CHAPTER 6
STORMWATER STANDARDS

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6.1 PURPOSE

The purpose of this Chapter is to set forth design standards and criteria for storm drainage infrastructure so there is reasonable degree of assurance that the health, safety, welfare, and property of the Town and citizens may be safeguarded and protected through the proper control and drainage of stormwater, the water quality of the Blue River and its tributaries will be protected, and there will be a certain uniformity in performance with respect to design and construction of drainage facilities.

Additionally, the Town of Breckenridge's mountain environment is known for steep grades, wetland areas, perennial streams, reservoirs, and the Blue River. This unique environment and topography results in grading, slope stability, stormwater runoff contamination, and phosphorus contamination concerns. This chapter establishes standards to safeguard public health and safety, protect environmentally sensitive areas, and protect the water quality of the Blue River, its tributaries, and Dillon Reservoir.

The design Standards presented herein are intended to aid in the design of stormwater infrastructure and outline the minimum standards required.

6.2 OTHER STANDARDS

The Mile High Flood District (MHFD), formerly known as the Urban Drainage and Flood Control District (UDFCD), has developed detailed permanent water quality design guidance and criteria in Volume 3 of the *Urban Storm Drainage Criteria Manual* (USDCM). This document is referenced extensively in this section and provides extensive discussion on the topic of stormwater quality treatment. When used for design, the most recent version shall be referenced as the MHFD continually updates Volume 3 of the USDCM based on performance and maintainability of the treatment facilities discussed.

6.2 SUBMITTALS

An Infrastructure Permit is required for all construction projects involving storm drainage infrastructure. Requirements for an Infrastructure Permit are summarized in Chapter 2. Other submittals may be required in accordance with Chapter 2 or as determined by the Town Engineer.

6.3 RAINFALL

A rainfall analysis was completed for the Town of Breckenridge using the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 data. The equations and data in this section are based on this analysis. A detailed memo regarding the evaluation of NOAA Atlas 14 is available from the Town upon request.

6.3.1 Rainfall Intensity

Equation 6.1 shall be used to calculate rainfall intensity for a given time of concentration or to develop intensity-duration-frequency curves for runoff analysis using the Rational Method as discussed in Section 6.4.

$$I = P_1 \frac{40.81}{(t+7.63)^{0.881}} \tag{6.1}$$

Where:

I = rainfall intensity (in/hr)

P_1 = 1-hour rainfall depth (in), see Table 6.1

t = storm duration, time of concentration (min)

Rainfall intensities as a function of storm duration and recurrence interval are provided in Table 6.1. This table was developed using Equation 6.1. The values in Table 6.1 are subject to revision and users of these Standards are encouraged to check for updates.

Table 6.1. Intensity-Duration-Frequency Data

Return Period	P_1	Peak Rainfall Intensity for Storm Duration (in/hr)				
		5-min	10-min	15-min	30-min	60-min
2-year	0.546	2.39	1.78	1.43	0.91	0.54
5-year	0.749	3.27	2.44	1.96	1.25	0.75
10-year	0.939	4.10	3.06	2.45	1.57	0.94
25-year	1.23	5.37	4.01	3.21	2.05	1.23
50-year	1.49	6.51	4.85	3.89	2.49	1.48
100-year	1.76	7.69	5.73	4.60	2.94	1.75
500-year	2.5	10.92	8.14	6.53	4.17	2.49

6.3.2 Rainfall Distribution

The 6-hour rainfall depths provided in Table 6.2 shall be used to develop a 6-hour SCS Type II rainfall distribution for runoff analyses that use hydrographs as discussed in Sections 6.4.4 and 6.4.5.

Table 6.2. 6-Hour Rainfall Depths for SCS Type II Distribution

Return Period	6-hour Rainfall Depth (in)
2-year	0.859
5-year	1.070
10-year	1.28
25-year	1.61
50-year	1.89
100-year	2.21
500-year	3.07

6.3.3 Design Storms

The objectives of establishing the minor and major design storms are to minimize inconvenience, protect against recurring minor damage, reduce maintenance costs caused by the minor storm, and eliminate substantial property damage and loss of life caused by the major storm. The goal is a functional drainage system at a reasonable cost. The storm drainage system may include streets, curb and gutter, roadside ditches, inlets, storm sewers, open drainageways, culverts, and detention facilities. In the Town of Breckenridge, the minor storm is the 10-year event and the major storm is the 100-year event.

6.4 RUNOFF

The hydrologic analysis of a site shall be based on the proposed land use for that site, as opposed to existing conditions. Calculation of contributing runoff from offsite upstream areas shall be based on the existing land use and topography. Flows specified in the Flood Insurance Study, Town of

Breckenridge (2011) shall be incorporated in the analysis where applicable and with the approval of the Town Engineer.

6.4.1 Approved Hydrologic Methods

All hydrologic methods have limitations and should be used only for appropriate scenarios. Accepted hydrologic methods are the Rational Method, the Soil Conservation Service (SCS) Dimensionless Unit Hydrograph Method, and the SCS Runoff Method. The last two methods are referred to collectively herein as the SCS Method developed by the Natural Resource Conservation Service (NRCS). The Rational Method may be used for watersheds of 90 acres or less and when only peak flows, as opposed to a hydrograph, are required for design (e.g., pipes, culverts, inlets). The SCS Method may be used for watersheds of any size or when hydrograph routing is required for design (e.g., detention ponds and volume-based water quality facilities). Information on the computer models that may be used to develop and route hydrographs is included later in this section.

6.4.2 Subwatershed Sizing

Determination of peak runoff at any downstream design point is affected by the size, number, and characteristics of the upstream subwatersheds within the overall drainage basin. Typically, the more homogenous each of the subwatersheds is, the more accurate the calculated peak flow is when compared to analysis of a single, larger watershed. Recommended guidelines are:

1. For an overall watershed of up to 100 acres, the maximum subwatershed size should be approximately 20 acres. Delineation should be conducted so that imperviousness, slope, and land use are similar for each subwatershed.
2. For an overall watershed over 100 acres, increasingly larger subwatersheds may be used provided the land use and surface characteristics within each subwatershed are homogeneous. In addition, the subwatershed sizing should be consistent with the level of detail needed to determine peak flow rates at various design points within the larger watershed.

6.4.3 Rational Method

The Rational Method uses Equation 6.2 to determine a peak runoff rate based on drainage area, rainfall intensity, and imperviousness. Imperviousness is represented by a coefficient, C.

$$Q = CIA \tag{6.2}$$

Where:

Q = peak rate of runoff (cfs)

C = runoff coefficient

I = rainfall intensity (in/hr)

A = drainage area (acres)

The rainfall intensity is given by Equation 6.1 or Table 6.1 based on the design storm and storm duration, which is assumed to be equal to the total time of concentration to the design point being analyzed. Determining values for the runoff coefficient, C, is discussed in the section below.

6.4.3.1 Runoff Coefficient

The runoff coefficient is based on land cover and soil classification. Soils are classified by the NRCS as belonging to Hydrologic Soil Group A, B, C, or D. The Hydrologic Soil Group(s) present at a site can be found using the Web Soil Survey tool available via an internet search for *NRCS Web Soil Survey*. Types of land use and their typical corresponding imperviousness are summarized in Table 6.3. Composite imperviousness should be obtained for each subwatershed that is not completely homogenous. The runoff coefficient, C, can then be calculated using the equations in Table 6.4. The coefficients are dependent on the Hydrologic Soil Group, composite imperviousness, and the return period of the design storm. The values below are approximations of average imperviousness for typical land uses. If a development's land use does not match a land use in the table below, or if the development's imperviousness does not match the values in the table below, imperviousness may be calculated for each subbasin within the site by dividing the total impervious area within the subbasin by the total subbasin area.

Table 6.3. Land Use Imperviousness

Land Use	Imperviousness (%, i_p)	Land Use	Imperviousness (%, i_p)
Urban:		Playgrounds	25
Downtown Area	95	Schools	55
Suburban Area	75	Parks, Cemeteries	10
Residential Single-family:		Paved Streets	100
2.5 acres or larger	15	Drive and Walks	90
0.75 - 2.5 acres	20	Roofs	90
0.25 - 0.75 acres	30	Lawns	2
0.25 acres or less	50	Undeveloped Areas:	
Apartments/Duplex/Townhomes	75	Historic Flow Analysis	2
Industrial:		Greenbelts, Agricultural	2
Light Areas	80	Off-site Flow Analysis (land use not defined)	45
Heavy Areas	90		

Table 6.4. Runoff Coefficient Equations

NRCS Soil Group	Storm Return Period			
	2-year	5-year	10-year	25-year
A	$C=0.84i_p^{1.302}$	$C=0.86i_p^{1.276}$	$C=0.87i_p^{1.232}$	$C=0.88i_p^{1.124}$
B	$C=0.84i_p^{1.169}$	$C=0.86i_p^{1.088}$	$C=0.81i_p+0.057$	$C=0.63i_p+0.249$
C/D	$C=0.83i_p^{1.122}$	$C=0.82i_p+0.035$	$C=0.74i_p+0.132$	$C=0.56i_p+0.319$

NRCS Soil Group	Storm Return Period		
	50-year	100-year	500-year
A	$C=0.85i_p+0.025$	$C=0.78i_p+0.110$	$C=0.65i_p+0.254$
B	$C=0.56i_p+0.328$	$C=0.47i_p+0.426$	$C=0.37i_p+0.536$
C/D	$C=0.49i_p+0.393$	$C=0.41i_p+0.484$	$C=0.32i_p+0.588$

6.4.3.2 Time of Concentration

The storm duration used in Equation 6.1 is the time required for runoff to flow from the most hydraulically distant point within the subwatershed to the design point of interest. This time is known as the time of concentration. Time of concentration is calculated by adding the flow times for each type of flow along most hydraulically distant flow path. Types of flow include sheet flow and shallow concentrated, or channelized flow. Time of concentration for the Rational Method is calculated using Equations 6.3 through 6.5.

$$t_c = t_i + t_t \tag{6.3}$$

Where:

t_c = time of concentration (min), minimum is 5 min in urban areas, 10 min in rural areas

t_i = initial or overland sheet flow time (min)

t_t = travel time for shallow concentrated flow in a ditch, channel, gutter, pipe, etc. (min)

Initial or overland flow is the sheet flow that occurs at the beginning of the flow path characterized by a flow depth less than 0.1 feet. It can be calculated using Equation 6.4 for flow path lengths up to 500 feet in rural areas and 300 feet in urban areas. However, in highly urbanized areas, the overland flow path is typically shorter than 300 feet because of the presence of drainage systems that collect and convey runoff.

$$t_i = \frac{0.395(1.1-C_5)\sqrt{L_i}}{S_o^{0.33}} \tag{6.4}$$

Where:

t_i = initial or overland sheet flow time (min)

C_5 = runoff coefficient for 5-year return period (see Table 6.4)

L_i = length of overland flow segment (ft)

S_o = average slope along the overland flow path (ft/ft)

Travel time for shallow concentrated flow is calculated based on the hydraulic properties of the conveyance element. The channelized travel time, t_t, is estimated by dividing the length of conveyance by the velocity. Equation 6.5 can be used to determine travel time in conjunction with the conveyance factors in Table 6.5.

$$t_t = \frac{L_t}{60K\sqrt{S_o}} = \frac{L_t}{60V_t} \tag{6.5}$$

Where:

t_t = travel time for shallow concentrated flow in a ditch, channel, gutter, pipe, etc. (min)

L_t = shallow concentrated flow path length (ft)

S_o = average slope of shallow concentrated flow path (ft/ft)

K = NRCS conveyance factor (see Table 6.5)

V_t = travel time velocity (ft/s) = $K\sqrt{S_o}$

For shallow concentrated flow in a pipe or well defined channel, Manning's Equation for open channel flow in Section 6.5 may be used to calculate velocity. The time of concentration, t_c, is then the sum of the initial flow time, t_i, and the travel time for shallow concentrated flow, t_t.

Table 6.5. NRCS Conveyance Factors, K

Land Surface	Conveyance Factor, K
Heavy meadow	2.5
Tillage/field	5

Short pasture and lawns	7
Nearly bare ground	10
Grassed waterway	15
Paved areas and shallow paved swales	20

Reference: NRCS (1986)

6.4.3.3 Limitations and Considerations

The minimum time of concentration in urban areas is 5 minutes. The minimum time of concentration in rural areas is 10 minutes. A common mistake in urban areas is to assume travel velocities that are too slow. Another is to not check the runoff peak resulting from only part of the design catchment. Sometimes a portion of the design catchment closer to the design point or a highly impervious area will produce a larger peak design flow than that computed for the entire catchment. This most often happens when the catchment is long and narrow, or when the upper portion is undeveloped while the lower portion is, or will be, fully developed.

6.4.4 SCS Method

For drainage areas larger than 90 acres, the SCS Method is one method than can be used to develop hydrographs. The procedures used in the SCS Method are described in Technical Release 20 (TR-20) and Technical Release 55 (TR-55) both prepared by the NRCS. Three parameters are needed to use the SCS Method: rainfall, curve number, and time of concentration. This section provides some general information needed to use the SCS Method in a hydrologic model. However, the reader should refer to TR-20 and TR-55 for more detail describing the SCS Method; the applicable hydrologic equations and theory; and all other background information.

6.4.4.1 SCS Method - Rainfall

The SCS Method includes four rainfall time distributions that are assigned geographically across the country. The SCS Type II storm distribution is applicable to the entire State of Colorado and therefore should be used when preparing hydrologic analysis for the Town. The rainfall distribution is based on a 24-hour duration.

The 24-hour distribution was developed to include the maximum intensities of all smaller duration storms; the 24-hour storm theoretically includes the 2-hour, 6- hour, etc. distributions within one longer distribution and can be used to estimate peak flows for all storm events. However, it will typically overestimate the volume of a storm event with a duration less than 24 hours. Therefore, the SCS also developed a 6-hour storm distribution, which was derived from the original 24-hour distribution. The 6-hour distribution can be used if the drainage area being modeled has a total time of concentration less than 6 hours. The total rainfall depths should be taken from NOAA Atlas 14 for the project design storm return period.

6.4.4.2 SCS Method – Curve Number

The SCS Method curve number (CN) is a variable used to predict infiltration and runoff based on land use and soil types. When using the SCS Method in a hydrologic model, the user will be required to input the percent imperviousness and CN for each watershed or subwatershed modeled. The CN is dependent on land cover and the NRCS Hydrologic Soil Group (A, B, C, or D). The modeled watershed or subwatersheds may be composed of multiple land uses and soil types. In these cases, a composite CN must be calculated as the representative CN for the area. Refer to Tables 6.6 and 6.7 for CN values for specific land uses and hydrologic soil groups.

The initial abstraction is another important parameter used in the CN analysis. It represents all the losses that occur prior to runoff including infiltration, interception, depression storage, and evaporation. The hydrologic model will calculate the default initial abstraction based on the selected CN. However, in some unique cases it may be desirable to provide a user-defined initial abstraction based on the site conditions.

The table below provides runoff curve numbers for typical land uses based on average imperviousness. If a development's land use does not match a land use in the table below, or if the development's imperviousness does not match the values in the table below, curve numbers for each subbasin within the development may be calculated using TR-20 and TR-55.

Table 6.6. Runoff Curve Numbers

Land Use or Surface Characteristic	Average Imperviousness (percent)	Runoff Curve Number by Soil Type			
		A	B	C	D
Commercial/Mixed Use					
Downtown and Base Areas*	95	95	96	97	97
All Other Commercial Areas	75	83	89	92	94
Residential Single Family					
2.5 acres or larger lot size	12	46	65	77	82
0.75 – 2.5 acres lot size	20	51	68	79	84
0.25 – 0.75 acres lot size	30	74	83	88	91
0.25 acres or smaller lot size	45	66	78	85	88
Multifamily and Resort Residential	75	83	89	92	94
Industrial					
Light	80	86	91	93	94
Heavy	90	92	94	96	96
Public Facilities/Open Spaces					
Parks, cemeteries	10	45	63	75	81
Playgrounds	25	45	63	75	81
Schools	55	69	80	86	89
Lawns and golf courses	2	40	62	74	80
Undeveloped Areas					
Pre-development conditions	2	40	62	74	80
Greenbelts, agriculture	2	40	62	74	80
Off-site analysis, unknown land use	45	66	78	85	88
Outcrops	70	80	87	91	93
Streets/Roads & Surfacing					
Paved	100	98	98	98	98
Road base or recycled asphalt	80	86	91	93	94
Gravel (uniformly graded)	40	63	76	84	87
Drives/Walks	90	92	94	96	96
Roofs	90	92	94	96	96

Reference: Values are from a combination of UDFCD (2016) and USDA NRCS (2004)

Table 6.7. Runoff Curve Numbers for Arid and Semiarid Rangelands

Cover Type	Hydrologic Condition[1]	Runoff Curve Number for Soil Type			
		A[2]	B	C	D
Herbaceous – mixture of grass, weeds and low-growing brush, with brush the minor element	Poor Fair Good		80 71 62	87 81 74	93 89 85
Oak-aspen – mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor Fair Good		66 48 30	74 57 41	79 63 48
Coniferous, general; grass understory	Poor Fair Good		75 58 41	85 73 61	89 80 71
Sage-grass – sage with an understory of grass	Poor Fair Good		67 51 35	80 63 47	85 70 55

1. Poor: <30% cover (litter, grass, and brush overstory); Fair: 30 to 70% cover; Good: >70% cover
2. Curve numbers for group A have not been developed for these types of cover

Reference: USDA NRCS (2004)

6.4.4.3 SCS Method – Time of Concentration

The time of concentration (T_c) is defined as the time it takes for water to travel from the most hydraulically distant point in the watershed to the point of interest. The SCS Method for calculating T_c is done by splitting the watershed into three distinct flow types: sheet flow, shallow concentrated flow, and open channel flow.

6.4.4.3.1 Sheet Flow

Sheet flow occurs in the upper part of the basin where there is not yet a defined channel. The assumption is that the depth of flow is very shallow, less than 0.1 feet, and that the total flow length is less than 300 feet. The equation for sheet flow is as follows:

$$T_t = \frac{0.007(nL)^{0.8}}{(s^{0.4})(P_2{}^{0.5})} \qquad (6.6)$$

Where:

T_t = travel time (hr)

n = Manning's n roughness coefficient (refer to TR-55 for example values)

L = flow length (ft)

P_2 = 2-year, 24-hour rainfall (in)

s = slope of hydraulic grade line (land slope, ft/ft)

6.4.4.3.2 Shallow Concentrated Flow

Travel time for shallow concentrated flow is estimated by multiplying the average velocity by the total length of flow. Average velocity is found using the Figure 6.1 below.

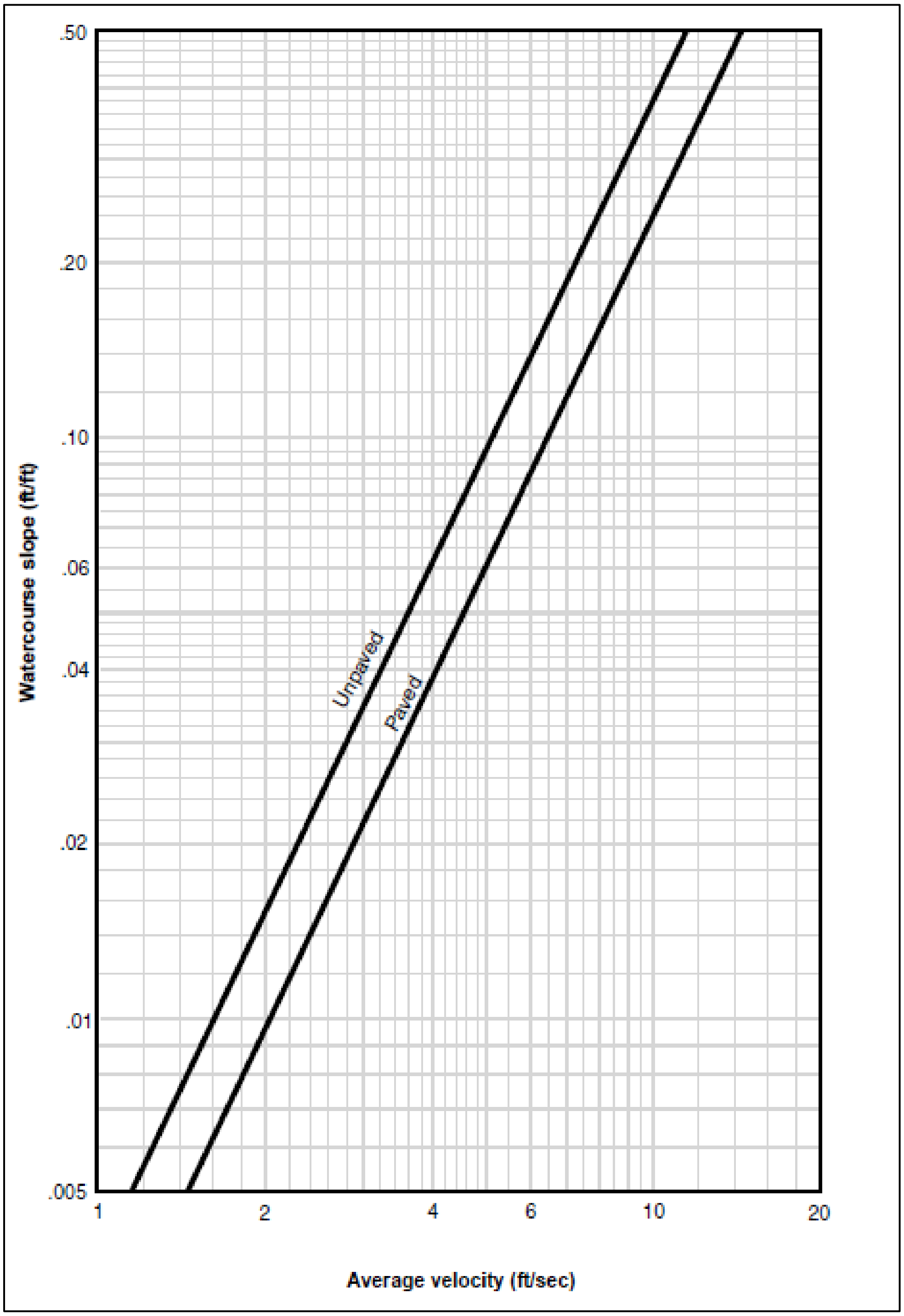


Figure 6.1. Average Velocities for Shallow Concentrated Flow
Reference: TR-55

6.4.4.3.3 Open Channel Flow

Open channel flow begins where there is a visible channel conveying storm water. Average velocity in the channel is estimated assuming the bank full flow. Velocity at bank full flow is calculated using Manning's equation or other available hydraulic modeling for the stream.

