TOWN OF RIDGWAY

STORMWATER STANDARDS

September 2020

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1. <u>INTRODUCTION</u>

These Stormwater Management Minimum Design Standards (Standards) are established by the Town of Ridgway and are intended to apply to all Development in the Town. Note that within the rights-of-way of State Highways 62 and 550, which fall in part under the jurisdiction of the Colorado Department of Transportation (CDOT), the more stringent requirements of the Town or CDOT will apply.

The purpose of these Standards is to provide a reasonable degree of assurance that the Development of public and private improvements will safeguard and protect the health, safety, welfare and property of the Town and citizens; and to assure a degree of uniformity in performance of public and private improvements thereby securing for Town residents benefits of Development while protecting against deterioration of the quality of the natural and manmade environment. These Standards provide the minimum acceptable standards for safe, consistent, effective, and economical infrastructure. Actual site design may require additional detail or more conservative design parameters to address site-specific issues. All proposed Development as defined below and including any improvements that alter the flow of storm water shall submit to the Town a drainage report that contains all design calculations, imperviousness's spreadsheets, nomographs, and other documentation necessary for the design and review of the proposed improvements in accordance with these standards. This design report and the design of all required improvements shall be signed and stamped by a Colorado Registered Professional Engineer.

1.1. Standards Overview

The following is an overview of general requirements pertaining to stormwater. Detailed standards can be found is sections 2 through 9 below.

- Development shall not exceed Historical peak flow from the parcel unless a stormwater system is in place which anticipated such improvements when the parcel was created.
- If a parcel cannot maintain its Historical flow rate(s) after Development, all downstream stormwater systems must be sized to accompany the flow rate(s) identified in the Stormwater Master Plan.
- Building entries shall be 12” above adjacent drainage features with positive drainage away from the foundation.
- A parcel may be required to treat for Water Quality Capture Volume even if Development does not affect peak flows. Refer to Section 6.5 Water Quality Capture Volume.
- Any Development that causes changes in runoff patterns must demonstrate how that water will be rerouted to a location acceptable to the Town.

1.2. Definitions and Abbreviations

1.2.1. Definitions

Words defined below are capitalized throughout the document for reference. Wherever the following words, phrases or abbreviations appear in the specifications, they shall have the following meanings when in reference to stormwater:

<u>CDOT STANDARDS</u> shall refer to Colorado Department of Transportation Standard Specifications for Road and Bridge Construction or the Colorado Department of Transportation M&S Standards Plans List

DEVELOPMENT shall mean any increase of imperviousness greater than 0.05 acre, or an improvement which results in a parcel's imperviousness percentage greater than the land use default impervious values (Table 3 below), or the creation of a PUD, or any parcel within the Uncompahgre River Overlay District. Development does not include improvements to existing Town rights of way, except improvements in the right of way that are for private benefit. "Development, "project", and "improvement" may be used interchangeably in these Standards.

HISTORIC shall refer- to the condition as it relates to runoff, drainage, flows or any other reference condition present at the time of adoption of these Standards with the impervious area being what is legally on record with the Ouray County Assessor or the impervious area that can be clearly delineated with 2019 or earlier National Agriculture Imagery Program (NAIP) Imagery and is still present on the parcel at the time of Development.

MAJOR STORM shall refer to a storm with a recurrence interval of 100 years.

MINOR STORM shall refer to a storm with a recurrence interval of 25 years.

RIPRAP refers to a protective blanket of large loose stones which are usually placed by machine to achieve a desired configuration.

SOIL RIPRAP is a mix of riprap and native soil. Soil riprap consists of 35% by volume of native soil taken from the channel excavation and 65% by volume of riprap of the specified gradation.

STANDARDS shall mean the Stormwater Management Minimum Design Standards

TOWN STANDARDS shall refer to the Town of Ridgway Standard Specification and Typical Drawings for Infrastructure Construction

1.2.2. Abbreviations

Wherever any of the following abbreviations appear, they shall have the following meaning:

CMP	Corrugated metal pipe
FAA	Federal Aviation Authority
FHWA	Federal Highway Administration
Fr	Froude number
HDPE	High density polyethylene
HGL	Hydraulic grade line
MHFD	Mile High Flood District, formerly known as the Urban Drainage and Flood Control District (UDFCD).
NAIP	National Agriculture Imagery Program
NOAA	National Oceanic and Atmospheric Administration
PVC	Polyvinyl chloride

RCBC	Reinforced concrete box culvert
RCP	Reinforced concrete pipe
SWMM	Storm Water Management Model
UDFCD	See MHFD
USDCM	Urban Storm Drainage Criteria Manual
WQCV	Water Quality Capture Volume resulting from a 1.25year storm

2. RUNOFF

Runoff can be calculated using the Rational Method for watersheds up to a total size of 160 acres. Theoretically SWMM may be used to calculate runoff for basins of any size, however for simplicity, the Town has limited its use to basins over 160 acres unless approved by the Town. Runoff from a parcel after any Development or improvement shall not exceed Historical flows for any parcel unless:

1) The parcel is part of a larger Development plan where its impact has already been incorporated into an existing stormwater drainage plan or system.

2) The increase in impervious area is less than 0.05 acres, the improvement results in a parcel's imperviousness percentage below the land use default values (Table 3), and the increased flow will result in less than a 1% increase in peak flow on any portion of the downstream stormwater system from current conditions at the time of adoption of these Standards.

2.1. Rational Method

The Rational Method may be used for watersheds up to a total size of 160 acres. When using the Rational Method, individual subwatershed sizes are to be no greater than 20 acres, and each subwatershed should be reasonably homogeneous for existing and projected land use.

The Rational Method is based on the following formula:

$$Q = CIA \qquad \text{Equation 1}$$

Where:

Q = maximum rate of runoff (cfs)
C = runoff coefficient per Table 1
I = rainfall intensity (inches per hour)
A = contributing watershed area (acres)

The rainfall intensity discussed below is the peak rainfall rate for a given return period storm having a duration equal to the time of concentration, calculated using Equation 2.

The time of concentration, t_c, is the time it takes for runoff to flow from the most remote part of the watershed to the point of interest. This parameter is necessary to determine the maximum flow at a specific point within the watershed. The time of concentration for both non-urbanized and urbanized watersheds is calculated as follows:

$$t_c = t_i + t_t \qquad \text{Equation 2}$$

Where:

t_c = time of concentration (minutes)
t_i = initial flow time (minutes)
t_t = travel time in the ditch, channel, swale, gutter, storm drain, etc. (minutes)

For non-urban watersheds, those with up to 20% imperviousness, the minimum recommended total time of concentration is 10 minutes, and the initial flow time can be calculated as follows:

$$t_i = 0.395\,(1.1 - C_{25}) * \frac{L_i^{1/2}}{S^{1/3}} \qquad \text{Equation 3}$$

Where:

C_{25} = runoff coefficient for 25-year return period per Table 1
L_i = length of initial flow (feet, 300 max)
S = average slope along the initial flow path (percent)

Table 1: Runoff Coefficients

Percent Impervious	Runoff Coefficients, C_x						
	1.25-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
0%	0.003	0.04	0.15	0.25	0.37	0.44	0.50
5%	0.03	0.08	0.18	0.28	0.39	0.46	0.52
10%	0.06	0.11	0.21	0.30	0.41	0.47	0.53
15%	0.10	0.14	0.24	0.32	0.43	0.49	0.54
20%	0.13	0.17	0.26	0.34	0.44	0.50	0.55
25%	0.16	0.20	0.28	0.36	0.46	0.51	0.56
30%	0.18	0.22	0.30	0.38	0.47	0.52	0.57
35%	0.22	0.25	0.33	0.40	0.48	0.53	0.57
40%	0.25	0.28	0.35	0.42	0.50	0.54	0.58
45%	0.28	0.31	0.37	0.44	0.51	0.55	0.59
50%	0.31	0.34	0.40	0.46	0.53	0.57	0.60
55%	0.34	0.37	0.43	0.48	0.55	0.58	0.62
60%	0.36	0.41	0.46	0.51	0.57	0.60	0.63
65%	0.42	0.45	0.49	0.54	0.59	0.62	0.65
70%	0.47	0.49	0.53	0.57	0.62	0.65	0.68
75%	0.52	0.54	0.58	0.62	0.66	0.68	0.71
80%	0.58	0.60	0.63	0.66	0.70	0.72	0.74
85%	0.64	0.66	0.68	0.71	0.75	0.77	0.79
90%	0.71	0.73	0.75	0.77	0.80	0.82	0.83
95%	0.79	0.80	0.82	0.84	0.87	0.88	0.89
100%	0.88	0.89	0.90	0.92	0.94	0.95	0.96

The initial flow length for both non-urbanized and urbanized watersheds is the length over which flow is expected to be sheet flow, prior to becoming concentrated in a swale. If the distance between the most remote part of the basin and the point of interest is longer than 300 feet, travel time must be added to initial flow time to calculate total time of concentration. Time to concentration shall be calculated using Manning's equation (discussed later in these Standards) or can be approximated by using Equation 4 to determine the time to concentration. The minimum conveyance factor that shall be used for a developed site shall be 7.

$$t = \frac{d}{K\, S_w^{1/2}} \qquad \text{Equation 4}$$

Where:

t = approximated time to concentration (seconds)
d = distance between most remote part of basin and point of interest (ft)
S_w = watercourse slope (ft/ft)
K = conveyance factor per Table 2

Table 2: Travel Time Conveyance Factors

Land Surface	Conveyance Factor, K
Heavy meadow	2.5
Tillage/Field	5
Short pasture and lawns	7
Nearly bare ground	10
Grassed waterways	15
Paved areas, shallow swales, and storm sewers	20

In urbanized watersheds, those with greater than 20% imperviousness, the total time of concentration shall not exceed that calculated using Equation 5 below and shall be no less than 5 minutes.

$$t_c = (26 - 17i) + \frac{L_t}{60\,(14i + 9)\sqrt{S_t}} \qquad \text{Equation 5}$$

Where:

t_c = minimum time of concentration for the first design point (minutes)
L_t = length of channelized flow path (feet)
i = imperviousness as a decimal
S_t = slope of the channelized flow path (feet/foot)

The runoff coefficients in Table 1 correspond with the composite imperviousness of the drainage basin for which peak flow is being calculated. General imperviousness values for several types of land use are in Table 3. If more specific data on imperviousness is known for a drainage basin, that data is to be used to develop a composite imperviousness to be used with Table 1 to obtain applicable C values.

Table 3: Default Imperviousness Values

Land Use or Cover	Percent Imperviousness
Undeveloped/Vacant Land	2
Parks, Open Space and Natural Areas	10
Rural Neighborhoods	20
Institutional	50
Residential Single-Family Neighborhoods	50
Town Core Residential Neighborhoods	50
Commercial/Industrial/Employment Areas	60
Mixed Use Residential AKA Mixed Neighborhoods	60
Mixed Use Business	70
Town Core	90
Class 6	80

2.2. Rainfall Intensity

When using the Rational Method to determine runoff, rainfall intensity shall be determined using Equation 6 below. Equation 6 is used to calculate rainfall intensities for a given time of concentration and return period. The equation was developed based on data from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 8 for the Ridgway area.

$$I = P_1 \times \frac{36.65}{(T_d+7.05)^{0.855}} \qquad \text{Equation 6}$$

Where:

I = rainfall intensity (inches per hour)
P_1 = 1-hour rainfall depth (inches)
T_d = storm duration (minutes)

Table 4 includes 1-hour rainfall depths associated with various return periods to use with Equation 6 in the first two columns. Within the other columns of Table 4, various rainfall intensities are displayed which were calculated from the corresponding return period and storm duration utilizing Equation 6. The design of stormwater infrastructure within the Town will have different design standards for the Minor and Major Storm events. The Minor Storm shall be defined as a storm with a 25-year return period, and the Major Storm shall be defined as a storm with a 100-year return period. In some locations, designing infrastructure using the Major or Minor storm may be impractical. If the designer feels that there are site specific challenges that prohibit using this Standard, the designer shall prepare and submit a report to the Town explaining the challenges and proposing an alternative Major or Minor Storm event for the location-specific challenges with the rationale for using that storm. The Town will review the request for deviation and work with the designer to determine the storm to be accommodated while also assessing downstream impacts. Such deviations Minor Storm will be determined on a site-specific basis rather than a project-wide basis.

