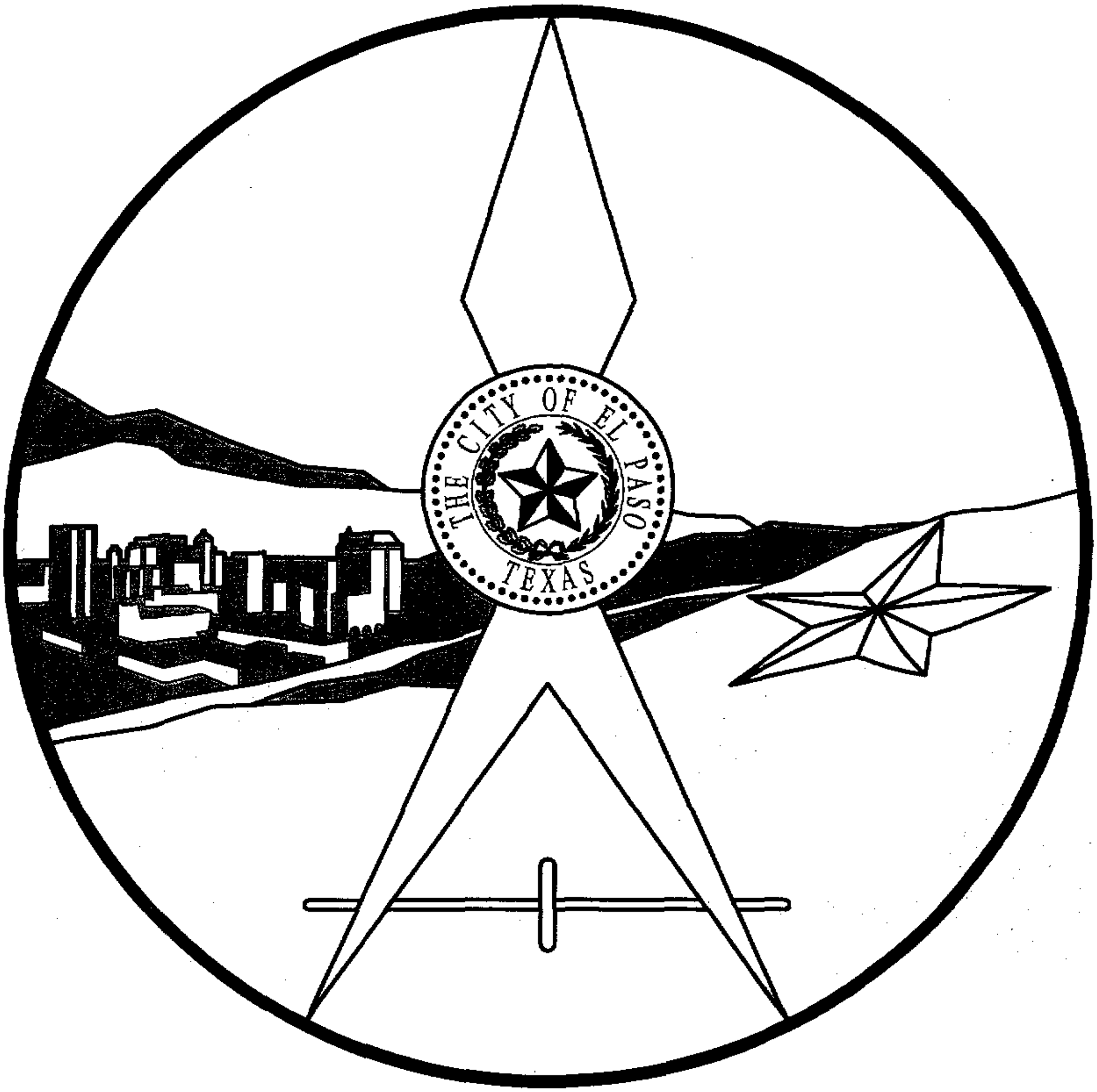

CITY OF EL PASO
ENGINEERING DEPARTMENT

DRAINAGE DESIGN MANUAL

JUNE, 2008

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1. Introduction

This Drainage Design Manual (DDM) is an official document authorized under the City of El Paso Subdivision Ordinance Chapter 19.19 Storm Water Management Requirements. Under that authority, the Chief Technical Officer of the Storm Water Utility (El Paso Water Utilities) and the City Engineer may revise and/or approve design plans and construction related to the DDM provisions in their respective areas described in the ordinance. Both entities are committed to successfully implementing and achieving the goals of the DDM for the benefit of the citizens of El Paso relative to Storm Water Management.

Adopted on ____________________________, 2008.

Approved By: ___________________________ ___________________________

Bert Juarez, P.E., Chief Technical Officer	R. Alan Shubert, P.E., City Engineer
Storm Water Utility – Technical Services	Engineering Department
El Paso Water Utilities	City of El Paso

2. Drainage Design Manual User Instructions

Revision sheets to the DDM will be distributed by the City of El Paso-Engineering Department to all registered holders of the Drainage Design Manual. Contact the City of El Paso-Engineering Department at (915) 541-4200 for registration.

3. Design Criteria

It is the policy of the City of El Paso and El Paso Water Utilities to design drainage structures to meet certain minimum standards. In general, drainage structures are designed to safely pass a flood flow, the magnitude of which is commensurate with an appropriate level of public safety and economic risk. This document establishes minimum standards in terms of design frequency floods and their effect on a drainage facility. Design frequency floods shall be estimated using the procedures described in subsequent chapters of this Drainage Design Manual.

Drainage structures must also be designed to meet all applicable laws including those concerning:

- Alterations of floodplains established in flood insurance studies.
- Construction in flood hazard areas.
- Encroachments or effects on the waters of the United States.
- Water pollution control, including sediment control.
- Protection of fish and wildlife.
- Protection of neighboring property owners.
- Prevention of adverse social and economic impacts.
- Protection of historic properties and archaeological sites.

Designers should use the flood magnitudes, as presented in Table 3-1, for design, unless otherwise directed in writing by the City Engineer or El Paso Water Utilities. Maximum water surface limits shown in this table should also be observed. Hydrologic and Hydraulic analysis of drainage structures should be performed using the methods identified in the subsequent chapters of this DDM.

Table 3-1: Hydrologic Design Criteria

Conveyance Structures	Design Storm and Duration	Maximum Water Surface
Streets (Arterial and Major Collector)	25-year	One lane high and dry (on roadways of four lanes or more)
	100-year	Within curb height
Streets (Local Streets)	25-year	Within curb height
	100-year	Within Right-of-Way
Storm Drain	100-year	HGL below finished grade
Channels	100-year	Contains Water Surface Elevation (WSEL) plus freeboard requirements
Culverts	50-year	Headwater (HW) to soffit
	100-year	HW to edge of road
Bridges	100-year	WSEL plus 1-foot freeboard below the low chord
	500 year	Withstand the structural forces
Retention Basins (no outlet)	100-year	Contain 100% of the runoff volume
Detention Basins	100-year	Basins to be designed utilizing good engineering practices and accepted methods (HEC-1) whereby 100% of the runoff volume to be properly managed through the use of channels and basins.

General Design Notes:

1. Developers are responsible for the additional runoff generated by proposed development; they must ensure that the historic runoff volume, peak, and duration are maintained.
2. The design frequency flood may be changed when a risk analysis indicates that an inappropriate design flood has been used.
3. The duration for all design frequency floods is 24-hours.
4. Drainage structures must be sized so that the 100-year floodplain is not made worse on adjacent properties.
5. Structure sizes should account for sediment bulking of the flow.
6. In no case shall a culvert size be less than 24" circular pipe culvert or its equivalent hydraulic capacity.
7. In no case shall storm drain pipe size be less than 18" circular pipe or its equivalent hydraulic capacity.
8. All storm water storage shall drain within 72 hours through infiltration, gravity outlet, or mechanical means.

4. Hydrology

Hydrology is the process of quantifying storm water runoff discharge and volume for a specified drainage basin at a chosen concentration point. Hydrology is a means to define flood-prone areas, historical conditions, and post-developed conditions. Concentration points are located at outfall locations (lowest point of the basin) and are commonly located at storm drain systems, culverts, and detention ponds. Drainage runoff is collected in waterways and conveyed to concentration points via arroyos, streams, channels, and streets with curb and gutter.

Natural drainage basins are defined by existing ridgelines and can be delineated from published topographic maps, such as United States Geological Survey (USGS) topographic maps, topographic surveys, or available Digital Terrain Models (DTMs). For developed lands, drainage basins are defined by high and low point grade breaks and are depicted on site specific topographic surveys.