The total T_c is calculated as the sum of the three flows: sheet flow, shallow concentrated flow, and open channel flow. Some hydrologic models represent T_c as the lag time for the watershed. Lag time is defined as follows:

$$\text{Lag Time} = 0.6 * T_c \qquad (6.7)$$

Where:

T_c = time of concentration (hr)

6.4.5 Hydrograph Development and Routing Models

Runoff hydrographs can be developed using CUHP, SWMM, or HEC-HMS modeling software. CUHP, which stands for Colorado Urban Hydrograph Program, was developed by the Mile High Flood District for use within SWMM. SWMM, which stands for Stormwater Management Model, was developed by the United States Environmental Protection Agency (EPA). SWMM can route hydrographs developed internally or those developed with CUHP. HMS, which stands for Hydrologic Modeling System, was developed by the United States Army Corps of Engineers' Hydrologic Engineering Center (HEC). HEC-HMS can also route the hydrographs it develops.

SWMM and HEC-HMS can evaluate runoff through larger or less homogenous basins or through complex drainage systems by routing flows through these elements. Other routing programs may be used if they are approved by the Town Engineer. The SCS Method should be used within HEC-HMS to generate runoff hydrographs. The EPA developed its own runoff method inherent to the SWMM software program. The SCS Type II storm distribution and NOAA Atlas 14 rainfall depths should be used to develop the hyetograph for input into the model. Other methodology may be used if it is better suited to site characteristics. A narrative describing the selected hydrologic methodology and the reasons for its selection should be included with the Drainage Report.

Basins should be divided into smaller and more homogeneous subwatersheds. The runoff hydrographs from upper subwatersheds are then routed through and combined with hydrographs from the lower subwatersheds, channels, detention basins, or other elements to develop a runoff hydrograph for the entire watershed.

The sizing of the detention storage may be based upon hydrograph storage routing techniques rather than direct calculation of volume and discharge requirements. A HEC or EPA SWMM model shall be used when reservoir routing of hydrographs is conducted, such as for the design of a detention pond. A HEC or SWMM model shall be used when channel routing of hydrographs is required.

6.4.5.1 HEC-HMS

HEC-HMS simulates the complete hydrologic process of a watershed system. The software includes traditional hydrologic analyses including infiltration, unit hydrographs, and hydraulic routing including through detention facilities. HEC-HMS also provides analysis tools for snowmelt. It should be noted that when using the SCS Type II storm distribution, the model is assuming that the storm distribution is for a 24-hour storm. If a 6-hour storm is desired, then the hyetograph must be input as a user-defined table.

6.4.5.2 EPA SWMM

SWMM is a rainfall-runoff model that analyzes a collection of subwatersheds that receive a rainfall hyetograph and generates runoff hydrographs at design points. SWMM can route runoff hydrographs through a system of pipes, channels, basins, and storage elements such as detention ponds; track the quantity of runoff generated at each design point; and track the flow rate and depth of water in each element over a specified simulation time. EPA SWMM is typically used for larger watersheds; for designing detention basins and water quality facilities; and for modeling large storm sewer systems. Runoff hydrographs can be developed internally or within CUHP.

6.4.5.3 CUHP

CUHP is a rainfall-runoff model that analyzes a collection of subwatersheds that receive a rainfall hyetograph and generates runoff hydrographs and total runoff volumes at design points. The runoff hydrographs are then routed by SWMM through a system of pipes, channels, basins, and storage elements such as detention ponds to determine peak flows and volumes.

6.4.6 FEMA Flows

FEMA has established regulatory 100-year peak flow rates for some of the larger streams within the Town. Where these flow rates have been established, the Town Engineer should be consulted to determine the validity of these flow rates as it is not uncommon for them to be outdated. In some instances, a new hydrologic study may be required to evaluate current peak 100-year flow rates along FEMA-regulated streams. This study would then be submitted to FEMA for review and approval to revise the regulatory flow rates. The 2018 Breckenridge Flood Damage Prevention Ordinance presents the Town’s requirements for work near FEMA floodplains and shall be consulted prior to design of any work near a FEMA floodplain.

6.5 OPEN CHANNELS

Open channel flow occurs when water has its surface exposed to the atmosphere. Open channels can be natural waterways, canals, ditches, swales, culverts, flumes, and gravity pipes that are not flowing full. Once water fully fills a closed conduit, flow in that conduit becomes pressure flow. This section discusses types of open channel flow and gives general design guidance for open channels including roadside ditches. Any work in natural channels may be subject to FEMA floodplain regulations. Chapter 2 (Section 2.4) discusses the permits that will be required for work in open channels as well as the applicability of the 2018 Breckenridge Flood Damage Prevention Ordinance to work in open channels.

For a more thorough discussion of open channel design principles, the user is encouraged to review the most recent version of the Urban Storm Drainage Criteria Manual (USDCM) by the Mile High Flood District (MHFD).

6.5.1 Open Channel Hydraulics

The hydraulics of an open channel can be complex, ranging from steady state uniform flow to unsteady, rapidly varied flow. Most drainage design involves uniform, gradually varied, or rapidly varied flow states. Steady uniform flow occurs when the depth of flow remains constant. The calculations for both uniform and gradually varied flow are relatively simple and assume parallel streamlines. In contrast, rapidly varied flow calculations, for hydraulic elements like hydraulic jumps and flow over spillways, have solutions that are generally empirical in nature. This section presents basic equations and computational procedures for uniform, critical, gradually varied, and rapidly varied flow for hydraulic jumps and weirs.

6.5.1.1 Uniform Flow

Open channel flow is considered uniform if the depth of flow is the same at every section of the channel. For a given channel geometry, roughness, discharge, and slope, there is only one possible depth for maintaining uniform flow. This is called the normal depth. For a prismatic channel cross section, the water surface will be parallel to the channel bottom during uniform flow. Uniform flow rarely occurs in nature and is difficult to achieve, even in a laboratory. However, channels are designed by assuming uniform flow as an approximation that is adequate for planning purposes.

Calculations for normal flow depth shall be based on Manning's equation shown as Equation 6.8. A spreadsheet can be developed for this equation as an effective tool for quick analysis.

$$Q = \frac{1.49}{n} A^{5/3} P^{-2/3} \sqrt{S} = \frac{1.49}{n} AR^{2/3} \sqrt{S} \qquad (6.8)$$

Where:

Q = flow rate (ft^3/s)

n = Manning’s roughness coefficient (see Table 6.8)

A = area (ft^2)

R = A/P = hydraulic radius (ft)

P = wetted perimeter (ft)

S = Channel or pipe slope (ft/ft)

Open channel flow velocity can be more easily calculated by rearranging Equation 6.8 to yield Equation 6.9. Equation 6.9 may be used to calculate travel time for the Rational Method. It may also be used to design channel revetment.

$$V = \frac{1.49 R^{2/3} S^{1/2}}{n} \qquad (6.9)$$

Where:

V = average velocity (fps)

For prismatic channels with uniform flow, the slope of the energy grade line (EGL), hydraulic grade line (HGL), and bottom of channel are assumed to be equal. Table 6.8 provides recommended Manning roughness coefficients for various channel linings and conditions. As channel roughness increases, a given flow rate will have a greater depth and slower velocity. Decreased roughness results in shallower depth and faster velocity. Selection of roughness coefficients for both the main channel and the overbanks is a critical part of the design and evaluation of an open channel, but it is also based on engineering judgement. Two engineers may look at the same natural channel and assign different Manning’s n values.

Manning’s n values may be selected to be slightly conservative for the variable being calculated. For example, when calculating a channel velocity, a slightly lower Manning’s n value will result in a slightly higher calculated velocity, ensuring that channel revetment is adequately sized. When calculating channel capacity, a slightly higher Manning’s n value will result in a slightly larger channel cross section, offering a factor of safety for channel sizing. Another factor to consider might be the anticipated long-term condition of the channel, as Manning’s n values are prone to increasing if the channel is not properly maintained. Ultimately the Manning’s n value selected by the design engineer must be defensible given the values presented in Table 6.8.

Table 6.8. Manning's Roughness Coefficients for Channels

Type of Channel & Description	Manning's n	Type of Channel & Description	Manning's n
Excavated or Dredged		**Lined or Built-Up Channels**	
Earth, straight & uniform		Riprap	
Clean, recently completed	.018	Riprap	Eq. 6.12
Clean, after weathering	.022	Concrete	
Gravel, uniform section, clean	.025	Trowel Finish	.013
With short grass, few weeds	.027	Float Finish	.015
Earth, winding & sluggish		Gunite, good section	.019
No vegetation	.025	Gunite, wavy section	.022
Grass, some weeds	.030	Concrete Bottom	
Dense weeds in deep channels	.035	Dressed stone in mortar	.017
Earth bottom & rubble sides	.030	Random stone in mortar	.020
Stony bottom & weedy banks	.035	Dry rubble or riprap	.030
Cobble bottom & clean sides	.40	Asphalt	
Dragline-excavated or dredged		Smooth	.013
No vegetation	.035	Rough	.023
Light brush on banks	.040	Gravel bottom with sides of	
Rock cuts		Formed concrete	.020
Smooth & uniform	.035	Random stone in mortar	.013
Jagged & irregular	.040	Dry rubble or riprap	.016
Channels not maintained, weeds & brush		Grassed	
Dense weeds	.080	Short grass prairie	0.15
Clean bottom, brush on sides	.050	Dense grasses	0.24
Natural Channels		Range (natural)	0.13
Natural channels, good condition	0.025	Wooded	
Natural channels, stones & weeds	0.035	Light underbrush	0.4
Natural channels, poor condition	0.06	Dense underbrush	0.8

References: Chow, V.T., Open Channel Hydraulics (1959), NRCS (1986)

For riprap lined channels, Equation 6.10 shall be used to calculate Manning's n value.

$$n = 0.0395 D_{50}^{1/6} \qquad (6.10)$$

Where:

D_{50} = mean riprap stone size (ft)

6.5.1.2 Critical Flow

Critical flow in an open channel is characterized by the following conditions:

1. The specific energy is at a minimum for a given discharge.
2. The discharge is at a maximum for a given specific energy.
3. The specific force is at a minimum for a given discharge.
4. The velocity head is equal to half the hydraulic depth in a channel with a minimal slope.
5. The Froude Number (Fr) is equal to 1.0.

When critical flow exists for uniform flow, the channel slope is at the critical slope. A slope flatter than critical will cause subcritical flow and result in a Froude number smaller than 1.0. A slope steeper than critical will cause supercritical flow and result in a Froude number larger than 1.0. When flow is at or near critical, it is unstable because minor changes in specific energy, such as from channel debris, will cause a major change in depth. Equation 6.11 should be used to calculate the Froude Number for all open channel designs.

$$\mathrm{Fr} = \frac{\mathrm{v}}{\sqrt{\mathrm{g}\mathrm{D_h}}} \tag{6.11}$$

Where:

Fr = Froude number (dimensionless)

v = velocity (ft/s)

g = gravitational acceleration (32.2 ft/s^2)

D_h = hydraulic depth, A/T (ft)

A = channel flow area (ft^2)

T = top width of flow area (ft)

6.5.1.3 Gradually Varied Flow

Gradually varied flow is most often seen as backwater created by culverts, inlets, and channel constrictions. For these conditions, flow depth will be greater than normal depth in the channel and the water surface profile must be computed using a backwater technique—either the direct step or the standard step method. The direct step method is best suited to the analysis of simple prismatic channels, whereas the standard step method is best suited for irregular or nonuniform cross-sections.

Hydrologic Engineering Center's River Analysis System (HEC-RAS), developed by the U.S. Army Corps of Engineers, is recommended for calculating water surface profiles in the Town of Breckenridge. If a designer would like to compute water surface profiles by hand, the methodology for using both the direct-step and standard-step methods can be found in the HEC-RAS Hydraulic Reference Manual (Brummer, 2016), as well as in Open Channel Hydraulics (Chow, 1959).

6.5.1.4 Rapidly Varied Flow

Rapidly varied flow has a very pronounced curvature of the streamlines. The change in curvature may be so abrupt that the flow profile is virtually broken, resulting in high turbulence. Common instances of rapidly varied flow include weir flow, orifice flow, and hydraulic jumps. Only hydraulic jumps will be discussed in this section. In the Town, weir and orifice flow are used almost exclusively for detention pond outlets and will be discussed in Section 6.9.

Hydraulic jumps may occur at grade control structures, inside storm drains or culverts, and at the outlet of a spillway and can be very erosive and affect hydraulic capacity. For grassed channels, the forces from a hydraulic jump must be controlled to prevent serious damage. Drops or other grade control structures can be used to direct the jump to an area specifically designed to resist the forces that come with it.

Jump locations within storm drain systems can be approximated by intersecting the energy grade line of the supercritical and subcritical flow reaches. There is little threat of damage to storm drains, but pipe capacity may be impacted. The effect on pipe capacity can be determined by evaluating the energy grade line and accounting for the energy lost by the jump. In general, for Froude Numbers less than 2.0, energy loss is less than 10%. For long concrete boxes, the concerns of the jump are the same as for storm drains. However, the jump can be adequately defined for box conduits and for spillways using the jump characteristics of rectangular sections. These Standards do not include a detailed evaluation of hydraulic jumps, but the USDCM has procedures that can be used. Calculations must be included with the required submittals in accordance with Chapter 2.

6.5.2 Open Channel Design

The design standards for all open channels in the Town, except for roadside ditches addressed in Section 6.5.3, are those in the most recent edition of the USDCM. The design standards in the USDCM include channel centerline alignment and cross section layout, hydraulic analysis, and using rocks and boulders for protection from erosion. The design process for an open channel can be somewhat circular because of a wide range of options available for materials, typical cross section, channel slope, and the frequency and height of drop structures.

6.5.2.1 Channel Selection Factors

Each type of channel must be evaluated for hydraulic, structural, environmental, sociological, maintenance, economic, and regulatory factors. Table 6.9 summarizes the multi-disciplinary factors that should be used when selecting the channel that is most suitable for a specific site.

Table 6.9. Factors to Consider for Channel Design

Hydraulic	Structural	Environmental	Sociological	Maintenance	Regulatory
Topography	Cost	Habitat	Pedestrian	Lifespan	Federal
Capacity	Shear Stress	Water Quality	Recreation	Accessibility	State
Slope	Momentum	Traffic Patterns	Demographics	Repair	Local
Offsite Drainage	Seepage & Uplift	Aesthetics	Social Patterns	Reconstruction	Right-of-Way
Basin Sediment Yield	Material Availability	Wetland Mitigation		Maintenance Activities	
	Haul Off Site	Green Area Need			

6.5.2.2 General Design Guidelines

Except for roadside ditches and the additional criteria in these Standards, all open channel improvements shall be designed in accordance with the latest versions of the Open Channels, Hydraulic Structures, and Stream Access and Recreational Channels chapters of the USDCM.

All open channels within the Town shall be designed to convey both the minor and major design storms in a subcritical flow condition with a Froude number of less than 0.80. The major storm shall not result in a flow depth greater than 4.0 feet at any point along the channel reach. All open channels shall also be designed with public safety in mind and adequate maintenance access shall be provided.

Natural channels and grass-lined channels are preferred, and concrete-lined and riprap-lined channels are discouraged. Channel improvements that drastically change the look, shape, lining, alignment, or flow characteristics of the existing channel should be avoided. Improvements to natural channels should strive to maintain the capacity and alignment of the existing channel. In the event an entirely new channel is required, such as through a new development, it should closely mimic natural channels in the surrounding area with the same capacity.

The design components that have the greatest potential effect on the performance and cost of the improvements should be evaluated early on to guide the design process. High cost items include riprap channel linings and boulder drop structures, and the engineer should strive to design open channels to minimize the need for and use of these elements. Consideration should also be given to long term maintenance and repair costs.

6.5.2.3 High Gradient Channels

In mountainous areas, natural channels can have steep grades with cobble or rock along their bottoms. While uniform flow calculations with standard channel roughness values generally predict supercritical flow, field observations show that these channels are often protected by natural armoring. Field investigations have resulted in procedures for estimating hydraulic roughness for these streams that result in lower calculated velocities than those obtained with the Manning's equation using a uniform roughness coefficient. The designer is encouraged to review Determination of Roughness Coefficients for Streams in Colorado by Robert D. Jarrett in cooperation with the Colorado Water Conservation Board.

Equation 6.12 may be used as an aid in predicting the roughness coefficient of a high-gradient channel provided the following conditions are met.

1. The channel must be a natural channel that has a relatively stable bank material and a cobble or boulder bed material.
2. The channel friction slope must be between 0.01 and 0.04 feet per foot and the hydraulic radius must be between 0.5 and 7 feet.
3. The channel must not be affected by backwater.

$$n = 0.39 S_f^{0.38} R^{-0.16} \quad (6.12)$$

Where:

n = Manning's roughness coefficient

S_f = channel friction slopes (ft/ft)

R = hydraulic radius, A/P (ft)

Additionally, while mountainous channels may have high average grades, they have often achieved this by cutting very steep drops interspersed along what are otherwise flatter reaches. The analysis of a natural mountain stream requires a careful topographical investigation. The hydraulic model must recognize that friction slope, hydraulic radius, and n value may change frequently along the length of the channel and take this into account by dividing the channel into reach lengths of reasonably uniform discharge, depth, slope, and channel and floodplain geometry. Determination of Roughness Coefficients for Streams in Colorado gives an in-depth discussion of suggested reach lengths and subdivision of cross sections to be used in the hydraulic model.

Natural channels have typically reached a reasonable state of equilibrium based on the amount of peak runoff they are accustomed to receiving. Although a new development may not encroach on the floodplain of a natural channel, it is also critically important that it does not increase the peak runoff the channel receives. This could easily cause erosion of the channel and require costly remediation.

If site conditions suggest use of Equation 6.12 might be appropriate, the designer shall consult with the Town to confirm its applicability and discuss any additional specific site concerns regarding the stability of the natural channel. Development shall be planned around natural channels so they remain in place in their natural alignment. If a Developer believes there is a benefit to realigning a natural channel, the proposed modifications shall be submitted to the Town Engineer for review. If the average slope of an existing natural channel through a development is greater than 1.0%, the existing natural channel should not be reconfigured either in horizontal or vertical alignment to suit development unless a geotechnical investigation identifies that the channel is unstable in its current condition. Rather, development should be planned to accommodate the location of the natural channel and its existing floodplain.

6.5.2.4 Ecological Channel Design Guidelines

The USDCM puts a considerable amount of emphasis on preserving and restoring natural stream corridors. The Town of Breckenridge strongly supports using ecological concepts to preserve and restore local channels. Ecological channel design includes bioengineering practices that utilize vegetation in a combination with natural structural measures to stabilize and protect stream banks while providing habitat.

Ecological channel design can have numerous public and environmental benefits when applied in an appropriate location, but care should be taken in selecting the location and completing design calculations to ensure an ecological channel design will hold up under the stream forces it is intended to withstand. Numerous types of bioengineering components can be used. Table 6.10 lists some of the potential advantages and disadvantages of an ecological channel design, as opposed to more traditional riprap and concrete design concepts. The potential for every channel restoration project to include ecological components shall be examined and discussed with the Town.

Table 6.10. Ecological Channel Design Advantages and Disadvantages

Advantages	Disadvantages
Environmental clearances (may facilitate permits)	Potentially more expensive
Aesthetically pleasing	Specialized vegetation
Fish passage	Additional maintenance required
Habitat for fish, birds, and macroinvertebrates	Susceptible to failure during larger storms
Open space creation and preservation	May require a larger footprint
Water temperature moderation	Specific hydrologic conditions required
Water quality enhancement	

Ecological channel design may be applied when the overall channel design is firmly rooted in engineering principles and when the following conditions are met:

1. Hydrologic conditions are favorable for establishment and successful growth of vegetation.
2. Designs are conservative in nature, and bioengineered features are used to provide redundancy.
3. Maintenance responsibilities are clearly defined.
4. Adequate structural elements are provided for stable conveyance of the major storm runoff.
5. Species are selected based on individual site characteristics.

