City of Steamboat Springs
Engineering Standards

SECTION 5.0
DRAINAGE CRITERIA

SEE SEPARATE DOCUMENT

Effective September, 2007

City of Steamboat Springs
Department of Public Works
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DRAINAGE CRITERIA INTRODUCTION

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All figures and tables are numbered and are placed either within the text or at the end of their respective sections.

5.1.2 PURPOSE AND INTENT

The purpose of these drainage criteria is the promotion of public health, safety, environmental stewardship, and general welfare of City roads and property while minimizing the possible flood damage to surrounding properties and structures by adopting policies, procedures, standards, and criteria for storm drainage.

All new development, redevelopment, and significant remodel projects submitted for acceptance under the provisions of these drainage criteria shall include adequate storm drainage system analysis and appropriate drainage system design. Such analysis and design shall meet or exceed the criteria set forth herein.

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5.2.1 PURPOSE

Establishing consistent drainage criteria will help to maintain, control, and implement sound stormwater practices to help safeguard citizens, City infrastructure, and property against adverse conditions from storm events. The primary objective of drainage design shall be the protection of City roads and property while minimizing the possible flood damage to surrounding properties and structures. Good drainage is one of the most important factors in road design.

The purpose of these criteria is to outline the minimum technical requirements governing storm water facilities design in the City of Steamboat Springs. The criteria outline general requirements for drainage studies, site design, and storm water system design.

5.2.2 AUTHORITY

The Community Development Code (CDC) (Section 26 of the City Municipal Code) requires that in conjunction with development project applications, adequate information is provided to support the proposed project. Section 20-1 of the City Municipal Code authorizes the Public Works Director to implement standards governing engineering design.

Regulations of land use activity, including drainage, are granted to a municipality under Colorado Revised Statues as noted below:

CRS 31-15-701 et seq. Grants municipalities the power to establish, improve, and regulate such improvements as streets and sidewalks, water and water works, sewers and sewer systems, and water pollution controls. In addition, a municipality may, among other powers, deepen, widen, pipe, cover, wall, alter or change the channel or watercourses.

CRS 31-25-501 et seq. Authorizes municipalities to construct local improvements and assess the cost of the improvements wholly or in part upon property specially benefited by such improvements. By ordinance, a municipality may order construction of district sewers for storm drainage in districts called storm sewer districts.

CRS 31-25-601 et seq, Authorizes municipalities to establish improvement districts as taxing units for the purpose of constructing or installing public improvements. The organization of districts is initiated by a petition filed by a majority of registered electors of the municipality who own real or personal property in the district.

CRS 31-35-401 et seq. Authorizes municipalities to operate, maintain, and finance water and sewage facilities for the benefit of users within and without their territorial boundaries. Sewerage facilities are defined as "any one or more of the various devices used for the collection, treatment or disposition of sewerage or industrial wastes of a liquid nature or storm, flood or surface drainage waters".

5.2.3 AMMENDMENTS AND REVISIONS

These drainage criteria may be amended periodically by the Public Works Director to reflect current technical practices or other policy revisions.

5.2.4 REVIEW AND APPROVAL

The Public Works Department will review and approve all drainage studies and storm water system design for general compliance with these criteria and standard civil engineering

practices. The approval does not relieve the owner, engineer, or designer from responsibility of ensuring that the calculations, plans, specifications, construction and record drawings comply with these criteria. Designs should also be coordinated with the utility companies as appropriate. See Section 5.3, Stormwater Planning and Submittals, for further guidance on drainage studies.

5.2.5 APPLICABILITY

In general, any project (new, redevelopment, or significant remodel) that alters stormwater drainage from existing conditions, including increasing impervious area, shall be required to evaluate the changes in a drainage report including a Stormwater Quality Plan. Existing sites undergoing redevelopment or remodel that do not alter storm water drainage are required to prepare a storm water quality plan to upgrade the site to meet current storm water quality requirements. See Section 5.3, Stormwater Planning and Submittals, for further guidance.

These criteria shall be followed for every drainage design and drainage study submitted within the City of Steamboat Springs.

US 40 (Lincoln Avenue) is a State highway and design of any drainage facilities on or affecting US 40 must conform to the Colorado Department of Transportation (CDOT) requirements.

5.2.6 OTHER STANDARDS

Where no requirement is given in these criteria, the requirements of the Urban Drainage and Flood Control District's *Urban Drainage Control Manual*, latest edition with modifications made as appropriate for Steamboat's climatic conditions shall govern. Where the City's documents do not cover a specific situation, consult the Public Works Department to confirm appropriate standards.

5.2.7 VARIANCES

On occasion, the unique conditions of a site may not fit within the criteria established herein. The Public Works Department may grant variances. Variances will be evaluated on a site-specific basis by the Public Works Department. The Department shall consider the site-specific constraints contributing to the need for the variance, the effect on safety, constraints to City right-of-way, public benefit, availability of other alternatives, economic feasibility, and the need for mitigation measures. Variances must be requested in writing and at a minimum include:

1. A list of the standards to be varied
2. A summary of the variances proposed to replace the standards
3. Technical sources supporting the variances
4. A description of the unique site-specific constraints contributing to the variance request
5. A summary of alternatives considered and a discussion as to why the standards cannot be accommodated
6. A summary of proposed mitigation measures, if needed

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5.3.1 PLANNING AND EASEMENTS

During the development process, consideration must be given to the location of easements. Easements are required for all water courses, drainageways, channels, streams, roadside ditches, culverts, storm sewers, and detention ponds that are public and located outside of the ROW or private and extending across multiple properties to allow for construction, maintenance, and operation. No trees or permanent structures shall be installed within the easement, and the owner shall not be allowed to change the function of the facility. Property owners shall be responsible for maintenance of stormwater facilities within the easements.

For open channels not including roadside ditches, the width of the easement shall be the width of the top of the channel bank plus 15 feet on at least one side that will allow for maintenance access. For roadside ditches not completely contained within the roadway right-of-way, the width of the easement shall be the width of the top of the ditch bank plus 5 feet on the side away from the road. For culverts and storm drains, the easement shall be 20 feet wide centered on the conduit unless pipe size, topography, or other site condition requires additional width.

Although driveway culverts are typically located in the public right-of-way, they are considered private improvements. Driveway culverts may be periodically maintained by the City, but it is each property owner's responsibility to ensure that his driveway culvert remains open and free from debris and sediment and to repair or replace the culvert as necessary.

For detention and water quality ponds, access to both the forebay and pond floor by maintenance equipment is essential. All reservoirs or ponds which serve more than a single lot or site must be provided a maintenance easement or other appropriate access way to allow a vehicle to access the basin.

5.3.2 SUBMITTAL TYPES

Site development includes development of drainage design and supporting calculations in a drainage study, finalizing the design on the site plan or civil drawings, amending the drainage study with final design changes, and documentation of construction.

The design information for the stormwater system is identified in the Drainage Study. The Drainage Study identifies existing drainage conditions and constraints, estimates proposed drainage conditions, and identifies mitigation measures required to accommodate changes due to the proposed site activity. The Drainage Study is generally prepared during the development planning process to allow the site to be designed to include the necessary stormwater design features and mitigation measures. The Drainage Study also identifies potential site pollutants and establishes which stormwater quality features will be used to minimize the possibility of releasing contaminants from a site. The primary objective of drainage design shall be the protection of City roads and property while minimizing the possible flood damage to surrounding properties and structures. There are five types of drainage studies: the Drainage Letter, the Conceptual Drainage Study, the Final Drainage Study, the Drainage Study Addendum, and the Stormwater Quality Plan. .

5.3.3 SUBMITTAL REQUIREMENTS

All drainage studies must be submitted simultaneously with the development application or construction plans for which they are required. Two copies of the draft study shall be

submitted to the Public Works Department (one for the file and one to be returned with redline comments). All applicable elements in the checklist for each study shall be included or the study may be returned for completion prior to review.

The unique characteristics of some sites may require additional analysis beyond what is listed for each study to clearly define a site's impacts or to adequately design a stormwater or stormwater quality system. Site-specific requirements beyond the minimum standards will be identified during the Technical Advisory Committee (TAC) development review process or can be determined in advance by contacting the Public Works Department.

Public Works will review the study and provide comments. The applicant shall revise and resubmit the plans incorporating the City's comments. Two copies of the final report can be submitted when indicated by the City's response comments (one copy to be retained by Public Works and one by Planning). Where plans are required to satisfy requirements for an application with public hearing or administrative approval, the plans must be finalized and approved by Public Works prior to Public Works approving a development application to move to public hearing or administrative review. In limited cases Public Works may accept conditions of approval to complete minor outstanding issues after the public hearing, but prior to issuance of any permits.

All studies shall be typed on 8 ½" x 11" paper, bound, and clearly and cleanly reproduced. The drawings, figures, plates and tables shall be either 11" x 17" or 24" x 36" and bound within the study or included in a pocket attached to the study. Blurred, washed out or unreadable portions of the study are unacceptable and could warrant resubmittal of the study. The study shall include a cover letter presenting the design application for review and shall be prepared by or supervised by a civil engineer licensed in Colorado. Revision dates must be included on all resubmittals.

5.3.4 DRAINAGE LETTER

A Drainage Letter is a short letter prepared for a site with only minimal storm water improvements or changes. Minimal changes are items such as sizing a single ditch or culvert. Drainage letters are also used for sites that require no stormwater system improvements but require stormwater quality (this may be a redevelopment site, a change of use site, or a site with an approved drainage study that did not include stormwater quality). A Drainage Letter may also be used for a site with an approved drainage study or master drainage plan where the development proposal makes changes to the development that affect drainage characteristics that do not effect sizing of stormwater components; the Drainage Letter can be used to demonstrate that proposed conditions are generally consistent with previous design assumptions. (Where changes affect system sizing or design, a Drainage Addendum is required instead of a Drainage Letter.) The Drainage Letter shall identify offsite flows passing through the site and on site flows that need to be managed, or it shall identify the approved drainage study that contains that information. The Drainage Letter shall include any calculations required to design the minimal stormwater system The Drainage Letters must include a Stormwater Quality Plan as Exhibit A. The Drainage Letter shall also contain the certification statement contained in Section 5.3.7 below.

Standard Form No. 1 (SF1), Drainage Letter Checklist in the Standard Forms Section lists the minimum requirements for a Drainage Letter. SF1 will be used to determine the adequacy of the Drainage Letter submittal in addition to any site-specific requirements identified at the pre-application or TAC review which may alter the checklist. Incomplete or absent information may result in the Drainage Letter being rejected for review.

5.3.5 CONCEPTUAL DRAINAGE STUDY

The Conceptual Drainage Study is a drainage document that outlines general drainage requirements and develops conceptual design for a site. The Conceptual Drainage Study accompanies the conceptual level development applications such as a Development Plan, Preliminary Plat, or Annexation. The purpose of the Conceptual Drainage Study is to:

- Identify existing and potential drainage issues which may occur on-site or off-site due to the development and identify potential mitigation measures as needed.
- Identify existing drainage conditions prior to proposed development including designated floodplain boundaries and off-site flows that need to be managed.
- Identify proposed solutions to drainage issues, the proposed stormwater management system concept, and the proposed stormwater quality concept design including defining the general component requirements and space needed on and off site.

The study shall be stamped and signed by a professional engineer licensed in Colorado and include the certification statement contained in Section 5.3.7.

For sites where the development plan and final development plan are moving through development approval concurrently, or where the developer has a more detailed site design at the conceptual stage, the Conceptual Drainage Study can be skipped and a Final Drainage Study can be provided instead.

5.3.5.1 Conceptual Drainage Study Report Contents

The Conceptual Drainage Study shall contain general design information describing the proposed drainage facilities (type, estimated pipe/ditch/pond size, inlet locations, etc) for the development. Detailed final calculations can be included but are not typically required. On constrained sites, or high density sites with little space for drainage features, more detailed information may be required to demonstrate the proposed layout can accommodate drainage facilities. All Conceptual Drainage Studies must include a Stormwater Quality Plan as Exhibit A. The Conceptual Drainage Study Report shall also contain a certification sheet having the certification statement contained in Section 5.3.7.

Standard Form No. 2 (SF2), Conceptual Drainage Study Checklist, in the Standard Forms Section lists the minimum required components of the Conceptual Drainage Study. The checklist will be used to determine the adequacy of the Conceptual Drainage Study submittal. Additional information may be required based on site-specific conditions, and if so, it will be identified through the TAC process or during the draft study review. Incomplete or absent information may result in the Conceptual Drainage Study being rejected for review.

5.3.6 FINAL DRAINAGE STUDY

The Final Drainage Study provides the specific drainage elements and design required to manage stormwater and stormwater quality at a site. The Final Drainage Study accompanies a Final Plat or Final Development Plan. The purpose of the Final Drainage Study is to:

- Identify existing and potential drainage issues which may occur on site or off site due to the development and any required mitigation measures
- Identify existing drainage conditions prior to proposed development including designated floodplain boundaries and off site flows that need to be managed.

- Identify proposed solutions to drainage issues, the proposed stormwater management system and stormwater quality system detailed design including defining the specific component requirements and sizing for on- and off-site elements.
- Provide final design and details for drainage facilities discussed in the Conceptual Drainage Study.
- Demonstrate adequate drainage improvements will be provided to support the development.

The location, type, and design of the stormwater system shall be identified at a construction plan level or 95% design level. The Final Drainage Study shall provide details of the proposed drainage facilities, including grading and water quality enhancement. Not all design may be completely finalized during the development process, and minor changes may occur during the development of the construction documents. If changes occur during final construction plan preparation or during construction, a Drainage Study Addendum shall be submitted for review and approval along with the construction documents.

The Final Drainage Study shall be stamped and signed by a professional engineer licensed in Colorado.

5.3.6.1 Final Drainage Study Report Contents

The Final Drainage Study shall provide details of the proposed drainage facilities including grading, erosion control, and water quality enhancement. Each Final Drainage Study must include a Stormwater Quality Plan as Exhibit A. The Final Drainage Study Report shall also contain a certification sheet having the certification statement contained in Section 5.3.7 below.

Standard Form 3 (SF3), Final Drainage Study Checklist, in the Standard Forms Section outlines the minimum requirements for a Final Drainage Study. The checklist will be used to determine the adequacy of the Final Drainage Study submittal, in addition to any requirements identified at the pre-application review which may alter the checklist. Incomplete or absent information may result in the Final Drainage Study being rejected for review.

5.3.7 CERTIFICATION STATEMENT

The following certification statement shall be included as a separate sheet for all Drainage Letters, Conceptual Drainage Study Reports, and Final Drainage Study Reports:

> "I hereby affirm that this (Type of Letter/Report) (Plan) for the (Type of Design) of (Name of Development) was prepared by me (or under my direct supervision) for the owners thereof and is, to the best of my knowledge, in accordance with the provisions of the City of Steamboat Springs Storm Drainage Criteria and approved variances. I understand that the City of Steamboat Springs does not and will not assume liability for drainage facilities designed by others.

_____
Registered Professional Engineer
State of Colorado No.
(Affix Seal)

5.3.8 STORMWATER QUALITY PLAN

The Stormwater Quality Plan identifies site-generated storm water flow and potential site contaminants, including sediment. The Plan presents the design for permanent stormwater quality features to minimize the potential for pollutants to leave the site. The purpose of the Plan is to identify post-construction best management practices (BMPs) for the site.

The Plan may also include construction BMPs, or it may refer to a separate Construction SWMP that is prepared to address stormwater quality during construction. Engineers are encouraged to design and phase sites to minimize erosion and contaminants and use features that require minimal maintenance. Section 5.12, Water Quality Enhancement, provides additional information on the Construction SWMP.

The Plan will discuss the permanent water quality facilities in sufficient detail and include appropriate calculations to verify their technical feasibility. Each post-construction BMP shall be identified as to its location and the WQCV it provides (or other basis for sizing the BMP). Maintenance requirements shall be listed in the Plan. Stormwater quality facilities shall be designed per Section 5.12, Water Quality Enhancement.

Standard Form 4 (SF4), Stormwater Quality Plan Checklist, in the Standard Forms Section outlines the minimum requirements for the Stormwater Quality Plan. The checklist will be used to determine the adequacy of the Stormwater Quality Plan submittal, in addition to any requirements identified at the pre-application review which may alter the checklist. Incomplete or absent information may result in the Stormwater Quality Plan being rejected for review.

5.3.9 FINAL DRAINAGE REPORT WITH CIVIL CONSTRUCTION PLANS

Where changes are made to the approved final drainage design during preparation of the construction plans or during construction, and those changes affect such items as flow rates, pipe sizing, pond sizing, or offsite flow type or direction, an addendum shall be submitted along with the construction plans. The addendum shall identify the specific change and provide supporting calculations. The addendum shall be reviewed and approved in conjunction with the civil construction plans.

5.3.10 CONSTRUCTION SWMP

The Construction SWMP must be prepared and certified by a qualified erosion control specialist, and shall be submitted for approval with or after the Final Drainage Study and prior to construction. Additional information regarding the Construction SWMP can be found in Section 5.12, Water Quality Enhancement and in the Municipal Code for Construction Site Management Plans. If there are direct discharges to a named tributary, the Public Works Department may require the drainage engineer to design temporary sediment ponds for construction as part of the Final Drainage Study.

5.3.11 RECORD DRAWINGS AND FINAL ACCEPTANCE CERTIFICATE

5.3.11.1 Record Drawings

Where as-built conditions vary from final design for public facilities, record drawings shall be submitted to the City. Drawings shall be stamped by a professional engineer and shall be submitted with the request for Probationary Acceptance of public improvements or requesting a Certificate of Occupancy for commercial, industrial or multi-family residential building sites.

5.3.11.2 Certifications

An engineer licensed in the state of Colorado shall certify, based on a survey from a registered land surveyor, the as-built detention pond volumes and surface areas at the design depths, and outlet structure sizes and elevations. The engineer shall also certify that, to the best of his/her knowledge, the storm drain sizes and invert elevations, manhole and discharge locations, representative open channel cross-sections, dimensions of all the drainage structures and other pertinent design features are in accordance with the approved drainage study and construction plans.

The responsible design engineer shall submit the documentation of the above with the request for preliminary acceptance.

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5.4.1 INTRODUCTION

The regulation of floodplains is necessary to preserve and promote the general health, welfare, and economic well-being of an area. A few of the general purposes of floodplain regulation are:

- Reducing risk of loss of life and property damage
- Protecting hydraulic characteristics of water courses
- Reducing public expenditures for flood control and relief

This Section gives guidance relevant to development near waterways within the limits of the City of Steamboat Springs.

5.4.2 FEMA-DESIGNATED FLOODPLAIN

The National Flood Insurance Program is administered by FEMA. Its basic function is to designate flood-prone areas and subsequently make available varying degrees of flood insurance protection for those requesting it or those required to possess it. FEMA has identified these areas on the Flood Insurance Rate Map (FIRM) associated with the "Flood Insurance Study (FIS) of Routt County, Colorado and Incorporated Areas" dated February 4, 2005.

For development or redevelopment located within the FEMA-identified areas of special flood hazard, Chapter 26, Article VI, Flood Damage Prevention, of the City Code will apply. In addition to the requirements outlined in Article VI, it is the developer's responsibility and financial obligation to meet and fulfill all FEMA rules and regulations and to prepare any appeals or revisions that may be required as a result of the proposed development. It is the developer's responsibility to prepare any Conditional Letters of Map Revision (CLOMR) or Conditional Letters of Map Amendment (CLOMA) that may be required prior to being granted approval by the City for development within the floodplain. **Figure 5.6.4, Local FEMA Studies** in Section 5.6, Storm Runoff, shows the various waterways that have been studied by FEMA.

Note that floodplain issues often take a substantial amount of time to resolve, and it is recommended that any developer wishing to develop a property within the limits of the FEMA floodplain begin to address these issues at a very early stage in the development process.

5.4.3 DEVELOPMENT OUTSIDE THE FEMA-REGULATED FLOODPLAIN

Even though an area is not located within a FEMA-designated floodplain, it may still be subjected to flooding from unstudied waterways. Developments being divided into parcels of less than 35 acres that also convey offsite flows are potentially subjected to additional requirements as specified in this Section. These developments have the option of either conveying these offsite flows across the surface of the development or enclosing these flows in one or more storm drain systems through the development.

5.4.3.1 Surface Conveyance

If channelized flow (as opposed to sheet flow) from an offsite watershed larger than 130 acres is conveyed across the surface of a new development, the developer must delineate the 100-

year floodplain through the development. At a minimum this will entail delineating the watershed boundary draining to the development, determining peak 100-year flows at appropriate points along the waterway, and developing a hydraulic model of the waterway during the 100-year storm event. A professional engineer registered in the State of Colorado is required to complete and certify the analysis.

Although any construction within the limits of the 100-year floodplain is highly discouraged, it may be allowed provided it meets the requirements set forth in Article VI of the City's Code. In any event, the runoff discharged from the site must not exceed historical runoff rates as set forth in Section 5.11, Detention. The City considers surface conveyance preferable to enclosed conveyance, because of cost, maintenance, aesthetics, and stormwater quality, and sites are encouraged to maintain surface flow where feasible.

5.4.3.2 Enclosed Conveyance

The other option for conveying offsite flows across a development is to enclose them. Although this option is likely substantially more costly than surface conveyance, should a developer wish to pursue this option, there are several criteria that must be met. The conveyance must meet all the criteria set forth in Section 5.9, Storm Drain Systems, and Section 5.10, Culverts and Bridges, including limits of inundation for different design storms. An emergency overflow path for the 100-year event must also be provided in the event that the enclosed conveyance system becomes completely clogged. As with the surface conveyance option, the runoff discharged from the developed site must not exceed historical runoff rates as set forth in Section 5.11, Detention.

5.4.4 FLOODPLAIN DEVELOPMENT PERMIT REQUIREMENTS

Article VI of the City Code gives general guidelines for floodplain development. Regardless of whether or not a development is within a FEMA-designated floodplain, all developments that convey channelized offsite flow (as opposed to sheet flow) and change the grades or cross-sections of that channel without enclosing it, must submit a floodplain development permit for approval. The floodplain development permit may require a floodplain modification study that includes a detailed topographic survey of the site having a minimum of 1-foot contours, a hydrologic study of the site comprised of a HEC-1 or HEC-HMS model and any field investigation notes, and a hydraulic model (HEC-2 or HEC-RAS) of the existing and proposed conveyance facilities. If the surface flows across the development have been studied and modeled by FEMA, the developer will not be required to complete a separate hydrologic model to determine flow rates. FEMA flows and water surface elevations shall be used in these cases for existing conditions. However, any modifications to the waterway will be hydraulically modeled and submitted to the City as part of the floodplain modification study, in addition to the modeling and submittals required by FEMA.

SECTION 5.5
STORM PRECIPITATION

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5.5.1 INTRODUCTION

Presented in this Section are design rainfall data for the minor and major storm events. These data are used to determine storm runoff peak flows and volumes in conjunction with the runoff models described in Section 5.6, Storm Runoff. All hydrologic analyses for Steamboat Springs shall utilize the rainfall data presented herein for calculating storm runoff.

5.5.2 RAINFALL ANALYSIS

For the City of Steamboat Springs, 24-hour point precipitation values are provided in **Table 5.5.1** for various recurrence interval storms. These data are from the NOAA Atlas 2, Volume III, 1973.

Table 5.5.1 24-Hour Point Rainfall Values for Steamboat Springs

Recurrence Interval	24-Hour Precipitation Depths (inches)
2-year	1.3
5-year	1.7
10-year	1.9
25-year	2.4
50-year	2.6
100-year	2.8

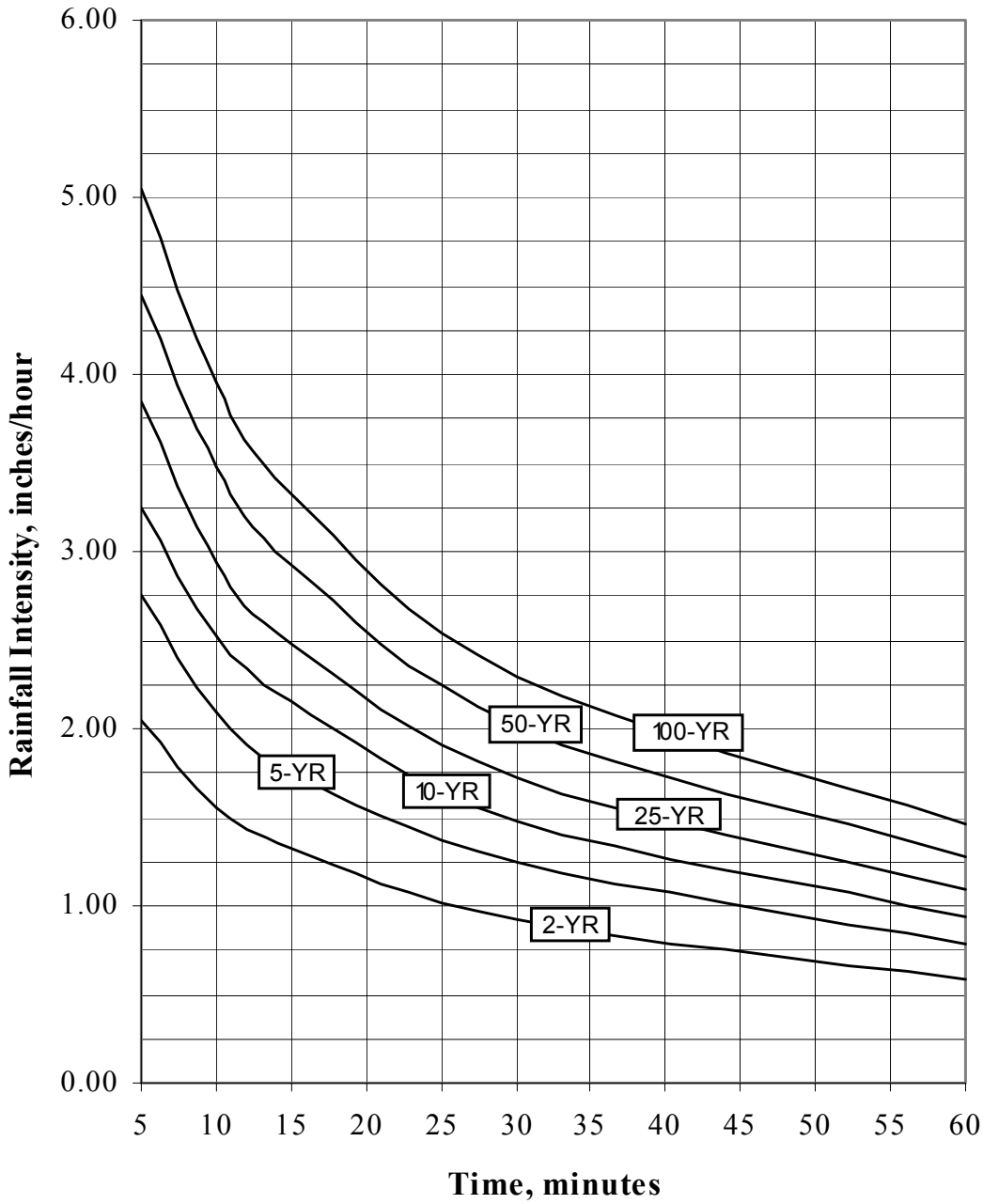
5.5.3 INTENSITY-DURATION-FREQUENCY CURVES FOR RATIONAL METHOD

Rainfall intensities as a function of storm duration and recurrence interval are provided in **Table 5.5.2** and **Figure 5.5.1**. These values were taken or derived from the values in the NOAA Atlas 2, Volume III, 1973.

Table 5.5.2 Intensity-Duration-Frequency Values

Storm Duration (min)	Precipitation Intensity – Steamboat Springs (inches/hour)					
	2-year Recurrence	5-year Recurrence	10-year Recurrence	25-year Recurrence	50-year Recurrence	100-year Recurrence
5	2.04	2.76	3.24	3.84	4.44	5.04
10	1.56	2.1	2.52	2.94	3.48	3.96
15	1.32	1.76	2.16	2.48	2.92	3.32
30	0.92	1.24	1.48	1.72	2.02	2.3
60	0.58	0.78	0.94	1.09	1.28	1.46

Figure 5.5.1 Intensity – Duration – Frequency Curves



5.5.4 DESIGN STORMS

For hydrograph analysis, the recommended minimum design storm duration is a 24-hour SCS Type II Distribution. Unless specifically noted otherwise, the major storm shall be the 100-year recurrence interval storm and the minor storm shall be the 5-year recurrence interval storm. If HEC-HMS is used, the designer shall choose the SCS Type II Distribution option and enter the point precipitation value given in **Table 5.5.1** for the required storm recurrence interval within the meteorologic model of the program. The designer is not required to consider evapo-transpiration or snowmelt in the meteorologic model, but if it is considered, a discussion of the methodologies chosen, parameters used, and rational for each shall be submitted as part of the required drainage studies for the site.

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5.6.1 INTRODUCTION

The City of Steamboat Springs recognizes two models that can be used to estimate storm runoff: the Rational Method and the U.S. Army Corps of Engineers' (USACE) Hydrologic Engineering Center (HEC) hydrologic computer programs (HEC-1 and HEC-HMS).

For certain circumstances, where adequate recorded stream flow data are available and the watershed area is greater than 10 square miles, a statistical analysis may be required to predict storm runoff peaks or for calibration of the model.

In the City of Steamboat, snowmelt can cause significant runoff. Statistical analysis of historical local data, however, shows that the intense rainstorms that can come in June, July, and August produce higher peak runoff than snowmelt. It is possible that rainfall *in combination with* snowmelt, such as a rain on snow event, may produce peak runoff in excess of the peaks from these storm events in the major rivers (i.e. the Yampa River), but not in the tributaries within the City of Steamboat Springs. Additionally lack of data for snowmelt and the complex interaction of snowmelt and rain make it difficult to model this scenario. Thus, for the purposes of these criteria, snowmelt runoff analysis is not required.

5.6.2 BASIC METHODOLOGY

Each of the accepted methods requires the user to determine the watershed time of concentration and watershed characteristics such as area, length of flow path, slope, and imperviousness. The HEC models also require calculation of the precipitation losses.

For watersheds less than 160 acres, the Rational Method shall be used. For watersheds larger than 160 acres, a HEC model shall be used. A 24-hour, SCS Type II Distribution of the design storm shall be analyzed.

5.6.2.1 Time of Concentration

The time of concentration, t_c, is the time required for runoff to flow from the most remote part of the watershed area to the point of interest. For the Rational Formula, the time of concentration is calculated so that the average rainfall rate for a corresponding duration can be determined from the rainfall intensity-duration-frequency curves.

For consistency between runoff analyses, the time of concentration equations presented in this Section shall be used for all small watershed (less than one square mile) runoff calculations.

Time of concentration consists of an initial time or overland flow time, t_i, plus travel time, t_t. In both urban and non-urban environments, the initial or overland flow is assumed to occur as sheet flow and as a function of surface type and slope, with an upper limit on the distance over which this type of flow can occur. Travel time, t_t, may be in a single element such as a swale or paved ditch, or it may be in a combination of many elements.

5.6.2.1.1 Non-Urbanized Watersheds

Non-urban watersheds shall be those that have less than 20% imperviousness at full development based on current zoning. In non-urban areas, the travel time will be in a small swale, channel, or wash, whereas in urban areas, the travel time will be in a storm drain, paved gutter, roadside drainage ditch, or drainage channel. Travel time can be estimated from the hydraulic properties of the storm drain, gutter, swale, ditch, or wash.

The time of concentration for both urban and non-urban areas is calculated as follows:

$$t_c = t_i + t_t \qquad (5.6.1)$$

Where:

t_c = Time of concentration (min)
t_i = Initial, Inlet, or overland flow time (min)
t_t = Travel time in the ditch, channel, gutter, storm drain, etc. (min)

The initial or overland flow time, t_i, may be calculated using the following equation:

$$t_i = 1.8\ (1.1 - K)\ L_o^{1/2} / S^{1/3} \qquad (5.6.2)$$

Where:

ti = Initial or Overland Flow Time (min)
K = Flow Resistance Coefficient
Lo = Length of Overland Flow, (ft, 300-ft maximum)
S = Average Watershed Slope (percent)

Equation 5.6.2 was originally developed for use with the Rational Formula method. The 5-year runoff coefficient, C_5, presented in **Table 5.6.1** is recommended for the flow resistance coefficient, K.

Table 5.6.1 Design Runoff Coefficients

Percentage Imperviousness	Runoff Coefficients					
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.04	0.15	0.25	0.37	0.44	0.50
5%	0.08	0.18	0.28	0.39	0.46	0.52
10%	0.11	0.21	0.30	0.41	0.47	0.53
15%	0.14	0.24	0.32	0.43	0.49	0.54
20%	0.17	0.26	0.34	0.44	0.50	0.55
25%	0.20	0.28	0.36	0.46	0.51	0.56
30%	0.22	0.30	0.38	0.47	0.52	0.57
35%	0.25	0.33	0.40	0.48	0.53	0.57
40%	0.28	0.35	0.42	0.50	0.54	0.58
45%	0.31	0.37	0.44	0.51	0.55	0.59
50%	0.34	0.40	0.46	0.53	0.57	0.60
55%	0.37	0.43	0.48	0.55	0.58	0.62
60%	0.41	0.46	0.51	0.57	0.60	0.63
65%	0.45	0.49	0.54	0.59	0.62	0.65
70%	0.49	0.53	0.57	0.62	0.65	0.68
75%	0.54	0.58	0.62	0.66	0.68	0.71
80%	0.60	0.63	0.66	0.70	0.72	0.74
85%	0.66	0.68	0.71	0.75	0.77	0.79
90%	0.73	0.75	0.77	0.80	0.82	0.83
95%	0.80	0.82	0.84	0.87	0.88	0.89
100%	0.89	0.90	0.92	0.94	0.95	0.96

The overland flow length, L_o, is generally defined as the length over which the flow characteristics appear as sheet flow or very shallow flow in broad, grassed swales. Changes in land slope, surface characteristics, and small drainage ditches or gullies will tend to force the overland flow into a combined flow condition, which results in higher flow velocities and shorter travel times. The initial flow time in both urban and non-urban areas shall be limited to the time to travel a distance of 300 feet

For watersheds longer than 300 feet, the travel time, t_t, must be added to the overland flow time. Travel time can be calculated using Manning's equation and the hydraulic properties of the storm drain, gutter, swale, ditch, or channel or can be approximated from Equation 5.6.3 and **Table 5.6.2**:

$$V = C_v S_w^{0.5} \qquad (5.6.3)$$

Where:

V = Velocity, fps
S_w = watercourse slope, ft/ft
C_v = Conveyance coefficient

The minimum conveyance coefficient, C_v, that shall be used for a developed site shall be 7.0, corresponding to short pasture and lawns.

Table 5.6.2 Travel Time Conveyance Coefficients

Land Surface	Conveyance Coefficient, C_v
Heavy meadow	2.5
Tillage/Field	5.0
Short pasture and lawns	7.0
Nearly bare ground	10
Grassed waterways	15
Paved areas and shallow swales	20

The time of concentration is then the sum of the initial flow time t_i and the travel time, t_t. The minimum recommended t_c for non-urban watersheds is 10 minutes.

5.6.2.1.2 Urbanized Watersheds

Overland flow in urbanized watersheds can occur from the back of the lot to the street, in parking lots, in landscape areas, or within park areas and can be calculated using the procedure described for non-urbanized watersheds. Travel time, t_t, to the first design point or inlet is often determined based on the conveyance coefficient for paved areas and shallow swales, but can be estimated using Manning's equation.

The time of concentration for the first design point in an urbanized watershed using this procedure should not exceed the time of concentration calculated using Equation 5.6.4, which was developed using rainfall/runoff data collected in urbanized regions (USDCM,1969).

$$t_c = L / 180 + 10 \qquad (5.6.4)$$

Where:

t_c = Time of Concentration at the first design point (min)
L = Watershed Length (ft)

Equation 5.6.4 may result in a lesser time of concentration at the first design point and thus would govern in an urbanized watershed. The recommended minimum t_c to the first urban design point is 5 minutes. For subsequent design points, the time of concentration is calculated by accumulating the travel times in downstream reaches.

5.6.2.2 Rational Method

For watersheds of less than 160 acres, the design storm runoff shall be calculated using the Rational Method.

The Rational method is based on the Rational Formula:

$$Q = CIA \qquad (5.6.5)$$

Where:

Q = Maximum rate of runoff in cubic feet per second (cfs)
C = Runoff coefficient
I = Average intensity of rainfall in inches per hour
A = Contributing watershed area in acres

5.6.2.2.1 Limitations on Methodology

The Rational Formula method adequately estimates the peak rate of runoff from a rainstorm in a given watershed, but it does not provide information on the full hydrograph and only approximates the runoff volume.

Because of the limitations of the Rational Method, the following guidelines on its application are provided:

- The individual sub-watershed sizes should not be greater than 20 acres.
- The aggregate of all sub-watershed areas should not be greater than 200 acres.
- The sub-watersheds should be reasonably homogeneous for existing and for projected land use.