4.1. Methodology Selection

Hydrology methodology should be chosen in accordance with Figure 4-1 and Figure 4-2 based on the drainage area and characteristics of the watershed.

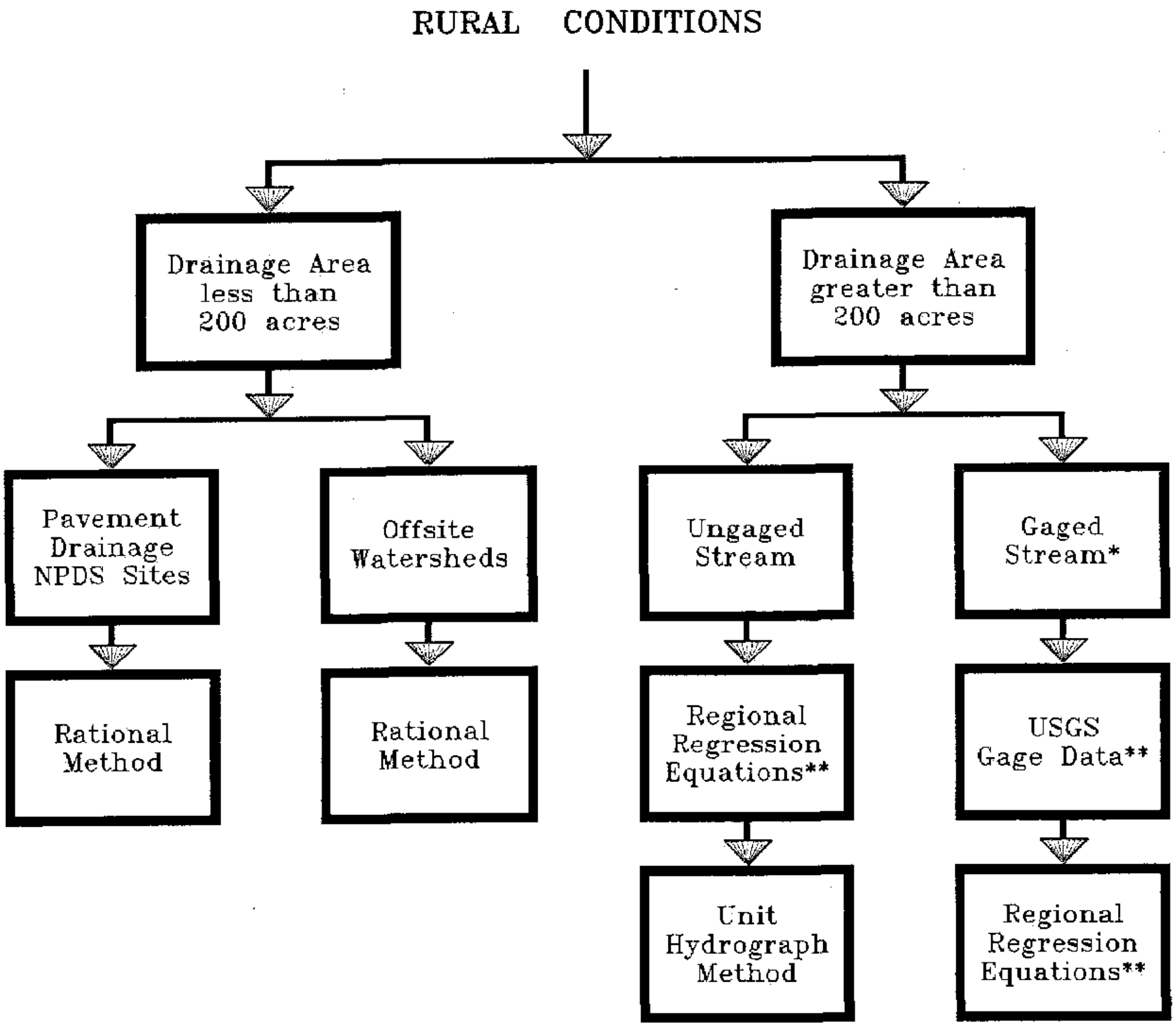


Figure 4-1: Rural Conditions

*Only gage data from USGS gages will be allowed for use on City projects.
**The City may require designers to provide a supplementary Unit Hydrograph calculation for comparison purposes.

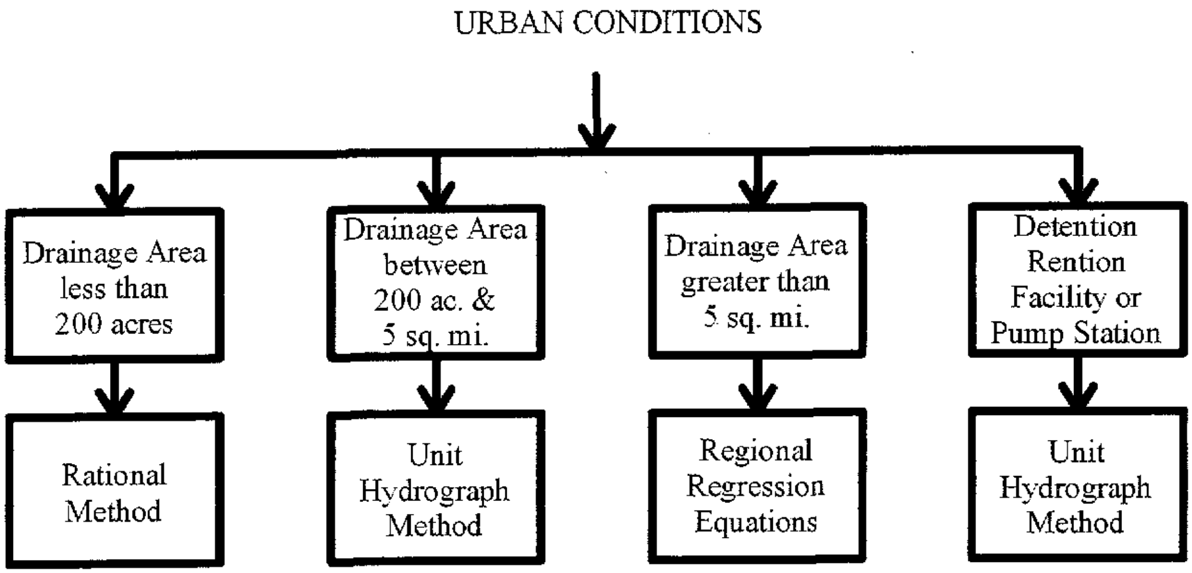


Figure 4-2: Urban Conditions

The decision for rural and urban design condition relies on if the project is within the incorporate regions of the City of El Paso. If so, then the urban condition will be used and all others will be designed under the rural condition. It is recommended that users of this manual check the reasonableness of their calculation values by the use of two methods.

4.2. Drainage Basins with Gage Data

Stream gage data must come from either a local, state, or federal governing agency. Where gage data is available for a watershed, statistical analysis to develop frequency estimates is preferable to other hydrologic analysis methods. Results of this type of analysis will provide the most realistic estimation of runoff values for a watershed. Development of runoff frequency events should conform to the procedures outlined in *Guidelines for Determining Flood Flow Frequency, Bulletin #17B* (USGS, 1981). It is recommended that at least 10 years of non-zero data be used to ensure more accurate statistical analysis.

4.3. Drainage Basins without Gage Data

4.3.1. General Data for Hydrologic Analysis

4.3.1.1. Basin Delineation

Many factors determine the hydraulic character of the natural drainage system such as drainage area, slope, hydraulic roughness, natural and channel storage, drainage density, channel length, antecedent moisture conditions, urbanization, and other factors. The effect that each of these factors has on the important characteristics of runoff is often difficult to quantify. Figure 4-3 shows the effects of the hydraulic character of a given drainage system.