It is important to note that bioengineered elements are commonly designed to withstand flows from more frequently occurring storms. Design events are typically between the 1.5-year to 10-year storm, with the 100-year storm occasionally being a consideration. While designing for a larger event is prudent, stability during such events may often be achieved by traditional engineering techniques because bioengineered elements may not remain stable above a certain threshold. If stability is critical at a given location, such as at bridge piers, bioengineering measures may not be enough without the addition of traditional engineering techniques. Bioengineering techniques can be incorporated into almost all traditional engineering projects, often to great ecological benefit. The design approach must balance ecological function with the need for channel stability when selecting a design discharge. Both the Town and the design engineer should discuss and agree upon the various ecological and hydraulic criteria the design will meet.

The key elements to consider in an ecological channel design include hydrology, hydraulics, geomorphology, physiochemistry, and biology. Each of the following elements should be addressed when designing the channel:

1. Future hydrologic changes associated with urbanization
2. Channel stability
3. Hydrology to support vegetation
4. Supplemental structural measures

The USDCM should be reviewed as part of the design process because it offers valuable guidance on typical minimum standards. The Natural Channel Design Review Checklist published by the US Fish and Wildlife Service should also be reviewed to ensure that all appropriate parameters have been considered.

No specific criteria or design guidance is included in these Standards because each site is unique and will require a solution based on the characteristics of and goals for each site. However, there are publications that offer guidance on ecological channel design, and these should be consulted to ensure the design will stand up to the chosen design hydraulic event. The Technical Supplements contained in *Stream Restoration Design* (National Engineering Handbook 654) (NRCS, 2007) offer extensive and detailed guidance on the physical design of ecological channels. Specifically, Technical Supplement 14, letters I through O, offer design guidance and equations for soil bioengineering, using large woody material for habitat and bank protection, vegetated rock walls, fish passage, and fish lunkers, among many other components.

Monitoring and maintenance should be performed throughout the life of the ecological channel design. The following list consists of four periods when a bioengineered structure is most at risk:

1. Immediately after construction
2. During the driest time of the year
3. During high magnitude discharge events
4. When a shift in plant community occurs away from plants chosen for biostabilization.

6.5.2.5 Revegetation

To achieve the highest likelihood of establishment of the specified vegetation, a 3-year maintenance plan from a certified landscaping company that understands native vegetation is required. The Town shall be consulted to provide a site-specific seed mix for each project. Plantings need to be completed in the fall or late winter to provide the best odds of establishment. Depending on the site, irrigation may also be required. Other techniques to improve the odds of successful vegetation establishment are in Chapter 7.

6.5.3 Roadside Ditch Design

Much like the design of any open channel, design of roadside ditches is a balance of several design components, including velocity, capacity, available right-of-way, slope, and cross-sectional geometry. Chapter 5 discusses several constraints and factors to consider when laying out a roadside ditch. The capacity requirements of a roadside ditch are based on the roadway encroachment criteria discussed in Section 6.6.

This section discusses permissible velocities and Froude numbers for a roadside ditch. Roadside ditch hydraulic calculations will be completed using Manning's equation. The Manning's roughness coefficients for calculating velocity, Froude number, shear stress, and capacity included in Table 6.8 will be used for all roadside ditch calculations. The designer should note that if a ditch is expected to be vegetated there is a much higher potential for erosion until revegetation is complete. The use of erosion control measures such as turf reinforcing mat prior to revegetation will minimize this potential.

Roadside ditch flow with design depths less than or equal to 1.0 feet have no Froude number or velocity limitations. For ditch flow depths greater than 1.0 feet, velocity shall not exceed 7.0 feet per second, and the Froude number shall not exceed 0.8. These criteria are shown in Table 6.11.

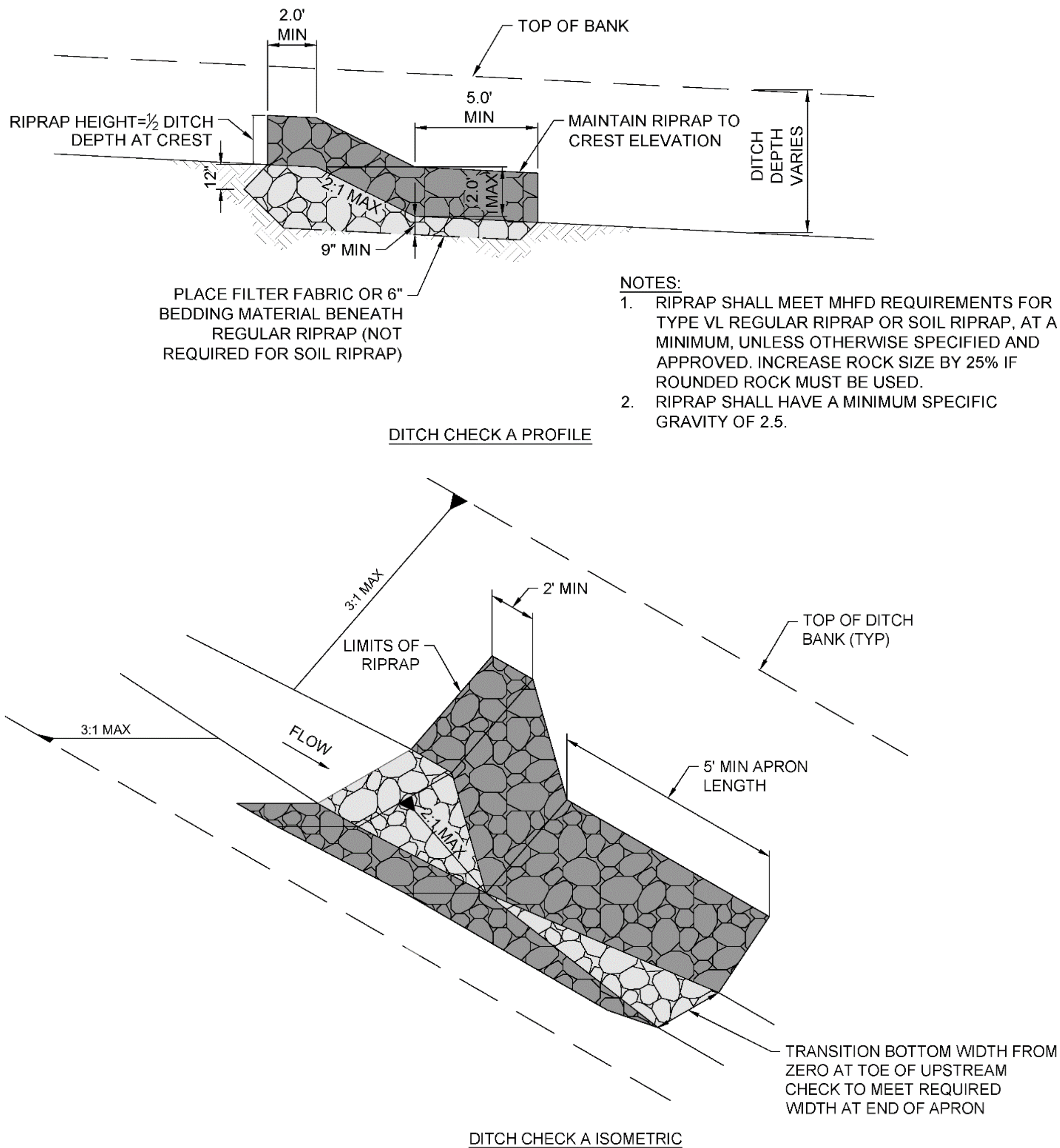
Table 6.11. Allowable Velocity and Froude Number for Roadside Ditches

Design Component	Maximum Allowable Values	
	Flow Depth ≤ 1.0 feet	Flow Depth > 1.0 feet
Velocity	No maximum	7 ft/s
Froude Number	No maximum	0.8

It is preferable that roadside ditches have side slopes no steeper than 3H:1V. If right-of-way is constrained, the ditch slope may be steepened to 2H:1V. Slopes steeper than 3H:1V shall be stabilized with erosion netting or stabilized with a method approved by the Town Engineer. Where right-of-way allows, roadside ditches will have a flat bottom at least two feet wide, but ditches may be V-shaped if right-of-way is constrained, with approval of the Town Engineer. Roadside ditches should ideally be designed as grass-lined channels without the need for riprap revetment. If riprap revetment is required, buried soil riprap shall be used in accordance with the design guidance in the USDCM and the ditch shall be revegetated.

Where roadway slopes are too steep to accommodate a ditch design that will meet velocity or Froude number criteria, a flattened ditch slope may be used with ditch checks placed at intervals to make up for grade discrepancies. An example of a ditch check is shown in Figure 6.2. In no case shall a roadside ditch have a slope steeper than 4%, regardless of whether allowable velocity and Froude number values are met.

Figure 6.2. Ditch Check Schematic



6.6 STREETS AND ROADSIDE CONVEYANCE

The primary function of public streets is the movement of traffic. The use of streets as part of the drainage system must be limited to prevent interference with traffic. Streets also typically convey runoff collected on the street surface as well as from some limited portion of the surrounding area. Streets must be capable of conveying that runoff to either a storm drain or open channel system. The drainage system in many parts of the Town is an open channel system with roadside ditches, which are discussed in Section 6.5. In dense and newly developed areas, an enclosed storm drain system is more appropriate, as discussed in Section 6.7.

This section presents the limitations on stormwater in public streets regardless of the type of roadside conveyance used. Limitations are established with roadway function and public safety in mind and are presented in terms of ponding depths at the curb face, the depth of flow permitted at the roadway crown, and the width of roadway that must remain clear during a storm event. Criteria vary based on the design storm and the roadway classification. When runoff in the street exceeds the allowable limits, a storm drain system, an open channel, or a combination of both is required to convey the excess flow. In all cases, the most stringent criteria will apply.

6.6.1 Allowable Flow Depth and Spread

Each street or roadway in the Town of Breckenridge is classified based on its role in connecting and providing access within and between various land uses. These classifications are available from the Town. The extent to which runoff from the minor or major design storm may encroach onto a roadway is based on that roadway's classification. Limiting the encroachment of stormwater onto a roadway section is the primary criteria by which public safety is maintained during a storm event.

Although many roads will utilize a roadside ditch to convey stormwater along the road, some will have a curb and gutter section. Using a curb and gutter or roadside ditch to convey flow along a street does not affect encroachment criteria because safety concerns remain the same for all types of roadways. The allowable encroachment onto the roadway for each roadway classification is presented in Table 6.12. These criteria may include the width of the roadway that must remain free of water or the allowable depth of flow at certain points along the roadway cross section. Curb overtopping criteria applies only to streets with a curb and gutter section.

In no case shall any roadway improvement, reconstruction, or expansion cause more flow encroachment on a parcel or structure outside the public right-of-way than currently exists. These criteria apply to roads with roadside ditches, curb and gutter sections, and culvert crossings. They do not apply to bridge crossings. Criteria for bridges are included separately in Section 6.8. Street inundation during both the minor and major storms must be analyzed for compliance with the criteria in this section.

Table 6.12. Maximum Allowable Flow Depth and Encroachment

Roadway Classification	Minor Storm Encroachment	Major Storm Encroachment
Arterial	10 feet clear each way; No curb overtopping; No encroachment on adjacent property	15 feet clear in center; Ponding below finished floor of all occupied structures
Major Collector	10 feet clear in center; No curb overtopping; No encroachment on adjacent property	Allowable depth at crown = 3 inches; Ponding below finished floor of all occupied structures
Minor Collector	10 feet clear in center; No curb overtopping; No encroachment on adjacent property	Allowable depth at crown = 6 inches; Ponding below finished floor of all occupied structures
Local	Flow may spread to crown; No curb overtopping; No encroachment on adjacent property	Allowable depth at crown = 9 inches; Ponding below finished floor of all occupied structures

Where roadside ditches are used to convey flow, they shall have sufficient capacity to meet the maximum encroachment and flow depth criteria in Table 6.12. Ditch geometry requirements are in Section 6.5 Open Channels.

6.6.2 Minimum and Maximum Grades

The minimum concrete or paved gutter grade shall be 0.5%. The minimum open channel grade shall be 1.0%. Maximum grades in roadside ditches shall meet criteria in Section 6.5 Open Channels.

6.6.3 Cross Street Flow

For all roadway classifications, flow in cross pans shall not exceed the limits set forth in Table 6.12.

6.6.4 Calculations

For roadway drainage, the minor and major storm must be evaluated separately for each side of the street using a Manning's n value of 0.016 for the gutter and street flow areas and a Manning's n value of 0.025 for sidewalk and grass areas, if needed. When a roadside ditch is used, a Manning's n value must be assigned based on the ditch lining.

6.6.4.1 Streets with Curb and Gutter

Design calculations can be performed manually, but this section assumes UD-Inlet will be used to calculate street capacity for those streets with curb and gutter. The USDCM provides additional details on the equations and methodologies that have been incorporated into the UD-Inlet spreadsheet, and guidance in the most recent version of the USDCM can be used for manual design. Note that as the MHFD updates their design spreadsheets they may change the prefix from UD to MHFD. The most recent version of UD-Inlet or MHFD-Inlet should be used.

A reduction factor from Figure 6.3 must also be applied to streets with curb and gutter, which will reduce effective street capacity. The reduction factor accounts for the increased effect on capacity that items like debris and parked cars can have at steeper roadway slopes. UD-Inlet includes these reduction factors automatically.

Street capacity calculations for the minor and major event shall be based on the following procedure:

1. Calculate the theoretical street capacity based on the allowable spread in Table 6.12.
2. Calculate the theoretical street capacity based on the allowable depth in Table 6.12.
3. Apply the appropriate reduction factor from Figure 6.3 to the theoretical flow rate based on allowable depth.
4. The lesser value from steps 1 and 3 is the allowable street capacity.
5. An inlet should be added whenever the runoff reaching the street exceeds the allowable street capacity for the major or minor event.

6.6.4.2 Streets with Roadside Ditches

Design calculations for streets with roadside ditches can be completed with a spreadsheet using Manning's equation in accordance with the design procedures in Section 6.5 Open Channels. The flow areas for the roadway and ditch must be separated so the appropriate Manning's n values can be used for each.

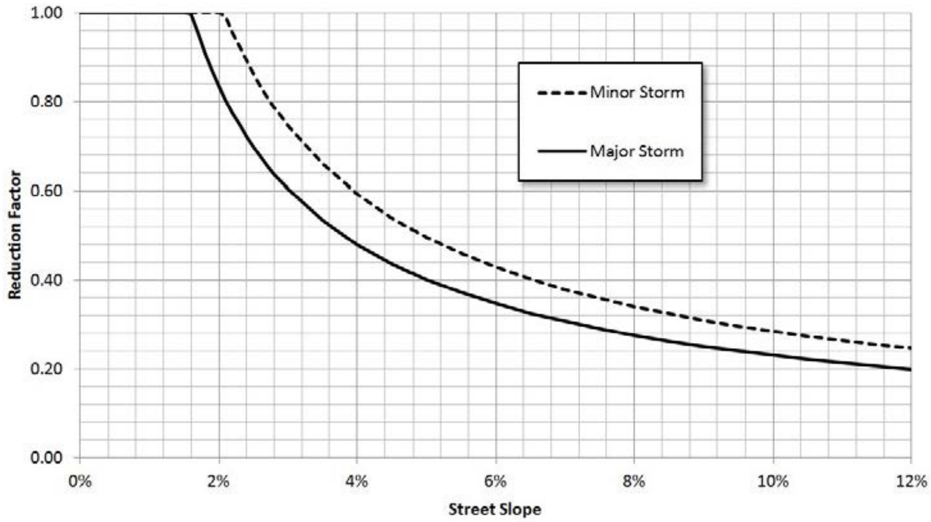


Figure 6.3. Reduction Factor for Gutter Flow
(UDFCD, 2018)

6.7 STORM DRAIN SYSTEMS

The criteria presented in this section shall be used to design and evaluate storm drain systems in the Town. A storm drain system refers to the system of inlets, pipes, manholes or junctions, outlets, and other appurtenant structures that are designed to collect and convey the initial or minor storm runoff. The storm drain system is a part of the local drainage system, which may also include curb and gutter, streets, roadside ditches, swales, and channels. This section presents both technical criteria and the general procedures for design and evaluation of pipes and inlets. Allowable roadway encroachment is in Section 6.6.

6.7.1 Storm Drain Design Criteria

A storm drain system is required when the allowable street capacity is exceeded during the minor storm event, or as required to eliminate the need for cross pans, or to prevent ponding/icing issues on roads. All criteria and guidelines below apply to the minor storm event unless site conditions offer no viable overflow option for the major storm event. Storm drain systems shall also be required where sump conditions exist and on major collectors and arterials to eliminate cross pans and ponding and icing near intersections.

6.7.1.1 Construction Materials

Pipe materials suitable for storm drains include reinforced concrete and high density polyethylene (HDPE). If the Town Engineer has reason to believe that pipe corrosion may be a problem, the Town Engineer may require a soils report to evaluate the corrosive potential of the soils and groundwater to be determined in the Geotechnical Report.

All pipe joint fillers, sealing compounds, gaskets, and the installation thereof, shall be in accordance with CDOT specifications. Rubber gaskets shall be used at pipe joints where the HGL is five feet higher or more than the pipe crown.

6.7.1.2 Pipe Size

The minimum allowable pipe diameter for storm drains shall be 18 inches for trunk lines and 15 inches for laterals. The minimum inside diameter of all pipes shall be no less than 14 inches for elliptical and arch pipe. In areas where debris, sediment deposition, adjacent wildfire burn area, or freezing are potential problems, the Town Engineer may require larger pipes. There is no maximum pipe size;

however, using multiple smaller barrels in lieu of very large pipes may be physically or economically advisable under some circumstances. For pipes below 5' diameter, a single barrel pipe is preferred over multiple smaller barrel pipes.

6.7.1.3 Horizontal and Vertical Alignment

Storm drains shall be designed and installed with sufficient cover to support an HS-20-44 loading in accordance with the pipe manufacturer's recommendations. Minimum and maximum cover are determined by the size, material, and class of pipe, as well as by the characteristics of the cover material and the expected surface loading. The designer should consult appropriate data sources to determine these values, but the minimum cover in all instances shall be greater than 24 inches of cover.

To prevent freezing issues, the following cover requirements shall be met as well, where feasible. In areas that are regularly plowed, storm sewers shall have a minimum cover of 5 feet. In areas not typically plowed, storm sewers shall have a minimum cover of 4 feet. Less cover may be allowed if winter flows are diverted and an alternate means of flow disposal is provided, such as a dry-well. If utility conflicts or grade restrictions do not allow the minimum values above, additional measures may be required to prevent freezing, including insulation around pipes, slotted pipes, deep inlets that are perforated or bottomless, and other measures as determined by the Town Engineer.

The following resources should be consulted to assist the designer in determining cover requirements:

1. Colorado Department of Transportation Standard Specifications for Road and Bridge Construction, Section 700 (Materials Details)
2. Concrete Pipe Design Manual (ACPA)
3. Handbook of Steel Drainage and Highway Construction Products (AISI)
4. Pipe Manufacturer Specifications
5. Other applicable references

Trench installations, including bedding, shall be in accordance with the most recent edition of the CDOT M&S Standard Plans. In manholes and junction boxes, the lowest inlet pipe invert elevation must be at least 0.2 feet higher than the outlet pipe invert elevation. In cases where inlet pipes are smaller than outlet pipes, the top of the inlet pipes shall be at least 0.2 feet higher than the top of the outlet pipes.

6.7.1.4 Utility Crossings

The Town's required minimum clearance between storm drains and other utilities is 12 inches, regardless of whether the crossing utility is above or below the storm drain. If either the storm drain or crossing utility is encased, the minimum clearance may be reduced to 6 inches. In all cases, backfill material, compaction, and additional protection shall be designed and provided to prohibit settling or failure of either the storm drain or crossing utility.

When storm drains cross above or within 18 inches below water mains, the storm crossing shall be constructed of a 20-foot section of pipe centered on the water main alignment, and the bounding joints shall be encased. Storm drain joints shall also be encased at all locations less than 10 feet horizontally from a water main. Encasement shall be a reinforced concrete collar, 6 inches thick, extended to 12 inches on either side of the joint. The minimum reinforcement shall be a minimum of four continuous #4 bars, equally spaced around the pipe, and tied with #3 bars around the pipe at 8 inches on center.

All work shall be in accordance with these Standards, the Town's Standard Drawings, CDOT M&S Standard Plans, or other approved details, and the design must be approved by both the Town and the utility owner. Utility owners may have more stringent requirements and local utilities shall be consulted to ensure these requirements are met.

6.7.1.5 Inlets and Manholes

The standard inlets permitted in the Town are a CDOT Type 13 combination inlet, a CDOT Type 13 valley inlet, and a CDOT Type C grated inlet. Type 13 combination inlets may also be called a Type 16 inlet. Type 13 combination inlets shall be used with a 6-inch vertical curb and gutter section and installed with a localized depression at least 2 inches below the gutter flowline elevation. Type 13 combination inlets shall be located, at a minimum, just upstream of curb ramps, and never in the ramp itself. Type C inlets shall be used in roadside ditches and installed in accordance with the CDOT standard plans, including creating a sump condition where one does not naturally exist. Type C inlets are not traffic rated and shall not be placed in roadways. Type C inlets may not be placed in sidewalks or other pedestrian routes. Other types of inlets, including those made of PVC, HDPE, or other materials are only allowed in landscaped areas outside of roadway and sidewalk clear zones.

Table 6.13 provides the maximum allowable manhole or junction box spacing for storm drains. Manholes are also required where there is a change in pipe size, vertical or horizontal alignment, elevation, or slope, and where there is a junction of two or more pipes or laterals. All manholes and junction boxes must provide access to the storm drain system for maintenance and inspection. All manhole and junction box inverts shall be formed with a minimum of a half bench to provide more hydraulically efficient flow through the manhole.

