Chapter 9: Streets, Inlets & Conveyance

Contents

1.0 Overview

The vast majority of this Chapter is taken directly from the Streets, Inlets and Storm Drains Chapter in the 2016 UDFCD Manual. There are segments of that chapter of the UDFCD Manual that show the derivation of calculating complex street capacities, capture efficiencies of inlets, the hydraulics of piping networks as well as several example calculations that have not been included here. The Design Engineer should reference the UDFCD Manual or other appropriate reference material for thorough discussion and understanding of these items.

1.1 Purpose and Background

The purpose of this Chapter is to provide design guidance for stormwater collection and conveyance utilizing streets, inlets, storm drains and other conveyances. Procedures and equations for the hydraulic design of street drainage, locating inlets and determining capture capacity, and sizing storm drains are not presented here but can be referenced in the UDFCD Manual.

The design procedures presented in this Chapter are based upon fundamental hydrologic and hydraulic design concepts. It is assumed that the reader has an understanding of basic hydrology and hydraulics. A working knowledge of the Rational Method (discussed in Chapter 5: Hydrology Standards Chapter) and open channel hydraulics (discussed in the UDFCD Manual) is particularly helpful. The design equations provided are well accepted and widely used. They are presented without derivations or detailed explanation but are properly referenced if the reader wishes to study their background.

Inlet capacity has been studied in great depth at the UDFCD. Determining inlet capacity and further refinement of the methodologies through multi-jurisdictional partnerships led by UDFCD, where hundreds of physical model tests of inlets commonly used in Colorado were performed at the Colorado State University (CSU) Hydraulics Laboratory. The physical model study is further detailed in technical papers available at www.udfcd.org.

UDFCD Reference: UDFCD has developed an inlet design tool, UD-Inlet, which incorporates the findings of the physical model. UD-Inlet is commonly used and an acceptable software tool for use in determining street capacity and sizing inlets for systems in Fort Collins. The UD-Inlet spreadsheet is available at www.udfcd.org/software

Other design tools may also be available and utilized with prior approval from FCU.

1.2 Urban Stormwater Collection and Conveyance Systems

Proper and functional urban stormwater collection and conveyance systems:

- Promote safe passage of vehicular traffic during minor storm events
- Maintain public safety and manage flooding during major storm events
- Minimize capital and maintenance costs of the system

Urban stormwater collection and conveyance systems are critical components of the urban infrastructure. Proper design is essential to minimize flood damage and limit disruptions. The primary function of the system is to collect excess stormwater in street gutters, convey it through storm drains and along the street right-of-way, and discharge it into a detention basin, water quality best management practices (BMP), or nearest receiving water body (FHWA 2009).

1.3 System Components

Urban stormwater collection and conveyance systems are comprised of three primary components:

1) Street gutters and roadside swales
2) Storm drain inlets
3) Storm drains (with appurtenances like manholes, junctions, etc.)

Street gutters and roadside swales collect runoff from the street (and adjacent areas) and convey the runoff to a storm drain inlet while maintaining the street's level of service.

Inlets collect stormwater from streets and other land surfaces, transition the flow into storm drains, and provide maintenance access to the storm drain system. Storm drains convey stormwater in excess of street or swale capacity along the right-of-way and discharge into a stormwater management facility or directly into a receiving water body. All of these components must be designed properly to achieve the objectives of the stormwater collection and conveyance system.

1.4 Minor and Major Storms

Rainfall events vary greatly in magnitude and frequency of occurrence. Major storms produce large flow rates but rarely occur. Minor storms produce smaller flow rates but occur more frequently. For economic reasons, stormwater collection and conveyance systems are not normally designed to pass the peak discharge during major storm events without some street flooding.

Stormwater collection and conveyance systems are designed to pass the peak discharge of the minor storm event (and smaller events) with minimal disruption to street traffic. To accomplish this, the spread and depth of water on the street is limited to a maximum mandated value during the minor storm event. Inlets must be strategically placed to pick up excess gutter or swale flow once the limiting allowable spread or depth of water is reached. The inlets collect and convey stormwater into storm drains, which are typically sized to pass the peak flow rate (minus the allowable street flow rate) from the minor storm without any surcharge. In Fort Collins, the magnitude of the minor storm event is defined as the 2-year storm.

In Fort Collins, the return period for the major storm event is defined as the 100-year storm. During this event, runoff exceeds the minor storm allowable spread and depth in the street and capacity of storm drains. Street flooding may occur and traffic may be disrupted as the street functions as an open channel. The Design Engineer must evaluate and design for the major event with regard to maintaining public safety and minimizing flood damages.

2.0 Street Drainage

Although streets play an important role in stormwater collection and conveyance, the primary function of a street or roadway is to provide for the safe passage of vehicular traffic at a specified level of service. If stormwater systems are not designed properly, this primary function will be impaired. Proper street drainage is essential to:

- Maintain the street's level of service
- Minimize danger and inconvenience to pedestrians during storm events (FHWA 1984)
- Reduce potential for vehicular skidding and hydroplaning
- Maintain good visibility for drivers (by reducing splash and spray)
- Maintain access for emergency vehicles

Reference: The Larimer County Urban Area Street Standards (LCUASS) shall be referenced for all street classification and design requirements for each project.

2.1 Encroachment Standards

The encroachment criteria provided in the tables below applies to public streets. Where there is a floodplain designation, Chapter 10 of the City code shall also apply. Encroachment in this context is defined as the extent of which stormwater is allowed to extend into the public roadway in terms of width and depth.

Table 2.1-1: Street Encroachment Standards for the Minor (2-Year) Storm

Street Classification	Maximum Encroachment
Local, Alley	• No curb-overtopping. • Flow may spread to crown of street.
Collector, Arterial (without median)	• No curb-overtopping. • Maximum allowable depth at gutter is 6 inches (6”). • Flow spread must leave a minimum of 6 feet (6’) wide clear travel lane on each side of the centerline.
Arterial (with median)	• No curb-overtopping. • Maximum allowable depth at gutter is 6 inches (6”). • Flow spread must leave a minimum of 12 feet (12’) wide travel lane in both directions of travel.

Note: Encroachment may not extend past the public right-of-way or into private property.

Table 2.1-2: Street Encroachment Standards for the Major (100-Year) Storm

Street Classification	Maximum Encroachment
Local, Alley, Collector, Arterial (without median)	• Maximum allowable depth at crown is 6 inches (6”) and must allow for the operation of emergency vehicles. • Maximum allowable depth at gutter is 12 inches (12”). • The most restrictive of these criteria will apply.
Arterial (with median)	• Maximum allowable depth must not exceed bottom of gutter at the median and must allow for the operation of emergency vehicles. • Maximum allowable depth at gutter is 12 inches (12”). • The most restrictive of these criteria will apply.

Note: Encroachment may not extend past utility easements that parallel the public right-of-way.

Table 2.1-3: Allowable Cross-Street Flow

Street Classification	Minor (2-Year) Storm	Major (100-Year) Storm
Local	Maximum allowable depth in crosspan is 6 inches	Maximum allowable depth at flowline is 18 inches (18”)
Collector	Maximum allowable depth in crosspan is 6 inches (only where crosspans are allowed)	Maximum allowable depth at flowline is 12 inches (12”)
Arterial	No cross-flow allowed	No cross-flow allowed. Maximum depth at arterial/local intersections shall not exceed arterial depth maximums (i.e. 12 inches (12”))

Note: Encroachment may not extend past utility easements that parallel the public right-of-way.

STREET HYDRAULIC CAPACITY:

REFERS TO THE CAPACITY FROM THE FACE OF THE CURB TO THE CROWN (FOR THE MINOR EVENT.) TYPICALLY, THE HYDRAULIC COMPUTATIONS NECESSARY TO DETERMINE STREET CAPACITY AND REQUIRED INLET LOCATIONS ARE PERFORMED INDEPENDENTLY FOR EACH SIDE OF THE STREET. ADDITIONALLY, FLOW AND STREET GEOMETRY MAY DIFFER FROM ONE SIDE OF THE STREET TO THE OTHER.

Once the allowable street encroachment has been established for the minor storm, the placement of inlets can be determined. The inlets will remove some or all of the excess stormwater and thus reduce the spread. It should be noted that proper drainage design utilizes the full allowable capacity of the street gutter in order to limit the cost of inlets and storm sewers.

At street sump locations, proper inlet sizing and design will be required to ensure that the 100-year flows can be carried to the storm pipes or an overflow channel to an acceptable outfall while the maximum water surface depth criteria are not surpassed. Inlet design is discussed in the next section of this Chapter.

A drainage easement for drainage overflow drainage must be granted to the City for access and maintenance if the stormwater flows are not contained within the public right-of-way.

Two additional design considerations are gutter geometry and street slope. Most urban streets incorporate curb and gutter sections. Various types exist, including spill shapes, catch shapes, curb heads and mountable curbs. The shape is chosen for function, cost or aesthetic reasons and does not dramatically affect the hydraulic capacity. Swales are used along some semi-urban streets and roadside ditches are common along rural streets. Cross-sectional geometry, longitudinal slopes and swale/ditch roughness values are important in determining hydraulic capacity and are covered in the next section.

2.2 Hydraulic Evaluation

Hydraulic computations are performed to determine the capacity of roadside swales and street gutters and the encroachment of stormwater onto the street. The design discharge is based on the peak flow rate and usually is determined using the Rational Method. Although gutter and street flows are unsteady and non-uniform, steady, uniform flow is assumed for the short time period of peak flow conditions.

2.2.1 Curb and Gutter

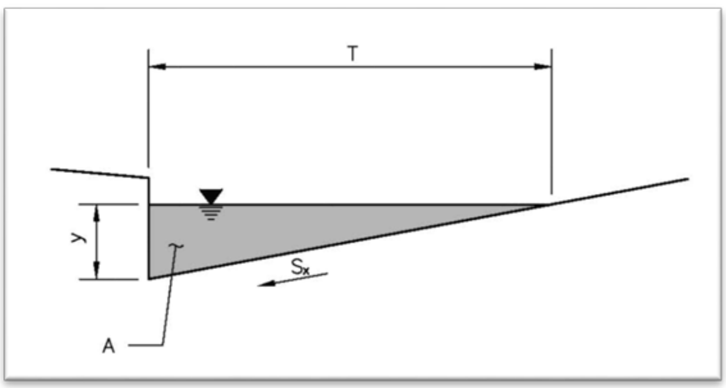
Both the longitudinal and cross (transverse) slope of a street are important in calculating hydraulic capacity. The capacity of the street increases as the longitudinal slope increases. Public safety considerations limit the maximum allowable flow capacity of the gutter on steep slopes. The cross-slope represents the slope from the street crown to the interface with the lip of the gutter, measured perpendicular to the direction of travel. Use of standard curb and gutter sections typically produces a composite section with milder cross-slopes for drive lanes and steeper cross-slopes within the gutter width for increased flow capacity.

Reference: LCUASS criteria will stipulate minimum and maximum allowable longitudinal and cross-slopes allowed for new and reconstructed roadways.

Capacity When Gutter Cross-Slope Equals Street Cross-Slope (Not Typical)

Streets with uniform cross-slopes like that shown in **Figure 2.2.1-1** are sometimes found in older urban areas. Since gutter flow is assumed to be uniform for design purposes, a modified Manning's equation is appropriate to use in this instance.

Figure 2.2.1-1. Gutter Section with Uniform Cross-Slope



For the triangular cross-section shown in the Figure above, flow rate in the gutter can be found using the Manning's equation, written as:

$$\mathbf{Q = \frac{0.56}{n} S_x^{5/3} S_o^{1/2} T^{8/3}}$$ **Equation 9-1**

Where:

Q = calculated flow rate for the half-street, cfs
n = Manning's roughness coefficient, dimensionless
S_x = street cross-slope, ft/ft
S_o = street longitudinal slope, ft/ft
T = top width of flow spread, ft

The flow depth can be found using:

$$\mathbf{y = TS_x}$$ **Equation 9-2**

Where:

y = flow depth at the gutter flowline, ft

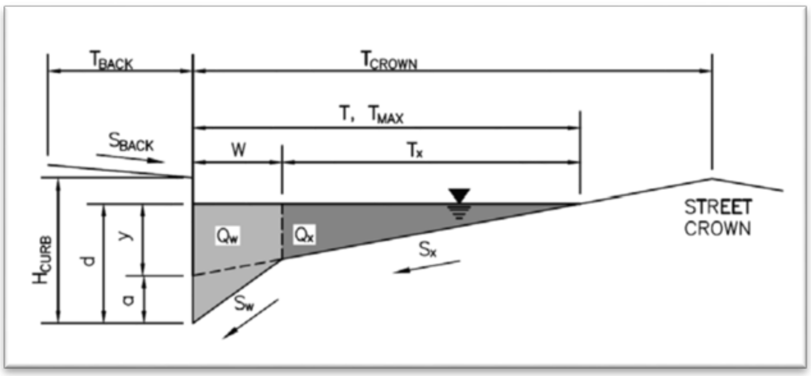
Note that the flow depth shall not exceed the curb height during the minor storm based on the criteria in **Table 2.1-1**.

Reference: The description and derivation of the Manning's equation modification can be found in the Streets, Inlets & Storm Drains Chapter of the UDFCD Manual.

Capacity When Gutter Cross-Slope is Not Equal to Street Cross-Slope (Typical)

Streets with composite cross-slopes like that shown in **Figure 2.2.1-2** are often used to increase the gutter capacity and keep nuisance flows out of the travel lanes.

Figure 2.2.1-2. Typical Gutter Section with Composite Cross-Slope



Determining the flow rate for composite street sections involves first determining the flow in the street (not the gutter) then determining the ratio of gutter flow to total flow, then computing the theoretical flow rate for the composite cross-section. Due to the complexity of this calculation procedure, it is recommended that the Design Engineer review the information presented in the UDFCD Manual for more thorough understanding. The UD-Inlet design workbook is an allowable design tool that incorporates these calculations into it.

Reference: The fundamentals of determining street capacities are further explored and presented in the other reference manuals including the UDFCD Manual.