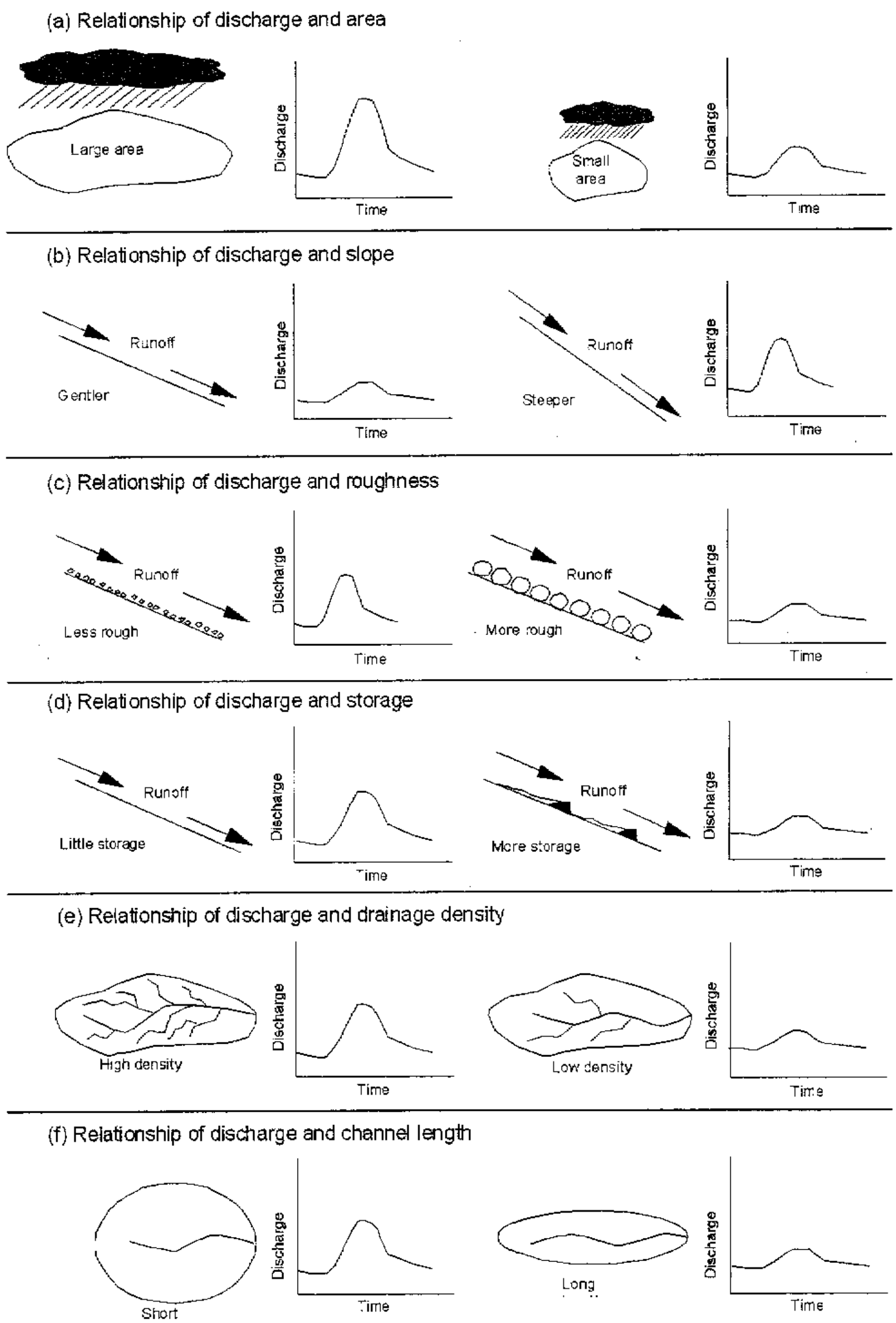


Figure 4-3: Effects of Basin Characteristics on Flood Hydrograph

The following procedure outlines basin delineation:

1. Obtain a viable source of maps that shows the project with contours.
2. Identify the outfall location or the outlet of the basin, generally the lowest point.

3. Identify ridgelines that encompass the basin by analyzing the contours. Mark the ridgelines.
4. Basin should be a closed polyline as to which a device can be used to measure area, such as a planimeter.

4.3.1.2. Rainfall

Intensity-Duration-Frequency Data for common design storm events for the El Paso Area has been developed and published in *Flood Frequency Determination, El Paso County and Incorporated Communities, Texas, Task Order 30* dated March 26, 2006, by Mapping Alliance Partnership. Drainage Regions for the City of El Paso were delineated and are shown in Figure 4-4. The intensity equations for typical storm events are presented following Figure 4-4.

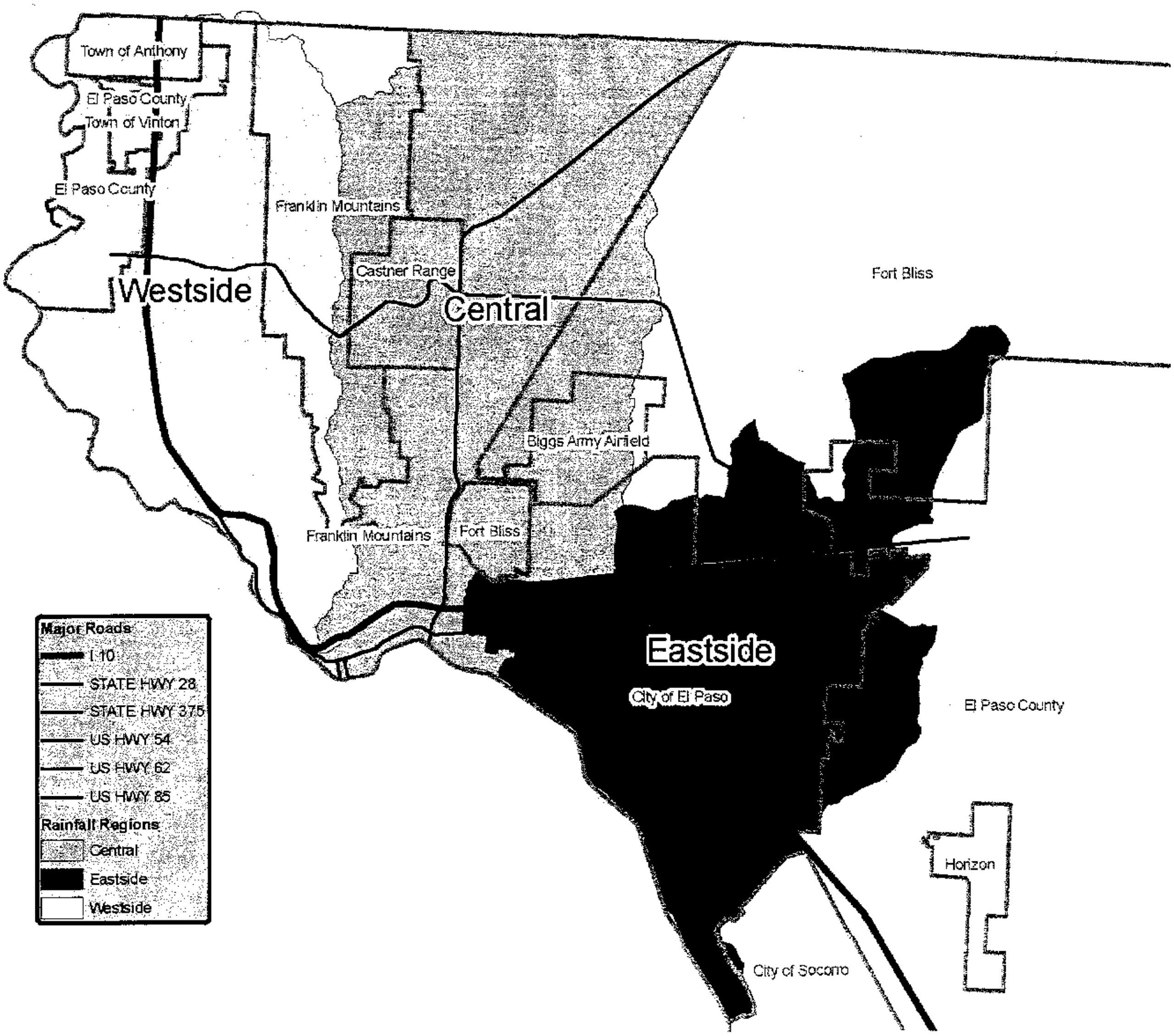


Figure 4-4: ***Drainage Regions (Revised)***

Central Region

Table 4-1: Central Duration Depth Frequency

Return Frequency	Total Rainfall Depth (inches) by Duration						
	1 hr	2 hr	3 hr	4 hr	6 hr	12 hr	24 hr
1	0.41	0.52	0.57	0.61	0.66	0.72	0.80
2	0.70	0.88	0.95	0.99	1.07	1.18	1.35
5	0.97	1.22	1.30	1.36	1.46	1.61	1.83
10	1.15	1.45	1.55	1.62	1.73	1.91	2.16
25	1.41	1.79	1.89	1.99	2.11	2.33	2.60
50	1.61	2.06	2.18	2.30	2.43	2.68	2.96
100	1.84	2.36	2.49	2.64	2.78	3.06	3.34
250	2.18	2.82	2.96	3.16	3.30	3.63	3.89
500	2.47	3.21	3.37	3.62	3.74	4.12	4.35