Table 6.13. Maximum Allowable Manhole Spacing

Vertical Pipe Dimension	Maximum Manhole or Junction Spacing
15 to ≤ 36 inches	400 feet
> 36 to ≤ 60 inches	500 feet
> 60 inches	750 feet

The required diameter of the manhole barrel is dependent upon the size of the largest pipe connecting to it. Minimum manhole sizes are in Table 6.14 and assume the storm drain alignment passes straight through the manhole with no incoming lateral lines. If a storm drain system changes alignment or must accommodate incoming lateral lines at a manhole, the manhole may need to be larger.

Table 6.14. Manhole Sizing

Maximum Pipe Dimension	Minimum Manhole Size
≤ 24 inches	4-foot diameter
> 24 to ≤ 42 inches	5-foot diameter
> 42 inches	6-foot diameter or box base

6.7.1.6 Capacity and Velocity

Minimum velocities are required in storm drains to reduce sedimentation and promote positive drainage through the pipe at all flow depths. Public and private storm drains shall have a minimum flow velocity of 3 feet per second when normal flow in the pipe is at 0.25D and never be constructed at less than 0.40% slope. Public and private storm drains shall have a maximum flow velocity of 16 feet per second for all storm events, although care should be taken with velocities this high as there is a higher risk for hydraulic jumps and extreme head loss through junctions.

The storm drain system shall be designed to convey the minor storm without resulting in pressure flow, and the energy grade line (EGL) for the minor storm shall be at or below finished grade at all manholes, inlets, or other junctions. Pressure flow during the major storm is discouraged, but if it does occur, it must not create a surcharged condition at any inlet. Where the 100-year hydraulic grade line (HGL) is above any manhole rim, or if any manhole or inlet is in a floodplain, all manhole and inlet covers must be bolted down.

The EGL and HGL for both the minor and major storm events shall be calculated and plotted for all storm drain systems. Hydraulic losses will include friction, expansion, contraction, and junction losses at a minimum. The methods for estimating these losses are presented in the following sections. Manning's n values for capacity and velocity calculations for storm drains shall be the values in Table 6.15. The design of storm drain outlets into open channels, including revetment shall meet the requirements of Section 6.8 for culvert outlets.

Table 6.15. Manning's Roughness Coefficients for Storm Drain Conduits

Pipe Interior	Manning's n
RCP (newer)	0.013
RCP (older)	0.015
RCP (preliminary design)	0.015
Smooth Plastic (HDPE)	0.011

When a planned storm drain system connects directly into the Town's existing storm drain system, an analysis must be provided showing the additional flow from the proposed project does not result in the capacity of the existing storm drain system being exceeded. The requirement for detention in Section 6.9 is intended to facilitate meeting this requirement.

6.7.2 Storm Drain Hydraulics

This section presents the general aspects of hydraulic design and evaluation of storm drains. Hydraulic design calculations can be performed manually with a spreadsheet or by using a computer model. Both methods are briefly discussed below. The user is assumed to possess a basic working knowledge of storm drain hydraulics and is encouraged to review technical literature available on the subject as needed.

6.7.2.1 Manual Calculations

Manual storm drain hydraulic calculations shall be performed in accordance with the HEC-22 (Brown et al., 2013) or the latest version of the USDCM. HEC-22 includes a discussion of both open channel and pressure flow and includes a design example.

Two of the critical design elements of a storm drain system are the HGL and the EGL. The HGL is a line that represents the water surface elevation along an open channel, including open channel flow within a pipe. In pressure flow, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. The EGL is an imaginary line that represents the total energy at any point in the system. Total energy includes elevation head, velocity head, and pressure head and is the HGL plus the velocity head ($V^2/2g$). The total energy at any location equals the energy at any downstream location plus the losses that occur between the two locations.

Losses are typically classified as either friction or form losses. Friction losses occur as water flows along the length of a pipe. Form losses occur at the exit from the system and at junction structures within the system. Because the Town does not allow transitions or bends outside of manholes, form losses will be restricted to exit losses when flow leaves the system, and structure losses, such as through inlets or manholes. These are referred to by HEC-22 as inlet and access hole losses.

6.7.2.2 Computer Model Calculations

Computer models are often used to calculate the HGL and EGL of storm drain systems. The benefits of using a computer model include consistency, speed, and the ability to check the validity of the model with relative ease. One disadvantage of computer modeling is that errors can occur and be hidden when the model user is inexperienced. Three common programs used throughout Colorado include those listed below. Additional software may be used as approved by the Town provided it utilizes industry standard calculation methods:

1. UD-Sewer 2009, a simple and free program developed by the MHFD that is easy to learn. Note that while UD-Sewer is still provided on the MHFD website and may still be used for new projects, the MHFD no longer supports this program.
2. Storm and Sanitary Analysis, a more complex design tool which runs within the AutoCAD Civil 3D design software package by Autodesk.
3. StormCAD, a more complex design tool which runs within the MicroStation design software package by Bentley Systems or within AutoCAD.

This section provides specific guidance for UD-Sewer 2009. The program uses Manning’s equation to analyze and size storm sewer systems. The program can also use the Rational Method to calculate runoff, perform HGL and EGL calculations, and provide plots of the storm drain, ground line, HGL, and EGL. The user’s manual is embedded in the software, which can be obtained from the MHFD website (https://mhfd.org/resources/software) or via an internet search for “MHFD UD-Sewer.”

6.7.2.2.1 Rational Method

UD-Sewer 2009 uses the Rational Method to calculate runoff based on input parameters provided by the user. The user can override Rational Method calculations by manually entering known flows that have been calculated separately; however, values must be entered for Rational Method parameters or the program will give an error.

6.7.2.2.2 Bend and Lateral Loss Coefficients

UD-Sewer 2009 requires bend and lateral loss coefficients for each storm drain segment within a model. Bend and lateral losses both occur at a manhole or inlet junction. Bend losses are the result of the angle between the incoming storm drain and the exiting trunk line at a junction. Lateral losses are the result of turbulence or eddies that occur from lateral flows joining a trunk line. These coefficients are calculated by the program based on user inputs that define the geometry of the system.

To calculate the bend loss coefficient, the user must select the shape of the manhole invert and enter the angle between the incoming and downstream pipe segments. To calculate the lateral loss coefficient, the user must enter the angle between the incoming lateral and downstream trunk line. When entering the angle, the user must select main line or lateral line. Lateral loss is only applied to the main lines of a storm sewer system in UD-Sewer 2009. For all lateral lines, the user should select lateral line and the program will default to zero. If more than one lateral enters a manhole, the user must exercise judgment to determine the appropriate loss coefficient. .

6.7.3 Inlet Hydraulics

This section presents the general procedures for sizing and spacing inlets for a storm drain system. Design calculations can be done manually, but this section focuses on the use of UD-Inlet to calculate street and inlet capacity. The USDCM provides additional details on the equations and methodologies that have been incorporated into the UD-Inlet spreadsheet, and guidance in the most recent version of the USDCM shall be used for manual design. Note that as the MHFD updates their design spreadsheets they may change the prefix from UD to MHFD. The most recent version of UD-Inlet or MHFD-Inlet should be used.

6.7.3.1 Introduction

Inlets on a continuous grade result in uncaptured flow bypassing the inlet and continuing to the next inlet in the system. Sump inlets are located at low points in the roadway vertical alignment which are known as sags or sumps; there is no way for excess water to bypass a sump curb inlet. A sump condition can occur at a change in street grade from positive to negative or at an intersection due to the crown of the cross street.

6.7.3.2 Inlet Capacity

Inlet capacity may be calculated using the UD-Inlet spreadsheet developed by the MHFD. UD-Inlet is an Excel-based program that calculates both street and inlet capacities based on several parameters entered by the user. In general, the procedure consists of defining the amount and depth of flow in the gutter and determining the theoretical flow interception by the inlet. The calculations within the spreadsheet program are based on physical research completed at Colorado State University. The most recent version of UD-Inlet can be obtained via an internet search for “UD-Inlet MHFD” or from the MHFD website (https://mhfd.org/resources/software).

Information required by the UD-Inlet spreadsheet includes design flow; height of curb; distance from curb face to street crown; gutter width; street cross and longitudinal slopes; gutter cross slope; Manning’s n for the street; maximum allowable spread from gutter flow line; maximum allowable depth at gutter flow line; and allowable flow depth at the street crown. Additionally, if flow is allowed behind the curb, the allowable spread width, side slope, and Manning’s n behind the curb must be entered. The spreadsheet can use the Rational Method to calculate a design flow at the inlet or will accept a flow entered by the user. If the inlet receives bypass from an upstream inlet, the bypass flow can be entered or referenced from another UD-Inlet worksheet. Default clogging factors included in the UD-Inlet spreadsheet shall be used to account for potential debris clogging, pavement overlaying, and varying design assumptions.

6.7.3.3 Continuous Grade Condition

The capacity of an inlet on grade is dependent on street slope, depth of flow in the gutter, height and length of curb opening, street cross slope, and the amount of local inlet depression. Cost effective inlet design allows for some bypass. The amount of bypass or carryover flow must be included in the drainage facility evaluation as well as in the design of the inlet.

6.7.3.4 Sump Condition

Due to frequent freezing conditions and the associated hazards, sump conditions will only be allowed where no practical alternatives for grading and drainage exist. Where a sump condition is unavoidable, sump inlets shall be sized to have twice the capacity as otherwise required by these Standards to decrease the likelihood of ice buildup preventing the inlet from functioning.

6.7.3.5 Inlet Spacing

The optimum spacing of storm inlets is dependent upon several factors, including traffic requirements, contributing land use, street slope, allowable street capacity, and distance to the nearest outfall system. The suggested sizing and spacing of the inlets is based on an ideal interception rate of 70% to 80%. This spacing has been found to be more efficient than a spacing that will accomplish a 100% interception rate; although, the downstream-most inlet will still need to be designed to intercept 100% of the flow.

Inlet spacing is typically an iterative process, and the designer may have to move inlet locations multiple times before determining the appropriate spacing to meet design criteria and maintain efficiency. After initial inlet locations are determined, the designer should recalculate the peak flow to each inlet and check that the allowable street capacity has not been exceeded at any location. If the actual flow is less than the allowable street capacity, inlets may be spaced further apart to prevent overdesign of a system. Locating inlets is a balance between meeting criteria and efficient design. It is not usually possible to have optimum inlet spacing throughout an entire storm drain system The inlets must be spaced so that no portion of a roadway exceeds the spread criteria of this chapter.

6.7.3.6 Inlet Grates

All grates used on storm inlets in the Town will be bicycle-safe grates in accordance with the most recent version of the AASHTO Guide for the Development of Bicycle Facilities. Grates shall be Type L vane grates or approved equal style meeting AASHTO bicycle and ADA requirements. Grates shall be cast iron and rated to withstand HS-20-44 loading.

6.7.4 Design of a Storm Drain System

This section presents the general procedure used to design a storm drain system from preliminary through final design. A typical local drainage system consists of flow in the storm drain and allowable flow in the gutter and street. These flows are ultimately discharged to a larger drainage system or an open channel with capacity for a much larger event.

6.7.4.1 Preliminary Design

The preliminary design of the storm drain system begins after a preliminary development plan has been prepared that delineates the general development areas, major drainage paths, and drainage outfall locations. Allocation of space for drainage facilities and considerations shall be incorporated into the preliminary development plan. The drainage engineer must have input into the development plan to ensure proper drainage planning.

1. The first step in any drainage project is the collection of basic data. Information typically required includes:
    a. Topographic maps of the development and drainage basins that show existing and proposed roadways, existing and proposed land uses, major drainage features such as creeks and streams, development area, and property boundaries
    b. Typical street cross sections
    c. Preliminary grading information, such as contours, profiles, and control elevations
    d. Soils information
    e. Existing and proposed utilities
    f. Existing irrigation and raw water facilities and requirements for maintaining facilities
    g. Rainfall information
2. Perform the hydrologic evaluation of the basin(s) for both the minor and major storms, typically using the Rational Method. Divide each basin into smaller subbasins and calculate the peak design flow for each hydrologic point of interest or potential inlet location. The degree of basin subdivision will depend on the detail of information available and the experience of the licensed professional drainage engineer.
3. Complete preliminary sizing for the minor storm. Beginning at the upper end of the basin, calculate the flow in the street until the allowable capacity of the street as calculated in Section 6.6 matches the design runoff. The storm drain system will typically start at this point. Removal of all street flow by inlets is not required, except at sump locations, and is typically not

economical. The sum of the flow in the storm drain and the street must be less than or equal to allowable capacity.

4. Assign a diameter, pipe material, and slope for preliminary sizing. Manning's n values should be those in Table 6.15. A profile may be required to check for utility conflicts or to confirm compatibility with the receiving drainage system. The preliminary vertical alignment should not be steeper than the proposed street grade. The designer should also be aware of existing utility locations, especially when crossing water and sanitary main and service lines.
5. After sizing the system for the minor storm, route the major storm through the system and evaluate the results. The combined total of the allowable street capacity and the storm drain capacity during the major storm should equal or exceed the 100-year runoff. A plan and profile of the pipes and minor and major storm EGL and HGL is required. If the combined allowable capacity is less than the design flow, some or all the following actions may be taken:
   a. Increase storm drain sizes and/or the number and size of inlets
   b. Increase street grade within acceptable limits or revise street cross-sectional geometry to allow additional capacity
   c. Provide additional onsite detention within the development to decrease peak flow.
6. Evaluate the preliminary design for costs and benefits. The impact of the system on downstream properties must be evaluated and mitigated as needed.

6.7.4.2 Final Design

Final design consists of final revisions to the storm drain system model and preparation of plans, profiles, and specifications for the storm drain system in enough detail for construction. Basic data, hydrologic analysis, and inlet sizing performed during preliminary design should be reviewed and verified. Drainage subbasin boundaries should be confirmed or revised as necessary, and design peak flows should be recalculated. The pipe and inlet sizes and locations are finalized while accounting for final street and storm drain grades, locations of existing and proposed utilities, and the design of the major drainage system. The EGL and HGL should be revised with updated energy losses at manholes and any other structures.

6.8 CULVERTS AND BRIDGES

Culverts and bridges convey surface water through or beneath an embankment such as a roadway, railroad, or canal. The size, alignment, and support structures of a culvert or bridge directly affect the capacity of the drainage system. An undersized culvert or bridge will force water out of the channel and cause flooding and damage. Culverts and bridges may significantly influence upstream and downstream flood risks, floodplain management, and public safety.

The criteria presented in this section shall be used to evaluate and design culverts and bridges in the Town, regardless of whether they are located within public right-of-way. The review of all submittals will be based on the criteria in this section. Stormwater crossings of CDOT roadways may have additional requirements.

6.8.1 Culvert Design Standards

Culverts shall be designed and constructed to the following standards. All proposed culverts, regardless of whether they are in public right-of-way, are subject to review and approval by the Town Engineer.

6.8.1.1 Materials and Structural Design

Allowable materials for culverts include HDPE and reinforced concrete. The culvert materials and joints shall meet the most recent versions of the standards listed in Table 6.16 for each type of culvert.

Each culvert installation shall be designed to maintain its full shape and function under an HS-20-44 loading in accordance with the design procedures in the latest edition of the AASHTO Standard Specifications for Highway Bridges, or appropriate ASTM standard, and with the pipe manufacturer's recommendations. For roadway crossing culverts, the minimum cover shall be 24 inches measured from the top of the pavement. For culverts crossing private driveways, the minimum cover shall be 8 inches. In all cases, minimum cover over roadway crossing culverts shall also include at least 6 inches of aggregate base course under the pavement. Total minimum cover may need to be increased for thicker pavement sections. In all cases, the minimum and maximum cover shall be in accordance with the manufacturer's recommendations. Trench installations shall be in accordance with the most recent edition of the CDOT M&S Standard Plans.

Table 6.16. Applicable Culvert Standards

Culvert Type	Standard
Reinforced Concrete Pipe—Round	ASTM C76 or AASHTO M 170
Reinforced Concrete Pipe—Elliptical	ASTM C507 or AASHTO M 207
Reinforced Concrete Pipe—Joints	ASTM C443 or AASHTO M 198
Reinforced Concrete Box Culvert—Joints	ASTM C1677
Reinforced Concrete Pipe—Arch	ASTM C506 or AASHTO M 206
Precast Concrete Box Culverts	ASTM C1433/C1577 or AASHTO M 259/M 273
Concrete for Cast-in-place culverts	CDOT 601
High Density Polyethylene Pipe (HDPE)	ASTM F2306
Gaskets for Joining Plastic Pipe (HDPE)	ASTM F477

6.8.1.2 Minimum Size

The minimum pipe diameter for culverts in the public right-of-way that are not driveway culverts shall be 18 inches. The minimum vertical pipe dimension shall be 18 inches where elliptical or arch pipe is used. Equivalent sizes for a 24-inch round pipe are a 29-inch by 18-inch arch and a 30-inch by 19-inch elliptical section.

Roadside ditch culverts crossing private driveways shall have a minimum diameter of 18 inches. Roadside ditches shall be re-graded if necessary to provide positive drainage below the culvert and prevent ponding or a sump condition near the culvert. The developer is responsible for re-grading and reconditioning existing swales to prevent ponding or a sump condition. If existing conditions prohibit the installation of an 18" culvert, the Town Engineer may grant a variance to allow a smaller culvert.

6.8.1.3 Allowable Headwater

Ponding above culvert entrances can cause property or roadway damage, culvert clogging, saturation of fills, detrimental upstream deposits of debris, an increase in floodplain elevation, or inundation of existing or future facilities. The maximum headwater for the 100-year design flow shall be 2.0 times the culvert diameter or culvert rise dimension for shapes other than round ($H_W/D \leq 2.0$) for culverts with a rise dimension less than or equal to 36 inches. For culverts with larger rise dimensions, the headwater to depth ratio for the 100-year design flow shall be less than 1.5. There is no maximum headwater value for the minor storm. Table 6.17 lists these criteria. The criteria in Table 6.17 are in addition to roadway encroachment and overtopping criteria, and do not apply to detention, water quality, or sedimentation facility outlets.

Table 6.17. Allowable Headwater

Culvert Diameter or Rise	100-Year Maximum Headwater/Diameter (H_W/D)
≤ 36 inches	2.0
> 36 inches	1.5

6.8.1.4 Roadway Overtopping

No overtopping of any public roadway at a roadway culvert crossing is permitted during the minor storm. Roadway overtopping of up to 6 inches may occur at culvert crossings of local roads during the major storm. Overtopping at arterial, major collector, and minor collector roadways may not occur during the major storm. Overtopping of driveways at culverts in roadside ditches may not result in roadway encroachment that exceeds that specified in Section 6.6. These criteria should be considered as headwater limitations in addition to those in Table 6.17 above.

Additionally, culverts under arterial and major collector roadways, or those conveying flows from drainage areas larger than 0.50 square miles shall pass the 100-year storm assuming 20% of the inlet is plugged.

The depth of roadway overtopping is assumed to be the difference between the headwater elevation and the roadway crown elevation along the centerline of the culvert. Where overtopping is not permitted, but some amount of encroachment is permitted, the culvert headwater elevation can be set at the elevation corresponding to the limits of encroachment.

During roadway overtopping, the roadway crown is assumed to act as a broad-crested weir. A weir coefficient of 2.8 shall be assumed along with a weir length not to exceed 100 feet, regardless of roadway geometry. The designer should first calculate weir flow using the allowable overtopping depth for the major storm. The designer should then calculate flow through the culvert in accordance with these Standards, with culvert headwater set at the allowable overtopping elevation. If the calculated weir flow plus the flow through the culvert exceeds the design flow, the allowable overtopping condition has been met.

6.8.1.5 Velocity and Outlet Protection

A minimum flow velocity within the culvert of 3 feet per second is required to prevent sediment from accumulating in the culvert. The minimum flow velocity should be calculated using Manning's equation with a flow depth equal to 0.25 times the vertical dimension of the culvert. Manning's n values are presented in Table 6.15 above. Regardless of calculated flow velocity, the minimum slope of any culvert shall be 0.40%.

Culvert design must include revetment to protect the outlet from erosion if exit velocity exceeds 6 feet per second during the 100-year event as calculated in accordance with these Standards. The most common type of outlet protection is riprap, either as a riprap apron or as a low tailwater basin. Procedures for designing a riprap apron or low tailwater basin downstream of a culvert outlet, including for multiple conduit installations, can be found in the USDCM. Culverts should not be designed with 100-year outlet velocities greater than 16 feet per second.

An economical culvert design that meets allowable headwater requirements should not result in a Froude number larger than 2.5 when design velocities are kept below 16 feet per second. Culvert slopes should be as flat as practicable to limit the amount of revetment required at the outlet. A riprap apron is typically used when the culvert discharges to a well-defined channel that can be expected to have a tailwater elevation equal to at least one-third of the height of the culvert. A low tailwater basin is typically used when the receiving channel may have little or no tailwater or where the receiving channel is not well defined.

6.8.1.6 Headwalls, Wingwalls, and End Sections

Except for private driveway culverts, all culverts in the public right-of-way shall be designed with headwalls and wingwalls or flared end sections at the inlet and outlet to minimize head loss. Stone headwalls and wingwalls are not allowed in roadside ditches. Private driveway culverts less than 36 inches in equivalent diameter may have projecting ends. Headwalls, wingwalls, and end sections shall be in accordance with the most recent edition of the CDOT M&S Standard Plans. Headwalls or end sections shall be located to provide a grade no steeper than 3H:1V between the back of the structure and the edge of the roadway shoulder or back of walk. Ditch and channel sections and profiles shall be transitioned at culvert inlets and outlets to allow for adequate cover over the culvert and to provide inlet and outlet conditions that will not cause erosion or sediment deposition.

