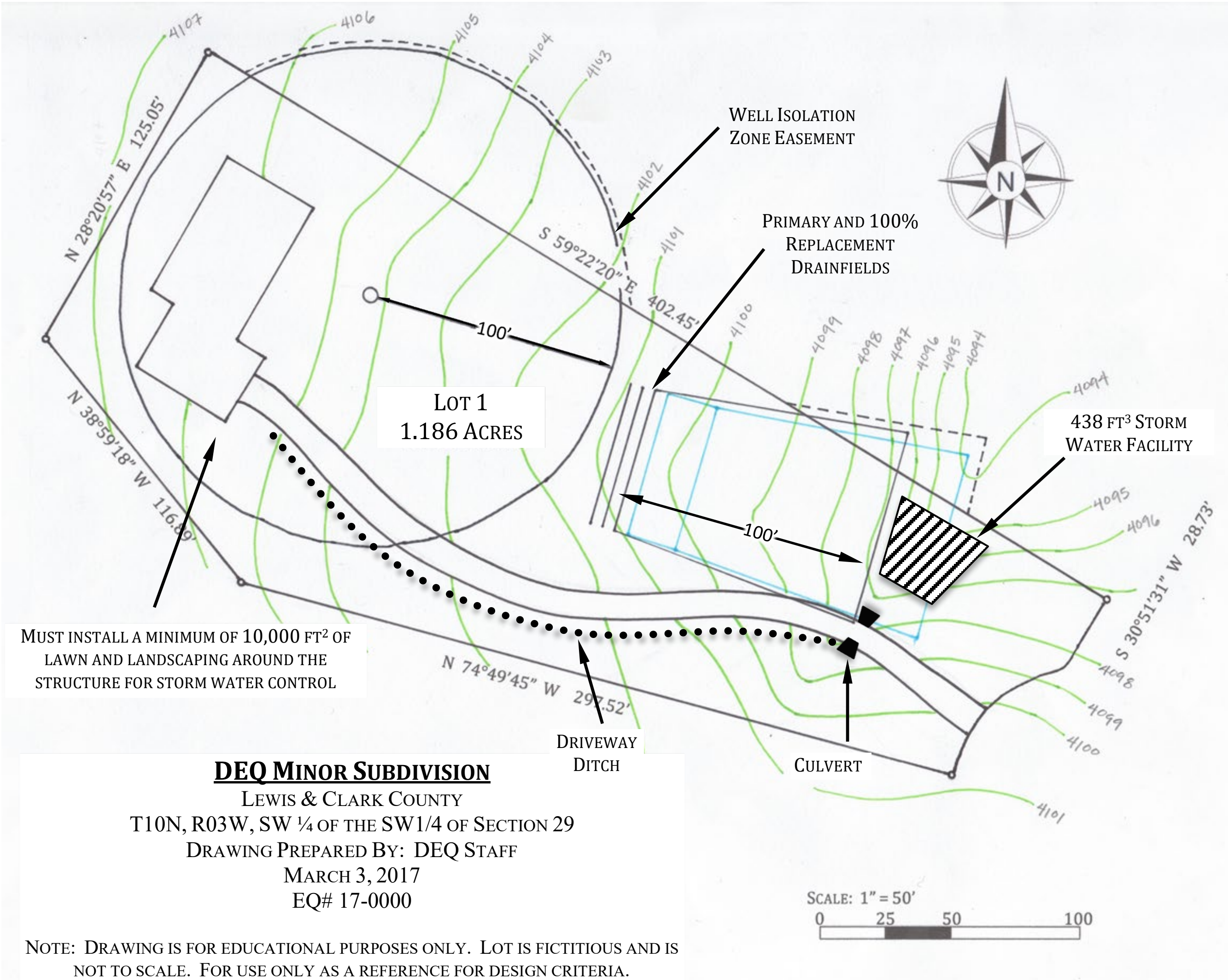


Figure 5. Standard Plan Retention Facility Lot Layout

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

E.1 INITIAL STORM WATER FACILITY

First, determine the total impervious area on the lot to calculate the minimum facility volume required to retain and/or infiltrate the first 0.5 inches of rainfall. Landscaping and undeveloped areas are not included in this facility sizing, so the total impervious area is the house/roof, patio, and gravel driveway.

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

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

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

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.

E.2 RETENTION FACILITY VOLUME

The minimum retention facility size can be determined using the ~~Modified~~ rational method (example spreadsheet provided in Appendix C) and the 2-year rainfall ~~intensity~~ depth for Helena of 1.32 inches (~~provided in Appendix A~~ Montana Department of Transportation Hydraulics Manual Appendix B-2017: Helena Airport COOP: 244055

~~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.~~

$$Q = Cf * C_w * i * A$$

The correction factor for the 2-year return period per Section 3.7.1 is Cf = 1.0.

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

Using the rational coefficients in ~~Appendix 0~~, Section 3.7.1, 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:

$$Q_{pre-development} = 0.2 * 1.32\ inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 1{,}036.57\ ft^3$$

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:

$$Q_{post-development} = 0.277 * 1.32\ inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 1{,}574.37\ ft^3$$

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

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

$$V_{minimum} = 1{,}574.37\ ft^3 - 1{,}036.57\ ft^3 = 437.80 ft^3$$

The **Standard Plan Retention Facility** must provide a minimum volume of ~~**415 ft^3**~~ **438 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.

E.3 TIME OF CONCENTRATION

Given the following hypothetical layout in Figure 5, the topography of the lot requires construction of a driveway ditch and culvert crossing to convey runoff from the proposed impervious area to the retention facility. For this example, the basin contributing runoff to the conveyance structures is assumed to be the boundary of the lot.

To correctly size the driveway ditch and culvert, the peak runoff flow rate must be determined. Per Section 3.7.5 and 4.1, the peak runoff flow rate must be determined at the time of concentration. The following presents the lot characteristics used to calculate the time of concentration.

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Length of sheet flow line: 155 ft
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:
The first step is to determine the time of concentration. In this example, the time of concentration consists of sheet flow time and shallow flow time.

Sheet Flow.
The first step is to 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 Section 3.7.5 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 to be 1.32 inches per the Montana Department of Transportation Hydraulics Manual Appendix B-2017: Helena Airport COOP: 244055

$$T_{t-sheet\ flow} = \frac{0.007(0.15 * 155)^{0.8}}{(1.32)^{0.5} 0.0323^{0.4}} = 0.298\ hours\ or\ 18\ minutes$$

Shallow Flow

Next, find the slope along the shallow 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 Section 3.7.5 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 the equation in Section 3.7.5.2., which is based on the slope and type of ground cover in the area of shallow concentrated flow.

Assume that the area of shallow 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.

$$V = 16.1345 * \sqrt{s} = 16.1345 * \sqrt{0.05} = 3.6\ ft/sec$$

The time of travel for shallow 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 flow.

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

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

E.4 CONVEYANCE STRUCTURES

The peak runoff flow rate can now be calculated for the driveway ditch and culvert crossing.

To ensure that the driveway is not overtopped during the 10-year event per Section 3.3.C. the driveway ditches must be capable of conveying the post-development flow rate.