Table 4: One-Hour Rainfall Depths and Intensity-Duration-Frequency Values

Return Period	P_1	Storm Duration and Resulting Rainfall Intensity (in/hr)				
		5-min	10-min	15-min	30-min	60-min
1.25-year	0.24	1.05	0.78	0.63	0.40	0.24
2-year	0.48	2.09	1.55	1.25	0.80	0.48
5-year	0.61	2.67	1.98	1.59	1.02	0.61
10-year	0.74	3.23	2.40	1.93	1.24	0.75
25-year	0.95	4.16	3.09	2.48	1.59	0.96
50-year	1.14	4.97	3.70	2.97	1.90	1.15
100-year	1.36	5.93	4.41	3.54	2.27	1.37
500-year	1.94	8.47	6.29	5.05	3.24	1.95

When using SWMM, rainfall should be determined using the latest NOAA Atlas 14, Volume 8 rainfall values. The SCS Type II 6-hr storm distribution should be used to generate the hyetograph in the model. This information can be found in an appendix of Ridgway's Stormwater Master Plan or online.

2.3. Storm Water Management Model

It is not anticipated that EPA's Storm Water Management Model (SWMM) will be used frequently in the Town to calculate runoff as it is typically used for large basins. If SWMM is used, recommendations in the SWMM user's manual shall be followed and design methodology shall be presented to the Town in the drainage design report detailing the parameters used including each basin's; imperviousness, area, characteristic width, slope, hydrologic soil group, precipitation losses using Horton's infiltration method, and any other relevant data. Within the model, dynamic wave routing shall be used to be consistent with the Town's Stormwater Master Plan unless the Town approves the use of kinematic wave routing. If kinematic wave routing is proposed, an explanation must be provided to the Town as to why this method is preferred.

3. ROADWAY DRAINAGE

Streets are a major component of the drainage system, but their use for stormwater drainage must be limited to prevent interference with traffic. A street's flow capacity is based upon its cross-sectional geometry, longitudinal slope, and the maximum allowed depth or spread of runoff. During a Major Storm event, streets may become emergency runoff channels, routing floodwaters away from structures. During these events many streets will be inundated to the point they are impassable to most vehicles.

3.1. Allowable Encroachment

Table 5 below lists the allowable encroachment criteria for different types of roadways within the Town of Ridgway. Roadway designation can be found in the Town's Master Plan or contact the Town for roadway designation.

Table 5: Allowable Roadway Encroachment

Criteria	Collector & Arterial Criteria	
	Minor Storm	Major Storm
Depth at gutter flow line[1]	6”	12”
Depth at outside edge of pavement[2]	6”	12”
Center clear lane for emergencies	N/A	12 feet
Street flow velocity	N/A	8 fps
Max flow spread	N/A	Stay within public ROW
Criteria	**All Other Roadways Criteria**	
	Minor Storm	**Major Storm**
Depth at gutter flow line[1]	N/A	12”
Depth at outside edge of pavement[2]	N/A	12”
Depth at roadway crown	0”	6”
Street flow velocity	N/A	8 fps
Max flow spread	N/A	Stay within public ROW

Where flow exceeds what is allowable within a street, a roadside ditch or enclosed drainage system must be used to capture and convey the excess flow. Areas of existing Development or redevelopment may have roadside ditches or enclosed drainage systems, while new areas of Development are expected to have enclosed drainage systems. Roadside ditches are discussed in detail later in these Standards. When designing or redesigning a roadway with buildings already present Table 16 will also need to be consulted and met regarding to building entry elevations. If the designer feels that there are site specific challenges that prohibit being able to meet existing entry height(s) the designer shall prepare and submit a report to the Town explaining the challenges and proposing an alternate.

3.2. Street Flow Calculations

Calculations for flow capacity and velocity in a 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 drainage 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. For new street designs, a combination of the storm drainage system and curb and gutter can be utilized to convey the Minor event. All street capacity calculations should be completed on a half-street basis and the same vertical-wall assumption applies to the street centerline as to the right-of-way. The Mile High Flood District (MHFD), maintains an excel spreadsheet that will calculate street hydraulic capacity given detailed user input. The spreadsheet is titled UD-Inlet and is available on the MHFD website under Technical Downloads. The website should be checked to ensure the most recent version of UD-Inlet is being used as the MHFD often updates its technical materials as new data becomes available. Detailed street flow calculations that can be completed by hand can be found in the Federal Highway Administration's (FHWA) HEC-22 Urban Drainage Design Manual.

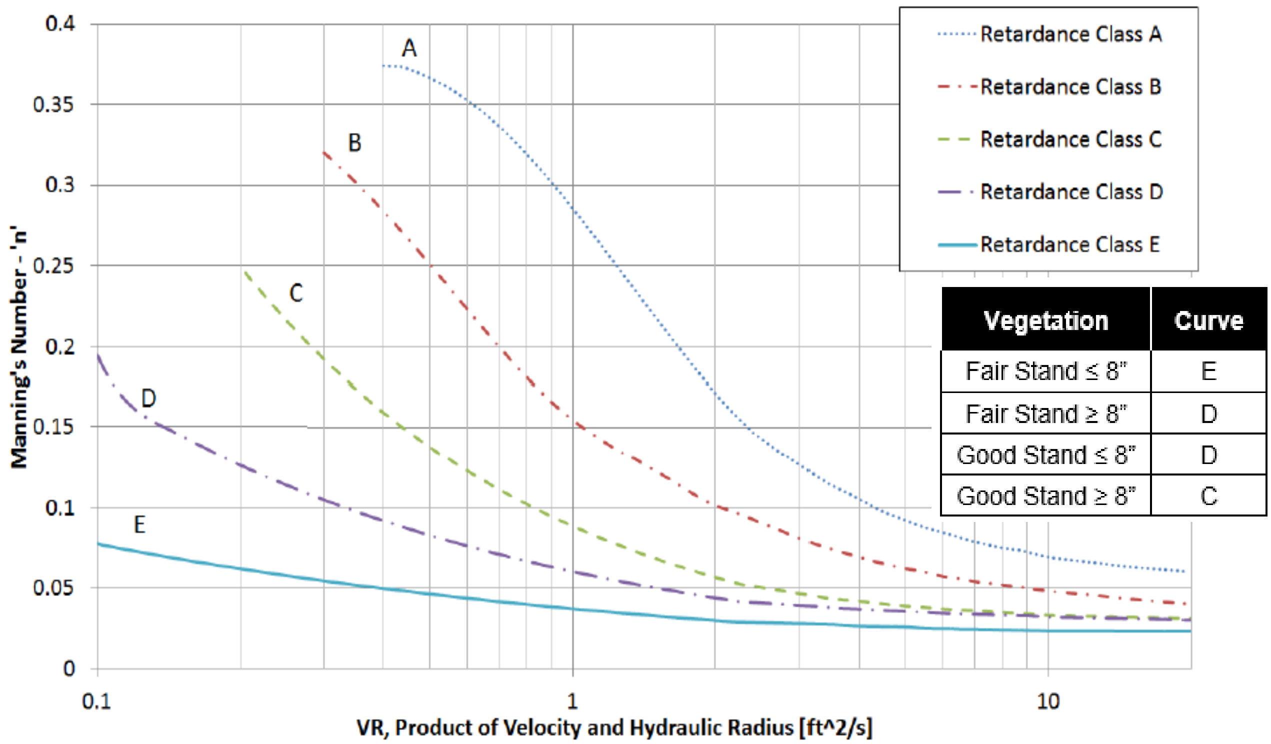
3.3. Roadside Ditches

Roadside ditches are open channels but are discussed here because they are specifically part of the roadway drainage system. They will ideally be grass-lined and shall be designed to prevent erosion of the ditch lining. If an enclosed drainage system is present or anticipated upstream or downstream, a roadside ditch will not be allowed unless approved by the Town. Maximum longitudinal slope shall result in a Froude number no higher than 0.80 and a maximum velocity of 7 feet per second during the Major Storm as calculated using Manning's equation. For grass-lined channels, the Manning's n value, velocity and capacity calculations shall use the appropriate retardance curve in Figure 1. Retardance curves A and B are not used within the Town. The Froude number for roadside ditches shall be calculated as follows:

$$\mathrm{Fr} = \frac{\mathrm{V}}{\left(\frac{\mathrm{gA}}{\mathrm{T}}\right)} 0.5 = \frac{\mathrm{V}}{(\mathrm{gD_h})} 0.5 \qquad \text{Equation 7}$$

Where: Fr = Froude number (dimensionless)
V = average ditch velocity (fps)
g = gravitational acceleration (32.2 ft/s^2)
A = cross sectional flow area (square feet)
T = top width of flow area (feet)
D_h = hydraulic depth = A/T (feet)

Using Figure 1 requires a trial-and-error approach, first assuming an "n" value and then calculating the various parameters repeatedly until the intersection of the product of velocity and hydraulic radius (VR) and the Froude number falls on the specified retardance curve dictated by vegetation.



Vegetation	Curve
Fair Stand ≤ 8"	E
Fair Stand ≥ 8"	D
Good Stand ≤ 8"	D
Good Stand ≥ 8"	C

Figure 1: Retardance Curves

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

The following criteria shall be used to design roadside ditches:

1. For all types of roadways and Development, encroachment and flow depth criteria shall be the same for roadside ditches as it is for enclosed drainage systems with curb and gutter for the Minor and Major Storms (seeTable 5).
2. The Major Storm water surface elevation shall meet the requirements of Table 16for the lowest point of entry of structures.
3. Side slopes of roadside ditches shall be no steeper than 3H:1V.
4. No roadside ditch shall have a flow depth greater than 3 feet.
5. A minimum velocity of 2.0 fps during a Minor storm is required to discourage sediment build-up unless approved by Town as a detention swale.

3.4. Ditch Checks

The natural topography of the area may result in relatively steep roadside ditch slopes. Ditch checks are required where the Froude number exceeds 0.80 or the velocity exceeds 8 feet per second. The upstream side of each ditch check shall be buried, and the downstream side can have no more than a 2-foot drop at a slope of 2H:1V. Each drop must have a soil riprap or ordinary riprap plus bedding apron extending at least 5 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 transitions from 0 at this point to 2 times the height of the next ditch check at its crest (assuming the ditch side slopes are 3H:1V and the face of the check is 2H:1V). In this fashion, the side slopes of the roadside ditch will remain constant through each of the ditch checks. Figure 2 is of a typical ditch check.

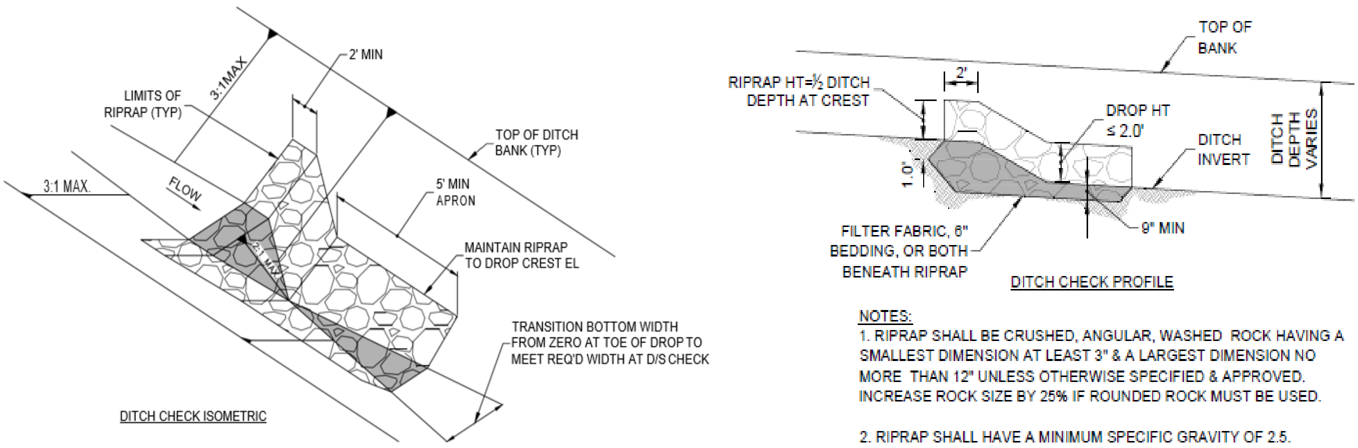


Figure 2: Ditch Check Schematic

3.5. Culverts

Design methodology in this section is generally based on the Federal Highway Administration (FHWA) Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5 (HDS-5), available online. Culvert flow is either under inlet control or outlet control. Identifying if the inlet or the outlet controls the culvert will depend on the flow rate being considered, the culvert and drainageway characteristics. Both inlet and outlet control conditions must be evaluated, and the condition which produces the greater energy loss for the design condition dictates which situation will control the design.

Under inlet control, the flow through the culvert is controlled by the headwater on the culvert and the inlet geometry. Inlet control can be unsubmerged or submerged. In an unsubmerged condition, the headwater is not enough to submerge the top of the culvert and the culvert slope is supercritical. In this situation, the culvert inlet acts like a weir. In a submerged condition, headwater submerges the top of the culvert, but the pipe does not flow full. In this situation, the culvert inlet acts like an orifice.