6.8.1.7 Grates

Where a clear and present danger exists such as a siphon, a drop in elevation adjacent to a sidewalk or road, a long pipe with one or more manholes, or at pipes which are near playgrounds, parks and residential areas, a grate at the inlet and outlet of the culvert may be required. For most pipes through embankments and under streets, grates will not be required.

6.8.1.8 Location and Alignment

Culverts shall be located to completely drain all rainfall and snowmelt runoff where drainageways intersect a roadway or sidewalk. All areas that water could be impounded, or flow restricted, by the new embankment must be identified and considered for culvert locations. Culverts shall be aligned to give drainageways a linear entrance and exit. Abrupt changes in alignment at either end of a culvert may retard flow and make a larger structure necessary. If possible, a culvert shall have the same alignment as the channel it conveys. If this is not practical, and the water must be turned into a culvert, headwalls, wingwalls, and aprons shall be used as protection against scour and to provide a more hydraulically efficient inlet.

Where the natural channel alignment would result in a culvert alignment skewed more than 30 degrees from perpendicular to a roadway, modification may be necessary. Such modifications will change the natural stability of the channel, and an investigation into other options is recommended. Although economic factors are important, hydraulic effectiveness of the culvert must be given primary consideration.

Roadway alignment also affects culvert design. The vertical alignment of roadways may define the maximum culvert diameter that can be used. Low vertical clearance may require the use of elliptical or arched culverts or the use of multiple barrels.

6.8.2 Bridge Design Standards

Based on hydraulic capacity requirements, bridges may be required to cross major open channels. Sizing bridge openings is of great importance. Improperly designed bridges may cause excessive scour or deposition or may not be able to pass the design flow. Backwater caused by bridges can cause flooding of upstream property, overtopping of roadways, or costly maintenance. Bridge openings should have as little effect on the flow characteristics as is reasonable, consistent with good design and economics. The criteria in this section apply to bridges on public and private roads in the Town. The Town will review bridge designs based on the guidance in this section.

It is possible that a bridge designed to meet the criteria of these Standards may be on a roadway that becomes flooded during the storm event the bridge is designed to pass. New bridges shall be designed to these Standards regardless of adjacent roadway flooding because roadways that experience frequent flooding may be reconstructed at a higher elevation in the future to achieve an overall greater level of protection.

6.8.2.1 Bridge Sizing

The low chord of all bridges must provide a minimum freeboard. All bridges on arterial and major collector roadways, or above the Blue River or its major tributaries, shall have a low chord elevation set at least 3 feet above the HGL. All other bridges shall have a low chord elevation at least 1 foot above the HGL. Arch culverts, large diameter culverts, or other structures may be classified as a bridge and subject to freeboard requirements. Freeboard requirements apply to both vehicular bridges, pedestrian bridges, and any other structures spanning a floodway.

6.8.2.2 Hydraulic Analysis

Hydraulic analysis and design calculations for all bridges must be prepared and certified by a licensed Colorado Professional Engineer. The procedures for design as outlined in the Federal Highway Administration (FHWA) publication *Hydraulic Design of Safe Bridges* shall be used for the hydraulic analysis of the proposed design. HEC-RAS may be used to complete the hydraulic analysis of bridge openings provided the guidance in the publication is followed. All bridges are assumed to remain in place during all storm events and shall not be assumed to break away or otherwise be removed from any modeling scenario. Design flow rates shall be those specified by FEMA or as calculated in accordance with these Standards.

6.8.2.3 Inlet and Outlet Configuration

Where bridge abutments and foundations are located below the 100-year water surface elevation, concrete wingwalls shall be tied to the existing side slopes to prevent erosion behind the abutments and to provide slope stabilization from the top of the embankment to the toe of slope. Riprap protection on the inlet and outlet transition slopes shall be provided to prevent erosion caused by eddy currents.

6.8.2.4 Scour Analysis and Countermeasures

Velocity limitations through the bridge opening are intended to limit potential scour. Regardless of the results of the scour analysis, a maximum 100-year average channel velocity of 16 feet per second shall be allowed through a bridge opening. Whenever a new or replacement bridge is designed, it is critical that scour depths at piers and abutments be estimated. The scour estimate must consider subsurface data and a hydraulic analysis of the proposed design.

The FHWA has published a set of Hydraulic Engineering Circulars (HEC) to provide guidance for bridge scour and stream stability analysis. The set includes HEC-18, Evaluating Scour at Bridges, HEC-20, Stream Stability at Highway Structures, and HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance. Latest editions of each shall be used in concert with each other to evaluate stream stability, potential scour, and appropriate scour countermeasures. HEC-RAS may be used to provide the raw data required for the HEC-18 equations. HEC-RAS may also be used to evaluate scour, but the user must be experienced in the nuances HEC-RAS presents in evaluating scour and the potential errors that can occur. Using HEC-RAS default values will cause inaccurate results.

The potential for local scour (pier and abutment) and general scour (contraction, stream degradation, and pressure) should be evaluated using HEC-18 to determine the extent of the various types of scour as applicable to each site. HEC-20 should be consulted to determine the general stability of the stream and whether lateral channel movement should be anticipated. If there is potential for scour during the scour design storm shown in Table 6.18, countermeasures shall be designed in accordance with HEC-

23. In all cases, the length of bridge piles shall be such that the design structural load may be safely supported entirely below the probable scour depth.

Table 6.18. Bridge Scour Design Standards

Roadway Classification	Design Storm for Abutment, Pier Cap, and Retaining Wall Design	Design Storm for Foundation Design
Arterial	500-year	500-year
Major Collector	100-year	500-year
Minor Collector	100-year	500-year
Local	50-year	500-year

6.8.2.5 Structural Design

The type of bridge foundation and foundation elevations should be determined by the bridge structural design engineer. During the design of the bridge foundations, the design engineer shall consider the design loading, the findings of the geotechnical investigation, scour depth as calculated using the procedures in HEC-18, anticipated frost depth, pressure flow during the 100-year event, and any other factors the engineer considers appropriate in his or her professional judgement. If scour is anticipated, the engineer can either design scour countermeasures using the procedures in HEC-23 for the applicable design storm or locate the bridge foundations below the anticipated depth of scour by a distance that provides a sufficient factor of safety in his or her professional judgement. Scour countermeasures will be required if anticipated scour depth is more than 5 feet.

Structural, scour, and foundation design calculations must be accompanied by a certification statement that is signed and sealed by a professional engineer licensed in the State of Colorado and submitted to the Town for review. The certification statement shall read as follows.

> I hereby affirm that the design calculations and plans for the bridge at [insert address] were prepared by me, or under my direct supervision, for the owners thereof, in accordance with the requirements of the International Building Code, the Breckenridge Town Code, the Breckenridge Town Standards, any approved variances and exceptions thereto, and my professional engineering judgment. I understand that the Town of Breckenridge does not and will not assume liability for facilities, structures, or improvements designed by others.
>
> Registered Professional Engineer   [Affix Seal]
>
> State of Colorado No. __________

All assumptions made by the bridge design engineer shall be provided in the calculations. Furthermore, the design of bridges may be subject to review by a third party at the Town's discretion. When located within a FEMA floodplain, all bridges are subject to requirements of the National Flood Insurance Program (NFIP) and local floodplain management regulations.

The Town recognizes that in certain limited instances, it may be exceptionally difficult to conform to these Standards. In these instances, the applicant may submit a variance request in accordance with the requirements in Chapter 1. The request must be signed and sealed by a professional engineer licensed in the State of Colorado.

6.8.3 Culvert Hydraulics

Presented in this section are the general procedures that shall be used for hydraulic design and analysis of culverts. The user is assumed to possess a basic working knowledge of culvert hydraulics and is encouraged to review the technical literature on the subject that is included in Hydraulic Design Series 5 (HDS-5), *Hydraulic Design of Highway Culverts*, published by the FHWA). The two primary types of culvert flow are inlet control and outlet control. Under inlet control, the cross-sectional area of the barrel, inlet geometry, and headwater are the factors that affect capacity. Outlet control involves the additional consideration of tailwater and the slope, roughness, and length of the culvert barrel. The Culvert Design Form, included as an attachment to this chapter, is a template for culvert hydraulic analysis that can be used with the information and equations below. All culvert designs shall include an analysis to determine whether inlet or outlet control conditions govern for both minor and major storm runoff.

6.8.3.1 Inlet Control Calculation

Under inlet control conditions, the slope of the culvert is steep enough that the culvert does not flow full. The control section of a culvert operating under inlet control is located just inside the entrance. Inlets may be either unsubmerged or submerged. In an unsubmerged condition, the headwater is high enough to submerge the top of the culvert and the culvert slope is supercritical. In a submerged condition, the headwater submerges the top of the culvert, but the pipe does not flow full. In this situation, the culvert inlet acts like an orifice.

In the submerged inlet condition, the equation governing the culvert capacity is the orifice flow equation. However, because of the uncertainty in estimating the orifice coefficient for a submerged culvert inlet, it is recommended that the inlet control nomographs published in HDS-5 be used to determine headwater for submerged inlets operating under inlet control. Nomographs may be found online in the second edition of HDS-5, publication number FHWA-NHI-01-020. Later editions do not have as many nomographs. Table 6.19 provides the appropriate inlet control nomograph to use for various types of culverts and end treatments. The FHWA has not published inlet control nomographs for plastic pipe. In their absence, the nomographs for concrete may be used for round HDPE with a smooth interior wall.

Table 6.19. Inlet Control Nomograph Selection

Material	Cross Section	End Treatment	Chart
Concrete/HDPE	Circular	None (Projecting), Headwall	1B
Concrete/HDPE	Circular	Flared end section	55B
Concrete	Horizontal Elliptical	Headwall or Projecting (use scale 1 for end section)	29B
Concrete	Rectangular	Wingwalls, angle and headwall bevel varies	8B-13B

6.8.3.2 Outlet Control Calculation

Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. Either subcritical or pressure flow exists in the culvert barrel under these conditions. Outlet control will govern if the headwater is deep enough, the culvert slope is sufficiently flat, or the culvert is sufficiently long.

Outlet control generally exists under two conditions. The first, and less common, occurs when headwater is not high enough to submerge the top of the culvert and the culvert slope is subcritical. The more common outlet control condition exists when the culvert is flowing full. A culvert with a submerged inlet and an unsubmerged outlet may also operate under outlet control, especially if it has a long barrel length or a flat enough slope. Culverts under outlet control may flow full or partly full, depending on various combinations of hydraulic factors.

Culvert capacity under outlet control is calculated using Bernoulli's equation. An energy balance is determined between the headwater at the culvert inlet and at the culvert outlet and includes inlet losses, friction losses, and velocity head. The general equation is expressed as:

$$H = h_e + h_f + h_v \qquad (6.13)$$

Where:

H = total energy head (headwater elevation minus tailwater elevation) (ft)

h_e = entrance head loss (ft), $K_eV^2/2g$

h_f = friction losses (ft)

h_v = velocity head (ft), $V^2/2g$

K_e = entrance loss coefficient per Table 6.20

Friction loss is the energy required to overcome the culvert barrel roughness and is calculated by the following equation.

$$h_f = (29n^2L/R^{1.33})(V^2/2g) \qquad (6.14)$$

Where:

n = Manning's coefficient per Table 6.15

L = length of culvert (ft)

R = hydraulic radius (ft)

V = velocity of flow (ft/s)

g = gravitational acceleration, 32.2 ft/s^2

Table 6.20. Culvert Entrance Loss Coefficients, K_e, for Outlet Control Calculations

Structure and Entrance Type	Ke	Structure and Entrance Type	Ke
RCP		RCB	
Headwall, socket end of pipe	0.2	Wingwalls at 30° to 75° to barrel	
Headwall, square edge	0.5	Square edge at crown	0.4
Projecting from fill, socket end	0.2	Rounded or beveled top edge	0.2
Projecting from fill, square cut end	0.5	Wingwalls at 10° to 25° to barrel	
Mitered to conform to fill slope	0.7	Square edge at crown	0.5
Side- or slope-tapered inlet	0.2	Wingwalls parallel (side extensions)	
Beveled edges, 33.7° or 45° bevels	0.2	Square edge at crown	0.7
Rounded (radius = D/12)	0.2	Side- or slope-tapered inlet	0.2
End section that conforms to fill slope(1)	0.5	No wingwalls	
HDPE (2)		Square edge on 3 sides	0.5
Projecting from fill	0.9	Rounded or beveled on 3 sides	0.2

(1) End sections that conform to fill slope are the sections commonly available from manufacturers. From limited hydraulic tests, they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, which incorporate a closed taper in their design, have a superior hydraulic performance. These latter sections can be designed by using the information given for the beveled inlet.

(2) Conditions not listed specifically for HDPE with a smooth interior wall have the same coefficient as RCP. The "projecting from fill" value given for HDPE is an approximation based on published values for CMP.

Combining the equations yields the following equation, which can be used to calculate culvert capacity directly only when the tailwater is at or above the crown of the culvert outlet.

$$H = (K_e + 1 + 29n^2L/R^{1.33})(V^2/2g) \tag{6.15}$$

When the tailwater is below the culvert outlet crown, the tailwater depth used for calculations shall be the larger of the tailwater anticipated in the downstream channel at the culvert outlet and the average of the critical depth in the culvert and the culvert diameter, $(D+d_c)/2$. The FHWA has determined the average of the critical depth and the culvert diameter to be an adequate approximation for tailwater depth for culverts that flow partially full. Critical depth calculation is a direct process for a box culvert but an iterative one for a circular pipe that is easily accomplished with a spreadsheet. Critical depth occurs when the Froude number is equal to 1.0. The flow area and top width will be those that occur at critical depth in the pipe. Many online tutorials are available.

$$Fr = \frac{v}{\sqrt{gD_h}} \tag{6.16}$$

Where:

Fr = Froude number (dimensionless)

v = velocity (ft/s)

D_h = hydraulic depth (ft), A/T

A = flow area (ft^2)

T = top width of flow area (ft)

g = gravitational acceleration, 32.2 ft/s^2

In addition to equation 6.15, outlet control nomographs published by the FHWA in HDS-5 can also be used to calculate the required headwater under outlet control conditions where the outlet is submerged. Outlet control nomographs can be found online in the second edition of HDS-5, publication number FHWA-NHI-01-020. Later editions do not have as many nomographs. Table 6.21 provides the appropriate outlet control nomograph to use for various types of culverts. The FHWA has not published outlet control nomographs for plastic pipe. In their absence, the nomographs for concrete may be used for round HDPE with a smooth interior wall. End treatments do not affect outlet control.

Table 6.21. Outlet Control Nomograph Selection

Material	Cross Section	Chart
Concrete/HDPE	Circular	5B
Concrete	Rectangular	15B
Concrete	Horizontal Elliptical	33B

Culvert capacity shall be computed using the Culvert Design Form, included as an attachment to this chapter. Two example calculations for culvert sizing are at the end of this section. The first is for a roadway crossing culvert and the second is for a driveway culvert in a roadside ditch. HDS-5 offers extensive guidance on the design of culverts that are under roadways and that may be used in conjunction with the requirements of these Standards.

6.8.3.3 Evaluation of Results

If the culvert selected will not fit the site, return to the design process, and select another culvert. Repeat the design process until an acceptable culvert configuration is determined. Compare the headwater elevations calculated for inlet and outlet control. The higher of the two is the controlling headwater elevation. The culvert can be expected to operate with that higher headwater for at least part of the time.

If outlet control governs and the headwater depth is less than 1.2D, it is possible that the barrel flows partly full through its entire length. In this case, caution should be used in applying the approximate method of setting the downstream elevation based on the greater of tailwater or (d_c + D)/2. If an accurate headwater is necessary, backwater calculations should be used to check the result from the approximate method. If the headwater depth falls below 0.75D, backwater calculations are required.

6.8.3.4 Outlet Velocity Calculation

The outlet velocity may be calculated as follows:

1. If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity.
2. If the controlling headwater is based on outlet control, determine the area of flow at the outlet based on the barrel geometry and the following:
   a. Critical depth if the tailwater is below critical depth.
   b. Tailwater depth if the tailwater is between critical depth and the top of the barrel.
   c. Height of the barrel if the tailwater is above the top of the barrel.

6.8.3.5 Computer Applications

While it is possible to use the procedures and nomographs for analyzing culvert hydraulics, it is more common to design culverts using computer applications. Among the applications approved for use by the Town is the FHWA’s HY-8 Culvert Analysis Program and the Mile High Flood District’s MHFD-Culvert spreadsheet, both of which may be used to calculate roadway overtopping, inlet and exit velocity, and hydraulic grade line.

6.8.4 Design Examples

Two design examples are included in this section. The first example is the analysis of an existing roadway cross culvert using the Culvert Design Form. Calculations from this design example are shown in the Culvert Design Form Example, included as an attachment to this chapter. The second example is the design of a private driveway culvert in a roadside ditch.

6.8.4.1 Crossing Culvert Analysis Example

The procedure to evaluate culverts is based on the procedures presented in HDS-5. The methodology consists of evaluating the culvert headwater requirements for both inlet and outlet control. The type of flow control that results in a larger required headwater is the governing flow condition.

An example calculation for rating an existing culvert is presented in the Culvert Design Form Example, included as an attachment to this chapter. The culvert is a 48-inch RCP. The length is 150 feet. The upstream invert elevation is 8540.0, and the downstream invert elevation is 8535.5. The slope is 0.030. The low point of the embankment over the culvert has an elevation of 8551.90. The n value is 0.015, in accordance with Table 6.15 for older concrete pipes. The culvert has flared end sections on each end. All depths are in feet unless noted otherwise.

The tailwater rating values are provided for this example and shown in Column 5. If the tailwater condition is unknown, it must be computed using the normal depth (subcritical or critical only) of a trapezoidal channel approximating the existing drainageway. A HEC-RAS model of the site could also be used to determine the tailwater rating curve.

The entrance loss coefficient, K_e, is determined from Table 6.20 as 0.5 for an end section that conforms to fill slope, which is the category used to represent a common flared end section. The full flow and the velocity are calculated from these values for comparison. The rating then proceeds in the following sequence:

1. The culvert design process begins with selecting a range of discharges or headwater depths and then using an inlet control nomograph to determine the corresponding flow values. This example begins with a range of headwater depths that are entered in Column 3. Headwater to pipe diameter ratios (H_W/D) are calculated and entered in Column 2. If the culvert is not circular, the culvert height is used for the calculation. Note that for design of new culverts, the Town has restrictions on the headwater-to-depth ratio in Table 6.17.
2. For each H_W/D ratio, inlet capacity is read from the appropriate inlet control nomograph (Chart 55B for this example because of the flared end sections) and entered in Column 1. Scale (1) for concrete should be used on Chart 55B to determine discharges, which then completes the inlet control rating.
3. For outlet control, the Q values that have been entered in Column 1 are used to determine the head values (H) in Column 4 from the appropriate outlet control nomograph, Chart 5B in this case. Note that flared end sections do not affect outlet control calculations. The first line drawn in Chart 5B is between the pipe diameter and the pipe length. The second line connects the Q value and passes through the turning line where the first line crossed it to determine H.
4. The known tailwater depths (T_W) for normal flow in the downstream channel are then entered into Column 5 for each Q value in Column 1. The depths have been provided in this example but must be calculated if they are not available using the normal depth of a trapezoidal channel approximating the existing drainageway downstream of the culvert. If the tailwater depth is less than the diameter of the culvert, Columns 6 and 7 must be calculated per Step 5, and the larger of the tailwater depth and the value of Column 7 shall be used as h_o. If the tailwater depth is greater than the diameter of the culvert, the tailwater values in Column 5 are entered into Column 8 as the values for h_o, and Step 6 should begin (Step 5 being skipped).
5. Approximate tailwater depths are calculated when tailwater depths in the downstream channel are less than the diameter of the culvert. The critical depth, d_c, for each Q value in Column 1 is calculated and entered in Column 6. For a circular pipe, the Froude number calculation is completed iteratively using a spreadsheet until the Froude number is 1. Alternately, Chart 4B from HDS-5 can be used to determine d_c for the pipe size and Q value. The average of the critical depth and the culvert diameter is calculated and entered in Column 7 as the approximate h_o value.
6. The headwater values (H_W) in Column 9 are calculated according to Equation 6.17:

$$H_W = H + h_o - LS_o \tag{6.17}$$

   where H is from Column 4 and h_o is either the value from Column 8 where $T_W \geq D$ or the larger value of Column 5 and Column 7 where $T_W < D$. L is the length of the culvert barrel and S_o is its slope.

7. The final step is to compare the inlet and outlet control headwater requirements (Columns 3 and 9) and record the higher of the two values in Column 10. The type of flow control is recorded in Column 11. The upstream water surface elevation is then calculated by adding the controlling headwater (Column 10) to the upstream invert elevation. Add this value to Column 12. The culvert rating curve can then be plotted from the values in Columns 12 and 1.