(Source; FEMA's Flood Frequency Analysis)

Central Intensity Equations

$$I_1 = \frac{22.99}{(T_c + 28.777)^{0.8970}} \qquad \text{4-1}$$

$$I_2 = \frac{31.46}{(T_c + 18.323)^{0.8705}} \qquad \text{4-2}$$

$$I_5 = \frac{44.63}{(T_c + 17.907)^{0.8768}} \qquad \text{4-3}$$

$$I_{10} = \frac{53.69}{(T_c + 18.000)^{0.8791}} \qquad \text{4-4}$$

$$I_{25} = \frac{70.95}{(T_c + 19.798)^{0.8915}} \qquad \text{4-5}$$

$$I_{50} = \frac{91.77}{(T_c + 24.562)^{0.9087}} \qquad \text{4-6}$$

$$I_{100} = \frac{111.04}{(T_c + 26.09)^{0.9177}} \qquad \text{4-7}$$

$$I_{250} = \frac{149.73}{(T_c + 30.146)^{0.9371}} \qquad \text{4-8}$$

$$I_{500} = \frac{198.78}{(T_c + 35.887)^{0.9602}} \qquad \text{4-9}$$

Westside Region

Table 4-2: Westside Duration Depth Frequency

Return Frequency	Total Rainfall Depth (inches) by Duration 1 hr	2 hr	3 hr	4 hr	6 hr	12 hr	24 hr
1	0.43	0.54	0.59	0.63	0.68	0.74	0.83
2	0.73	0.91	0.98	1.03	1.11	1.22	1.40
5	1.04	1.31	1.40	1.47	1.58	1.74	1.98
10	1.28	1.62	1.72	1.81	1.93	2.13	2.41
25	1.64	2.08	2.20	2.32	2.46	2.72	3.03
50	1.95	2.49	2.63	2.77	2.93	3.23	3.57
100	2.31	2.96	3.12	3.31	3.47	3.83	4.18
250	2.86	3.70	3.89	4.15	4.33	4.76	5.11
500	3.36	4.36	4.58	4.92	5.09	5.60	5.92

(Source; FEMA's Flood Frequency Analysis)

Westside Intensity Equations

$$I_1 = \frac{22.94}{(T_c + 26.034)^{0.8924}} \quad (4\text{-}10)$$

$$I_2 = \frac{31.78}{(T_c + 16.944)^{0.8670}} \quad (4\text{-}11)$$

$$I_5 = \frac{47.71}{(T_c + 18.323)^{0.8750}} \quad (4\text{-}12)$$

$$I_{10} = \frac{59.91}{(T_c + 18.202)^{0.8791}} \quad (4\text{-}13)$$

$$I_{25} = \frac{83.70}{(T_c + 20.638)^{0.8931}} \quad (4\text{-}14)$$

$$I_{50} = \frac{105.87}{(T_c + 22.270)^{0.9025}} \quad (4\text{-}15)$$

$$I_{100} = \frac{140.07}{(T_c + 26.090)^{0.9189}} \quad (4\text{-}16)$$

$$I_{250} = \frac{200.41}{(T_c + 31.034)^{0.9398}} \quad (4\text{-}17)$$

$$I_{500} = \frac{269.76}{(T_c + 35.887)^{0.9599}} \quad (4\text{-}18)$$

Eastside Region

Table 4-3: Eastside Duration Depth Frequency

Return Frequency	Total Rainfall Depth (inches) by Duration 1 hr	2 hr	3 hr	4 hr	6 hr	12 hr	24 hr
1	0.35	0.45	0.49	0.52	0.56	0.61	0.69
2	0.64	0.80	0.87	0.91	0.98	1.08	1.23
5	0.97	1.22	1.30	1.36	1.46	1.61	1.83
10	1.22	1.54	1.64	1.72	1.84	2.02	2.29
25	1.61	2.05	2.17	2.28	2.42	2.67	2.98
50	1.96	2.5	2.64	2.79	2.95	3.25	3.59
100	2.38	3.05	3.21	3.41	3.58	3.94	4.30
250	3.04	3.92	4.12	4.40	4.59	5.05	5.42
500	3.65	4.73	4.97	5.34	5.52	6.08	6.42

(Source; FEMA's Flood Frequency Analysis)

Eastside Intensity Equations

$$I_1 = \frac{19.37}{(T_c + 27.833)^{0.8950}} \qquad 4\text{-}19$$

$$I_2 = \frac{28.58}{(T_c + 18.000)^{0.8696}} \qquad 4\text{-}20$$

$$I_5 = \frac{44.63}{(T_c + 17.907)^{0.8768}} \qquad 4\text{-}21$$

$$I_{10} = \frac{57.38}{(T_c + 18.202)^{0.8803}} \qquad 4\text{-}22$$

$$I_{25} = \frac{83.76}{(T_c + 21.297)^{0.8956}} \qquad 4\text{-}23$$

$$I_{50} = \frac{111.89}{(T_c + 24.562)^{0.9095}} \qquad 4\text{-}24$$

$$I_{100} = \frac{144.20}{(T_c + 25.944)^{0.9190}} \qquad 4\text{-}25$$

$$I_{250} = \frac{202.11}{(T_c + 28.777)^{0.9327}} \qquad 4\text{-}26$$

$$I_{500} = \frac{293.32}{(T_c + 35.887)^{0.9602}} \qquad 4\text{-}27$$

Where:

> I_{xx}= *Rainfall Intensity for a given frequency, in inches per hour.*
>
> T_c = *Computed Time of Concentration, in minutes.*

4.3.1.3. Time of Concentration

Time of Concentration is computed using several methods depending on type. Please refer to Table 4-4. The minimum recommended Time of Concentration is 10 minutes.

Table 4-4: Time of Concentration

Type of Water Way	Time of Concentration Method
Shallow Concentrated Flow: Arroyos (USGS Blue Steam)	Kirpich Formula
Unconcentrated Flow (commonly areas upstream [approximately 500 feet] of USGS Blue Stream and sheet flow)	Upland Method utilizing Velocity Charts
Concentrated flows (commonly channels and streets)	Upland Method utilizing Manning's Equation for Velocity Computation

Kirpich Formula

The Kirpich Formula is used generally for waterways defined in USGS topographic maps as perennial, intermittent, and disappearing streams. USGS topographic maps show these streams as blue or light blue continuous lines. The formula is shown below:

$$T_c = 0.0078\left(\frac{L^{0.77}}{S^{0.385}}\right) \qquad \text{4-28}$$

Where:

> T_c = *Time for drainage to arrive at concentration point from the uppermost reach of the basin, in minutes.*
>
> L = *The longest length for drainage to arrive at concentration point from the uppermost reach of the basin, in feet.*
>
> S = *Average slope of the waterway gradient, in foot per foot.*

For overland flow on concrete or asphalt surfaces, the time of concentration developed using the Kirpich Formula can be used if it is multiplied by a factor of 0.4. For concrete channels, multiply by 0.2. No adjustment is needed for flows over bare soil or in roadside ditches.

Upland Method

The Upland Method is used for waterways of unconcentrated flows, commonly known as sheet flow. These flows are generally considered a mode of drainage prior to becoming concentrated in waterways such as an arroyo or stream. The maximum flow length is 500 feet. Common land covers for Upland Method are meadows, fallow cultivation, short grass pasture, bare ground, and paved areas. Figure 4-5 relates the gradient slope of the waterway and type of cover to obtain velocity. The general Time of Concentration equation is as follows:

$$T_c = \frac{L}{V(60)} \tag{4-29}$$

Where:

T_c = Time for drainage to arrive at concentration point from the uppermost reach of the basin, in minutes.

L = The longest length for drainage to arrive at concentration point from the uppermost reach of the basin, in feet.