Under outlet control, the flow through the culvert is controlled primarily by culvert slope, roughness, and tailwater elevation. This occurs when the culvert is not capable of conveying as much flow as the inlet opening will accept. The control section may be within the barrel, at the barrel exit or even further downstream. Outlet control will govern if the tailwater is high enough, the culvert slope is relatively flat, or the culvert relatively long. Outlet control will exist primarily under two conditions. The first and less common is when the headwater does not submerge the culvert inlet and the culvert slope is subcritical. The more common condition is when the culvert is flowing full. Culvert hydraulic calculations shall be performed using rating nomographs and/or culvert hydraulic analysis programs such as the FHWA's HY8 Culvert Analysis.

3.5.1. Culvert Design Procedure

Design shall consider design flow, culvert size and material, upstream channel and entrance configuration, downstream channel and outlet configuration, and erosion protection. Material and shape shall be based on required hydraulic capacity and the ability to meet the HS-20 loading.

Inlet control calculations shall be completed using inlet control nomographs from the FHWA's HDS-5 for typical configurations, available online. Various types of culvert entrance configurations included in HDS-5 may be used, except for projecting entrances, which must be approved by the Town. Note that if using a flared end section on a round culvert that chart 55B should be used.

Outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert in outlet control. Critical depth charts and outlet control nomographs are also used in the design process. The procedure and variables in HDS-5 for culvert design should be followed. Several commonly used inlet control and outlet control nomographs are included in the appendix. Additional nomographs and critical depth charts are available in HDS-5.

3.5.2. Culvert Design Criteria

The information below shall be used for culvert design and provided to the Town for review. Culverts crossing State Highway 62 (Sherman Street) and U.S. 550 are also subject to the requirements of CDOT.

1. Design discharge shall be the Major Storm event when crossing under an arterial or collector roadway and shall be the Minor event for other roadways unless the roadway is the only road providing access to an area, in which case the Major Storm shall be used. Check the Town's Master Plan or contact the Town for roadway classification.
2. Upstream headwater on culverts under all types of roadways shall meet the encroachment requirements in Table 5. Headwater divided by culvert diameter or height (HW/D) shall not exceed 1.5 during the design storm. No increase in backwater from a culvert will be permitted to extend onto an adjacent property.
3. Tailwater shall be calculated as the depth of water downstream of the culvert measured from the outlet invert. Backwater calculations from a downstream control point are required unless downstream channel normal depth approximations using Manning's Equation are considered by the designer to be adequate and there is no downstream structure, roadway, obstruction or improvements causing backwater. The designer shall receive Town approval prior to using normal depth approximations.
4. Velocity at the culvert outlet cannot exceed the maximum permissible velocity for the channel lining in
5. Table 8. Velocities exceeding these values require outlet protection discussed below. Minimum culvert velocity for the Minor Storm shall be 2 feet per second. Culverts shall have a maximum design-flow velocity of 8 feet per second during the Major Storm unless approved otherwise by the Town. Design shouldn't subject the culvert or surrounding channel lengths to velocities greater than the culvert or surrounding channel lining can handle.
6. Minimum cover over a culvert shall be 1 foot or 8 inches to the bottom of the pavement. Culverts passing under roadways shall maintain their shape and function under an HS-20 loading. Maximum or minimum allowable cover will depend on pipe size and material and should follow the manufacturer's recommendations.
7. The minimum size for all public culverts and private culverts placed in a public drainage way such as a roadside ditch shall be an 18-inch diameter round pipe or any shape with an equivalent area. For public drains through sidewalks a 12-inch-wide by 8-inch-deep chase

drain shall be used. Private drainage culverts on private property may be as small as 8-inch diameter if supported by a drainage letter or study.

8. Culverts shall comply with the Town Standards and CDOT Standards. Culverts shall be galvanized corrugated metal pipe (CMP) or dual walled high-density polyethylene (HDPE) with a smooth interior and a corrugated exterior. Reinforced concrete pipe (RCP), or reinforced concrete box culverts (RCBC) may be used with Town approval. Culverts may be circular, elliptical, arch, or box-shaped.
9. For public culverts and storm drain pipes placed in areas having corrosive soils, high density polyethylene pipe shall be used.
10. Culverts shall be placed to completely drain all runoff where a swale or channel intersects a road or sidewalk. All areas where water may be impounded shall be considered for culvert locations. Culvert placement shall not include any abrupt changes in flow direction at either end. If it is not possible for a culvert to have the same alignment as the channel, headwalls, wingwalls, and aprons shall be used as protection against scour and to provide a more efficient inlet.
11. Culvert inlets and outlets shall have a flared end section unless headwalls or wingwalls are included in the design.

3.5.3. Outlet Velocity Calculation

The outlet velocity is calculated as follows:

1. If design 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 design headwater is based on outlet control, determine the area of flow at the outlet (and corresponding velocity) 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.

3.5.4. Outlet Protection

Below presents maximum permissible mean channel velocities for various types of channel linings. When outlet velocities exceed allowable channel velocity, a riprap apron or riprap basin (also called a low tailwater basin) is required. The design of riprap aprons and basins shall be completed in accordance with the FHWA's HEC-14 or the USDCM published by the MHFD.

The gradation and materials for riprap shall be as specified in the CDOT Standards. Also note that the riprap sizing calculations are for angular rocks with fractured faces, nearly rectangular in shape with a breadth or thickness at least 1/3 its length. Where these riprap materials are not available, rounded river rock may be used if channel side slopes are flattened to 4H:1V and the required gradation is increased by at least 25%.

3.5.5. Computer Applications

The FHWA's HY8 Culvert Analysis may be used in lieu of nomographs. Other programs must receive Town approval.

4. STORM DRAIN SYSTEMS

Storm drains are used to convey runoff in locations where street capacity is exceeded. Typically, storm drains are sized to convey peak runoff from the Minor Storm in excess of street flow capacity as designated in Table 5. The first inlet will either be located at or upstream of 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 more than the Minor Storm event, up to the entire Major Storm event flow. Four examples of this situation are:

1. Locations where street flow is not in the desired direction and there is no other feasible drainage solution.

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.

3. Locations where there is no viable overflow option for the Major Storm event without adversely impacting private property.

4. When a storm drain system sized for the Minor event results in flooding during the Major event that exceeds the allowable encroachment criteria in Table 5, in which case the storm drain system must be upsized so that the criteria for all storm events are met.

4.1. General Storm Drain System Design Procedure

The general design process for a storm drainage system is below.

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. The runoff from multiple streets will eventually converge at a point, so all streets that are tributary to that point must be evaluated before moving on downstream. An inlet should be located wherever the Minor or Major Storm peak street flow exceeds the allowable capacity for that street and at all sump locations.

3. Initial storm drain sizing begins 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 significantly.

Regardless of if computer software is used to model storm drain systems, hand calculations should be used to spot-check the computer models to ensure the software is functioning properly.

4.2. Allowable Capacity and Velocity

A storm drain shall be designed to convey all the design storm runoff from areas tributary to it as identified in Ridgway's Stormwater Master Plan. The design of surcharged storm pipes is not allowed for the Minor Storm, and capacity and velocity should generally be calculated using the Manning's equation (Equation 9). A minimum design flow velocity of 2 feet per second is required for a Minor storm. The maximum design flow velocity is 10 feet per second during any storm. The Town will review requests for a storm drain design where the design storm velocity exceeds 10 fps. Maximum outfall velocities are more restrictive as discussed in this section.

Table 6 provides Manning's n value. The designer shall consider aging of the pipe and possible abrasions, corrosion, dents, deflection, joint conditions, and potential sediment buildup when selecting roughness values.

Table 6: Manning's Roughness Coefficients for Storm Drains

Type of Conduit	Interior Wall Description	Manning's n
Concrete Pipes and Boxes	Smooth	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

Reference: Adapted from HDS-4 and HEC-22

4.3. Storm System Requirements

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. Consult the CDOT Standards, the Concrete Pipe Design Manual, the Handbook of Steel Drainage and Highway Construction Products, and manufacturer specifications to determine cover requirements. Storm drains under railroads and roadways must comply with any cover requirements specified for culverts, as well as with any criteria the railroad and roadway owners may have. When designing and building a storm system, the Town Standards Section 02723 will act as the governing requirements for any details not called out.

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 foot of cover from finished grade and at least 8 inches below the bottom of the pavement to top of outside of pipe regardless of location unless special bedding is provided per the manufacturer's recommendations. In a manhole, the lowest inlet pipe invert elevation must be at least 0.2 feet higher than the outlet pipe invert elevation. Where the downstream pipe is larger than the largest upstream pipe, pipe crowns should be matched. The storm sewer system alignment shall be designed to minimize the length of pipe, stay a consistent distance from the right of way centerline, and provide a reasonably uniform pipe slope throughout. Local utility companies shall be consulted to determine the location of their existing lines and their required minimum clearances. Pipe encasement may be required in some locations where minimum utility clearances are not met. The Town and affected utility shall determine when encasement is required and approve the design of any required encasement. Designs that request relocation of utilities shall be avoided whenever possible.

Manholes or other junction structures are required at all bends, vertical drops, and changes in main line pipe size or slope. 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. The bench shall be flared up to the spring line along the length of the bench through the manhole for 12-inch pipes to facilitate camera work and cleaning. Maximum allowable manhole spacing is 400 feet.

All storm drain pipes shall have a minimum diameter of 12 inches. For non-circular pipes, these minimum diameters represent equivalent diameters based on cross-sectional areas. All storm drain pipes shall comply with the Town's Standards as well as the most recent edition of the CDOT Standards. Public storm drain pipes shall be dual walled high-density polyethylene (HDPE) with a smooth interior and a corrugated exterior with water tight bell and spigot joints or SDR 35 polyvinyl chloride (PVC). In limited cases where design constraints necessitate the use of reinforced concrete, Town approval is required.

Structure foundation drains up to 4-inch diameter may 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 wye-shaped fitting shall be installed on the main and the 4-inch leg of the wye shall be used to extend the foundation drain connection to the property line, where it will terminate in a clean-out for the foundation drain. The connection shall be done in a manner that the connecting pipe does not restrict the flow capacity of the mainline storm sewer pipe nor allows root entry. No strap-on taps are permitted. A restrained connection is required for pumped flow.

The required diameter of the manhole barrel is dependent upon the size and configurations of the pipes connecting to it. For all manholes, at least 12 inches of clearance must be present from the openings for the pipes in where they intersect the inside of the manhole to preserve the structural integrity of the manhole. Approved manhole designs are in the Town Standards for 4' and 5' manholes. See CDOT Standards for 6' diameter and larger manholes.

4.4. Hydraulic Calculations

The Mile High Flood District, maintains the program UD-Sewer which calculates the hydraulic grade line (HGL) within a storm sewer system. UD-Sewer is available on the MHFD website under Technical Downloads. The website should be checked to ensure the most recent version of UD-Sewer is being used as the MHFD often updates its technical materials as new data becomes available. The Town shall approve the use of any software other than UD-Sewer.

Detailed street flow calculations that can be completed by hand can be found in the FHWA's HEC-22 Urban Drainage Design Manual. Pipe friction and manhole losses are significant source of energy dissipation in storm drain systems. If calculating an HGL by hand, pipe friction losses need to be considered as well as manhole losses for changes in pipe diameter at a manhole, differences in flow depth upstream and downstream of a manhole, more than two pipes using a manhole, plunging flow, and manhole benching. Pipe sizes shall be initially selected based on capacity calculated using Manning's equation for open channel flow (Equation 9) starting at the uppermost reach of the storm drain system. Alternately, Equation 8 may be used to directly solve for the minimum required pipe diameter for circular pipes, rounding up to the nearest standard pipe size.

$$D_i = \left[\frac{2.16nQ_p}{S_o^{1/2}}\right]^{3/8} \quad \text{Equation 8}$$

Where:

D_i = initial design minimum pipe diameter (feet)
Q_p = initial design peak flow rate (cfs)
n = Manning roughness coefficient (see Table 7)
S_o = initial design pipe slope (feet/foot)

The HGL shall be calculated for each storm drain system 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 in accordance with procedures in HEC-22 or using UD-Sewer. Compliance with minimum and maximum flow velocities is based on peak design flow 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 for the Minor Storm.

4.5. Storm Inlet Selection, Sizing, and Location

When flow in a street impacted by new Development exceeds allowable limits of encroachment during either the Minor or Major event, an enclosed drainage system with inlets must be added. The standard street inlets for use in Ridgway are the CDOT Type 13 and Type R inlets with a bicycle safe grate. A Denver Type 13/16 Combination inlet may also be used. Note that if using a Type R inlet, the standard 2-inch local depression of the throat section should be reduced to 1 inch. Each inlet shall have a 1-foot sump (inlet sump) below the lowest pipe invert elevation to collect sediment and debris. Area inlets approved for public use where no potential traffic loads exist include CDOT Type C and D inlets with a close mesh or bicycle safe grates and the CDOT Type 13 area inlet with a No. 13 grate.