5.6.2.2.2 Rainfall Intensity

The rainfall intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration. Rainfall intensity for use with the Rational Method shall be the intensity of the design storm having a duration equal to the time of concentration. Section 5.5, Storm Precipitation, discusses rainfall intensity in more detail.

5.6.2.2.3 Runoff Coefficient

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all which effect the time distribution and peak rate of runoff. Determination of the coefficient requires judgment and understanding on the part of the engineer.

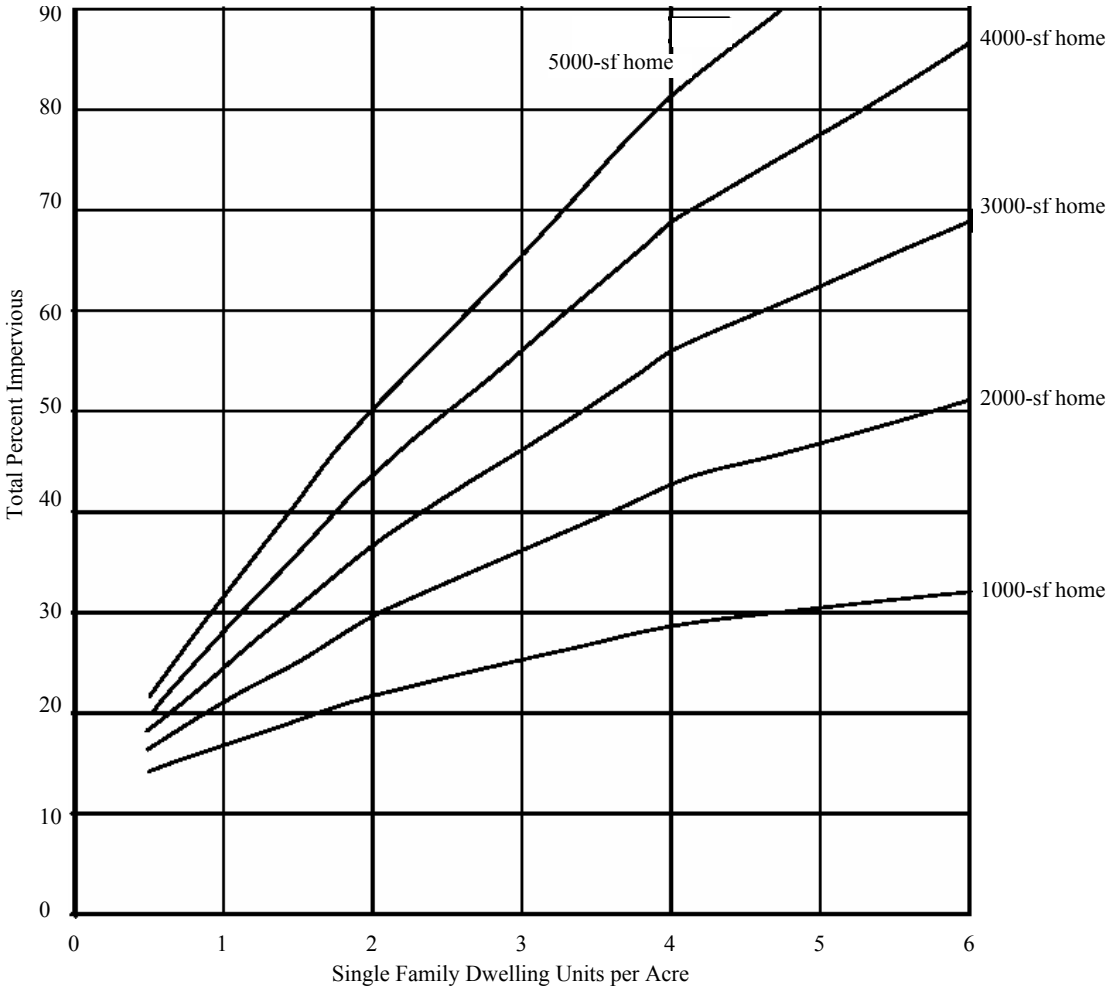
The first step in identifying the runoff coefficient is to determine the composite imperviousness of the watershed using recommended values in **Table 5.6.3** and **Figures 5.6.1** through **5.6.3**. Once the percentage impervious is determined, recommended C values for various recurrence interval storms are determined from **Table 5.6.1**.

Table 5.6.3 Recommended Imperviousness Values

Land Use or Surface Characteristic	Percent Impervious
Business	
Commercial Areas	95
Neighborhood Areas	85
Residential	
Single Family	(see figures)
Multi-Unit (detached)	60
Multi-Unit (attached)	75
Half-acre lot or larger	(see figures)
Apartments	80
Industrial	
Light industrial	80
Heavy industrial	90
Parks, cemeteries	5
Playgrounds	10
Schools	50
Railroad yards	15
Undeveloped Areas	
Historic Flow analysis	2
Greenbelts, agriculture	2
Off-site flow analysis (when land use not defined)	45
Streets	
Paved (concrete/asphalt)	100
Gravel	40
Drives and walks	90
Roofs	90
Lawns (all soils)	0

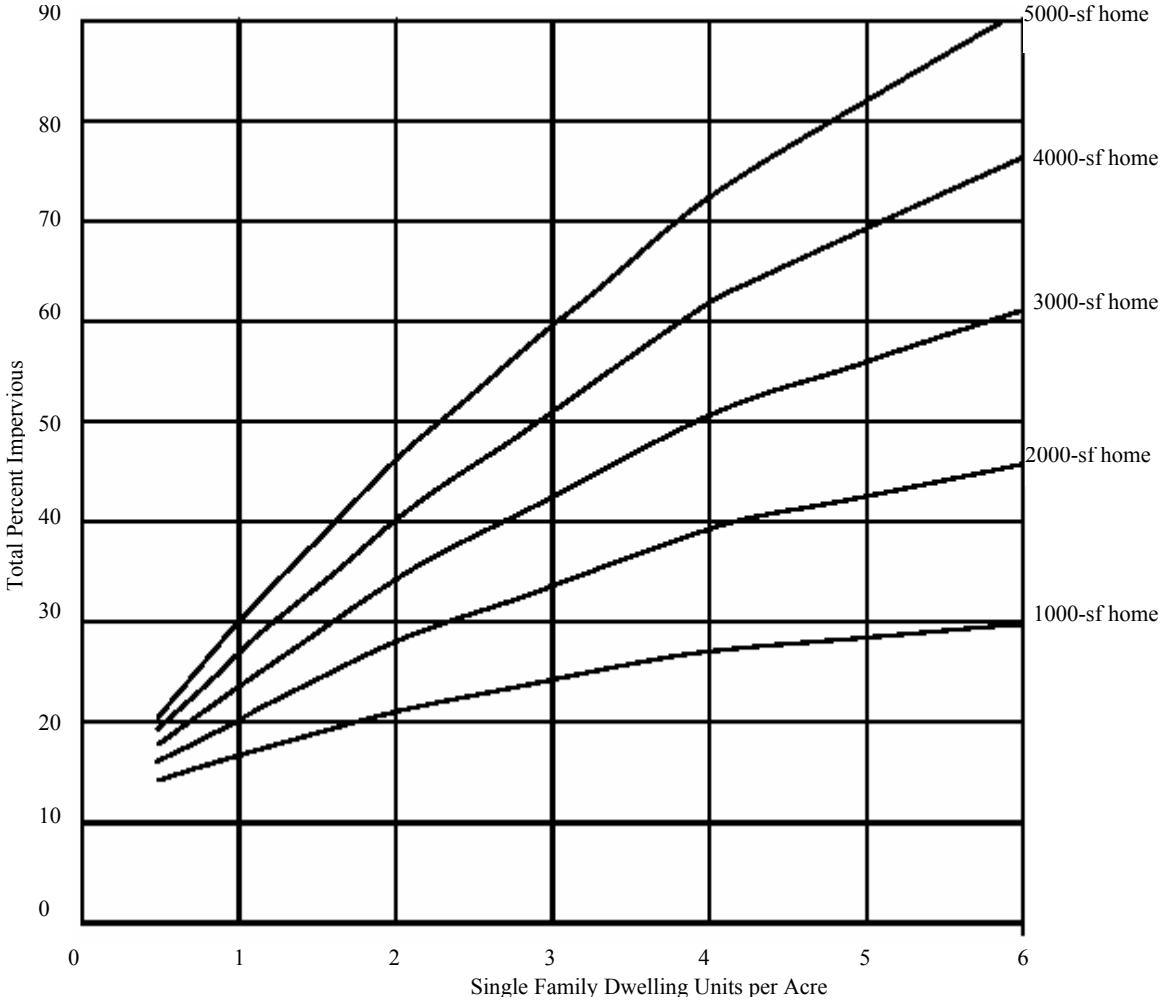
Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 1.

Figure 5.6.1 Watershed Imperviousness for Single-Family Residential Ranch Houses



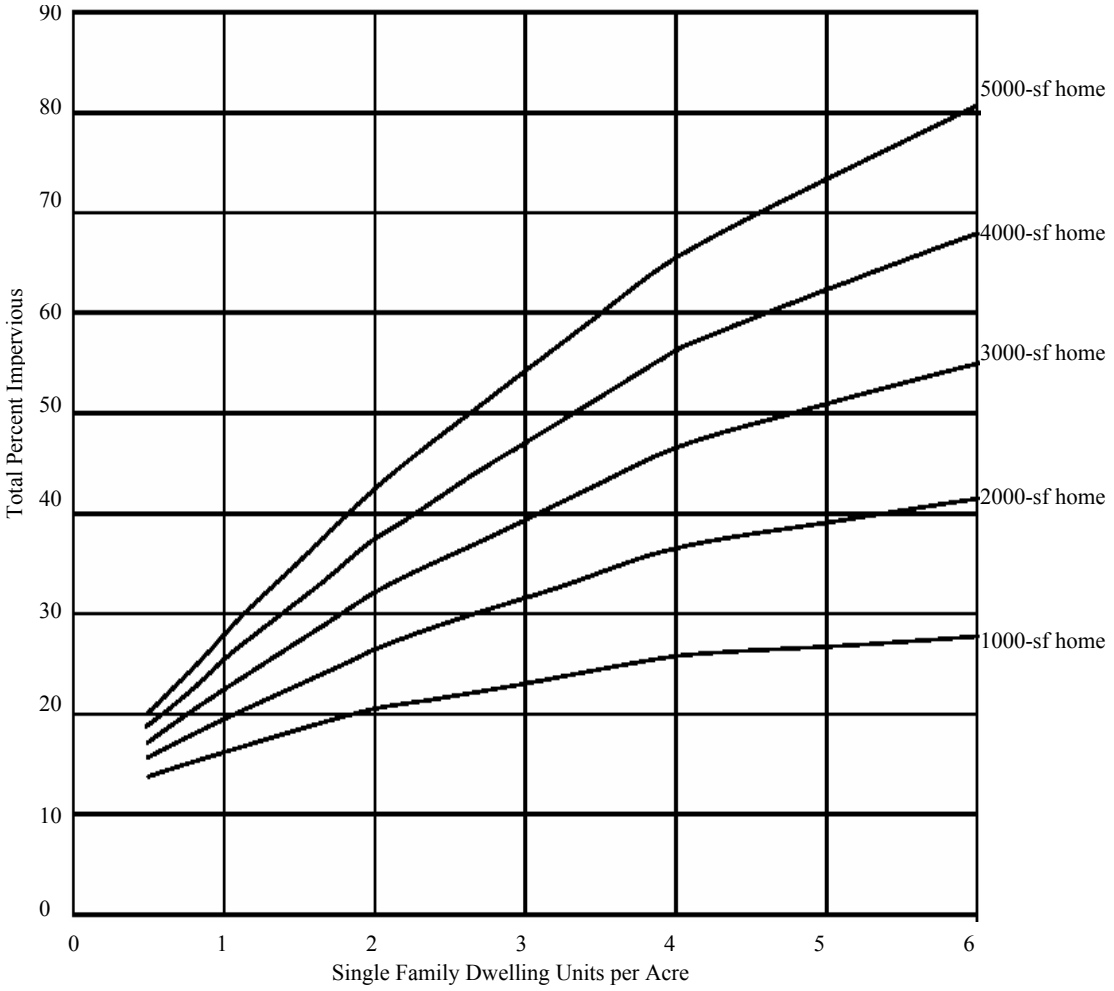
Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 1.

Figure 5.6.2 Watershed Imperviousness, Single-Family Residential Split-Level Houses



Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 1.

Figure 5.6.3 Watershed Imperviousness for Single-Family Residential Two Story Houses



Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 1.

5.6.2.3 **HEC Models**

Over the years, the USACE HEC has developed models designed to simulate various hydrologic and hydraulic processes. Chronologically, the HEC-1 Flood Hydrograph Package was the first hydrologic model developed to process all ordinary flood hydrograph computations associated with a single recorded or hypothetical storm. Its successor, HEC-HMS (Hydrologic Modeling System), is designed to simulate the precipitation-runoff processes of branching watershed systems. It is designed to be applicable in a wide range of geographic areas for modeling the widest possible range of hydrologic conditions. This includes large river basin water supply and flood hydrology, and small urban or natural watershed runoff.

Either program is acceptable for use in the City of Steamboat Springs. The designer is referred to the HEC-1 and HEC-HMS User's Manuals for additional guidance. The following subsections offer guidance for determining some of the inputs to the HEC programs.

5.6.2.3.1 CN Determination

If the SCS Method is specified for use in the basin model portion of the HEC-HMS model, a soil-cover curve number (CN) is used for computing excess precipitation. The curve number CN is related to hydrologic soil group (A, B, C, or D), land use, treatment class (cover), and antecedent moisture condition.

The soil group is determined from published soil maps for the area, which correlate each soil name with the soil group. Land use and treatment class are determined during field visits or from aerial photographs. Procedures for determining land use and treatment class are found in Chapter 8 of the National Engineering Handbook, Section 4 (SCS, 1985). An antecedent moisture condition II (AMC-II) is recommended for the City of Steamboat Springs.

Having determined the soil group, land use and treatment class, and the antecedent moisture condition, CN values can be determined from **Table 5.6.4** below. Additionally, for undeveloped areas, **Table 5.6.5** may be used if the site meets any of the more specific cover types listed there. In watersheds with varying land use, a composite CN may also be calculated directly from imperviousness estimates using the following equation.

$$CN = 98*Imp + X*(1-Imp) \qquad (5.6.6)$$

Where:

Imp = Imperviousness as a decimal
X = Adjustment factor based on NRCS Soil Type

NRCS Soil Type	Adjustment Factor
A	39
B	61
C	74
D	80

5.6.2.3.2 Losses

Once the curve number is determined, precipitation loss can be determined by first calculating the soil moisture storage deficit and then the initial abstraction using the equations below. The HEC-HMS model will also calculate loss and accumulated runoff when the initial abstraction is entered into the loss tab when using the SCS methodology.

$$Q = (P - IA)^2 / ((P - IA) + S) \qquad (5.6.7)$$

$$S = (1{,}000 / CN) – 10 \qquad (5.6.8)$$

$$IA = 0.2\ S \qquad (5.6.9)$$

Where:

Q = Accumulated Excess (in)
P = Accumulated Rainfall Depth (in)
IA = Initial abstraction (in)
S = Currently Available Soil Moisture Storage Deficit (in)
CN = SCS Curve number

Since this method results in total excess for a storm, the incremental excess (the difference between rainfall and precipitation loss) for a time period is computed as the difference between the accumulated excess at the end of the current period and the accumulated excess at the end of the previous period.

Table 5.6.4 Runoff Curve Numbers

Land Use or Surface Characteristic	Average Impervious (%)	Runoff Curve Number Soil Complex			
		A	B	C	D
Business					
Commercial Areas	95	95	96	97	97
Neighborhood Areas	85	89	92	94	95
Residential					
Single Family (see note)	(see note)				
Multi-unit (detached)	60	74	83	88	91
Multi-unit (attached)	75	83	89	92	94
Apartments	80	86	91	93	94
Industrial					
Light	80	86	91	93	94
Heavy	90	92	94	96	96
Parks, cemeteries	5	42	63	75	81
Playgrounds	10	45	63	75	81
Schools	50	69	80	86	89
Railroad yards	15	48	67	78	83
Irrigated Areas					
Lawns, parks, golf course	0	39	61	74	80
Agriculture	0	39	61	74	80
Undeveloped Areas					
Pre-development conditions	2	40	62	74	80
Greenbelts, agriculture	2	40	62	74	80
Off-site analysis when land use unknown	45	66	78	85	88
Outcrops	70	80	87		94
Streets/Roads					
Paved	100	98	98	98	98
Gravel	40	63	76	84	87
Drives/Walks	90	92	94	96	96
Roofs	90	92	94	96	96

Note: Estimate imperviousness from Figures 5.6.1, 5.6.2, and 5.6.3. Then compute the Curve Number from Equation 5.6.6 based on soil type. Values are from SCS Technical Release No. 55 (1986) with minor revisions for local conditions.

Table 5.6.5 Additional Runoff Curve Numbers for Arid and Semiarid Rangelands

Cover Type	Hydrologic Condition	Runoff Curve Number Soil Complex A	B	C	D
Herbaceous – mixture of grass, weeds and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen – mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper – pinyon, juniper, or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sage-grass – sage with an understory of grass	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub – major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, paloverde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

Reference: USDA NRCS, 1997. Part 630 Hydrology, National Engineering Handbook

5.6.2.3.3 Sub-watershed Sizing

The determination of the peak rate of runoff at a given design point is affected by the number of sub-watersheds within a larger watershed. Typically, the more sub-watersheds that are used to define a larger watershed, the more representative the resulting peak flow is of actual runoff conditions. The improved predictive capability of multiple sub-watersheds is due to better homogeny of the sub-watershed characteristics, as compared to analysis of the watershed with no sub-watersheds. Recommended guidelines are:

- For watersheds up to 100 acres in size, the maximum sub-watershed size should be approximately 20 acres.
- For watersheds over 100 acres in size, increasingly larger sub-watersheds may be used as long as the land use and surface characteristics within each sub-watershed are homogeneous. In addition, the sub-watershed sizing should be consistent with the level of detail needed to determine peak flow rates at various design points within a given watershed.

5.6.2.3.4 Parameters

For the basin portion of the HEC-HMS model, the designer shall use SCS methodology unless site conditions specifically indicate some other method. A brief discussion shall be submitted as part of the required drainage studies indicating the various methodologies and parameters that were utilized in the basin model including, but not limited to, loss, transform, baseflow, imperviousness, curve numbers, and initial abstraction.

5.6.3 FEMA FLOWS

The Federal Emergency Management Agency (FEMA) has completed full hydrologic models of several of the waterways within the City of Steamboat Springs. These include the Yampa River, Spring Creek, Butcherknife Creek, Soda Creek, Walton Creek and an unnamed tributary, Fish Creek and an unnamed tributary, and Burgess Creek and an unnamed tributary. **Figure 5.6.4** indicates these streams. Where FEMA hydrologic studies have been completed, the flow rates and water surface elevations for each of the return periods studied shall be used for design of improvements including site grading and layout as well as channel improvements.

5.6.4 CHANNEL ROUTING OF HYDROGRAPHS

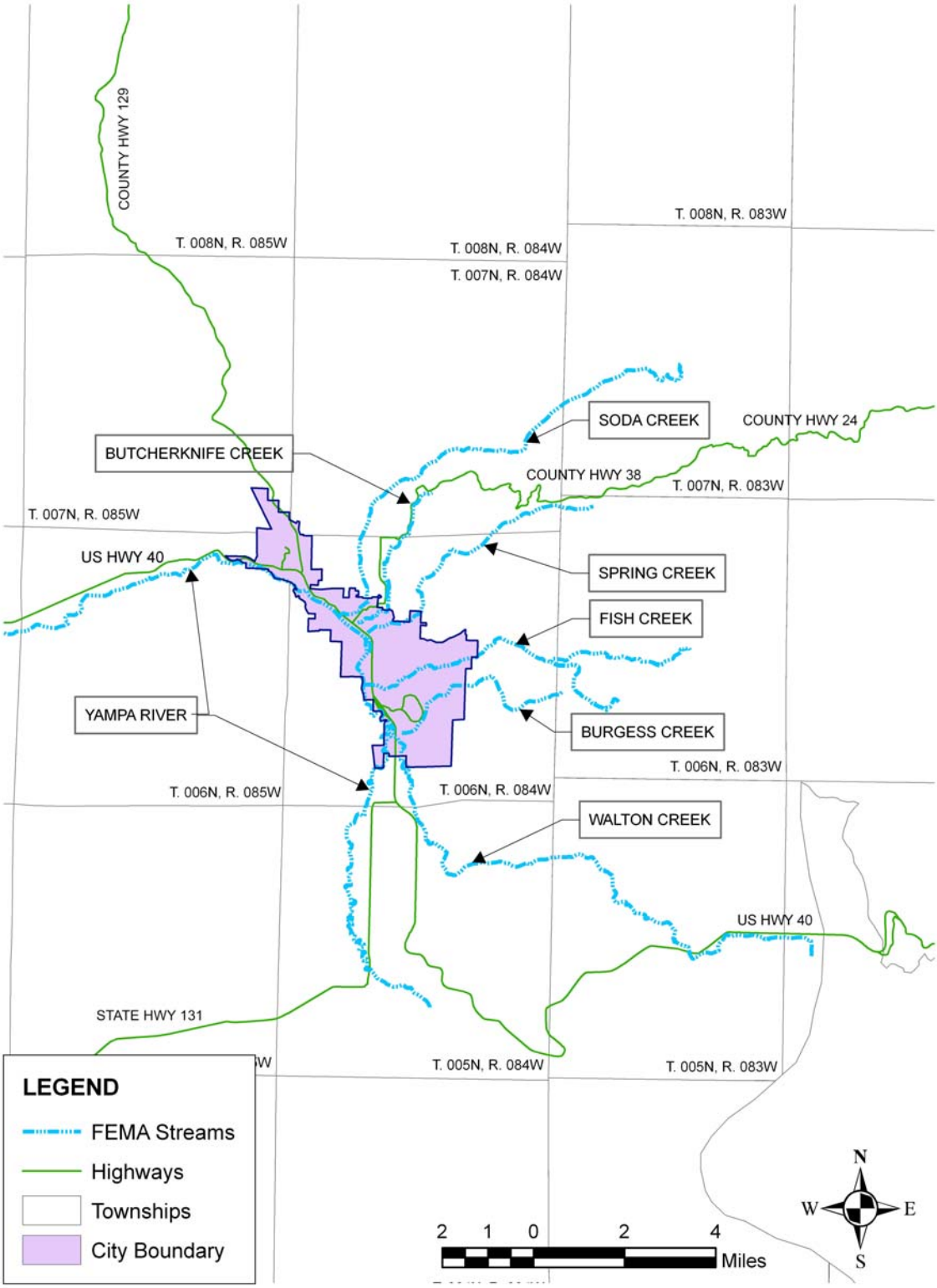
When a large or non-homogeneous watershed is being investigated, it should be divided into smaller and more homogeneous sub-watersheds. The storm hydrograph for each sub-watershed is then routed through the channel and combined with individual sub-watershed hydrographs to develop a storm hydrograph for the entire watershed. HEC-1 or HEC-HMS is recommended when channel routing of hydrographs is required.

Within the HEC programs, the Kinematic Wave method is recommended when the watershed has well defined channels, and the Muskingum-Cunge method is recommended for poorly defined channels that have cross sections that can be determined from detailed topography. Otherwise the Muskingum method is recommended.

5.6.5 RESERVOIR ROUTING OF HYDROGRAPHS

In some instances where detention is required, the sizing of the detention storage will be based upon hydrograph storage routing techniques rather than direct calculation of volume and discharge requirements. HEC-1 or HEC-HMS is recommended when reservoir routing of hydrographs is conducted. Information on when hydrograph routing is appropriate can be found in Section 5.11.7, Hydrologic Design Methods and Criteria.

Figure 5.6.4 Local FEMA Studies



SECTION 5.7
OPEN CHANNELS

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5.7.1 INTRODUCTION

Any water that is conveyed in such a manner that top surface is exposed to the atmosphere is defined as open channel flow. This type of flow occurs in all channel types including streams, rivers, canals, ditches, and drainage channels. This Section discusses all types of open channel flow, including roadside ditches, and gives general guidance for designing improvements to channels. For a more thorough discussion of open channel design principles, the user is encouraged to review Section 3 of Chapter 7, Major Drainage, of the *Urban Storm Drainage Criteria Manual* (USDCM) by the Urban Drainage and Flood Control District (UDFCD), 2006.

5.7.2 FLOW CHARACTERISTICS

The hydraulics of an open channel can be very complex, encompassing many different flow conditions from steady-state uniform flow to unsteady, rapidly varying flow. Most of the problems in storm water drainage involve uniform, gradually varying or rapidly varying flow states. Steady uniform flow, in which the depth of flow remains constant over the time interval studied, is the most commonly analyzed flow condition in open channel hydraulics. The calculations for uniform and gradually varying flow are relatively straight forward. Rapidly varying flow computations such as hydraulic jumps and flow over spillways, however, can be very complex, and the solutions are generally empirical in nature. This Section will only discuss uniform, gradually varied flow computations. For rapidly varying flow conditions, the designer is encouraged to review the many hydraulics textbooks written on this subject.

5.7.2.1 Normal Flow

Open channel flow is 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 depth is referred to as the “normal depth.” For uniform flow within a prismatic channel (i.e., uniform cross section), the water surface will be parallel to the channel bottom. Although uniform flow rarely occurs in nature and is difficult to achieve in a laboratory, a uniform-flow approximation is considered appropriate for planning and design purposes because of the straight forward calculation.

The computation of uniform flow and normal depth shall be based upon the Manning or Uniform Flow Equation:

$$Q = \frac{1.49}{n} A^{5/3} P^{-2/3} \sqrt{S} = \frac{1.49}{n} A R^{2/3} \sqrt{S} \qquad (5.7.1)$$

Where:

- Q = flow rate (ft^3/s)
- n = Manning roughness coefficient
- A = area (ft^2)
- P = wetted perimeter (ft)
- R = hydraulic radius = A/P (ft)
- S = slope of the energy grade line (ft/ft)

For prismatic channels, the energy grade line, hydraulic grade line, and the bottom can be assumed parallel for uniform, normal depth flow conditions. **Table 5.7.1** provides a list of Manning roughness coefficient values for many types of conditions.

Table 5.7.1 Manning's Roughness Coefficients

TYPE OF CHANNEL AND DESCRIPTION	Roughness Coefficients
EXCAVATED OR DREDGED	
Earth, straight and uniform	
Clean, recently completed	.018
Clean, after weathering	.022
Gravel, uniform section, clean	.025
With short grass, few weeds	.027
Earth, winding and sluggish	
No vegetation	.025
Grass, some weeds	.030
Dense weeds or aquatic plats in deep channels	.035
Earth bottom and rubble sides	.030
Stony bottom and weedy banks	.035
Cobble bottom and clean sides	.40
Dragline-excavated or dredged	
No vegetation	.035
Light brush on banks	.040
Rock cuts	
Smooth and uniform	.035
Jagged and irregular	.040
Channels not maintained, weeds and brush	
Dense weeds, high as flow depth	.080
Clean bottom, brush on sides	.050
Same as above, but highest state of flow	.070
Dense brush, high state	.100
LINED OR BUILT-UP CHANNELS	
Concrete	
Trowel Finish	.013
Float Finish	.015
Gunite, good section	.019
Gunite, wavy section	.022
Concrete Bottom	
Dressed stone in mortar	.017
Random stone in mortar	.020
Dry rubble or riprap	.030
Gravel bottom with sides of	
Formed concrete	.020
Random stone in mortar	.023
Dry rubble or riprap	.033
Asphalt	
Smooth	.013
Rough	.016
Grassed	.04

Reference: Chow, V.T., Open Channel Hydraulics, 1959

5.7.2.2 Gradually Varied Flow Computation

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm drain inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel, and the water surface profile can be computed using either the direct-step or standard step method. The Direct-Step Method is best suited to the analysis of simple prismatic channels, and the Standard-Step Method is required for the analysis of irregular or non-uniform cross-sections. The most general and widely used programs are the U.S. Army Corps of Engineers' HEC-2 and HEC-RAS programs. The design engineer may use these programs or proprietary computer software specifically approved by the Public Works Department to compute water surface profiles for channel and floodplain analyses.

5.7.2.3 Critical Flow Computation

Critical flow through a channel is characterized by several important conditions regarding the relationships between flow, specific energy, and slope of a particular hydraulic cross-section. The Froude Number (Fr) is a measurement used to identify when flow becomes critical. Flow is critical when the Froude Number is equal to 1.0.

Typically, channels must not be designed to flow at or near critical state ($0.80 < Fr < 1.2$) because flow is unstable in this range. Within this range, factors causing only minor changes in specific energy, such as channel debris or minor variation in roughness, will cause a major change in depth.

The Froude Number (Fr) is defined as follows:

$$Fr = \frac{v}{\sqrt{gD_h}} \qquad (5.7.2)$$

Where:

- Fr = Froude number (dimensionless)
- v = velocity (ft/s)
- g = gravitational acceleration (32.2 ft/s^2)
- A = channel flow area (ft^2)
- T = top width of flow area (ft)
- D_h = hydraulic depth, D_h=A/T (ft)

5.7.3 <u>TYPES OF OPEN CHANNELS</u>

There are many options available to convey surface water in an open channel. These range from existing natural channels to concrete rectangular channels. Open channels can be categorized as either natural or engineered. Natural channels include all watercourses that are carved and shaped by erosion and sedimentation. Engineered channels are those constructed by human efforts. Of the six different types of open channels described in the following subsections, grass-lined channels are preferred within the City of Steamboat Springs. Other channel types may be considered on a case-by-case basis based on site conditions and flow characteristics.

5.7.3.1 Natural Channels

Natural channels are carved and shaped by natural erosion processes before urbanization occurs. As the channel's tributary watershed urbanizes, natural channels often experience erosion and may need grade control checks and localized bank protection to stabilize. Natural channels are strongly influenced by urbanization. If watershed imperviousness exceeds around 10%, it is likely that a natural channel is no longer viable and mitigation measures will be required, such as bank and bed stabilization measures.

5.7.3.2 Grass-Lined Channels

Grass-lined channels are the most desirable of all the types of constructed or modified drainageways. They provide channel storage, lower velocities, groundwater recharge, and various multiple use benefits. Low flow areas may need to be concrete, rock-lined, or otherwise reinforced with vegetation to minimize erosion and maintenance problems. Turf reinforcing mats may be considered in private developments only.

5.7.3.3 Wetland Vegetation Bottom Channels

Wetland vegetation bottom channels are grass-lined channels that are designed to encourage the development of wetlands in the channel bottom. These channels offer potential benefits that may include wildlife habitat, groundwater recharge and water quality enhancement. In low-flow areas, the banks may need supplemental reinforcement to protect against undermining.

5.7.3.4 Concrete-Lined Channels

Concrete-lined channels are high velocity artificial drainageways that are not encouraged. However, in retrofit situations where existing flooding problems need to be solved and where right-of-way is limited, concrete channels may be appropriate. Special attention should be taken to provide safety measures around the concrete-lined channels. Concrete channels are not permitted for use in the City without specific approval by the Public Works Department.

5.7.3.5 Riprap-Lined Channels

Riprap-lined channels offer a compromise between a grass-lined channel and a concrete-lined channel. They can reduce right-of-way needs as compared to grass-lined channels and avoid the higher costs of concrete-lined channels. Riprap-lined uniform channels are not encouraged. Note, however, that riprap for use at culvert outlets is encouraged and shall be as specified in Section 5.10, Culverts and Bridges. If riprap is used, it shall be a soil-riprap mix of at least 30% soil, mixed prior to placement, and seeded or otherwise vegetated in accordance with the City's Standard Specifications.

5.7.3.6 Other Channel Linings

Additional channel liners are also available including gabion, interlocked concrete blocks, concrete revetment mats formed by injecting concrete into double layer fabric forms, and various types of synthetic fiber liners. As with rock and concrete liners, all of these types are best considered for helping to solve existing urban flooding problems and are not recommended for new developments. Each type of liner has to be scrutinized for its merits, applicability, how it meets other community needs, its long term integrity, and maintenance

needs and costs. Channels lined with artificial materials, except as noted above, are not permitted in new development areas.

5.7.4 CHANNEL SELECTION

Each type of channel must be evaluated for its longevity, integrity, maintenance requirements and costs, and general suitability for community needs, among other factors. Selection of a channel type that is most appropriate for the conditions that exist at a project site shall be based on a multi-disciplinary evaluation, which may include hydraulic, structural, environmental, sociological, maintenance, economic, and regulatory factors. In the City of Steamboat Springs, natural-looking channels, such as grass-lined channels, are far preferred to channel improvements that drastically change the look, shape, lining, alignment, or flow characteristics of the existing channel. The use of concrete-lined and riprap-lined channels is discouraged. In the event an entirely new channel is required, such as through a new development, it should closely mimic similarly-sized natural channels in the surrounding area whenever possible. In the event a harder channel lining such as concrete or riprap is required, the designer shall consult with the Public Works Department to arrive at solution that is acceptable to the City.

To the maximum extent possible, any channel improvements should strive to maintain the existing flow rate and alignment of an existing open channel. This Section presents general design standards that apply to all improved channels.

The following multi-disciplinary factors should be used when selecting the channel that is most suitable for a specific site.

Table 5.7.2 Channel Selection Factors

Hydraulic Factors	Structural Factors	Environmental Factors	Sociological Factors	Maintenance Factors	Regulatory Factors
Topography	Cost	Wildlife habitat	Pedestrian traffic	Life expectancy	Federal
Capacity needed	Shear stress	Neighborhood character	Neighborhood social patterns	Maintainability	State
Slope of thalweg	Momentum transfer	Wetland mitigation	Recreational needs	Accessibility	Local
Basin sediment yield	Seepage and uplift forces	Street and traffic patterns	Neighborhood population and age group	Repair and reconstruction	Right-of-way
Ability to drain adjacent lands	Area to waste excess material	Neighborhood aesthetic requirements		Proven performance	Municipal or county policies
	Pressures and pressure fluctuations	Water quality enhancement			
	Availability of material	Need for new green areas			

5.7.5 GENERAL DESIGN CRITERIA FOR IMPROVED OPEN CHANNELS

With the following exceptions, all open channel improvements shall be designed in accordance with the latest versions of Chapter 7, Major Drainage, and Chapter 8, Hydraulic Structures, of the *Urban Storm Drainage Criteria Manual* by the UDFCD. For each type of channel specified above, Chapter 7 discusses design velocity and Froude number, flow depth, longitudinal slope, curvature, bottom width, side slopes, riprap sizing, and freeboard requirements. Chapter 8 discusses energy dissipation structures such as grouted sloping boulder drop structures, baffled chute drops, and impact stilling basins should these be required in the City.

All open channels within the City of Steamboat Springs shall be designed to convey water in a subcritical flow condition (Fr<0.8). All open channels shall be designed with public safety in mind and adequate maintenance access shall be provided.

5.7.5.1 Manning Roughness Coefficients

Table 5.7.1 shows recommended values for the Manning roughness coefficient for various channel types and conditions. The values were taken from *Open Channel Hydraulics* by Chow, 1959. Manning roughness coefficients for riprap channels shall be calculated as:

$$n = 0.0395\, d_{50}^{1/6} \qquad (5.7.3)$$

Where:

n = Manning's roughness coefficient
d_{50} = the mean stone size in feet

5.7.5.2 Channel Velocity

The various channel linings are only stable up to certain velocities. Channel design should consider reducing the potential for erosion and may require a decrease in slope, change in channel bottom material, or the addition of revetment. **Table 5.7.3** presents the major storm maximum permissible velocity for common channel linings. Erosive soils include loams, sands, and noncolloidal silts. Less erosive soils include clays, shales, cobbles, and gravel.

Table 5.7.3 Maximum Permissible Mean Channel Velocity

Channel Lining	Maximum 100-Year Velocity (fps)
Grass in Erosive Soils	5.0
Grass in Less Erosive Soils	7.0
Cobble in Erosive Soils	5.0
Cobble in Less Erosive Soils	7.0
Angular Riprap	15.0
Semi-Angular Riprap	12.0
Grouted Riprap	15.0
Gabions	15.0
Soil Cement	15.0
Concrete	20.0

5.7.5.3 High Gradient Channels

In the City of Steamboat Springs, 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 with 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.

The following equation may be used to as an aid in predicting the roughness coefficient of a high-gradient channel provided the channel certain criteria are met.

$$n = 0.39\ S_f^{0.38}\ R^{-0.16} \qquad (5.7.4)$$

Where:

n = Manning's roughness coefficient
S_f = channel friction slope, ft/ft
R = hydraulic radius (A/P), feet

The following limitations on use of the above equation apply and are basic guidelines for when it should be used.