As previously calculated, the post-development time of concentration is 19 minutes.

Per Section 3.7.1 the Montana Department of Transportation Hydrology Manual – Appendix B has rainfall intensity values for various parts of the state. The 10-year return period rainfall intensities for Helena from that document for the 15 minute and 20-minute storm event are 2.15 inch/hour and 1.76 inch/hour respectively. A linear interpolation of the two values results in a rainfall intensity of 1.84 in/hour for the 19-minute storm event (time of concentration).

$$2.15 + \frac{(2.15 - 1.76)}{(15 - 20)} * (19 - 15) = 1.84\ inch/hour$$

Use the weighted rational coefficient of 0.277 previously calculated, in the rational method equation from Section 3.7.1.

$$Q = 1 * 0.277 * \frac{1.84\ in}{hr} * \frac{1\ hr}{3600\ sec} * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in}$$
$$= 0.60\ \frac{ft^3}{sec}$$

The post-development runoff peak flow rate for the 10-year storm event is **0.60 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 17. Assume a maximum water depth of 6 inches (0.5 ft), which would correspond to a 3-foot wide ditch.

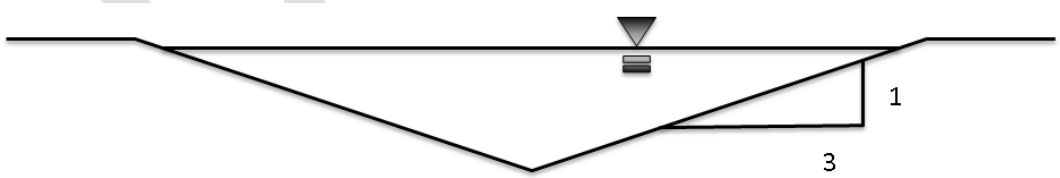


Figure 7. Typical Section View of V-Ditch

The Chezy-Manning Equation 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 0.60 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.

Per Section 3.7.1 the Montana Department of Transportation Hydrology Manual – Appendix B has rainfall intensity values for various parts of the state. The 100-year return period rainfall intensities for Helena from that document for the 15 minute and 20-minute storm event are 4.14 inch/hour and 3.38 inch/hour respectively. A linear interpolation of the two values results in a rainfall intensity of 3.53 in/hour for the 19-minute storm event (time of concentration).

Use the weighted rational coefficient of 0.277 previously determined, and a correction factor of 1.25 for the 100-year return period per Section 3.7.1 in the rational method.

$$Q = 1.25 * 0.277 * \frac{3.53\ in}{hr} * \frac{1\ hr}{3600\ sec} * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 1.46\ \frac{ft^3}{sec}$$

The post-development runoff peak flow rate for the 100-year storm event is **1.46 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. The post-development flow rate for the 10-year storm event at the time of concentration was calculated above as **0.60 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 of the culvert, 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:

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

$$0.60\frac{ft^3}{sec} = \frac{1.486}{0.022} * 2\pi * \left[\frac{D\ ft}{4}\right]^{8/3} * \left(0.05\frac{ft}{ft}\right)^{1/2}$$

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 8 inches. Per Section 4.4, the minimum culvert diameter is **12 inches**.

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.

The post-development flow rate for the 100-year storm event at the time of concentration was calculated above as **1.46 cfs**.

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:

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

$$1.46\frac{ft^3}{sec} = \frac{1.486}{0.022} * 2\pi * \left[\frac{D\ ft}{4}\right]^{8/3} * \left(0.05\frac{ft}{ft}\right)^{1/2}$$

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 **10 inches**. The 12-inch culvert proposed for the 10-year storm will be sufficient to pass the 100-year storm without inundating any homesites or drainfields.

~~STANDARD PLAN – INFILTRATION FACILITY EXAMPLE~~

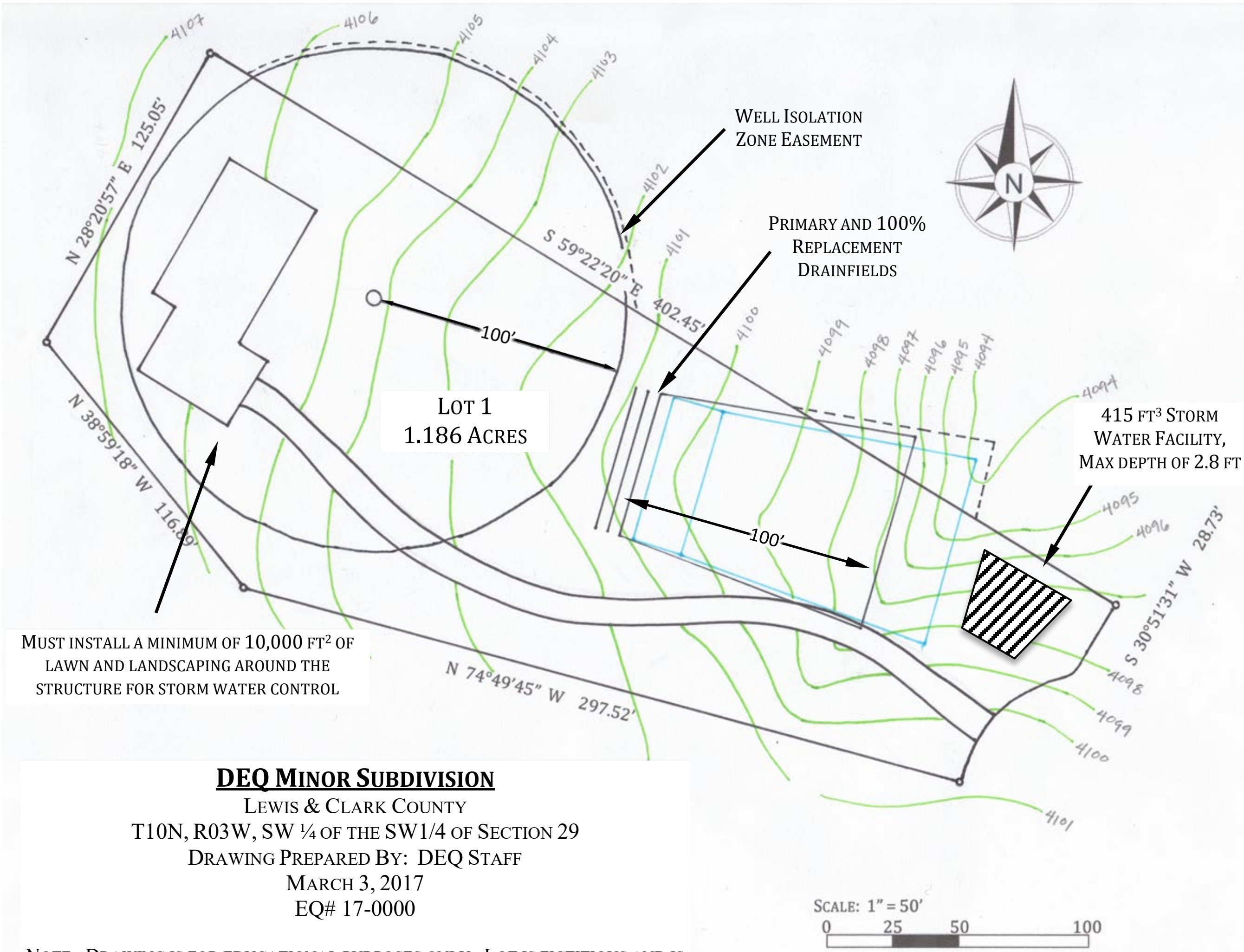


Figure 8. Standard Plan Infiltration Facility Lot Layout

~~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 ft² of house/roof~~
  - ~~300 ft² patio~~
  - 10,000 ft² of lawn and landscaped area
  - ~~3,750 ft² 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 0 . 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 (ft³/sec or, in-ac/hour)~~
~~C = runoff coefficient (unitless)~~
~~i = intensity (in/hour)~~
~~A = Area (acres)~~