Allowable Capacity

Stormwater flows along streets exert momentum forces on cars, pavement and pedestrians. To limit the hazardous nature of large street flows, it is necessary to set limits on flow velocities and depths. As a result, the allowable half-street hydraulic capacity is determined as the lesser of:

$$Q_A = Q_T \qquad \textbf{Equation 9-3}$$

Or

$$Q_A = RQ_d$$ **Equation 9-4**

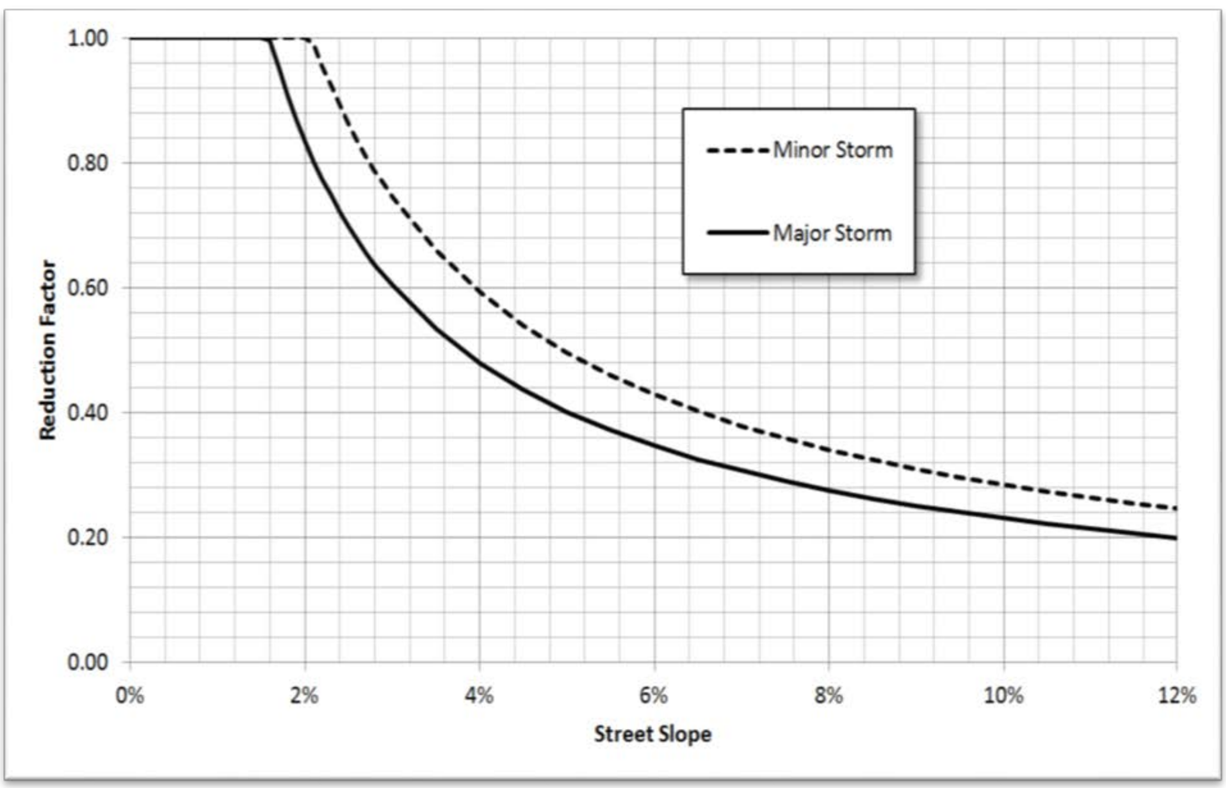
Where:

Q_A = allowable street hydraulic capacity, cfs
Q_T = street hydraulic capacity where flow spread equals allowable spread, cfs
R = reduction factor (allowable street and gutter flow for safety), dimensionless
Q_d = street hydraulic capacity where flow depth equals allowable depth, cfs

There are two sets of safety reduction factors developed for the UDFCD region (Guo 2000b) and included in the design standards of this Manual. One is for the minor event and the other is for the major event. **Figure 2.2.1-3** shows that the safety reduction factor does not apply unless the street longitudinal slope is more than 1.5% for the major event and 2% for the minor event. The safety reduction factor, representing the fraction of calculated gutter flow at maximum depth that is used for the allowable design flow, decreases as longitudinal slope increases.

It is important that street drainage design includes the allowable street hydraulic capacity using reduction factors. Where the accumulated stormwater amount on the street approaches the allowable capacity, a street inlet should be installed.

Figure 2.2.1-3. Reduction factor for gutter flow (Guo 2000b)



2.2.2 Swale Capacity

Where curb and gutter are not used to contain flow, swales are frequently used to convey runoff and disconnect impervious areas. It is very important that swale depths and side slopes be shallow for safety and maintenance reasons. Street side drainage swales are not the same as roadside ditches. Street side drainage swales provide mild slopes and are frequently designed to provide water quality enhancement. For purposes of disconnecting impervious area and reducing the overall volume of runoff, swales should be considered as collectors of initial runoff for transport to other larger means of conveyance. To be effective, they need to be limited to a stable velocity, depth and cross-slope geometries.

Equation 9-5 can be used to calculate the flow rate in a V-section swale (using the appropriate roughness value for the swale surface) with an adjusted cross-slope found using:

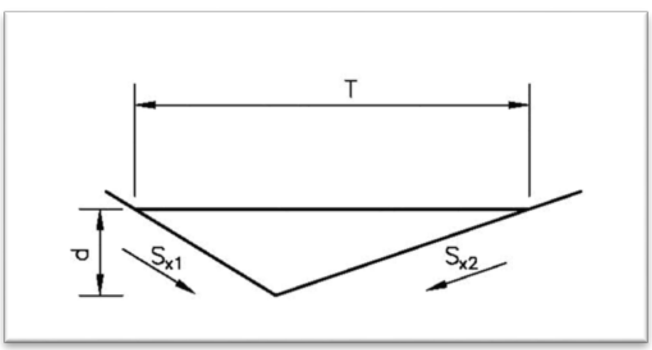
$$S_x = \frac{S_{x1} S_{x2}}{S_{x1}+S_{x2}} \quad \textbf{Equation 9-5}$$

Where:

S_x = adjusted side slope, ft/ft
S_{x1} = right side slope, ft/ft
S_{x2} = left side slope, ft/ft

Figure 2.2.2-1 shows the geometric variables.

Figure 2.2.2-1. Typical V-Shaped Swale Section



Note that the slope of a roadside ditch or swale can be different than the adjacent street. The hydraulic characteristics of the swale can therefore change from one location to another and should be analyzed where appropriate.

VELOCITY x DEPTH:

FOR SAFETY REASONS, PAVED SWALES (E.G. SWALES WITH CONCRETE TRICKLE CHANNELS) SHOULD BE DESIGNED SUCH THAT THE PRODUCT OF VELOCITY AND DEPTH IS NO MORE THAN SIX (6) FOR THE MINOR STORM AND EIGHT (8) FOR THE MAJOR STORM.

3.0 Inlets

3.1 Inlet Function and Selection

Inlets collect excess stormwater from the street, transition the flow into storm drains and can provide maintenance access to the storm drain system. There are three major types of inlets: grate, curb opening and combination. **Table 3.1-1** provides considerations in proper selection.

Table 3.1-1. Inlet Selection Considerations

Inlet Type	Applicable Setting	Advantages	Disadvantages
Grate	Sumps and continuous grades (should be made bicycle safe)	Perform well over wide range of grades	Can become clogged. Can lose some capacity with increasing grade
Curb-Opening	Sumps and continuous grades (but not steep grades)	Do not clog easily, bicycle safe	Lose capacity with increasing grade
Combination	Sumps and continuous grades (should be made bicycle safe)	Intercept flow over wide section	Susceptible to clogging

3.2 Design Considerations

Frequently, roadway geometry dictates the location of inlets. Inlets are placed at low points (sumps), median breaks and at intersections. Additional inlets should be placed where the design peak flow on the street half is approaching the allowable capacity of the street half. Allowable street capacity will be exceeded and storm drains will be underutilized when inlets are not located properly or not designed for adequate capacity. (Akan and Houghtalen, 2002)

Inlets placed on continuous grades are generally designed to intercept only a portion of the gutter flow during the minor storm (i.e. some flow bypasses to downgradient inlets).

The effectiveness of the inlet is expressed as efficiency defined as:

$$\mathbf{E = Q_i/Q} \qquad \textbf{Equation 9-6}$$

Where:

E = inlet efficiency (fraction of gutter flow captured by the inlet)
Q_i = intercepted flow rate, cfs
Q = total half-street flow rate, cfs

Bypass (or carryover) flow is not intercepted by the inlet. By definition,

$$\mathbf{Q_b = Q - Q_i}$$ **Equation 9-7**

Where:

Qb = bypass (or carryover) flow rate, cfs

The ability of an inlet to intercept flow (i.e. hydraulic capacity) on a continuous grade increases to a degree with increasing gutter flow, but the capture efficiency decreases. In general, the inlet capacity depends on:

- The inlet type and geometry (length, width, curb opening, etc.)
- The flowrate
- The longitudinal slope
- The cross (transverse) slope

The capacity of an inlet varies with the type of inlet. For grate inlets, the capacity is largely dependent on the amount of water flowing over the grate, the grate configuration and spacing. For curb-opening inlets, the capacity is largely dependent on the length of the opening, street and gutter cross-slope and the flow depth at the curb. Local gutter depression at the curb opening will increase capacity. FCU requires that all curb-opening throats must be installed with the bottom of the opening at least two inches (2”) below the flowline elevation. The minimum transition length allowed is five feet (5’).

Combination inlets on a continuous grade (i.e. not a sump condition) intercept up to 18% more than grate inlets alone and are much less likely to clog completely (CSU 2009).

TYPE R INLET RESTRICTIONS:

- THROAT OPENINGS SHALL BE AT LEAST 2” BELOW FLOWLINE ELEVATION
- FOR PUBLIC SAFETY CONCERNS, THROAT OPENINGS MUST NOT EXCEED 6”.
- MINIMUM TRANSITION LENGTH FROM FLOWLINE TO THROAT IS 5’
- TYPE R INLETS ARE DISCOURAGED FROM BEING PLACED ON LOCAL STREETS OR RESIDENTIAL AREAS UNLESS THERE ARE PHYSICAL CONSTRAINTS THAT WOULD EXCLUDE THE USE OF COMBO INLETS.

Inlets in sumps operate as weirs at shallow ponding and as orifices as depth increases. A transition region exists between weir flow and orifice flow, much like a culvert. Grate inlets and slotted inlets have a higher tendency to clog with debris than do curb opening inlets, so calculations should take that into account.

Reference: The methodology for determining the hydraulic capacity of the various inlet types is documented in the UDFCD Manual. Refer to that manual for in-depth hydraulic design information for inlets.

Photograph 3.2-1. These street inlets are the most commonly used in Fort Collins. Their performance was tested for both on-grade conditions and in sump conditions in a 1/3 scale physical model at CSU.

(a) CDOT Type 13 grated inlet in combination configuration

(b) CDOT Type R curb opening inlet

3.3 Inlets on a Continuous Grade

3.3.1 Grate Inlets on a Continuous Grade

The capture efficiency of a grate inlet on a continuous grade is highly dependent on the width of the grate and, to a lesser degree, the length. In general, most of the flow within the width of the grate will be intercepted and most of the flow outside of the width of the grate (i.e. in the street) will not. The velocity of gutter flow also affects capture efficiency. If the gutter velocity is low and the spread of water does not exceed the grate width, all of the flow will be captured by the grate inlet. However, this is not normally the case, even during the minor storm. The spread of water often exceeds the grate width and the flow velocity can be high. Thus, some of the flow within the width of the grate may splash over the grate, and unless the inlet is very long, very little of the flow outside the grate width is captured.

3.3.2 Curb-Opening Inlets on a Continuous Grade

The capture efficiency of a curb-opening inlet is dependent on the length of the opening, the depth of flow at the gutter flow line, street cross-slope and the longitudinal gutter slope. If the curb opening is long, the flow rate is low and the longitudinal gutter slope is small, all of the flow will be captured by the inlet. It is generally uneconomical to install a curb-opening long enough to capture all of the flow during the minor storm. Thus, some water gets by the inlet, and the inlet efficiency needs to be determined.

3.3.3 Combination Inlets on a Continuous Grade

Combination inlets take advantage of the debris removal capabilities of a curb-opening inlet and the capture efficiency of a grate inlet. Combination inlets on a continuous grade (i.e. not in a sump location) intercept 18% more than grate inlets alone and are much less likely to clog completely (CSU 2009). A special case combination where the curb opening extends upstream of the grated section is called a sweeper inlet. The inlet capacity is enhanced by the additional upstream curb-opening length and debris is intercepted there before it can clog the grate. The construction of sweeper inlets is more complicated and costly, however, and they are not commonly seen in Fort Collins. To calculate interception efficiency for a sweeper inlet, the upstream curb-opening efficiency is calculated first and then the interception efficiency for combination section based on the remaining street flow is added to it. To analyze this within UD-Inlet, select *user-defined combination*, select a grate type, and check the *sweeper configuration* box.

3.3.4 Inlet Location and Spacing on Continuous Grades

Although one should always perform interception capacity computations on stormwater inlets, the ultimate location (or positioning) of those inlets is rarely a function of interception alone. Often, inlets are required in certain locations based upon street design considerations and topography (low points). One notable exception is the location and spacing of inlets on continuous grades. On a long continuous grade, stormwater flow increases as it moves down the gutter and picks up more drainage area. As the flow increases, so does the spread (encroachment) and depth (inundation). Since the spread and depth are not allowed to exceed the specified maximum (see **Tables 2.1-1** and **2.1-2**), inlets must be strategically placed to remove some of the stormwater from the street. Locating these inlets requires design computations by the Design Engineer.

Proper design of stormwater collection and conveyance systems makes optimum use of the conveyance capabilities of street gutters, such that an inlet is not needed until the spread (encroachment) and depth (inundation) reach allowable limits during the design (minor) storm. To place an inlet prior to that point on the street is not economically efficient. To place an inlet after that point would violate the encroachment and inundation standards. Therefore, the primary design objective is to position inlets along a continuous grade at the locations where the allowable spread and/or depth is about to be exceeded for the design storm. The ultimate goal is to always place an inlet just upstream of the point where the allowable spread and/or criteria would otherwise be exceeded.