4.5.1. Guidelines for Inlet Location and Spacing

Inlets should be placed where allowable encroachment limits are exceeded. At no time shall inlets be located within a curb ramp, but an inlet shall be located within approximately 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 meet the requirements of Table 5. Additional street inlet locations shall be determined using the following iterative process:

1. The location of sump inlets is fixed at the sag of the roadway vertical alignment. The inlet should be sized to maintain water depth and spread within the allowable limits in Table 5. If the Town determines a sump inlet becomes excessively large, additional inlets upgradient from the sump shall be considered.

2. Consider the change in tributary area to the inlet associated with any upstream or downstream location adjustment and recalculate flow depth and spread.

3. A typical design interception efficiency of an on-grade inlet is 70 to 80 percent. On-grade inlets designed to capture 100 percent of runoff are less effective hydraulically and economically.

4. 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.

5. 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. Sump inlets should be located prior to the placement of any on-grade inlets during the design process.

Sumps in paved areas or in unpaved open spaces shall not pond more than 6 inches during the Minor Storm. Building entrances shall be no less than 12 inches above the Major Storm ponding depth.

4.5.2. Inlets on Continuous Grade

Inlets on a continuous grade may allow some flow to bypass to the next downstream inlet and this bypass flow must be accounted for. Inlet capacity calculations shall include standard clogging factors. UD-Inlet, the spreadsheet developed by the MHFD and mentioned earlier in this section, will calculate hydraulic capacity of an inlet on grade given detailed geometric input. This spreadsheet 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 are typically designed to capture between 70 and 80 percent of the design discharge.

4.5.3. Inlets in Sump Conditions

Street inlets in sump conditions, such as the low point in a vertical sag, must have the capacity to capture all the runoff draining to them without exceeding maximum allowable flow depth and spread. To ensure maximum allowable ponding depth is not exceeded, and to protect against failure, flanker inlets shall be considered. Flanker inlets are located upgradient 10 to 50 feet from the primary sump inlet. Two flanker inlets shall have a combined design capacity equal to or greater than that of the primary sump inlet or inlets.

UD-Inlet will calculate hydraulic capacity of a street or area inlet in a sump condition and shall be used to calculate the hydraulic capacity of an inlet in a sump condition. The default values for clogging factors and for orifice and weir coefficients shall be used unless site conditions specifically dictate the use of different values.

4.5.4. Inlet Grate Selection

Bicycle-safe grates must be used in all areas that may receive pedestrian or bicycle traffic unless specifically approved by the Town. The types of grates permitted for use with the Type 13/16 Combination inlet are valley grates in single, double, and triple-inlet configurations. Vane grates are not allowed.

When a combination inlet is to be installed with a valley grate, this is sometimes designated as a Type 13/16 Combination inlet depending on the manufacturer. The designer should consult with the manufacturer to ensure compliance with these Standards and not unconditionally specify a Type 13/16 Combination inlet. There are several valley grates that may be used with a Type 13/16 Combination inlet. Neenah Foundry and East Jordan Ironworks are nationwide manufacturers. 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. A CDOT Type 13 area inlet may also be used.

A “drains to river” stamp shall be included in all grate inlets. Inlet grates shall be submitted to the Town for approval.

5. CHANNEL AND RESERVOIR ROUTING

When a large or non-homogeneous watershed is being investigated, it will be required to be divided into smaller and more homogeneous subwatersheds. The storm hydrograph for each subwatershed can then be routed through the channel and combined with individual subwatershed hydrographs to develop a storm hydrograph for the entire watershed. A SWMM model must be used to route hydrographs if the complexity of the relationship between the subbasins is such that simply adding the peak flows together is not an appropriate solution. Additionally, detention storage volume may be sized using routing techniques rather than the direct Federal Aviation Authority (FAA) Method calculation provided later in these Standards.

5.1. Improved Open Channel Design Criteria

All open channel improvements for grassed channels and channels composed of native materials shall be designed in accordance with the latest version of the Urban Storm Drainage Criteria Manual (USDCM) by the Mile High Flood District (MHFD). If localized energy dissipation is required along an open channel, such as at a drop, it will also be designed in accordance with the latest version of the USDCM. All open channels within the Town of Ridgway shall be designed to convey water in a subcritical flow condition (Fr<0.8) where achievable. All open channels shall be designed with public safety in mind and adequate maintenance access shall be provided.

5.2. Open Channel Flow

The computation of uniform flow and normal depth in any open channel shall be based upon the Manning's or Uniform Flow Equation:

$$Q = \frac{1.49}{n} AR^{2/3}\sqrt{S} \qquad \text{Equation 9}$$

Where:

Q = flow rate (cfs)
n = Manning roughness coefficient (see Table 7)
A = area (square feet)
P = wetted perimeter (feet)
R = hydraulic radius = A/P (feet)
S = slope of the energy grade line (feet/foot)

Table 7: Manning's Roughness Coefficients

Type of Channel and Description	Roughness Coefficient
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 plants in deep channels	.035
Earth bottom and rubble sides	.030

Stony bottom and weedy banks	.035
Cobble bottom and clean sides	.040
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	**Roughness Coefficient**
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	Figure 1
Riprap	Equation 14

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

5.3. Flow Depth and Froude Number

Backwater from culverts, storm drain inlets, or channel constrictions can cause channel flow depth to be greater than normal depth. In these cases of gradually varied flow, the water surface can be computed using HEC-RAS. Other computer software may be used for water surface calculation if approved by the Town.

Critical flow depth in a channel occurs when the Froude number (Fr) is equal to 1.0. Channels should 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 is defined as follows:

$$\mathrm{Fr} = \frac{\mathrm{v}}{\sqrt{\mathrm{gD_h}}} \qquad \text{Equation 10}$$

Where:

Fr = Froude number (dimensionless)
v = velocity (fps)
g = gravitational acceleration (32.2 ft/s^2)
D_h = hydraulic depth = A/T (feet)

Where:
A = channel flow area (square feet)
T = top width of flow area (feet)

5.4. Channel Velocity

Each channel lining is only stable up to a certain velocity. Channel design shall consider reducing the potential for erosion and may require a decrease in slope, change in channel bottom material, or the addition of revetment.

Table 8 gives 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. Channel velocities may be restricted to values below those listed in

Table 8 in other sections of this document.

Table 8: 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

5.5. Types of Channels

Native materials, grass, concrete, and riprap are generally the different types of channel linings found within the Town. Channels composed of native materials and channels that are grass-lined are preferred within the Town as concrete and riprap-lined channels have higher capital, maintenance costs and potential safety concerns. The latter channel types may be considered on a case-by-case basis based on site conditions and flow characteristics. Each channel lining should be evaluated for its longevity, integrity, maintenance requirements and costs, and general suitability for community needs, among other factors.

Selection of a channel lining that is most appropriate for the site should be based on a multi-disciplinary evaluation to include hydraulic, structural, environmental, sociological, maintenance, economic, and regulatory factors. New channels should closely mimic similarly sized natural channels in the area if possible. If a hard channel lining is proposed, the designer shall consult with the Town to arrive at an acceptable design using the criteria in this section. Channel improvements should maintain the existing flow rate and alignment. New Development must not increase the peak runoff a natural channel receives unless it was designed to accommodate the added flow.

5.5.1. Concrete Lined Channels

Rigid channel linings such as concrete are not recommended due to safety concerns, potential loss of long-term structural integrity, and aesthetics, but they may be required at some sites because of restrictive site characteristics. HEC-15 by the Federal Highway Administration (FHWA) offers extensive guidance on the design of concrete channels. If design flow is supercritical in a concrete-lined channel, imperfections at joints can cause their rapid deterioration or complete failure. High velocities at cracks or joints can cause uplift forces under the liner. Concrete linings must be designed by a structural engineer. Concrete linings shall be continuously reinforced longitudinally and laterally to resist hydrostatic uplift forces, including from groundwater and potential local inflow behind the lining. All joints shall be designed to prevent differential movement. Construction joints are required for all cold joints and where the lining thickness changes. Reinforcement shall be continuous through the joint. The design criteria for a concrete lined channel can be found in Table 9.

Table 9: Concrete-Lined Channel Design Criteria

Criteria	Controlling Values
Maximum Velocity	15 fps
Froude Number	$Fr \leq 0.8$ or $Fr \geq 1.2$
Max Side Slope	1.5H:1V
Min Channel Radius Subcritical	2 times 100-year top width
Min Channel Radius Supercritical	Not Allowed
Min Concrete Thickness Subcritical	5”
Min Concrete Thickness Supercritical	7”
Outfalls into Concrete Channel	12” above invert
Min Bedding Layer Subcritical	6”
Min Bedding Layer Supercritical	9”
Min Freeboard	1.0’ and per Equation 11
Concrete Finish	Per Table 10
Maintenance Access	Per Town
EGL and HGL	Plotted on channel profiles
Safety Fencing and Steps	Required unless waived by the town due to very low hazard

Flow in a concrete channel with a Froude number between 0.8 and 1.2 is unstable and increases the possibility of unanticipated hydraulic jumps forming in the channel. It should be avoided at all flows, not just the design flow. To calculate velocity and capacity, the designer should use Manning's Equation with the n values in Table 10. Contact the Town to determine acceptable concrete finishes, typically a troweled and broomed finish is preferred.

Table 10: Concrete-Lined Channel Manning's n Values

Type of Concrete Finish	Manning's n Values		
	Minimum	Typical	Maximum
Trowel finish	0.011	0.013	0.015
Float finish	0.013	0.015	0.016
Finished, with gravel on bottom	0.015	0.017	0.020
Broomed	-	0.016	-
Unfinished	0.014	0.017	0.020
Shotcrete, troweled, not wavy	0.016	0.018	0.023
Shotcrete, troweled, wavy	0.018	0.020	0.025
Shotcrete, unfinished	0.020	0.022	0.027
On good excavated rock	0.017	0.020	0.023
On irregular excavated rock	0.022	0.027	0.030

Freeboard in a concrete channel shall be no less than 1 foot for channels with a top width up to 10 feet and the concrete lining shall be extended above the flow depth to provide the required freeboard. The Town shall be consulted for larger channels. Freeboard will be calculated as:

$$H_{fb} = 2.0 + 0.025V(y_o)^{1/3} + \Delta y \qquad \text{Equation 11}$$

Where:

H_{fb} = freeboard height (feet)
V = velocity of flow (fps)
y_o = depth of flow (feet)
Δy = increase in water surface elevation due to super elevation at bends

and $$\Delta y = \frac{V^2 T}{2 g r_c} \qquad \text{Equation 12}$$

Where:

V = mean flow velocity (fps)
r_c = radius of curvature (feet)
T = top width of channel under design flow conditions (feet)

g = standard gravity ($32 ft/s^2$)

Longitudinal underdrains shall be provided along the channel bottom on 10-foot centers within a free-draining bedding layer. The underdrains shall be free draining and daylight at drops or at other locations suggested by the designer and approved by the Town. A weep hole detail and installation pattern in channel side slopes to relieve hydrostatic pressure shall be provided to the Town for review.

The Town may require a low-flow swale if a small base flow is expected to avoid bottom slime, noxious odors, and mosquito breeding. Fencing and gates for maintenance should be considered if the 100-year design depth exceeds 3ft or is near areas of public access. Manhole-type steps are required for design flows which result in depths over 2ft in case emergency evacuation is required.

5.5.2. Riprap Lined Channels

Riprap lined channels are not preferred by the Town but may be required based on site specific conditions such as high velocities that cannot be lowered by flattening the channel slope, limited space requiring channel side slopes steeper than 3H:1V, and where rapid changes in channel geometry occur. Use of riprap-lined channels must be approved by the Town. FHWA's HEC-15 offers extensive guidance on the design of riprap-lined channels.

Riprap refers to a protective blanket of large loose stones which are usually placed by machine to achieve a desired configuration. Soil riprap is a mix of riprap and native soil. Soil riprap consists of 35% by volume of native soil taken from the channel excavation and 65% by volume of riprap of the specified gradation. It is mixed on-site, before placement. When a riprap lining is used, all areas above frequent flow zones be protected with soil riprap, covered with 6 inches of topsoil, and revegetated with native grasses. Due to its small size, all riprap linings with a d_{50} of 6 inches and smaller should be soil riprap. Recommended seed mixtures for where riprap is buried are shown in Table 11. The riparian seed mix is for perennial streams (near constant water in the invert) and the upland seed mix is for ephemeral streams (typically dry stream bed). Seed mixes other than those listed in Table 11 require Town approval.