~~Using the rational coefficients in Appendix 0, the pre-development coefficient of runoff is $C_{w-pre} = 0.2$. 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:~~

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

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

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

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

~~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.~~

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

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

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

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

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

~~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**.~~

~~STANDARD PLAN – DETENTION FACILITY EXAMPLE~~

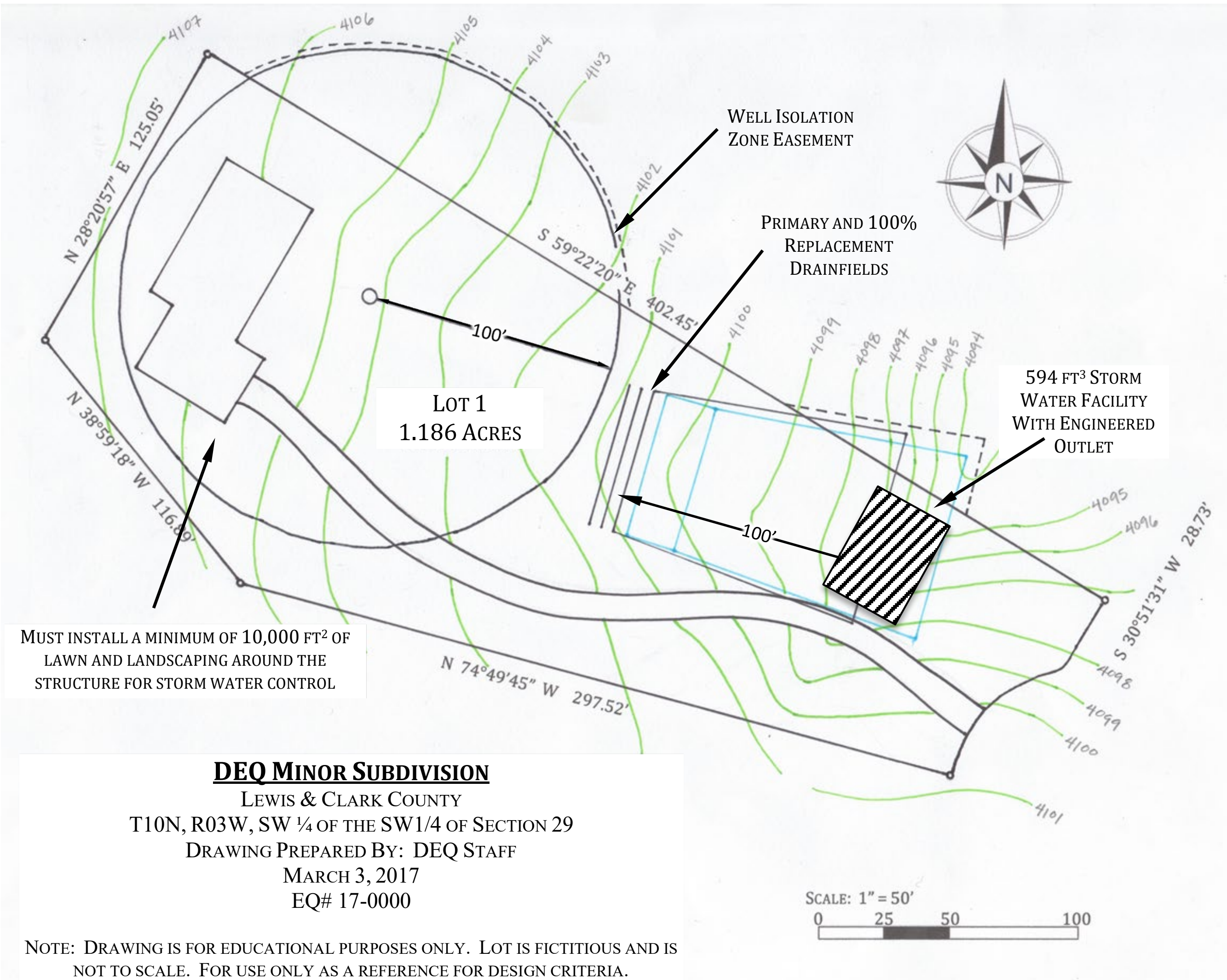


Figure 9. Standard Plan Detention Facility Lot Layout

~~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.~~

~~E.5 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.~~

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

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

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

~~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.~~

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

~~E.6 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 0 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 10, locate an intensity for the 29-minute duration and the 2-year storm event of i = 0.68 in/hr.~~

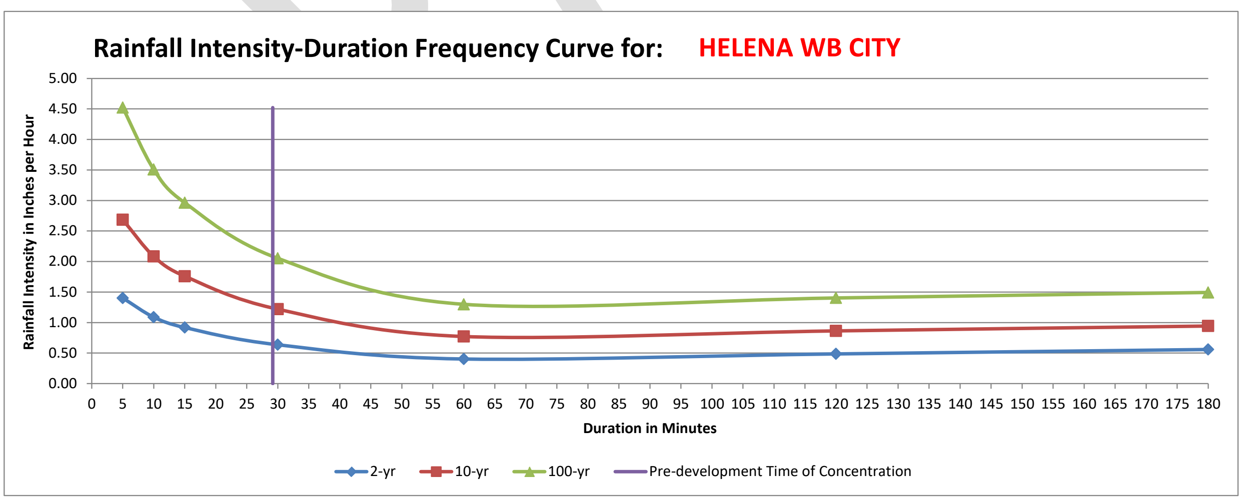
~~Using the rational method:~~

$$\cancel{Q = C * i * A}$$

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

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

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



~~**Figure 10. Pre-Development IDF Curve**~~

~~E.7 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 ft² of house/roof~~
- ~~300 ft² patio~~
- ~~10,000 ft² of lawn and landscaped area~~
- ~~3,750 ft² 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.~~