Once the first inlet location is identified along a continuous grade, an inlet type and size can be specified. The first inlet's hydraulic capacity is then assessed. Generally, it is uneconomical to size an inlet (on continuous grades) large enough to capture all of the gutter flow. Instead, some carryover flow is expected. This practice reduces the amount of new flow that can be picked up at the next inlet. However, each inlet should be positioned at the location where the spread or depth of flow is about to reach its allowable limit. For placement of inlets on a continuous grade, the Design Engineer should not only analyze length of the grate opening to capture a required amount of flow (which may result in a very long inlet bank), but also analyze the placement of dispersed inlets along the continuous grade to capture the required amount of flow. As discussed further in Section 3.4.2, weir performance decay can also play a part in reducing the effectiveness of long inlet banks.

The gutter discharge for inlets (other than the most upstream inlet), consists of the carryover (bypassed) flow from the upstream inlet plus the stormwater runoff generated from the intervening local drainage area. The carryover flow from the upstream inlet is added to the peak flow rate obtained from the Rational Method for the intervening local drainage area. The resulting peak flow is conservatively approximate since the carryover flow peak and local runoff peak do not necessarily coincide.

Reference: UD-Inlet design workbook is available for download from the UDFCD website and is a widely used design tool accepted by FCU. The examples provided at the end of the Street, Inlets & Storm Drains Chapter in the UDFCD Manual for inlet calculations show how to calculate the capture efficiency and the overall flow capture for inlets.

3.4 Inlets in a Sump

3.4.1 Grate Inlets in a Sump (UDFCD-CSU Model)

All of the stormwater draining to a sump inlet must pass through an inlet grate or curb-opening to enter the storm drain. This means that clogging due to debris can result not only in underutilized pipe conveyance, but also ponding of water on the surface. Surface ponding can be a nuisance or hazard. Therefore, the capacity of inlets in sumps must account for this clogging potential. Grated inlets alone are not allowed on roadways for this reason. Curb-opening and combination (including sweeper) inlets are more appropriate. In all sump inlet locations, consider the risk and required maintenance associated with a full clogged condition and design the system accordingly.

Photograph 3.3.4-1. Inlets that are located in street vertical sag curves (sumps) are highly efficient.

Photograph 3.3.4-1 shows a curb-opening inlet in a sump condition. At this location, if the inlet clogs, standing water will be limited to the elevation at the back of the walk.

Flow through a grated sump inlet varies with respect to depth and continuously changes from weir flow (at shallow depths) to mixed flow (at intermediate depths), and also orifice flow (at greater depths). For commonly used grated street inlets in the UDFCD region, a UDFCD-CSU physical model study was conducted to more accurately measure the interception capacity of grated inlets.

Reference: The UDFCD-CSU physical model study is discussed in the Streets, Inlets & Storm Drains chapter of the UDFCD Manual.

3.4.2 Curb-Opening Inlets in a Sump (UDFCD-CSU Model)

Like a grate inlet, a curb-opening inlet operates under weir, orifice, or mixed flow. From the UDFCD-CSU physical model study, the HEC-22 procedure was found to overestimate the capacity of the CDOT Type R and other similar curb-opening inlets for the minor storm event and underestimate capacity for the major storm event. From the UDFCD-CSU study of these inlets, the interception capacity is based on the depression and opening geometry.

WEIR PERFORMANCE DECAY:
INLETS BECOME LESS EFFECTIVE IN WEIR FLOW AS THEY GROW IN LENGTH. WHAT THIS MEANS IS THAT ADDING INLETS TO REDUCE THE DEPTH OF FLOW WILL TYPICALLY NOT INCREASE TOTAL CAPACITY WHEN THE INLET IS IN WEIR FLOW. THIS IS IMPORTANT TO CONSIDER THIS WHEN DESIGNING FOR THE MINOR EVENT. IN AN EFFORT TO MEET MINOR EVENT DEPTH CRITERIA, THE SYSTEM MAY NEED TO BE EXTENDED FURTHER UPSTREAM.

The UDFCD-CSU study demonstrated a phenomenon referred to as weir performance decay, which is a function of the length of the inlet. It was found that inlets become less effective in weir flow as they grow in length, if the intent is to limit ponding to less than or equal to the curb height.

Photograph 3.4.2-1. Weir performance decay can be observed in this picture as flow appears to enter only the first two inlets while exceeding the height of the upstream curb.

From the UDFCD-CSU study, an empirical equation to estimate interception capacity for the CDOT Type R curb-opening inlet was developed and is shown in **Figure 3.4.2-1**.

The UDFCD-CSU study demonstrated that the grate and curb-opening function of combination inlets do not operate independently, but interfere with each other and affect the actual capacity of combination inlets. As such, the study demonstrated that the CDOT Type 13 combination inlets are also subject to weir performance decay. Empirical equations to estimate interception capacity for the CDOT Type 13 combination inlet was developed and is shown in **Figure 3.4.2-2.**

Figure 3.4.2-1. CDOT Type R interception capacity in a sump

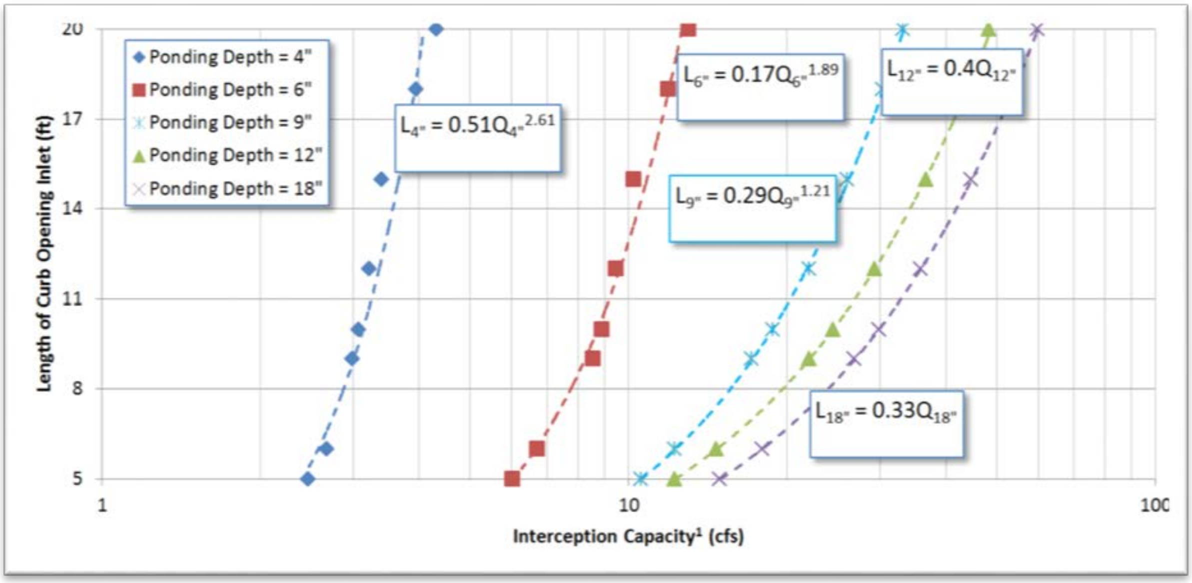
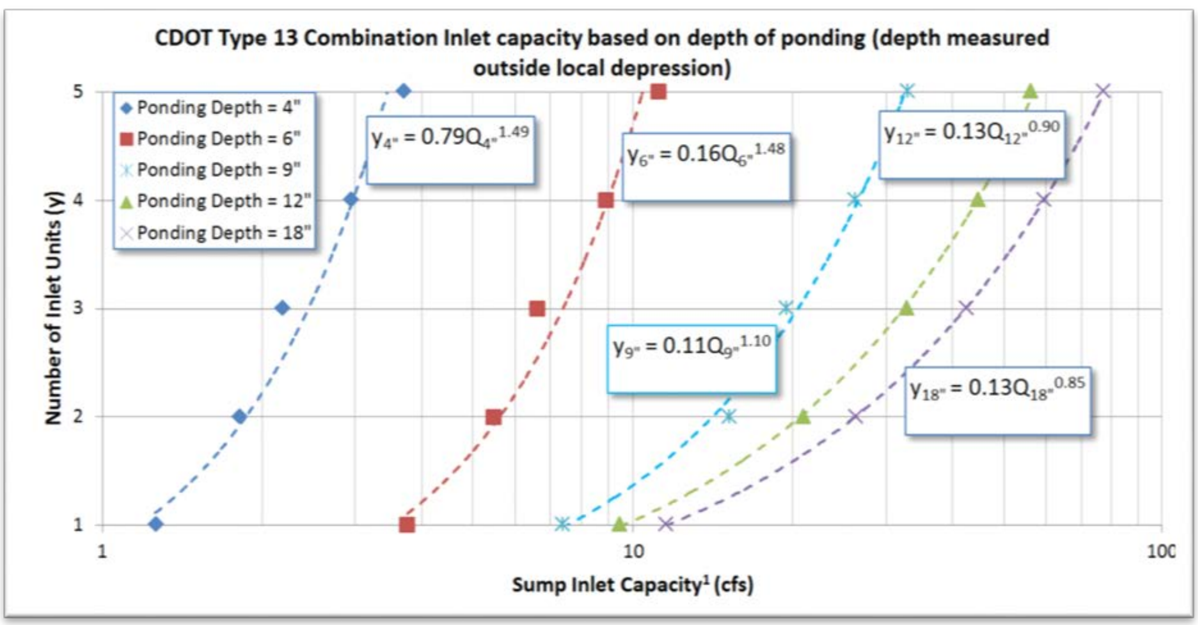


Figure 3.4.2-2. CDOT Type 13 Combination inlet interception capacity in a sump



3.4.3 Other Inlets in a Sump (Not Modeled in the UDFCD-CSU Study)

The hydraulic capacity of grate, curb-opening and slotted inlets operating as weirs is expressed as:

$$\mathbf{Q_i = C_w L_w d^{1.5}}$$ **Equation 9-8**

Where:

Q_i = inlet capacity, cfs
C_w = weir discharge coefficient
L_w = weir length, ft
D = flow depth, ft

Values for C_w and L_w are presented in **Table 3.4.3-1** for various inlet types. Note that the expressions given for curb-opening inlets without depression should be used for depressed curb-opening inlets if L>12 feet.

The hydraulic capacity of grate, curb-opening and slotted inlets operating as orifices is expressed as:

$$\mathbf{Q_i = C_o A_o (2gd)^{0.5}}$$ **Equation 9-9**

Where:

Q_i = inlet capacity, cfs
C_o = orifice discharge coefficient
A_o = orifice area, ft^2
d = characteristic depth as defined in **Table 3.4.3-1**, ft
g = 32.2 ft/sec^2

Values for C_o and A_o are presented in **Table 3.4.3-1** for different types of inlets.

Combination inlets are commonly used in sumps. The hydraulic capacity of combination inlets in sumps depends on the type of flow and the relative lengths of the curb opening and grate. For weir flow, the capacity of a combination inlet (grate length equal to the curb opening length) is equal to the capacity of the grate portion only. This is because the curb opening does not add any effective length to the weir. If the curb opening is longer than the grate, the capacity of the additional curb length should be added to the grate capacity. For orifice flow, the capacity of the curb opening should be added to the capacity of the grate.

Table 3.4.3-1. Sump Inlet Discharge Variables and Coefficients (Modified from Akan and Houghtalen 2002)

Inlet Type	**C_w**	**L_w [1]**	**Weir Equation Valid for**	**Definitions of Terms**
Grate Inlet	3.00	L+2W	$d<1.79(A_o/L_w)$	L = length of grate, ft W = width of grate, ft d = depth of water over grate, ft A_o = clear opening area [2], ft^2
Curb-Opening Inlet	3.00	L	d < h	L = length of curb opening, ft h = height of curb opening, ft $d = d_i-(h/2)$, ft d_i = depth of water at curb opening, ft
Depressed Curb-Opening Inlet [3]	2.3	L + 1.8W	d < (h + a)	W = lateral width of depression, ft
Slotted Inlets	2.48	L	d < 0.2 ft	L = length of slot, ft d = depth at curb, ft

1. The weir length should be reduced where clogging is expected.
2. Ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sump locations unless in combination with curb openings.
3. If L > 12 ft, use the expressions for curb-opening inlets without depression.

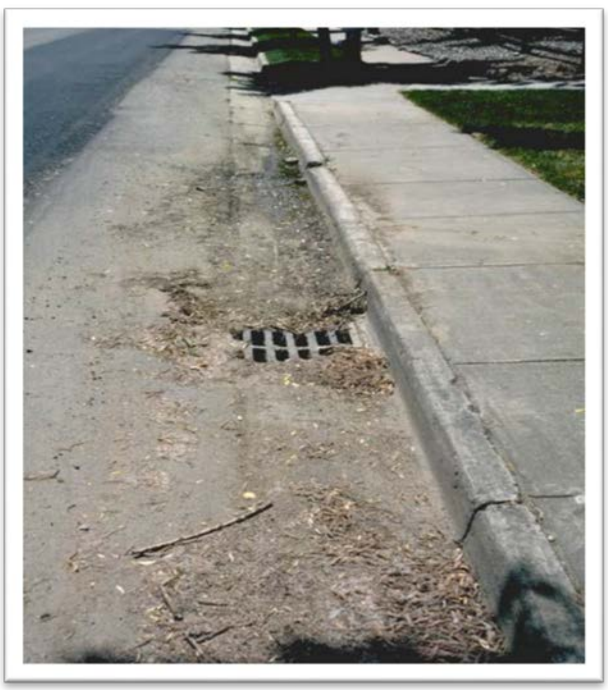
Inlet Type	**C_o**	**A_o [4]**	**Orifice Equation Valid for**	**Definition of Terms**
Grate Inlet	0.67	Clear opening area[5]	d > 1.79(A0/Lw)	d = depth of water over grate, ft
Curb-Opening Inlet (depressed or undepressed, horizontal orifice throat)	0.67	(h)(L)	di > 1.4h	d = di-(h/2), ft di = depth of water at curb opening, ft h = height of curb opening, ft
Slotted Inlet	0.80	(L)(W)	d > 0.40 ft	L = length of slot, ft W = width of slot, ft d = depth of water over slot, ft

4. The orifice area should be reduced where clogging is expected.
5. The ratio of clear opening area to total area is 0.8 for P-1-7/8-4 and reticuline grates, 0.9 for P-1-7/8 and 0.6 for P-1-1/8 grates. Curved vane and tilt bar grates are not recommended at sump locations unless in combination with curb openings.