V = Average velocity of the waterway gradient, in feet/second obtained from Figure 4-5 (based on Figure 15-2, National Engineering Handbook Section 4)

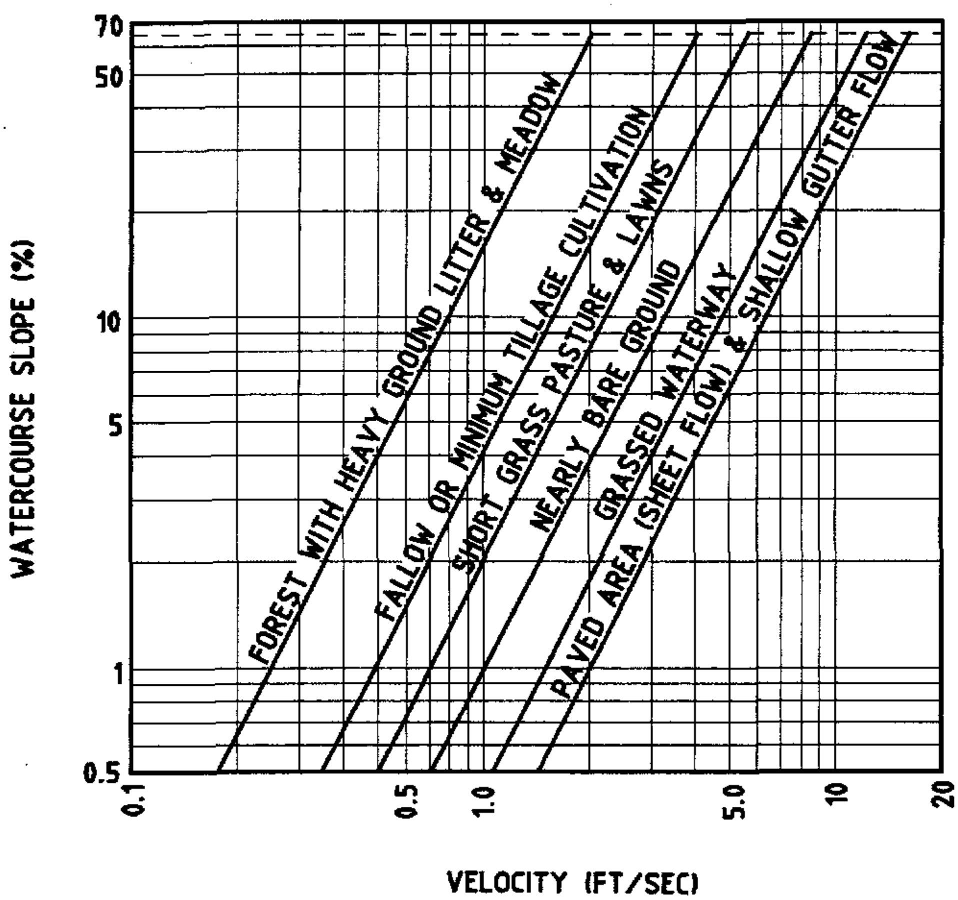


Figure 4-5: Average Velocity

An alternate way to compute the velocity component of the Upland Method is to apply Manning's formula. This equation relates the longitudinal slope of the section, wetted perimeter, and roughness coefficient for velocity.

$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad 4\text{-}30$$

Where:

V = Average velocity for section for the entire drainage waterway, in feet per second.

R = Hydraulic radius = Flow Area divided by Wetted Perimeter, in square foot per foot.

S = Average slope of the waterway gradient, in foot per foot.

Use the following procedure for estimating Time of Concentration:

1. Divide the flow path into reach lengths along which flow conditions remain reasonably consistent. Characterize the runoff along a travel path as either overland (or sheet) flow, shallow concentrated flow, or concentrated channel.
2. For each identified reach length, estimate the travel time using a method that is appropriate for the flow conditions.
3. Determine the total time by adding the individual travel times to determine the total time.
4. Select the path that results in the longest time, the recommended minimum Time of Concentration is 10 minutes.

4.3.2. Rational Method

The use of the Rational Method to compute peak discharge is confined to small basins. It can be applied to either undeveloped or developed lands. This method is a function of the size of the basin area, time for drainage to traverse the longest route to the concentration point (Time of Concentration), and land cover.

The Rational Method formula is presented below:

$$Q_{xx} = CI_{xx}A \qquad 4\text{-}31$$

Where:

Q = Peak discharge for a given frequency, in cubic feet per second.

C = Rational coefficient dependent on land cover and frequency.

I = Intensity for the given frequency, in inch per hour.

A = Computed area of the basin, in acres

The following procedure outlines the Rational Method for estimating peak discharge:

1. Determine the Time of Concentration with consideration for future characteristics of the watershed.

2. Choose and apply the rainfall Intensity-Duration-Frequency (IDF) equation according to the frequency.
3. Select or develop appropriate runoff coefficients for the watershed. Where the watershed comprises more than one characteristic, you must estimate C values for each area segment individually.
4. Apply the appropriate return period multiplier to the runoff coefficient.
5. Calculate the peak discharge for the watershed for the desired frequency using the Rational Method Equation.

4.3.2.1. Runoff Coefficient

The El Paso area is unique in natural, urban, and rural landscapes, creating a vast array of surface roughness that rainfall must saturate before drainage collection. Below, in Table 4-5 are tabulated "C" coefficients related to the surface land use. The coefficient is a function of land cover and land use.

Table 4-5: Rational Method Developed Condition Coefficient (C)

*** The engineer may submit a "weighted" average coefficient on a case by case basis subject to the approval of the City Engineer.**

LAND USE	Runoff Coefficient			
	2-,5-, and 10-Year	*25-Year*	50-Year	100-Year
Rural Residential	0.42	0.46	0.50	0.53
Single Family Residential	0.48	0.53	0.58	0.60
Multi Family Apartment	0.75	0.83	0.90	0.94
General Commercial	0.85	0.94	0.95	0.95
* Educational	0.75	0.83	0.90	0.94
* Public Facilities	0.85	0.94	0.95	0.95
General Transportation	0.95	0.95	0.95	0.95
Pavement and Rooftops	0.95	0.95	0.95	0.95
Gravel Vehicular Travel Lanes & Shoulders	0.70	0.77	0.84	0.88
Golf Courses & Cemeteries	0.25	0.28	0.30	0.31
General Open Space	0.40	0.44	0.48	0.50
Passive Open Space (Includes mountain preserves and washes)	0.55	0.61	0.66	0.69
Mountain Terrain (High topographic relief, slopes >10%)	0.80	0.88	0.95	0.95
Agriculture Flat (0-2% Slope)	0.34	0.39	0.43	0.47
Desert	0.10	0.18	0.25	0.33

LAND USE	Runoff Coefficient			
	2-,5-, and 10-Year	*25-Year*	50-Year	100-Year
Landscaping with Impervious Under Treatment	0.85	0.94	0.95	0.95
Landscaping without Impervious Under Treatment	0.40	0.44	0.48	0.50
Alluvial Fan Areas	Rational Method not recommended			

*** The engineer may submit a "weighted" average coefficient on a case by case basis subject to the approval of the City Engineer.**

For a watershed comprising more than one characteristic, the composite C value is a weighted average dependent upon individual area segments. The composite C value is calculated by the following equation:

$$C = \frac{\sum C_n A_n}{\sum A_n} \qquad \text{4-32}$$

Where:

C = Weighted runoff coefficient.

n = nth sub-area.

Cn = Runoff coefficient for nth sub-area.

An = nth sub-area size, in acres.

Example: Determine the weighted C coefficient for unimproved area

Existing conditions (unimproved):

Land Use	Area, ha (ac)	Runoff Coefficient, C
Unimproved Grass	8.95 (22.1)	0.25
Grass	8.6 (21.2)	0.22
	Total = 17.55 (43.3)	

Solution:

Determine Weighted C for existing (unimproved) conditions:

$$\text{Weighted C} = \sum \frac{C_x A_x}{A} = \frac{[(22.1 \times 0.25) + (21.2 \times 0.22)]}{43.3} = 0.235 \qquad \text{4-33}$$

4.3.3. USGS Regression Equations

4.3.3.1. Regional Regression Equations

The following regional regression equation applies to rural, uncontrolled watersheds within the El Paso area and is used to compare validity of results obtained from using the Unit Hydrograph Method. This equation was developed by the Texas Department of Transportation (TXDOT). El Paso County is located well within Region 2 of the State of Texas. Table 4-6 provides coefficient values necessary for the use of the regression equations.

$$QT = aA^{b}SH^{c}SL^{d} \qquad 4\text{-}34$$

Where:

QT = T-year discharge, in cubic feet per second.

A = Contributing drainage area, in square miles.

SH = Basin-shape factor defined as the ratio of main channel length squared to contributing drainage area, unitless.

SL = Mean channel slope defined as the ratio of headwater elevation of the longest channel minus main channel elevation at site to main channel length, unitless.

a, b, c, d = Multiple linear regression coefficients dependent on region number and frequency.