Table 11: Seed Mixes

PLS (lbs/ac)	Mix %	Riparian Seed Mix	
		Botanic Name	**Common Name**
1.9	10%	Juncus articus	Arctic Rush
1.9	10%	Calamagrostis canadensis	Bluejoint Reedgrass
1.9	10%	Poa secunda	Cany Bluegrass
1.9	10%	Glyceria striata	Fowl Mannagrass
3.7	20%	Elymus trachycaulus	Slender Wheatgrass
3.7	20%	Deschampsia Caespitosa	Tufted Hairgrass
3.7	20%	Pascopyrum smithii	Western Wheatgrass
18.7	100%	Total lbs/acre	
PLS (lbs/ac)	**Mix %**	**Upland Seed Mix**	
		Botanic Name	**Common Name**
3.0	11.2%	Achnatherum hymenoides 'Rimrock'	'Rimrock' Indian Ricegrass
4.0	14.9%	Agropyron desertorum 'Hycrest'	'Hycrest' Crested Wheatgrass
2.5	9.3%	Bouteloua curtipendula	Sideoats Grama
2.0	7.4%	Bouteloua gracilis 'Lovington'	'Lovington' Blue Grama
3.0	11.2%	Bromus marginatus 'Garnet'	'Garnet' Mountain Brome

2.1	7.8%	Cleome lutea VNS	Yellow Beeplant
3.2	11.9%	Cleome serrulata VNS	Rocky Mountain Beeplant
0.7	2.6%	Chrysothamus nauseosus albicaulis	Tall Blue Rabbitbrush
0.4	1.5%	Eschscholzia californica VNS	California poppy
0.5	1.9%	Eriogonum umbellatum VNS	Sulphur-flower buckwheat
0.5	1.9%	Gaillardia pulchella VNS	Indian blanketflower
0.4	1.5%	Glandularia gooddingii VNS	Desert Verbena
0.5	1.9%	Machaeranthera bigelovii var. bigelovii VNS	Bigelow's tansyaster
0.1	0.4%	Linum lewisii	Blue Flax
0.3	1.1%	Lupinus prunophilus	Chokecherry Lupine
2.5	9.3%	Pascopyrum smithii 'Arriba'	'Arriba' Western Wheatgrass
0.2	0.7%	Penstemon eatonii	Firecracker Penstemon
0.3	1.1%	Penstemon palmeri VNS	Palmer Penstemon
0.7	2.6%	Penstemon strictus VNS	Rocky Mountain Penstemon
26.9	100%	Total lbs/acre	

Riprap failures result from: too many undersized individual rocks within the size range; poor gradation of the rock; and/or improper bedding. There is no maximum depth criterion for riprap-lined channels. Stone sizing for ordinary riprap shall be calculated as:

$$d_{50} = \sqrt{\left(\frac{VS^{0.17}}{4.5(G_s-1)^{0.66}}\right)} \qquad \text{Equation 13}$$

Where:

d_{50} = mean rock size (feet)
V = mean design channel velocity (fps)
S = longitudinal channel slope (feet/foot)
G_s = specific gravity of stone (2.50 minimum)

The riprap layer thickness shall be 2.0 times the calculated d_{50} and should be increased by 50% at the upstream and downstream termination of a riprap lining for at least 3 feet to prevent undercutting. The physical characteristics of riprap stone and the gradation resulting from the d_{50} calculated using Equation 14 as defined in the CDOT Standards. The designer should round up the d_{50} size specified in the construction plans if the calculated d_{50} falls between two standard gradations. Equation 13 is not to be used for sizing riprap for rundowns (chutes) or culvert outlet protection.

Manning roughness coefficients for manmade ordinary riprap or soil riprap channels shall be calculated as:

$$n = 0.0395\ d_{50}{}^{1/6} \qquad \text{Equation 14}$$

Where:

n = Manning's roughness coefficient

d_{50} = mean stone size (feet)

Proper bedding is required for long-term stability of riprap channel protection and should extend up the side slopes at least 1 foot above the design water surface. Bedding is not required for a soil riprap lining. Table 12 shows bedding thickness for different riprap gradations and native soil conditions. When a channel is excavated where 50% or more of the native material is retained on the #40 sieve by weight, only a single layer of Type II material (see Table 13) is required. Otherwise, a two-layer system is required. Alternatively, a single 12-inch layer of Type II bedding can be used.

Table 12: Granular Bedding Layer Requirements

Size of d_{50}	Minimum Bedding Thickness		
	Fine-Grained Native Soils		Coarse-Grained Native Soils
	Type I	Type II	Type II
d_{50} = 6" and d_{50} = 9"	4"	4"	6"
d_{50} = 12"	4"	4"	6"
d_{50} = 18"	4"	6"	8"
d_{50} = 24"	4"	6"	8"

Type I bedding is the lower layer in a two-layer system and Type II is the upper layer. Type I and Type II bedding material specifications are given in Table 13. Type I is equivalent to the CDOT specification for fine aggregate for concrete and Type II is equivalent to the CDOT specification for Class A filter material. Landscaping, filter or other types of fabric are not a substitute for granular bedding.

Table 13: Granular Bedding Gradation Requirements

Bedding Layer Requirements	Percent Passing by Weight	
	Type I	Type II
3 inches	-----	90-100
1½ inches	-----	-----
¾ inches	-----	20-90
3⁄8 inches	100	-----
#4	95-100	0-20
#16	50-85	-----
#50	10-30	-----
#100	2-10	-----
#200	-----	0-3

The potential for erosion increases along the outside bank of a channel bend so it may be necessary to provide additional erosion protection at those locations. The minimum radius of curvature for a riprap-lined channel is two times the top width of the design flow. When radius of curvature divided by the flow top width is equal or greater to 8.0, no increase in protection is needed. Where the radius is smaller than this, an adjusted velocity shall be used to size the bends riprap size. Velocity along the outside of a bend shall be estimated using Equation 15. Bend riprap protection is to be applied to the outside quarter of the channel bottom and to the outside channel side slope a distance of at least 2 times the top width of the flow. Riprap does not need to extend upstream of the start of the curve.

Velocity along the outside of a bend can be calculated as:

$$V_a = \left(-0.147\frac{r_c}{T} + 2.176\right)V \qquad \text{Equation 15}$$

Where:

V_a = adjusted channel velocity for riprap sizing along the outside of channel bends (fps)
V = mean channel velocity for the peak flow of the Major Storm event (fps)
r_c = channel centerline radius (feet)
T = flow top width during Major Storm event (feet)

Riprap protection for other channel transitions where the Froude number is 0.8 or less can be calculated using Equation 13 with the maximum velocity in the transition increased by 25%. Transition protection should extend upstream of the transition entrance at least 5 feet and downstream of the transition exit for a distance of at least 5 times the design flow depth. Design criteria for riprap lined channels in is

Table 14.

Table 14: Riprap-Lined Channel Design Criteria

Criteria	Controlling Values
Maximum Velocity	12-15 fps (see
Froude Number	Fr ≤ 0.8
Manning's n	Per Equation 9
Steepest Side Slope	2.5H:1V
Stone Specific Gravity	Minimum of 2.5
Riprap Gradation (d_{50})	Per Equation 13
Riprap Blanket Thickness	2x d_{50}
Minimum Radius of Curvature	2x flow top width
Riprap for Bend Protection	Use Equations 13 and 15
Outfalls into Concrete Channel	1' to 2' above invert
Bedding Layer	Per Table 12 and Table 13
Minimum Freeboard	1.0' and per Equation 11
Use of Soil Riprap	Riprap d_{50}
Use of Buried Soil Riprap	Riprap d_{50}
Seed Mix	Water Availability
Maintenance Access	Site Specific
EGL and HGL	Plotted on channel profiles
Safety Fencing and Steps	Required unless waived by the Town due to very low hazard

5.5.3. High Gradient Channels

Natural channels can sometimes have steep longitudinal slopes with rip-rap, cobble or rock along their bottoms. These channels are often predicted to have supercritical flow and very high velocities. However, field observations show these channels are often configured so that they are protected by natural armoring. These configurations include short, steep drops with larger rocks situated to resist flow followed by longer, flatter sections of channel. For a more in-depth discussion, 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. n = $0.393\ Sf^{0.38}\ R\text{-}0.16$

Equation 16 may be used as an aid to predict the roughness coefficient of a high-gradient channel provided the criteria below are met. The designer may research how to determine the friction slope if unknown as it is outside the intent of these standards.

$$n = 0.393\ S_f^{0.38}\ R^{-0.16} \qquad \text{Equation 16}$$

Where:

n = Manning's roughness coefficient

S_f = friction slope or water surface slope (feet/foot)

R = hydraulic radius, (wetted area/wetted perimeter) (feet)

The basic guidelines for when this equation should be used are:

1. The channel must be a natural channel that has a relatively stable bank material and a cobble or boulder bed material.

2. The channel friction slope must be between 0.01 and 0.04 feet per foot and the hydraulic radius must be between 0.5 and 7 feet.

3. The channel must not be affected by backwater.

In all cases, a Major Storm flow shall never exceed a depth greater than 5.0 feet or have less than 1.0 foot of freeboard at any point along the channel.

5.5.4. Grouted Boulders

Grouted boulders may be used for drop structures in channels. The design of all grouted boulder drop structures, including materials specifications, shall be in accordance the latest version of the USDCM. Consult with the Town prior to designing a grouted boulder drop structure. A riprap ditch check (Section 3.4) is preferred over a grouted feature.

6. DETENTION AND WATER QUALITY

Development of a parcel must not result in peak runoff rates of a Minor or Major Storm that are greater than Historic conditions, unless the entire site drains directly to a public storm sewer designed to carry the undetained flow, or the Uncompahgre River. Detention basins can help Developments meet these criteria. Total site runoff rate is the sum of detention basin release and direct runoff, both of which must be considered separately and in combination. All detention basins must include a water quality outlet to drain the water quality capture volume within between 12 and 40 hours and must fully drain within 72 hours. Peak runoff rates can be calculated as indicated previously in Section 2 of these Standards. The maximum discharge rate from a detention basin shall not exceed the peak runoff rate of a respective storm at the time of adoption of these regulations.

New subdivisions that include multiple lots should provide a coordinated system of detention for the entire subdivision to minimize the number of detention basins and maintenance requirements. Individual facilities on each lot within a subdivision are not permitted. Detention ponds should be designed as landscaped areas integrated into the site.

6.1. Maintenance

All detention basins must facilitate and plan for maintenance; all private facilities must be regularly maintained by their owners to remove accumulated sediment and ensure the outlet drains freely. All basins that serve more than a single lot or site must have access ramps to the basin outlet to facilitate sediment removal and other maintenance within the basin. Detention basin maintenance is the responsibility of the Development served by the basin.

6.2. Basin Geometry

An emergency overflow weir shall be included at the 100-year water surface elevation. The design flow rate for sizing the overflow weir and revetment shall be the 100-year peak inflow into the pond to account for outlet becoming fully clogged. The embankment above the overflow weir and around the basin shall provide 1.0 foot of freeboard minimum above the design flow depth over the weir. Figure 3 offers a schematic. Weir flow is discussed later in this section. Basin side slopes may be no steeper than 3H:1V. The basin invert shall be sloped at 2 percent or more towards the outlet. Detention basins should have a length-to-width ratio not less than 2. A forebay is required to consolidate incoming sediment for ease of maintenance. Forebay sizing shall be per the Urban Storm Drainage Criteria Manual (USDCM) developed by the MHFD.

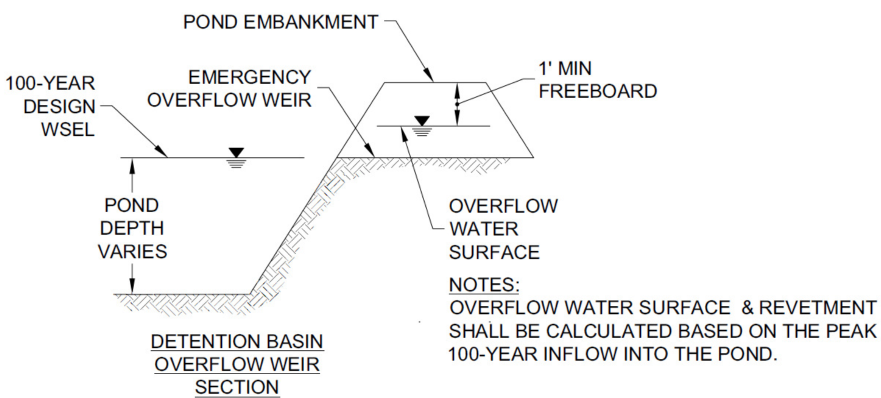


Figure 3: Basin Freeboard Schematic

6.3. Basin Sizing Using the FAA Method

If the Rational Method is used to calculate peak flows, the FAA Method described in this subsection shall be used to determine required basin 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 basins or unusual watershed storage characteristics. Rainfall intensity values can be determined by NOAA Atlas 14 estimates which can be found online or in Ridgway’s Stormwater Master Plan.