1. The channel must be a natural main 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.

In each case the major storm shall not result in a Froude number greater than 0.80, a flow depth greater than 5.0 feet, or less than 1.0 feet of freeboard at any point along the channel reach.

Although overall slopes of natural channels can be very steep in mountainous areas, channels often times have achieved these high average grades by cutting very steep drops along what otherwise are flatter channel reaches. The analysis of a natural mountain stream requires a careful topographical investigation. In constructing the hydraulic model of a natural channel, it is important to recognize that friction slope and hydraulic radius, and the *n* value as a consequence, can change frequently. The hydraulic model should 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 very easily cause erosion of the channel and require potentially costly remediation. The guidance in Section 5.11, Detention,

specifies that no new development shall increase peak runoff from the area it occupies because of the erosive damage that allowing this could cause, among other adverse effects.

If site conditions suggest use of Equation 5.7.4 might be appropriate, the designer shall consult with the Public Works Department to confirm its applicability and discuss any additional specific site concerns regarding the stability of the natural channel. If the average slope of an existing natural channel though 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.

5.7.6 ROADSIDE DITCHES

Roadside ditches shall be grass-lined and shall be designed so that flow velocities do not cause erosion of the ditch lining. Maximum longitudinal channel slopes shall be dictated by a maximum allowable Froude number of 0.80 and a maximum allowable velocity of 7 feet per second. Velocities shall be estimated using Manning's equation. For grass-lined channels, Manning's *n* value as well as velocity and capacity calculations shall be based on the Soil Conservation Service (SCS) Retardance Curve C shown in **Figure 5.7.1**. The Froude number for roadside ditches shall be calculated as follows:

$$Fr = \frac{V}{\left(\frac{gA}{T}\right)^{0.5}} = \frac{V}{\left(gD_h\right)^{0.5}} \qquad (5.7.5)$$

Where:

Fr = the Froude number

V = average ditch velocity, fps

A = cross sectional area of flow, ft^2

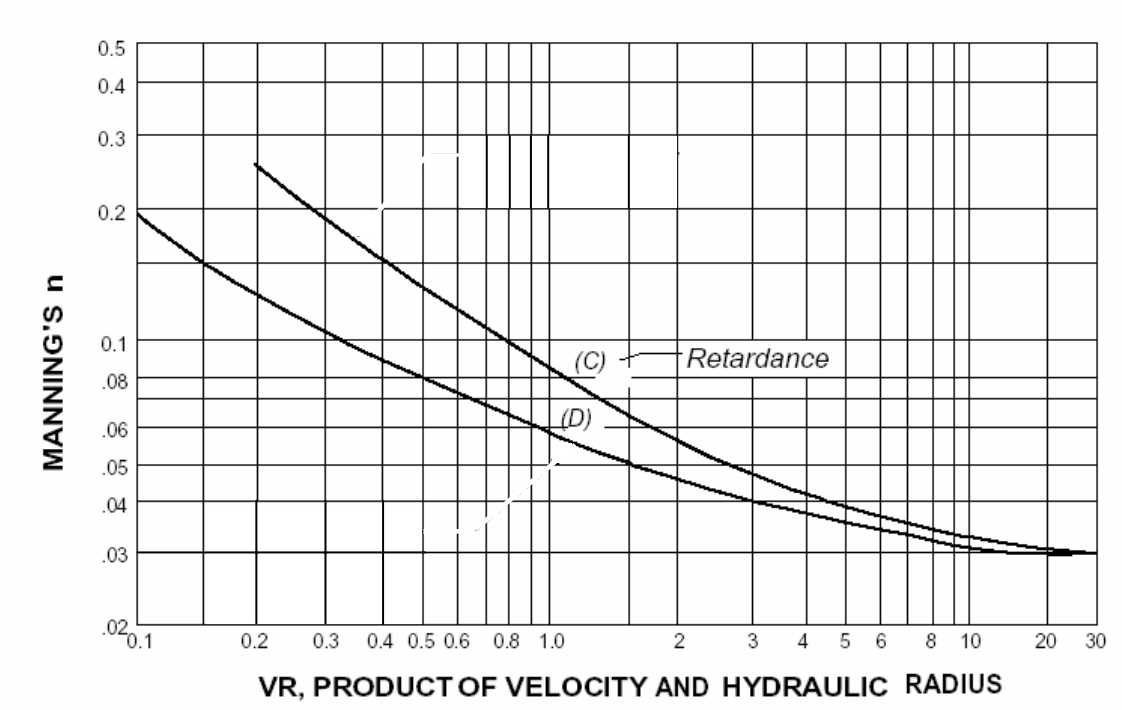
T = width of the channel at the surface of flow, ft

D_h = hydraulic depth, ft

g = gravitational constant, 32.2 ft/s^2

Note that using **Figure 5.7.1** requires a trial-and-error approach, first assuming an *n* value and then calculating the various parameters repeatedly until the intersection of VR (the product of the velocity and the hydraulic radius) and the Froude number falls on the specified SCS Retardance Curve.

Figure 5.7.1 SCS Retardance Curves



Alternately, the UDFCD has developed a spreadsheet that will calculate a number of parameters given a user-specified ditch geometry, flow depth, and Retardance Curve. The spreadsheet is titled UD-Channels and is available on the UDFCD website under Technical Downloads. The user should use the "Rating" worksheet.

The following criteria also apply:

1. For arterial and collector roadways, the major storm shall not encroach upon any drive lane. **Figure 5.7.2** is a schematic of this.
2. For local roadways, the major storm shall not inundate the outside edge of the outside drive lane by more than 6". **Figure 5.7.3** is a schematic of this requirement.
3. Side slopes of roadside ditches shall be no steeper than 2H:1V.
4. No roadside ditch shall have a flow depth greater than 3 feet.
5. A minimum velocity of 2.0 fps is required to discourage sediment build-up.
6. Residences shall be no less than 12 inches above the major storm water surface elevation at the ground line or at the lowest point of entry.

Figure 5.7.2 Collector and Arterial Inundation Limits

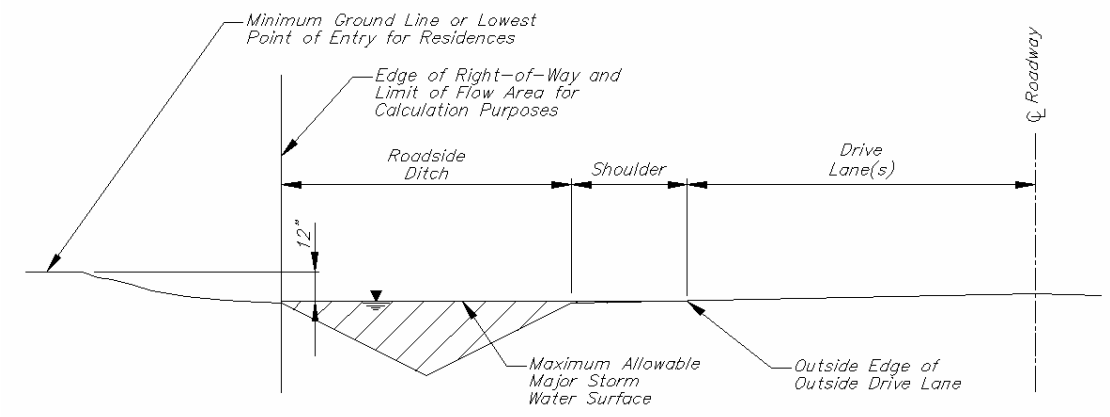
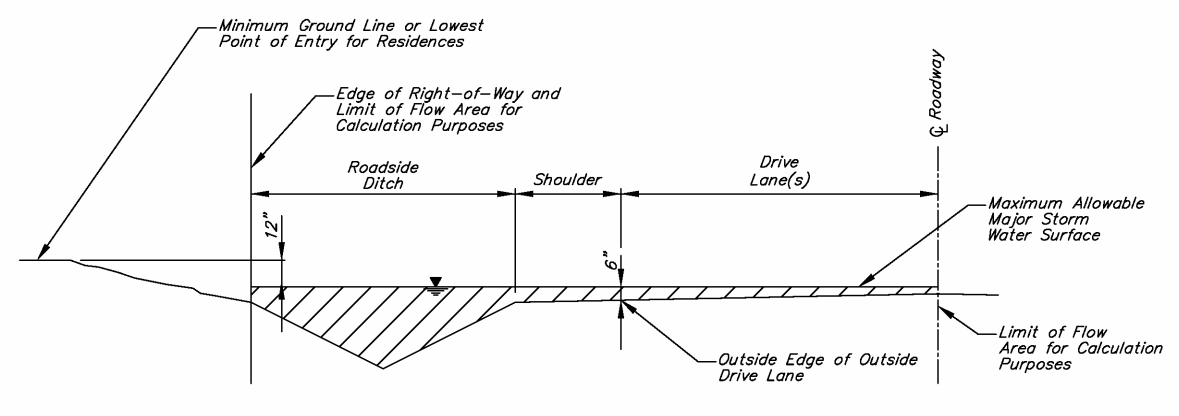


Figure 5.7.3 Local Roadway Inundation Limits



5.7.6.1 Ditch Checks

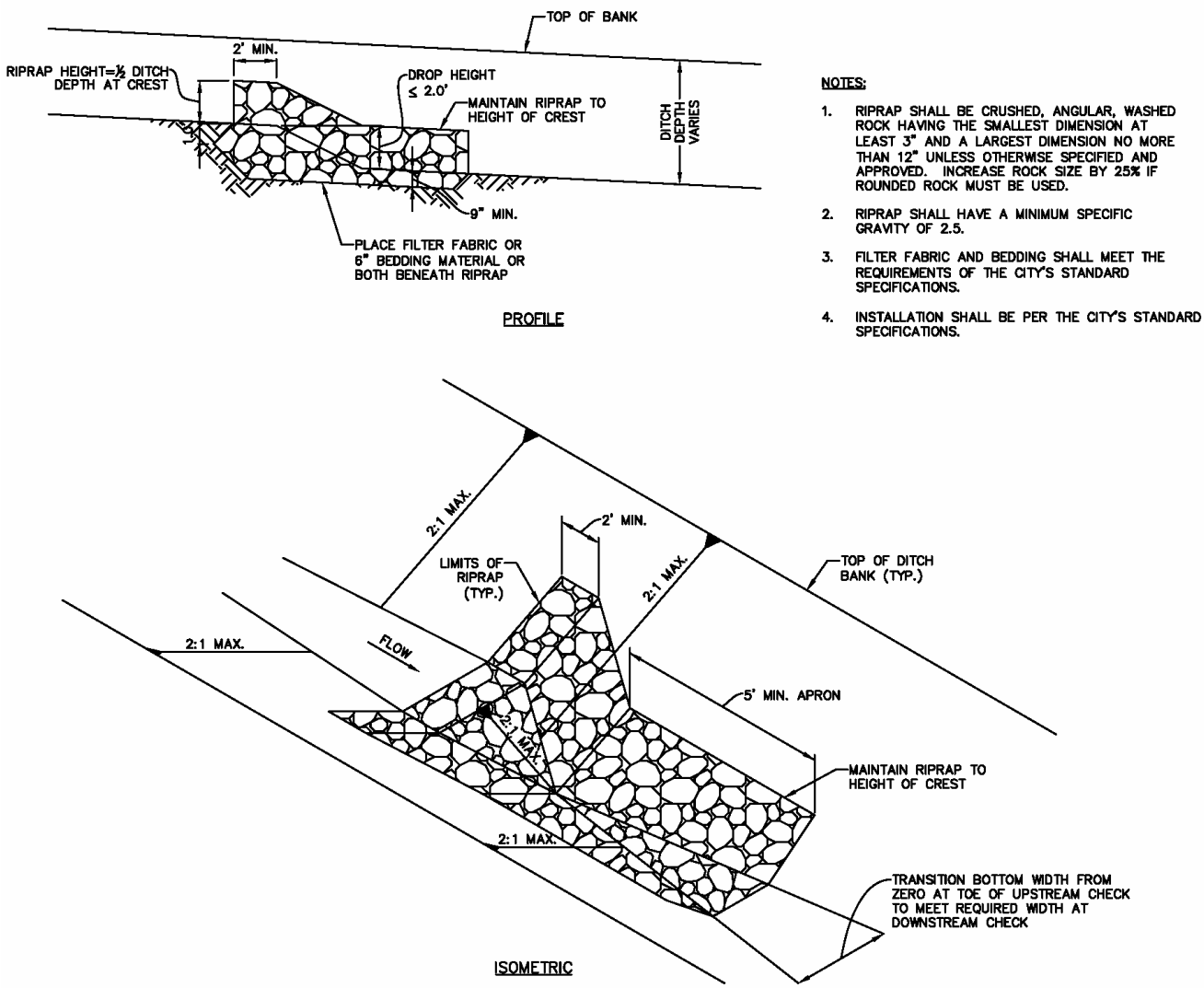
Due to the natural topography of the area, it is not unusual for developments within the City of Steamboat Springs to have relatively steep roadside ditch slopes. When ditch slopes are steeper than 3%, the potential for erosion is especially high. Ditch checks provide a method by which to slow down ditch flow velocities and prevent unnecessary erosion. Ditch checks are required where the Froude number exceeds 0.80 or the velocity exceeds 7 feet per second.

Ditch checks act like miniature drop structures. They allow for flattening the channel slope to achieve the required Froude number and velocity values. The upstream side of each ditch check shall be buried while the downstream side will allow for no more than a 2-foot drop at a slope of 2H:1V. Each drop will be required to have a riprap apron extending a minimum of five feet downstream of the toe of the ditch check. Ditch checks shall be installed longitudinally at the interval required to meet Froude number and velocity requirements for the design storm.

The ditch cross section at the downstream toe of each ditch check will be a standard v-ditch. At this location, the bottom of the ditch has no width. The ditch width will transition from zero at this point to four times the height of the next ditch check at the crest of the next ditch check (assuming the ditch side slopes are 2H:1V) In this fashion, the side slopes of the roadside

ditch will remain constant through each of the ditch checks. **Figure 5.7.4** gives a detail of a typical ditch check.

Figure 5.7.4 Ditch Check Schematic Details



5.7.6.2 Ditch Stabilization

When a new roadway is cut through a development or an existing roadway is widened or otherwise disturbed, the adjacent roadside ditch is typically completely bare, at least initially. Although it may be seeded very promptly upon completion of construction, it will remain vulnerable to erosion until a good stand of grass is actually established. To prevent roadside ditch erosion during this sensitive period of time, ditches shall be stabilized in accordance with the project Specifications.

SECTION 5.8
STREETS AND ROADSIDE CONVEYANCE

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5.8.1 INTRODUCTION

The drainage system in the City of Steamboat Springs is generally intended to be an open channel system. Section 5.7, Open Channels, discusses the design of roadside ditches for this purpose. In some areas of town such as the downtown business district, central park area, and on private roads within private developments, however, it is desirable to have an enclosed drainage system that typically includes curb and gutter, curb inlets, and storm sewers. This Section presents criteria for storm drainage flows on public streets having curb and gutter as well as the selection and placement of storm drain inlets. Section 5.9, Storm Drain Systems, discusses storm drain design criteria. Any new curb and gutter must be approved by the Public Works Department if it is proposed outside the downtown business district or central park area, and the design must consider not only drainage, but plowing and maintenance requirements as well.

5.8.2 STREETS AS PART OF THE DRAINAGE SYSTEM

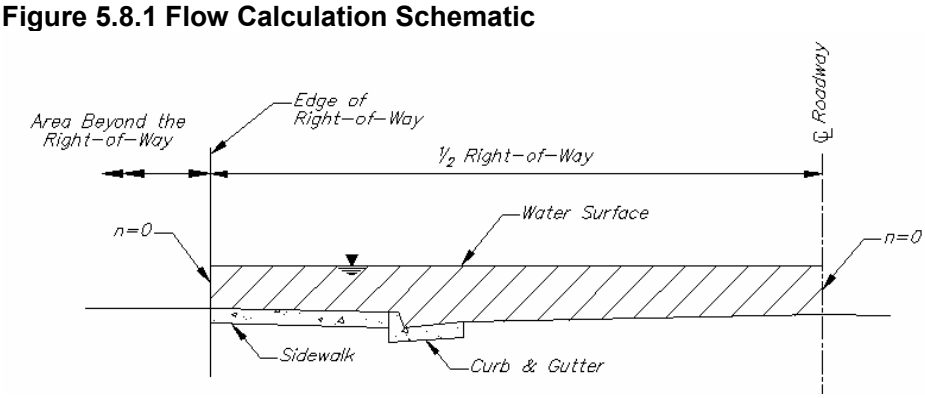
The primary function of public streets is the movement of traffic, and use of streets as part of the drainage system must be limited to prevent interference with traffic. Street inundation limits are specified in this Section in order to limit this interference.

Streets typically convey runoff collected on the street surface itself 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 maximum allowable capacity of a street is based upon its cross-sectional geometry, longitudinal slope, and the maximum allowed depth of runoff. Where the minor storm event exceeds the maximum depth, inlets must be used to reduce street flow. During a major storm event, streets may become emergency runoff channels, routing floodwaters away from structures. During such an event, many streets will be inundated to the point they are impassable to most vehicles.

5.8.3 ALLOWABLE FLOW DEPTH, SPREAD, AND VELOCITY

Calculations for flow capacity and velocity in a given street section are based upon the limits specified for each type of roadway and the assumption that area outside the street right-of-way does not contribute to the capacity of the street system. For calculation purposes, it is assumed that an infinitely high vertical wall of zero roughness exists at the right-of-way boundary, and any flow area outside this boundary is not considered in analysis. Due to the potential for a single street cross-section to have different half-street cross-sections, all street capacity calculations are to be completed on a half-street basis. Therefore, the same vertical-wall assumption applies to the street centerline as to the right-of-way where the calculated flow width exceeds the half-street width. **Figure 5.8.1** illustrates this concept.

Figure 5.8.1 Flow Calculation Schematic



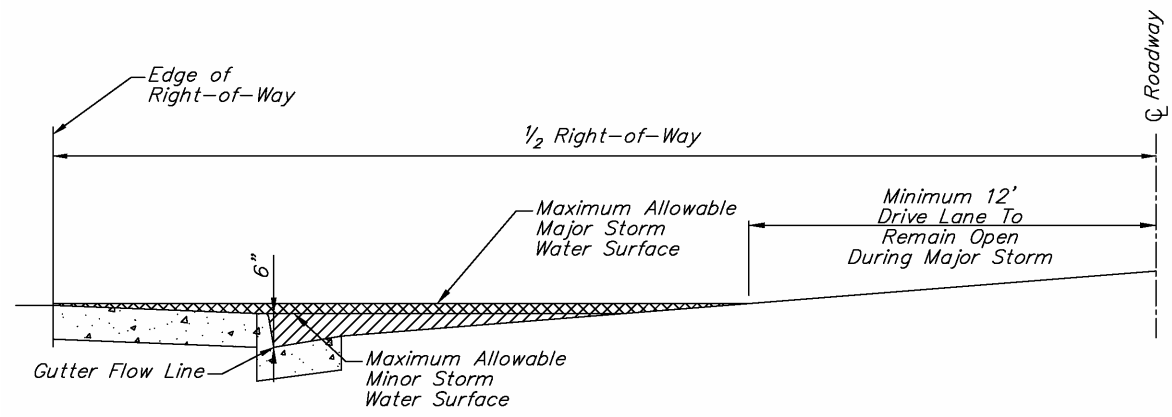
At sump locations, pipes and/or channels must be provided to facilitate compliance with maximum limits as described above. Maintenance access must be provided for these facilities, including easements where outside the right-of-way.

When a roadway has an enclosed drainage system, the system is typically designed to carry the minor storm. As such, the roadway itself must be designed to fully carry the difference between the minor storm flows and the major storm flows.

5.8.3.1 Collector and Arterial Streets

The enclosed drainage system on a collector or arterial street shall be designed so that the minor storm flow depth in the street does not exceed 6 inches at the gutter flow line. The system shall also ensure that at least one 12-foot lane of traffic remains open in each direction during the major storm event. Velocity shall be less than 8 feet per second. Each residence shall have both its ground line and lowest point of entry no less than 24 inches above the gutter flow line. Where existing buildings are not 24 inches (for residential buildings) or 12 inches (for commercial buildings) above the gutter flow line, major storm flow depth shall be limited to 6 inches. **Figure 5.8.2** is a schematic diagram of the allowable flow on collector and arterial streets not having a contingency caused by existing buildings.

Figure 5.8.2 Collector and Arterial Inundation Limits



5.8.3.2 Local Streets

The enclosed drainage system on a local street shall be designed so that the minor storm flow depth in the street does not exceed 6 inches at the gutter flow line. The system shall also ensure that the major storm flow depth does not exceed 12 inches at the gutter flow line. Velocity shall be less than 8 feet per second. Each residence shall have both its ground line and lowest point of entry no less than 24 inches above the gutter flow line. Where existing buildings are not 24 inches (for residential buildings) or 12 inches (for commercial buildings) above the gutter flow line, major storm flow depth shall be limited to 6 inches.

5.8.4 STREET HYDRAULIC CAPACITY EVALUATION

Gutter and street flow are assumed to be uniform for the purpose of hydraulic evaluation and design, but as street flow depth increases, flow width increases at a much faster rate. This wide, relatively shallow flow has the effect of decreasing the hydraulic radius, rendering the standard Manning's equation somewhat inaccurate. The following equation shall be used instead:

$$Q_S = \frac{K_u}{n} S_X^{5/3} S_L^{1/2} T_S^{8/3} \qquad (5.8.1)$$

Where:

Q_S = Street Flow Capacity Not Including Gutter (cfs)

n = Manning's Roughness Coefficient

$K_u = 0.56$

S_X = Street Cross Slope

S_L = Street Longitudinal Slope

T_S = Flow Top Width Not Including Gutter (ft)

For streets with a single cross slope for the gutter and street section, the above equation will suffice for determining total capacity if the T_S term is modified to include the gutter. However, when the gutter has a steeper cross slope than the street, the above equation specifies capacity in the flow area between the edge of pavement (not including the gutter itself) and the edge of flow. A Manning's roughness value of *n* = 0.016 should be used.

Where flow stays within the gutter section, the standard Manning's equation is used:

$$Q = \frac{1.49}{n} A \cdot R^{2/3} \cdot S_L^{1/2} \qquad (5.8.2)$$

Where:

Q = Flow Capacity (cfs)

S_L = Street Longitudinal Slope

R = Hydraulic Radius (ft) = A/P

A = Cross – Sectional Flow Area (sf)

P = Wetted Perimeter (ft)

For streets with differing slopes in the gutter section and the street section, Equation 5.8.3 shall be used. **Figure 5.8.3** is a graphic showing some of the variables used.

$$Q = \frac{Q_s}{1-E_o} \qquad (5.8.3)$$

Where:

Q = Total Flow Capacity (cfs)

Q_s = Flow Outside the Depressed Gutter Section (cfs)

$$E_o = \frac{1}{1 + \dfrac{S_w / S_x}{\left[1 + \dfrac{S_w / S_x}{(T/W) - 1}\right]^{8/3} - 1}}$$

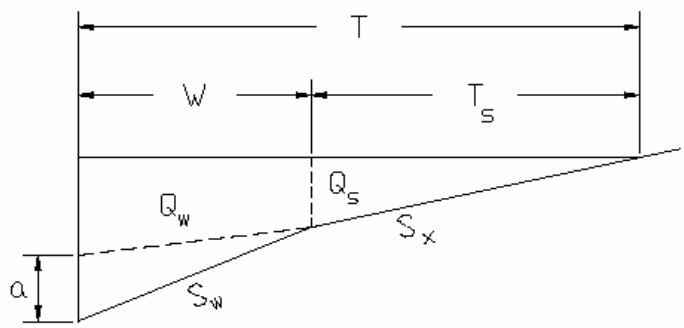
S_w = Gutter Cross Slope $= S_x + \dfrac{a}{W}$

a = Gutter Depression (feet)

W = Width of Gutter (feet)

T = Total Top Width of Flow Including Gutter (feet)

Figure 5.8.3 Gutter Section with Composite Cross Slope



The user should first determine flow outside the gutter section using Equation 5.8.1, then calculate E_o, and finally compute total flow in the composite street and gutter section.

The maximum allowable gutter velocity is eight feet per second. Velocities exceeding this value can create safety issues, cause erosive damage to the street and other surfaces, and reduce the effectiveness of storm drain inlets.

Alternately, the Urban Drainage and Flood Control District (UDFCD) has developed an excel spreadsheet that will calculate street hydraulic capacity given detailed user input. The spreadsheet is titled UD-Inlet and is available on the UDFCD website under Technical Downloads. The website should be checked to ensure the most recent version of UD-Inlet is being used as the UDFCD often updates its technical materials as new data becomes available.

5.8.5 STORM INLET SELECTION, SIZING, AND LOCATION

Wherever storm flow in the street exceeds allowable flow spread, velocity, or depth, some or all of the flow must be intercepted by a storm drainage inlet. The standard street inlet permitted for use in the City of Steamboat Springs is the Denver Type 16 Combination grated

inlet with a bicycle safe grate. Each inlet shall be constructed with a 2-foot sump below the lowest pipe invert elevation to allow for collection and removal of sediment and debris that can accumulate in storm sewers. A detail of this inlet for single, double, and triple configurations is found at http://www.denvergov.org/Portals/526/documents/StandardDetailDrawings.pdf. The formed lean fill shown in the bottom of inlet in the detail is not required for inlets constructed with the 2-foot sump. Area inlets approved for public use include CDOT Type C, Type D, and Type 13 inlets.

Inlets may be located on a continuous grade where flow not intercepted by the inlet will pass to another location. Or they may also be located in the sag portion of a street's vertical alignment or at any other sump location such as in a parking lot or unpaved area. Computation of inlet capacity involves several factors including type of inlet, location, grate type, inlet geometry, flow width and depth, and longitudinal and cross slopes.

Due to the winter climate in Steamboat Springs, inlets may experience temporary clogging due to snow and ice accumulation. Because of the variability of the freeze-thaw cycle, it is difficult to quantify the effects of snow and ice on the inlets. The standard clogging factors provide some design contingency and no additional capacity factors beyond those required herein for clogging are required to address winter conditions.

5.8.5.1 Hydraulic Capacity of Inlets on a Continuous Grade

The amount of flow an inlet intercepts is affected by different factors for different inlet types. Grate inlet capacity is affected by the amount of water flowing over the grate, the gutter flow velocity, and blockage due to debris. Curb inlet capacity varies primarily with inlet length, depth of flow, and longitudinal and cross slopes of the gutter and street. Combination inlets have essentially the same interception capacity as grate inlets standing alone; however, the curb opening portion of a combination inlet provides much greater debris-handling capability than a grate inlet has on its own. As specified above, the Denver Type 16 Combination grated inlet with a bicycle safe grate shall be used.

UD-Inlet, the spreadsheet developed by the UDFCD and mentioned earlier in this Section, will calculate hydraulic capacity of an inlet on grade given detailed geometric input. The spreadsheet is available on the UDFCD website and shall be used to calculate the hydraulic capacity of an inlet on grade. Any carryover flow calculated at an inlet on grade shall be added to the design discharge at the next inlet. Note also that inlets on grade should be designed to capture between 70 and 80 percent of the design discharge.

5.8.5.2 Hydraulic Capacity of Inlets in Sump Conditions

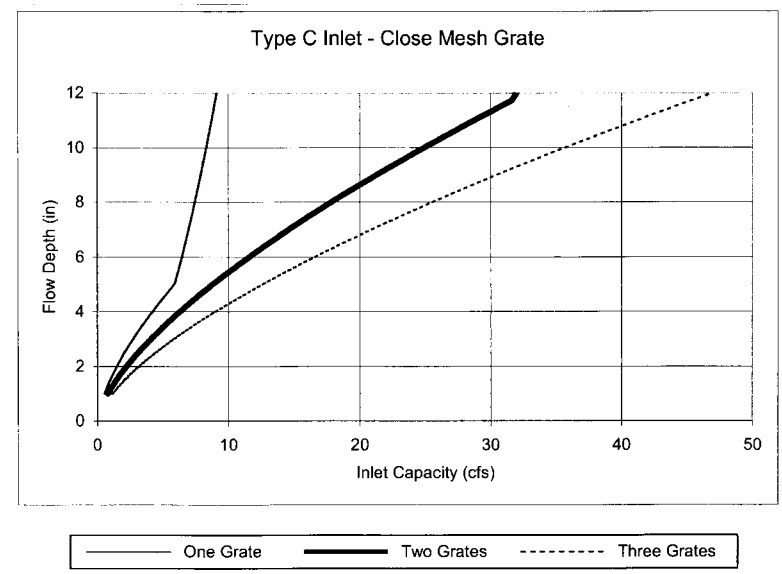
Street inlets in sump conditions must have the capacity to capture all of the runoff draining to the sump without exceeding maximum allowable ponding depths. To ensure maximum allowable ponding depth is not exceeded, a secondary flow path must be provided in the case of inlet failure. The preferred secondary flow path is a designated emergency overflow weir and channel located within an accessible drainage easement. It must be protected from erosive effects by pavement or riprap. If no easement is available at the inlet location, flanker inlets must be installed in the same gutter on each side of the primary inlet. Flanker inlets are located upgradient 10 to 50 feet from the primary sump inlet. The two flanker inlets shall have a combined design capacity equal to or greater than that of the primary inlet.

UD-Inlet, the spreadsheet developed by the UDFCD and mentioned earlier in this Section, will calculate hydraulic capacity of a street inlet in a sump condition given detailed geometric input. The spreadsheet is available on the UDFCD website and shall be used to calculate the

hydraulic capacity of an inlet in a sump condition. Where the option exists, the user shall accept the default values for clogging factors and for orifice and weir coefficients unless site conditions specifically dictate the use of different values.

The capacity of Type C and Type D area inlets with close mesh grates in a sump condition shall be determined from **Figure 5.8.4**. This figure gives the capacity of a Type C inlet with a close mesh grate and includes a 75% reduction factor. The capacity of a Type D inlet shall be equal to the capacity of a Type C inlet with two grates. The capacity of a Type 13 area inlet with a valley grate shall be determined from the manufacturer's inlet capacity curve.

Figure 5.8.4 Type C and D Area Inlet Capacity



Reference: Douglas County Storm Drainage and Technical Criteria Manual

5.8.5.3 Grate Selection

The City of Steamboat Springs requires that a bicycle-safe grate be used in all paved areas that may receive pedestrian or bicycle traffic unless specifically approved by the Public Works Department. The types of grates permitted for use with the Type 16 Combination grated inlet are vane grates and valley grates in single, double, and triple-inlet configurations. Vane grates, however, shall not be used in sump conditions.

Due to variances in nomenclature among various casting facilities, the designer should note that a "Type 16 Combination" inlet may sometimes imply the use of a vane grate. When a combination inlet is to be installed with a valley grate, this is sometimes designated as a "Type 13 Combination" inlet depending on the manufacturer. The designer should consult with the manufacturer to ensure compliance with these criteria and not unconditionally specify a "Type 16 Combination" inlet. The typical vane grate for use with a Type 16 Combination inlet is Neenah/Deeter Foundry #2502L or East Jordan Ironworks #7567M. The typical valley grate to be used with a Type 16 Combination inlet is Neenah/Deeter Foundry #2502A or East Jordan Ironworks #7567M2. In public sump areas not in a roadway, such as a parking lot or unpaved open area, the CDOT close mesh grate may be used with the Type C and Type D area inlets and Neenah/Deeter Foundry #2501-A or East Jordan Ironworks #7567M2 may be used with the Type 13 area inlet.

5.8.5.4 Inlet Location and Spacing

In streets, inlets should be placed at any location where water may encroach on street traffic beyond the allowable limits. At no time shall inlets be located within a curb ramp, but an inlet shall be located within 50 feet upstream of all curb ramps. Inlets shall be located to prevent bypass flows from the minor storm from crossing any street, although minor storm flows shall be allowed to cross alleys. During the major storm, flow depth across any street shall be limited to 6 inches at the gutter flow line. Valley pans shall not be allowed on public streets.

Additional street inlet locations shall be determined using the following iterative process:

1. Determine a preliminary location for the inlet based on street configuration and estimated runoff to the gutter.
2. If the inlet is in a sump, location is essentially fixed during the remainder of the design process. The inlet should be sized to maintain water depth and spread within the limits set by this Manual. If the required inlet size becomes excessively large, the designer is urged to install additional inlets upgradient from the sump.
3. For inlets on a grade, the designer must find the flow characteristics at the selected preliminary inlet location to determine whether the inlet needs to be placed further upstream or may be moved downstream based on maximum allowable parameters.
4. The designer should take into account the change in tributary area to the inlet associated with any upstream or downstream movement.
5. A typical design interception efficiency of an on-grade inlet is 70 to 80 percent. As mentioned previously, on-grade inlets designed to capture 100 percent of runoff tend to be significantly less effective both hydraulically and economically.
6. The designer should include any carryover or bypass flow from an upstream inlet when calculating the flow at a downstream inlet. Although the peak runoff to an inlet may not coincide with the peak carryover flow from an upstream inlet, these two peak flows shall be added to find the total peak flow to the downstream inlet.
7. Maximizing the use of sump inlets tends to increase the overall efficiency of the inlet system, and inlets must be installed at all street sags and at all sumps formed by intersections except where other drainage provisions have been made. Therefore, it is suggested that sump inlets are located prior to the placement of any on-grade inlets during the design process.

When incorporating sumps into paved areas such as parking lots or unpaved open spaces, they should be configured so that ponding at the sump inlets does not exceed 12 inches during the minor storm. Buildings shall be no less than 12 inches above the major storm ponding depth at the ground line or at the lowest point of entry.

SECTION 5.9
STORM DRAIN SYSTEMS

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5.9.1 INTRODUCTION

Storm drains are used to convey runoff in locations where streets exceed their capacity or are otherwise unable to drain. Runoff is typically introduced into a storm drain via a street inlet, discussed in Section 5.8, Streets and Roadside Conveyance. However, water may also enter the system via grated area inlets or culvert inlets. The design of a storm drain system is dependant on topography, street rights-of-way and drainage easements, the need to convey flows from multiple locations, existing and proposed structures and utilities, outfall locations, local hydrology, and design criteria.

Typically, storm drains are sized to convey peak runoff from the minor storm in excess of street flow capacity. This means the upper end of a storm drain system will usually be located at the first inlet encountered by runoff in a given watershed. As discussed in Section 5.8, Streets and Roadside Conveyance, the first inlet will either be located where runoff first exceeds street capacity or where there is a vertical sag in the street.

Occasionally, inlets and storm drains must be sized to convey the entire major storm event flow. Two examples of this situation are:

1. Locations where street flow is not in the desired direction and there is no other feasible drainage solution (such as closed basins).

2. Locations where the standard allowable major storm street capacities do not apply, such as negative slopes outside the curb but within the right-of-way.

Peak runoff values are found using the methods set forth in Section 5.6, Storm Runoff.

5.9.2 STORM DRAIN DESIGN CRITERIA

This Section presents certain parameters relating to the design and construction of storm drain systems in the City of Steamboat Springs. Storm drain systems shall be sized for the minor storm event. All criteria and guidelines below apply to the minor storm event unless site conditions offer no viable overflow option for the major storm event.

5.9.2.1 Allowable Capacity

A storm drain shall be designed to convey all the design storm runoff from areas tributary to it. The design of surcharged storm pipes is not allowed for the minor storm. Methodology for the calculation of the energy grade line (EGL) and the hydraulic grade line (HGL), indicating all hydraulic losses due to friction, junctions, and other structures and phenomenon is included in this Section. The minor storm HGL shall at no time or location exceed finished grade.

For the purpose of completing a conceptual storm drain system design, calculation of an EGL and HGL is not required. In these cases, the initial design methods presented at the end of this Section are considered sufficient.

5.9.2.2 Allowable Velocity

Minimum velocities are required in storm drains to reduce sedimentation and promote positive drainage through the pipe at all depths. A minimum design-flow velocity of 2 feet per second is required for all public and private storm drains.

Public and private storm drains shall have a maximum design-flow velocity of 10 feet per second. Velocity may be increased to 15 feet per second, but the required gage thickness of corrugated metal pipes must be increased by one increment and the wall thickness of reinforced concrete pipes must increased from Wall B to Wall C in these instances. See the City's Standard Specifications for required pipe gage and thickness. Note that maximum outfall velocities are more restrictive to protect those areas from extensive erosion. See Sections 5.7, Open Channels, and 5.10, Culverts and Bridges, for details.

5.9.2.3 Pipe Roughness

Table 5.9.1 provides a range of Manning's *n* values for many pipe materials and configurations. For capacity calculations, hydraulic roughness shall be the largest Manning's *n* value in the provided range. The designer may choose to use a higher Manning's *n* value if conditions warrant.