Table 4-6: Regression Coefficients and Limits

Freq. (yrs)	a	b	c	d	Wt Error %	Limits	
2	826	0.376	0.869	-0.689	120	A lower:	0.32
5	6500	0.372	0.738	-0.933	92	A upper:	4305
10	18100	0.369	0.673	-1.050	88	SH lower:	0.51
25	55300	0.366	0.604	-1.190	92	SH upper:	14.8
50	108000	0.363	0.566	-1.270	99	SL lower:	9.67
100	199000	0.361	0.531	-1.340	107	SL upper:	130

4.3.3.2. Nationwide Equations

The Nationwide Urban Regression Equations are based on multiple regression analyses of urban flood frequency data from 199 urbanized basins. The following seven-parameter equations are valid for urbanized areas that do not contain peak controlling structures and should not be used if any of the seven variables are larger or smaller than those used in the original regression study. No other applicability restrictions exist for use of these equations. The equations are as follows:

$$UQ_2 = 2.35A^{0.41}SL^{0.17}(RI2+3)^{2.04}(ST+8)^{-0.65}(13-BDF)^{-0.32}IA^{0.15}RQ2^{0.47} \qquad 4\text{-}35$$

$$UQ_5 = 2.70A^{0.35}SL^{0.16}(RI2+3)^{1.86}(ST+8)^{-0.65}(13-BDF)^{-0.31}IA^{0.11}RQ5^{0.54} \qquad 4\text{-}36$$

$$UQ_{10} = 2.99A^{0.32}SL^{0.15}(RI2+3)^{1.75}(ST+8)^{-0.57}(13-BDF)^{-0.30}IA^{0.09}RQ10^{0.58} \qquad 4\text{-}37$$

$$UQ_{25} = 2.78A^{0.31}SL^{0.15}(RI2+3)^{1.76}(ST+8)^{-0.55}(13-BDF)^{-0.29}IA^{0.07}RQ25^{0.60} \quad 4\text{-}38$$

$$UQ_{50} = 2.67A^{0.29}SL^{0.15}(RI2+3)^{1.74}(ST+8)^{-0.53}(13-BDF)^{-0.28}IA^{0.06}RQ50^{0.62} \quad 4\text{-}39$$

$$UQ_{100} = 2.50A^{0.29}SL^{0.15}(RI2+3)^{1.76}(ST+8)^{-0.52}(13-BDF)^{-0.28}IA^{0.06}RQ100^{0.63} \quad 4\text{-}40$$

$$UQ_{500} = 2.27A^{0.29}SL^{0.16}(RI2+3)^{1.86}(ST+8)^{-0.54}(13-BDF)^{-0.27}IA^{0.05}RQ500^{0.63} \quad 4\text{-}41$$

Where:

UQ_T = Urban T-year Peak Discharge, in cubic feet per second.

A = Drainage Area, in square miles.

SL = Main Channel Slope, in feet per mile.

RI2 = Rainfall for the 2-hour, 2-year recurrence interval, in inches.

ST = Basin Storage, in percent.

BDF = Basin Development Factor, unitless.

IA = Impervious Surfaces, in percent.

RQT = Peak discharges for an equivalent rural drainage basin in the same hydrologic area as the urban basin for a recurrence interval of T years, in cubic feet per second

Source: United State Geological Survey-National Flood Frequency Program Version 3, Updated 2002, Chapter ***"Urban Flood - Frequency Estimating Techniques"*** by. V.B. Sauer

4.3.4. Unit Hydrograph Methods

4.3.4.1. Runoff Curve Number

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. Runoff Curve Number is a function of the hydrologic soil group and land cover condition. TXDOT general definitions for the four hydrologic soil groups are described in Table 4-7.

Table 4-7: Hydrologic Soil Groups

Soil Group	Description
Group A	Group A soils have a low runoff potential due to high infiltration rates even when saturated (0.30 in/hr to 0.45 in/hr). These soils primarily consist of deep sands, deep loess, and aggregated silts.

Soil Group	Description
Group B	Group B soils have a moderately low runoff potential due to moderate infiltration rates when saturated (0.15 in/hr to 0.30 in/hr). These soils primarily consist of moderately deep to deep, and moderately well to well drained soils with moderately fine to moderately coarse textures (shallow loess, sandy loam).
Group C	Group C soils have a moderately high runoff potential due to slow infiltration rates (0.05 in/hr to 0.5 in/hr). These soils primarily consist of soils in which a layer near the surface impedes the downward movement of water or soils with moderately fine to fine texture, such as clay loams, shallow sandy loams, soils low in organic content, and soils usually high in clay.
Group D	Group D soils have a high runoff potential due to very slow infiltration rates (less than 0.05 in./hr if saturated). These soils primarily consist of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material, such as soils that swell significantly when wet or heavy plastic clays or certain saline soils.

Effects of Urbanization

Consider the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the selected soil group.

The hydrologic soil group is available online for the entire El Paso area on the Natural Resources Conservation Service (NCRS) Web Soil Survey. Once the hydrologic soil group and cover type is determined, the Runoff Curve Number (CN) can be estimated using the following tables.

Table 4-8: Runoff Curve Numbers for Arid and Semi Arid Rangelands[1]

Cover description		Curve number for hydrologic soil group			
Cover Type	Hydrologic Condition[2]	A[3]	B	C	D
Herbaceous--mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen--mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	71
	Fair		48	57	63
	Good		30	41	48
Piñon, juniper, or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub – major plants include salt brush, greasewood, creosote-bush, black brush, bursage, palo verde mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

Notes:
[1] Average runoff condition, Ia = 0.2S. For range in humid regions use Table 4-11
[2] Poor is < 30% ground cover (litter, grass, and brush overstory)
Fair is 30% to 70% ground cover
Good is >70% ground cover
[3]Curve numbers for Group A have been developed only for desert shrub.

Table 4-9: Runoff Curve Numbers for Urban Areas[1]

Cover Type and Hydrologic Conditions	Percent developed	Curve number for hydrologic soil group A	B	C	D
Fully developed urban areas (vegetation established)					
Open Space: (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious Areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved: curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94

Notes:

[1] Average runoff condition, $Ia = 0.2S$. For range in humid regions use Table 4-11

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows:
impervious areas are directly connected to the drainage system,
impervious areas have a CN of 98,
and pervious areas are considered equivalent to open space in good hydrologic condition.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

[4] Composite CNs for natural desert landscaping should be computed based on the impervious area percentage (CN=98) and the precious area CN. The pervious area CNs are assumed equivalent to desert shrub in poor hydrologic condition.

Table 4-10: Runoff Curve Numbers for Cultivated Agricultural Land[1]

Cover Type	Treatment[2]	Hydrologic Condition[3]	A	B	C	D
Fallow	Bare soil		77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	68	85	89
	SR+CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C+CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR+CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C+CR	Poor	62	73	81	84
		Good	60	72	80	93
	C&T	Poor	61	72	79	82
		Good	58	70	78	81
	C&T+CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast Legumes or Rotation Meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

Notes:

[1]Average runoff condition, and Ia=0.2S.

[2]Crop residue cover applies only if residue is on at least 5 percent of the surface throughout the year.

[3]Hydrologic condition is based on a combination of factors affecting infiltration and runoff: including, (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legume in rotations, (d) percent of residue cover on land surface (good > 20 percent), and (e) degree of roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better infiltration and tend to decrease runoff.

Table 4-11: Runoff Curve Numbers for Other Agriculture Land

Cover Type	Hydrologic Condition	A	B	C	D
Pasture, grassland, or range-continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow - continuous grass, protected from grazing and generally mowed for hay		30	58	71	78
Brush - brush-weeds-grass mixture, with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30[4]	48	65	73
Woods - grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods[6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[4]	55	70	77
Farmsteads - Buildings, lanes, driveways and surrounding lots		59	74	82	86

[1]Average runoff condition

[2]Poor: <50% ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: >75% ground cover and lightly or only occasionally grazed.

[3]Poor: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

[4]Actual curve number is less than 30; use CN = 30 for runoff computations.