3.4.4 Inlet Clogging

Inlets are subject to clogging effects (see **Photograph 3.4.4-1**). Selection of a clogging factor reflects the condition of debris and trash on the street. During a storm event, street inlets are usually loaded with debris by the first flush runoff volume. As a common practice for street drainage, 50% clogging is considered for the design of a single grate inlet and 10% clogging is considered for a single curb-opening inlet. Often, it takes multiple units to collect the stormwater on the street. Since the amount of debris is largely associated with the first flush volume in a storm event, the clogging factor applied to a multiple-unit street inlet should be decreased with respect to the length of the inlet. Linearly applying a single-unit clogging factor to a multiple-unit inlet will lead to an excessive increase in inlet length. For example, if a 50% clogging factor is applied to a six-unit inlet, the inlet would be presumed to function as a three-unit inlet. In reality, the upgradient units of the inlet would be more susceptible to clogging (perhaps at the 50% level) than the downgradient portions. In fact, continuously applying a 50% reduction to the discharge on the street will always leave 50% of the residual flow on the street. This means that the inlet will never reach a 100% capture and leads to unnecessarily long inlets.

Photograph 3.4.4-1. Clogging is an important consideration when designing inlets. With the concept of first-flush volume, the decay of clogging factor to grate or curb-opening length is described as (Guo 2000a):



3.5 UD-Inlet Design Workbook

The UD-Inlet design workbook provides quick solutions for many of the street capacity and inlet performance computations described in this Chapter. A brief summary of each worksheet of the workbook is provided below. Note that some of the symbols and nomenclature in the worksheets do not correspond exactly with the nomenclature of the text. The text and the worksheets are computationally equivalent.

- The **Q-Peak** tab calculates the peak discharge for the inlet tributary area based on the Rational Method for the minor and major storm events. Alternatively, the user can enter a known flow. Information from this tab is exported to the *Inlet Management* tab.
- The **Inlet Management** tab imports information from the *Q-Peak* tab and *Inlet [#]* tabs and can be used to connect inlets in series so that bypass flow from an upstream inlet is added to flow

calculated for the next downstream inlet. This tab can also be used to modify design information imported from the *Q-Peak* tab.

- **Inlet [#]** tabs are created each time the user exports information from the *Q-Peak* tab to the *Inlet Management* tab. The *Inlet [#]* tabs calculate the allowable half-street capacity based on allowable depth and allowable spread for the minor and major storm events. This is also where the user selects an inlet type and calculates the capacity of that inlet.

- The **Inlet Pictures** tab contains a library of photographs of the various types of inlets contained in the worksheet and referenced in this Chapter.

Reference: The UD-Inlet design workbook, available for download at the www.udfcd.org/software website is a common design tool used by Design Engineers and is accepted for use by FCU.

3.6 Nuisance Flows

The location of inlets is important to address the effects of nuisance flows and avoid icing. Nuisance flows are urban runoff flows that are typically most notable during dry weather and come from sources such as over-irrigation and snow melt. Nuisance flows can cause problems in both warm and cold weather months. Problems include algae growth and ice. While it is possible to minimize nuisance conditions through design, irrigation practices in the summer and snow and ice removal in the winter make it very difficult to eliminate nuisance flows entirely. Because these practices are somewhat controlled by residents and businesses; homeowner's associations and business associations should plan for maintenance on private roadways and parking areas to address nuisance flow conditions, particularly in the winter when ice accumulation can impede the ability of the drainage system to serve its purpose. Design Engineers should work with property owners and development teams to implement a storm drainage design that minimizes the impact of nuisance flows to the greatest degree possible. These include the maintenance objectives of removal of snow and ice promptly and frequently, keeping drains and gutters clear and placing shoveled snow onto lawns or grassy areas.

In the summer months, over-irrigation of lawns and landscaping can be a major contributor to nuisance flows. Car washing is another summertime cause of excess flows. In homes with poor or improper drainage, excessive sump pump discharge may also contribute.

Flows over sidewalks and driveways due to summertime nuisance flows can cause algae growth, especially if fertilizer is being used in conjunction with over-irrigation. Such algae growth is both a safety issue due to increased falling risk resulting from slippery surfaces and an aesthetic issue. Nuisance flows laden with fertilizer, sediment and other pollutants also have the potential to overload stormwater BMPs, which are generally designed for lower pollutant concentrations found in typical wet weather flows. Homeowners are required to direct downspout and sump pump discharges to swales, lawns and gardens (keeping away from foundation backfill zones) where water can infiltrate.

In winter months, snow and ice melt are the primary causes of nuisance flows and associated icing problems (see **Photograph 3.6-1**).

Photograph 3.6-1. The location of inlets is important to address the effects of nuisance flows.

Snow and ice melt can re-freeze on streets and sidewalks, where it poses hazards to the public and is difficult to remove. Often, icing is most significant on east-west streets that have less solar exposure in the winter. Trees, buildings, fences and topography can also create shady areas where ice accumulates. Snow and ice may also clog drains and inlets leading to flooding. Snowmelt has been found to have high pollutant concentrations which can stress water quality facilities. Because many of the issues related to winter nuisance flows are beyond the control of the City (especially in areas that are already developed), identifying problem areas and incorporating maintenance objectives into the planning and design process is often the most effective practice for minimizing nuisance conditions. **Table 3.6-1** provides the various sources, problems and avoidance strategies associated with nuisance flows.

Table 3.6-1. Nuisance Flows: Sources, Problems and Avoidance Strategies

	Warm Weather	Cold Weather
Examples/Sources	• Over-irrigation of landscaping • Car washing • Sump pump discharge	• Snow melt • Ice melt • Sump pump discharge
Problems	• Poor water quality • High-nutrient concentration • High-pollutant concentration • Algae growth	• Icing leading to inlet blockage and flooding • Ice on streets and sidewalks • High-pollutant concentration
Avoidance Strategies	• Irrigation, drainage and fertilizer education • Proper drainage design • Minimization of directly connected impervious area • Sidewalk chase drains	• Inlet, chase and sidewalk maintenance • Prompt and frequent snow and ice removal • Consider additional inlets in strategic locations • Shoveling snow onto grassy areas away from streets and inlets • Locate inlets and sumps away from shaded areas

Photograph 3.6-2. Inlets frequently need maintenance.

For new development projects, locating inlets in areas where water can be intercepted before it accumulates or slows down and has the opportunity to freeze is the most effective way to minimize icing from the design perspective. To the extent practical, locate inlets away from areas that will be heavily shaded during winter months (in particular the north side of buildings to help prevent ice build-up and allow proper flow. For areas where shading is unavoidable, consider providing additional inlet capacity at strategic locations. For example, if a street with a southern exposure will drain to an east-west street that is shaded, having additional inlet capacity at the intersection may be advisable, especially if the flow is intended to turn and follow the east-west street. It is also important to consider potential future vegetative growth when evaluating shading effects. Although trees may be small and have little canopy when originally planted, they will grow and ultimately provide far greater tree canopy than when initially planted. Tree canopy may vary seasonally; depending on the tree species (e.g. deciduous trees lose their leaves in the fall and less canopy is present in the winter). Ultimately, even with careful placement of inlets and avoidance of shading to the extent practical, icing in some locations will likely occur due to shading from buildings, fences and other improvements on private property and maintenance to remove accumulated ice will be necessary.

CITY OF FORT COLLINS POLICY ON THE USE OF SUMP PUMPS:

- DISCHARGE FROM FOUNDATION DRAINS, PRIVATE LOT STORM DRAINAGE PIPES AND SUMP PUMPS MUST COMPLY WITH ALL APPLICABLE STATE AND LOCAL REQUIREMENTS. CITY CODE, SECTION 26-214 STATES THAT STORMWATER AND ALL OTHER UNPOLLUTED DRAINAGE WATER SHALL ONLY BE DISCHARGED TO SUCH STORMWATER FACILITIES AS ARE SPECIFICALLY AUTHORIZED FOR SUCH DISCHARGE BY THE UTILITIES EXECUTIVE DIRECTOR, PROVIDED HOWEVER, THAT IN NO EVENT SHALL NON-STORMWATER RUNOFF (WHICH INCLUDES LANDSCAPE IRRIGATION, UNCONTAMINATED PUMPED, INFILTRATED OR RISING GROUND WATER, AND FLOWS FROM PROPERLY INSTALLED, OPERATED AND CITY-APPROVED FOOTING, FOUNDATION OR CRAWL SPACE DRAIN OR PUMP) OR WATER FROM NATURAL SPRINGS BE PERMITTED TO BE DISCHARGED ONTO OR UPON ANY STREET, SIDEWALK OR GUTTER. ADDITIONALLY, CITY CODE, SECTION 26-498 PROHIBITS CONNECTIONS TO A STORM DRAINAGE FACILITY TO CONVEY FLOWS OTHER THAN STORM DRAINAGE AND UNCONTAMINATED GROUNDWATER FLOWS.
- DISCHARGE FROM SUMP PUMPS MAY BE TIED TO THE CITY'S <u>STORMWATER SYSTEM</u> UPON APPROVAL FROM THE UTILITIES EXECUTIVE DIRECTOR, BUT MAY NOT DISCHARGE DIRECTLY TO A STREET SURFACE. ALL TIE-IN POINTS MUST BE INSTALLED AT APPROVED LOCATIONS SUCH AS AT A MANHOLE OR AT AN INLET. <u>NO DIRECT TIE-IN TO A STORM DRAIN PIPE WILL BE ALLOWED</u>. SUMP PUMP DISCHARGE FLOWS CAN ONLY BE RELEASED INTO A STORMWATER CONVEYANCE SYSTEM (SUCH AS PIPE JUNCTIONS, CHANNELS OR PONDS) SPECIFICALLY DESIGNED AND APPROVED BY THE CITY TO ACCEPT SUCH DISCHARGE.
- PLEASE REFER TO CITY CODE SECTIONS 26-214, 26-331, 26-491 AND 26-498 FOR FURTHER GUIDANCE.

Control of nuisance waters such as shallow ponding that occasionally concentrate on flat lawns, landscaped, paved or other such areas is strictly the responsibility of the property owner of the land where ponding occurs. The City will make reasonable efforts to minimize the occurrence of such nuisances through its review and inspection authorities, but if such nuisances do occur, the City is not responsible or obligated to correct or require any other party to correct such a problem.

For more information on nuisance flows, multiple Colorado-based publications are available to provide guidance related to landscape management practices and snow and ice removal. Representative resources include:

- UDFCD Manual, Volume 3, Source Control BMPs
- GreenCO BMP Manual
- Colorado State University Extension Yard and Garden Fact Sheets

4.0 Storm Drain Systems

4.1 Introduction

Once stormwater is collected from the street by an inlet, it is directed into the storm drain system. The storm drain system is comprised of inlets, pipes, manholes, outlets and other appurtenances. For specific information regarding the applicability of a number of available pipe materials, a document titled "Storm Sewer Pipe Material Technical Memorandum" is available for download at www.udfcd.org

Apart from inlets, manholes are the most common appurtenance in storm drain systems. Their primary functions include:

- Providing maintenance access
- Serving as junctions when two or more pipes merge
- Providing flow transitions for changes in pipe size, slope and alignment
- Providing ventilation

Manholes are generally made of precast or cast-in-place reinforced concrete. They are typically 48 inches (48") or 60 inches (60") in diameter depending on the pipe size and orientation. Manholes are required at regular intervals for maintenance requirements. Maximum spacing of 400' is required, even along straight sections of piping. Standard size

STORM SYSTEM MANHOLES:

- REQUIRED TO BE PLACED AT ALL JUNCTIONS, INTERSECTIONS, CHANGE IN PIPE DIAMETER AND CHANGE IN SLOPE
- MUST BE PLACED AT 400' MAX SPACING, EVEN ALONG STRAIGHT SECTIONS
- INVERT DROPS IN MANHOLES SHOULD BE 0.1' WHEREVER POSSIBLE
- MAXIMUM VELOCITY OF 20 FPS THROUGH STORM SYSTEMS
- OUTLET TRANSITIONS (I.E. FLARED-END SECTIONS) ARE REQUIRED FOR TRANSITIONS FROM PIPE TO OPEN CHANNEL FLOW TO REDUCE VELOCITY AND EROSION.

manholes cannot accommodate large pipes, so special junction vaults are constructed for that application.

Outlet structures are transitions from pipe flow to open channel flow or still water (e.g. ponds, lakes, etc.). Their primary function is to provide a transition that minimizes erosion and controls flow rates into the receiving water body. Occasionally, flap gates or other types of check valves are placed on outlet structures to prevent backflow from high tailwater or flood-prone receiving waters.

Reference: FCU requires that the construction of all stormwater facilities must be built in accordance the Development Construction Standards for Water, Wastewater and Stormwater.

4.2 Easements for Storm Pipes

Required minimum widths of drainage easements for common types of drainage facilities are listed in **Table 4.2-1**.

Table 4.2-1: Required Drainage Easements for Pipes

Drainage Facility: **Storm Sewer Pipe Diameter < 36”**	**Minimum Easement Width**
Depth to Invert < 5’	20’
5’ < Depth to Invert ≤ 10’	30’
Depth to Invert > 10’	30’ minimum or [Pipe I.D. + 6 + Depth x 2]
Storm Sewer Pipe Diameter ≥ 36”	
Depth to Invert < 5’	20’ minimum or [Pipe I.D. + 7 + Depth x 2]
5’ < Depth to Invert ≤ 10’	30’ minimum or [Pipe I.D. + 7 + Depth x 2]
Depth to Invert > 10’	[Pipe I.D. + 7 + Depth x 2]

4.3 Design Process, Considerations and Constraints

The design of a storm drain system requires a large data collection effort. The data requirements in the proposed service area include topography, drainage boundaries, imperviousness, soil types and locations of any existing storm drain conduits, inlets and manholes. In addition, identification of the type and location of other utilities in the ground is critical. Alternative layouts of a new system (or modifications to an existing system) can be investigated using these data.

System layouts rely largely on street rights-of-way and topography. Most layouts are dendritic (tree) networks that follow the street pattern. Dendritic networks collect stormwater from a broad area and converge in the downstream direction. Networks with parallel branches are possible but sometimes less desirable. Each layout should depict inlet and manhole locations, drainage boundaries services by the inlets, pipe locations, flow direction and outlet locations. A final layout selection is made from the viable alternatives based on likely system performance and cost.

STORM PIPES IN THE RIGHT-OF-WAY:

- MINIMUM DIAMETER IS 15" OR EQUIVALENT.
- NEW INLET/PIPE SYSTEMS AND STREETS TO BE SIZED TO CONVEY THE 100-YEAR STORM.
- PIPELINE HGL AND EGL TO BE A MINIMUM OF 12" BELOW THE SURFACE.