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

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

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

~~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.~~

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

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

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

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

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

~~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 20 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 11 for how this determination was made).~~

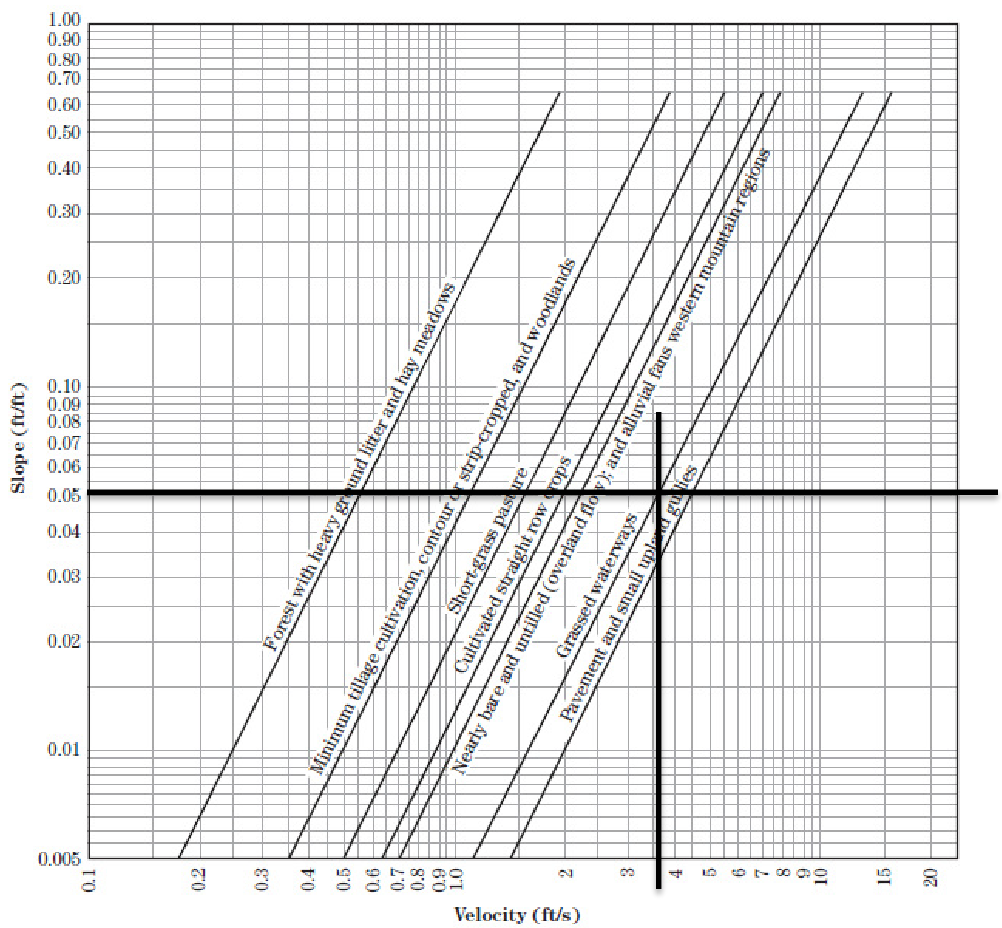


Figure 11. Detention Example Slope and Velocity Graph

~~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.~~

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

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

~~E.8 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 ft² of house/roof~~
- ~~300 ft² patio~~
- ~~10,000 ft² of lawn and landscaped area~~
- ~~3,750 ft² gravel driveway~~

~~Post-Development Time of Concentration: 19 minutes~~

~~Solution: As with the pre-development peak flow calculation, the Rational Method described in Appendix 0 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:~~

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

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

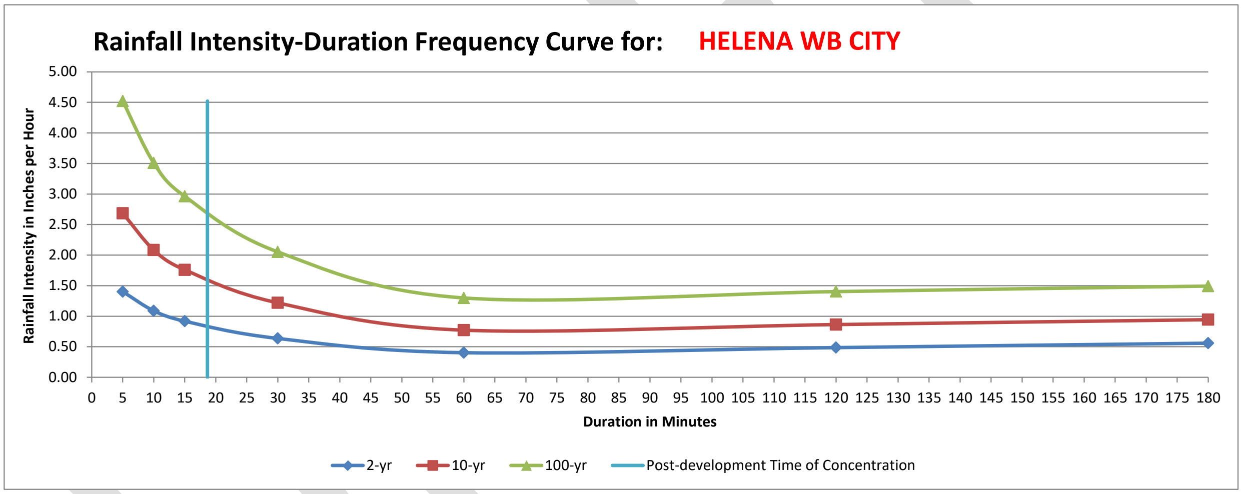
$$\cancel{C_{w-post} = \frac{(3{,}510 + 3{,}000 + 1{,}000 + 6{,}802.4) ft^2}{51{,}662.16 ft^2} = 0.277}$$

~~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.~~

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

~~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 12. Post-Development IDF Curve**~~

~~E.9 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 E.6 and the post-development peak flow rate was calculated as **0.28 cfs** in Appendix E.8. Additionally, both the pre- and post-development times of concentration were determined in Appendices E.5 and E.7 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 13. The post-development peak flow rate of **0.28 cfs** occurs at the post-development time of concentration of **19 minutes**.~~