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

$$V_i = (CIA)(T_c)(60\ \text{seconds/minute}) \qquad \text{Equation 17}$$

Where:

V_i = inflow volume (cubic feet)
C = Rational Method runoff coefficient for the Major or Minor Storm
I = design rainfall intensity (inches/hour)
A = watershed area draining to the detention pond (acres)
T_c = Rational Method time of concentration (minutes)

2. Determine the outflow volume by multiplying the allowable release rate by the same time of concentration used in step 1.

$$V_o = (R_a)(T_c)(60 \text{ seconds/minute}) \qquad \text{Equation 18}$$

Where:

V_o = outflow volume (cubic feet)
T_c = Rational Method time of concentration used in step 1 (minutes)
R_a = allowable release rate as determined per these Standards (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 basin must be decreased to compensate for site runoff that is not detained. The allowable release rate from the detention basin is the total site existing conditions peak runoff rate minus the post-Development undetained flow rate from areas not draining to the detention basin. A maximum of 5 percent of the total site may bypass the detention basin unless approved by the Town.

6.4. Basin Sizing Using SWMM

If SWMM is used to calculate peak runoff rates, it can be used to develop inflow hydrographs at the detention basin site. The program can then be used to determine the required storage volume and outlet design based on an iterative 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 peak flow and an acceptable drain time are achieved. Assumptions made during detention basin design, all design calculations, and SWMM input and output text files shall be provided to the Town for review. Files shall be highlighted and design values shall reference calculations. The outputs shall include comments and/or be summarized periodically to ease in the review process.

6.5. Water Quality Capture Volume

The water quality capture volume (WQCV) represents the volume associated with the 1.25-year return period storm. Detaining this volume is considered to provide the best value in water quality treatment. All detention basins will be designed with a water quality outlet in addition to the Minor and Major Storm outlets, but the WQCV can be assumed to be contained within the Minor and Major Storm volume for FAA Method basin sizing. Any increase of imperviousness greater than 0.05 acres, or an improvement which results in a parcel's imperviousness percentage over land use default values (Table 3), or the creation of a PUD or a parcel within the Uncompahgre River Overlay District is required to provide WQCV detention for the entire parcel onsite, even if other detention is not required. The WQCV detention is to be based on the entire parcels imperviousness, including existing or Historic features when further Development occurs. The MHFD has spreadsheets that can aid in the design of the WQCV outlet. The equation to calculate the WQCV in Ridgway is:

$$\mathrm{WQCV} = \frac{((\mathrm{A}*\mathrm{a}*(0.91*(\mathrm{i}^3)-(1.19*(\mathrm{i}^2)+0.78\mathrm{i}))}{12}$$ Equation 19

Where:

WQCV = water quality capture volume (acre-feet)
A = area draining to the detention basin (acres)
a = 0.8, the WQCV drain time coefficient corresponding to a 12-hour drain time
i = imperviousness as a decimal percentage

Assuming 100% imperviousness, the above equation can be simplified to approximate required treatment volume in cubic feet if desired.

$$\mathbf{WQCV}_{\boldsymbol{cft}} = \mathbf{0.022A}_{\boldsymbol{ft}}$$ Equation 20

Where:

$WQCV_{cft}$ = water quality capture volume (cubic feet)
A_{ft} = total impervious area (square feet)

The particular treatment method for the WQCV can be determined by the owner or developer but is subject to Town approval. Treatment methods shall be recognized by the Mile High Flood District or other referenced standards.

6.6. Outlet Design Concepts

Detention basin outlets are complex because of the need to detain multiple events to different release rates. Several different outlet design examples can be found in the Urban Storm Drainage Criteria Manual (USDCM) developed by the MHFD. The MHFD also provides spreadsheets that can be used to aid in designing multi-stage outlets. While flow out of a detention basin is often controlled by an orifice place, no outlet pipe shall be smaller than 12 inches in diameter so that it may be easily cleaned. The invert of the lowest outlet shall be set at the lowest point in the basin or at the minimum pool elevation, if applicable. The outlet pipe shall discharge into a standard manhole or into a drainageway with proper erosion protection. All orifice plates shall be removable. The outlet structure shall be located along the downstream embankment of the basin and in a location that can be accessed for maintenance. In no case shall the outlet structure be in the middle of the pond.

Each detention basin shall include a water quality outlet designed to drain the WQCV in 12 hours, a Minor Storm outlet, and a Major Storm outlet that allows for the release of any detained water at the allowable flow rate. An emergency overflow path shall be provided in the event the outlet becomes clogged or a storm larger than the Major Storm occurs. The emergency overflow shall provide conveyance of the Major Storm inflow so that there is no damage to the surrounding area or to downstream facilities. The invert of the emergency overflow should be set at or above the 100-year water surface elevation.

6.7. Outlet Hydraulic Design

Hydraulic design of outlets consists of one or more weirs and orifices. The equation for a broad crested weir is:

$$\mathrm{Q} = \mathrm{CL}(\mathrm{H})^{3/2}$$ Equation 21

Where:

Q = discharge (cfs)
C = weir coefficient (see Table 15)
L = horizontal length (feet)
H = total energy head (feet)

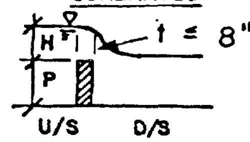
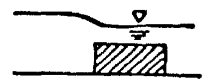
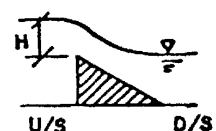
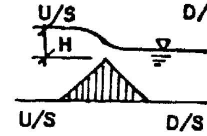
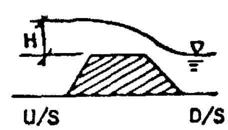
Another common weir is the v-notch, whose equation is as follows:

$$Q = 2.5 \tan(\Theta/2)\ H^{5/2} \qquad \text{Equation 22}$$

Where:

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

Table 15: Weir 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	

Reference: King & Brater, Handbook of Hydraulics, 1963

The equation for orifice flow is:

$$Q = (C_d)(A)(2gH)^{1/2} \qquad \text{Equation 23}$$

Where:

Q = flow (cfs)

C_d = orifice coefficient
A = area (square feet)
g = gravitational constant (32.2 feet/second^2)
H = head on orifice measured from orifice centerline (feet)

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

6.8. 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.

Colorado Revised Statute (CRS) §37-92-602 (8) provides legal protection for any detention basin in the Town, provided it meets the following criteria:

1. It is owned or operated by a governmental entity or is subject to oversight by a governmental entity;

2. It continuously releases or infiltrates at least 97% of the runoff from a rainfall event that is less than or equal to a 5-year storm within 72 hours after the end of the event;

3. It continuously releases or infiltrates as quickly as practicable, but in all cases releases or infiltrates at least 99% of the runoff within 120 hours after the end of events greater than a 5-year storm; and

4. It operates passively and does not subject the stormwater runoff to any active treatment process (e.g., coagulation, flocculation, disinfection, etc.)

All new detention basins including individual site basins built by private parties must meet the criteria above and be reported by the engineer of record to the state stormwater notification portal online.

Operation, maintenance, repair, and replacement of all detention basins is the responsibility of the party that develops the basin or its successors in interest. The Town assumes no responsibility for the operation, maintenance, or function of any detention or water quality basin.

7. BUILDING ENTRIES

To help prevent flooding of a building, all building entrances must be at an elevation above the adjacent drainage feature or roadway. The burden shall be on the owner to show that any criteria required below have been met. Minimum building elevations can be seen in Table 16. Where one is allowed to build to lot line, garage floors will be allowed to be 12 inches lower than minimum building entry elevation if all doors entering into habitable space, mechanical, plumbing, electrical and other appliances meet the elevation requirements below. Buildings shall also have positive drainage away from the foundation and shall not result in flooding of a neighboring property.

Table 16: Minimum Building Entry Elevations

Road Drainage Type	Min. Building Entry Elevation
Areas with curb and gutter only	12" above top of gutter

Areas with roadside ditches only	12” above outside edge of roadway or top of ditch, whichever is higher

If a property owner or their designated representative can display practical difficulty or unnecessary hardship in achieving the above minimum building entry elevation requirements, the entry requirements may be a minimum of 12 inches above surrounding final parcel grade with Town Staff approval.

In the event building entry elevations cannot meet either of the above situations, a mitigation plan can be provided by a licensed engineer and the following sections of Ridgway Municipal Code 6-2 “Flood Plain Management Regulations” shall apply; 6-2-1, 6-2-2(C)(1) through 6-2-2(C)(6), 6-2-3, 6-2-4(B) through 6-2-4(P), 6-2-5, 6-2-8 and 6-2-9. Any reference to flood, flood plain or similar when referencing Ridgway Municipal Code 6-2 shall be interpreted as 12” above final parcel grade for the purpose of these stormwater regulations. The mitigation plan shall be submitted to Town staff for review and approval.

8. BRIDGES

If a bridge is required or desired within the Town, its design must consider flow velocity through the bridge, pier and abutment scour, backwater effects, and roadway overtopping. Bridge openings should result in as little change in flow characteristics as is reasonable, consistent with good design and economics. The Town will review bridge designs based on the guidance in this section, however, the designer is required to contact FEMA for additional requirements if the bridge is on a FEMA-regulated waterway. At the time of adoption, the Uncompahgre River and Cottonwood Creek are the only two regulated waterways within Town limits.

8.1. Hydraulic Analysis

The hydraulic analysis of bridges shall be completed in accordance with the FHWA Hydraulics of Bridge Waterways, FHWA HY-4, or HEC-RAS.

8.2. 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 anywhere on the channel.
2. The 100-year water surface elevation within the bridge shall also be a minimum of 1.0 foot below the lowest 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 all bridges and low water crossings shall be prepared and certified by a Colorado Registered Professional Engineer (as is required for all stormwater design work).
6. In all instances, all bridges shall meet all applicable FEMA floodplain regulations. The Town of Ridgway requires a Floodplain Development Permit for any work located within a FEMA designated floodplain per Ridgway Municipal Code 6-2.

9. FEMA FLOWS AND FLOODPLAINS

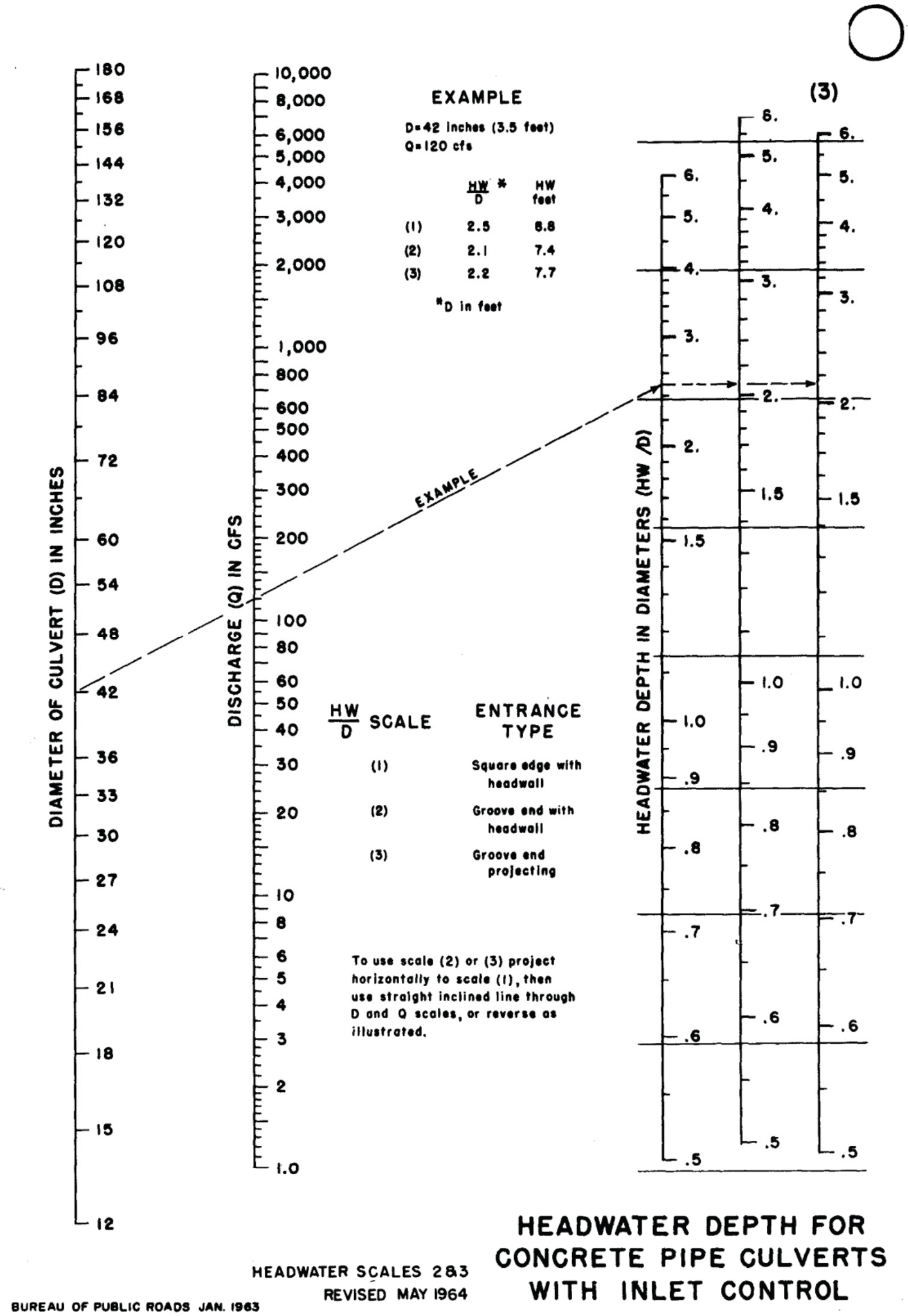
The Federal Emergency Management Agency (FEMA) maintains a floodplain map of the Uncompahgre River. For this waterway, the flow rates and water surface elevations for each of the return periods studied in the effective model shall be used for design of improvements including site grading and layout as well as channel improvements. Additionally, no Development of any kind may be completed within the floodplain boundaries designated by FEMA without a floodplain development permit issued by the Town per Ridgway Municipal Code 6-2. Contact the Town for additional floodplain restrictions and requirements. FEMA has defined a flood zone for Cottonwood Creek but does not have a model available.