Once a final layout is chosen, storm drain pipes are sized based on the hydrology (peak flows) and hydraulics (pipe capacities). This is accomplished by designing the upstream pipes first and moving downstream. Pipe diameters less than 15 inches (15") are not recommended for storm drains. The City requires that the minimum pipe diameter for public storm pipes and all pipes located in the public right-of-way is 15 inches (15"), or a minimum vertical dimension of twelve inches (12") if elliptical or arch pipe is used.

Pipes generally increase in size moving downstream since the drainage area (and thus flow) is increasing. Downstream pipes should never be smaller than upstream pipes, even if a steeper slope is encountered that will provide sufficient capacity with a smaller pipe. The potential for clogging at the resulting "choke point" is always a concern.

Storm pipes are typically sized to convey the minor storm without surcharging; using open channel hydraulics calculations to determine normal depth 100% full pipe depth. However, storm pipes need to be sized for the full amount of stormwater that is able to reach the pipes from inlets or other appurtenances. For example, if an inlet is able to convey 20 cfs to the storm piping system during the 100-year storm, then the pipes need to be sized to safely convey the 20 cfs while keeping the HGL and EGL below the surface of the roadway.

Because the maximum capacity of a circular pipe occurs at approximately 93% of the depth of full pipe flow, designing for full flow results in a slightly conservative design. FCU requires that the combination of storm piping systems and streets are required to accommodate the major storm without exceeding encroachment standards or hydraulic/energy grade line requirements as set forth in this Manual.

Manholes are located in the system in conjunction with pipe sizing and inlet placement, where manhole locations are dictated by standard design practices. For example, manholes are required whenever there is a lateral pipe servicing an inlet, and where a change occurs in pipe size, alignment, or slope. In addition, manholes are required at pipe branch junctions. Manholes are also required along long straight section of pipe for maintenance purposes, with the distance between manholes dependent on pipe size, but not more than 400 feet. Whenever possible, the invert of a pipe leaving a manhole should be at least 0.1 foot lower than the incoming pipe to ensure positive flow flows through the manhole. However, FCU allows for 0 foot drop across the inlet or manhole when a 0.1 foot drop is not possible. Whenever possible, match the pipe soffit elevations when the downstream pipe is larger to minimize backwater effects on the upstream pipe. Additional manholes may be necessary to "step down" a steep grade, allowing pipe slopes to be much flatter than the slope of the street above. This is done to prevent velocities in storm drain pipes from exceeding the recommended maximum velocity of 20 fps.

Once storm drain pipes are sized and manhole locations are determined, the performance of the storm drain system must be evaluated using energy grade line (EGL) calculations starting at the downstream system outlet. As stormwater flows through the storm drain system, it encounters many flow transitions. These transitions include changes in pipe size, slope and alignment, as well as entrance and exit conditions. All of these transitions consume energy, resulting in energy losses expressed as head losses. These losses must be accounted for to ensure that inlets and manholes do not surcharge to a significant degree (i.e. produce street flooding). This is accomplished using hydraulic grade line (HGL) calculations as a check on pipe sizes and system losses. If significant surcharging occurs, the pipe sizes should be increased. High tail water conditions at the storm drain outlet may also produce surcharging. This can also be accounted for using HGL calculations.

FCU requires that if HGL is surcharged along the pipe, the EGL will need to be determined and shown on the design plans to ensure that the EGL does not elevate above the finished surface. FCU requires that the EGL is a minimum of twelve inches (12") below the manhole lid elevation and/or flowline elevation at the inlet. Bolt-down lids are not allowed except by variance. This requirement applies to both public and private storm drainage systems.

4.4 Storm Drain Hydrology and Hydraulics

The Streets, Inlets and Storm Drains chapter in the UDFCD Manual provides a comprehensive section on the hydraulic design for pipe systems, the details of which are not included in this Manual. The UD-Culvert and UD-Sewer software downloads are available at www.udfcd.org are common tools used to properly size culverts and pipe systems. Bentley Flowmaster and other pipe calculator software's are also accepted for use by FCU. Care must be taken by the Design Engineer use the proper loss coefficients for input into the software. The methodology behind determining the proper loss coefficients are provided in this same chapter of the UDFCM Manual.

The depth of flow in the receiving stream must be taken into consideration for backwater computations for both the minor and major storm runoff. An analysis of the joint probability of occurrence may be warranted based on the standards described below. FEMA recommends modeling a 10-year water surface in the receiving stream for a 100-year tributary discharge. HEC-22 also provides guidance based on the ratio of main stream watershed area to that of the tributary stream. FCU follows FEMA recommended standards for hydraulic modeling tie-in to the following waterways:

- Poudre River – 2-year water surface elevation

- Spring Creek – 10-year water surface elevation

Backwater hydraulics analysis for storm pipe systems entering detention basins:

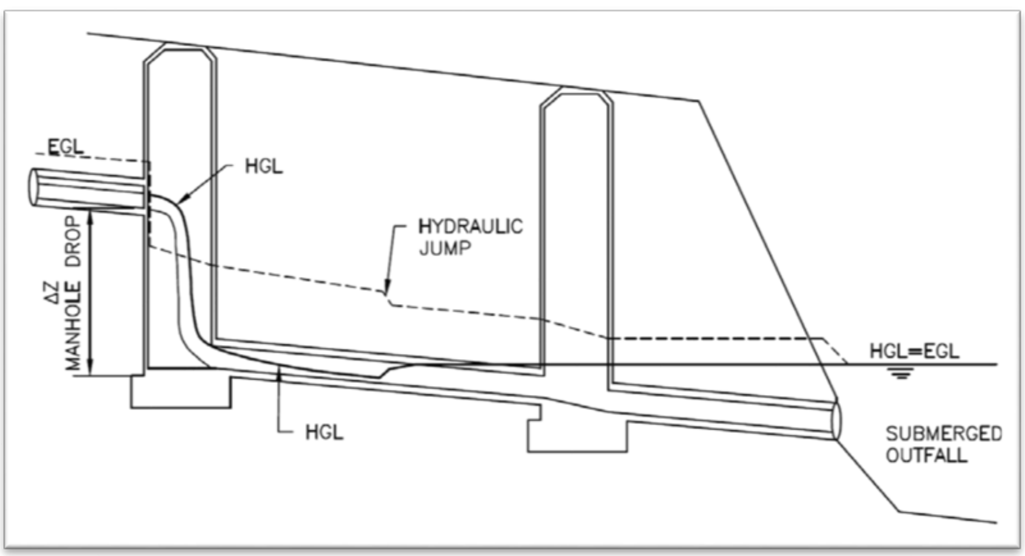
- Shall be based upon the 100-year water surface elevation in the detention basin or the emergency spillway elevation if that is higher. Alternatively, if a SWMM model is prepared for the site, it may be utilized in sizing the storm pipes.

- Storm pipe systems (including roof drains and underdrains) entering detention basins are required to enter at the bottom elevation of that area of the basin and are not allowed to enter at a higher elevation due to erosion issues

Backwater hydraulics analysis for storm pipe systems entering irrigation ditches:

- Shall be based upon the normal operating water surface elevation (as determined by the irrigation ditch or reservoir company). It is typical, however, for the irrigation ditch or reservoir company to require storm pipe tie-ins above the normal operating water surface elevation and/or include flap gates at the outfall. Specific approvals and coordination would need to be conducted with the irrigation ditch or reservoir company for this circumstance.

Figure 4.4.1-1. Hydraulic and Energy Grade Lines



5.0 Swales

The functions and benefits from natural streams can be extended further upstream in the watershed by conveying runoff on the surface in vegetated channels and swales rather than in underground storm drains. Besides the aesthetic and habitat value of surface channels, stormwater quality can be enhanced by promoting beneficial interaction between water, soil and vegetation. Conveyance in storm drains produces no such interactions or water quality enhancement.

Guidance is provided in this subsection for the design of swales, draining areas from less than an acre up to about 10 impervious acres (e.g. 20 acres at 50% imperviousness). A series of design charts are provided to guide the designer in determining stable conditions in vegetated or void-filled riprap swales of varying cross-sections based on design flow rate and slope. The charts show flow rates as high as 100 cfs (stable at relatively flat slopes) and slopes as steep as ten percent (10%) (stable at relatively low flows). It should be noted that the design criteria in this section differs from those in Chapter 7: Water Quality, of this Manual. Those criteria are intended to provide a higher level of water quality treatment. These criteria are intended for stable conveyance more so than water quality benefits.

5.1 Design Criteria for Swales

All open channels shall be designed with freeboard. Freeboard for major channels (defined as those with capacity in excess of one hundred (100) cfs) must be a minimum of one foot (1') of extra depth. Freeboard for minor channels (defined as those carrying less than one hundred (100) cfs design flow) must be designed to handle a minimum of an additional 33 percent of runoff, over and above the 100-year design flow.

Design criteria are described for grass and rock (soil riprap or void-filled riprap) swales. Where indicated by **Figures 5.1.1-1** through **5.1.1-4**, grass swales meeting these criteria are preferred, but when conditions require, swales lined with soil riprap or void-filled riprap are advisable. When designing grass-lined swales, a Froude No. ≤ 0.8 is required.

In order to maximize the use of grass swales, and increase the likelihood that the swale will remain functional and stable over time, two key design principles should be considered.

1) **Adopt shallow swale section with flat bottom.** Swale cross-sections that allow runoff to spread out (shallow, flat bottom with gentle side slopes) promote lower velocities and shear stresses than triangular (or "V" shaped) swales. This is also good for water quality. In general, the wider the bottom width of the swale, the more stable it will be, although concentrated flow paths may still form. It is generally recommended that swales be of a trapezoidal shape with a bottom width of 2 feet or more with side slopes that are 5:1 or flatter.

2) **Establish dense turf-forming grass in suitable soils**. The single most important factor in creating stable grass swales is to establish a dense stand of turf-forming grass in the bottom and side slopes of the swale. This requires good soils or amendments and proper soil

preparation and planting. Irrigation may also be necessary. See Chapter 4: Construction Control Measures, for more information.

5.1.1 Stability Charts

Swale stability based on slope, flow rate, swale geometry and grass or rock lining are shown graphically in **Figures 5.1.1-1** through **5.1.1-4**. Design guidance is provided in the stability charts for design discharges up to 100 cfs for longitudinal slopes up to ten percent (10%). Although these figures go up to 100 cfs, it may be appropriate to design a more naturalized channel section for flow rates greater than 30 or 40 cfs. This is largely dependent on site-specific considerations. As already mentioned, steep swales are most feasible for small discharges while swales carrying large discharges are most feasible at flatter slopes. If the chart is indicating that riprap greater than Type H (see **Figure 5.1.1-3**) is required, a swale for those hydraulic conditions will not be allowed. Typically, if Type H riprap is shown to be required, other design options such as widening the swale or flattening the slope must be explored.

The use of **Figures 5.1.1-1** through **5.1.1-4** for swale stability analysis requires that geometric parameters indicated at the top of each chart apply and the requirements of Section 5.2 for grass swales and Section 5.3 for soil riprap or void-filled riprap are met.

Table 5.1-1 below summarizes the appropriate stability chart to reference based upon the swale geometry.

Table 5.1-1. Summary of swale properties for stability chart reference

Bottom Width	Side Slope	Stability Chart
2-4 feet	Between 5:1 and 10:1	Figure 5.1.1-1
2-4 feet	10:1 or flatter	Figure 5.1.1-2
Greater than 4 feet	Between 5:1 and 10:1	Figure 5.1.1-3
Greater than 4 feet	10:1 or flatter	Figure 5.1.1-4

For swales outside the range of application of **Figures 5.1.1-1** through **5.1.1-4**, specific analysis of the proposed swale parameters may be required.

5.2 Grass Swales

5.2.1 Soil and Vegetation Properties

The single most important factor governing the stability of grass swales is the quality of vegetation. Chapter 4: Construction Control Measures provides recommended seed mixes when specific seed mixes are not provided in the Landscape Plans. Turf-forming grasses that include a variety of species work best.

In addition to seeding, it is recommended that grass plugs of the dominant species in the seed mix be planted to provide some immediate vegetative cover and improve overall establishment. Place drier species on the side slopes. Placing sod is also an option for grass swales.

5.2.2 Construction

It is imperative that the construction drawings and specifications address seedbed preparation; installation of seed, blankets and plugs; temporary irrigation; weed control; and follow-up reseeding and maintenance. Specific construction recommendations, including for submittals and inspections, can be found in Chapter 4: Construction Control Measures. Good temporary erosion controls are critical during establishment of vegetation.