Table 5.9.1 Manning's Roughness Coefficients for Storm Drain Conduits

Type of Conduit (see note)	Interior Wall Description	Manning's n
Concrete Pipes	Smooth	0.011-0.013
Concrete Boxes		
Wood forms	Smooth	0.012.-0.014
Steel forms	Smooth	0.012-0.013
Spiral-Rib Metal Pipes	Smooth	0.012-0.013
Corrugated Metal Pipes & Boxes		
Annular Corrugations	68mm x 13mm (2-2/3" x ½ ") corrugations	0.022-0.027
Helical Corrugations	68mm x 13mm (2-2/3" x ½") corrugations	0.011-0.023
	150mm x 25 mm (6" x 1") corrugations	0.022-0.025
	125mm x 25mm (5" x 1") corrugations	0.025-0.026
	75mm x 25mm (3" x 1") corrugations	0.027-0.028
Structural Plate Corrugations	230mm x 64mm (9" x 2 ½") corrugations	0.033-0.037
	150mm x 50mm (6" x 2") corrugations	0.033-0.035
Corrugated Polyethylene (HDPE)	Smooth	0.008-0.015
	Corrugated	0.018-0.025
Polyvinyl Chloride (PVC)	Smooth	0.008-0.012
Cast-Iron Pipe, uncoated		0.013
Steel Pipe		0.009-0.013
Vitrified Clay Pipe		0.012-0.014
Vitrified Clay Liner Plates		0.015
Cemented Rubble Masonry Walls		
Concrete Floor and Top		0.017-0.022
Natural Floor		0.019-0.025
Brick		0.014-0.017
Laminated Treated Wood		0.015-0.017

Reference: Adapted from HDS-4 and HEC-22

Note: The designer should take into account the age of a pipe and possible abrasions, corrosion, deflection, and joint conditions when selecting roughness values. Additionally, inclusion of pipe materials in the table does not necessarily constitute approval for their use by the City.

5.9.2.4 System Layout

The layout of a storm drain system is dependent on topography, hydrology, surface hydraulics, easements and right-of-ways, existing structures and utilities, outfall locations, and other factors. General criteria for the design of a storm drain layout are presented below.

5.9.2.4.1 Vertical Alignment

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 including:

- Colorado Department of Transportation *Standard Specifications for Road and Bridge Constructio*n, Section 700 (Materials Details)
- *Concrete Pipe Design Manual* (ACPA)
- *Handbook of Steel Drainage and Highway Construction Products* (AISI)
- Pipe Manufacturer Specifications
- Other applicable references

Storm drains crossing under railroads and roadways must comply with any cover requirements specified for culverts in Section 5.10, Culverts and Bridges, as well as with any criteria the railroad owner may have.

Pipes installed under any driving or parking area shall be designed for H-20 minimum live load, and all pipes shall have a minimum of 1' of cover from finished grade to top of outside of pipe regardless of location.

In a manhole, the lowest inlet pipe invert elevation must be at least 0.2 feet higher than the outlet pipe invert elevation.

5.9.2.4.2 Horizontal Alignment

All bends in storm drain alignment must be accommodated by a manhole or other appropriate structure, and no bend may be acute.

The storm sewer system alignment shall be designed to minimize the length of pipe and to provide a reasonably uniform pipe slope throughout.

5.9.2.4.3 Utility Clearances

The designer shall consult with each of the local utility companies to determine the location of their existing lines and shall comply with their required minimum clearances.

Pipe encasement may be required in some locations where minimum utility clearances are unable to be met. The City and affected utility shall approve the design of any required encasement.

5.9.2.4.4 Manholes

All manholes must provide access to the storm drain for maintenance and inspection. All manhole inverts shall be formed with a minimum of a half bench to provide more hydraulically-efficient flow through the manhole.

A manhole must be located at all changes in main line pipe size or slope, at abrupt changes in main line invert elevation, and at main line bends. For storm drain pipes of less than 48-inch diameter, manholes are also required at all lateral junctions, however, lateral pipes may be connected to main lines larger than 48" without the use of a manhole with City approval.

Maximum allowable manhole spacing for pipes 24" and smaller is 300 feet. Maximum allowable manhole spacing for pipes larger than 24" is 400 feet.

Structure foundation drains shall be connected directly into a storm drain pipe where an enclosed storm drain system exists but there is no storm drain manhole conveniently located to connect into. In these instances, a stub-out shall be installed in the storm drain line that is minimally large enough to allow the foundation drain line to be inserted into it. A concrete collar shall then be poured around the connection.

5.9.3 STORM DRAIN HYDRAULICS

This Section presents the hydraulic methodology used to calculate storm drain capacities and thereby to design a storm drain system. The actual design process is presented in Section 5.9.6.

5.9.3.1 Gravity-Flow Analysis

Initial storm drain design is completed by selecting pipe sizes based on capacity calculated using open-channel flow computations. Starting at the uppermost reach of the storm drain, at the first inlet, the designer applies Manning's equation (Equation 5.9.1) for each segment of drain. A segment is a reach of pipe with a junction, transition, grade change, horizontal bend, or pipe size change at each end.

$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} S_o^{1/2} \qquad (5.9.1)$$

Where:

- Q_f = Full Flow Discharge (cfs)
- n = Manning's Roughness Coefficient
- A_f = Full Flow Area, $\pi D^2/4$ for circular pipes, (sf)
- R_f = Full Flow Hydraulic Radius, D/4 for circular pipes, (ft)
- S_o = Pipe Slope (S_o=S_f for full flow)
- D = Pipe Diameter (ft)

Alternately, Equation 5.9.2 may be used to directly solve for the minimum required pipe diameter for circular pipes. The designer should always round up to the nearest standard pipe size, keeping in mind that losses in the pipe may decrease available capacity. Initial pipe size, D_i, is based on the peak design flow for that pipe segment, Q_P.

$$D_i = \left[\frac{2.16 n Q_P}{S_o^{1/2}} \right]^{3/8} \qquad (5.9.2)$$

For non-circular pipes, Equation 5.9.2 provides an equivalent diameter based on flow area.

5.9.3.2 HGL and EGL Calculation

Following the initial storm drain design, the system is analyzed using energy-momentum theory to account for specific energy losses. This method allows for the calculation of the HGL and EGL for a given storm drain line by starting with the water surface elevation of the outfall and working upstream, accounting for losses due to pipe friction, manholes, bends, junctions, and pipe entrances and exits. Compliance with minimum and maximum flow velocities is based on peak design flow in the final selected pipe size for each segment. Note that pressure flow is not allowed for the minor storm, and the depth of water in a pipe shall not exceed 0.8 times the pipe diameter.

Energy-momentum theory is based upon the concept that energy, typically expressed in hydraulics as "head" in a linear dimension such as feet, is conserved along a given conduit segment. For a segment where *A* is the upstream end and *B* is downstream, the steady-flow energy equation can be expressed as:

$$z_A + \frac{p_A}{\gamma} + \frac{V_A^2}{2g} + h_p = z_B + \frac{p_B}{\gamma} + \frac{V_B^2}{2g} + \sum h_L \qquad (5.9.3)$$

Where:

$z =$ Invert Elevation above any Horizontal Datum
$p =$ Fluid Pressure
$\gamma =$ Specific Weight of Water $\cong$ 62.4 lbf/ft^3
$V =$ Flow Velocity
$h_p =$ Head Added by a Pump (if applicable)
$\sum h_L =$ Sum of Head Losses in Segment A - B as calculated per the methods prescribed in this section.

The EGL is calculated by Equation 5.9.4. Each term represents the hydraulic head contributed to the total energy head by an energy component. For instance, the third term, $V^2/2g$, is the velocity head. The EGL elevation at a given point is equal to:

$$EGL = z + \frac{p}{\gamma} + \frac{V^2}{2g} \qquad (5.9.4)$$

The HGL elevation is simply the EGL minus the velocity head:

$$HGL = EGL - \frac{V^2}{2g} \qquad (5.9.5)$$

In cases where outfall water surface is equal to or higher than the outlet flow elevation, the EGL and HGL are assumed to be equal, i.e. velocity is zero at the downstream point where calculations start. However, if the outfall water surface is lower than the outlet pipe flow elevation, the flow elevation is used as the outlet HGL. Note that the outfall water surface elevation used must be determined coincident with the time of peak flow from the storm drain.

The HGL at the next structure up the line is determined by the equations presented in **Table 5.9.2**. The equations are separated by HGL at the pipe inlet downstream of the manhole and the pipe outlet at the inlet to the manhole. For non-surcharged flow (less than 80% pipe depth), the free water surface at the pipe inlet (downstream end of the manhole) is added to

head loss across the manhole to find the next pipe outlet HGL (upstream end of the manhole). All storm drain systems shall be designed for non-surcharged flow for the minor storm.

Table 5.9.2 Equations for Determining HGL

Surcharge Conditions	Outlet Submergence	HGL in Manhole/Junction	At	Equation Number
$d_n/D > 0.80$	N/A	$= HGL_{Pipe\ Outlet} + h_f$	Pipe Inlet (D/S from MH)	(5.9.6)
$d_n/D > 0.80$	N/A	$= HGL_{Pipe\ Inlet} + h_{mh}$	Pipe Outlet (U/S from MH)	(5.9.7)
$d_n/D \leq 0.80$	Unsubmerged	$= WSE_{Pipe\ Inlet}$	Pipe Inlet (D/S from MH)	(5.9.8)
$d_n/D \leq 0.80$	Unsubmerged	$= WSE_{Pipe\ Inlet} + h_{mh}$	Pipe Outlet (U/S from MH)	(5.9.9)
$d_n/D \leq 0.80$	Submerged	= Larger of Equations 5.9.6 and 5.9.8 OR = Larger of Equations 5.9.7 and 5.9.9		

Where:

d_n = Normal Flow Depth in Pipe (feet)

$HGL_{Pipe\ Outlet}$ = Larger of Tailwater Elevation, Flow Depth Elevation at Pipe Outlet, and HGL at Next Downstream Pipe Inlet

$WSE_{Pipe\ Inlet}$ = Free Water Surface Elevation at Pipe Inlet

h_f, h_{mh} = Head losses as described in this section

Occasionally, design flow through a pipe may be not only gravity-flow but also supercritical. Pipe losses (h_f) in a supercritical pipe section are not carried upstream.

In locations where two adjoining pipe segments flow in supercritical conditions, manhole losses are also ignored for that line. The designer should be careful to include these losses where only one of the pipes on the line contains supercritical flow.

Inlet pipes to a manhole must occasionally have an invert significantly above that of the outlet pipe. In locations where the outlet pipe water surface elevation (or HGL if pressure flow) is below the invert of an inlet pipe, that inlet pipe is treated as an outfall pipe. In this case, the outfall water surface elevation is always lower than the pipe outlet water level, so the latter elevation is used for the initial HGL of the new upstream reach. The outflow pipe from the manhole in such a situation acts as a culvert under either inlet or outlet control. See Section 5.10, Culverts and Bridges, for information regarding the computation of an HGL at the manhole and calculation of head loss due to a culvert inlet.

The following Sections prescribe methods for determining the energy losses induced by pipe friction and manholes that may be encountered by storm drain flows.

5.9.3.2.1 Pipe Friction Losses

Pipe friction is a significant source of energy dissipation in storm drains. For gravity flow, friction slope (S_f) can be assumed to be equal to the slope of the pipe invert (S_o). For pipes with a surcharge flow condition ($d_n/D > 0.80$), Equations 5.9.6 and 5.9.7 define friction slope:

$$S_f = \frac{n^2 V_{avg}^2}{K_Q R^{4/3}} \qquad (5.9.10)$$

Where:

$K_Q = 2.21$

$$S_f = \left[\frac{Q_{avg} n}{K_Q D^{8/3}}\right]^2 \qquad (5.9.11)$$

Where:

$K_Q = 0.46$

Since flow rate and cross-sectional area typically remain constant through one segment of pipe, average velocity can be assumed to equal flow rate divided by flow area. Equation 5.9.11 is based on the average flow rate in the pipe segment.

Once the friction slope is known, pipe friction head loss is calculated by multiplying the friction slope by the pipe segment length:

$$h_f = S_f L \qquad (5.9.12)$$

5.9.3.2.2 Manhole Junction Losses

This Section details calculating approximate head loss through a manhole. This method applies to any junction of two or more pipes accessible by a manhole.

For each manhole, the designer first calculates the initial head loss coefficient (K_o) and all applicable coefficient correction factors (C_x). The adjusted head loss coefficient (K) and head loss in the manhole (h_{mh}) are then computed.

$$h_{mh} = K\left(\frac{V_o^2}{2g}\right) \qquad (5.9.13)$$

$$K = K_o C_D C_d C_Q C_p C_B \qquad (5.9.14)$$

$$K_o = 0.1\frac{b}{D_o}(1 - \sin\theta) + 1.4\left(\frac{b}{D_o}\right)^{0.15} \sin\theta \qquad (5.9.15)$$

Where:

$\theta =$ Angle Between Inflow and Outflow Pipes ($\leq 180°$)

$b =$ Manhole or Junction Diameter (at water level)

$D_o =$ Outlet Pipe Diameter

The coefficient correction factors are calculated using the equations presented below and are applied to the initial head loss coefficient per Equation 5.9.14. Note that some correction factors do not apply to all manhole configurations. These non-applicable factors are set to unity.

C_D – Correction Factor for Pipe Diameter

This factor applies to pressure flow when the ratio of water depth in the manhole above the outlet pipe invert to outlet pipe diameter is greater than 3.2 ($d_{mho} / D_o > 3.2$).

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \qquad (5.9.16)$$

Where:

D_o = Outlet Pipe Diameter

D_i = Inlet Pipe Diameter

C_d – Correction Factor for Flow Depth

This factor applies to gravity flow and low-pressure flow when the ratio of water depth in the manhole above the outlet pipe invert to outlet pipe diameter is less than 3.2 ($d_{mho} / D_o < 3.2$).

$$C_d = 0.5\left(\frac{d_{mho}}{D_o}\right)^{0.6} \qquad (5.9.17)$$

Where:

d_{mho} = Water Depth in Manhole above Outlet Pipe Invert

D_o = Outlet Pipe Diameter

For purposes of this calculation, water depth in the manhole is approximated as the vertical distance from the outlet pipe invert to the HGL at the upstream end of the outlet pipe.

C_Q – Correction Factor for Relative Flow

This factor applies to manholes with three or more pipes entering the structure at similar elevations (one of these pipes will be the outlet pipe). This correction factor does not apply to the effects of inflow pipes with flowlines far enough above the outlet pipe to qualify as plunging flow.

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \qquad (5.9.18)$$

Where:

θ = Angle between the Inflow Pipe of Interest and the Outflow Pipe

Q_i = Flow in the Inflow Pipe of Interest

Q_o = Flow in the Outflow Pipe

The Pipe of Interest is the inlet pipe to the manhole on the line being investigated. This factor accounts for streamline interference by flow from other pipes entering the manhole.

C_p – Correction Factor for Plunging Flow

This factor applies to manholes with an inflow pipe of interest that is affected by plunging flow from another inflow pipe having a higher flowline. The factor does not apply to the line with the pipe that is discharging the plunging flow. It only applies when the height of the plunging-flow pipe flowline above the outlet pipe center exceeds the manhole water depth: $h > d_{mho}$

$$C_p = 1 + 0.2\left(\frac{h}{D_o}\right)\left(\frac{h - d_{mho}}{D_o}\right) \qquad (5.9.19)$$

Where:

$h =$ Vertical Distance of Plunging Flow (height of plunging flow pipe flowline above center of outlet pipe)

$d_{mho} =$ Water Depth in Manhole above Outlet Pipe Invert

$D_o =$ Outlet Pipe Diameter

A common application of this correction factor occurs at locations where inlets convey intercepted flow directly to the storm drain main line via drop inlets or where laterals enter a manhole well above the main line invert.

C_B – Correction Factor for Benching

This factor applies to all flow conditions. See **Table 5.9.3** for proper correction factor selection.

Table 5.9.3 Benching Correction Factors

Bench Type	Outlet Pipe Conditions	
	Fully Submerged, Pressure Flow*	Unsubmerged, Free Surface Flow**
Flat or Depressed	1.00	1.00
Benched: ½ Pipe Diameter	0.95	0.15
Benched: 1 Pipe Diameter	0.75	0.07
Improved Bench	0.40	0.02

*Applies for $d_{mho}/D_o \geq 3.2$

**Applies for $d_{mho}/D_o \leq 1.0$

Note that the submerged pressure-flow factors do not apply until flow depth in the manhole has exceeded 3.2 times the outlet pipe diameter. For depths between free surface flow and full pressure-flow conditions ($1.0 > d_{mho}/D_o < 3.2$), the designer should use a linear interpolation to compute the benching correction factor.

5.9.3.3 Computer Hydraulic Modeling

HGL and EGL calculations may be prepared using computer software. The Urban Drainage and Flood Control District has a program entitle UD-Sewer that is available on their website that will aid in storm drain system design. The designer is urged to use sound professional

judgment to select the program that is most applicable to local design standards and the requirements of a given project. The designer shall consult with the Public Works Department before using any software other than UD-Sewer.

5.9.4 CONSTRUCTION STANDARDS

This Section outlines standards for the construction of storm drain systems.

5.9.4.1 Storm Drain Pipe

5.9.4.1.1 Minimum Size

All public and private storm drain pipes shall have a minimum diameter of 12". For non-circular pipes, these minimum diameters represent equivalent diameters based on cross-sectional areas.

5.9.4.1.2 Maximum Size

There is no maximum pipe size specified. However, the designer should consider the possibility of utilizing multiple barrels where physically and economically advisable.

5.9.4.1.3 Pipe Material and Shape

All storm drain pipes shall comply with the City's Standard Specifications as well as the most recent revision of the CDOT Standard Specifications. Public storm drain pipes shall be corrugated metal except in limited cases where capacity or other design constraints necessitate the use of reinforced concrete. These limited cases must be individually approved by the Public Works Department. Private storm drain pipes may be circular, elliptical, arch, or box-shaped and constructed of reinforced concrete, corrugated aluminized or galvanized steel, corrugated aluminum, corrugated or profile wall high density polyethylene, or polyvinyl chloride. Public sites may use circular, elliptical, arch, or box-shaped conduit as well but material shall be limited to reinforced concrete, corrugated aluminum, and aluminized or galvanized corrugated steel pipe.

In the downtown area there is an existing vitrified clay pipe (VCP) that was formerly a sanitary sewer line. There are several foundation and storm drains connected to this line, however, new connections to this line are prohibited. Furthermore, where new construction is adjacent to a VCP, the VCP shall be abandoned and replaced, and all existing connections shall be reconnected to the replacement pipe as appropriate.

5.9.4.1.4 Joint Fillers, Sealants, and Gaskets

All pipe joint fillers, sealing compounds, and gaskets, and the installation thereof, shall be governed by the City's Standard Specifications. Rubber gaskets shall be used at pipe section joints where greater than five feet of pressure head is expected in the design storm. This is equivalent to locations where the HGL elevation is five feet higher than the pipe crown.

5.9.4.1.5 Backfill Loading

At a minimum, backfill shall be governed by the City's Standard Specifications. Backfill shall be as required to ensure the pipeline maintains full function under an HS-20 loading if subjected to traffic loading. A minimum of 1' of cover between the top of the pipe and finished subgrade is required at all times.

5.9.4.1.6 Pipe Bedding

Specifications for pipe trenching and bedding, and backfill shall be as outlined in the City's Standard Specifications.

5.9.4.2 Manholes

The required diameter of the manhole barrel is dependent upon the size of the largest pipe connecting to it. For pipe diameters up to 24", a 4' manhole may be used. For pipe diameters up to 42", a 5' diameter manhole must be used. For pipes larger than 42", a 6' diameter, or box-base manhole must be used. Specific manhole designs approved for use are discussed in the City's Standard Specifications.

5.9.4.3 Inlets

Street inlets are discussed and specified in Section 5.8, Streets and Roadside Conveyance. Other enclosed inlets, such as area inlets, shall be CDOT Type C or Type D with bicycle safe grates if it is possible they will be subjected to pedestrian or bicycle traffic, or closes mesh grates if they will not be subjected to non-vehicular traffic.

Culvert-type inlets, such as those directing ditch flows into a storm drain, are required to include a flared end section to increase capacity and reduce erosive potential. See Section 5.10, Culverts and Bridges, for culvert inlet design criteria.

5.9.4.4 Outlets

Storm drain outlets typically discharge to a drainage channel, a natural stream or river, or a detention basin. In order to increase storm drain capacity and reduce erosion potential, outlets are required to include a flared end section equivalent to those required for culvert outlets as specified in Section 5.10, Culverts and Bridges.

Due to the erosive potential of high-velocity storm drain flow on unlined channels and in detention and retention basins, a riprap apron and/or an energy-dissipation structure shall be constructed at all storm drain outlets per requirements set forth in Section 5.10, Culverts and Bridges.

5.9.5 <u>STORM DRAIN SYSTEM DESIGN</u>

5.9.5.1 Initial Storm Drain Design

The following procedure is for the initial layout and sizing of a storm drain. The results of this process must be validated by the final design methodology before the system can be considered viable. However, this design process may be used for conceptual drainage report submittals.

1. Choose a system layout based on street rights-of-way and other drainage easements, developed topography, utility locations, and likely cost and performance. This layout should include preliminary inlet and manhole locations.

2. Complete the hydrologic analysis of the project area. Compute peak flow in each street starting at the upper end of the project area and working downstream. Typically, the runoff from multiple streets will converge at a point, so all streets that

are tributary to that point must be completed before moving on downstream. An inlet should be located wherever the minor storm peak street flow exceeds the allowable capacity for that street and at all sump locations.

3. Begin Initial storm drain sizing at the uppermost inlet for each street, combining individual street storm drains where appropriate. The design flow for a given storm drain segment is based on the sum of all flow from upstream pipes and the larger of the major and minor street flows exceeding the respective street capacity at the inlet just upstream from that segment.

4. Use Manning's open channel flow, including approximate junction head losses, to compute required pipe size and slope for each pipe segment. Evaluate pipe size and/or slope at locations where significant energy losses may occur, such as large or complex pipe junctions and major pipe bends and increase the pipe size as deemed appropriate. Downstream pipes should not be smaller than upstream pipes unless the flow rate decreases.

5.9.5.2 Final Storm Drain Design

Following the completion of an initial storm drain system design, final design may begin. While many designers may choose to utilize computer software to model storm drain systems, smaller projects are still often completed manually. Hand calculations are also useful for spot-checking of computer models to ensure the software is functioning properly. The level of hydraulic analysis presented in this Section shall be met for any final drainage report.

1. The hydraulics for each system shall be recomputed using the energy-momentum theory starting at each system's outfall point. All applicable energy losses must be included in the calculations, including head loss due to manhole/junction chambers, pipe transitions and bends, no-access junctions, and entrances/exits.

2. The HGL and EGL shall be calculated for each end of each pipe segment and each side of all locations of additional energy loss listed in Step 1. The design storm HGL shall not exceed finished grade at any location along the storm drain.

5.9.6 DISSIMILAR PIPE CONNECTIONS

Because pipe material requirements for private facilities are different from those for public ones, it is often required to join together pipes of different materials when connecting a private storm drain system to the public one. There are several requirements for these types of connections. The connection must be made at a manhole, area inlet, or other similar drainage structure that will easily accommodate both types and sizes of pipe. The connection must also occur outside the public right-of-way, with all portions of the connection being owned, maintained, repaired, and replaced, if necessary, by the developer.

SECTION 5.10
CULVERTS AND BRIDGES

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5.10.1 INTRODUCTION

This chapter provides information for the hydraulic design of culverts and bridges. It is intended for use by those with a good understanding of basic hydrologic and hydraulic methods and with experience in the design of hydraulic structures. The designer should also understand the variety of possible flow conditions in these complex hydraulic structures.

Culverts and bridges convey water beneath highways, railroads, and other embankments. The size, alignment, and support structures of a culvert or bridge will directly affect its flow capacity. Inadequate culvert or bridge capacity can force water out of the conveyance system, flood an alternate path, and cause damage away from the channel. Culvert and bridge design also involves structural design considerations. All culverts potentially subjected to vehicular traffic shall be designed structurally for an H-20 live load in accordance with the AASHTO recommendations. All culverts under railroads shall be designed in accordance with the railroad's standards. Aside from those stipulations, only the hydraulic aspects of design are covered in this Section.

5.10.2 CULVERT HYDRAULICS

This Section presents the general procedures for hydraulic design and evaluation of culverts. Information in this Section is based on the Federal Highway Administration (FHWA) *Hydraulic Design of Highway Culverts,* Hydraulic Design Series No. 5 (HDS-5). For situations not covered in this Section, the methodology in HDS-5 shall be followed.

Inlet and outlet control are the two basic types of flow in culverts. Under inlet control, the flow through the culvert is controlled by the headwater on the culvert and the inlet geometry. Under outlet control, the flow through the culvert is controlled primarily by culvert slope, roughness, and tailwater elevation.

When designing a culvert, the designer must evaluate both inlet and outlet control conditions for the given design constraints. The condition which produces the greater energy loss for the design condition determines the appropriate control to use for the culvert design. Culvert hydraulic calculations shall be performed using rating nomographs and/or culvert hydraulic analysis programs.

5.10.2.1 Required Design Information

Note that culverts crossing State roadways are additionally subject to the requirements of the Colorado Department of Transportation (CDOT). The governing criteria will be the stricter of CDOT criteria and the criteria specified herein.

5.10.2.1.1 Discharge

Culverts are required where natural or manmade channels are crossed by roads, streets, or other infrastructure. The amount of channel flow which encroaches upon the road should be minimized to protect the road embankment and pavement from erosion damage as well as to protect vehicles and pedestrians from dangerous flow depths and velocities. The major storm shall be used to design culvert crossings under arterial and collector roadways and the minor storm to design culvert crossings under local roadways unless the local roadway is the only road providing access to an area, in which case the major storm shall be used.

5.10.2.1.2 Headwater

The depth of the upstream water surface measured from the invert at the culvert entrance is referred to as headwater. For arterial and collector roadways, the major storm shall not cause headwater at any culvert to encroach on any drive lane and HW/D for the major storm shall not exceed 1.5. For local roadways, the minor storm shall not inundate the outside edge of the outside drive lane by more than 6" and HW/D shall not exceed 1.5. In no case shall an increase in backwater from a culvert extend upstream onto an adjacent property.

5.10.2.1.3 Tailwater

Tailwater is defined as the depth of water downstream of the culvert measured from the outlet invert. Tailwater may be caused by an obstruction in the downstream channel or by the hydraulic resistance of the channel. Backwater calculations from a downstream control point are required to precisely define tailwater. When appropriate, normal depth approximations may be used instead of backwater calculations.

5.10.2.1.4 Velocity

The flow velocity at a culvert outlet can cause local streambed scour and bank erosion at the outlet. The outlet should be designed so as not to discharge on unprotected fills or unstable material. **Table 5.7.3** in Section 5.7, Open Channels, presents the maximum permissible velocities for several types of channel linings. Velocities exceeding these values require outlet protection. See below in this Section for erosion protection requirements.

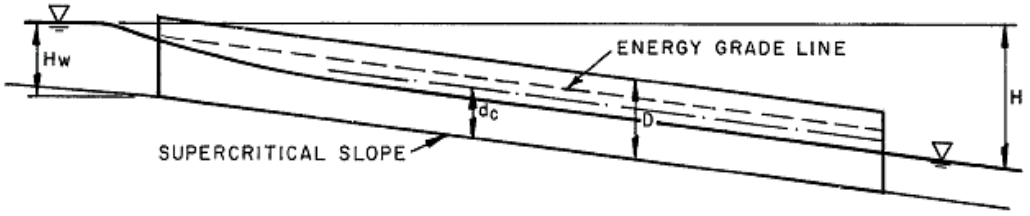
At a minimum, culverts shall be designed to be self-cleaning with a minimum design velocity of 2.5 feet per second when flowing half full. Public and private culverts shall have a maximum design-flow velocity of 10 feet per second. Velocity may be increased to 15 feet per second, but the required gage thickness of corrugated metal pipes must be increased by one increment and the wall thickness of reinforced concrete pipes must increased from Wall B to Wall C in these instances. See the City's Standard Specifications for required pipe gage and thickness.

5.10.2.2 Inlet Control

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. Headwater depth, cross-sectional area, inlet edge configuration, and barrel shape all affect inlet control. Under inlet control, the culvert barrel usually flows partially full. The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity.

Inlet control for culverts can occur in two ways, unsubmerged or submerged. In an unsubmerged condition, the headwater is not sufficient to submerge the top of the culvert and the culvert slope is supercritical, shown in **Figure 5.10.1**. In this situation, the culvert inlet acts like a weir.

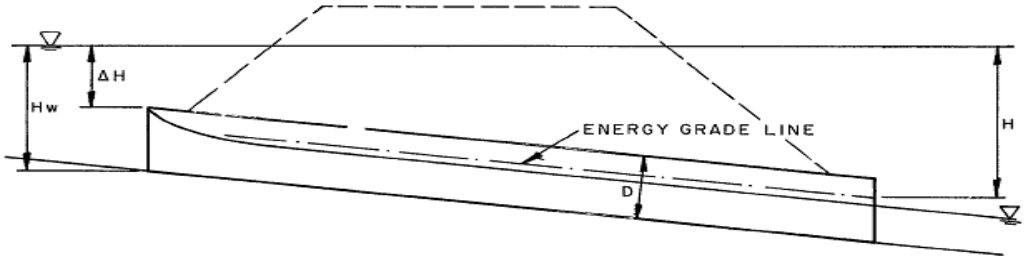
Figure 5.10.1 Inlet Control – Unsubmerged Inlet



Reference: UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 2.

In a submerged condition, the headwater submerges the top of the culvert but the pipe does not flow full as shown in **Figure 5.10.2**. In this situation, the culvert inlet acts like an orifice.

Figure 5.10.2 Inlet Control – Submerged Inlet



Reference: UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 2.

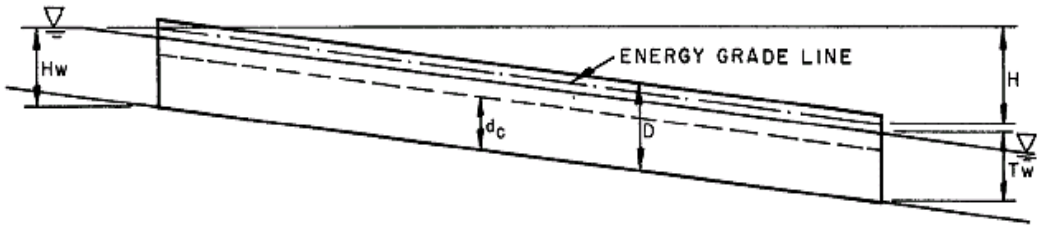
For a culvert operating with inlet control, the upstream water surface elevation and the inlet geometry represent the major flow controls. Culvert roughness, slope, length, and outlet conditions, including tailwater, are not factors in determining culvert hydraulic performance.

5.10.2.3 Outlet Control

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for the outlet control flow in a culvert is located at the barrel exit or even further downstream. All of the geometric and hydraulic characteristics of the culvert play a role in determining its capacity. These characteristics include all of the factors governing inlet control, water surface elevation at the outlet, and the slope, length, and roughness of the culvert barrel.

Outlet control will govern if the headwater and/or tailwater is high enough, the culvert slope is relatively flat, and the culvert relatively long. Outlet control will exist under two conditions. The first and less common is when the headwater does not submerge the culvert inlet and the culvert slope is subcritical as shown in **Figure 5.10.3**.

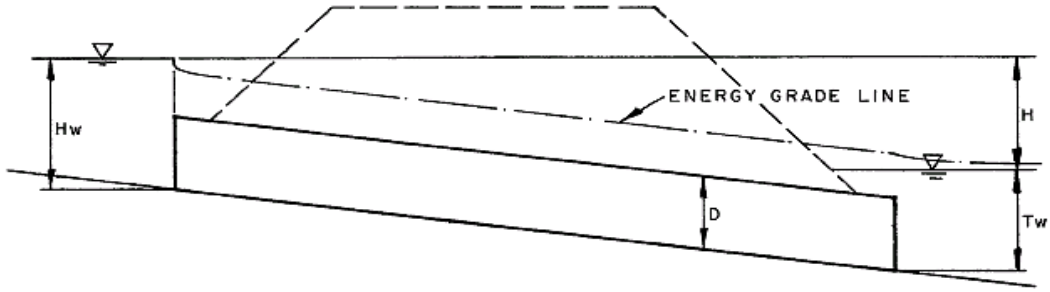
Figure 5.10.3 Partially Full Conduit



Reference: UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 2.

The more common condition exists when the culvert is flowing full as shown in **Figure 5.10.4**.

Figure 5.10.4 Full Conduit



Reference: UDFCD 2001. Urban Storm Drainage Criteria Manual, Volume 2.

Under outlet control, culverts may flow full or partly full depending on various combinations of the above factors. Performance of a culvert under outlet control can be affected by culvert length, roughness, and tailwater depth.

5.10.3 CULVERT SIZING AND DESIGN

All culverts shall be designed and constructed using the following standards. The analysis and design shall consider design flow, culvert size and material, upstream channel and entrance configuration, downstream channel and outlet configuration, and erosion protection.

5.10.3.1 Size and Material

All culverts shall comply with the City's Standard Specifications, as well as the most recent revision of the CDOT Standard Specifications. Public culverts shall be corrugated metal pipe (CMP) except in limited cases where capacity or other design constraints necessitate the use of reinforced concrete pipe (RCP) or reinforced concrete box culverts. These limited cases must be individually approved by the Public Works Department. Private culverts may be circular, elliptical, arch, or box-shaped and constructed of reinforced concrete, corrugated aluminized or galvanized steel, corrugated aluminum, corrugated or profile wall high density polyethylene, or polyvinyl chloride. Public sites may use circular, elliptical, arch, or box-shaped conduit as well but material shall be limited to reinforced concrete, corrugated aluminum, and aluminized or galvanized corrugated steel pipe. Guidance on joining two different pipe materials can be found in Section 5.9, Storm Drain Systems.

In all cases, material and shape shall be selected based on not only hydraulic capacity, but also the ability of a pipeline to maintain full cross-sectional area and function without excessive cracking, breaking, or undergoing excessive deflection.

The minimum size for all public culverts, except those under sidewalks, shall be an 18-inch diameter round pipe or any shape having a corresponding equivalent area. For public culverts under sidewalks and private drainage culverts, the minimum shall be 8".

For public culverts and storm drain pipes placed in areas having corrosive soils, required gage thickness shall be increased by one increment for corrugated metal pipes, and wall thickness shall be increased from Wall B to Wall C for reinforced concrete pipes.

5.10.3.2 Limitations on Cover

Minimum and maximum allowable cover over a pipe will depend on pipe size and material but in no instance shall be less than one foot. Culverts for which less than one foot of cover is available will require additional structural analysis and other provisions such as full depth concrete paving to compensate for the loss of proper cover. In all cases, culverts passing under roadways shall be designed to maintain their full shape and function under an HS-20 loading. The City's Standard Specifications details cover requirements for various types and sizes of pipe. If the City's Standard Specifications do not cover the pipe size or material being used, the manufacturer's recommendations shall dictate the limitations on cover.

5.10.3.3 Location

Culverts shall be located as required to completely drain all rainfall and snowmelt runoff where drainageways intersect a roadbed or sidewalk. The designer shall identify all areas that water could be impounded or flow restricted by the new embankment and consider them for culvert locations. Culverts shall be aligned to give drainageways a direct 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. 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 efficient inlet.