10. CONSTRUCTION WATER QUALITY

The Colorado Department of Public Health and Environment (CDPHE), as authorized by the Clean Water Act, issues permits to prevent the discharge of pollutants to waterways during construction. At the time of adoption, construction sites that will disturb one acre or more or are part of a common plan of development (such as a subdivision or commercial development) or sale are required to apply for and receive a permit for stormwater discharges associated with construction activities from the State. The CDPHE also issues permits for construction dewatering and other construction related activities. Information on the applicability of these permits and the associated requirements can be found via online for CDPHE water quality construction permits.

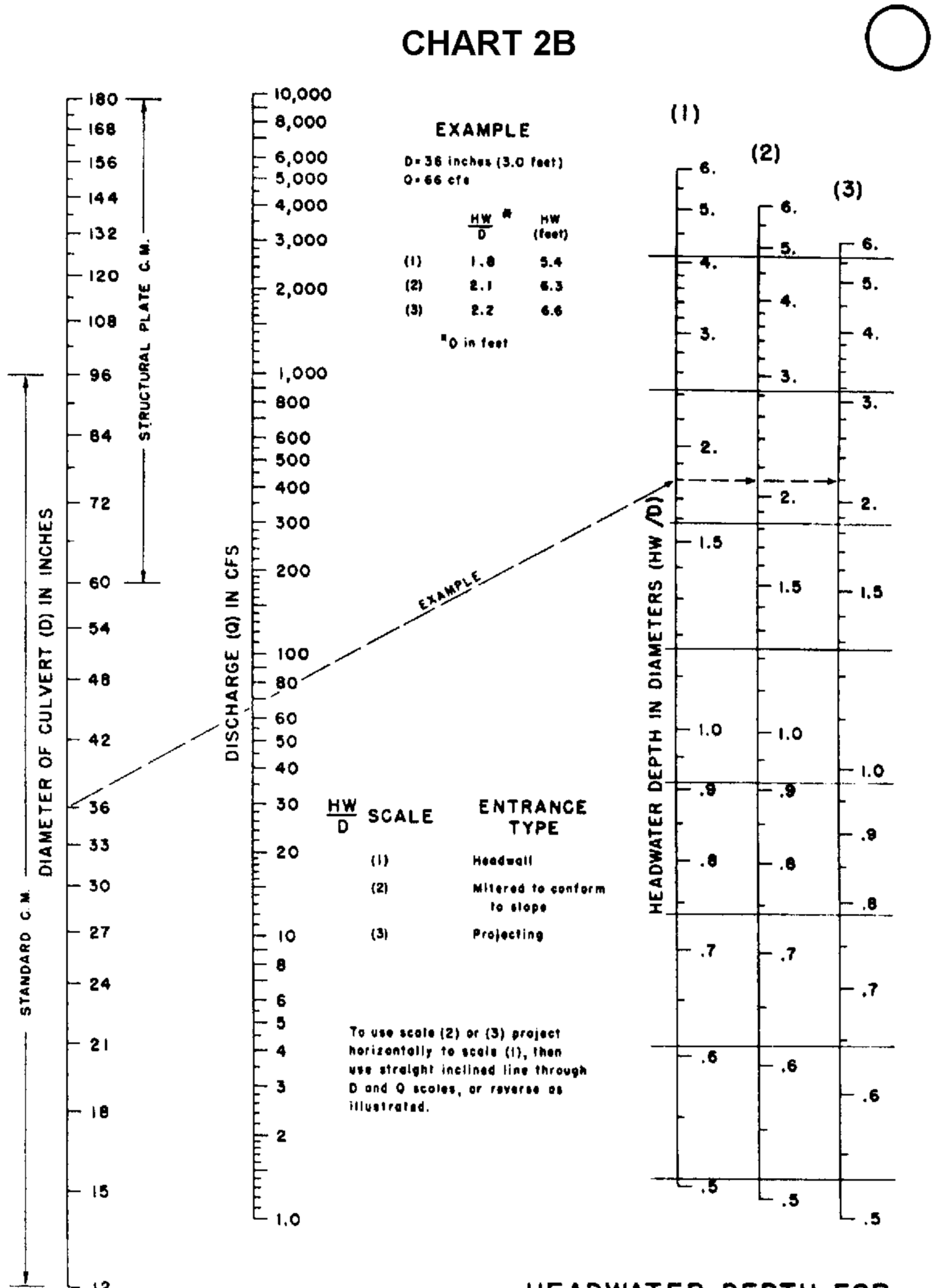
11. APPENDIX A: Nomographs