[5]CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

[6]Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

The above tables include CN values for numerous of urban land uses. For urban land use the CN is based on a specific percentage of imperviousness. For example, the CN values for commercial land use are based on an imperviousness of 85 percent. Curve numbers for other percentages of imperviousness can be computed using a weighted CN approach, with a CN of 98 used for the impervious areas and the CN for open space (good condition) used for the pervious portion of the area. Thus CN values of 39, 61, 74, and 80 are used for hydrologic soil groups A, B, C, and D, respectively. These are the same CN values for pasture in good condition. Thus, the following equation can be used to compute a weighted CN:

$$CN_w = CN_p(1-f) + f(98) \quad \text{4-42}$$

in which f is the fraction (not percentage) of imperviousness. To show the use of Equation 4-42,

the CN values for commercial land use with 85 percent imperviousness are:

A soil: 39(0.15) + 98(0.85) = 89
B soil: 61(0.15) + 98(0.85) = 92
C soil: 74(0.15) + 98(0.85) = 94
D soil: 80(0.15) + 98(0.85) = 95

For a commercial land area with 60 percent imperviousness of a B soil, the composite CN would be:

$$CN_w = 61(0.4) + 98(0.60) = 83$$

The same values can be obtained from the figure below:

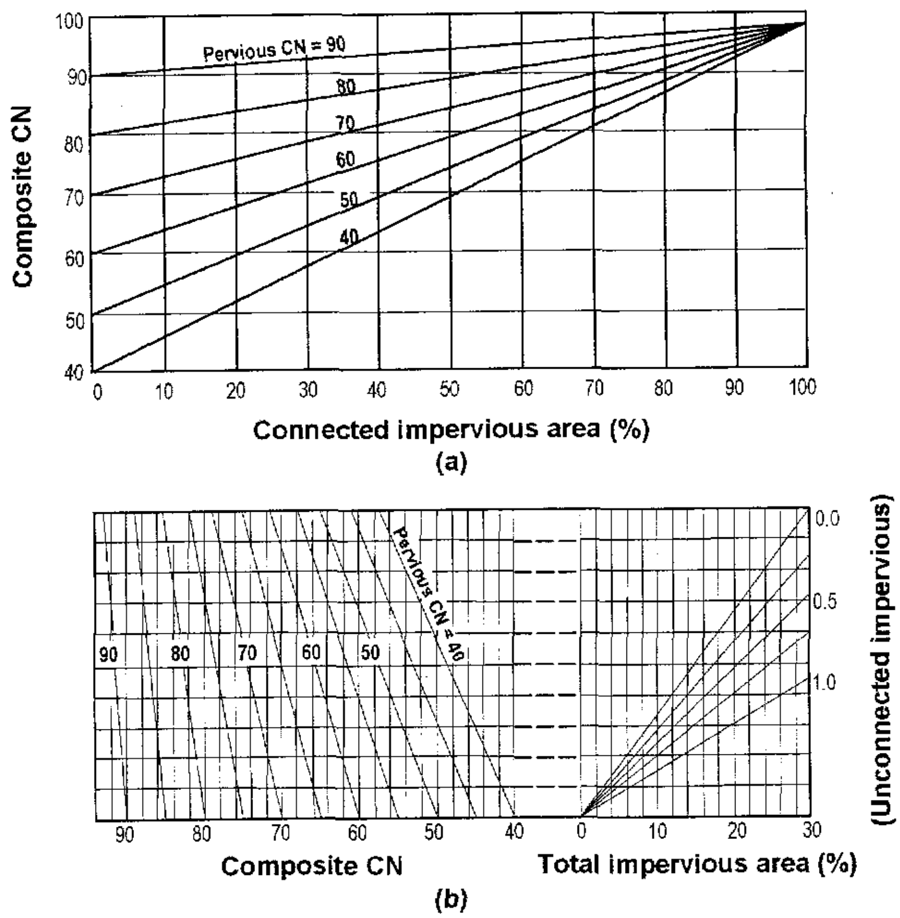


Figure 4-6: Composite Curve Number Estimation

Antecedent Moisture Condition

Rainfall infiltration losses depend primarily on soil characteristics and land use (surface cover). The NRCS method uses a combination of soil conditions and land use to assign runoff factors known as runoff curve numbers. These represent the runoff potential of an area when the soil is not frozen. The higher the CN, the higher the runoff potential. The following tables provide an extensive list of suggested runoff curve numbers. The CN values assume medium antecedent moisture conditions (CN II). If necessary, adjust the Runoff CN for wet or dry antecedent moisture conditions. Use a five-day period as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period. Equation 4-43 adjusts values for expected dry soil conditions (CN I). Use Equation 4-44 to accommodate wet soils (CN III). For help determining which moisture condition applies, see the table titled Rainfall Groups for Antecedent Soil Moisture Conditions during Growing and Dormant Seasons.

$$CN(I) = \frac{4.2CN(II)}{10 - 0.058CN(II)} \qquad \text{4-43}$$

$$CN(III) = \frac{23CN(II)}{10 + 0.13CN(II)} \qquad \text{4-44}$$

Table 4-12: Rainfall Groups for Antecedent Soil Moisture Conditions during Growing and Dormant Seasons

Antecedent Moisture Condition	Description	Growing Season 5-Day Antecedent Rainfall	Dormant Season 5-Day Antecedent Rainfall
Dry AMC I	An optimum condition of watershed soils, where soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes pace.	Less than 1.4 in. or 35 mm.	Less than 0.05 in or 12 mm.

4.3.4.2. Lag Time

SCS Equation

The most commonly used equation for lag time is the SCS equation. This equation may be used when computing the unit hydrograph.

$$T_{LAG} = L^{0.8}\frac{(S+1)^{0.7}}{1900\sqrt{Y}} \qquad 4\text{-}45$$

Where:

T_{LAG} = Lag time, in hours.

L = Hydraulic length of watershed, in feet.

S = Maximum retention in the watershed, in inches as defined by:

$$S = \frac{1000}{CN} - 10 \qquad 4\text{-}46$$

Where:

Y = Watershed slope, in percent.

CN = SCS curve number for the watershed as defined by the loss method.

Snyder's Equation

Snyder developed an equation, which has been widely used, to determine the lag time for a watershed. Snyder's study was based on data from the Appalachian Mountain region for large natural watersheds.

$$t_L = C_t(L_{ca}L)^{0.3} \qquad 4\text{-}47$$

Where:

t_L = Lag Time, in hours.

L_{ca} = Distance along the main stream from the base to a point nearest the center of gravity of the basin, in miles.

L = Length of main stream channel from the base outlet to the upstream end of the stream and including the additional distance to the watershed divide, in miles.

C_t = Coefficient representing variations of types and locations of streams.

Typical values for Snyder's coefficient are provided in the following table:

Table 4-13: Snyder's Stream Variance Coefficients

Location	Range of C_t	Average C_t
Southern California	-	0.4
Central Texas	0.4-2.3	1.1
Sewered Urban Areas	0.2-0.5	0.3

Location	Range of C_t	Average C_t
Mountainous Watersheds	-	1.2
Foothill Areas	-	0.7
Valley Areas	-	0.4
SW Desert	0.7-1.9	1.4

4.3.4.3. SCS (NRCS) Unit Hydrograph Method

For drainage basins larger than 200 acres, the unit hydrograph method must be used for peak discharge calculations. A project specific hydrograph is a plot of discharge versus time. For a particular drainage basin, the project specific hydrograph is an adjustment of the unit hydrograph. The area beneath the hydrograph curve is equal to the volume of direct runoff of the entire drainage basin. Figure 4-7 and Tc = *Time of Concentration* Tr = Duration of Rainfall Excess

Tp = Time of Peak Tb = Base Time

Figure 4-8 shows a dimensionless unit hydrograph and its associated cumulative mass curve. The NRCS unit hydrograph was developed through the analysis of a large number of natural unit hydrographs from a broad cross section of geographic locations and hydrologic regions.