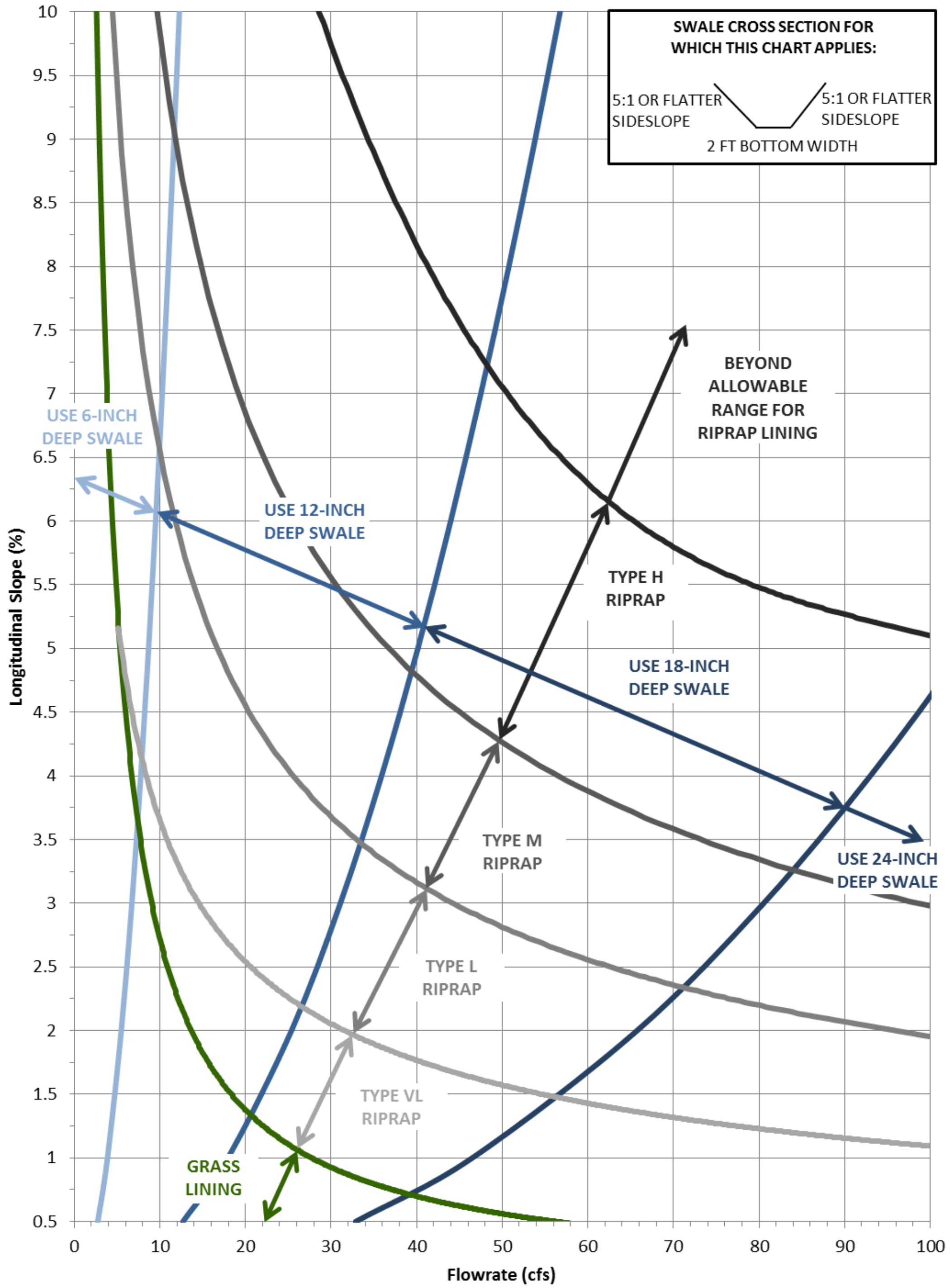


Figure 5.1.1-1. Swale stability chart; 2-4 foot bottom width and side slopes between 5:1 and 10:1
(Note: Riprap classifications refer to gradation for riprap used in soil riprap or void-filled riprap. See Figure 8-34 for gradations.) (Source: Muller Engineering Company)

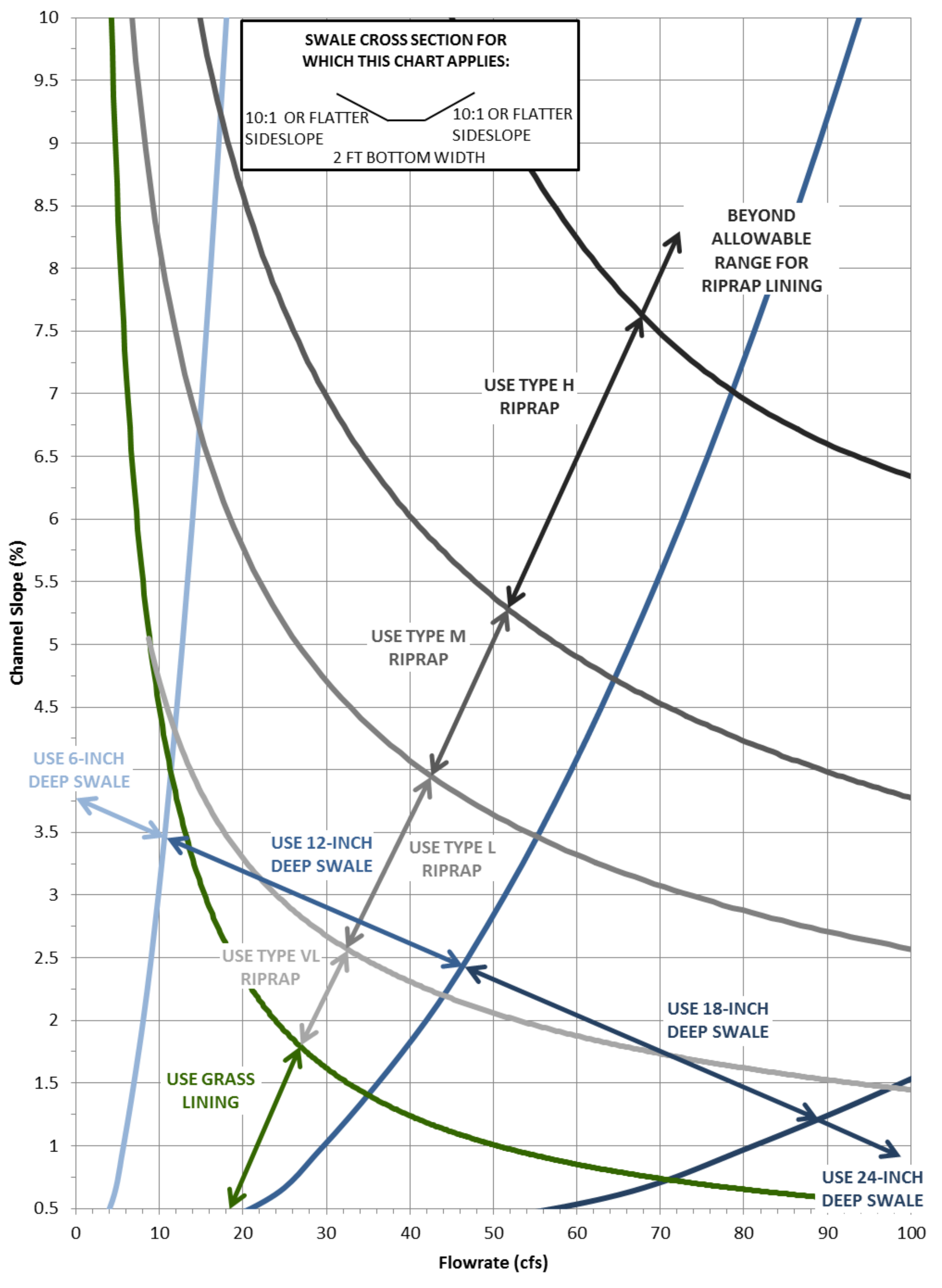


Figure 5.1.1-2. Swale stability chart; 2-4 foot bottom width and 10:1 (or flatter) side slopes

(Note: Riprap classifications refer to gradation for riprap used in soil riprap or void-filled riprap. See Figure 8-34 for gradations.) (Source: Muller Engineering Company)

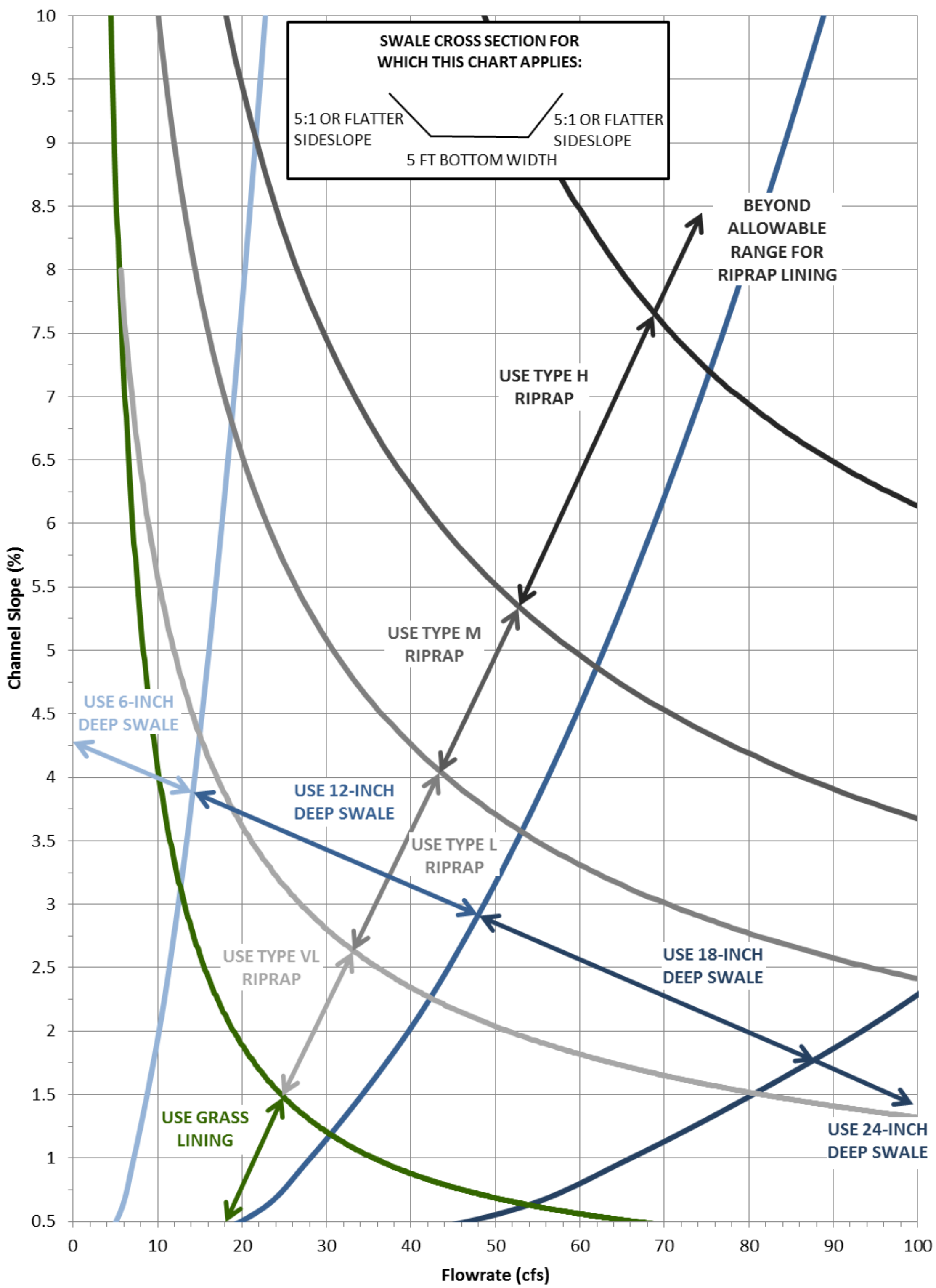


Figure 5.1.1-3. Swale stability chart; greater than 4 foot bottom width and side slopes between 5:1 and 10:1

(Note: Riprap classifications refer to gradation for riprap used in soil riprap or void-filled riprap. See Figure 8-34 for gradations.) (Source: Muller Engineering Company)

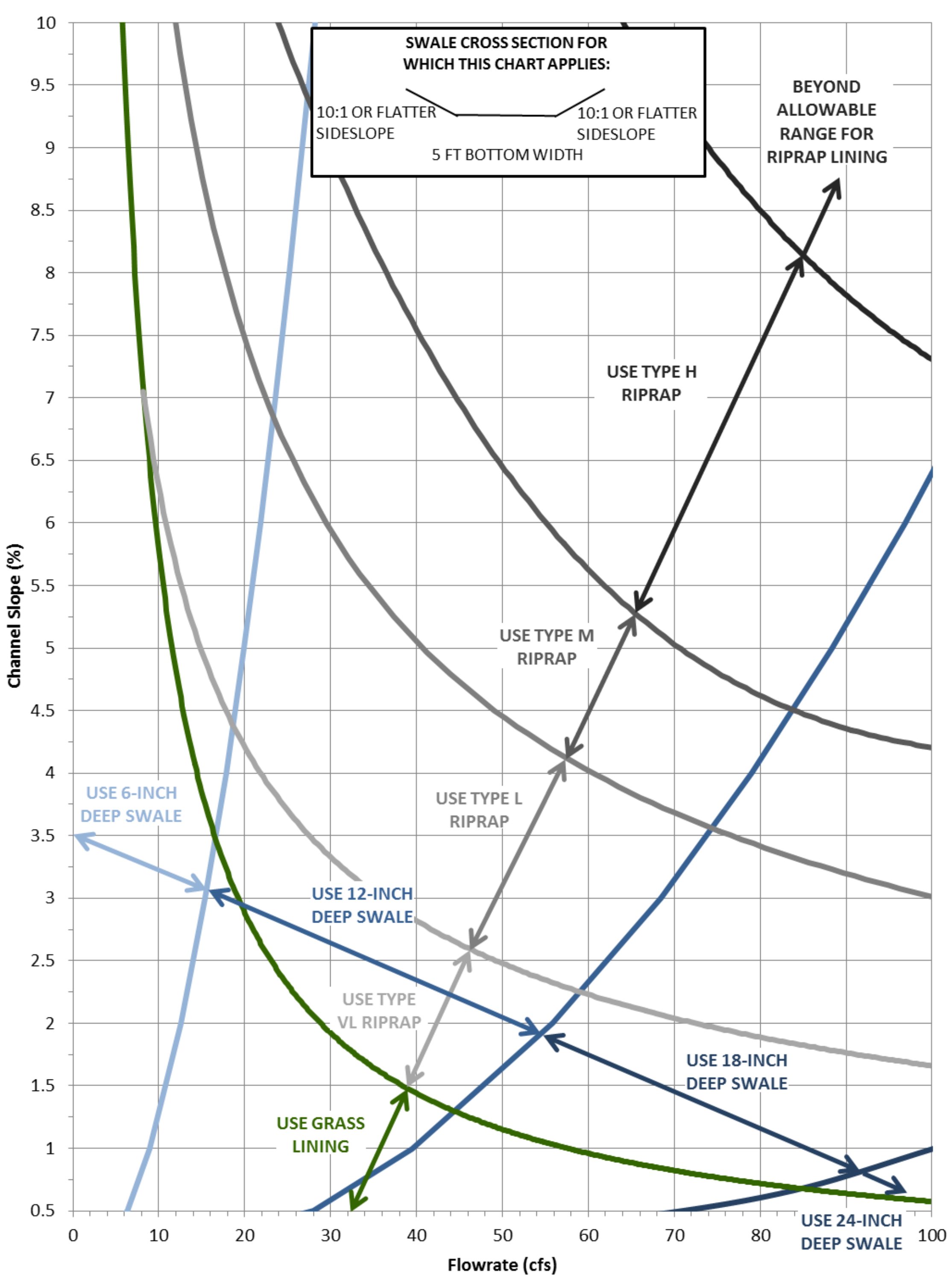


Figure 5.1.1-4. Swale stability chart; greater than 4 foot width and 10:1 (or flatter) side slopes
(Note: Riprap classifications refer to gradation for riprap used in soil riprap or void-filled riprap. See Figure 8-34 for gradations.) (Source: Muller Engineering Company)

6.0 Use of Irrigation Ditches

The use of irrigation ditches for stormwater conveyance or outfall purposes must be in accordance with the policy discussed in Chapter 1: Drainage Principles and Policies.