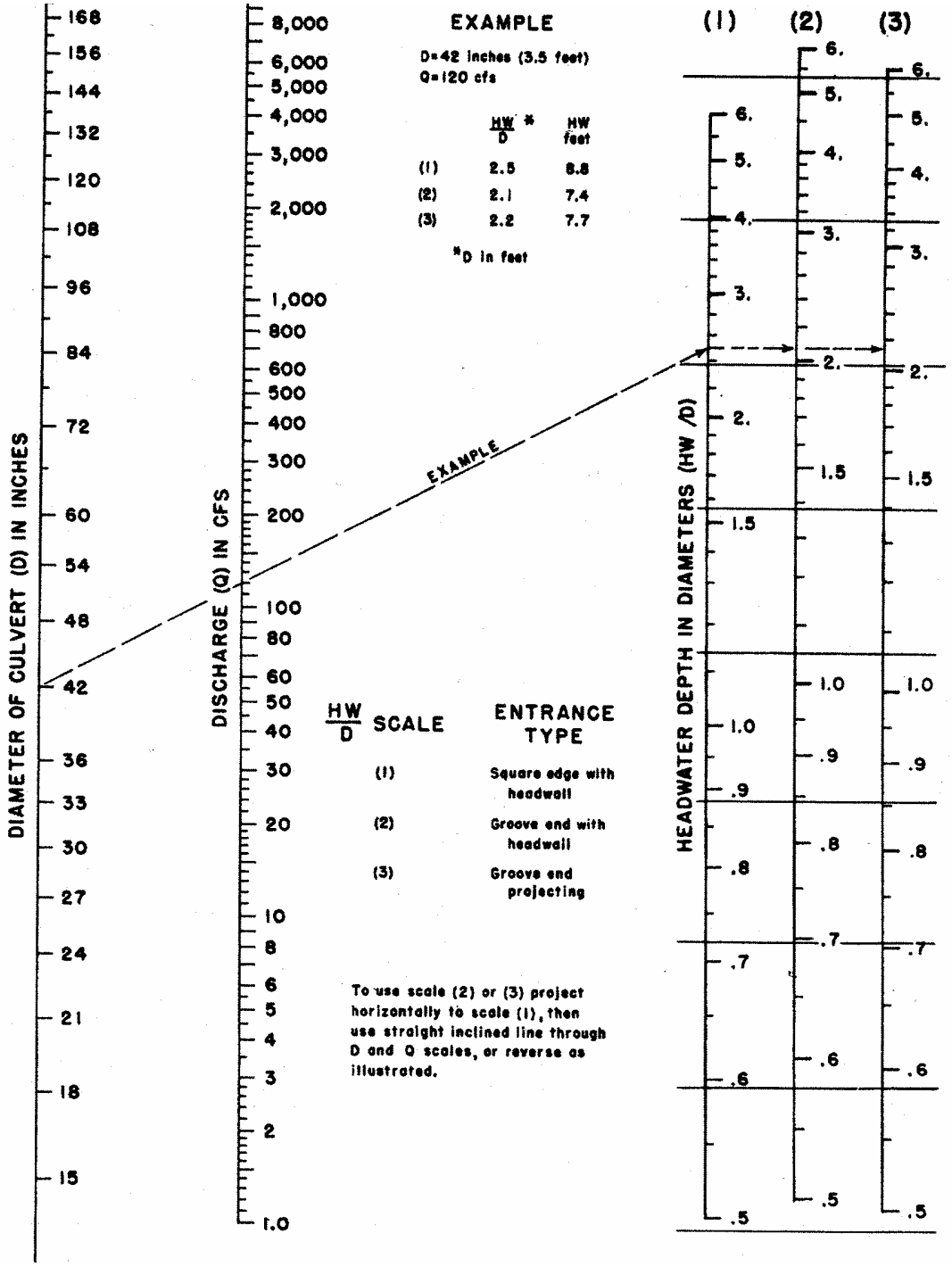
Where the natural channel alignment would result in a culvert alignment skewed more than 30 degrees from perpendicular to a roadway, modification may be necessary. Such modifications will change the natural stability of the channel, and an investigation into other options is recommended. Although economic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

5.10.3.4 Inlet Control Calculation

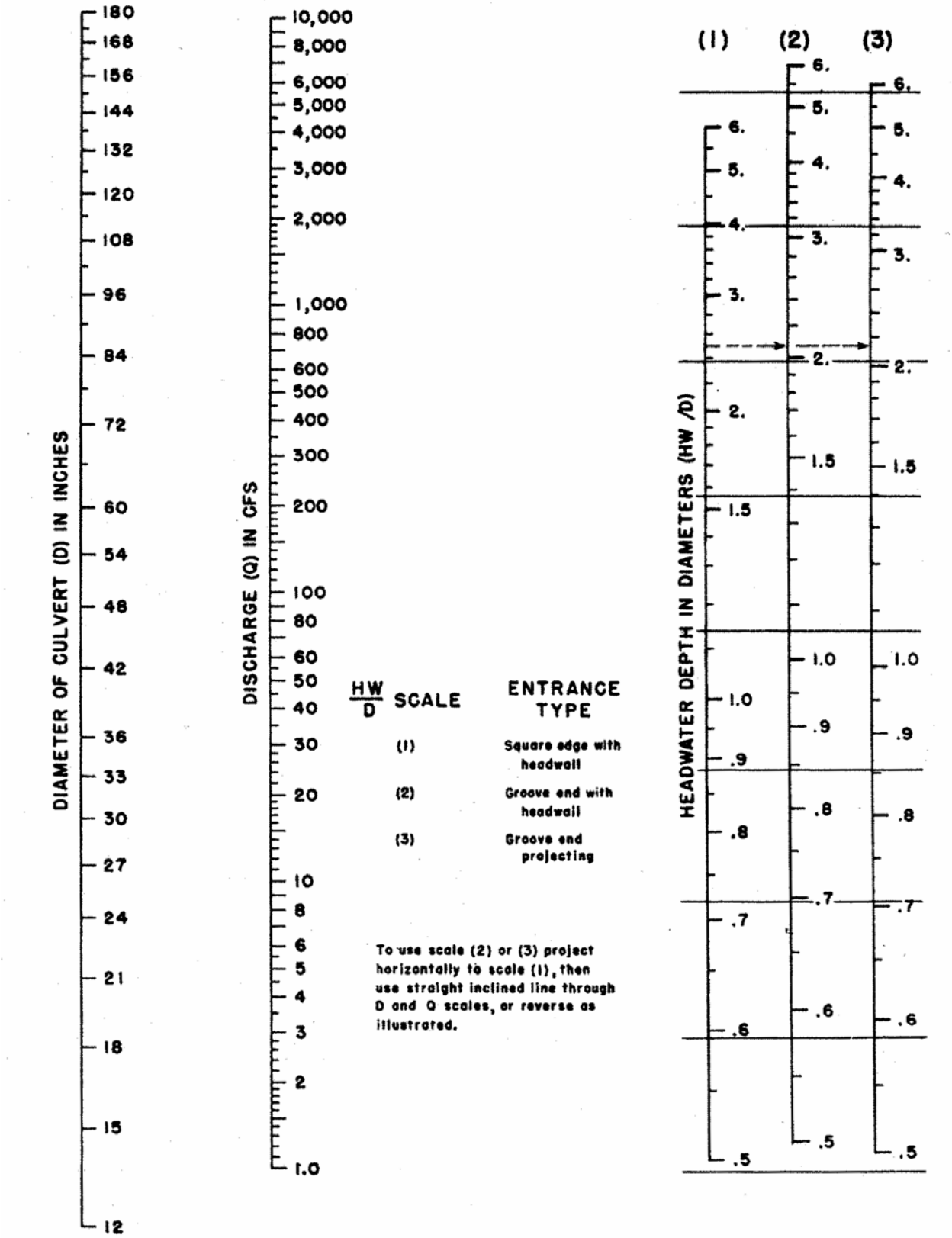
Inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert if it is under inlet control. Approach velocity head may not be included as part of the headwater. Inlet control nomographs from HDS-5 for typical configurations are included in this Section. **Figure 5.10.5** gives an example of the use of an inlet control nomograph. **Figures 5.10.6** through **5.10.8** are inlet control nomographs for circular concrete and corrugated metal pipes and for concrete box culverts. For all other situations, refer to FHWA's HDS-5. To evaluate outlet control, the following procedure shall be followed.

Figure 5.10.5 Inlet Control Nomograph Example



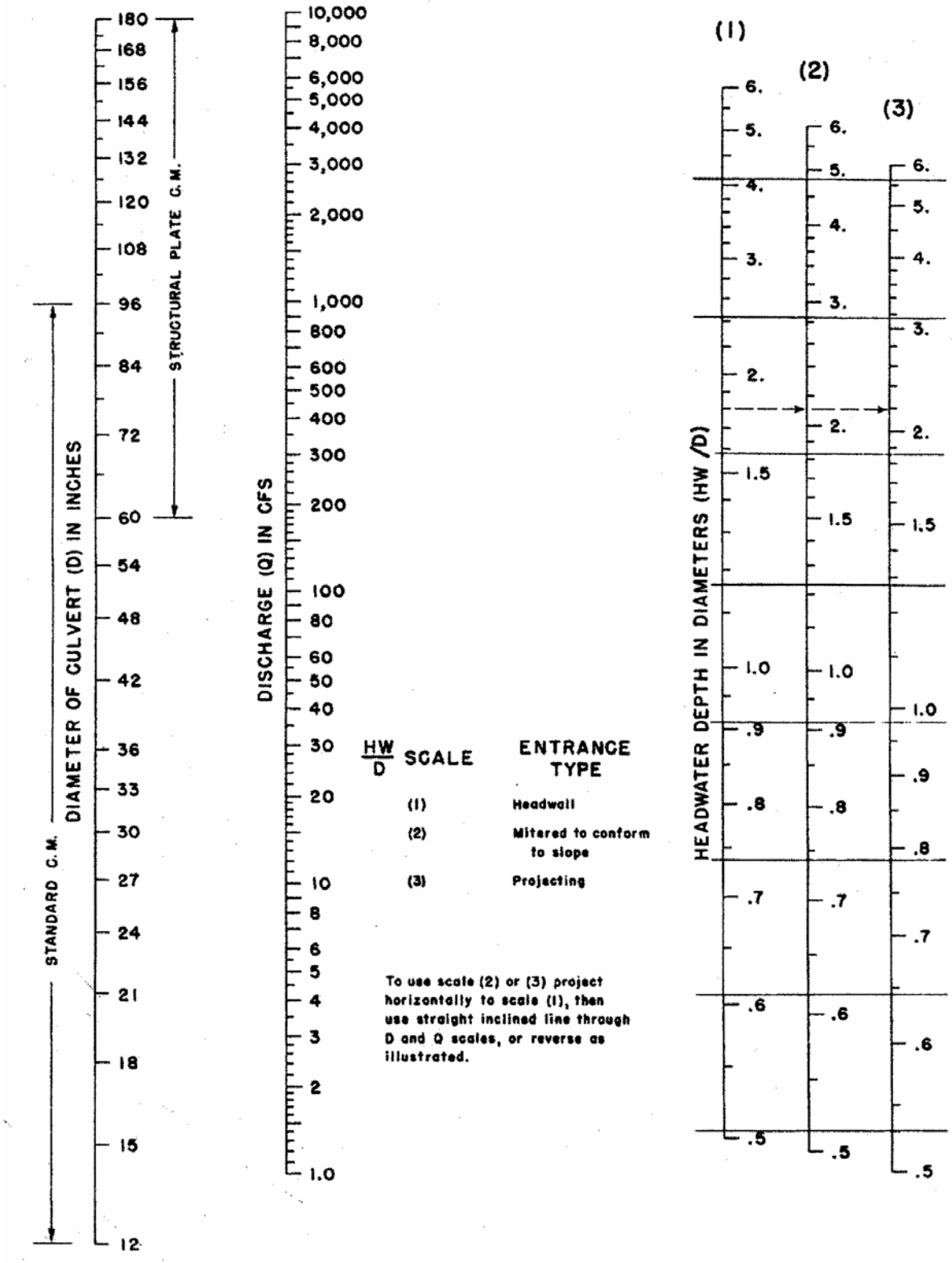
Reference: HDS-5, FHWA

Figure 5.10.6 Inlet Control Nomograph for Circular Concrete Pipes



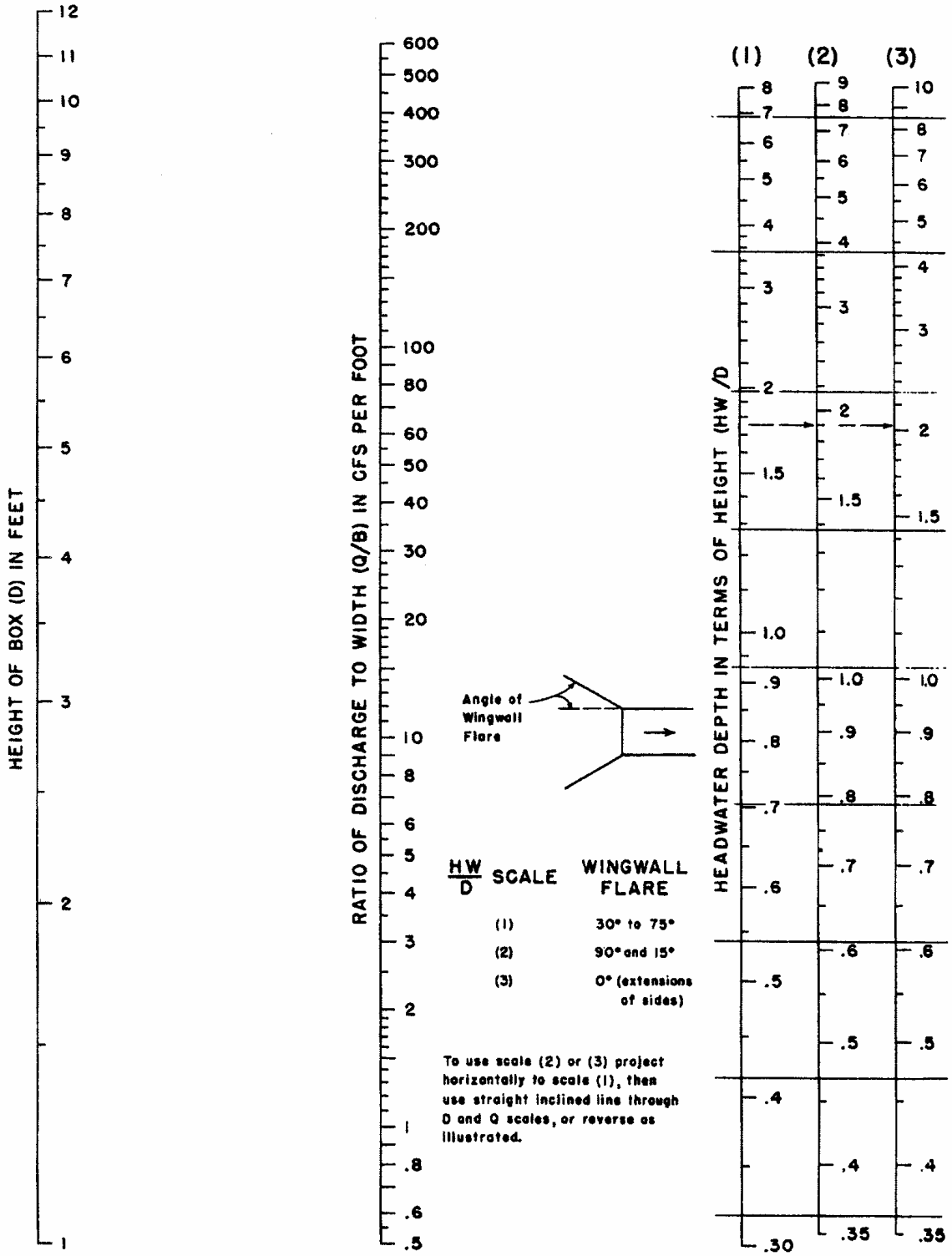
Reference: HDS-5, FHWA

Figure 5.10.7 Inlet Control Nomograph for Circular Corrugated Metal Pipes



Reference: HDS-5, FHWA

Figure 5.10.8 Inlet Control Nomograph for Concrete Box Culverts



Reference: HDS-5, FHWA

5.10.3.5 Outlet Control Calculation

Outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert in outlet control. The approach and downstream velocities may be included in the design process, if desired. Critical depth charts and outlet control nomographs are used in the design process and are included in this Section. For illustration of their use, refer to the example outlet control nomograph and example critical depth chart shown in **Figures 5.10.9** and **5.10.13**. Outlet control nomographs and critical depth charts for circular concrete and corrugated metal pipes and for concrete box culverts are shown in **Figures 5.10.10** through **5.10.12** and in **Figures 5.10.14** through **5.10.15**. For all other situations, refer to FHWA's HDS-5. The following procedure should be followed.

1. Determine the tailwater depth, TW, above the outlet invert at the design flow rate. This is obtained from backwater or normal depth calculations or from field observations.
2. Enter the appropriate critical depth chart with the flow rate and read the critical depth, d_c. The critical depth, d_c, cannot exceed the culvert diameter, D. The d_c curves are truncated for convenience when they converge. If an accurate d_c is required for d_c > .9D consult the *Handbook of Hydraulics* or other hydraulic reference.
3. Calculate $(d_c + D)/2$
4. Determine the depth from the culvert outlet invert to the hydraulic grade line, h_o, with the following equation:

   h_o = TW or $(d_c + D)/2$, whichever is larger
5. From **Table 5.10.1**, obtain the appropriate entrance loss coefficient, K_e, for the culvert inlet configuration.
6. Determine the head losses through the culvert barrel, *H*, using the outlet control nomograph.
   a. Required Manning's *n* values are presented in **Table 5.9.1** of Section 5.9, Storm Drain Systems. If the Manning's *n* value given in the outlet control nomograph is different than the required Manning's *n* for the culvert, adjust the culvert length using the formula:

      $$L_1 = L(n_1/n)^2 \qquad (5.10.1)$$

      Where:
      L_1 = adjusted culvert length (ft)
      L = actual culvert length (ft)
      n_1 = desired Manning's *n* value
      n = Manning's *n* value from the outlet control chart

   b. Using a straightedge, connect the culvert size with the culvert length on the appropriate k_e scale. This defines a point on the turning line.
   c. Again using the straightedge, extend a line from the discharge through the point on the turning line to the head loss, *H*, scale. *H* is the energy loss through the culvert, including entrance, friction, and outlet losses. Careful alignment of the straightedge is necessary to obtain good results from the outlet control nomograph
7. Calculate the required outlet control headwater elevation, EL_{ho}.

   $$EL_{ho} = EL_o + H + h_o \qquad (5.10.2)$$

Where:

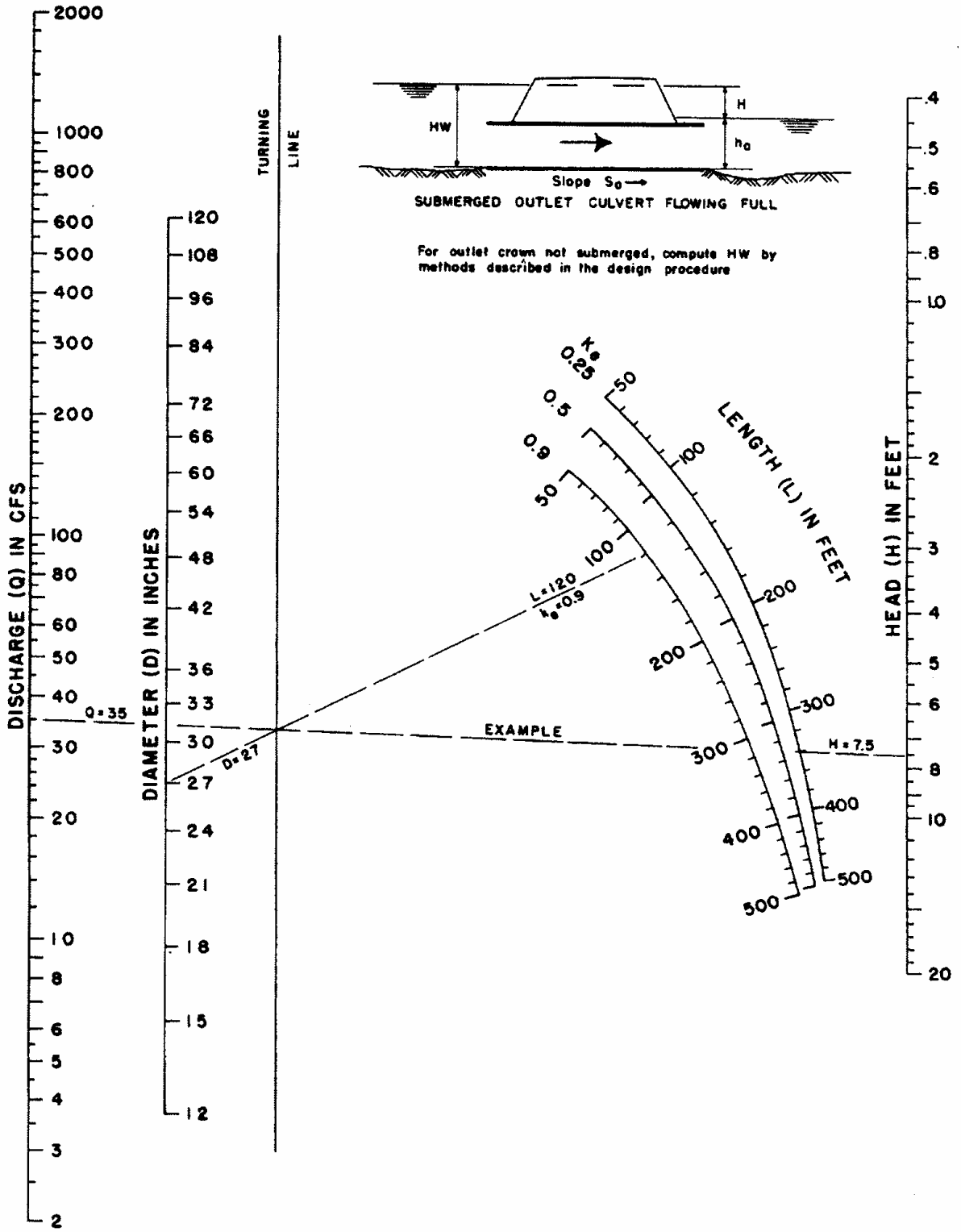
EL_o = invert elevation at the outlet

8. If the outlet control headwater elevation exceeds the design headwater elevation, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

Table 5.10.1 Culvert Entrance Loss Coefficients

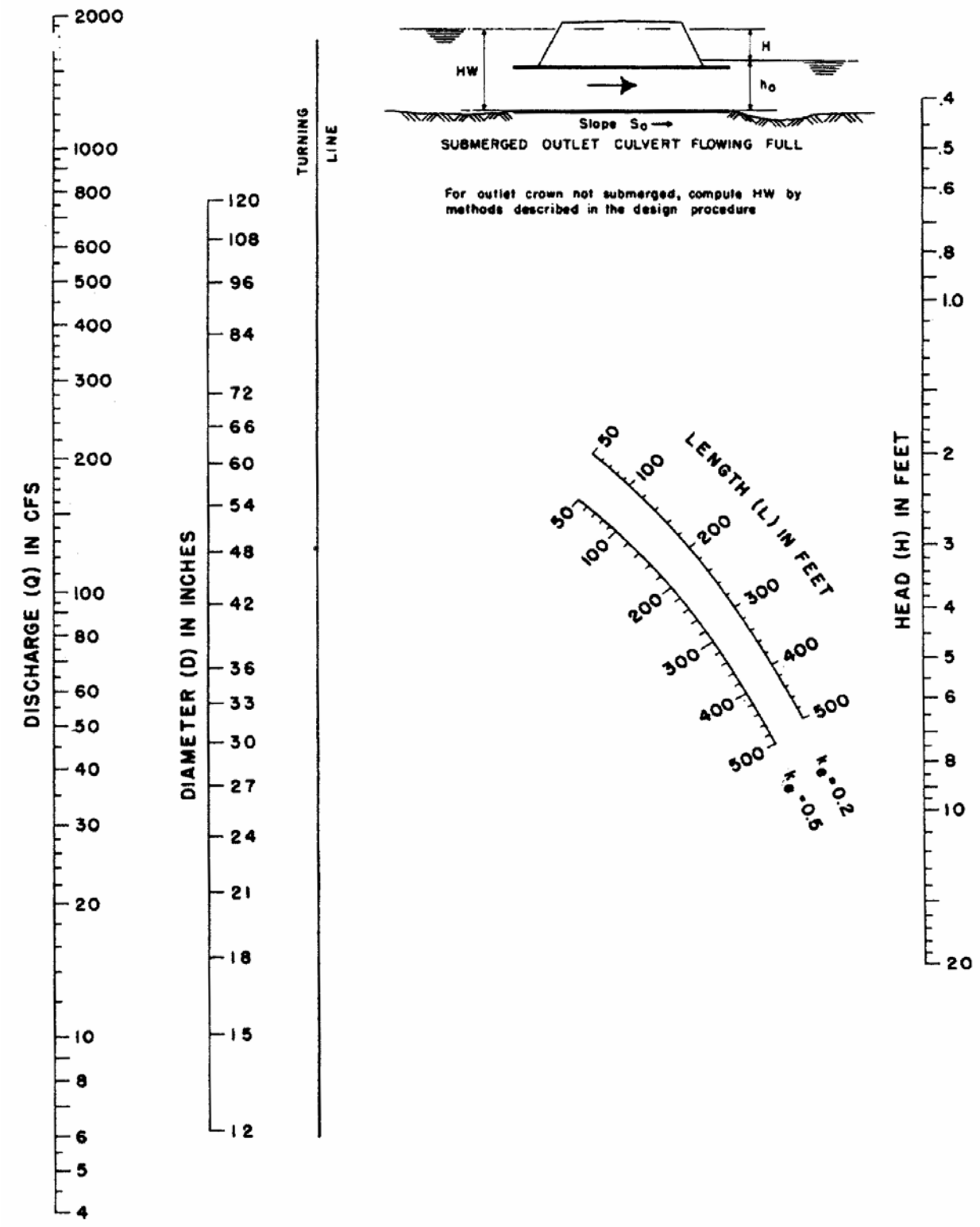
Type of Structure and Design of Entrance	Coefficient K_e
• Pipe Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end.	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius = D/12)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7˚ or 45˚ bevels	0.2
Side- or slope-tapered inlet	0.2
• Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7˚ or 45˚ bevels	0.2
Side- or slope-tapered inlet	0.2
• Box Reinforced Concrete	
Headwall parallel to embankment (no wingwalls):	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30˚ to 75˚ to barrel:	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10˚ to 25˚ to barrel:	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides):	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Figure 5.10.9 Example Outlet Control Nomograph