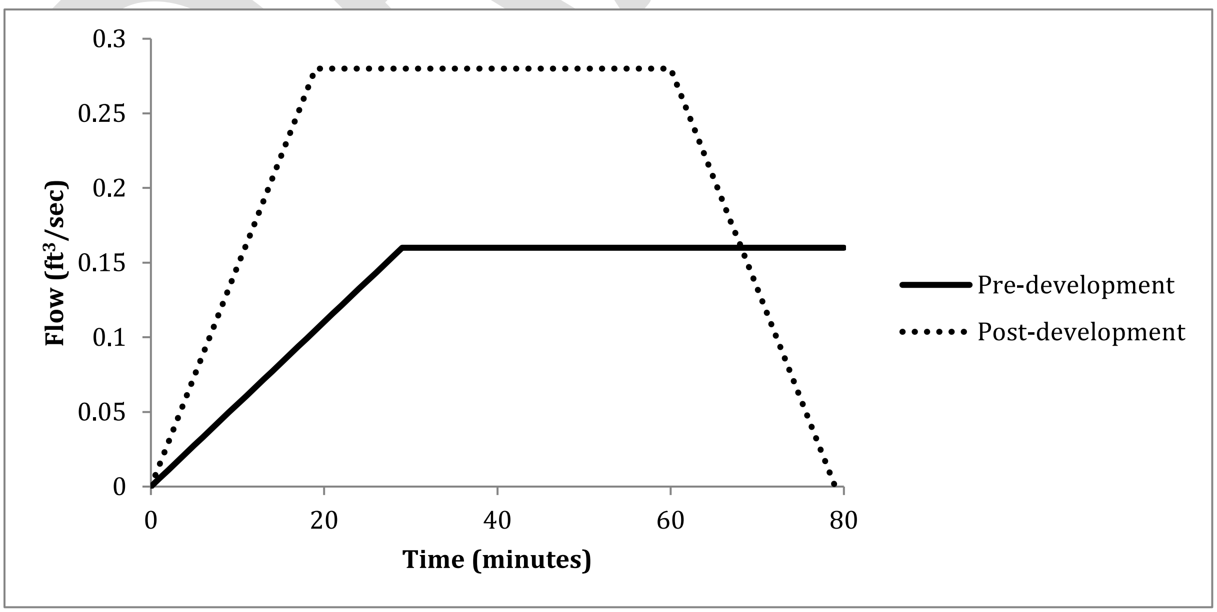


Figure 13. Pre- and Post-Development Runoff Hydrographs

~~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 14 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.~~

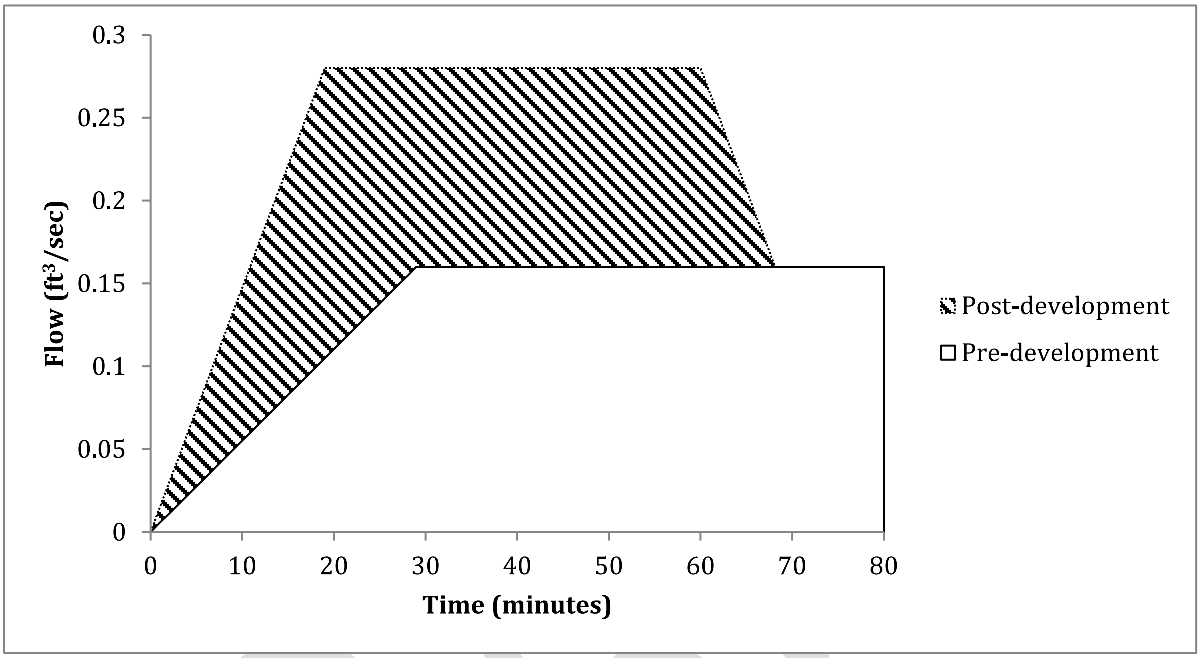


Figure 14. Hydrographs Showing Required Storage

~~Use the pre-development peak flow rate of **0.16 cfs** and the equation for discharge from a circular orifice given in A.4 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.~~

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

~~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)~~

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

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

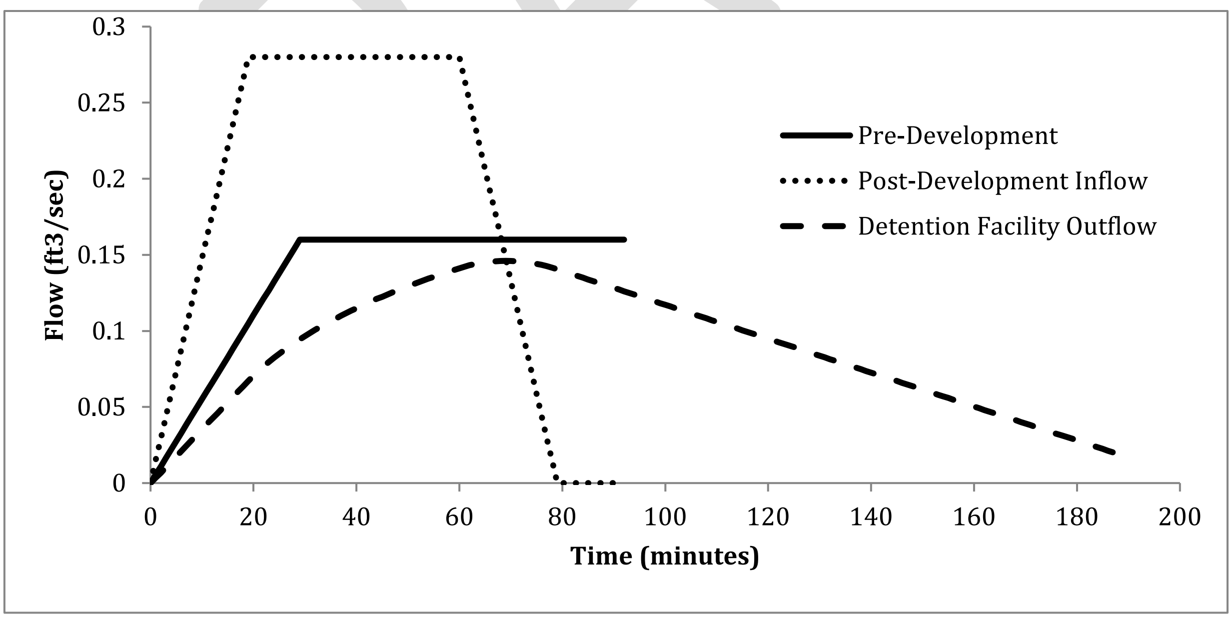
~~Now that the outlet size, depth of the facility, and a starting point for the minimum facility volume have been determined a stage-storage relationship can be developed for the specified facility. Based on the selected geometry of the detention facility, each "stage" or specific depth of the facility will correspond to a unique outlet value calculated using the Storage Indication Routing Method described in Appendix 0.~~