CHART 1B



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

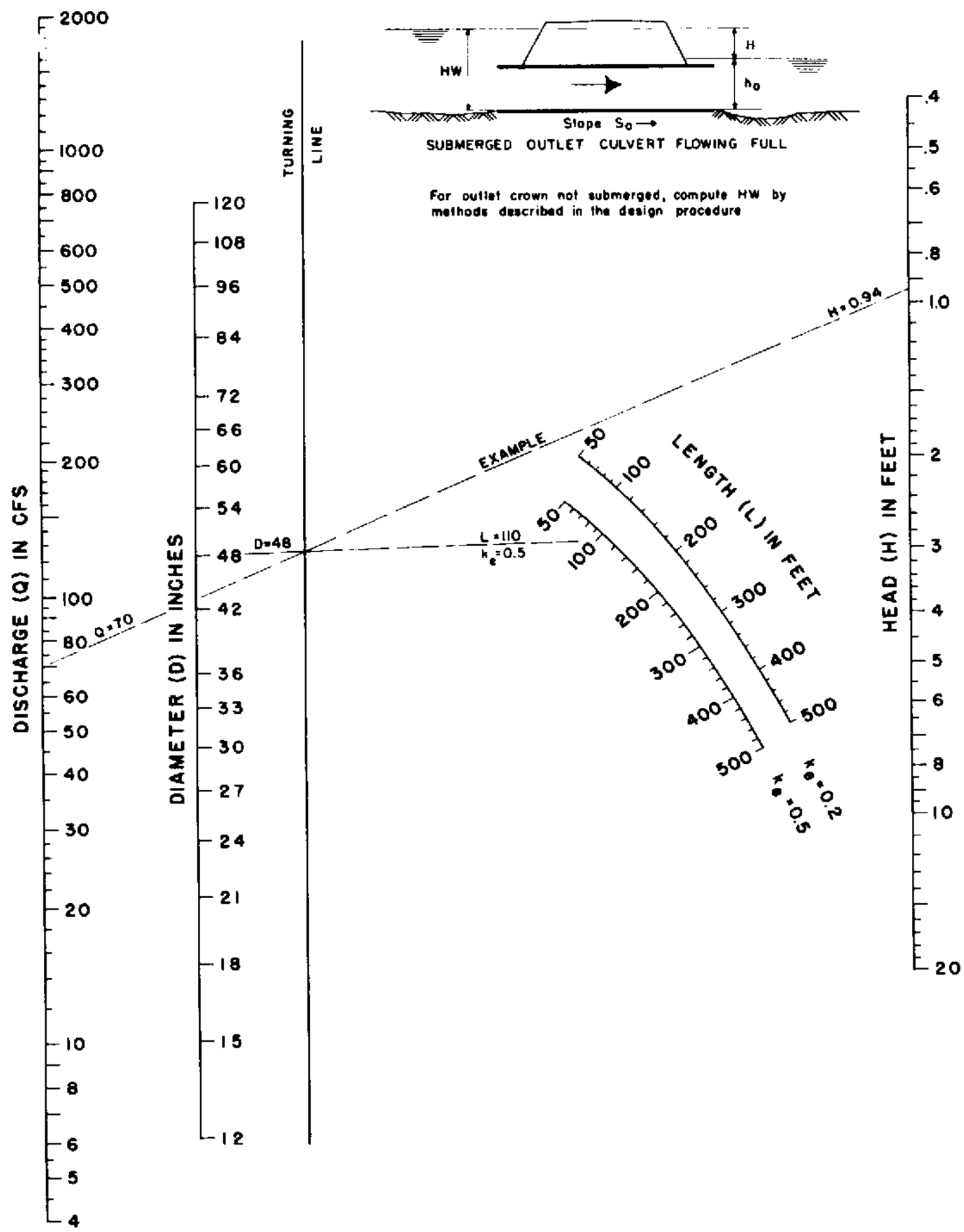
CHART 2B



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

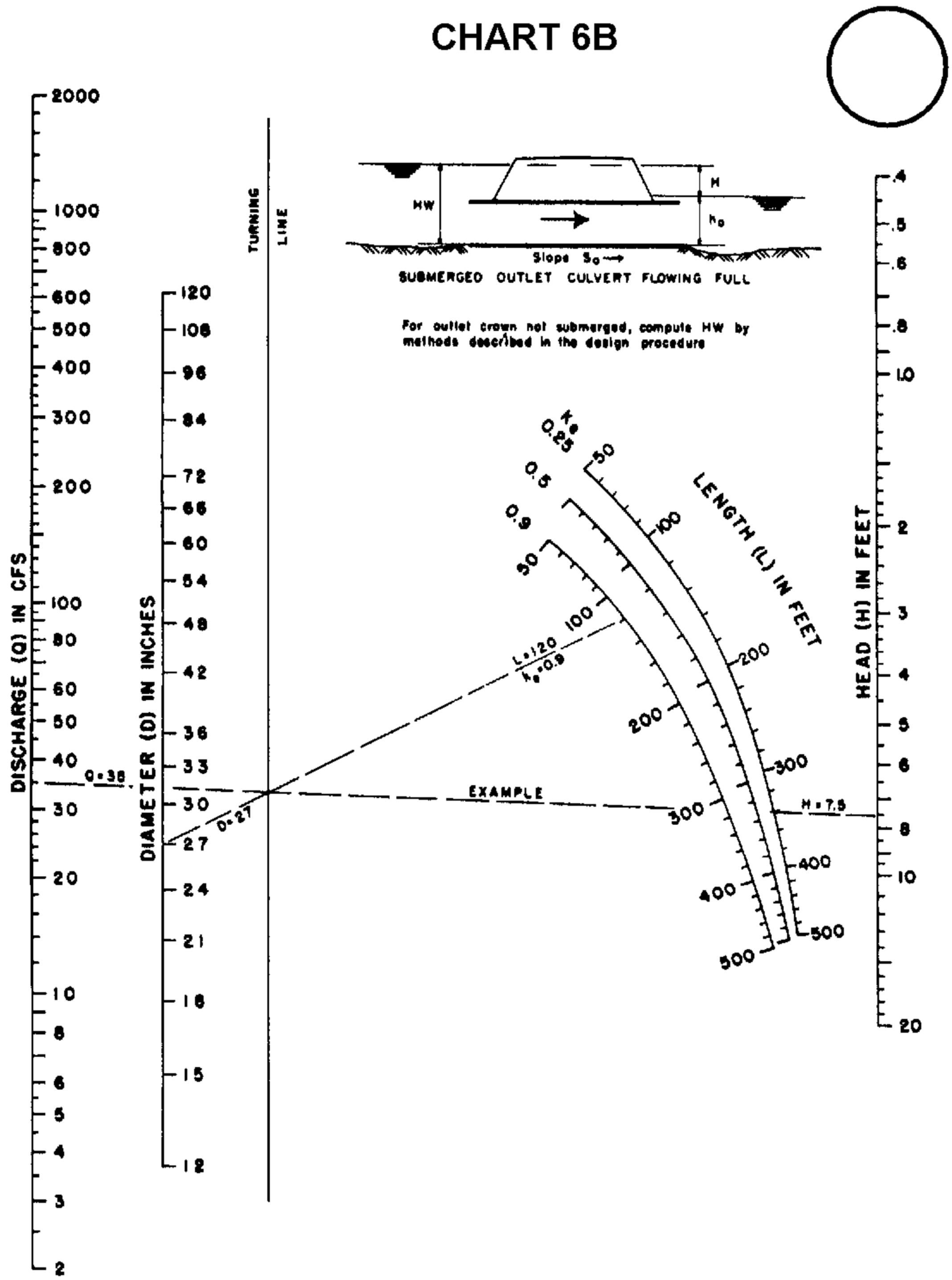
CHART 5B



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

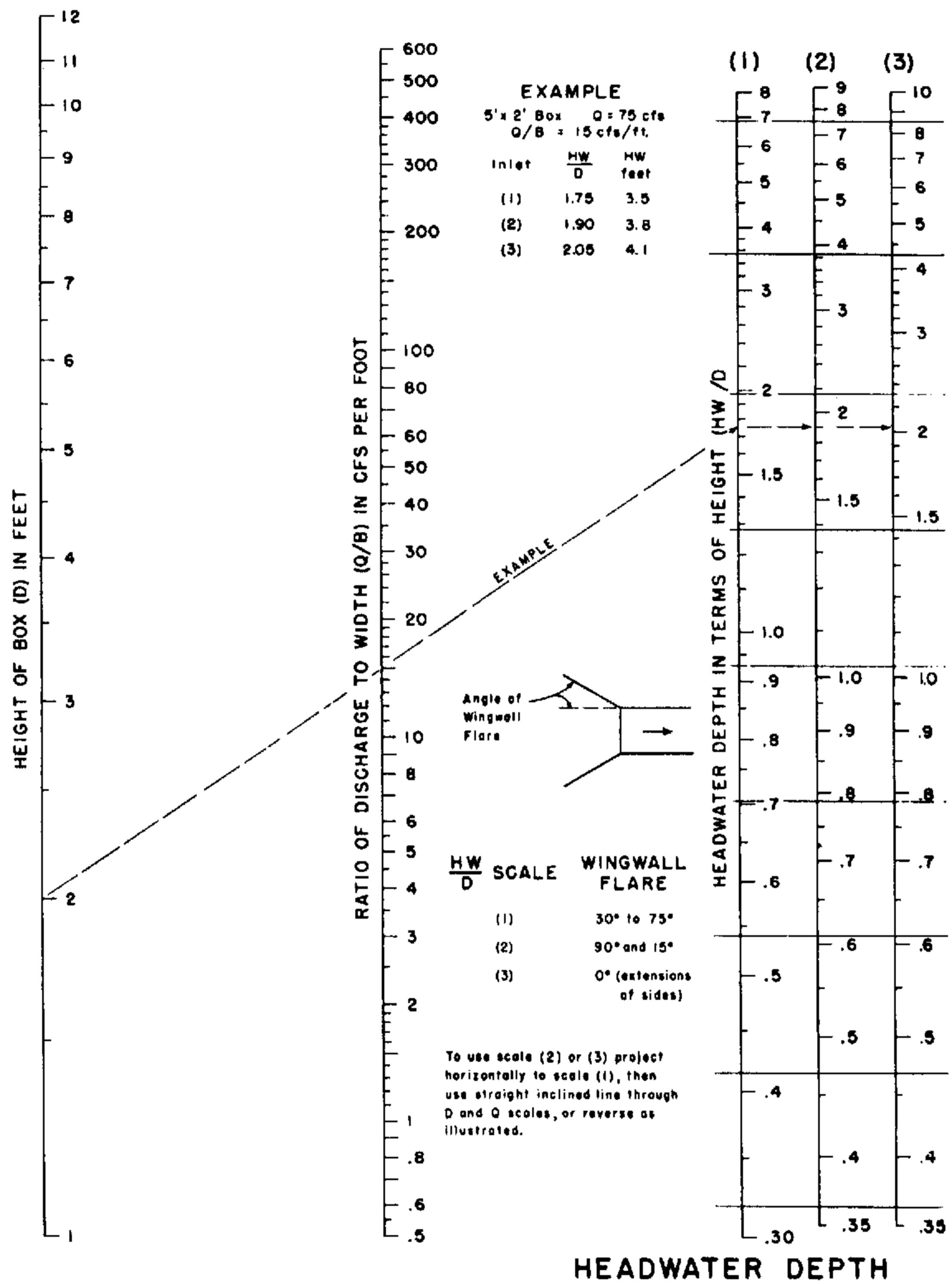
CHART 6B



HEAD FOR
STANDARD
C. M. PIPE CULVERTS
FLOWING FULL
n = 0.024

BUREAU OF PUBLIC ROADS JAN. 1963

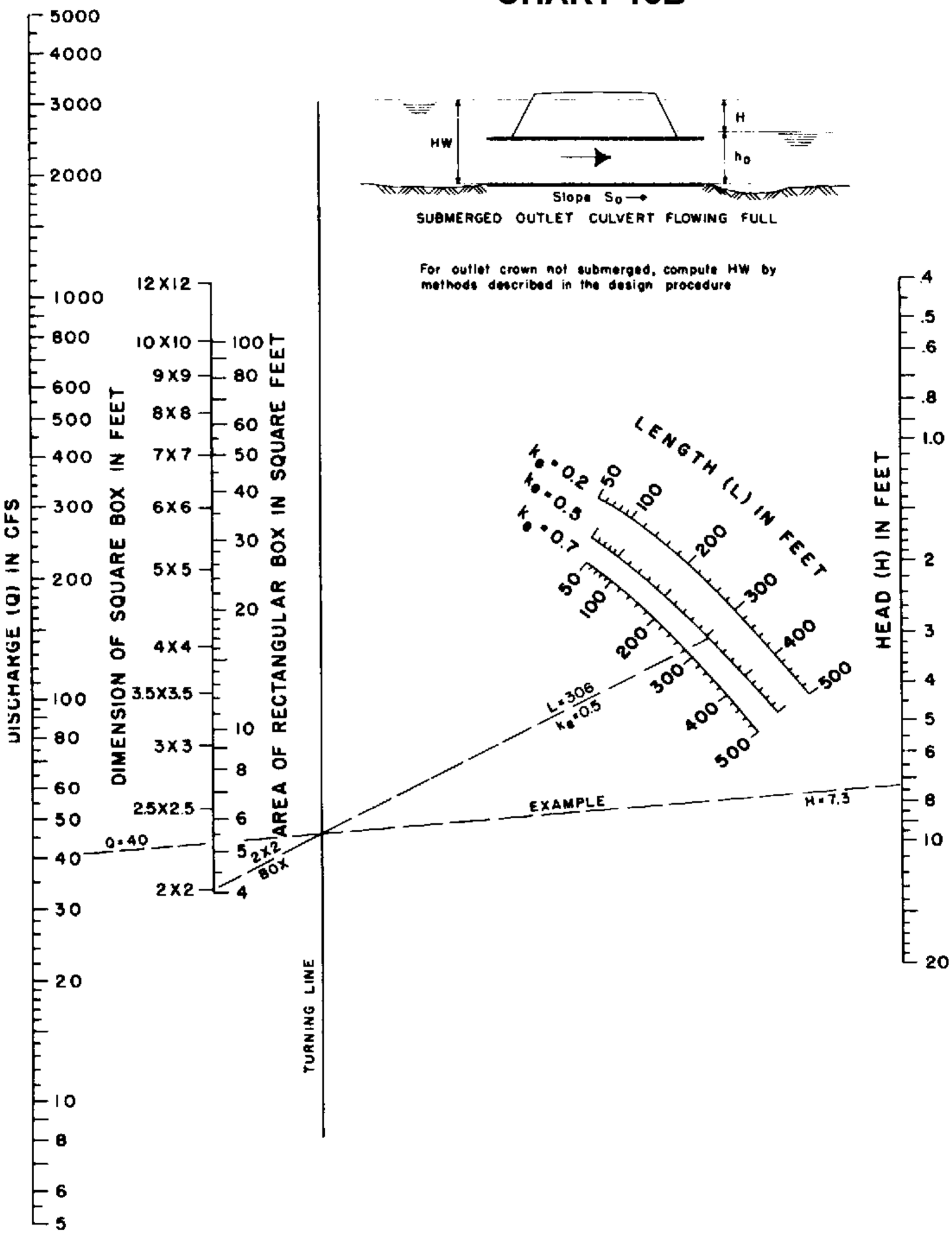
CHART 8B



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

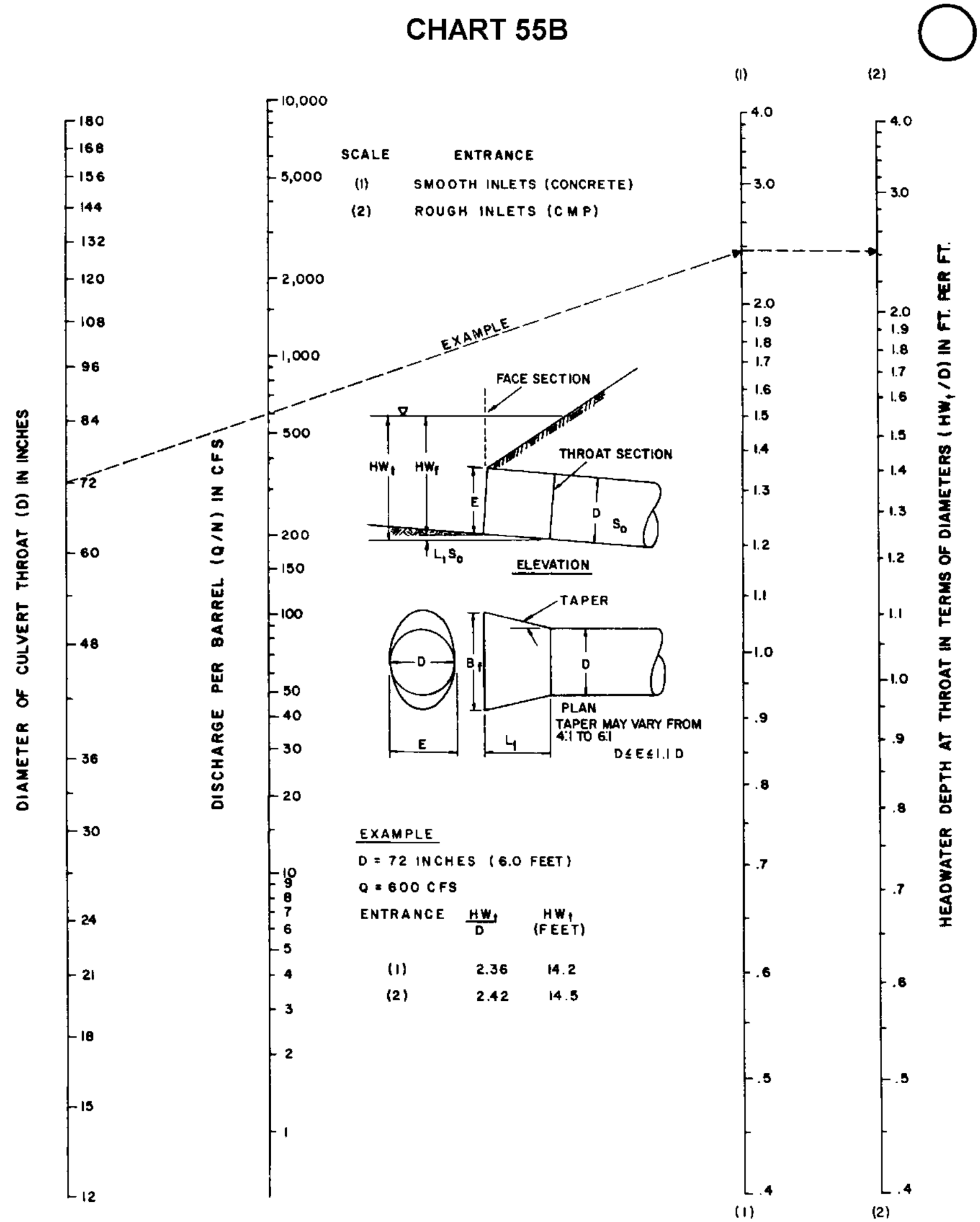
CHART 15B



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

AU OF PUBLIC ROADS JAN. 1963

CHART 55B



ENTRANCE	$\frac{HW_t}{D}$	HW_t (FEET)
(1)	2.36	14.2
(2)	2.42	14.5

THROAT CONTROL
FOR SIDE-TAPERED INLETS TO PIPE CULVERT
(CIRCULAR SECTION ONLY)