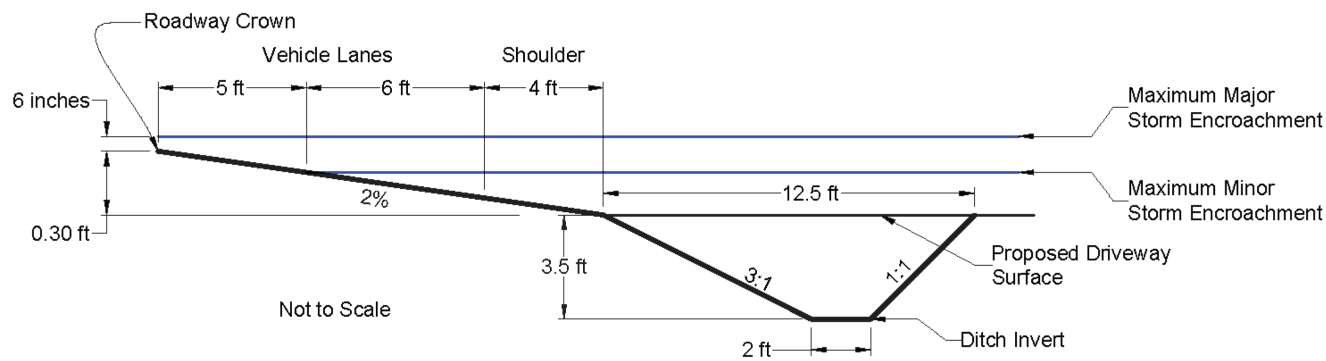
Outlet velocity for designing downstream protection can be computed using V = Q/A. For full flow conditions, the culvert area is the full cross sectional area of the culvert. For partially full conditions, the culvert area is the area calculated at a depth of h_o. Channel protection shall be in accordance with guidance in the USDCM. Velocity values are not shown in the Culvert Design Form but should be calculated for the 100-year event.

To size a culvert crossing, the same form can be used, with some variation in the basic data. First, a design Q is selected, and the maximum allowable headwater is determined. An inlet type is selected, and the invert elevations and culvert slope are estimated based on site constraints. A culvert type and size is then selected and rated for both inlet and outlet control. If the controlling headwater exceeds the maximum allowable headwater, the design data must be modified, and the procedure repeated, until the desired results are achieved.

6.8.4.2 Private Driveway Culvert Design Example

A driveway is planned to provide access to a new residence from a minor collector roadway with an existing roadside ditch. The minor collector has a transverse slope of 2%. The roadside ditch is trapezoidal with a 3:1 side slope down from the road, a 1:1 backslope, a 2-foot bottom width, and a 3.5-foot depth as shown in Figure 6.4.

Figure 6.4. Private Driveway Culvert Example Ditch Geometry



The driveway is assumed to have no slope over the ditch for calculation purposes. The calculated peak flow is 28 cfs for the major storm and 12 cfs for the minor storm. Floodwater encroachment onto the road must not exceed the limitations set forth in Table 6.12 of these Standards. Inlet control is assumed for this example; however, actual projects should use the Culvert Design Form to determine if culverts are under inlet or outlet control.

1. During the major storm, water on a minor collector may be 6 inches deep at the crown of the road. The depth of the water from the allowable water surface to the ditch invert is 4.3 feet. This value is assumed as the headwater depth.

2. For the major storm, calculate the discharge through an 18-inch HDPE (D = 1.5 feet) with projecting ends and a headwater depth of 4.3 feet using Chart 1B:

$$\mathrm{H_W/D = 4.3\ ft/1.5\ ft = 2.9}$$

   A H_W/D ratio of 2.9 on Scale 3 of Chart 1B for projecting culvert ends gives a discharge of 18.0 cfs for an 18-inch HDPE.

3. Calculate weir flow over the road and driveway during the major storm. Flow over the road and over the driveway are calculated independently. Because the road grade is sloped across the cross section, the average depth of flow over the road is used. Flow outside the top of the ditch side slope is assumed to be negligible for this example, but it may be considered if the designer feels it is appropriate. Assume a weir coefficient of 2.8.

$$\mathrm{Q_{weir} = Q_{road} + Q_{drive}}$$

$$Q_{weir} = CLH^{3/2} + CLH^{3/2}$$

$$Q_{weir} = (2.8)(15)((0.85 + 0.50)/2)^{3/2} + (2.8)(12.5)(0.85)^{3/2}$$

$$Q_{weir} = 23.3\ \text{cfs} + 27.4\ \text{cfs} = 50.7\ \text{cfs}$$

Total flow over the road and driveway is 50.7 cfs.

4. The combined flow through the 18-inch HDPE and over the road/driveway is 68.7 cfs, which is significantly more than the major storm design flow of 28 cfs. Encroachment onto the minor collector will not exceed allowable and the chosen culvert is acceptable for the major storm. If the combined flow would have been less than the major storm flow, a larger culvert would be required, and Steps 2–3 would be repeated using a 24-inch HDPE. In this case, a shallower ditch or a smaller pipe may be considered.

   Verify that the design meets the minor storm criteria. During the minor storm, flow may spread to within 5 feet of the crown of a minor collector to allow a single 10-foot lane to remain clear during the minor storm. Assuming encroachment extends only to within 5 feet of the roadway crown yields a headwater depth of 3.7 feet and a H_W/D ratio of 2.5.

5. Scale 3 of Chart 1B indicates a discharge of 16.0 cfs for an 18-inch HDPE with a H_W/D ratio of 2.5, which is greater than the 12 cfs peak flow during the minor storm. The design will meet the minor storm criteria.
6. Verify that the culvert has a minimum 8 inches of cover. The driveway surface is 3.5 feet or 42 inches above the ditch invert. The 18-inch HDPE has more than the minimum 8 inches of cover and meets all criteria for this location.

6.9 DETENTION

The imperviousness of any site typically increases when it is developed. Roof areas, sidewalks, and parking all contribute to site imperviousness. Rain falling on this added impervious area cannot infiltrate into the ground after development. The rainfall discharges from the site instead, impacting adjacent properties, storm drain capacity, and even the capacity of larger drainageways when development is considered in the aggregate. The purpose of detention facilities is to store the additional runoff volume associated with development and discharge it from the developed site at the rate experienced during pre-development conditions. Detention helps to minimize peak flows in urbanized areas. Detention can include individual site options such as small landscaped basins and larger regional options that serve multiple sites. Detention facilities can also be dual purpose when they are designed to meet water quality requirements as well as hydraulic detention requirements. This section presents the Town’s criteria and guidance for designing detention ponds.

6.9.1 Applicability

Detention is required for all new development and redevelopment. Subdivided developments may use a single detention facility provided it captures runoff from the entire development. All detention facilities in the Town are subject to oversight by the Town. A maximum of 5% of the total development site may discharge directly from the site, without hydraulic detention, provided the peak site discharge does not exceed the peak historic discharge from the site.

Exemptions from the detention requirement may be granted if the project has either of the following characteristics, provided the additional undetained runoff will cause no adverse impacts to any downstream properties.

1. Impervious area is increased by no more than 0.10 acres or 4360 square feet.

2. Other situations as may be determined by the Town to be in the best interest of the Town.

Exemptions may also be provided for single family homes in subdivisions without regional detention provided low-impact development principles are included in the design to minimize the increase in runoff and not aggravate flooding or erosion problems. Low impact development principles include using pervious pavers or pavement for driveways and walkways, minimizing directly connected impervious areas, and routing drainage from impervious areas such as roofs and driveways through infiltrating swales or across vegetated pervious areas prior to discharging it from the site. Detailed guidance on low impact development can be found in Volume 3 of the USDCM.

Supporting analysis and certification by a professional engineer that exclusion from the detention requirement will not cause any adverse downstream impacts must be submitted to the Town Engineer for consideration if an exemption is requested. An exemption to provide detention issued by the Town does not eliminate potential liability to others.

Although development increasing impervious area by less than 0.10 acres is exempted from detention requirements, any new development or redevelopment is required to analyze and correct any inadequate drainage, including insufficient drainage away from buildings, runoff adversely affecting downstream properties, inadequately sized drywells, drainage exceeding existing storm drains, pipes, or other conveyance, or other drainage concerns identified by the Town Engineer.

6.9.2 Detention Facility Design

All detention facilities shall be designed as full-spectrum detention facilities. The Mile High Flood District has developed detailed design guidance for detention basins. The USDCM provides discussion on the applicability of detention; an explanation for why full-spectrum detention is the preferred approach; and calculations for sizing a detention facility and designing the outlet structure.

The most recent versions of the USDCM and the design tool MHFD-Detention.xlsm (formerly UD-Detention.xlsm) may be used for sizing and designing all detention facilities in the Town. Detention facilities in the Town may be extended detention basins, sand filters, and rain gardens. Constructed wetlands may also be used provided water rights are accommodated. The USDCM also includes weir and orifice equations for the design of detention basin outlets that may also be used for other applications within these Standards as needed. Guidance on the use of MHFD-Detention within the Town is included in this section.

6.9.2.1 Historic Flow Rates

The policy of the Town is to require detention storage of stormwater runoff to limit peak discharges from new development and redevelopment sites to historic rates. Detention facilities shall be designed to release stored runoff volumes at or below the calculated historic peak rate for the 2-, 5-, 10-, 25-, 50-, and 100-year storms.

For new development, historic peak runoff rates per acre shall be calculated in accordance with the procedures in these Standards using a site imperviousness of 2% to represent historic conditions in accordance with Table 6.3. For redeveloped areas, historic rates shall be those calculated for the condition immediately prior to redevelopment in accordance with these Standards unless other criteria are specified by the Town Engineer in writing. Calculated historic rates will vary based on the methodology chosen to calculate peak flow, whether it is the Rational Method, the SCS Method, or a computer model. Calculated historic rates for each storm event shall be presented as part of the drainage analysis in the Drainage Report.

Post-development peak runoff from a site may not be greater than pre-development runoff from a site for any storm event. Total site runoff is typically a combination of detention basin release and direct runoff from areas not draining to a detention facility, both of which must be considered. A maximum of

5% of the total site area may contribute direct runoff. Note that the allowable peak discharge from a detention facility will be less than the allowable peak runoff from the whole site unless the entire site drains to the detention facility.

For redevelopment sites, any existing detention facilities shall be factored into the runoff calculations and accounted for with the revised runoff characteristics to preserve the pre-development runoff rates as identified in any previous drainage studies. If a HEC model of the watershed exists, it can be used to generate historic runoff rates by changing the imperviousness of the watershed to historic conditions.

6.9.2.2 MHFD-Detention

The last three tabs in the MHFD-Detention spreadsheet contain helpful information on how to use the MHFD-Detention spreadsheet as a design tool. Users are highly encouraged to thoroughly review the information in these tabs, including the video provided before beginning design. For designing detention facilities within the Town, the following inputs shall be used in lieu of default values.

1. While most detention basins in the Town will include water quality treatment, the first drop down menu on the Basin worksheet below Watershed Information should be set to Flood Control Only if no water quality treatment is to be provided. If water quality treatment is to be provided within the detention facility, the user should select the type of facility being designed.
2. Location for 1-hr Rainfall Depths on the Basin worksheet shall be set to User input and the P_1 values from Table 6.1 of these Standards shall be entered into the appropriate, blue-shaded cells on the Basin worksheet.
3. In the three cells used to Define Zones and Basin Geometry in the Basin worksheet, the Zone 1 Volume shall be the WQCV if water quality treatment will be provided. If water quality will not be provided, the Zone 1 Volume shall be the 2-year event. In all cases, the Zone 2 Volume shall be set to EURV – Zone 1, and the Zone 3 Volume shall be set to 100-year minus Zones 1 & 2.
4. The remainder of the user inputs on the basin worksheet tab shall follow the recommended guidelines indicated as notes in each cell. Assuming the design of the facility was completed in accordance with guidelines in the USDCM, the design values for these inputs will be within the recommended ranges of values for each variable. Note that the Total Available Detention Depth should be set at the maximum allowable 100-year water surface, not at the downstream embankment crest, to allow for design freeboard.
5. On the Outlet Structure worksheet, an Outlet Type must be selected from the drop down menu at the top of the worksheet for each Zone. When water quality treatment is provided, the lowest outlet is typically an orifice plate, which is a plate with multiple smaller orifices cut into it. When water quality treatment is not provided, the lowest outlet may be a vertical orifice, which is also a plate, but with a single, larger hole cut into it. The options available in the drop down menu for Outlet Type will automatically only be those applicable to the design completed up to that point. The user should only enter additional design values into the User Input rows in the remainder of this worksheet that correspond to the Outlet Types selected at the top of the worksheet. For example, if there is no orifice plate, there is no need for the user to enter data describing an orifice plate. But each Outlet Type selected must have its design data entered for the worksheet to yield correct results.
6. Several rows of hydraulic results are presented at the bottom of the Outlet Structure worksheet. While several of the values are of interest, there are only a handful that are critical for detention facilities in the Town. If these criteria are not met, the design must be adjusted until they are.
    a. Peak Outflow for each design storm may not exceed the historic rates calculated for the site.

b. Time to Drain 97% of Inflow Volume must not exceed 72 hours after the end of a 5-year storm.

c. Time to Drain 99% of Inflow Volume must not exceed 120 hours after the end of storms greater than the 5-year storm.

d. Maximum Ponding Depth must be at or below the emergency spillway and at least 12 inches below the surrounding embankment.

6.9.4 Snow Storage in Detention Ponds

If it is intended that a detention pond will serve as a snow storage area, the calculation of the pond's required volume must be increased by 50% of the full design snow storage volume and the pond must provide permanent water quality treatment in the form of the water quality outlet.

6.9.5 Maintenance

The performance of detention facilities is extremely sensitive to a lack of maintenance, and all detention facilities must be designed to facilitate maintenance. Section 6.10 of these Standards includes design guidance related to maintenance. It is the responsibility of all private detention facility owners to regularly maintain their detention facilities, except as modified by a recorded agreement. Outlets, especially, must be regularly maintained to ensure the basins do not detain water longer than allowed by Colorado water law.

The Town may require an enforceable Ownership and Maintenance (O&M) Agreement to be in place for detention and water quality facilities before issuing any applicable local permits. The O&M Agreement must include the party responsible for maintaining the facility, inspection frequency, and proposed maintenance activities in an Operations and Maintenance Plan (O&M Plan). Should the responsible party fail to adequately maintain the detention facility, the Town shall have the right to enter the property for the purpose of maintenance. All such maintenance costs and associated legal fees will be assessed to the property owner. Guidance on maintenance frequency for various types of facilities is included in Section 6.10. Guidance on the development of the O&M Plan is included in Chapter 2.

6.9.6 Detention and Water Rights

Senate Bill 15-212 became effective on August 5, 2015, as Colorado Revised Statute (CRS) §37-92-602 (8), *Concerning a Determination that Water Detention Facilities Designed to Mitigate the Adverse Effects of Storm Water Runoff Do Not Materially Injure Water Rights*. This statute provides legal protection for any stormwater detention and infiltration facility in Colorado, provided the facility does not materially injure water rights and meets the following criteria.

1. It is owned or operated by a governmental entity or is subject to oversight by a governmental entity.
2. It continuously releases or infiltrates at least 97% of all the runoff from a rainfall event that is less than or equal to a 5-year storm within 72 hours after the end of the event.
3. It continuously releases or infiltrates as quickly as practicable, but in all cases releases or infiltrates at least 99% of the runoff within 120 hours after the end of events greater than a 5-year storm.
4. It operates passively and does not subject the stormwater runoff to any active treatment process such as coagulation, flocculation, and disinfection.

There are reporting requirements for any owner or operator of any detention facility constructed after August 5, 2015 that seeks protection under the new statute. A data sheet and online map-based compliance portal website has been developed that will allow owners and operators in the Town to upload the required notification information. The notification requirement applies only to facilities constructed after August 5, 2015. Facilities in existence before August 5, 2015, are defined in the statute as materially noninjurious to water rights and do not require notification.

The compliance portal can be found online (*https://maperture.digitaldataservices.com/gvh/?viewer=cswdif*). A document containing frequently asked questions, links to a video tutorial, and the link to the compliance portal can be found online (*http://www.crgov.com/DocumentCenter/View/12225*) or via an internet search for "Colorado water rights compliance portal." The owner or operator must report new detention via the portal, and the county must approve the portal entry once it is complete. The owner or operator shall inform the county once the portal documentation is ready for approval.

6.9.7 Jurisdictional Dams

Detention facilities with a downstream embankment height in excess of 10 feet, 100-year water surface area in excess of 20 acres, or 100-year volume in excess of 100 acre feet are considered jurisdictional dams and require approval by the State Engineer's office.

6.10 PERMANENT WATER QUALITY

The naming convention for facilities that treat stormwater quality after construction has varied over time. These facilities have been called both best management practices (BMPs) and control measures by various entities. The Town's term for these facilities is permanent stormwater treatment facilities. The term treatment facilities may also be used. This term distinguishes these facilities from those used during construction.

The goal of the requirements in this section is to keep the Town's streams and drainageways healthy. This section presents runoff reduction and site planning principles, the applicability of the requirements for permanent water quality treatment, design requirements for permanent stormwater treatment facilities, and design requirements associated with maintenance.

6.10.1 Planning Principles

Increases in impervious area that typically come with development can negatively impact flow volumes, temperature, and stormwater quality. To minimize these impacts, site planning should consider how the site will be used as well as how stormwater runoff will be conveyed and treated. While no specific design criteria is associated with many of these guidelines, general site planning goals for every development include:

1. Considering stormwater quality needs early in the development process to better integrate stormwater treatment facilities into the site.
2. Minimizing impacts to the natural environment including water quality, air quality, wildlife habitat, vegetation, and natural landforms and protecting areas with high ecological value such as those with mature trees, stream corridors, wetlands, and soils with high infiltration rates.
3. Developing creative site layouts to reduce the extent of paved and other impervious areas.
4. Reducing runoff from the site and maximizing infiltration by minimizing directly connected or continuous impervious areas and slowing runoff through pervious and/or vegetated areas. Developing the layout of a site to reduce runoff will also reduce the required size of WQCV treatment facilities.
5. Centralizing water quality treatment for larger developments and integrating them with site operations to minimize land use, achieve greater economy of scale, and reduce the number of treatment facilities requiring maintenance.

6. Developing operational procedures to minimize the risk of spills and designing the site layout to prevent any spills from leaving sites with operations that include washing, fueling, manufacturing, materials storage, and vehicle maintenance, among others.
7. Using pervious drainage conveyances where appropriate, routing downspouts across pervious areas, and incorporating vegetated areas into locations that generate and convey runoff like parking lots and driveways. Grass buffers, grass swales, and bioretention can all be used.
8. Discharging site runoff across a vegetated area prior to discharge from the site. No impervious area should discharge directly into wetlands, the Blue River, or one of its tributaries.
9. Maintaining natural drainage patterns and implement sheet flow.
10. Selecting permanent stormwater treatment facilities based on expected pollutant type.
11. Providing areas for snow storage so that snow melt will not be a nuisance and will drain to a permanent water quality facility.

Chapter 1 of Volume 3 of the USDCM includes a section on ways to minimize the adverse impacts of development on water quality, including ideas to minimize site runoff. This document should be reviewed to gain a better understanding of planning principles as they relate to water quality. A discussion on how design of the site minimizes site runoff and the impacts to water quality shall be included in the permanent water quality portion of the Drainage Report.

6.10.2 Applicability

Permanent stormwater treatment facilities are required to be designed and installed for all new development and redevelopment sites that meet one of the following criteria:

1. Disturb at least one acre of land.
2. Disturb less than one acre but are part of a larger common development plan. In these cases, the individual development need not provide a permanent stormwater treatment facility, provided the larger common development provides a centralized facility that will provide treatment for all the sites within it.
3. Increase impervious area by more than 0.10 acres or 4360 square feet.
4. Include any of the following land uses: auto service station, auto repair, auto body work/paint, auto wash/polish, equipment repair, lumberyard, nursery, asphalt plant, concrete batch plant, industry, manufacturing, crushing gravel/rock, milling, mining, sawmill, silviculture, junkyard, sludge, sanitary landfill, truck terminals, impound yard, motor vehicles parking/storage, or storage of pesticides, herbicides, fertilizer, or other potentially hazardous materials.

New development is classified as any land disturbing activity or construction of any building or structure. Redevelopment is any creation, addition, or removal and replacement of any impervious area, or any building construction or land disturbing activity, on a site that is already substantially developed.

For sites that require permanent stormwater quality treatment, treatment must be provided for at least 80% of the added impervious area on the site. At least 80% of the disturbed site pervious area must also be treated unless justification is provided showing that runoff from these areas will not negatively impact water quality within the Town. An area larger than the minimum required may necessarily drain to the treatment facility, and this runoff must be accounted for in the design of the facility. Calculations documenting this has been achieved shall be included in the permanent water quality portion of the Drainage Report.

Development that is exempted from permanent water quality requirements may still be required by the Town to analyze and correct any inadequate drainage, including insufficient drainage away from buildings, runoff adversely affecting downstream properties, inadequately sized drywells, drainage exceeding existing storm drains, pipes, or other conveyance, or other drainage concerns identified by the Town Engineer.

6.10.3 Treatment Facility Types

There are three basic methods by which stormwater quality can treated in the Town.

1. Collecting and storing the water quality capture volume (WQCV) and releasing it slowly, thereby settling out pollutants in the stormwater instead of discharging them downstream,
2. Infiltrating the WQCV into the ground within a given time, and
3. Using a proprietary vault or structure that has been developed to treat stormwater quality.

Volume 3 of the USDCM provides graphics that offer some general guidance as to the treatment facilities that are typically used for sites having specific characteristics. Details for the design of each type of treatment facility, as well as design constraints, are in the next section.

6.10.4 Design Criteria

The Town's general criteria for permanent water quality treatment is 80% removal of the 60 micron particle from the WQCV of the area being treated or from the peak runoff rate from the 2-year return period storm over the area being treated, depending on the type of treatment facility being designed. For sites that require permanent stormwater quality treatment, treatment must be provided by one of the methods in Section 6.10.3. Meeting the criteria in this section for each type of treatment mechanism is assumed to result in this criteria being met. Supporting calculations must be included in the Drainage Report.