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

~~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 15. 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.~~

~~Standard Plan – Conveyance Example~~

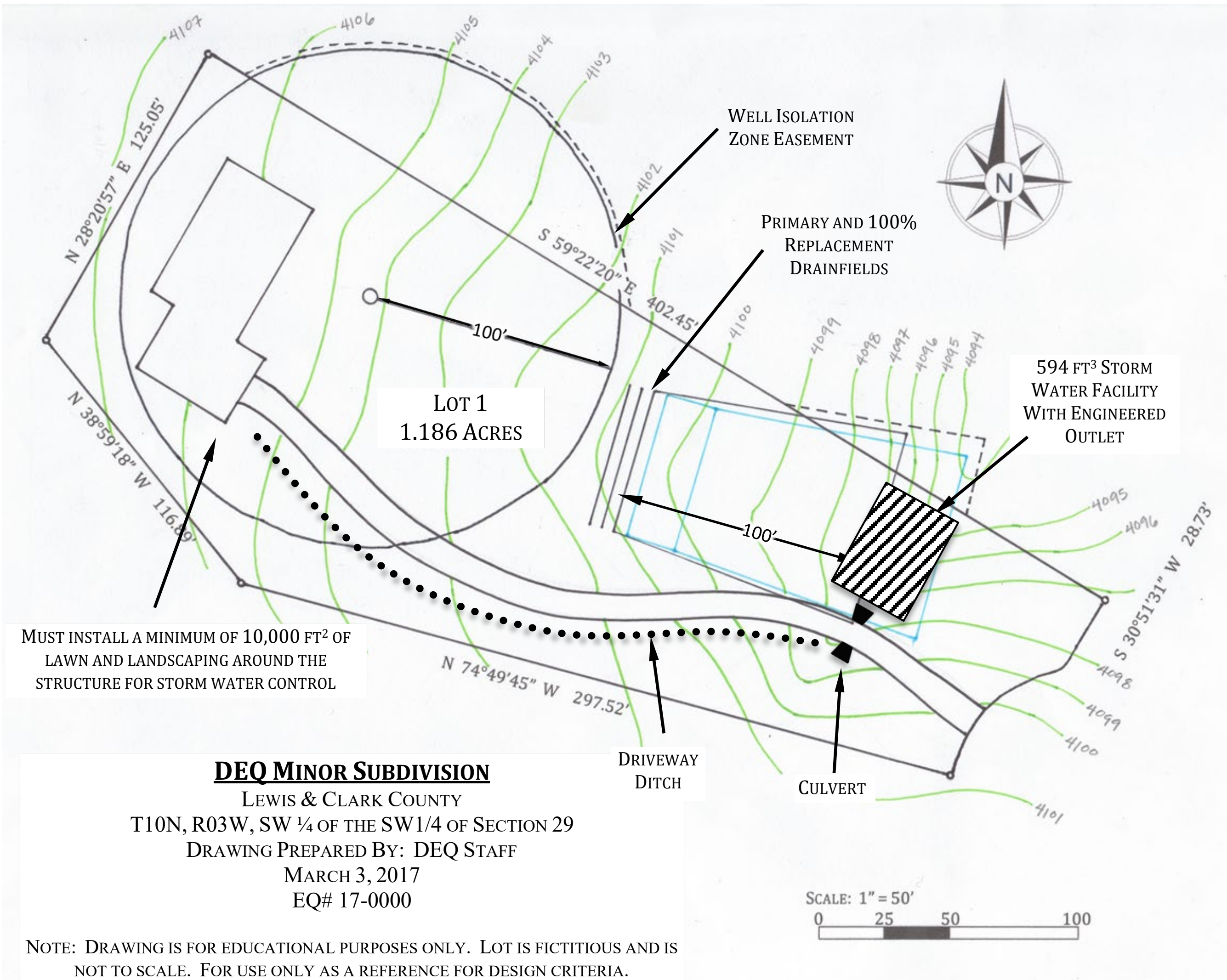


Figure 16. Standard Plan Conveyance Lot Layout

~~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 ft² of house/roof~~
  - ~~300 ft² patio~~

~~10,000 ft² of lawn and landscaped area~~
~~3,750 ft² gravel driveway~~

~~Some impervious areas have slopes in excess of 3%.~~

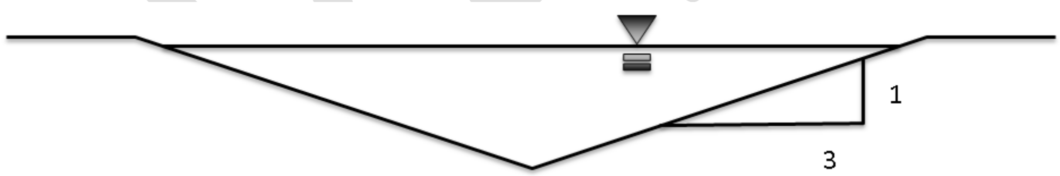
~~Additionally, the site has a basin to the south of roughly 10 acres with agricultural use that contributes historic storm water runoff to the natural drainage on the eastern portion of Lot 1.~~

~~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 E.8.~~

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

~~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 17. Assume a maximum water depth of 6 inches (0.5 ft), which would correspond to a 3-foot wide ditch.~~



~~**Figure 17. 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 (ft²)~~
~~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.~~

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

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

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

~~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 E.8.~~

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

~~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 0 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.~~