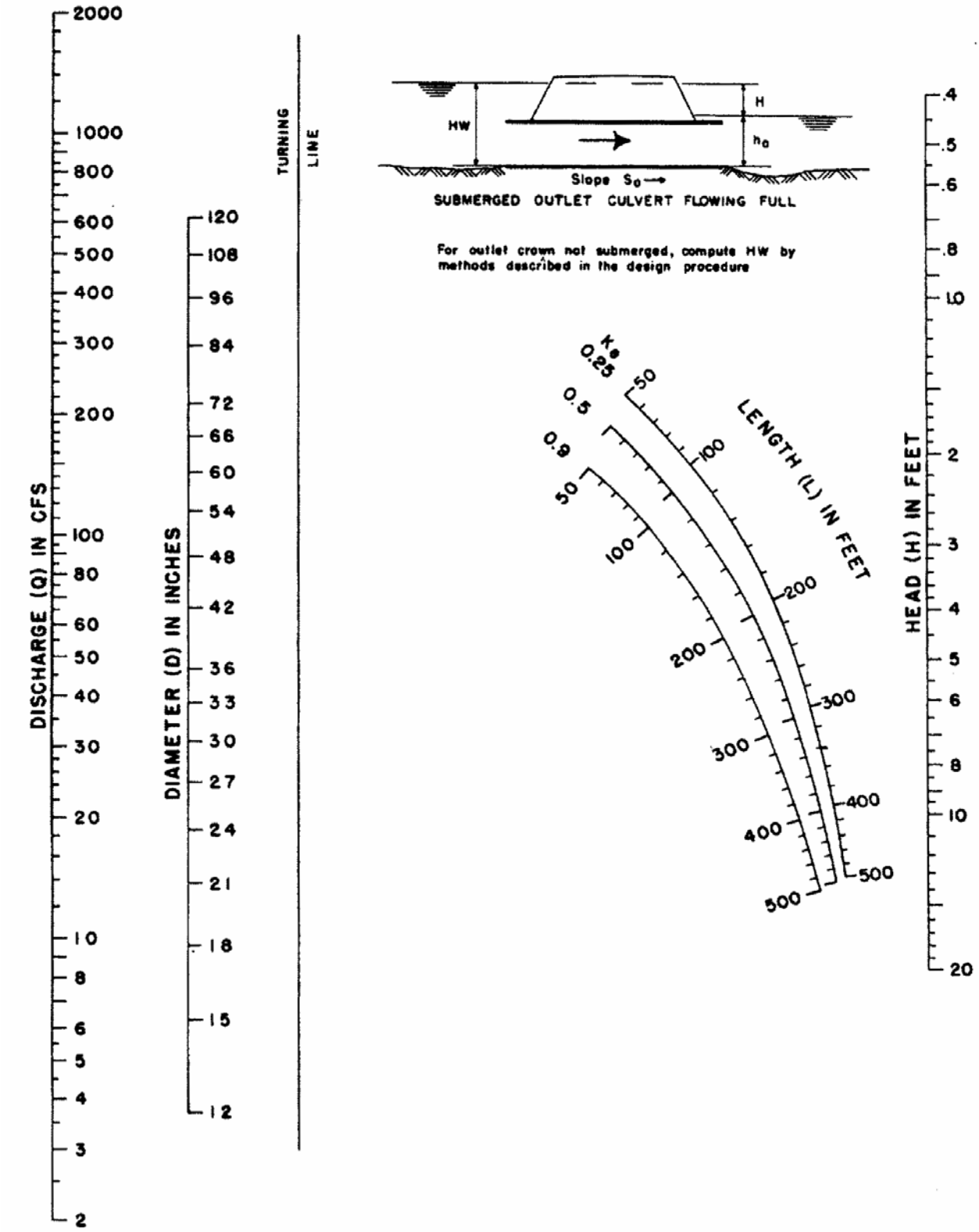
Reference: HDS-5, FHWA

Figure 5.10.10 Outlet Control Nomograph for Concrete Pipe Culverts



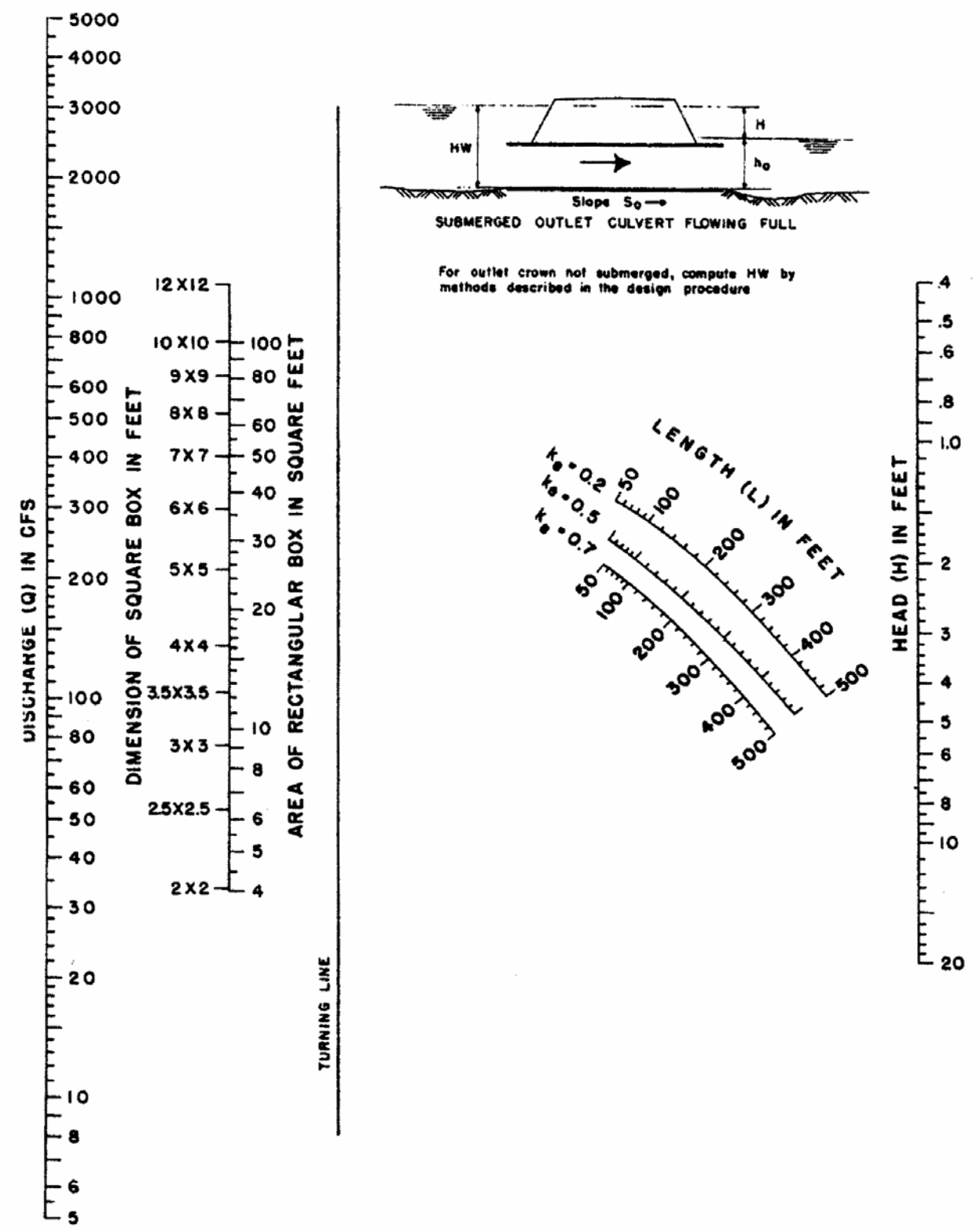
Reference: HDS-5, FHWA

Figure 5.10.11 Outlet Control Nomograph for Corrugated Metal Pipe Culverts



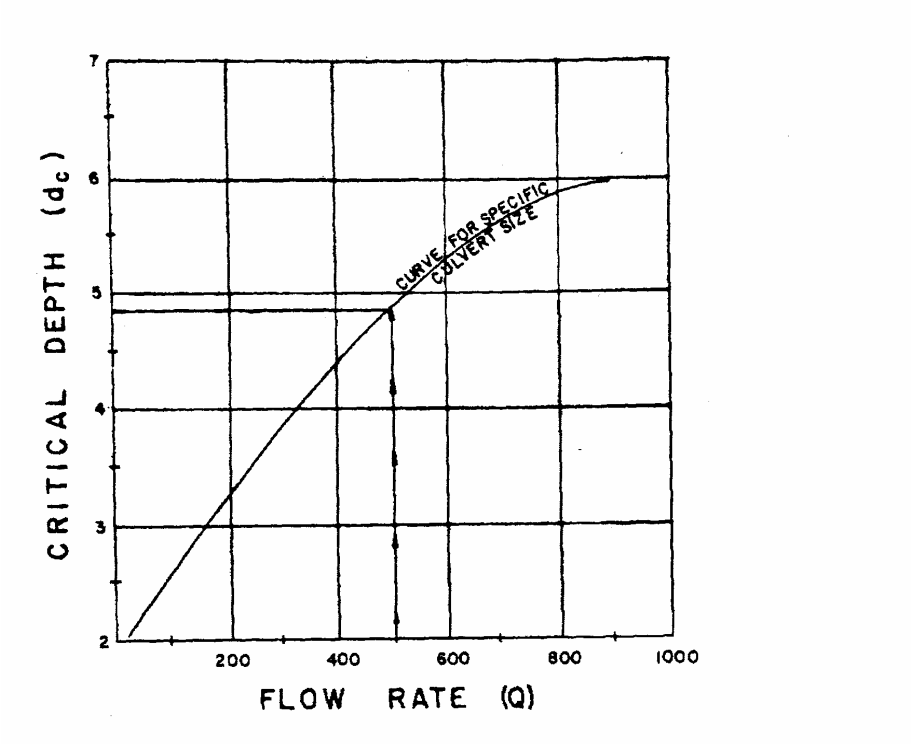
Reference: HDS-5, FHWA

Figure 5.10.12 Outlet Control Nomograph for Concrete Box Culverts



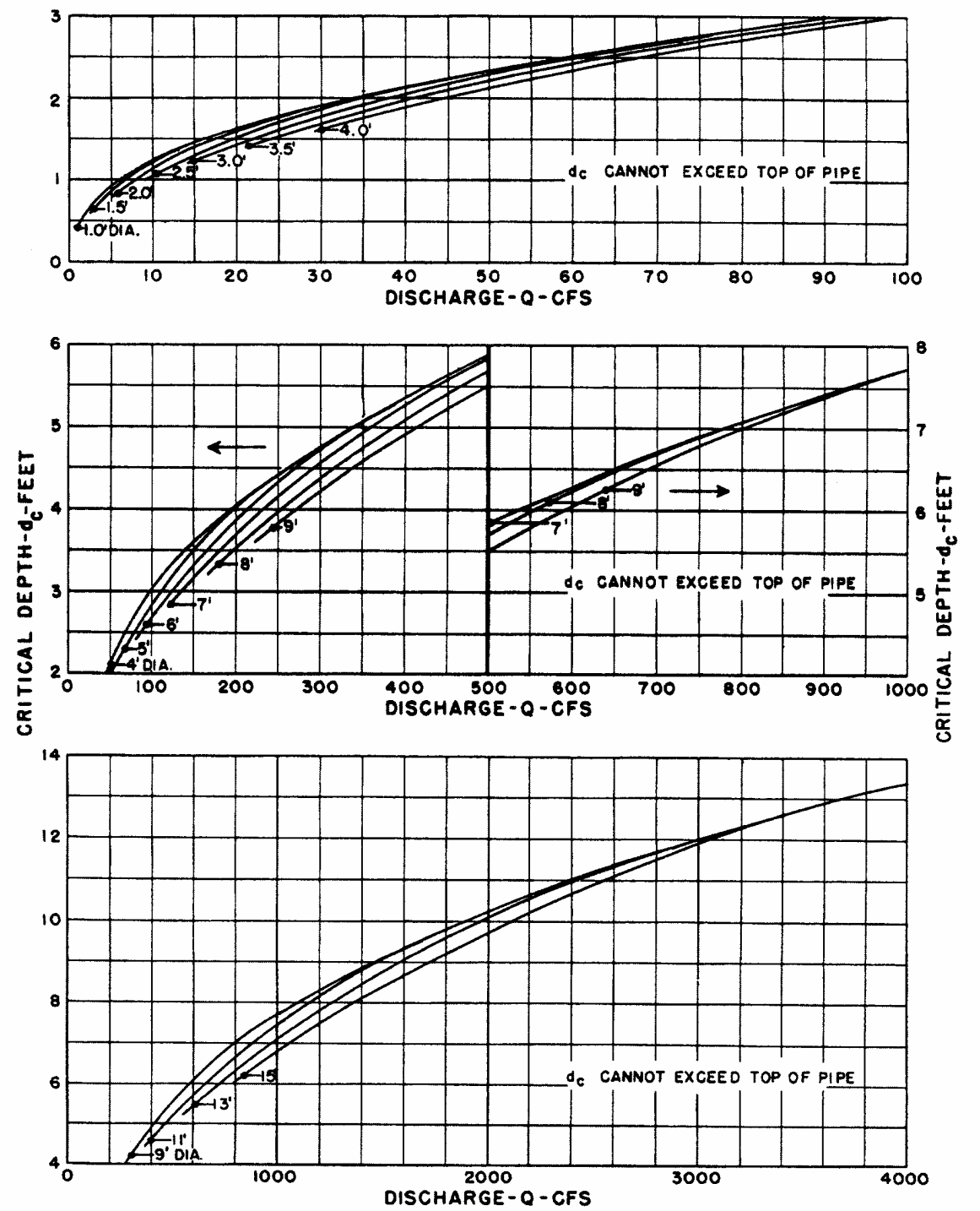
Reference: HDS-5, FHWA

Figure 5.10.13 Critical Depth Example Chart



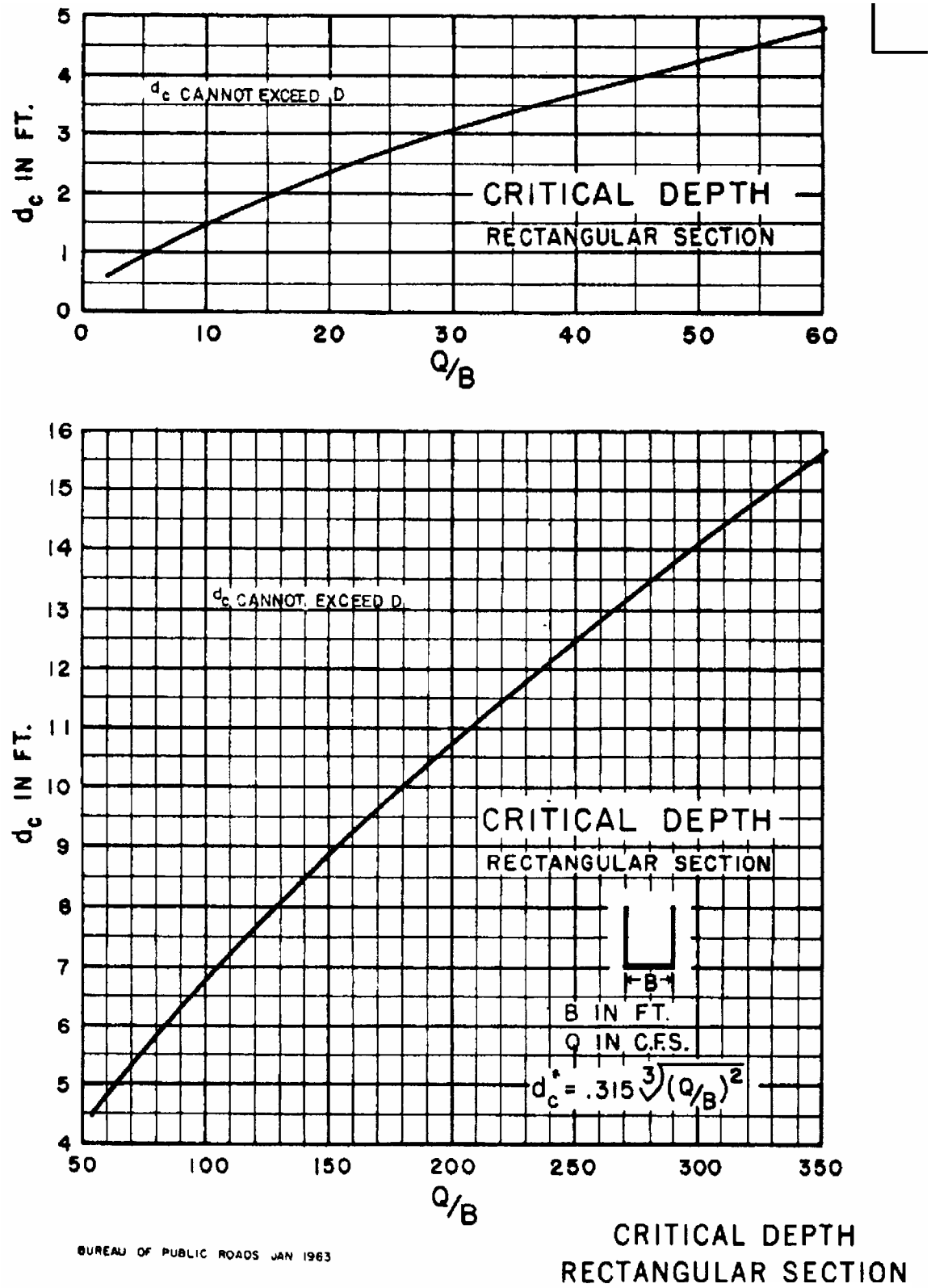
Reference: HDS-5, FHWA

Figure 5.10.14 Critical Depth Chart for Circular Pipes



Reference: HDS-5, FHWA

Figure 5.10.15 Critical Depth Chart for Rectangular Sections



Reference: HDS-5, FHWA

5.10.3.6 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 designated 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 1.2*D*, 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 0.75*D*, backwater calculations are required.

After inlet or outlet control is determined and design parameters are identified, there are some cases for which further design modifications may be required to finalize the design to accommodate topography, building constraints, utility conflicts, or other site conditions. To arrive at a final design it may be necessary to try several combinations of entrance types, invert elevations, and pipe diameters to determine the most economic and effective design that will meet the conditions of the site.

5.10.3.7 Outlet Velocity Calculation

The outlet velocity is 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.

5.10.3.8 Computer Applications

Although the nomographs discussed in this Section are still used, engineers are increasingly designing culverts using computer applications. Among these applications are the FHWA's HY8 Culvert Analysis and numerous proprietary applications. If a computer application other than HY8 is to be used, the designer must first submit documentation of the program to the Public Works Department for approval.

5.10.3.9 Outlet Protection

Table 5.7.3 of Section 5.7, Open Channels, presents maximum permissible mean channel velocities for various types of channel linings. A horizontal riprap-lined apron is required for all culvert outlets when the outlet velocity exceeds the values presented in **Table 5.7.3**. For culverts less than or equal to 36 inches in diameter or equivalent open area and outlet velocities less than 15 fps, the following procedure should be used. For larger culverts or

outlet velocities greater than 15 fps, the outlet protection design provided for in USDOT, 1983 should be used.

The length of the apron, L_a, is determined using the following empirical relationships:

$$L_a = \frac{1.8Q}{D_0^{3/2}} + 7D_0 \text{, for } TW < \frac{D_0}{2} \qquad (5.10.3)$$

and

$$L_a = \frac{3Q}{D_0^{3/2}} + 7D_0 \text{, for } TW > \frac{D_0}{2} \qquad (5.10.4)$$

Where:

D_0 = maximum inside culvert width (ft)
Q = pipe discharge (cfs)
TW = tailwater depth (ft)

Where there is no well defined channel downstream of the apron, the width, *W*, of the apron shall be as follows, as shown in **Figure 5.10.16**:

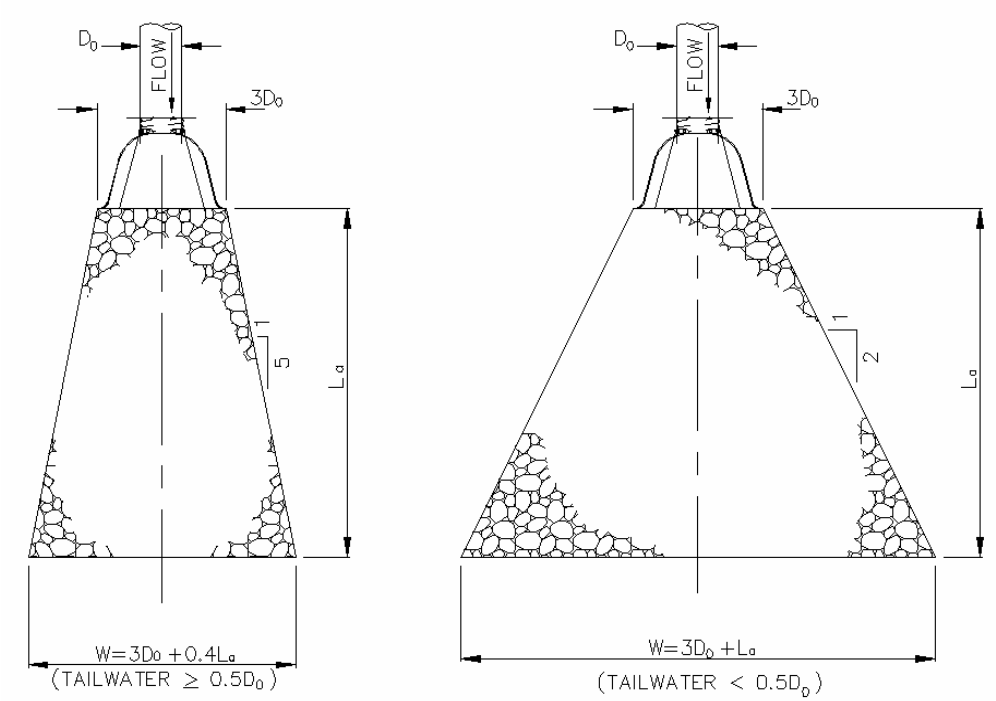
$$W = 3D_0 + 0.4L_a \text{, for } TW \geq \frac{D_0}{2} \qquad (5.10.5)$$

and

$$W = 3D_0 + L_a \text{, for } TW < \frac{D_0}{2} \qquad (5.10.6)$$

The width of the apron at the culvert outlet shall be at least 3 times the culvert width.

Figure 5.10.16 Culvert Outlet Protection Configuration



Reference: US EPA (modified to show flared end section)

Where there is a well-defined channel downstream of the apron, the bottom width of the apron shall be at least equal to the bottom width of the channel and the lining shall extend at least one foot above the tailwater elevation and at least two-thirds of the vertical conduit dimension above the invert.

The side slopes shall be 2:1 or flatter, the bottom grade shall be level, and there shall be a toewall at the end of the apron or culvert.

The riprap median stone diameter required, d_{50}, is determined from the following equation:

$$d_{50} = \frac{0.02(Q)^{4/3}}{TW(D_0)} \qquad (5.10.7)$$

Preformed scour holes may be used where flat aprons are impractical. **Figure 5.10.17** shows a general design of a scour hole. The stone diameter is determined using the following equations:

$$d_{50} = \frac{0.0125(Q)^{4/3}}{TW(D_0)}, \text{ for } y = \frac{D_0}{2} \qquad (5.10.8)$$

and

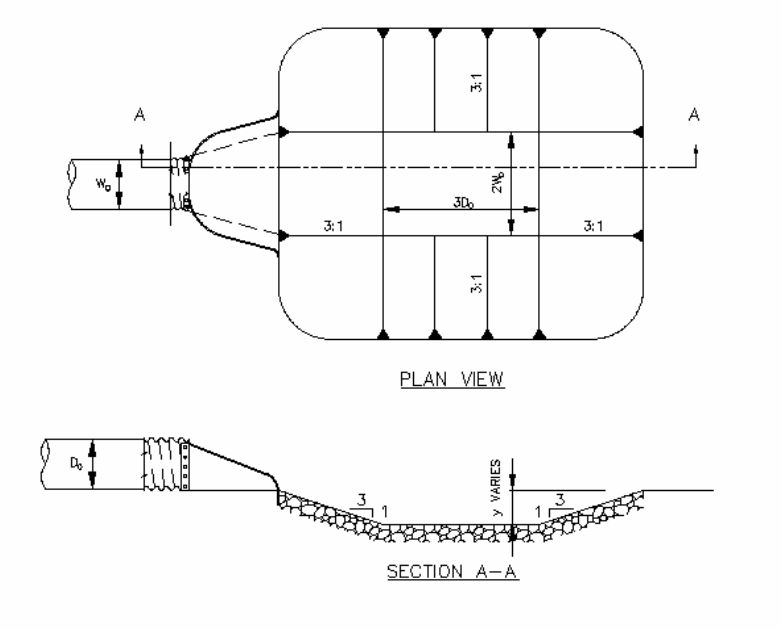
$$d_{50} = \frac{0.0082(Q)^{4/3}}{TW(D_0)}, \text{ for } y = D_0 \qquad (5.10.9)$$

Where y = depth of scour hole below culvert invert.

The gradation and materials for riprap shall be as specified in the CDOT Standards and Specifications. The CDOT gradation shall correspond to the d_{50} calculated as specified in this Section. If the calculated d_{50} is between two CDOT gradations, the larger to the two shall be used.

Also note that the riprap sizing calculations included here are for rocks that are angular with fractured faces, nearly rectangular in shape with a breadth or thickness at least 1/3 its length, as required in the City's Standard Specifications. Where these riprap materials are not available, rounded river rock may be used with some modifications. Channel side slopes shall be flattened to at least 3H:1V and the required gradation shall be increased by at least 25% when using river rock.

Figure 5.10.17 Preformed Scour Hole



Reference: Design and Construction of Urban Stormwater Management Systems, ASCE (modified to show flared end section)

5.10.4 CULVERT INLETS

One of the most important considerations in the design of a culvert is the inlet configuration. The culvert inlet edge usually represents a flow contraction and may be the primary flow control. Providing a more gradual flow transition will lessen the energy loss and create a more hydraulically efficient inlet condition. All concrete box culverts shall be designed with headwalls and wingwalls at the inlet and outlet.

A multitude of different inlet configurations are used on culvert barrels. These include both prefabricated and cast-in-place options. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope. Structural stability, aesthetics, erosion control, and fill retention should be considered in the selection of an inlet configuration. At a minimum, all public culverts shall have flared end sections at their inlet and outlet.

5.10.4.1 Projecting Inlets

Projecting inlets vary greatly in hydraulic efficiency and adaptability to requirements with the type of pipe material used. The primary advantage of projecting inlets is relatively low cost. Corrugated metal pipe projecting inlets have a low efficiency and are susceptible to damage. A projecting entrance of corrugated metal pipe is equivalent to a sharp-edged entrance with a thin wall and has an entrance coefficient of approximately 0.9 as given in **Table 5.10.1**. Bell-and-spigot concrete pipe or tongue-and-groove concrete pipe with the bell end or grooved end used as the inlet section are quite efficient hydraulically having an entrance coefficient of approximately 0.2. For concrete pipe that has been cut, the entrance is square edged, and the entrance coefficient is approximately 0.5. Projecting inlets are not allowed on public culverts.

5.10.4.2 Inlets with Headwalls

Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, and providing embankment protection against erosion. The relative efficiency of the inlet varies with the pipe material used. Corrugated metal pipe in a headwall is essentially a squared-edge entrance with an entrance coefficient of approximately 0.5. The entrance losses may be reduced by rounding the entrance. For tongue-and-groove or bell-end concrete pipe, little increase in hydraulic efficiency is realized by adding a headwall. The primary reasons for using headwalls are embankment protection and ease of maintenance.

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow. Wingwalls offer little increase in hydraulic efficiency regardless of the pipe material used, and their use should be justified by reasons other than an increase in hydraulic efficiency.

Headwalls and wingwalls are required where standard roadside grades cannot be achieved without them.

5.10.4.3 Tapered Inlets

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. Tapered inlets improve culvert performance by providing a more efficient control section (the throat). However, tapered inlets are not recommended for use on culverts flowing under outlet control because, in that case, a simple beveled edge is of equal benefit. The two most common improved inlets are the side-tapered inlet and the slope-tapered inlet. FHWA's HDS-5 provides guidance on the design of improved inlets.

5.10.5 <u>BRIDGES</u>

Based on hydraulic capacity requirements, bridges may be required to cross major open channels. Sizing the 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 City will review bridge designs based on the guidance in this Section, however, the designer is required to contact FEMA for additional requirements.

5.10.5.1 Hydraulic Analysis

The hydraulic analysis of bridges shall be completed in accordance with the FHWA *Hydraulics of Bridge Waterways*. Alternately, the City of Steamboat Springs also permits the use of two computer models, FHWA HY-4 and HEC-RAS, for bridge hydraulic analysis. If a computer model other than HY-4 or HEC-RAS is to be used, the designer must first submit documentation of the program to the City for approval.

5.10.5.2 Bridge Design Standards

The method of planning for a bridge opening begins with calculation of the channel's 100-year water surface profile without the presence of the bridge. The following criteria shall then be met:

1. The addition of the bridge to the channel shall cause no more than 1.0 foot of rise in the 100-year water surface elevation on the channel.
2. The 100-year water surface elevation within the bridge shall also be a minimum of 1.0 foot below the low chord of the bridge.
3. Where bridge abutments and foundations are located below the 100-year water surface elevation, concrete wingwalls at angles of 40 degrees to 60 degrees shall be tied to the existing side slopes to prevent erosion behind the abutments.
4. Where supercritical flow exists in a lined channel, the bridge shall have no influence on the flow. There shall be no encroachment into the 100-year water surface elevation.
5. The design and supporting calculations for both private bridges and low water crossings shall be prepared and certified by a Colorado Registered Professional Engineer.
6. In all instances, all bridges shall meet all applicable FEMA floodplain regulations.

SECTION 5.11
DETENTION

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5.11.1 INTRODUCTION

The main purpose of a detention basin is to store runoff and reduce peak discharge by allowing flow to be discharged at a slower, more controlled rate. This controlled discharge rate is the lesser of available downstream capacity and historic site runoff rates. Detention helps to control flood peaks in urbanized areas. Use of detention includes individual site options such as a channel or small landscaped basin and regional options serving multiple sites such as construction of a large pond or reservoir.

5.11.2 DETENTION VERSUS RETENTION

Stormwater storage reservoirs are either detention or retention basins. A detention basin detains water temporarily, releasing it slowly through a pipe or channel. Because of its ability to release flow during inflow, the required storage volume is reduced. Detention basins also have a positive means of outflow, eliminating problems that come with a residual pool. Alternately, a retention basin retains water without any release during inflow. Once the storm event is over, basin drainage may occur due to evaporation and percolation into the soil. The use of retention basins is not permitted within the City of Steamboat Springs.

5.11.3 HISTORIC FLOWS

Historic runoff from a site is generally the amount of runoff a site produced during a given storm prior to anything being constructed on the site. When there is no construction on a site (i.e., no man-made imperviousness), or when construction on a site was completed without an approved drainage study, and the Rational Method is being used to determine peak runoff, historic flow rates shall be calculated using the flow rates listed in **Table 5.11.1**. For the purposes of these criteria, when a site already has construction that was completed in conjunction with an approved drainage study, that construction may be considered part of the site's historic condition and historic flows for these sites shall be computed based on a composite C value determined in accordance with Section 5.6, Storm Runoff. 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 (as defined above) as specified in Section 5.6, Storm Runoff.

Total allowable peak runoff rates from a developed, redeveloped, or significantly remodeled site shall be the historic flow rates for the minor and major design storm events.

Table 5.11.1 Historic Flow Rates (cfs/acre)

CONTROL FREQUENCY	SOIL GROUP		
	A	B	C and D
Minor Storm	0.04	0.08	0.10
Major Storm	0.27	0.46	0.54

The predominant soil group for the watershed area tributary to the detention pond shall be used for determining the historic flow rates. Information on the soils in Steamboat Springs can be found in published SCS Soil Surveys.

Note that the allowable peak discharge from a detention pond will be less than the allowable peak runoff from the site as a whole unless the entire site drains to the detention pond.

5.11.4 LOCAL AND REGIONAL DETENTION FACILITIES

5.11.4.1 Local Detention Facilities

Local detention facilities are designed and built by developers or local property owners. The facilities typically serve a single development and are intended to allow development by protecting a site from existing flooding conditions or by protecting downstream property from increased runoff caused by the development. The outlet capacity is generally based on pre-development hydrology. For smaller developments, detention storage volume may be provided in the form of small landscaped basins.

New subdivisions that include multiple lots are required to provide a coordinated system of detention for the entire subdivision to minimize the number of detention facilities and maintenance requirements. Ideally this means one detention facility for the entire subdivision. However, based on topography, this may not be feasible and more than one detention facility may be required. In these cases, the number of detention facilities shall be minimized to the extent practicable, and individual facilities on each lot are not permitted.

An example of this is a housing development of 10 acres being divided into 10 properties. A single detention facility should be provided for all 10 acres of development in lieu of one for each property. Another example is a commercial shopping area. Instead of having multiple hard basin detention areas throughout the development, it is preferable to have the entire development drain to one larger, landscaped basin.

5.11.4.2 Regional Detention Facilities

Regional detention facilities are generally those facilities that control flow in major channels, are large in size, and are owned and maintained by public agencies. The purpose of these facilities is to significantly reduce downstream flows in order to maximize the capacity of existing systems and maintain flows at or below historic rates. The City does not currently have any regional detention facilities.

5.11.5 DETENTION FOR MANAGEMENT OF STORMWATER QUALITY

The primary purpose of detention facilities is to reduce peak flows, but they can also be adapted to enhance stormwater quality. Requirements for stormwater quality management are in Section 5.12, Water Quality Enhancement.

5.11.6 GENERAL CRITERIA

Detention ponds may not be required if the development site can be designed to generate equal to or less than historic flows or can demonstrate that there is adequate capacity in the downstream storm sewer system to convey additional site flows to the City's outfall (the Yampa River). Underground detention and rooftop detention are not allowed unless specific permission is granted from the City. Detention ponds should be designed as landscaped areas integrated into the site design with multiple provisions, a coordinated development design effort, and minimal maintenance.

A Colorado licensed professional engineer shall observe construction and post-construction activities and certify the pond was built according to the approved plans and specifications. The certification must be submitted and approved by the Public Works Department prior to issuance of any CO within the development.

5.11.6.1 Maintenance

All detention facilities must be designed to facilitate maintenance. All private facilities shall be regularly maintained by their owners. Detention facilities shall have maintenance access a minimum of 10 feet wide, and any turning radius shall be at least 30 feet along the inside edge of the access. The access shall have a maximum slope of 10 percent and a cross slope of two percent.

Typical detention pond maintenance includes removing accumulated sediment and debris, ensuring that facility outlets are not clogged and are otherwise functioning properly, and maintaining landscaped areas that may be incorporated into the facility, all on a regularly scheduled basis with a minimum annual frequency. Detention ponds should be inspected after each significant storm event to check that they are functioning properly.

5.11.6.2 Volume

Detention basins shall be designed for the minor and major storm events. The minimum required freeboard for detention facilities is 1 foot above the computed 100-year water surface elevation. An overlap in detention volume and water quality volume will be permitted for all storm events. The larger volume of the two shall be the design detention basin volume.

5.11.6.3 Runoff Criteria

Post-development peak runoff from a site may not be greater than historic runoff from a site for any storm event storm. Total site runoff is typically a combination of detention basin release and direct runoff, both of which must be considered. If direct runoff is allowed from the developed site, the sum of the direct runoff plus the release from the detention basin must not exceed historic runoff rates. Note that there is no minimum time in which the detention volume must drain, only that the allowable release rate must not be exceeded. A maximum of 5% of the total site area is allowed to contribute direct runoff.

It may be permissible to release flows from a site in excess of historic runoff rates if it can be demonstrated that the downstream facilities, all the way to the Yampa River, are adequate to accept the new peak flow rate. The new peak flow rate will be calculated assuming fully-developed conditions in the entire watershed in which the development is located. This will apply only to sites in close proximity to the Yampa River. Direct release of flows in excess of historic rates must be approved by the Public Works Department and supported by design calculations showing the adequacy of the system to manage the increased flow rates.

5.11.6.4 Geometric Requirements

For proper function and safety, there are several geometric requirements for detention basins. While 4H:1V side slopes are preferred, 3H:1V side slopes are permitted. Detention basins shall be sloped at a minimum of 2% towards the basin outlet to ensure adequate drainage. All basins shall include an access ramp a minimum of 10 feet wide sloped no steeper than 6H:1V to allow for routine maintenance. To promote pollutant removal, detention basins should have a length-to-width ratio not less than 2, with a ratio of 4 being ideal. A sedimentation forebay is also recommended to promote long-term functioning of the structure.

Access to both the forebay and pond by maintenance equipment is essential. All reservoirs or ponds which serve more than a single lot or site must also be provided a maintenance easement to allow a vehicle to access the basin. Maintenance of required volume and inflow and outflow works is necessary for the facility to function as required.

5.11.6.5 Bottom Drainage

Most drainage conveyance systems are designed to divert even minor nuisance flows to stormwater storage facilities. Conveyance facilities to and within a detention pond should be capable of transporting small flows all the way to the outlet facility rather than causing a soggy bog condition that cannot be properly maintained. Facilities shall be capable of conveying at least 0.5 cfs of trickle or nuisance flows.

5.11.6.6 Ground Cover and Landscaping

After final grading, the slopes and bottom of each detention basin shall be protected from erosion by seeding and mulching, sodding, or other approved ground cover within 30 days. The planting of native trees and shrubs on the slopes of storm water basins is also encouraged as long as they will not interfere with maintenance operations when fully mature. Plants native to the area should require no permanent irrigation. However, temporary irrigation shall be provided as required to ensure establishment of newly planted vegetation.

5.11.6.7 Inlet and Outlet Design

The inlet to a detention pond can be by way of surface inlets and/or by a local private storm sewer system. The inlet to the detention pond shall be designed so as not to cause any degradation of the pond's banks or invert.

The outlet of a detention basin can be a single pipe or can be a more complex design including a combination of pipes, orifice plates, or overflow weirs. No outlet pipe shall be smaller than 12 inches in diameter. Multiple pipe outlets may be required to control different design storms. The invert of the lowest outlet pipe shall be set at the lowest point in the detention pond or at the minimum pool elevation, if present. The outlet pipe shall discharge into a standard manhole or into a drainageway with proper erosion protection. If orifice plates are required to control the release rates, the plates shall be hinged to open into the detention pond to facilitate back flushing of the outlet pipes.

An emergency overflow spillway shall be provided in the event the basin outlet becomes clogged or a storm larger than the major design storm occurs. The emergency overflow shall allow safe passage of the 100-year storm event so that there is no damage to the surrounding area or to downstream facilities. The invert of the emergency spillway should be set at or above the 100-year water surface elevation.

5.11.6.8 Drain Time

Detention basins shall be designed to drain so that the site's allowable release rate is not exceeded.

5.11.6.9 State Engineer's Office

Dams constructed for the purpose of storing water, with a surface area, volume, or dam height as specified in Colorado Revised Statues 37-87-105 as amended, shall require approval by the State Engineer's Office. These include dams which impound water above the elevation of the natural surface of the ground creating a reservoir with a capacity of more than 100 acre-feet, or which create a reservoir with a surface area in excess of 20 acres at the high-water line, or which exceed 10 feet in height measured vertically from the elevation of the lowest point of the natural surface of the ground along the longitudinal center line of the dam up to the bottom of

the emergency spillway. These facilities are subject to state statutes shall be designed and constructed in accordance with the criteria of the State.

5.11.7 HYDROLOGIC DESIGN METHODS AND CRITERIA

The method used for hydrologic design of detention facilities depends on whether a HEC model (HEC-1 or HEC-HMS) or the Rational Method is used to calculate runoff. If a HEC model is used, a full hydrograph is available for traditional storage routing. If the Rational Method is used, the modified FAA Method shall be used to estimate required detention volumes. These two methods are discussed below. In either case, both the minor and the major storms shall be analyzed.

5.11.7.1 HEC Method

HEC-1 and HEC-HMS may be used to develop inflow hydrographs for hydrologic basins of any size. Inflow hydrographs for detention basin design shall be based on ultimate development conditions. The HEC programs can calculate a hydrograph at any location in the watershed, but the designer must structure the data input file so that the proposed detention basin site is a hydrograph-routing or hydrograph-combining point.

After the inflow hydrograph has been developed and the allowable release rate has been determined, the required storage volume can be estimated. In order to calculate the required storage volume of a particular detention basin, the following additional information must be available or prepared:

1. Proposed outlet discharge versus elevation data for the proposed basin site
2. Proposed storage volume versus elevation data for the proposed basin site
3. Proposed drain time for the proposed basin site

The HEC computer programs can be used to determine the required storage volume and outlet design based on a reservoir routing procedure. Initial estimates of outlet size are made and the program is run. The output is reviewed and changes are made to the outlet configuration as needed until the desired degree of flood peak attenuation and an acceptable drain time are achieved.

5.11.7.2 FAA Method

When a HEC model is not used, the FAA Method shall be used to determine the minimum required detention pond volume. The FAA Method is a simplified hydrograph routing procedure that is appropriate for watersheds smaller than 200 acres that don't have multiple detention ponds or unusual watershed storage characteristics.

1. Determine the inflow volume by multiplying the peak flow rate by the time of concentration as calculated by the Rational Method.

$$V_i = (CiA)(T_c)(60\,\text{sec/min}) \qquad (5.11.1)$$

Where:

V_i = inflow volume (ft^3)
C = Rational Method runoff coefficient for the major or minor storm
A = watershed area draining to the detention pond (acres)
T_c = Rational Method time of concentration (min)

i = design rainfall intensity (in/hr)

2. Determine the outflow volume by multiplying the allowable release rate by the time of concentration.

$$V_o = (Allowable\,Release\,Rate)(T_c)(60\,\text{sec/min}) \qquad (5.11.2)$$

Where:

V_o = outflow volume (ft^3)

T_c = Rational Method time of concentration (min)

Allowable release rate shall be determined per this Section (cfs).

3. The required detention pond volume for each design storm is the difference between the inflow volume and the outflow volume at the design time of concentration and rainfall intensity.

If the entire site is not tributary to the detention pond, the allowable release rate from the detention pond must be decreased in order to compensate for site runoff that does not pass through the detention pond. The allowable release rate is calculated as the total site historic runoff rate minus the post-development undetained flow rate. The result will be that the post-development peak flow from the entire site will meet the historic runoff rate criteria. A maximum of 5% of the total site is allowed to bypass the detention pond.

5.11.8 HYDRAULIC DESIGN OF OUTLET WORKS

Hydraulic design data for sizing of detention facilities outlet works is detailed in this subsection. **Figure 5.11.1** at the end of this subsection shows two commonly used outlet configurations.

5.11.8.1 Weir Flow

The general form of the equation for horizontal crested weirs is:

$$Q = CL(H)^{3/2} \qquad (5.11.3)$$

Where:

Q = discharge (cfs)
C = weir coefficient (see **Table 5.11.2** below)
L = horizontal length (feet)
H = total energy head (feet)

Another common weir is the v-notch, whose equation is as follows:

$$Q = 2.5 \tan(\Theta/2)\, H^{5/2} \qquad (5.11.4)$$

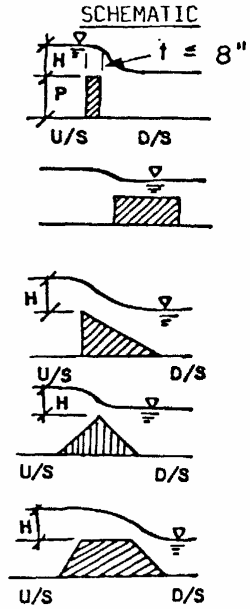
Where:

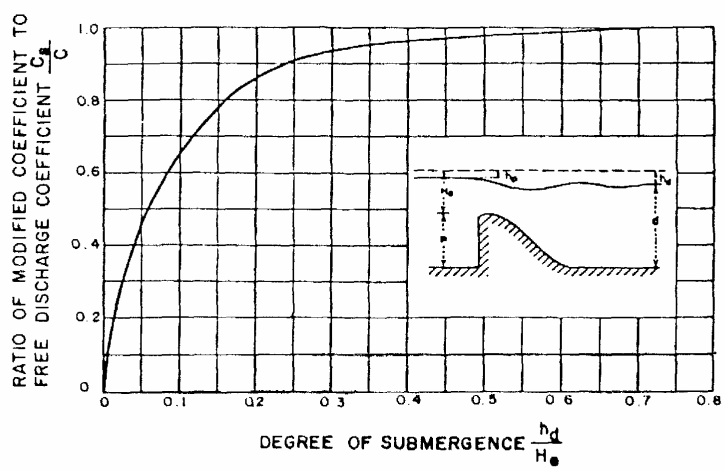
Θ = angle of the notch at the apex (degrees)

When designing or evaluating weir flow the effects of submergence must be considered. A single check on submergence can be made by comparing the tailwater to the headwater depth.

Table 5.11.2 Weir Flow Coefficients

SHAPE	COEFFICIENT	COMMENTS	SCHEMATIC
Sharp Crested	-		
Projection Ratio (H/P = 0.4)	3.4	H< 1.0	
Projection Ratio (H/P = 2.0)	4.0	H> 1.0	
Broad Crested	-		
W/Sharp U/S Corner	2.6	Minimum Value	
W/Rounded U/S Corner	3.1	Critical Depth	
Triangular Section	-		
A) Vertical U/S Slope	-		
1:1 D/S Slope	3.8	H > 0.7	
4:1 D/S Slope	3.2	H > 0.7	
10:1 D/S Slope	2.9	H > 0.7	
B) 1:1 U/S Slope	-		
1:1 D/S Slope	3.8	H >0.5	
3:1 D/S Slope	3.5	H >0.5	
Trapezoidal Section			
1:1 U/S Slope, 2:1 D/S Slope	3.4	H > 1.0	
2:1 U/S Slope, 2:1 D/S Slope	3.4	H > 1.0	
Road Crossings			
Gravel	3.0	H > 1.0	
Paved	3.1	H > 1.0	





ADJUSTMENT FOR TAILWATER

Reference: King & Brater, Handbook of Hydraulics, 1963
Design of Small Dams, USBR, 1977

5.11.8.2 Orifice Flow

The equation governing the orifice opening and plate is the orifice flow equation:

$$Q = C_d A\,(2gh)^{1/2} \qquad (5.11.5)$$

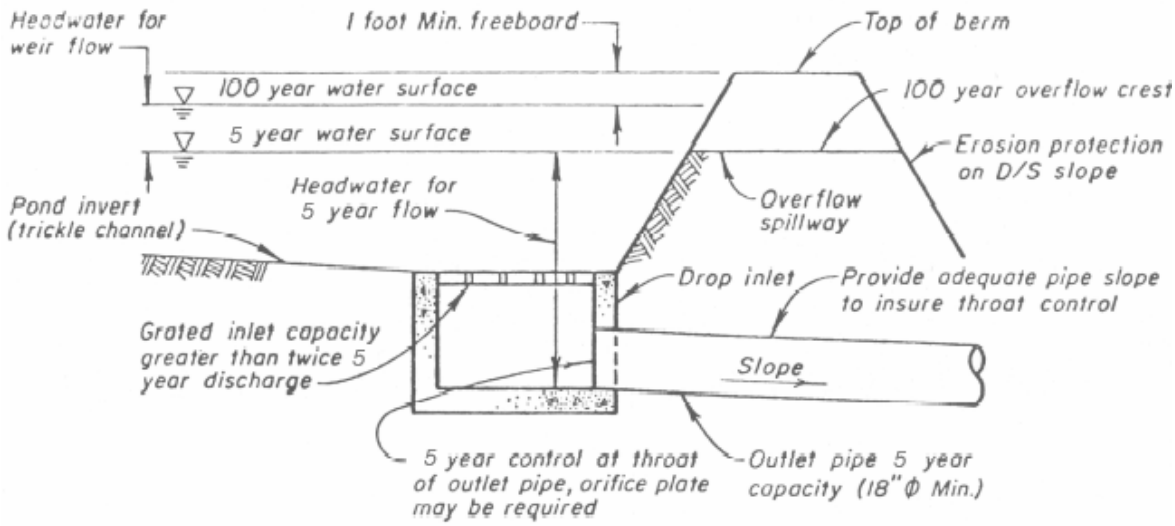
Where:

- Q = Flow (cfs)
- C_d = Orifice coefficient
- A = Area (ft^2)

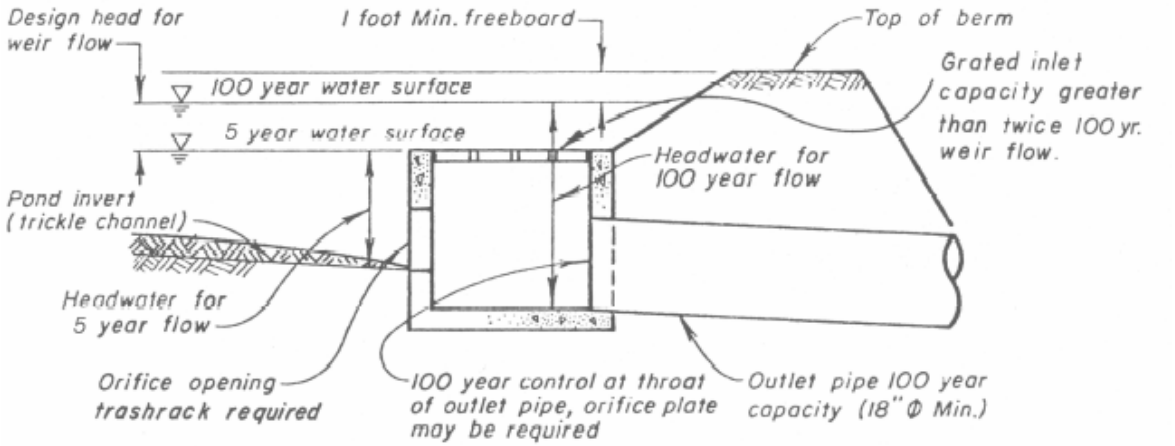
G = Gravitational constant = 32.2 ft/sec^2
h = Head on orifice measured from centerline (ft)

An orifice coefficient (C_d) value of 0.65 shall be used for sizing of squared edged orifice openings and plates.