FCU requires the appropriate owner's / ditch and reservoir company's approval, whether public or private if improvements cause any of the following:

1) Alteration of the existing patterns of drainage into irrigation ditches
2) Increased volumes discharged into the ditch
3) Changes in the quality of runoff entering the ditch
4) Change in the historic point of discharge into the ditch
5) Any proposed ditch crossing(s) or relocation(s)
6) Any proposed grading within the ditch easement
7) Access to the ditch easement during construction activities

This approval may be in the form of signature on the construction plans or documents. If determined by the Utilities Executive Director to be sufficient, other formal legal agreements may be substituted for an approval signature on the construction plans. The list above is not exhaustive and represents examples of circumstances when ditch or reservoir company approval is required. Early contact with affected companies may be beneficial.

In the rare instance where an irrigation ditch is allowed to serve as the outfall for a stormwater facility the following provisions, at a minimum, must be met:

1) The maximum water surface elevation must be determined based on the maximum amount of irrigation flow in the ditch. The appropriate owner / ditch or reservoir company is the determining authority in regard to the maximum irrigation flow in the ditch. Written verification of the maximum irrigation flow from the owner / ditch or reservoir company must be submitted with the hydraulic analysis of the ditch water surface elevation.
2) The maximum water surface elevation of the ditch must then be determined by combining the maximum irrigation flow in the ditch with the 100-year stormwater flows in the ditch.
3) The detention outlet must be designed such that backflow from the ditch into the detention facility is prevented.

4) The backwater effects caused by the design of a detention outlet, if any, must be reviewed and approved by both FCU and the appropriate ditch or reservoir company.

5) The outlet design must consider tailwater effects on the outlet pipe resulting from the combination of the maximum irrigation flow and the 100-year storm discharge within the ditch.

6) The 100-year water surface elevation of the ditch must be determined using the appropriate Master Drainage Plan or if not available, additional studies may be required from the party seeking to discharge into the ditch. For cases where 100-year discharges are not available, upstream restrictions or structure capacities can be considered for determining ditch flows.

If new developments are adjacent to irrigation facilities but no flows are being directed into the ditch, the owner/ ditch or reservoir company must still be notified of the proposed development. In such cases, ditch or reservoir company approval shall be required prior to any approval by FCU, unless upon written request by the applicant, the Utilities Executive Director determines that the development will result in no impact on or to the ditch or reservoir company and that there will be no impact on stormwater flows or improvements from the adjacent irrigation facilities.

The party seeking modifications to existing ditch conditions must obtain the appropriate owner / ditch or reservoir company approvals and signatures prior to seeking FCU approval for such modifications.

When privately owned and maintained irrigation facilities abut private property, it is the responsibility of the private parties involved to develop and implement a policy regarding safety.

7.0 Energy Dissipation and Erosion Protection

Local scour is typified by a scour hole produced at a pipe or culvert outlet. This is the result of high exit velocities, and the effects extend only a limited distance downstream. Coarse material scoured from the circular or elongated hole is deposited immediately downstream, often forming a low bar. Finer material is transported farther downstream. The dimensions of the scour hole change due to sedimentation during low flows and the varying erosive effects of storm events. The scour hole is generally deepest during passage of the peak flow.

Protection against scour at outlets ranges from limited riprap placement to complex and expensive energy dissipation devices. Pre-formed scour holes (approximating the configuration of naturally formed holes) dissipate energy while providing a protective lining to the streambed.

This section addresses energy dissipation and erosion control utilizing riprap and other measures that can be used to minimize or eliminate local scour at a pipe outlet. In general, these measures may pose risks to the public. Discourage public access and minimize the risk of falls at these structures.

Scour and Stream Degradation: Scour is typically found at culvert outlets and other isolated transitional areas within a stream. Frequently, scour holes fill in with sediment over time only to be reformed during infrequent high flows. Degradation is a phenomenon that is independent of culvert performance. Natural causes can produce a lowering of the streambed over time. Contributing factors include the slope of the stream and the size and availability of the sediment load. Degradation can also be a result of other constructed features such as upstream detention or increased watershed imperviousness. The identification of a degrading stream is an essential part of the original site investigation.

Reference: Methods for predicting scour hole dimensions are found in the Hydraulic Design of Energy Dissipators for Culverts and Channels (FHWA 1983 and 2000).

7.1 Use of Riprap Policy

Riprap should only be used when other methods of protection or stabilization are not appropriate or possible. Alternatives to riprap are generally recommended:

- Manufactured channel lining or revetment treatments such as Turf Reinforcement Mats (TRMs)
- Erosion control matting
- Geotextiles
- Articulating Concrete Blocks (ACBs)
- Other flexible linings

These alternates will be considered by FCU on a case-by-case basis in order to determine the most appropriate material that should be specified under particular conditions and for different applications.

When riprap is determined to be the best or only appropriate method for stabilization soil riprap may be utilized. Soil riprap is intended for use in applications where vegetative cover can be established in the riprap.

- FCU requires that four to six inches (4-6”) of topsoil on top of soil riprap is required to help establish vegetation.
- FCU requires that the minimum d_{50} (mean particle size intermediate dimension) by weight for riprap, is twelve inches (12”), or Type M riprap.

Gabions are not allowed.

7.2 Riprap Apron

This section addresses the use of riprap for erosion protection downstream of conduit and culvert outlets.

The length of the riprap protection downstream from the outlet depends on the degree of protection desired. If it is necessary to prevent all erosion, the riprap must be continued until the velocity has been reduced to an acceptable value. The acceptable major event velocity is set at five feet per second (5 fps) for non-cohesive soils and at seven feet per second (7 fps) for erosion resistant soils. The rate at which the velocity of a jet from a conduit outlet decreases is not well known. For the procedure recommended here, it is assumed to be related to the angle of lateral expansion, θ, of the jet. The velocity is related to the expansion factor, $(1/(2\tan\theta))$, which can be determined directly using **Figure 7.2-2 or 7.2-3**, by assuming that the expanding jet has a rectangular shape:

$$\mathbf{L_p} = \left(\frac{\mathbf{1}}{\mathbf{2\tan\theta}}\right)\left(\frac{\mathbf{A_t}}{\mathbf{Y_t}} - \mathbf{W}\right) \qquad \textbf{Equation 9-10}$$

Where:

L_p = length of protection, ft
W = width of the conduit (ft, use diameter for circular conduits)
Y_t = tailwater depth, ft
Θ = the expansion angle of the culvert flow

And:

$$\mathbf{A_t} = \frac{\mathbf{Q}}{\mathbf{V}} \qquad \textbf{Equation 9-11}$$

Where:

Q = design discharge, cfs
V = the allowable non-eroding velocity in the downstream channel, fps
A_t = required area of flow at the allowable velocity, ft^2

In no case should L_p be less than 3H or 3D, nor does L_p need to be greater than 10H or 10D whenever the Froude parameter, $Q/WH^{1.5}$ or $Q/D^{2.5}$ is less than 8.0 or 6.0, respectively. Whenever the Froude parameter is greater than these maximums, increase the maximum L_p required by $1/4D_c$ or 1/4H for circular or rectangular culverts, respectively, for each whole number by which the Froude parameter is greater than 8.0 or 6.0, respectively.

Once L_p has been determined, the width of the riprap protection at the furthest downstream point should be verified. This dimension is labeled "T" on **Figure 7.2-1**. The first step is to solve for θ using the results from **Figure 7.2-2 or 7.2-3**.

$$\boldsymbol{\theta} = \tan^{-1}\left(\frac{1}{2(\text{ExpansionFactor})}\right)$$ **Equation 9-12**

Where:

Expansion Factor = determined using **Figure 7.2-2** or **7.2-3**

T is then calculated using the following equation:

$$\mathbf{T} = 2\left(\mathbf{L_p}\tan\boldsymbol{\theta}\right) + \mathbf{W}$$ **Equation 9-13**

7.2.1 Multiple Conduit Installations

The procedures outlined in this section can be used to design outlet erosion protection for multi-barrel culvert installations by replacing the multiple barrels with a single hydraulically equivalent hypothetical rectangular conduit. The dimensions of the equivalent conduit may be established as follows:

1) Distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed; otherwise, the flow through each barrel must be computed.

2) Compute the Froude parameter $Q_i/D_{ci}^{2.5}$ (circular conduit) or $Q_i/W_iH_i^{1.5}$ (rectangular conduit), where the subscript "*i*" indicates the discharge and dimensions associated with an individual conduit.

3) If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit.

4) Make the height of the equivalent conduit, H_{eq}, equal to the height, or diameter, of the selected individual conduit.

5) The width of the equivalent conduit, W_{eq}, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $Q/W_iH_{eq}^{1.5}$.