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

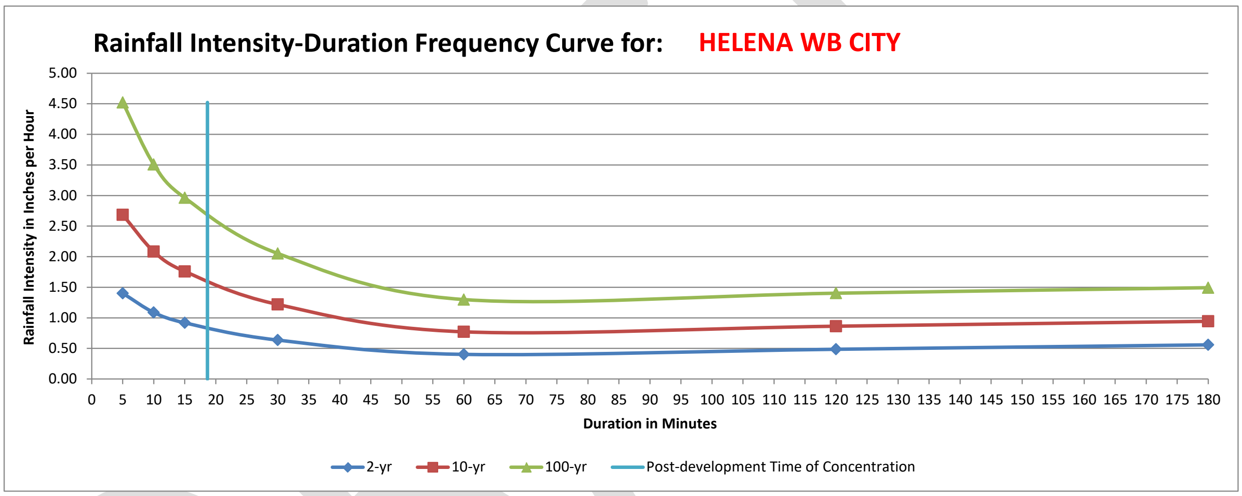
~~Estimate that the Manning's roughness coefficient, n, is 0.022 for a corrugated metal culvert using the Federal Highway Administration Hydraulic Engineering Circular No. 22, Third~~

~~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:~~

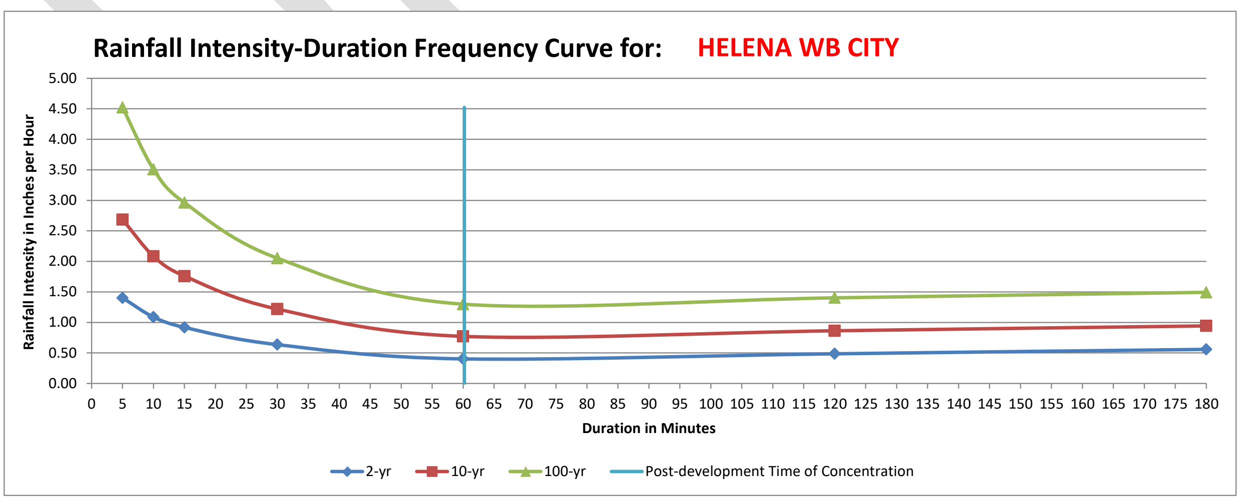
$$Q = \frac{1.486}{n} * 4\pi * \left(\frac{D}{4}\right)^{8/3} * S^{1/2}$$

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

~~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 18. Post-Development IDF Curve for Site**~~