Figure 5.11.1 Detention Pond Outlet Configurations



TYPE 1 OUTLET
No Scale



TYPE 2 OUTLET
No Scale

Reference: Prepared by WRC Engineering, Inc.

5.11.9 DESIGN EXAMPLE

Example 1: Detention Basin Sizing

A new development is located within a 23-acre watershed. When the development is complete the watershed will be 55 percent impervious. The predominant soil group in the basin is soil group C. The time of concentration to the inlet to the detention pond is 19 minutes. Find the major and minor storm storage volumes and release rates.

Step 1: Determine the allowable release rate for the major and minor storm using Table 5.11.1.

Allowable Release Rate for minor storm = (0.10 cfs/acre)(23 acres) = 2.30 cfs

Allowable Release Rate for major storm = (0.54 cfs/acre)(23 acres) = 12.42 cfs

Step 2: Calculate C values for the major and minor storms using Table 5.6.1.

With an imperviousness of 55 percent, C_5 is 0.43 and C_{100} is 0.62.

Step 3: Calculate rainfall intensity for the major and minor storms using Figure 5.5.1.

Based on a time of concentration of 19 minutes, i_5 is 1.60 in/hr and i_{100} is 2.95 in/hr.

Step 4: Calculate the inflow volume for the minor and major storms using Equation 5.11.1.

$V_{i(5)}$ = (0.43)(1.60 in/hr)(23 acres)(19 min)(60 s/min) = 18,039 ft^3

$V_{i(100)}$ = (0.62)(2.95 in/hr)(23 acres)(19 min)(60 s/min) = 47,956 ft^3

Step 5: Calculate the outflow volume for the minor and major storms using Equation 5.11.2.

$V_{o(5)}$ = (2.30 ft^3/s)(19 min)(60 s/min) = 2,622 ft^3

$V_{o(100)}$ = (12.42 ft^3/s)(19 min)(60 s/min) = 14,159 ft^3

Step 6: Calculate the required detention pond volumes for the major and minor storms as described in Section 5.11.7.2.

$V = V_i - V_o$

$V_{(5)}$ = 18,039 ft^3 – 2,622 ft^3 = 15,417 ft^3 = 0.35 acre-feet

$V_{(100)}$ = 47,956 ft^3 - 14,159 ft^3 = 33,797 ft^3 = 0.78 acre-feet

Step 7: Design the physical layout of the detention pond.

Assume a 5-foot water depth for the major storm. Assuming a constant pond side slope, the surface area of the pond at half the design depth is calculated as follows.

33,797 ft^3 / 5 ft = ~6800 ft^2

Assume a length-to-width ratio of 3:1. The shorter dimension of the pond at half the design depth will be:

√(6800/3) = ~48 feet

The longer dimension of the pond will be approximately 3 x 48' = 143' at the average design depth.

Assuming 4:1 side slopes, the floor of the pond will be 28' x 123'.

Step 8: Confirm the pond volume and top of bank. Assume the floor of the pond is at elevation 5800.00.

Water surface elevation (feet)	Dimensions (feet x feet)	Area (ft^2)	Incremental Volume (ft^3)	Cumulative Volume (ft^3)
5800	28' x 123'	3444		
5801	36' x 131'	4716	4080	4080
5802	44' x 139'	6116	5416	9496
5803	52' x 147'	7644	6880	16376
5804	60' x 155'	9300	8472	24848
5805	68' x 163'	11084	10192	35040

Interpolating, the 100-year volume of 33,797 ft^3 is provided at elevation 5804.88. The top of the detention pond must be at a minimum elevation of 5805.88 to provide the required 1 foot of freeboard.

Note that the above calculations offer only a simple example of the general procedure for laying out a detention pond. In reality, access ramps and water quality geometric considerations such as the forebay and bottom stage will alter the required geometry of the detention basin.

Example 2: Outlet Design

Design a Type 2 outlet (see Figure 5.11.1) for the detention pond in Example 1 for the minor and major storm.

Step 1: Interpolate the values in the table to determine the minor storm design water surface elevation.

$$\text{Minor Storm Water Surface} = \frac{\left(15417\,ft^3 - 9496\,ft^3\right)}{\left(16376\,ft^3 - 9496\,ft^3\right)}\left(5803 - 5802\right) + 5802$$

Minor Storm Water Surface = 5802.86

Step 2: Determine the minor storm orifice opening size and depth to the centerline of the orifice. Initially assume h = 2.0, putting the centerline of the orifice at 5800.86.

$A = Q / [(C_d)(2gh)^{1/2}]$ (Rearranged Equation 5.11.5)

$A = 2.3 / [(0.65)[(2)(32.2)(2)]^{1/2}]$

$A = 0.312\ ft^2$

Diameter = $(4A/\pi)^{1/2}$

Diameter = $(4*0.312/\pi)^{1/2}$

Diameter = 0.63 feet (7.56 inches)

The flow line of the orifice will be at: 5802.86 – 2.0 - (1/2)(0.63) = 5800.54

5800.54 is higher than the pond floor, so the initial assumption of h = 2.0 feet is physically possible. A 7.56-inch diameter orifice at flow line elevation 5800.54 is required at the entrance to the outlet box.

Alternately, an iterative process could be undertaken to arrive at an orifice diameter so that the flow line of the orifice is located at the exact pond floor elevation.

Step 3: Using Equation 5.11.5, determine the discharge though the minor storm outlet for the major storm headwater (h = 5804.88 – 5800.86 = 4.02 feet).

$Q = C_d A\ (2gh)^{1/2}$

$Q = (0.65)(0.312)[(2)(32.2)(4.02)]^{1/2}$

Q = 3.26 cfs

Step 4: Determine the discharge for sizing the major storm weir:

Q_{weir} = 12.42 cfs – 3.26 cfs

Q_{weir} = 9.16 cfs

Step 5: Size the orifice plate for the major storm outlet assuming an 18-inch outlet pipe with its flow line at the pond floor elevation (h = 5804.88 – (5800 + 18”/2) = 4.13 feet).

$A = Q / [(C_d)(2gh)^{1/2}]$ (Rearranged Equation 5.11.5)

$A = 12.42 / [(0.65)[(2)(32.2)(4.13)]^{1/2}]$

$A = 1.17\ ft^2$

$D = (4A/\pi)^{1/2}$

$D = [(4)(1.17)/\pi]^{1/2}$

D = 1.22 feet (14.65 inches)

The plate requires a 14.65-inch diameter orifice centered on the 18-inch outlet pipe.

Step 6: Determine the minimum box dimensions (i.e., the weir length) to assure control of the pipe inlet.

Based on H/P = 2.02/2.86 = 0.71, C for a sharp crested weir is interpolated to be 3.51 from the values in Table 5.11.2.

$L = Q_{weir} / (C(H)^{3/2})$ (Rearranged Equation 5.11.3)

$L = 9.16 / (3.51(2.02)^{3/2})$

L = 0.91 feet

Since the required weir length is only 0.91 feet, the selected box dimensions should be suited to construction and maintenance access. A minimum size of 3’ x 3’ is recommended.

SECTION 5.12
WATER QUALITY ENHANCEMENT

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Figure 5.12.7 Excess Runoff from 0.5” Rainfall with an SCS Type II 24-Hour Distribution

5.12.1 INTRODUCTION

Stormwater management has historically focused solely on ways to reduce the frequency and severity of flooding. There is another aspect of stormwater management, however, that has been given attention relatively recently, and that is water quality management. Stormwater management is environmentally important for all communities, but particularly to river communities like Steamboat Springs. A water quality program is essential to help keep streams and drainageways healthy and to minimize impacts from pollutants and debris.

To determine the appropriate best management practices (BMPs) for a site, the potential pollutants must first be identified. The most common pollutant in the Steamboat Springs area is sediment, and most BMPs will be focused on managing sediment. The volume of sediment requiring treatment can be significantly reduced by minimizing disturbed areas as well as the amount of directly-connected impervious area. Other common pollutants to be treated are oil, gas, grease from automobiles, and scoria from snow management practices. Once the potential pollutants are identified, site operations and proposed land uses can then be reviewed to determine where BMPs can be incorporated into the site. The focus should be on providing BMPs that are integrated with site operations and are part of the site design. For larger developments, centralized BMPs shall be designed for the entire site during the development process. Remote and isolated BMPs are not permitted as they use more land, have less economy of scale, and require greater maintenance.

The BMPs listed in this Section are based on typical site characteristics, common pollutants, and typical site designs. All BMP design shall consider the winter climate as BMPs may be required to operate during freeze/thaw cycles. This Section presents structural and site planning BMPs that are to be used both during and after construction. Refer to the latest edition of the *Urban Storm Drainage Criteria Manual* for additional descriptions and detail regarding stormwater quality facility design.

The designer shall submit a Stormwater Quality Plan to the City as Exhibit A of the Conceptual and Final Drainage Studies and the Drainage Letter (when required) as outlined in Section 5.3, Stormwater Planning and Submittals.

5.12.2 PERMANENT BMP FACILITIES

Each development will strive to reduce runoff and maximize stormwater infiltration by minimizing continuous impervious area and preserving riparian habitat by utilizing broad, shallow drainageways, open areas, and existing natural channels. In addition to this, any new development or redevelopment of property that disturbs one acre or more shall also implement permanent post-construction BMPs on site to control the discharge of pollutants after construction is completed. Permanent BMPs may be structural or non-structural. Structural BMPs include natural facilities such as ponds and swales and proprietary facilities such as vortex-type mechanical manholes. Private site BMPs shall be installed and maintained by the site and shall be outside the public right-of-way and offline from public stormwater systems.

An individual site that is smaller than one acre is still required to implement stormwater quality practices if it is part of a subdivision or larger development that is more than one acre. Sites that repair or replace parking lots not in conjunction with a redevelopment are encouraged to upgrade the site to include BMPs.

When a development or redevelopment site requires a detention facility, this facility may be designed to serve as both the detention pond and the stormwater quality facility. Detention facilities that are also water quality facilities shall have outlets to provide for both the

stormwater quality and detention release rates. See Section 5.11, Detention, for design requirements for detention ponds.

Sites large enough to require a volume-based BMP (see below) must develop centralized facilities that will serve the entire development. Numerous remote BMPs will not be permitted.

5.12.2.1 Minimum Structural BMPs

The following natural structural BMPs are acceptable for use in Steamboat Springs. The design of the first four is based on the design water quality capture volume (WQCV). They capture and temporarily detain the WQCV then release it over a period of time. The design of the last two is based on the design rate of runoff from the site. They all allow sufficient time for pollutants, mostly sediment, to settle out. The effectiveness of each is discussed in greater detail in the Urban Drainage and Flood Control District's *Urban Storm Drainage Criteria Manual, Volume 3 – Best Management Practices*, 2007 (USDCM).

Figure 5.12.1 Porous Pavement Detention (with under-drain) (PPD)

Figure 5.12.2 Porous Landscape Detention (with under-drain) (PLD)

Figure 5.12.3 Extended Detention Basin (EDB)

Figure 5.12.4 Constructed Wetland Basin (CWB)

Figure 5.12.5 Grass Buffer (Filter Strip) (GB/FS)

Figure 5.12.6 Grass Swales (GS)

Small sites (less than 1 acre) that are adjacent to a grass-lined roadside ditch may use the ditch as a grass swale BMP provided 1) the roadside ditch meets all USDCM criteria for a grass swale BMP, 2) there is adequate hydraulic capacity in the ditch per Section 5.7, Open Channels, 3) the identified potential pollutants are only those associated with parked vehicles, and 4) space constraints limit opportunity to utilize another kind of BMP. Under-drains shall be included with both porous detention BMPs unless it can be demonstrated that the subsurface soils provide adequate infiltration.

Examples of these BMPs can be found in:

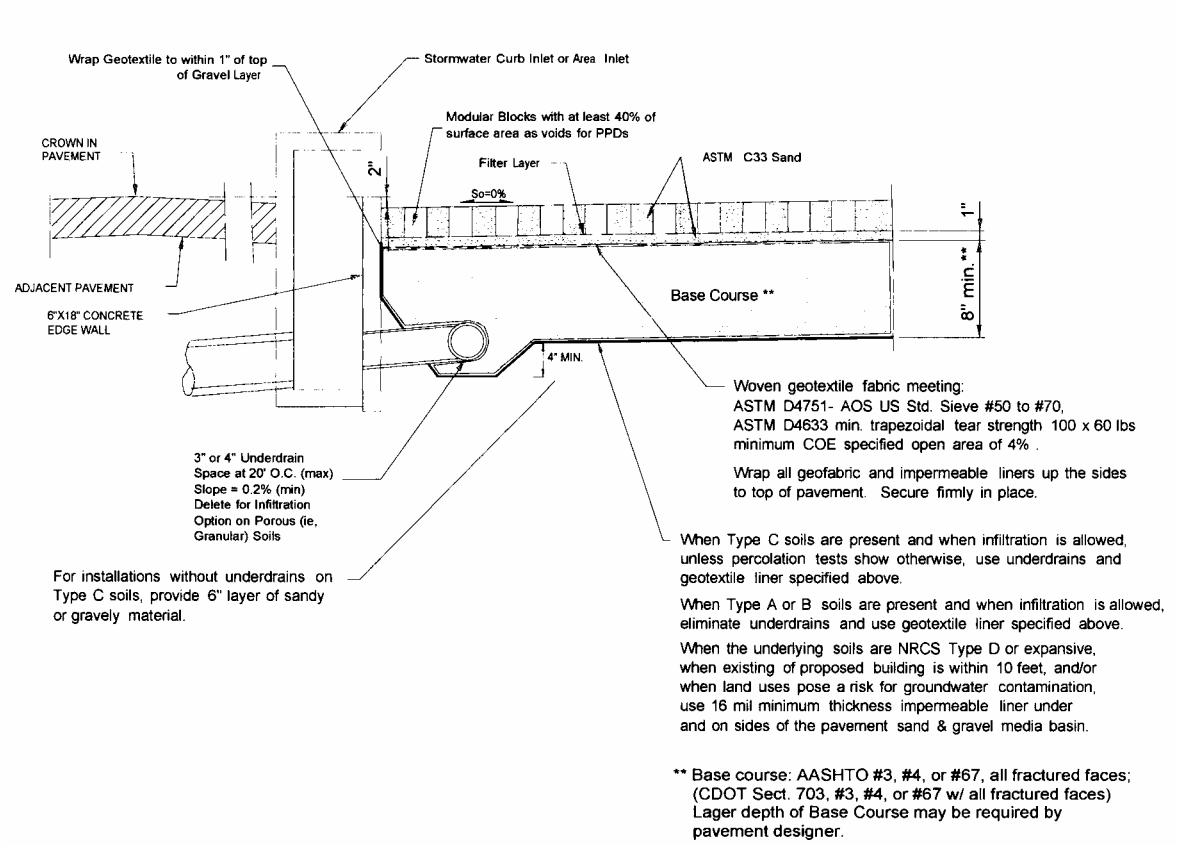
a. Urban Drainage & Flood Control District (UDFCD) 2007. *Urban Storm Drainage Criteria Manual (USDCM), Volume 3 - Best Management Practices*. This document is available at http://www.udfcd.org/usdcm/vol3.htm.

b. Water Environment Federation (WEF) and American Society of Civil Engineers (ASCE) 1998. *Urban Runoff Quality Management. WEF Manual No. 23, ASCE Manual No. 87.*

c. EPA 2002. *Consideration in the Design of Treatment Best Management Practices (BMP) to Improve Water Quality*. EPA 600 R-03/103. This document is available at http://www.epa.gov/ORD/NRMRL/pubs/600r03103/ 600r03103chp5.pdf.

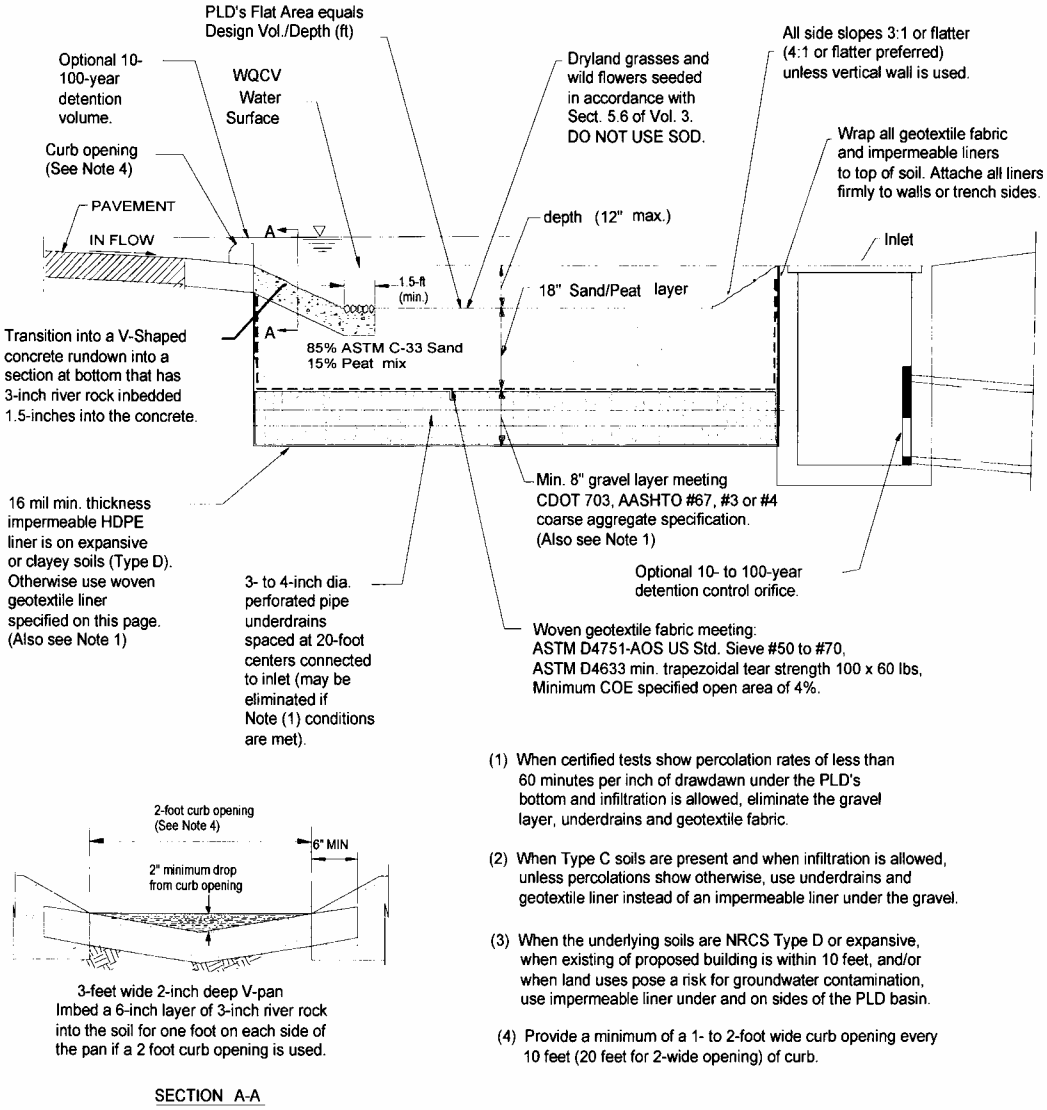
Use of alternate BMPs not specified above is subject to approval by the City.

Figure 5.12.1 Porous Pavement Detention (with under-drain) (PPD)



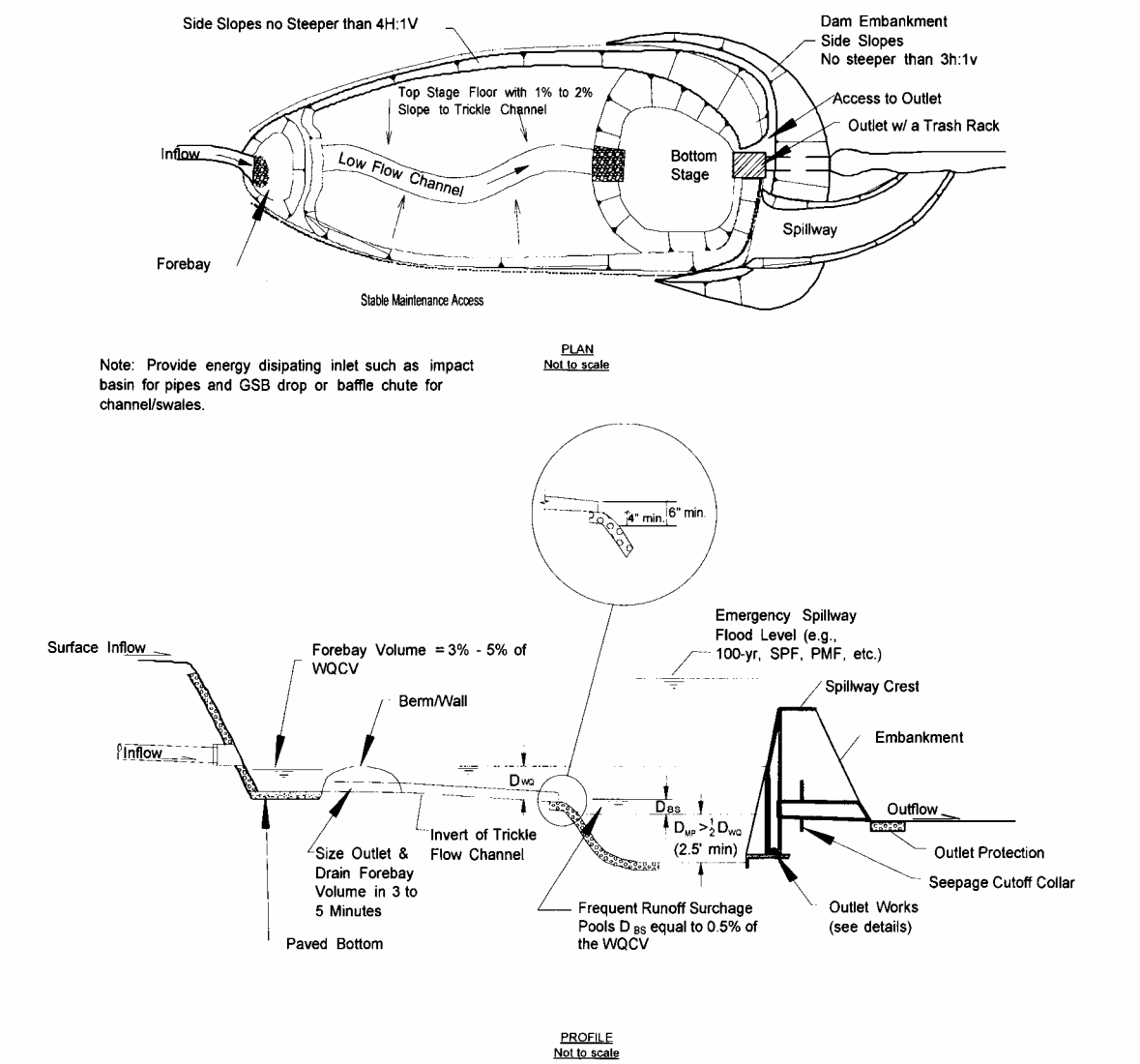
Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 3.

Figure 5.12.2 Porous Landscape Detention (with under-drain) (PLD)



Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 3.

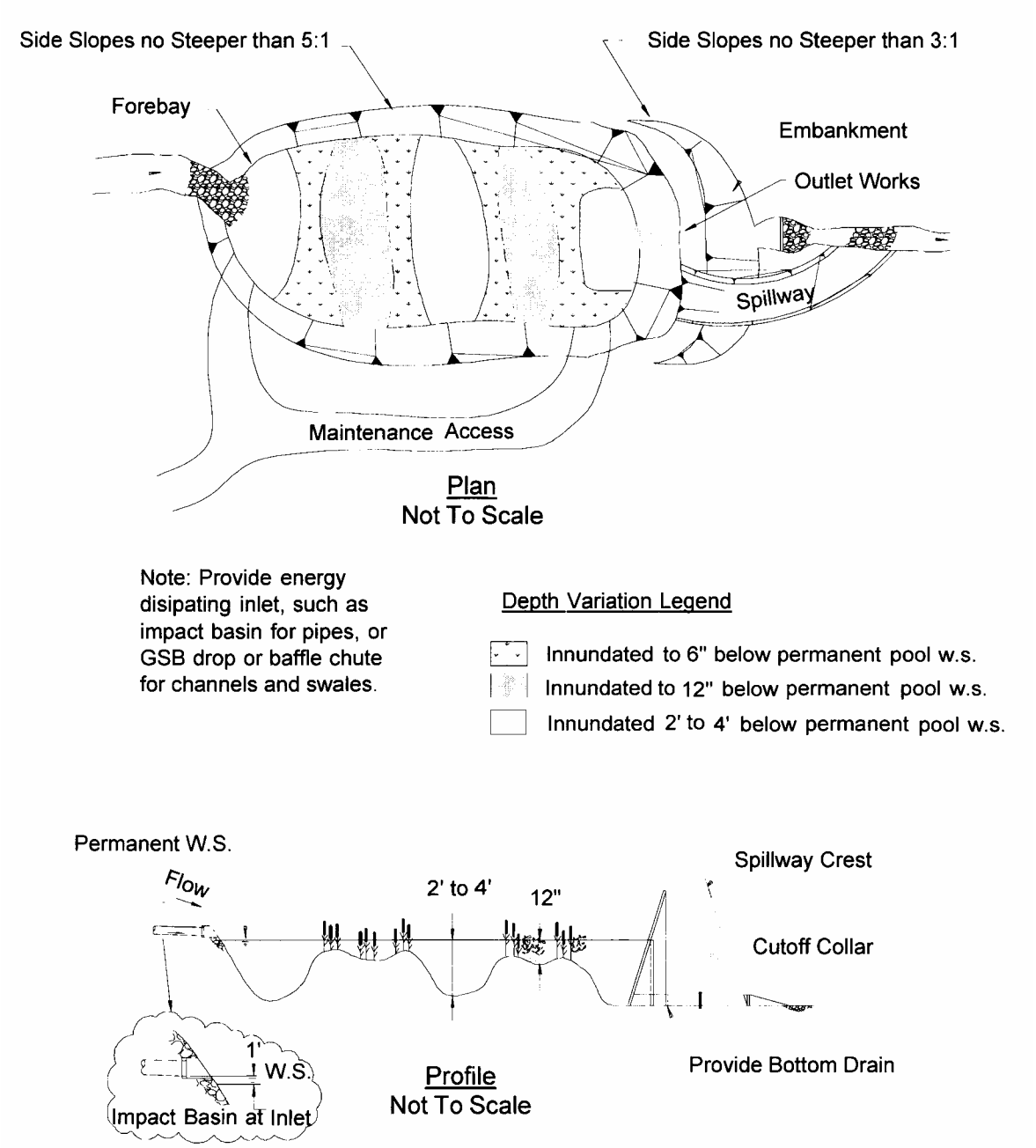
Figure 5.12.3 Extended Detention Basin (EDB)



Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 3.

When an extended detention basin is used as a structural BMP, the City does not require the use of a bottom stage or mircopool. All other details should be designed to meet the criteria presented in the USDCM.

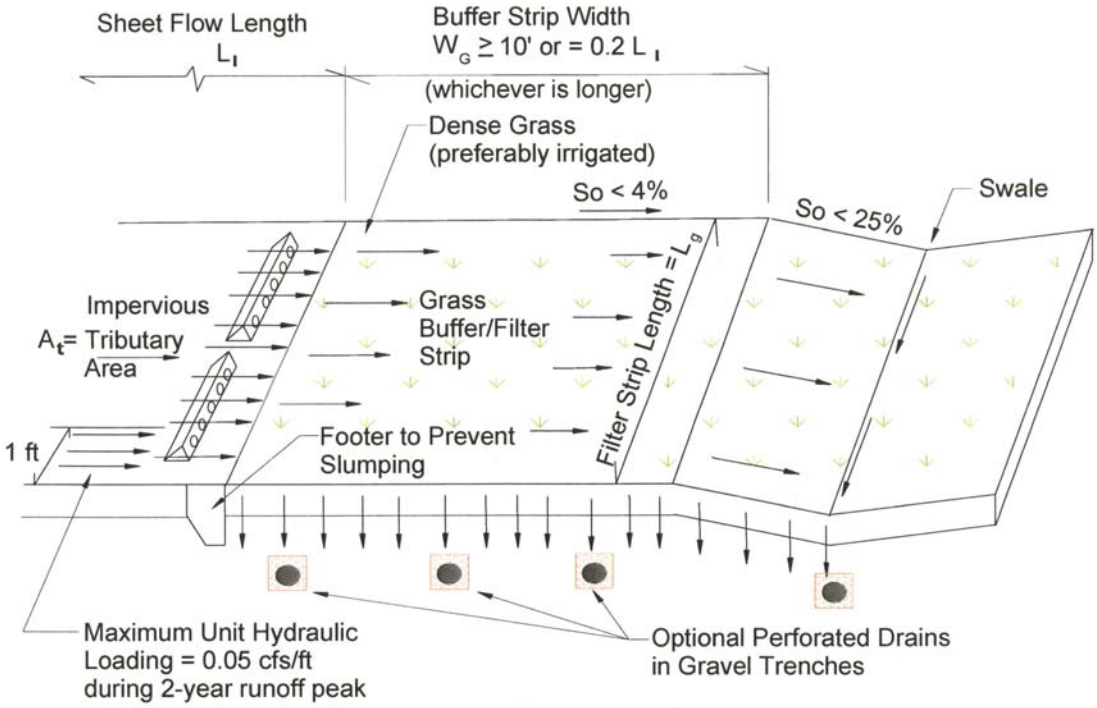
Figure 5.12.4 Constructed Wetland Basin (CWB)



Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 3.

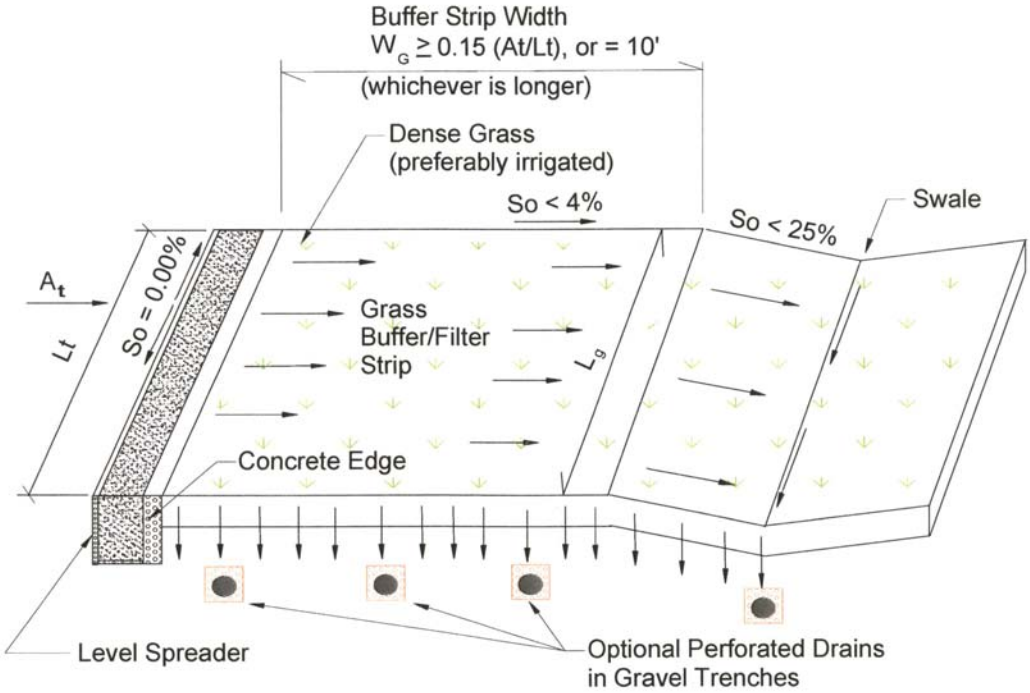
In some cases, a constructed wetland basin may be used for wetland mitigation as well as for water quality and detention. Any area of the constructed wetland basin not intended to ever have the ground disturbed as part of maintenance or other operations can be counted towards a required wetland mitigation area.

Figure 5.12.5 Grass Buffer (Filter Strip) (GB/FS)



SHEET FLOW CONTROL

N.T.S

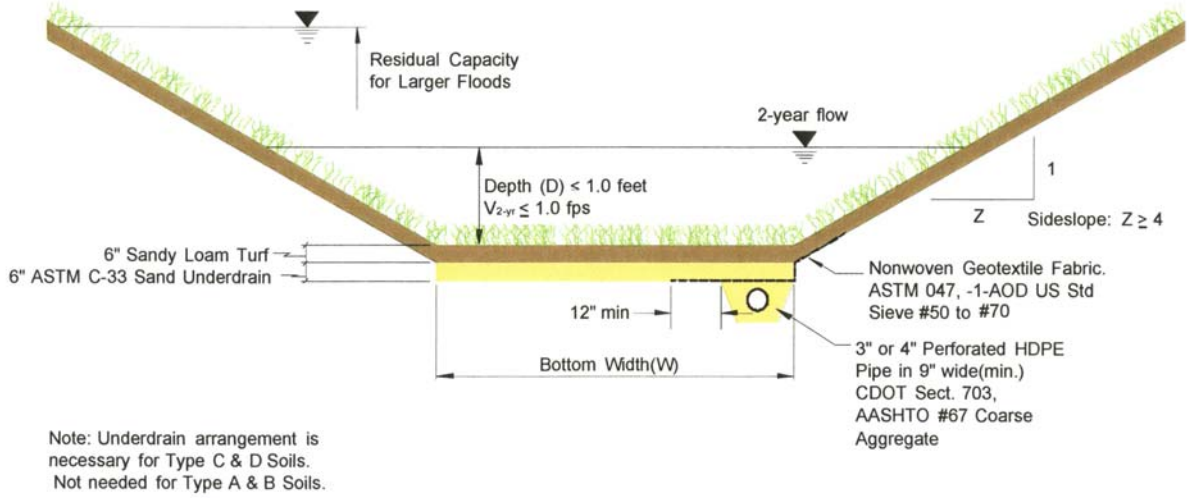


CONCENTRATED FLOW CONTROL

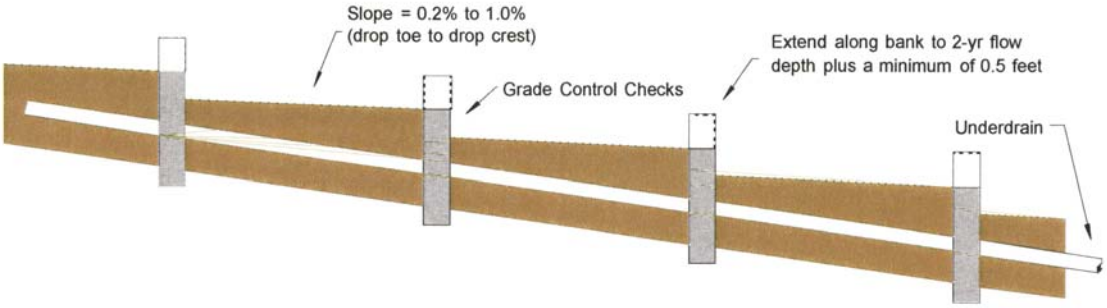
N.T.S

Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 3.