6.10.4.1 Tributary Offsite Area

When offsite area is tributary to a permanent stormwater treatment facility there are two options. The first option is to intercept the offsite flow and route it around or through the site in a separate conveyance system. The second option, if offsite flow cannot be separated, is to size the treatment facility for the entire tributary area. This could mean a much larger design volume or a larger peak runoff rate.

6.10.4.2 Right-of-Way Restrictions

Privately owned and maintained treatment facilities must be located outside the public right-of-way and offline from public stormwater conveyance systems. Easements are required for the area occupied by all facilities located outside the public right-of-way including for access.

6.10.4.3 WQCV Storage and Release

The WQCV is a volume of water designated for treatment that has been determined to provide the most water quality benefit for the cost to construct the improvements required to treat it. Volume 3 of the USDCM provides detailed discussion on the development of the WQCV. Capturing, storing, and slowly releasing the WQCV of a site will meet the Town's requirements for permanent water quality treatment by storing it long enough that pollutants settle out. WQCV treatment facilities acceptable for use in the Town include extended detention basins, sand filter basins, rain gardens, and infiltration galleries and trenches. Other types of facilities may be considered on a case-by-case basis.

The first step in designing a WQCV facility is calculating the WQCV. Two variables are required to calculate the WQCV. The first is the total imperviousness of the area draining to the treatment facility. Recommended imperviousness values are in Section 7.5. The total imperviousness of a site can be determined by taking an area-weighted average of the different imperviousness values for the site.

Total imperviousness can also be adjusted to an effective imperviousness if certain practices are implemented as part of the site design. Effective imperviousness applicability and calculations are discussed below the calculation for the WQCV.

The second variable is the design drain time of the treatment facility. Recommendations for design drain time for different types of WQCV treatment facilities can be found in Volume 3 of the USDCM. The most common WQCV facility is an extended detention basin, for which the recommended drain time is 40 hours. WQCV drain time coefficients are in Table 6.22 below. The general equation to calculate the WQCV in Breckenridge is expressed as:

$$\mathrm{WQCV} = 0.84\mathrm{Aa}(0.91\mathrm{I}^3 - 1.19\mathrm{I}^2 + 0.78\mathrm{I})/12 \qquad (6.18)$$

Where:

WQCV = water quality capture volume (acre-feet)

a = WQCV drain time coefficient

I= imperviousness as a decimal percentage

A = area draining to the treatment facility in acres

Table 6.22. Drain Time Coefficients for WQCV Calculations

Drain Time	Coefficient, a
12 hours	0.8
24 hours	0.9
40 hours	1.0

The WQCV equation was initially developed based on rainfall data from the Denver metro area. However, the precipitation depth of the average runoff producing storm in Breckenridge is 0.36 while in Denver it is 0.43. The WQCV equation above includes a coefficient of 0.84 to adjust the equation for use in Breckenridge. A map showing the variance in the average runoff producing storm across Colorado is shown as Figure 6.5.

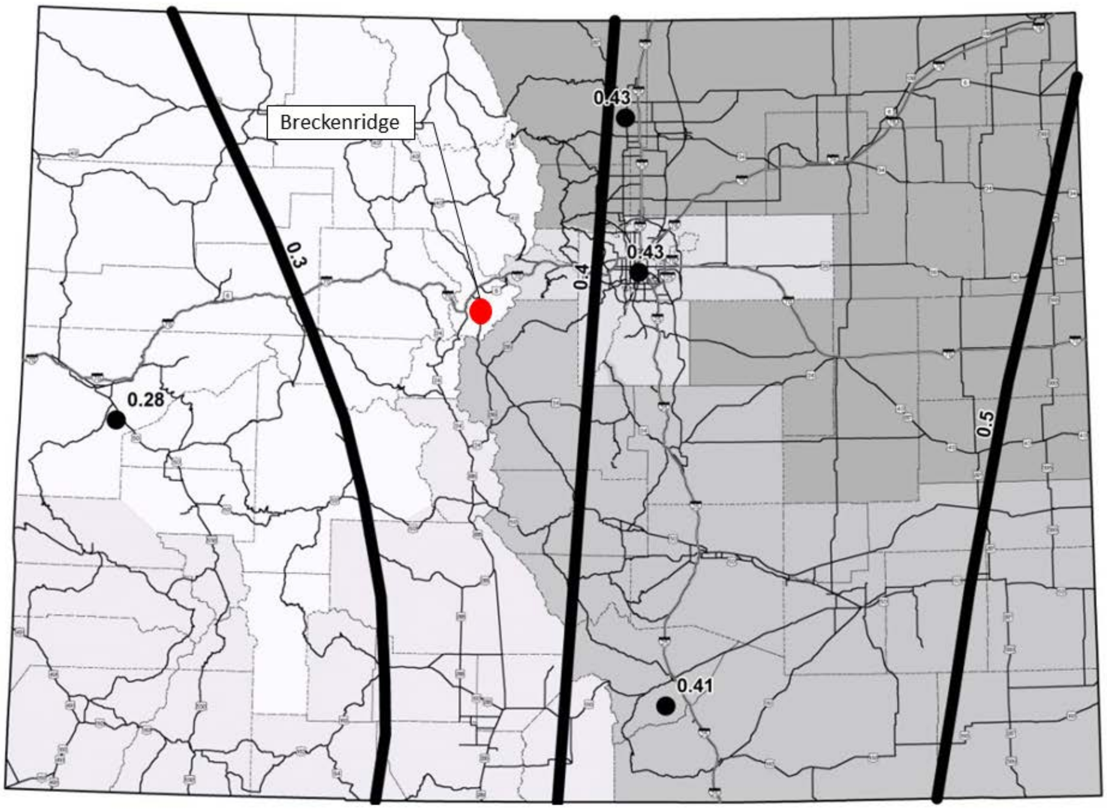


Figure 6.5. Map of the Average Runoff Producing Storm's Depth
(Modified from UDFCD, 2016)

Guidance on how to design extended detention basins, sand filter basins, and rain gardens to meet the WQCV standard are included in the Fact Sheets in Volume 3 of the USDCM. Sand filter basins are referred to as simply sand filters and rain gardens are referred to as bioretention.

6.10.4.3.1 Effective Imperviousness

The imperviousness value used in the WQCV calculation for sites that implement low impact development (LID) principles such as green infrastructure and MDCIA may be reduced to reflect the site's effective imperviousness. The effective imperviousness is dependent on the level of MDCIA implemented for high-level planning applications. Level 1 includes designing impervious surfaces to drain over a grass buffer or other pervious surface prior to reaching any stormwater conveyance system. Level 2 is an enhancement to Level 1 and includes eliminating curb and gutter or using slotted curbs; low-velocity pervious grass- or rock-lined swales instead of storm sewers, and pervious street shoulders. Guidance on calculating effective imperviousness for Level 1 and Level 2 MDCIA can be found in Volume 3 of the USDCM.

Where a detailed site plan has been developed and the square footage of directly connected impervious area, unconnected impervious area, receiving pervious area, and separate pervious area have all been defined, a more detailed effective imperviousness can be determined using the methods in Volume 3 of the USDCM.

6.10.4.3.2 Design Volume and Bypass Capacity

If it is intended that a WQCV treatment facility will serve as a snow storage area, the calculation of the required volume must be increased by 50% of the full design snow storage volume.

WQCV treatment facilities will likely receive more runoff than they were designed to treat during larger storm events. The design of any treatment facility designed to store and release the WQCV must include bypass capacity for the 100-year peak inflow rate into the facility. This is often provided in the form of an overflow weir set at the elevation of the design WQCV. Deign guidance is provided in the Fact Sheets in Volume 3 of the USDCM.

6.10.4.3.3 Detention

When a site requires detention in addition to permanent water quality treatment, a single facility may be designed to serve both needs. Detention facilities that are also water quality treatment facilities have outlets to provide for both the stormwater quality treatment and detention release rates. For these facilities, the guidance in the USDCM for Excess Urban Runoff Volume (EURV) facilities shall be followed. Section 7.12 discusses detention further.

6.10.4.4 Infiltration

It is common within the Town of Breckenridge for the subgrade soils to have a high infiltration rate. Infiltration testing is required when infiltration facilities are planned. Infiltration rates must be established by a double ring infiltrometer test performed by a geotechnical engineer at each proposed infiltration site at the proposed elevation of the bottom of the filter material prior to beginning detailed design. Treatment facilities acceptable for use in the Town that can be designed to infiltrate the WQCV include sand filter basins, rain gardens, drywells, infiltration trenches, and infiltration galleries. Other types of facilities may be considered on a case-by-case basis. However, if the same area being used as an infiltration treatment facility will serve as a snow storage area, the calculation of the required volume must be increased by an additional 50% of the full design snow storage volume.

Infiltration facilities must have a high enough subsoil permeability to infiltrate the entire WQCV of the area draining to them within 6 hours without the use of underdrains. Infiltration facilities shall be designed to drain 150% of the WQCV (see attached fact sheet for more information) within 72 hours. Infiltration facilities are not allowed within 300 feet of any active waterway without appropriate pretreatment prior to infiltration (examples include sand filters, water quality vaults, detention basins, and rain gardens). Infiltration facilities located between 300 and 1000 feet from an active waterway must not drain the WQCV in less than 4 hours unless appropriate pretreatment is designed. If subgrade soils at these facilities result in infiltration of the WQCV in less than 4 hours, a sand layer must be designed to slow the infiltration of the WQCV to a minimum of 4 hours. There are no restrictions on infiltration facilities located more than 1000 feet from an active waterway.

Guidance on how to design sand filter basins and rain gardens to infiltrate the WQCV standard are included in the Fact Sheets in Volume 3 of the USDCM. Sand filter basins are referred to as simply sand filters, rain gardens are referred to as bioretention. Guidance on how to design drywells, infiltration trenches and galleries are included in the Fact Sheets attachment to this chapter.

Permeable pavements may be considered with a legally binding maintenance agreement in place as alternative to conventional pavement in pedestrian areas and lower-speed vehicle areas. They will not be allowed for treatment of tributary areas with high sediment yields that could easily clog the system and they are not allowed in the public right-of-way. Permeable pavements will not be allowed on steep slopes or in areas receiving runoff from bare or nearly bare earth. Permeable pavements will be considered impervious materials for detention and water quality calculations and must be designed in accordance with manufacturer's recommendation.

See the subsurface infiltration facility fact sheet in Appendix D of these Standards for additional information and requirements for designing infiltration facilities.

6.10.4.5 Proprietary Structures

Proprietary structures are designed to treat a design flow rate instead of a design volume, and typically function by gravitational separation, vortex separation, filtration, or by screening and retaining pollutants within the structure. The use of proprietary facilities is acceptable but generally discouraged as more frequent maintenance is typically required to maintain adequate performance, proprietary structures often have high long-term costs, and they are often very large while treating only a small flow rate. Proprietary facilities that require removable or replacement cartridges and those that require confined space entry procedures or remote camera operation for routine inspections may be considered on a case-by-case basis.

The two most recognized national programs that test the TSS removal of proprietary structures are in Washington state and New Jersey. Proprietary structures acceptable for use in the Town are those that have been tested by one of these programs and have received one of the following:

1. General or Conditional Use Level Designation for the Pretreatment or Basic test protocols of the Washington State Department of Ecology (WSDOE) Test Assessment Protocol – Ecology (TAPE) for emerging stormwater treatment technologies
2. Certification by the New Jersey Department of Environmental Protection (NJDEP) with verification by the New Jersey Corporation for Advanced Technology (NJCAT) that the manufactured treatment device (MTD) is adequate for TSS removal.

The Town may consider allowing a proprietary structure that is not approved by WSDOE or NJCAT provided that, as part of the drainage report, a qualified professional engineer submits adequate documentation, as determined by the Town, of the manufacturer's test data showing similar performance to that required by either the WSDOE or the NJCAT. The level of scrutiny during review for approval of such devices may be significant.

Proprietary structures must be designed to provide water quality treatment for the 2-year peak design flow rate from the area they are treating. Bypass capacity must be included for the 100-year peak flow rate from this same area. If bypass capacity is not included within the structure, it must be provided before the structure in the form of a diversion for higher flows provided the first flush flows pass to the treatment facility.

6.10.4.6 Treatment in Series

Treatment in series is a very effective way to provide water quality treatment and is highly encouraged. Treatment in series, also referred to as a treatment train, involves passing stormwater from one treatment facility to the next, with each facility providing additional treatment. The facilities used in a treatment train are positioned so facilities that can handle a larger, coarser pollutant load are first, while those facilities that are more suited to a smaller, finer pollutant load are last. This allows for longer periods between required maintenance, especially for the facilities that provide more refined removal.

One option for calculating TSS removal rates is presented in the *Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality* (EPA,1986). The development of this manual was supported by the EPA Nationwide Urban Runoff Program (NURP). While the entire document is worth reviewing, Chapter 4.2.1 of this document presents an analytical method under which TSS removal can be evaluated under the dynamic conditions expected in permanent stormwater treatment facilities. Removal due to sedimentation in a dynamic system is expressed by the equation below.

$$R = 1 - \left[1 + \frac{1}{n} * \frac{V_s}{Q/A}\right]^{-n} \qquad (6.19)$$

Where:

R = fraction of solids removed

V_s = settling velocity of particles

Q/A = rate of applied flow divided by surface area

n = turbulence parameter

The turbulence factor offers a way to factor in poor performance caused by turbulence and short circuiting, with n=1 representing very poor performance and n=5 or more indicating very good performance. With n equal to infinity, removal efficiency is linked to detention time. This equation is useful in areas without enough relief to drain a pond that could hold the entire WQCV, or to design basins in series or those receiving inflow that may have already been partially treated by a different type of upstream treatment facility.

Particle settling velocity is calculated as the submerged weight of a particle minus the drag. This calculation requires the minimum particle size of interest be specified. It also requires the viscosity of water, which varies with temperature. To meet Town requirements, a spherical particle with a diameter of 60 microns may be used. A water temperature of 40 degrees Fahrenheit may be assumed, having a viscosity of 1.664×10^5 ft^2/s.

Given the Town requirement for 80% removal of the 60 micron particle, an appropriate treatment train composed of any types of permanent treatment facilities discussed herein may be designed using the equation above. Grass buffers and grass swales are especially helpful at the upstream end of a treatment train. Each of the facilities in the treatment train must include physical components in accordance with these Standards and with the manufacturer’s recommendations for permeable pavement, if applicable. A Treatment Train Example Calculation is included as an attachment to this chapter.

6.10.5 Designing for Inspection and Maintenance Requirements

Long-term maintenance requirements are a critical component of treatment facility selection and design because facilities that are not properly maintained do not function properly and/or do not treat stormwater runoff to the extent required by the Town. All new facilities shall be designed to facilitate maintenance operations.

Maintenance considerations that must be evaluated during the selection and design process include accessibility, required equipment, frequency of maintenance, special required training, the freeze/thaw nature of the Town’s climate, and the need for replacement materials.

Property owners shall be responsible for the maintenance of stormwater detention and water quality facilities to ensure proper functioning. See Section 6.10.5.7 for additional maintenance requirements.

6.10.5.1 Access

An easement shall be provided to allow Town staff to access the facility if the responsible agency is negligent. Access roads shall be provided to all structures including forebays, outfalls, inlets, micropools, outlet structures, and proprietary facilities. Access roads must support an 80,000-pound vehicle load, be at least 10 feet wide, and have an inside turning radius of at least 25 feet. Access roads must pass close enough to each treatment facility component needing maintenance (forebays, micropools, subgrade vaults, etc.) that the equipment to be used will be able to reach the structure. For example, a backhoe with a reach of 25 feet must be able to pass within 25 feet of the structure without leaving the access road. The Town’s vacuum truck has a reach of only 5 feet. If a vacuum truck is required for maintenance, the access road must pass within 5 feet of the facility being maintained. If the access road cannot pass close enough to the structure for the equipment to reach, a concrete access ramp or similar supportable material, such as a 6-inch layer of angular 4-inch-minus riprap, shall be provided at a slope no steeper than 10:1. Concrete ramps shall be scored for wet weather traction.

6.10.5.2 Forebays

Accessible forebays shall be provided at all concentrated inflow points of extended detention basins and sand filter basins for removal of accumulated sediment and floatables. The forebay must be designed in accordance with the Fact Sheets in Volume 3 of the USDCM.

6.10.5.3 Stockpile Areas

Facilities should be designed with stockpile areas for temporary storage and drying of mucked out material. A stockpile area should be directly adjacent to each structure to be cleaned out. Stockpile areas shall be located outside the low flow area and should be as flat as possible. Total stockpile area shall be twice the square footage of all forebays and the micropool.

6.10.5.4 Extended Detention Basins

Extended detention basins shall include a micropool with a hard bottom against which to excavate that is accessible to a vacuum truck or backhoe. Extended detention basins shall include a trickle channel at least 48 inches wide with no cross slope. Trickle channels shall have a concrete bottom and sides at least 6 inches high to allow access for lightweight maintenance equipment if the trickle channel longitudinal slope is less than 5%. If there is a desire for a more aesthetically natural trickle channel, the Town Engineer shall be consulted to discuss and determine maintenance requirements and protocols.

6.10.5.5 Infiltration Facilities

Sand filter basins, rain gardens, drywells, and infiltration trenches have a short lifespan before needing replacement filter material. These facilities shall include pretreatment to remove trash and larger sediment. If located next to a roadway, infiltration facilities shall also include a barrier to prevent degradation of the roadway subgrade. Infiltration testing is required prior to facility design if an infiltration facility is planned. Infiltration testing shall also be conducted after construction and prior to acceptance to ensure the facility functions as intended.

During construction and maintenance operations, special care shall be taken to avoid compaction of subsurface soils that will reduce infiltration rates. If a full infiltration basin is not used, an underdrain system including a loading evaluation will be required and cleanouts must be provided every 300 feet.

Guidance on how to design drywells and infiltration trenches and galleries is included in the Fact Sheets included as attachments to this chapter.

6.10.5.6 Proprietary Structures

Proprietary structures must not require confined space entry procedures for maintenance. Careful design of access to vaults and use of a vacuum truck may eliminate confined space entry requirements.

6.10.5.7 Ownership and Maintenance Plan and Agreement

Treatment and detention facilities require regular inspection and maintenance, to be completed by the property owner. Table 6.23 lists the minimum required inspection and maintenance schedule and typical maintenance activities and operational protocols for various types of treatment and detention facilities. Based on site conditions, the design engineer may require additional maintenance measures, a more frequent schedule, or unique protocols for a site. Volume 3 of the USDCM should be consulted when determining the maintenance schedule, activities, and protocols to be included on the O&M Plan.

An Ownership and Maintenance (O&M) Plan must be developed, submitted, and approved as part of the Drainage Report and then also included as part of the Ownership and Maintenance (O&M) Agreement. An executed O&M Agreement may be required prior to facility approval and project close out. The Town will have the right to access the property to maintain the treatment or detention facility and invoice the owner for the cost of such work if the owner fails to maintain the facility. An example O&M Plan, including a template for the O&M Plan text, and an O&M Plan checklist are included as attachments to Chapter 2. Town staff is available for consultation during the treatment and/or detention facility selection and design process to ensure the design meets the requirements of these Standards. Minimum required components for maintenance are presented in Table 6.23 below.

Town staff will routinely inspect facilities or respond to complaints relating to facilities that may not be performing properly. Facility owners should expect notification of inspections and subsequent findings to be communicated by inspection personnel.

Table 6.23. Recommended Inspection and Maintenance Schedule

Extended Detention Basin	
Activity	**Required Frequency**
Inspection for debris at outlet, sediment in the forebay, and damage to structures or embankments; maintain or repair as necessary.	Twice annually
Remove sediment from forebay, trickle channel(s), and micropool; aeration of vegetated areas	Annually
Mowing	As needed to maintain 6” height and control weeds
Irrigation and application of fertilizer, herbicide, and pesticide	As needed to maintain vegetative health

Notes: Maintenance frequency is highly dependent on construction activity within the tributary area, associated erosion control measures, and the design of the facility. More frequent removal of accumulated sediment may be required, but detention basins are generally low maintenance facilities.

Sand Filter Basin, Rain Gardens, Dry Wells, and Infiltration Trenches and Galleries	
Activity	**Required Frequency**
Inspection to confirm infiltration rate after rainfall; maintain as necessary. Debris and litter removal.	Twice annually
Mowing, plant care, irrigation, and application of fertilizer, herbicide, and pesticide (for bioretention only)	As needed to maintain vegetative health
Mulch replacement (for rain gardens only)	As needed to maintain 3” depth
Inspection of underdrain	When ponding lasts longer than 24 hours
Sediment removal and replacement of media	When ponding lasts longer than 24 hours and underdrain is not clogged

Proprietary Structure	
Activity	**Required Frequency**
Inspection for debris that may cause bypass of design treatment flow rate; maintain as necessary.	Quarterly for first 2 years, as indicated based on first 2 years after that

Filter cartridge inspection; replace as necessary.	Twice annually
Debris removal, filter cartridge replacement, and vacuuming	As recommended by the manufacturer

6.11 CONSTRUCTION STORMWATER MANAGEMENT

Management of stormwater runoff from construction sites in the Town of Breckenridge contributes to our community goal of protecting and maintaining water quality within our local watercourses. A significant amount of sediment can be discharged with stormwater runoff from a construction site. This sediment has the potential to end up in receiving streams, lakes, and rivers, negatively impacting the riparian and aquatic habitat. Establishing a program to minimize untreated runoff from construction sites is essential to keeping streams and drainageways healthy and to minimizing impacts from pollutants and litter.