Figure 7.2-1. Riprap apron schematic for culverts inline with the channel

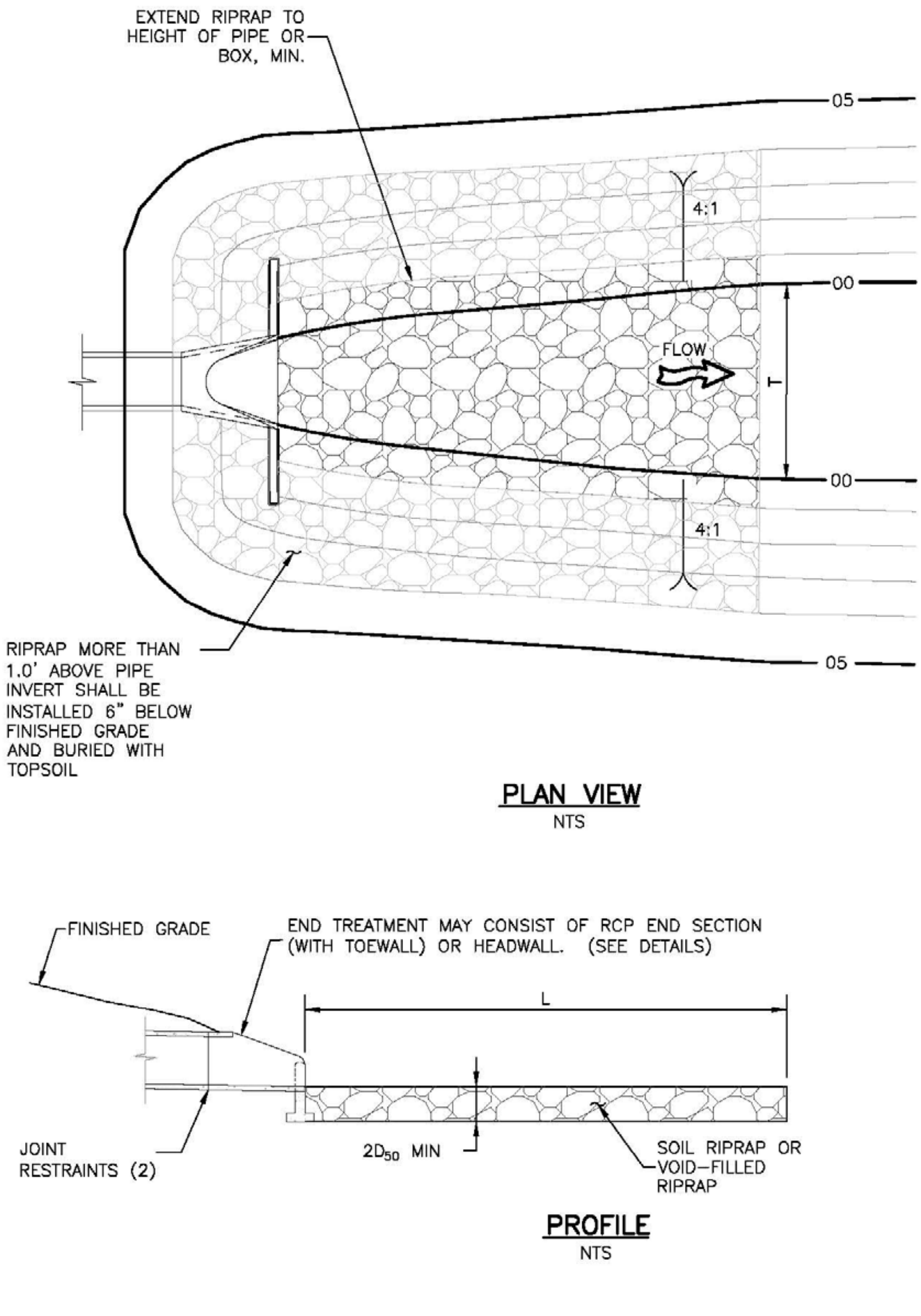


Figure 7.2-2. Expansion factor for circular conduits

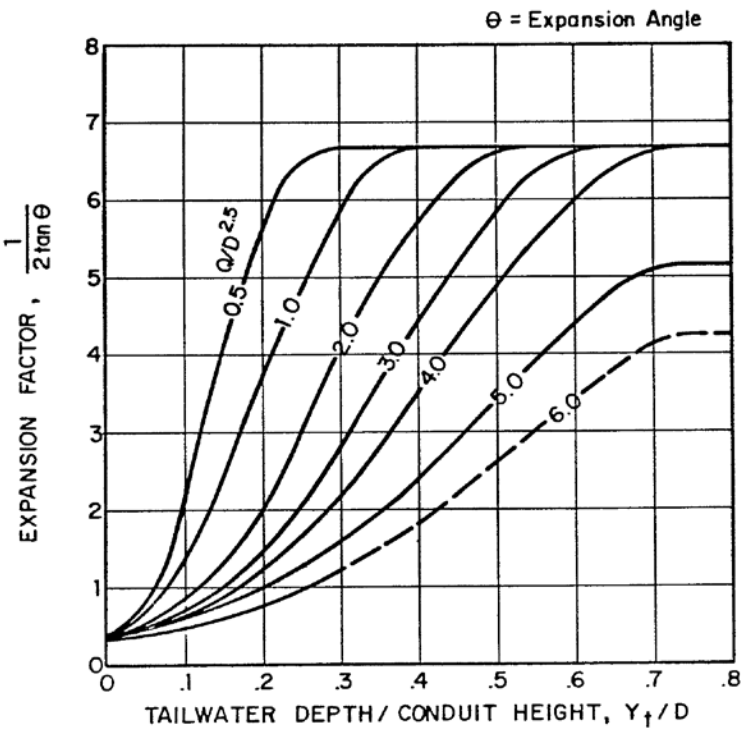
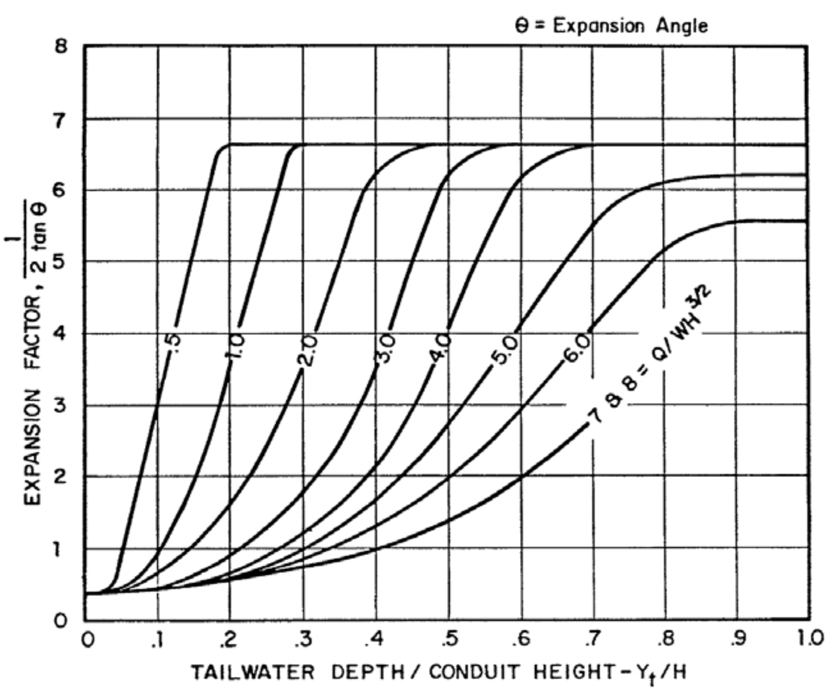


Figure 7.2-3. Expansion factor for rectangular conduits



7.3 Rock Sizing for Riprap Apron

Scour resulting from highly turbulent, rapidly decelerating flow is a common problem at conduit outlets. The following section summarizes the method for sizing riprap protection for both riprap aprons and low tailwater basins.

The required rock size may be selected from **Figure 7.2-2** for circular conduits and from **Figure 7.2-3** for rectangular conduits. **Figure 7.2-2** is valid for $Q/D_c^{2.5}$ of 6.0 or less and **Figure 7.2-3** is valid for $Q/WH^{1.5}$ of 8.0 or less. The parameters in these two figures are:

1) $Q/D^{1.5}$ or $Q/WH^{0.5}$ in which Q is the design discharge in cfs, D_c is the diameter of a circular conduit in feet, and W and H are the width and height of a rectangular conduit in feet.

2) Y_t/D_c or Y_t/H in Y_t is the tailwater depth in feet, D_c, is the diameter of a circular conduit in feet, and H is the height of a rectangular conduit in feet. In cases where Y_t is unknown or a hydraulic jump is suspected downstream of the outlet, use $Y_t/D_t = Y_t/H = 0.40$ when using **Figures 7.3-1** and **7.3-2**.

3) The riprap size requirements in **Figures 7.3-1** and **7.3-2** are based on the non-dimensional parametric **Equations 9-14** and **9-15** (Steven, Simons and Watts 1971 and Smith 1975).

Circular culvert:

$$\mathbf{d_{50} = \frac{0.023Q}{Y_t^{1.2}D_c^{0.3}}} \qquad \textbf{Equation 9-14}$$

Rectangular culvert:

$$\mathbf{d_{50} = \frac{0.014H^{0.5}Q}{Y_tW}} \qquad \textbf{Equation 9-15}$$

These rock requirements assume that the flow in the culvert is subcritical. It is possible to use **Equations 9-14** and **9-15** when the flow in the culvert is supercritical (and less than full) if the value of Dc or H is modified for use in **Figures 7.3-1** and **7.3-2**. Whenever the flow is supercritical in the culvert, substitute D_a for D_c and H_a for H, in which D_a is defined as:

$$\mathbf{D_a = \frac{(D_c + Y_n)}{2}} \qquad \textbf{Equation 9-16}$$

Where the maximum value of D_a shall not exceed D_c, and:

$$\mathbf{H_a = \frac{(H + Y_n)}{2}} \qquad \textbf{Equation 9-17}$$

Where the maximum value of H_a shall not exceed H, and:

D_a = parameter to use in place of D, in **Figure 7.3-1** when flow is supercritical, ft

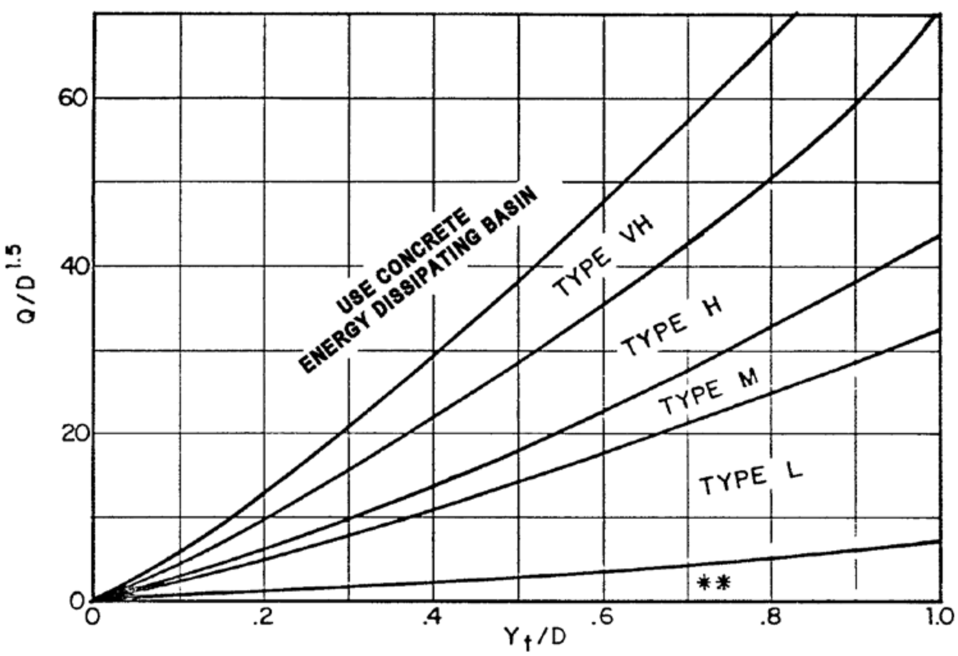
D_c = diameter of circular culvert, ft

H_a = parameter to use in place of H in **Figure 7.3-2** when flow is supercritical, ft

H = height of rectangular culvert, ft

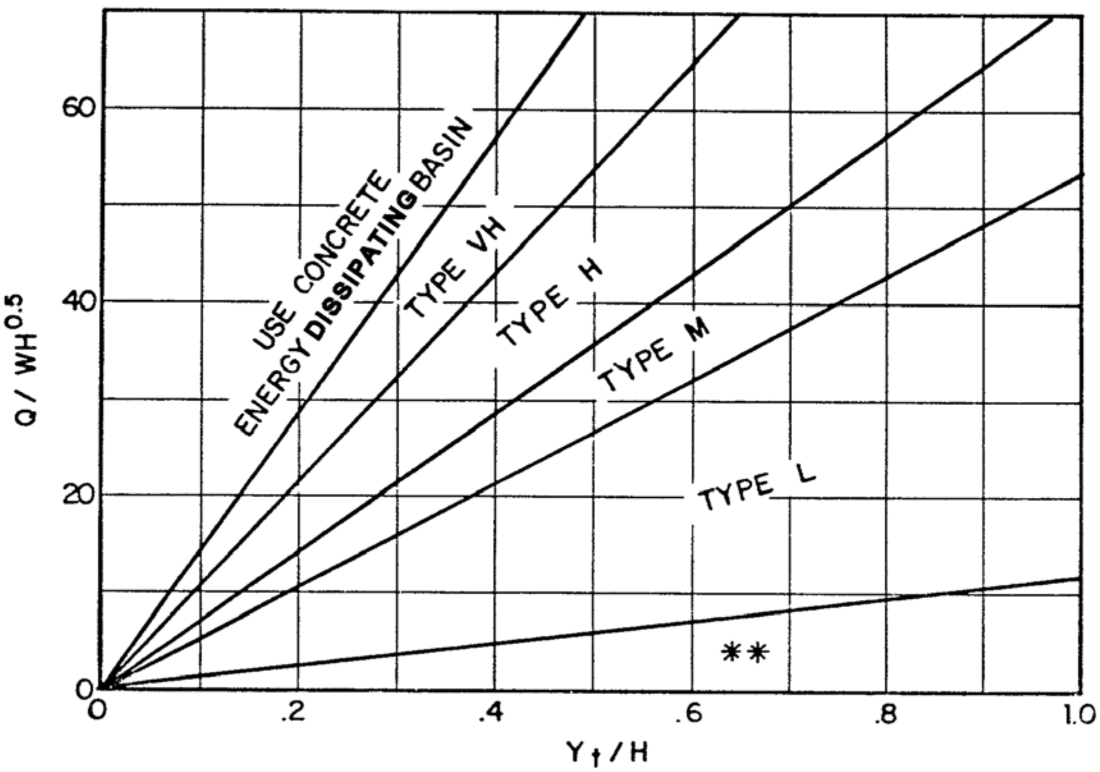
Y_n = normal depth of supercritical flow in the culvert, ft

Figure 7.3-1. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{1.5} \le 6.0$)



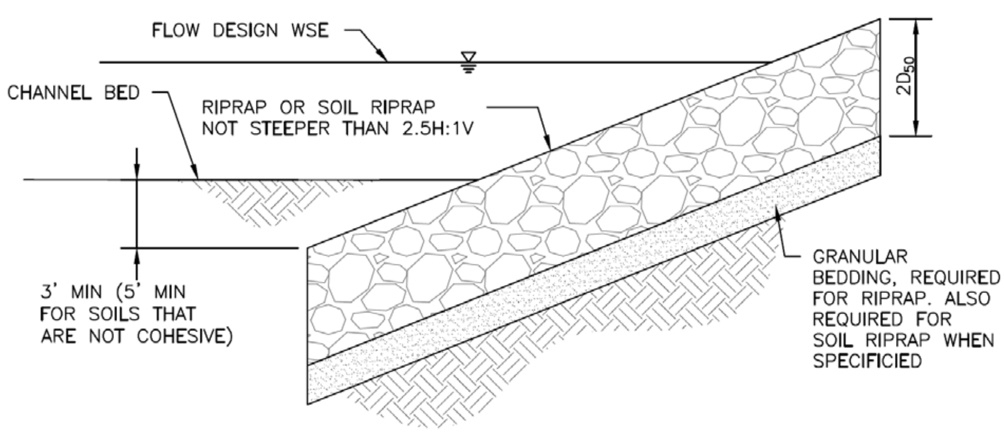
Use D_a instead of D whenever flow is supercritical in the barrel.

Figure 7.3-2. Riprap erosion protection at rectangular conduit outlet (valid for $Q/WH^{0.5} \leq 8.0$)



Use H_a instead of H whenever culvert has supercritical flow in the barrel.

Figure 7.3-3. Riprap and soil riprap placement and gradation (part 1 of 3)



Riprap and Soil Riprap Placement and Gradation (Part 1 of 3)

Riprap Designation	% Smaller Than Given Size by Weight	Intermediate Rock Dimension (inches)	Mean Rock Size, D_{50} (inches)
Type M	70-100	21	12
	50-70	18	
	35-50	12	
	2-10	4	
Type H	70-100	30	18
	50-70	24	
	35-50	18	
	2-10	6	

Soil Riprap Notes:

1.) Elevation tolerances for the soil riprap shall be 0.10 feet. Thickness of soil riprap shall be no less than thickness shown and not more than two inches (2”) greater than the thickness shown.

2.) Where “soil riprap” is designated on the contract drawings, riprap voids are to be filled with native soil. The riprap shall be pre-mixed with the native soil. The soil used for mixing shall be native topsoil. The soil riprap shall be installed in a manner that results in a dense, interlocked layer of riprap with riprap voids filled completely with soil. Segregation of materials shall be avoided and in no case shall be combined material consist primarily of soil; the density and interlocking nature of riprap in the mixed material shall essentially be the same as if the riprap was placed without soil. Mix proportions and riprap gradations to be provided by the Design Engineer.

3.) Where specified typically as “buried soil riprap”, a surface layer of topsoil shall be placed over the soil riprap according to the thickness specified on the contract drawings. The topsoil surface layer shall be compacted to approximately 85% of maximum density and within two percentage points of optimum moisture in accordance with ASTM D698. Topsoil shall be added to any areas that settle.

4.) All soil riprap that is buried with topsoil shall be reviewed and approved by the Design Engineer prior to any topsoil placement.

Riprap and Soil Riprap Placement and Gradation (Part 2 of 3)

Gradation for Granular Bedding		
US Standard Sieve Size	Percent Passing by Weight	
	Type I CDOT Section 703.01	Type II CDOT Section 703.09 Class A
3"	-	90-100
1 1/2"	-	-
3/4"	-	20-90
3/8"	100	-
#4	95-100	0-20
#16	45-80	-
#50	10-30	-
#100	2-10	-
#200	0-2	0-3

Riprap and Soil Riprap Placement and Gradation (Part 3 of 3)

Thickness Requirements for Granular Bedding			
Riprap Designation	Minimum Bedding Thickness (inches)		
	Fine-Graded Soils [1]		Coarse-Graded Soils [2]
	Type 1 (Lower Layer)	Type II (Upper Layer)	Type II
Type M	4	4	6
Type H	4	6	8
Type VH	4	6	8

Notes:

1.) May substitute one twelve inch (12”) layer of Type II bedding. The substitution of one layer of Type II bedding shall not be permitted at drop structures. The use of a combination of filter fabric and Type II bedding at drop structures is acceptable.

2.) 50% or more by weight retained on the #40 sieve.