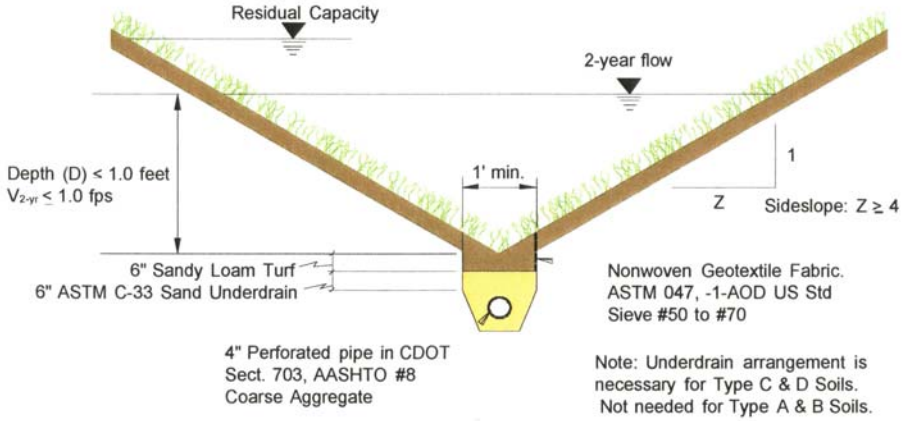
Figure 5.12.6 Grass Swales (GS)



TRAPEZOIDAL GRASS-LINED SWALE SECTION
NOT TO SCALE



GRASS-LINED SWALE PROFILE
NOT TO SCALE



TRIANGULAR GRASS-LINED SWALE SECTION
NOT TO SCALE

Reference: UDFCD 2007. Urban Storm Drainage Criteria Manual, Volume 3.

5.12.2.2 Proprietary BMPs

Proprietary BMPs for stormwater management include items such as filtration devices and hydrodynamic separators. Most proprietary BMPs function by gravitational separation, vortex separation, filtration, or by screening and retaining pollutants within the system. More frequent cleaning of these devices may be required in order to maintain adequate performance and, for some devices, to prevent the release of accumulated pollutants.

Proprietary systems may be permitted on constrained sites that have insufficient area to provide the WQCV required for one of the six natural BMPs shown above. Proprietary systems must consider factors such as the cost of initial installation, maintenance, and the ability to assure long-term function. Proprietary BMPs are limited to commercial sites with less than 5 acres of tributary area and must provide for the 5-year flow rate or be a supplement to other BMPs which meet these requirements.

5.12.2.3 Maintenance

All drainage and water quality facilities must be designed to facilitate future maintenance. On-site facilities require regular inspection and maintenance by the property owner. The following is the minimum required maintenance schedule for various BMPs. Based on site conditions, the design engineer may require additional maintenance measures for a site. If a BMP is used that is not listed in the table, a maintenance schedule shall be created and followed.

Table 5.12.1 Minimum Required Inspection and Maintenance Schedule

Type of BMP	Maintenance Activity	Minimum Frequency
Detention/Retention		
EDB CWB	Cleaning and removal of debris after 2-year storm Repair of outlet control structure Repair of embankment and side slopes Harvest vegetation	Annually
	Removal of accumulated sediment from sediment storage areas when 60% of original volume is lost	Every 5 years
	Removal of accumulated sediment from main cells of pond once 50% of the original volume has been lost	Every 20 years
Infiltration Practices		
PPD PLD	Avoid sealing or repaving with non-porous materials	N/A
	Inspect for appropriate function	After Large Storm Events
	Ensure that paving area is clean of debris Ensure that paving dewaters between storms Ensure that the area is clean of sediments	Monthly
	Mow upland and adjacent areas and seed bare areas Vacuum sweep to keep the surface free of sediment	Every 3 months
	Inspect the surface for deterioration or spalling	Annually
Filtration Practices		
GS	Mowing and removing litter/debris Stabilizing eroded side slopes and bottom Managing nutrient and pesticide use Dethatching swale bottom and removing thatching Discing or aerating swale bottom	Annually

Type of BMP	Maintenance Activity	Minimum Frequency
	Scraping swale bottom and removing sediment to restore original cross section and infiltration rate Seeding or sodding to restore ground cover (use proper erosion and sediment control)	Every 5 years
GB/FS	Mowing and removing litter/debris Managing nutrient and pesticide use Aerating soil on the filter strip Repairing eroded or sparse grass areas	Annually

5.12.3 WATER QUALITY CAPTURE VOLUME

The WQCV requirements for structural BMPs are to be determined using the criteria adopted by the Northwest Colorado Council of Governments (NWCCOG), of which Steamboat Springs is a member. Removal of pollutants shall be accomplished by sizing detention basins to incorporate a WQCV emptying time of 40 hours for a 0.5-inch rainfall in 24 hours with no more than 50% of the stored water being released in 12 hours.

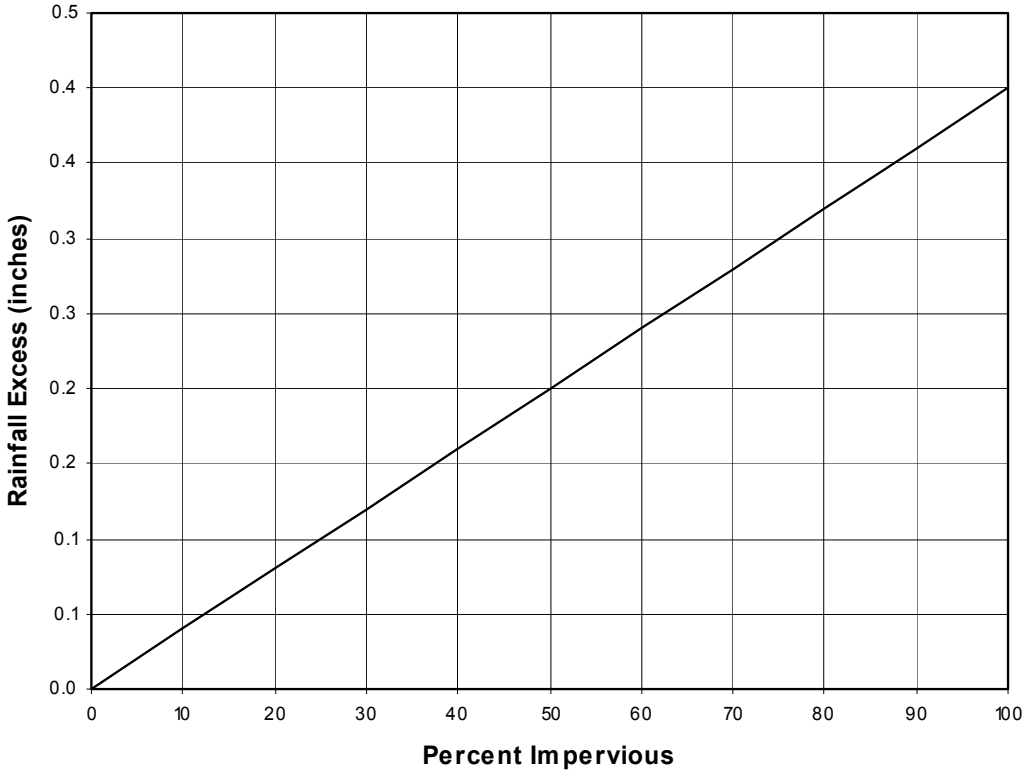
The WQCV can be calculated two different ways. If a HEC model is available that uses the location of the proposed water quality pond as a hydrograph-routing or hydrograph-combining point, a 0.5-inch rainfall with a 24-hour, SCS Type II Distribution can be specified and the total runoff volume to the pond can be generated. If a HEC model is not available, **Figure 5.12.7** may be used to approximate rainfall excess for the watershed area tributary to the pond. This figure assumes an initial abstraction of 0.1 inch, consistent with commonly accepted guidance. The WQCV will be the excess depth of rainfall multiplied by the watershed area draining to the pond.

The design of the outlet for the WQCV shall not be completed by routing flows using a HEC program, but rather by following the procedures detailed in the USDCM.

Required storage volume for water quality facilities shall be based on 120% of the calculated WQCV to allow for sediment accumulation. Where a water quality facility is also required to function as a detention facility, the volume required for detention does not need to be increased to account for the WQCV.

Some sites cannot be graded in a reasonable manner so that the entire site drains to the water quality facility. If at least 90% of the site drains to the water quality pond, the City may accept the design. Any area not draining to the water quality facility should have a very low pollutant load and should be graded to drain to vegetated areas prior to discharge from the site. No impervious area is permitted to discharge directly into wetlands, the Yampa River, or one of its tributaries

Figure 5.12.7 Excess Runoff from 0.5" Rainfall with an SCS Type II 24-Hour Distribution



5.12.4 SITE PLANNING BMPs

For existing and new development sites of one acre or more or smaller sites that are part of an overall development one acre or more, site planning BMPs shall be applied. All developments are encouraged to incorporate site planning BMPs into site operations both during construction and once construction is completed. Some of the site planning BMPs may be required by other regulatory agencies. Examples of site planning BMPs include the following controls or management of activities:

- Pesticides, Herbicides and Fertilizer Use
- Vehicle Washing
- Illicit Discharge
- Above Ground Storage Tanks
- Spill Prevention and Response
- Good Housekeeping
- Preventative Maintenance
- Loading and Unloading
- Painting Operations
- Fueling
- Outside Materials Storage
- Exposure Minimization
- Outside Manufacturing

The EPA requires a Spill Prevention, Control, and Countermeasure (SPCC) Plan for "facilities which may reasonably be expected to discharge oil" to streams or wetlands. Information on the EPA's requirements can be found at http://www.epa.gov/region8/compliance/spcc.html. The CDPHE requires spill prevention and containment measures to be identified in a

Stormwater Management Plan (SWMP) for several types of sites. Guidance can be found at: http://www.cdphe.state.co.us/wq/PermitsUnit/stormwater/index.html.

5.12.5 INDUSTRIAL AND COMMERCIAL ACTIVITIES

The Colorado Department of Public Health and Environment (CDPHE) is currently the authorizing agency for Stormwater Management Plans. Owners of industrial or commercial activities that have the potential to create illicit discharges due to contaminated material coming in contact with precipitation or stormwater are required to follow the applicable rules and permitting requirements of the State. There are several different types of permits covering different types of activities. See http://www.cdphe.state.co.us/wq/PermitsUnit/index.html for Colorado's water quality permitting requirements.

5.12.6 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 construction and stabilization of the site, unless they are designated to remain as permanent BMPs in the Stormwater Quality Plan. Temporary BMPs shall be identified on the Construction Site Management Plan that is required to be submitted in conjunction with a building permit. The Stormwater Quality Plan does not need to include temporary BMPs except for the following:

1. A general note that a Construction Site Management Plan is required with a building permit (by others)
2. A general discussion on how to manage the particular site to minimize the potential for erosion such as suggestions for phasing or minimizing exposed soil.
3. Design of temporary sedimentation ponds or other major erosion control measures for sites with an excavated area of greater than 10acres or sites adjacent to a wetland or major drainageway. Design shall meet the criteria presented in the USDCM.

For construction projects disturbing one acre or more, the U. S. Environmental Protection Agency (EPA) requires that the project owner/operator apply for a permit under the National Pollutant Discharge Elimination System (NPDES) program. In Colorado, the EPA has delegated management of this program to the Water Quality Control Division (WQCD) of the Colorado Department of Public Health and Environment (CDPHE). The Colorado Discharge Permit System (CDPS) General Permit for Stormwater Discharges Associated with Construction Activity (Permit No. COR-030000) is issued in compliance with the provisions of the Colorado Water Quality Control Act and the Federal Water Pollution Control Act. To fall under this permit, each project owner must submit an application to the CDPHE. For information on this permit and other permits that may be required during construction (such as construction dewatering) please see the state website at: http://www.cdphe.state.co.us/wq/PermitsUnit/stormwater/index.html

5.12.7 COMPLIANCE WITH CITY PERMIT

The City of Steamboat Springs obtained a permit to discharge stormwater under the Colorado Discharge Permit System, Permit #COR-090087. This permit requires the City to develop various stormwater management programs including those for construction, inspection, education, and land development so that pollutants are reduced in all stormwater runoff.

In addition, Section 5-3 of the Municipal Code outlines requirements for a Construction Site Management Plan and general requirements for erosion control. Any stormwater management program must be consistent with the City's Permit Program. As part of the City's Permit Program, construction site inspections will be conducted to assure compliance with the City Permit and individual site plans. Work that is not in compliance with the City Permit, site plan, or Municipal Code is subject to enforcement action as permitted by the Municipal Code.

5.12.8 CONSTRUCTION DISCHARGE PERMIT (CDP) REQUIREMENTS

All stormwater discharges from construction sites disturbing one-acre or more shall meet the following minimum standards:

1. Stormwater discharges from construction activities shall not cause or threaten to cause pollution, contamination or degradation of Waters of the State.
2. Concrete wash water shall not be discharged to State waters or storm sewer systems.
3. Bulk storage structures for petroleum products and other chemicals shall have adequate protection so as to contain all spills and prevent any spilled material from entering State waters.
4. All wastes composed of building materials must be removed from the site for disposal in licensed disposal facilities. No building material wastes or unused building materials shall be buried, dumped, or discharged at the site.
5. Off-site vehicle tracking of sediments shall be minimized.
6. Land disturbances shall be conducted in a manner to effectively reduce accelerated soil erosion and sedimentation.

5.12.8.1 Minimum Temporary BMP Requirements

Each construction site shall implement structural, non-structural, and planning measures. The primary goal of BMPs is to reduce erosion at the source, followed by trapping eroded materials before they leave the site. The following is required at a minimum:

1. BMPs that control erosion at the source, such as those that stabilize earth disturbances with vegetation or mulch after grading is substantially complete on any portion of the site not otherwise permanently stabilized. Typical methods include surface roughening, mulching, vehicle tracking control, and installation of blankets, straw wattles, tackifiers, netting, and matting.
2. BMPs that trap sediment before it leaves the site or enters the municipal storm sewer system, which ever comes first. Such BMPs shall be installed prior to initiating earth disturbances. Typical examples include check dams, inlet protection, sediment basins, and silt fence.
3. BMPs that prevent spills of petroleum products and other chemicals and contain storm runoff from construction wastes to a designated area, if applicable.
4. Construction sequencing for all BMPs to reduce the duration a disturbed area is exposed. Temporary disturbed areas shall be exposed no longer than 30-days. Disturbed areas that are to be permanently stabilized shall be exposed no longer than 7-days.
5. BMPs that capture and retain runoff from equipment washing operations, such as cleaning of concrete trucks.
6. Program and schedule for regular inspection and maintenance of BMPs.

7. Sites that have more than 1 acre of tributary area or are directly adjacent to a wetland or major tributary are required to construct temporary sedimentation ponds to help control the release of sediment from the site.

5.12.8.2 Acceptable BMPs

BMPs shall be selected for a specific site to address the minimum requirements above and shall consider the criteria for selection and design of BMPs found in the following documents:

1. Erosion and Sediment Control During Construction, Routt County, Colorado
2. Colorado Department of Transportation 2002. Erosion Control and Stormwater Quality Guide. This document is available along with CAD drawings for BMPs at *http://www.dot.state.co.us/environmental/envWaterQual/wqms4.asp*.
3. Urban Drainage & Flood Control District (UDFCD) 2007. *Urban Storm Drainage Criteria Manual (USDCM), Volume 3 - Best Management Practices*. This document is available at http://www.udfcd.org/usdcm/vol3.htm.
4. *EPA 2002. Consideration in the Design of Treatment Best Management Practices (BMP) to Improve Water Quality. EPA 600 R-03/103.* This document is available at http://www.epa.gov/ORD/NRMRL/pubs/600r03103/600r03103chp5.pdf.

The BMPs presented in the documents referenced above shall be used in the preparation of the Construction SWMP. Use of alternate BMPs not specified above is subject to approval by the City.

STANDARD FORMS

LIST OF TABLES

Standard Form No. 1 Drainage Letter Checklist
Standard Form No. 2 Conceptual Drainage Study Checklist
Standard Form No. 3 Final Drainage Study Checklist
Standard Form No. 4 Stormwater Quality Plan Checklist

Standard Form No. 1 Drainage Letter Checklist

Instructions:

1. The applicant shall identify with a "check mark" if information is provided with letter. If applicant believes information is not required, indicate with "N/A" and attach separate sheet with explanation.
2. The reviewer will determine if information labeled "N/A" is required and whether additional information must be submitted.

I. General

_____ A. Typed and legible in 8½ x 11" format.
_____ B. Drawings that are 8½" x 11" or 11 x 17 bound within letter, larger drawings (up to 24 x 36) included in a pocket attached to the letter. Drawings shall be at an appropriate size and scale to be legible and include project area.

II. Title Page

_____ A. Type of Letter.
_____ B. Project Name, Subdivision, Original Date, Revision Date.
_____ C. Preparer's name, firm, address, and phone number.
_____ D. Certifications, PE stamp, signature and date from licensed Colorado PE (for FINAL letter).
_____ E. "DRAFT" for 1st Submittal and revisions; "FINAL" once approved.
_____ F. Note: City of Steamboat Springs plan review and approval is only for general conformance with City design criteria and the City code. The City is not responsible for the accuracy and adequacy of the design, dimensions, and elevations that shall be confirmed and correlated at the job site. The City of Steamboat Springs assumes no responsibility for the completeness or accuracy of this document.

III. Introduction

_____ A. Description of site location, size in acres, existing and proposed land use, and any pertinent background info.
_____ B. Identify drainage reports for adjacent development.

IV. Drainage Criteria and Methodology Used

_____ A. Identify design rainfall and storm frequency.
_____ B. Identify runoff calculation method used.

V. Existing Conditions (Pre-Development/Historic)

_____ A. Indicate ground cover, imperviousness, topography, and size of site (acres).
_____ B. Describe existing stormwater system (sizes, materials, etc.).
_____ C. Describe other notable features (canals, major utilities, etc.).
_____ D. Note site outfall locations and ultimate outfall location (typically Yampa River).
_____ E. Note capacity of existing system and identify any constraints.
_____ F. Identify NRCS soil type.
_____ G. Identify the FEMA Map reviewed, if site is in floodplain/way, and zone designation.

VI. Proposed Conditions

_____ A. Indicate ground cover, imperviousness, topography, and disturbed area (acres).
_____ B. Describe proposed stormwater system (sizes, materials, etc.).
_____ C. Describe proposed outlets, and indicate historic and proposed flow for each.
_____ D. Include calculations for all pipes, inlets, culverts, ditches, ponds, etc. in appendix.
_____ E. Include a summary table for the 5- and 100-year events showing historic flow and proposed flow for total site and each basin.
_____ F. Include a summary of proposed water quality measures to be constructed.

VII. Conclusions

_____ A. Provide general summary.
_____ B. Note if site does or does not comply with criteria and any variances to criteria.
_____ C. Indicate if peak proposed flow is less than, equal to, or greater than peak historic flow for each outfall, design point, and for the total site.
_____ D. Indicate proposed stormwater quality system.

VIII. References

_____ A. Provide a reference list of all criteria, master plans, drainage reports and technical information used.

IX. Figures

_____ A. Vicinity Map.
_____ B. Site Plan (include the horizontal and vertical datum used and all benchmarks).
C. Existing conditions.
_____ 1. Delineate existing basin boundaries.
_____ 2. Show existing runoff flow arrows.
_____ 3. Show existing topography.
_____ 4. Show existing stormwater features (structures, sizes, materials, etc.).
_____ 5. Show floodplain limits and information.
_____ 6. For each basin, show bubble with basin number, acreage and percent impervious or provide information in summary table or figure.
_____ 7. For each outlet show bubble with acreage and historic flow and proposed flow or provide information in summary table on figure.
D. Proposed Conditions
_____ 1. Delineate proposed basin boundaries.
_____ 2. Show proposed runoff flow arrows.
_____ 3. Show existing and proposed topography at an interval of at least 5-ft.
_____ 4. For each basin show bubble with basin number, acreage and percent impervious or provide a summary table or figure.
_____ 5. For each outlet show bubble with acreage, historic flow, and proposed flow or provide a summary table or figure.
_____ 6. Show floodplain limits and information.
_____ 7. Show proposed stormwater system (components, sizes, materials, & slopes).
_____ 8. Show property lines and easements.
_____ 9. Show any new easements required.

X. Appendices

_____ A. Runoff Calculations
_____ B. Culvert Calculations
_____ C. Pond Calculations.
_____ D. Other Calculations

Acknowledgements:

Standard Form No. 1 was prepared by: _____ _____
Date

Standard Form No. 2 Conceptual Drainage Study Checklist

Instructions:

1. The applicant shall identify with a "check mark" if information is provided with letter. If applicant believes information is not required, indicate with "N/A" and attach separate sheet with explanation.
2. The reviewer will determine if information labeled "N/A" is required and whether additional information must be submitted.

I. General

_____ A. Typed and legible in 8½ x 11" format.
_____ B. Report bound (comb, spiral, or staple – no notebook).
_____ C. Drawings that are 8½" x 11" or 11 x 17 bound within letter, larger drawings (up to 24 x 36) included in a pocket attached to the letter. Drawings shall be at an appropriate size and scale to be legible and include project area.

II. Cover

_____ A. Report Type – Conceptual Drainage Study.
_____ B. Project Name, Subdivision, Original Date, Revision Date.
_____ C. Preparer's name, firm, address, phone number.
_____ D. "DRAFT" for 1st submittal and revisions; "FINAL" once approved.

III. Title Sheet

_____ A. Table of Contents
_____ B. Certification, PE Stamp, signature, and date from licensed Colorado PE.
_____ C. Note: City of Steamboat Springs plan review and approval is only for general conformance with City design criteria and the City code. The City is not responsible for the accuracy and adequacy of the design, dimensions, and elevations that shall be confirmed and correlated at the job site. The City of Steamboat Springs assumes no responsibility for the completeness or accuracy of this document.

IV. Introduction

_____ A. Description of site location, size in acres, existing and proposed land use, and any pertinent background info.
_____ B. Identify drainage reports for adjacent development.

V. Drainage Criteria and Methodology Used

_____ A. Identify design rainfall and storm frequency.
_____ B. Identify the runoff calculation method used.
_____ C. Identify culvert and storm sewer design methodology.
_____ D. Identify detention discharge and storage methodology.
_____ E. Discuss HEC-HMS methodologies and parameters, if HEC-HMS is used.

VI. Existing Conditions (Pre-Development/Historic)

_____ A. Indicate ground cover, imperviousness, topography, and size of site (acres).
_____ B. Describe existing stormwater system (sizes, materials, etc.).
_____ C. Describe other notable features (canals, major utilities, etc.).
_____ D. Note site outfall locations and ultimate outfall location (typically Yampa River).
_____ E. Note capacity of existing system and identify any constraints.
_____ F. Identify NRCS soil type.
_____ G. Discuss any existing easements.
_____ H. Identify the FEMA Map reviewed, if site is in floodplain/way, and zone designation.

VII. Proposed Conditions

_____ A. Indicate ground cover, imperviousness, topography, and disturbed area (acres).
_____ B. Describe proposed stormwater system (sizes, materials, etc.).
_____ C. Describe proposed outlets, and indicate historic and proposed flow for each.
_____ D. Include calculations for all culverts, ditches, ponds, etc. in appendix.
_____ E. Include a summary table for the 5- and 100-year events showing historic flow and proposed flow for total site and each basin.
_____ F. Discuss proposed easements.
_____ G. Describe offsite flows to be passed thru site.
_____ H. Summarize any impacts to downstream properties or indicate none.

I. Detention Ponds.
_____ 1. Indicate pond volume and area (size and depth) requirement.
_____ 2. Indicate release rates.
_____ 3. Discuss outfall design, location, and overflow location.
_____ 4. Discuss maintenance requirements.

J. Curb and Gutter
_____ 1. Indicate gutter capacity.
_____ 2. Indicate curb capacity.
_____ 3. Indicate design depth of flow in street.

K. Culverts
_____ 1. Indicate whether each culvert is under inlet or outlet control.
_____ 2. Show that headwater is less than the maximum allowable.
_____ 3. Indicate design velocity.
_____ 4. Indicate required and provided flow rates.
_____ 5. Discuss whether outlet protection is required and what will be used.

L. Inlets
_____ 1. Indicate inlet capacity.
_____ 2. Indicate the type of inlet(s) used.

M. Channels
_____ 1. Indicate design velocity (and type of dissipation if required).
_____ 2. Indicate required and provided flow capacity.
_____ 3. Show critical cross-section(s) including water surface.

N. Site Discharge
_____ 1. Discuss use and design of detention to ensure discharge is less than or equal to historic flow.
_____ 2. Provide documentation that downstream facilities are adequate and no adverse impacts to downstream property owners (i.e. no rise certification)

VIII. Post Construction Stormwater Management

_____ A. Discuss in general terms which permanent BMP practices will be used to control pollutant and sediment discharge after construction is complete. Exhibit A, Storm Water Quality Plan shall be attached that will give details (see separate checklist)

IX. Conclusions

_____ A. Provide general summary.
_____ B. Note if site complies with criteria and any variances to criteria.
_____ C. Indicate if peak proposed flow is less than, equal to, or greater than peak historic flow for each outfall, design point, and for the total site.
_____ D. List proposed new stormwater system requirements.

X. References

_____ A. Provide a reference list of all criteria, master plans, drainage reports and technical information used.

XI. Tables

_____ A. Include a copy of all tables prepared for the study.

XII. Figures

_____ A. Vicinity Map.
_____ B. Site Plan (include the horizontal and vertical datum used and all benchmarks).
C. Existing conditions.
_____ 1. Delineate existing basin boundaries.
_____ 2. Delineate offsite basins impacting the site.
_____ 3. Show existing and proposed topography at an interval of at least 5-ft.
_____ 4. Show existing runoff flow arrows.
_____ 5. Show existing stormwater features (structures, sizes, materials, etc.).
_____ 6. Show floodplain limits and information.
_____ 7. For each basin show bubble with basin number, acreage and % impervious.
_____ 8. For each outlet show bubble with acreage and historic flow and proposed flow or provide information in summary table on figure.
D. Proposed Conditions
_____ 1. Delineate proposed basin boundaries.
_____ 2. Show proposed runoff flow arrows.
_____ 3. Show existing and proposed topography at an interval of at least 5-ft.
_____ 4. For each basin show bubble with basin number, acreage and percent impervious or provide a summary table or figure.
_____ 5. For each outlet show bubble with acreage, historic flow, and proposed flow or provide a summary table or figure.
_____ 6. Show floodplain limits and information.
_____ 7. Show proposed stormwater system (components, sizes, materials, & slopes).
_____ 8. Show property lines and easements (existing and proposed).

XIII. Appendices

_____ A. Runoff Calculations.
_____ B. Culvert Calculations.
_____ C. Pond Calculations.
_____ D. Other Calculations.

Acknowledgements:

Standard Form No. 2 was prepared by: _____ _____
Date

Attach Exhibit A – Storm Water Quality Plan (see Standard Form No. 4)

Standard Form No. 3 Final Drainage Study Checklist

Instructions:

1. The applicant shall identify with a "check mark" if information is provided with letter. If applicant believes information is not required, indicate with "N/A" and attach separate sheet with explanation.
2. The reviewer will determine if information labeled "N/A" is required and whether additional information must be submitted.

I. General

_____ A. Report typed and legible in 8½" x 11" format.
_____ B. Report bound (comb, spiral, or staple – no notebook).
_____ C. Drawings that are 8½ x 11 or 11 x 17 bound within report, larger drawings (up to 24 x 36) included in a pocket attached to the report. Drawings shall be at an appropriate size and scale to be legible and include project area.

II. Cover

_____ A. Report Type – Final Drainage Study.
_____ B. Project Name, Subdivision, Original Date, Revision Date.
_____ C. Preparer's name, firm, address, phone number.
_____ D. "DRAFT" for 1st submittal and revisions; "FINAL" once approved.

III. Title Sheet

_____ A. Table of Contents.
_____ B. Certification, PE Stamp, signature, and date from licensed Colorado PE.
_____ C. Note: City of Steamboat Springs plan review and approval is only for general conformance with City design criteria and the City code. The City is not responsible for the accuracy and adequacy of the design, dimensions, and elevations that shall be confirmed and correlated at the job site. The City of Steamboat Springs assumes no responsibility for the completeness or accuracy of this document.

IV. Introduction

_____ A. Description of site location, size in acres, existing and proposed land use, and any pertinent background info.
_____ B. Reference planning application type and plan set date and preparer.
_____ C. Identify drainage reports for adjacent development.

V. Drainage Criteria and Methodology Used

_____ A. Identify design rainfall and storm frequency.
_____ B. Identify the runoff calculation method used.
_____ C. Identify culvert and storm sewer design methodology.
_____ D. Identify detention discharge and storage methodology.
_____ E. Discuss HEC-HMS methodologies and parameters, if HEC-HMS is used.

VI. Existing Conditions (Pre-Development/Historic)

_____ A. Indicate ground cover, imperviousness, topography, and size of site (acres).
_____ B. Describe existing stormwater system (sizes, materials, etc.).
_____ C. Describe other notable features (canals, major utilities, etc.).
_____ D. Note site outfall locations and ultimate outfall location (typically Yampa River).
_____ E. Note capacity of existing system and identify any constraints.
_____ F. Identify NRCS soil type.
_____ G. Discuss any existing easements.
_____ H. Identify the FEMA Map reviewed, if site is in floodplain/way, and zone designation.

VII. Proposed Conditions

_____ A. Indicate ground cover, imperviousness, topography, and disturbed area (acres).
_____ B. Describe proposed stormwater system (sizes, materials, etc.).
_____ C. Describe proposed outlets, and indicate historic and proposed flow for each.
_____ D. Include calculations for all culverts, ditches, ponds, etc. in appendix.
_____ E. Include a summary table for the 5- and 100-year events showing historic flow and proposed flow for total site and each basin.
_____ F. Discuss proposed easements.
_____ G. Describe off-site flows to be passed thru site.
_____ H. Summarize any impacts to downstream properties or indicate none. Reference CLOMR/LOMR and impacts.

I. Detention Ponds.
_____ 1. Indicate pond volume and area (size and depth) requirement.
_____ 2. Indicate release rates.
_____ 3. Discuss outfall design, location, and overflow location.
_____ 4. Discuss maintenance requirements.

J. Curb and Gutter
_____ 1. Indicate gutter capacity.
_____ 2. Indicate curb capacity.
_____ 3. Indicate design velocity
_____ 4. Indicate design depth of flow in street.

K. Culverts
_____ 1. Indicate whether each culvert is under inlet or outlet control.
_____ 2. Show that headwater is less than the maximum allowable.
_____ 3. Indicate design velocity.
_____ 4. Indicate required and provided flow rates.
_____ 5. Discuss whether outlet protection is required and what will be used.

L. Inlets
_____ 1. Indicate inlet capacity.
_____ 2. Indicate the type of inlet(s) used.

M. Channels
_____ 1. Indicate design velocity (and type of dissipation if required).
_____ 2. Indicate required and provided flow capacity.
_____ 3. Show critical cross-section(s) including water surface.

N. Site Discharge
_____ 1. Discuss use and design of detention to ensure discharge is less than or equal to historic flow.
_____ 2. Provide documentation that downstream facilities are adequate and no adverse impacts to downstream property owners (i.e. no rise certification)

VIII. Post Construction Stormwater Management

_____ A. Discuss in general terms which permanent BMP practices will be used to control pollutant and sediment discharge after construction is complete. Exhibit A, Storm Water Quality Plan shall be attached that will give details (see separate checklist)

IX. Conclusions

_____ A. Provide general summary.
_____ B. Note if site complies with criteria and any variances to criteria.
_____ C. Indicate if peak proposed flow is less than, equal to, or greater than peak historic flow for each outfall, design point, and for the total site.
_____ D. List proposed new stormwater system requirements.

X. References

_____ A. Provide a reference list of all criteria, master plans, drainage reports and technical information used.

XI. Tables

_____ A. Include a copy of all tables prepared for the study.

XII. Figures

_____ A. Vicinity Map.
_____ B. Site Plan (include the horizontal and vertical datum used and all benchmarks).
C. Existing conditions.
_____ 1. Delineate existing basin boundaries.
_____ 2. Delineate offsite basins impacting the site.
_____ 3. Show existing and proposed topography at an interval of at least 2-ft.
_____ 4. Show existing runoff flow arrows.
_____ 5. Show existing stormwater features (structures, sizes, materials, etc.).
_____ 6. Show floodplain limits and information.
_____ 7. For each basin show bubble with basin number, acreage and % impervious.
_____ 8. For each outlet show bubble with acreage and historic flow and proposed flow or provide information in summary table on figure.
D. Proposed Conditions
_____ 1. Delineate proposed basin boundaries.
_____ 2. Show proposed runoff flow arrows.
_____ 3. Show existing and proposed topography at an interval of at least 2-ft.
_____ 4. For each basin show bubble with basin number, acreage and percent impervious or provide a summary table or figure.
_____ 5. For each outlet show bubble with acreage, historic flow, and proposed flow or provide a summary table or figure.
_____ 6. Show floodplain limits and information.
_____ 7. Show proposed building footprints and FFE for commercial and multi-family
_____ 8. Show property lines and easements (existing and proposed).
_____ 9. Label public and private facilities. A general note can be placed on the plans in lieu of labeling all facilities, if applicable.

XIII. Appendices

_____ A. Runoff Calculations.
_____ B. Culvert Calculations.
_____ C. Pond Calculations.
_____ D. Other Calculations.

Acknowledgements

Standard Form No. 3 was prepared by: _____ _____
Date

Attach Exhibit A – Storm Water Quality Plan (see Standard Form No. 4)

Standard Form No. 4 Stormwater Quality Plan Checklist

This list is not an exhaustive list of every possible item required in a stormwater quality plan but provides a general guideline for preparation of the Stormwater Quality Plan.

Instructions:

1. The applicant shall identify with a "check mark" if information is provided with letter. If applicant believes information is not required, indicate with "N/A" and attach separate sheet with explanation.
2. The reviewer will determine if information labeled "N/A" is required and whether additional information must be submitted.

I. General

_____ A. Report typed and legible in 8½" x 11" format.
_____ B. Report bound (comb, spiral, or staple – no notebook).
_____ C. Drawings that are 8½" x 11" or 11" x 17" bound within letter, larger drawings (up to 24" x 36") included in a pocket attached to the letter. Drawings shall be at an appropriate size and scale to be legible and include project area.

II. Cover

_____ A. Report Type – Stormwater Quality Plan.
_____ B. Project Name, Subdivision, Original Date, Revision Date.
_____ C. Preparer's name, firm, address, phone number.
_____ D. "DRAFT" for 1st submittal and revisions; "FINAL" once approved.

III. Title Sheet

_____ A. Table of Contents.
_____ B. Certification, PE Stamp, signature and date from licensed Colorado PE (for Final).
_____ C. Note: City of Steamboat Springs plan review and approval is only for general conformance with City design criteria and the City code. The City is not responsible for the accuracy and adequacy of the design, dimensions, and elevations that shall be confirmed and correlated at the job site. The City of Steamboat Springs assumes no responsibility for the completeness or accuracy of this document.

IV. Introduction

_____ A. Description of site location, size in acres, existing and proposed land use, and any pertinent background info.

V. Design Criteria and Methodology Used

_____ A. Identify design rainfall and storm frequency.
_____ B Identify the runoff calculation method used.
_____ C. Identify stormwater quality methodology.

VI. Proposed Conditions

_____ A. Describe potential site contaminants including sediment.
_____ B. Identify site stormwater flows that need to be managed (type and quantity).
_____ C. Identify all stormwater quality control measures to be constructed.
D. Water quality (detention) pond(s)
_____ 1. Provide a summary of design calculations that supports the design features such as total pond volume, WQCV, sediment storage, water quality outlet structure, square feet and depth (include all calculations in the appendix).
_____ 2. Identify the actual WQCV volume, orifice size, and release rate.
_____ 3. Identify water quality aspect maintenance requirements.
_____ E. If proprietary BMPs are proposed, provide the justification and sizing requirements (see Section 5.12, Water Quality Enhancement).
_____ F. If compensating detention is provided, discuss practices to address water quality from area not tributary to detention area. No underground detention is allowed.

VII. BMP Summary and Maintenance Requirements

_____ A. Indicate and describe the post-construction stormwater quality features (BMPs) provided.
_____ B. If temporary construction sedimentation ponds or other BMPs are required during construction, identify them here or include the Construction SWMP. The Construction SWMP shall be submitted the City prior to construction regardless whether it is included here.
_____ C. List a maintenance schedule of permanent BMPs and who is responsible for it.
_____ D. Identify design specifications for construction.

Acknowledgements

Standard Form No. 4 prepared by: _____ _____
Date

Include this form as part of the Stormwater Quality Plan.