~~**Figure 19. IDF Curve for Offsite Basin**~~

~~SHALLOW CONCENTRATED FLOW~~

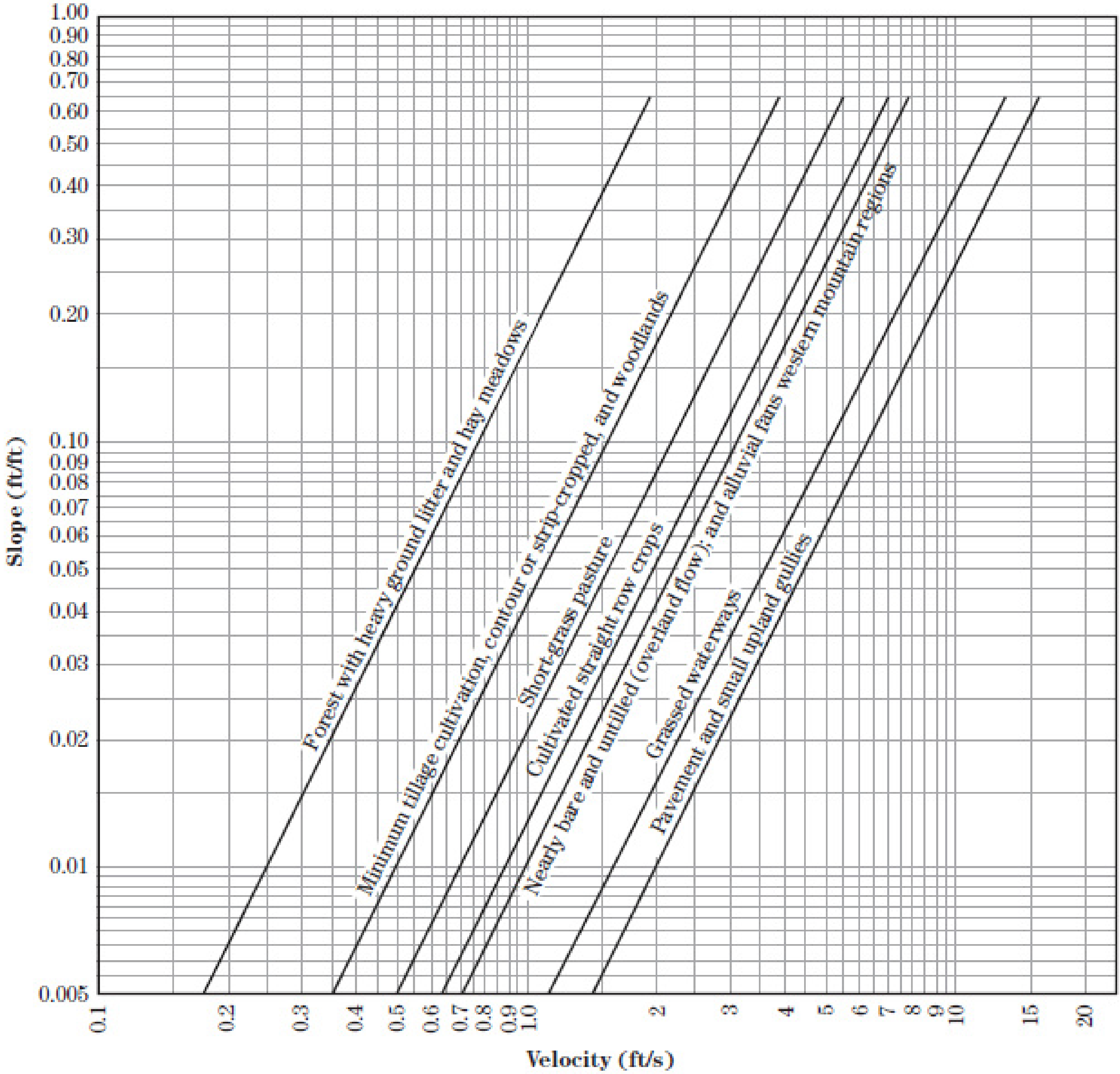
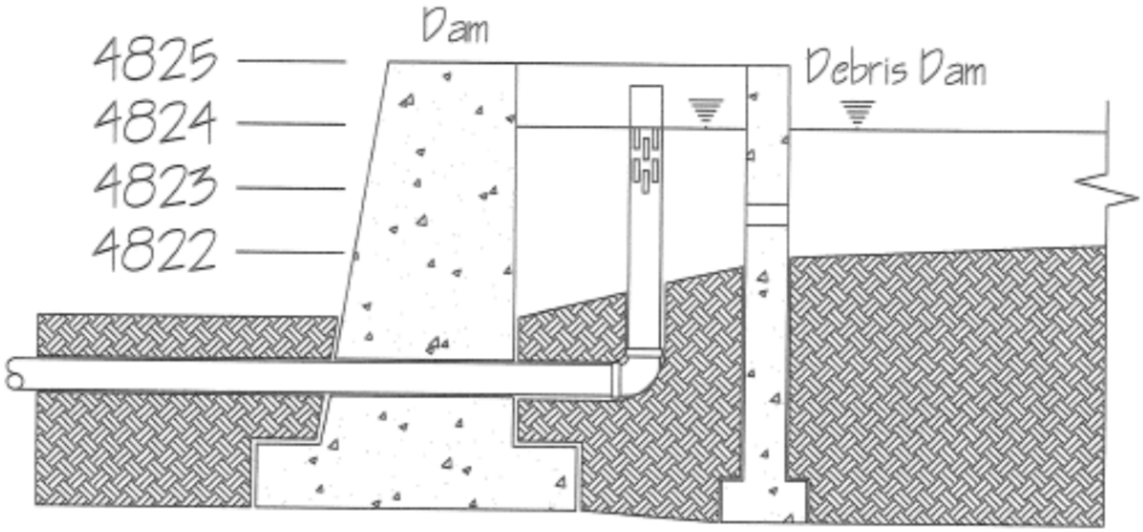
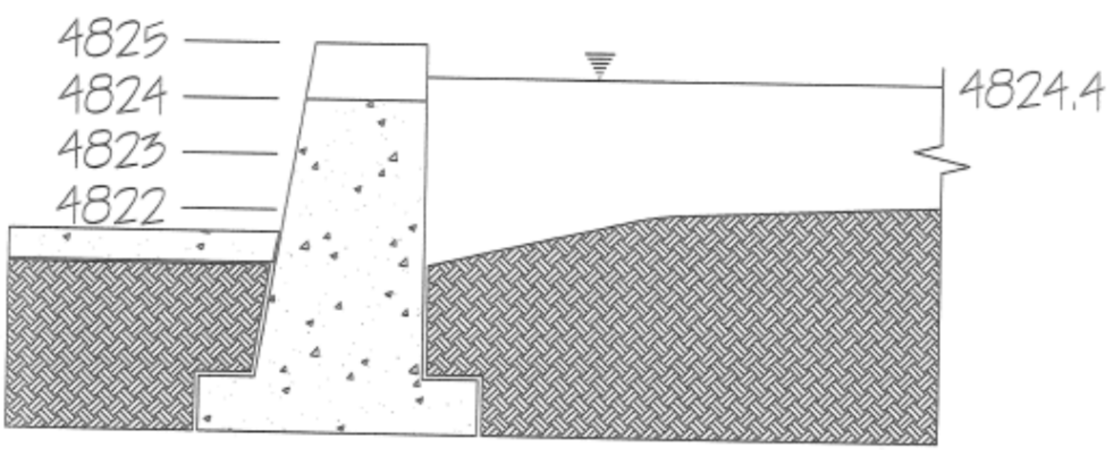


Figure 20. Shallow Concentrated Flow Slope and Velocity

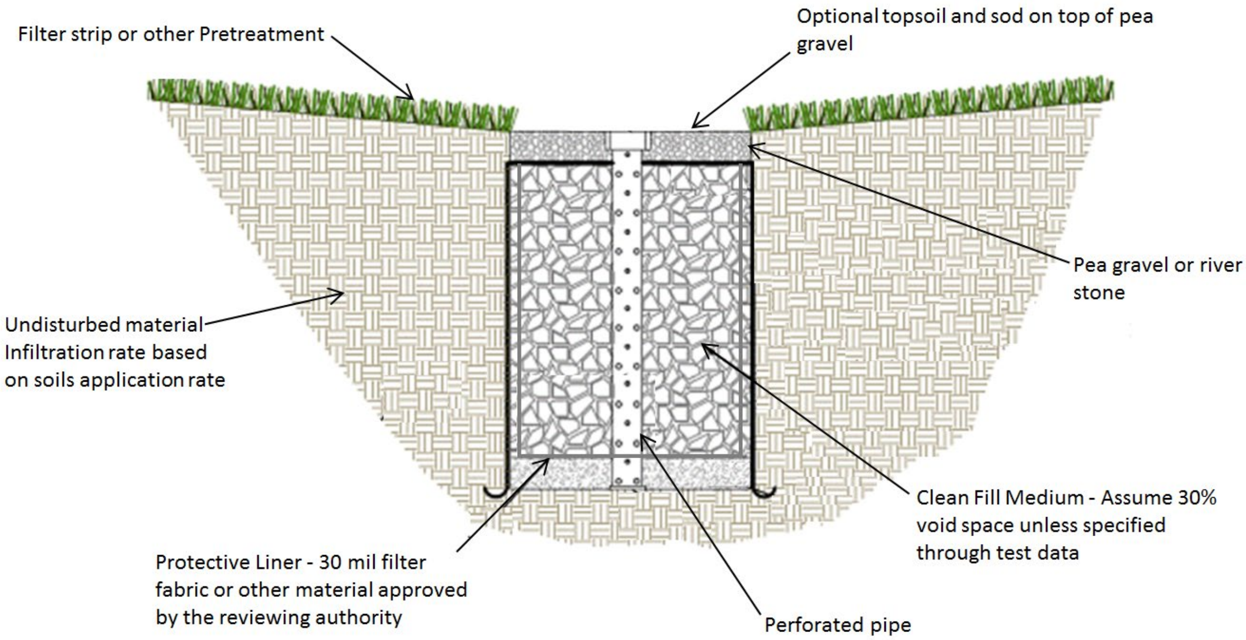
~~EXAMPLE DRAWINGS~~



~~**Figure 21. Slotted Riser Pipe Example**~~



~~**Figure 22. Rectangular Weir Example**~~



~~**Figure 23. Infiltration Facility Example**~~

APPENDIX F - REFERENCES

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