This section presents the Town's requirements regarding the control of stormwater runoff quality from construction sites, the applicability of the requirements, design guidance for selected BMPs, and references that provide additional information and details. The goal of the requirements in this section is to keep the Town's streams and drainageways healthy. Volume 3 of the USDCM includes a substantial amount of guidance on construction stormwater management and is referenced throughout this section.

The naming convention for facilities or practices that treat stormwater quality during construction has varied over time and between documents. These facilities and practices have been called best management practices (BMPs) in the past. While some more recent documents refer to them as control measures or temporary control measures, these Standards will continue to refer to them as BMPs.

6.11.1 Applicable Codes and Permits

The Town requires applicable construction activity to obtain at least one permit from the Town prior to beginning construction activity. See Chapter 2 for permit requirements. A Stormwater Management Plan (SWMP) or Construction Site Management Plan (CSMP) is required as part of these permits. CSMP requirements are in the Town Code.

Applicable construction projects must apply for a permit from the CDPHE to be covered by the CDPS General Permit for Stormwater Discharges Associated with Construction Activity (Permit No. COR400000). For activity involving dewatering operations, a Construction Dewatering Permit from the CDPHE is required. Information on these permits, and others that may be required during construction, can be found at the CDPHE's webpage for water quality construction permits.

CDOT Erosion Control and Stormwater Quality Field Guide will be used to ensure compliance designing, installing and maintaining BMPs through construction.

6.11.2 Applicability within the Town

The Town's requirements depend on the area disturbed by the project. Applicable projects must develop a Stormwater Management Plan (SWMP). Non-applicable projects, those disturbing less than one acre including any commercial, multi-family, or applicable single family/duplex development, must develop a Construction Site Management Plan (CSMP). CSMP requirements are in the Town Code

A SWMP consists of construction plans and details, specifications for BMPs, and a narrative report that collectively indicate how a construction site will implement structural, non-structural, and planning measures to reduce erosion at the source and prevent pollutants such as sediment and litter from leaving the site. Detailed SWMP requirements are included later in this section.

The Town also requires a Construction Site Management Plan (CSMP) for certain projects. For sites disturbing one acre or more, the CSMP applies only to traffic and site access control. For sites disturbing less than one acre, the CSMP applies to traffic, site access, and erosion and sediment control. Criteria in this section for the design of BMPs and for site inspections apply to sites that require a SWMP and those that require a CSMP.

6.11.3 Stormwater Management Plan Requirements

The intent of a Stormwater Management Plan (SWMP) for construction activities is to prevent pollution, contamination, or degradation of waters of the State and to prevent discharge of pollutants from a project site. Appropriate BMPs must be implemented prior to the start of construction activity, must control potential pollutants during each phase of construction, and must be maintained in operational condition through final stabilization. BMPs must be selected, designed, installed, implemented, and maintained to provide control of all potential pollutants including sediment, construction site waste, trash, discarded building materials, concrete truck washout water and materials, chemicals, sanitary waste, and contaminated soils in discharges from the site. The SWMP will include the following at a minimum:

6.11.3.1 SWMP Report

The SWMP Report is a narrative description and summary of the project site and proposed improvements, how construction activities will be conducted, and the erosion and sediment control practices, procedures, and physical BMPs that will be installed or implemented on site. Items to be included in the SWMP Report are:

1. Administrative Information: Include the project name, location, owner, operator, qualified stormwater manager that will conduct inspections, and the CDPHE certification number.
2. Existing Site Information: Include a description of the existing site, the existing site vegetation and percent density, and the name of the receiving water.
3. Proposed Improvements: Include a description of the proposed project, a description of construction activities, the total area of planned disturbance including access, staging, and storage areas, a sequence of major activities, an approximate construction schedule, and a description of how the project site will be phased.
4. Structural Control Measures: Include a description of the structural BMPs for each stage of construction. Each site must provide structural BMPs that:
    a. Trap sediment before it leaves the site or enters the municipal storm sewer system or watercourse. These shall be installed prior to initiating earth disturbances. Examples include check dams, inlet protection, sediment basins, sediment control logs, and silt fence. Sediment basins are required for sites that have more than 3 acres of tributary area or are directly adjacent to a wetland or major tributary.
    b. Capture and retain runoff from vehicle and equipment washing operations, such as cleaning of concrete trucks, chutes, and associated equipment. An example is the concrete washout. Equipment wash water may not be discharged to State waters or storm sewer systems.
    c. Stop erosion at the source and minimize off-site vehicle tracking of sediment. These include BMPs that stabilize earth disturbances with vegetation or soil stabilization techniques after grading is substantially complete on any portion of the site or for any portion of the site that is inactive for a certain duration of time. Examples include surface roughening, mulching, vehicle tracking control, and installation of blankets, straw wattles, tackifiers, netting, and matting.
    d. Provide bulk storage and prevent spills of petroleum products and other chemicals or fertilizers and contain storm runoff from construction wastes to a designated area, if

applicable. These BMPs shall be able to contain all spills and prevent any spilled material from entering State waters. Secondary storage must be provided, and all bulk storage shall be located as far away from State waters as possible.

e. Provide final stabilization of a site. These may include sod, seed, mulch, landscape plantings, decorative rock, or hard surfacing such as pavers, concrete, or asphalt. Seed mixes must be provided. An anticipated schedule must be provided. If a site is to be winterized, discuss temporary stabilization measures to be utilized. The Town may require a revegetation and stabilization bond for some projects.

5. Non-Structural Control Measures: Include a description of the non-structural BMPs that will be used throughout construction. This section of the report must include:
   a. Discussion about how the following operations will be conducted: trash management; dust and particulate management; materials loading and unloading; vehicle and equipment maintenance and fueling; building materials, chemical, fertilizer, and stockpile storage; routine maintenance involving fertilizers, pesticides, detergents, petroleum products, and solvents; and concrete and batch plants.
   b. A plan to remove from the site and dispose of all waste composed of building materials in licensed disposal facilities. No building material waste or unused building materials shall be buried, dumped, or discharged at the site.
   c. A program and schedule for regular inspection and maintenance of BMPs.
6. Include a description of how construction will be sequenced to reduce the duration any disturbed areas are exposed. Temporary disturbed areas and disturbed areas that will be permanently stabilized shall be exposed no longer than 30 days.
7. Addition Potential Pollutants: Identify and address pollutant sources associated with any other areas or operations not included above where spills can occur. Identify other non-stormwater discharges not included above including construction dewatering not covered under the Construction Dewatering Discharges general permit and wash water that may contribute pollutants to Town waters.

6.11.3.2 SWMP Plans

The SWMP Plans are construction plans that locate and identify the number, extent, and installation details of the structural and nonstructural BMPs included and discussed in the SWMP Report. The SWMP plans must also include property boundaries, construction site boundaries, existing and proposed utilities, limits of cut and fill, stockpile areas, porta-let locations, arrows to depict the direction runoff will flow, and the locations of all receiving waters and drainages.

6.11.3.3 SWMP Specifications

BMP materials specifications and installation requirements may be provided with the BMPs installation details included with the SWMP Plans or as separate technical specifications in the project manual. Installation and implementation specifications must be provided for all structural BMPs in one of these two documents.

6.11.4 SWMP Submittal, Inspection, & Maintenance

For applicable sites, a SWMP must be submitted to the Town for review and approval prior to construction. Once the SWMP is approved and construction begins, regular inspection and maintenance of the BMPs will proceed throughout the duration of construction. The subsections below provide more detail on these requirements and processes.

6.11.4.1 SWMP Submittal Requirements

SWMP plans are construction plans that depict the type and location of structural and procedural BMPs that will be implemented on site during various stages of construction. SWMP plans must be submitted to the Town for review and approval. A SWMP shall be developed by a qualified stormwater professional.

Each SWMP must have separate sheets for the initial/interim and final phases of construction as well as detail sheets with a design and installation detail for each BMP specified in the SWMP. A legend shall be provided that includes all abbreviations used (e.g. CWA is used for concrete washout area) as well as all symbols, blocks, and line types that represent various BMPs. Each kind of symbol, block, or line type shall be labeled at least once on each sheet on which it is used. Text on all SWMP plan sheets, including the detail sheets, shall be legible when the SWMP plan set is printed on 11 x 17 paper, using at least a 9-point font. In the case of overlapping or adjacent project sites that are separately managed, the SWMP shall include at least one plan sheet that clearly shows the site area managed as part of the submitted permit. If more than one sheet is required for each phase of the SWMP plan set, the first sheet of the initial phase shall match the first sheet of the interim phase, and so forth. For project sites that are adjacent to or within construction activity that is not being performed by the same owner or operator, the SWMP shall delineate the exact project site boundaries that each owner or operator is responsible for.

6.11.4.1.1 Initial SWMP Plan

Initial SWMP plans include all BMPs that will be installed before construction begins. These shall include vehicle tracking at all exits from the site and enough stabilized staging area to accommodate the site's operations. Initial SWMP plans will also include any construction fence, inlet protection, curb socks, perimeter controls, or concrete washout areas that are required on site. Initial BMPs shall be identified on the initial SWMP plan by their two- or three-letter abbreviation and a quantity such as linear feet (LF) or square feet (SF), where applicable. The initial plan may be combined with the interim plan for small sites that do not have complex phasing. Sites disturbing 40 acres or more must have separate initial and interim SWMP plan sheets that detail the planned phasing of the site, and it is likely that more than one of each will be required to cover the larger area at a reasonable scale.

6.11.4.1.2 Interim SWMP Plan

Interim SWMP plans include all BMPs that will be installed as construction progresses. These include all BMPs such as temporary stream diversions, check dams, sediment basins, stockpile areas, sediment control logs on exposed slopes, and culvert outlet protection, among others. The interim SWMP plan should include all the BMPs shown on the initial SWMP plan, but they should have their dimensions removed for clarity. Like the initial SWMP plan, interim BMPs shall be identified on the interim SWMP plan by their two- or three-letter abbreviation and a quantity such as linear feet (LF) or square feet (SF), where applicable. Operators are encouraged to phase construction sites to minimize erosion. A complicated site may require more than one interim SWMP plan. The interim plan may be combined with the initial plan for small sites that do not have complex phasing. Sites disturbing 40 acres or more must have separate initial and interim SWMP plan sheets that detail the planned phasing of the site, and it is likely that more than one of each will be required to cover the larger area at a reasonable scale.

6.11.4.1.3 Final SWMP Plan

Final SWMP plans include final stabilization BMPs such as seeding, mulching, and erosion control blanket. For some sites, the landscape plan, if it is comprehensive and included in the construction plan set, may be used as the final SWMP plan, provided the required information is included. In other cases, a separate final SWMP plan will be required. The final SWMP plan should generally not include initial and interim BMPs and abbreviation labels. If an initial or interim BMP is to remain, it shall be included on the final SWMP plan and noted to remain. Otherwise all initial and interim BMPs shall be assumed to removed.

6.11.4.1.4 SWMP Details

Details for all BMPs specified shall be provided on SWMP plan detail sheets. Details shall be in accordance with CDOT or MHFD standard details.

6.11.4.2 SWMP and CSMP Inspection

Each SWMP permittee shall designate a qualified stormwater manager to inspect the BMPs on the construction site. Inspections should occur either weekly or every 14 days and after each runoff-producing storm event to confirm they are installed and functioning as intended, beginning with an inspection of the initial BMPs prior to any excavation and ending with final approval of the site by the Town. The stormwater manager shall be an individual knowledgeable in the principles and practices of erosion and sediment control and pollution prevention, and with the skills to assess: 1) conditions at construction sites that could impact stormwater quality and 2) the effectiveness of stormwater controls implemented to meet the requirements of this permit.

The stormwater manager should keep a record of all inspections including the date and time of the inspection, recent precipitation, required maintenance activities to be completed, and a summary of maintenance completed since the previous inspection. The stormwater manager shall update the SWMP (or CSMP) plans to reflect current conditions by showing changes made to the location and/or type of BMPs based on their performance and/or inspection reports throughout the duration of construction. An updated set of SWMP plans, including installation details, shall be on site during construction activities.

The Town will periodically inspect construction sites for conformance to the SWMP, to confirm installation, maintenance, and function of the BMPs are adequate, and to assure compliance with the Town's permit(s) and Town Code. Work that is not in compliance with the Town's Standards, the SWMP, the CSMP, or the Town Code is subject to enforcement action.

6.11.4.3 SWMP Maintenance

Maintenance of BMPs is typically ongoing for the duration of construction. The SWMP permittee shall schedule any required maintenance noted during regular inspections to be completed prior to the next inspection or reinspection. The SWMP must be updated as needed to reflect current site conditions and be maintained on site. Possible modifications may include replacing and adding BMPs and identifying additional pollution sources. Hand-written notations, initialed and dated, are adequate for most plan updates.

6.11.4.4 SWMP Adequacy

If BMPs installed on the site are inadequate to properly control pollutants during construction as evidenced by their performance during or after storm events or as identified during inspections, the SWMP permittee shall immediately complete any modifications required to properly control pollutants or those noted during inspection. Modifications completed shall be physically noted in the SWMP plans. BMPs, or lack thereof, identified as inadequate based on a Town inspection shall be rehabilitated immediately.

6.11.5 Temporary BMPs

Temporary BMPs are structural or site planning BMPs that are utilized to minimize sediment or other pollutants during construction activities. These BMPs shall be removed from the site upon completion of site stabilization unless they are designated to remain by the SWMP or by the Town because, for example, they do not impede use or maintenance of the site or will biodegrade. This section presents some specific structural and site planning BMPs that are to be used during construction. It also offers several reference documents that provide design and construction details for BMPs that are not specifically discussed in this section.

6.11.5.1 Vehicle Tracking

Vehicle tracking pads are an essential part of preventing sediment from leaving a construction site. Vehicle tracking pads shall be implemented at every exit point to the site, regardless of the type of equipment that will be exiting at each location. The stone used should be hard, durable, angular stone, resistant to weathering with a long dimension not less than 3 inches. Larger stone is preferable, and stone should have a minimum specific gravity of 2.6. Other means of keeping sediment on site may be acceptable with proper documentation and provided performance is maintained. Any damage to sidewalk, curb, or gutter shall be replaced by the permittee. The minimum dimension for each vehicle tracking pad is 20 feet wide by 50 feet long by 6 inches thick, although more length may be required if the minimum size does not provide adequate performance.

6.11.5.2 Sediment Basins

Sediment basins to capture construction site runoff shall be installed on construction sites that disturb more than 3 acres. Runoff is detained in sediment basins and slowly drained so that sediment may settle out before the runoff leaves the site.

For sites with less than 40 acres of disturbance, at least 66% of disturbed area shall drain to a sediment basin. Multiple basins may be more efficient to achieve this requirement than a single basin, depending on the site configuration. Each basin must have a way to drain detained runoff as well as an emergency overflow with a revetted overflow path for runoff exceeding the sediment basin design volume. Sediment basins should drain passively through an outlet structure designed to drain the full basin volume within 12 hours. Designs for outlet structures should be per the criteria references provided in 6.11.5.7. Sediment basin storage volume may also be actively drained by pumping through a sediment bag with the basin being pumped empty when the basin volume reaches 50% of capacity or when rain is forecasted within 24 hours. When pumping is proposed in lieu of a passive outlet structure but is not performed per this criteria, the owner and operator are subject to enforcement action for improperly maintaining the selected BMP.

Each sediment basin shall provide 1600 cubic feet of storage per acre of tributary area provided it is not utilized as a BMP during the months of March, April, or May. If a sediment basin is utilized as a BMP during March, April, or May, it shall provide 3200 cubic feet of storage per acre of tributary area.

For sites with 40 acres or more of disturbance, every acre of disturbance exceeding 40 acres shall drain to a sediment basin. For example, a site with 39.9 acres of disturbance would be required to drain 26.3 of those disturbed acres to a sediment basin. A site with 45 acres of disturbance would be required to drain 31.7 of those disturbed acres to a sediment basin. This criteria encourages owners and operators to minimize the area of site disturbance at any one time to less than 40 acres.

Sediment basin design elements that must be in accordance with the Volume 3 of the USDCM include a spillway crest length based on tributary drainage area, a 12-inch minimum distance between the overflow crest and the surrounding embankment, outlet protection for the spillway, outlet works, and embankment slopes.

6.11.5.3 Check Dams

Check dams may be used to slow runoff in drainageways to limit erosion and sediment transport. Different agencies have specified different cross sections for check dams in their BMP details, however, all agencies specify that the elevation of the crest of the downstream check dam should be equal to the downstream toe of the next upstream check dam. This may become unreasonable in because of the steep grades of some drainageways. For check dams in Breckenridge, the following criteria apply.

1. For preliminary sizing and spacing, check dam height above existing grade shall be a minimum of 18 inches and a maximum of 3 feet. Check dams shall be spaced along the drainageway such that the crest of the downstream check dam is at the same elevation as the downstream toe of the next upstream check dam.
2. If preliminary sizing results in a check dam spacing of less than 200 feet (i.e. the drainageway is steeper than 1.5%), Check dam height may be increased to 4 feet or may spacing may be less than 200 feet for slopes up to 2%. If drainageway slope exceeds 2%, the SWMP preparer shall propose a check dam height and spacing that he or she believes will meet the intent of the intended purpose of the check dam. The proposal shall be discussed during a SWMP review meeting with the Town.
3. Check dam embedment in the underlying ground shall be at least 12 inches and stones used in the check dam shall have a D50 of at least 12 inches.
4. A plan view and sections of a typical check dam used on the site will be provided with the SWMP plan as an initial BMP.
5. All check dams must be removed as part of final stabilization unless designated or approved to remain by the Town.

6.11.5.4 Materials Storage and Stockpiles

Construction materials that are not earth or aggregate shall be stored on a stabilized staging area. Earth and aggregate may be stockpiled outside of a stabilized staging area, but earth materials must be bounded by silt fence or some other BMP that will prevent sediment from escaping the stockpile during a runoff event.

The stabilized staging areas shall be large enough to store all required materials, provide parking for vehicles and equipment on site, and accommodate loading and unloading activities. The stabilized staging area must be installed as an initial BMP, prior to any other construction activities. The size of the stabilized staging area will vary with each site but may be required to be enlarged if inspections show it is not sized sufficiently to contain all required items and activities. Each stabilized staging area shall consist of granular material at least 3 inches thick unless it is demonstrated that native materials provide adequate stabilization. If rutting occurs, or if the underlying subgrade becomes exposed, additional granular material will be required. Once construction is complete, the granular material shall be removed, and the site shall be stabilized in accordance with the final SWMP plan or landscape plan.

6.11.5.5 Temporary Stabilization and Winterization

All areas disturbed by construction or stockpiles shall be stabilized as soon as possible to reduce the duration bare soil is exposed to runoff. All disturbed areas which are either final graded or will remain inactive for a period of more than 30 days shall be stabilized after the completion of the grading activities. Acceptable temporary stabilization BMPs include surface roughening, seeding and mulching, erosion control blankets or turf reinforcement matting, and tarping for stockpiles. Temporary stabilization by revegetating should take place progressively as the project moves forward and as soon as feasible.

To prevent damage during spring runoff, all disturbed areas shall be temporarily stabilized with one of the acceptable methods listed above prior to winter. While the Town requires winterization activities by completed by December 1, it is recommended that winterization be completed by November 1.

6.11.5.6 Erosion Blanket

Any embankment, cut, or fill slope that is in its final graded state and steeper than 3H:1V shall be seeded and covered with erosion blanket within 30 days of final grading being completed. Erosion blanket installation shall be in accordance with the manufacturer's details and specifications which shall be included on the SWMP plan detail sheet.

6.11.5.7 Control Measure Specifications and Details

Unless the Town has included its own temporary BMP details or design guidance in this section, construction details and design procedures shall be as presented in the most recent version of one or more of the following references. Note that these documents are updated regularly and can be found via an internet search.

1. Colorado Department of Transportation Erosion Control and Stormwater Quality Guide and Standard Plan M-208-1.
2. Urban Drainage & Flood Control District Urban Storm Drainage Criteria Manual, Volume 3 Stormwater Quality.
3. Southeast Metro Stormwater Authority Grading, Erosion, and Sediment Control Manual.

The BMPs presented in the documents referenced above shall be designated in the SWMP where appropriate, and details for each BMP specified shall be included in the SWMP. Use of alternate BMPs not specified above is subject to approval by the Town.

6.11.6 Construction Site Inactivity

Temporary stabilization must be implemented for earth disturbing activities on any portion of the site where ground disturbing construction activity has permanently ceased, or temporarily ceased for more than 30 calendar days. Temporary stabilization methods may include, but are not limited to, hydromulching, tarps, soil tackifier, hydroseed, and erosion control blankets. The permittee may exceed the 30-day schedule when either the function of the specific area of the site requires it to remain disturbed, or physical characteristics of the terrain and climate prevent stabilization. The SWMP must document the constraints necessitating the alternative schedule, provide the alternate stabilization schedule, and identify all locations where the alternative schedule is applicable on the site map.

6.11.7 Final Stabilization

Final stabilization is reached when all surface disturbing activities at the site are complete and a uniform vegetative cover has been established with a plant density of at least 70% of pre-disturbance levels, or an equivalent amount of permanently stable surface has been constructed. Permanently stable surfaces include landscape rock, wood mulch, and landscape pavers. Only the vegetation specified in the SWMP planting plan or seed mix shall count toward plant density.