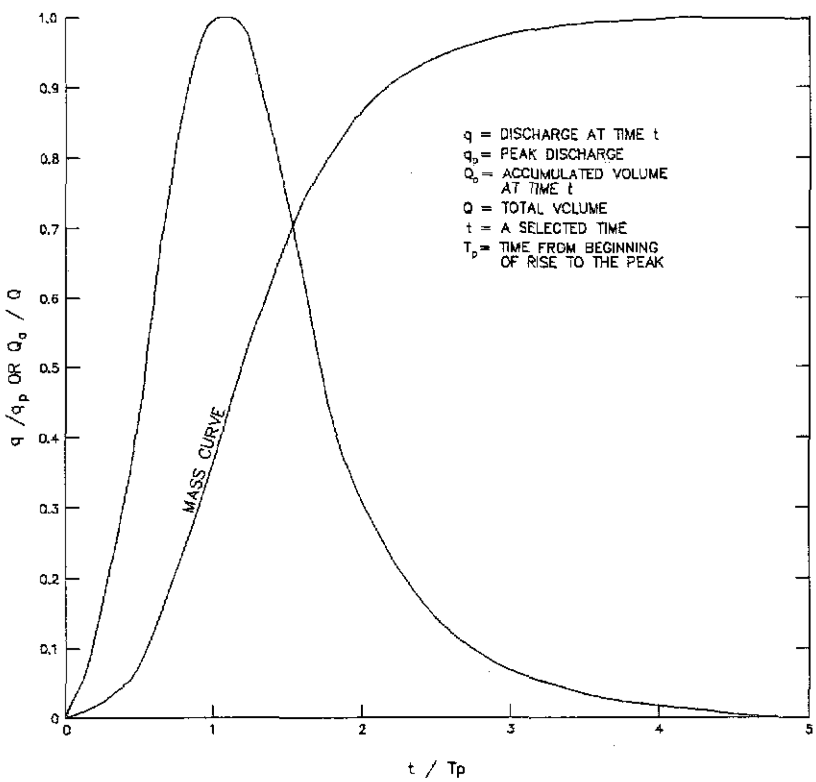
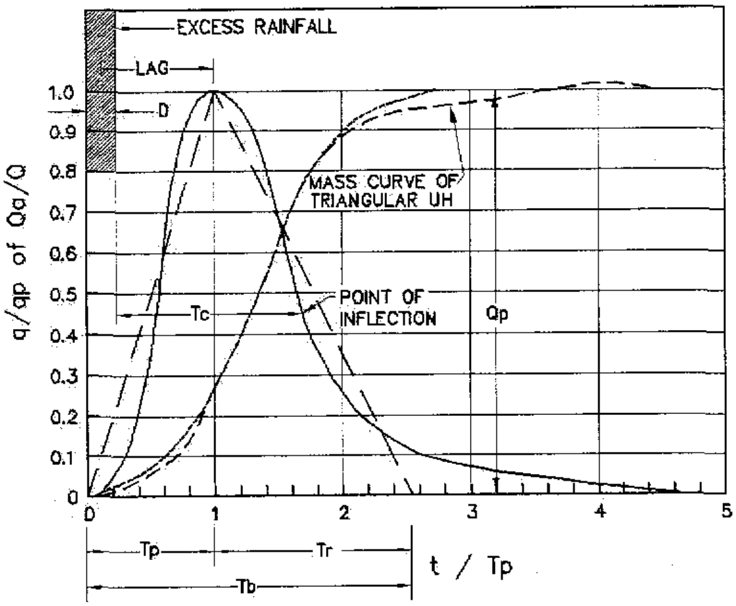


Figure 4-7: NRCS Dimensionless Unit Hydrograph and Mass Curve



Tc = Time of Concentration

Tp = Time of Peak

Tr = Duration of Rainfall Excess

Tb = Base Time

Figure 4-8: Dimensionless Curvilinear Unit Hydrograph

Manual construction of unit hydrograph can be tedious, and designers are encouraged to use computer models capable of computing an NRCS synthetic unit hydrograph. Computer models acceptable to the City of El Paso include:

HEC-HMS: United States Army Corp of Engineers (USACE) Hydrologic Engineering Center Version 3.1.0

HEC-1: USACE Flood Hydrograph Package, 1981

TR-20: United States Department of Agriculture (USDA) Soil Conservation Service, SCS 1983

TR-55: NRCS Technical Release 55

These computation programs make addition and routing of multiple hydrographs a relatively easy task. In addition, these programs have the capability of computing runoff hydrographs from several drainage basins, adding hydrographs together and estimating travel time for hydrographs once they have entered defined conveyance channels. Different rainfall distributions and total rainfall depths may be used in the model. All of the computer models require the same basic input data to compute an NRCS synthetic unit hydrograph, including:

Drainage Basin Area

Time of Concentration or Lag Time

Runoff Curve Number

Total Rainfall Depth

Rainfall Distribution

The precise input format for each of these elements varies with each computer program used. Guidelines for the use of a particular computer model are beyond the scope of this manual. Drainage designers wishing to use computer models for hydrologic analysis are encouraged to obtain license copies of the software.

SCS (NRCS) Type II-75 Rainfall Distribution

NRCS dimensionless rainfall distributions for Texas are shown to be Type II. However, discussions with NRCS representatives indicate that a Type II-75 distribution is more applicable to the El Paso Area. Adjusted Type II distributions were developed for the state of New Mexico based on rainfall patterns.

Figure 4-9 displays Type II-75 unit hydrograph and mass curve rainfall distribution.

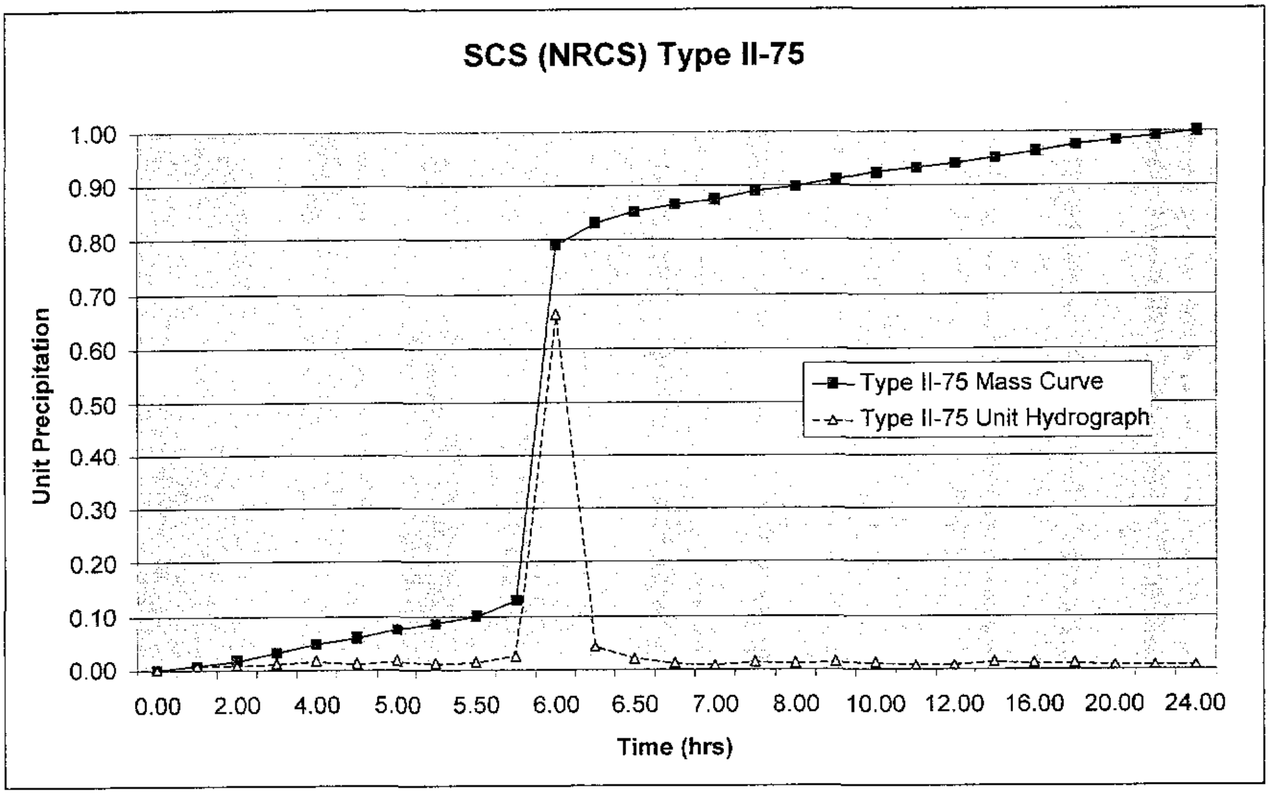


Figure 4-9: SCS (NCRS) Type II-75 Unit Hydrograph and Mass Curve

Table 4-14 provides a tabular format of **Figure 4-9**.

Table 4-14: SCS (NRCS) 24-Hour Type II-75 Rainfall Distribution

Time, t (hours)	Fraction of 24-hour Rainfall
0.00	0.0000
1.00	0.0080
2.00	0.0177
3.00	0.0301
4.00	0.0475
4.50	0.0591
5.00	0.0754
5.25	0.0863
5.50	0.1016
5.75	0.1280
6.00	0.7895
6.25	0.8323
6.50	0.8516
6.75	0.8644
7.00	0.8740
7.50	0.8875
8.00	0.8977
9.00	0.9122

Time, t (hours)	Fraction of 24-hour Rainfall
10.00	0.9230
11.00	0.9318
12.00	0.9392
14.00	0.9524
16.00	0.9640
18.00	0.9743
20.00	0.9836
22.00	0.9922
24.00	1.0000

4.3.4.4. Snyder's Method

A methodology commonly employed by the USACE and many others in El Paso is the Snyder Method. A unit hydrograph is developed by computing seven data points based on lag time, time base, unit hydrograph duration, peak discharge, and hydrograph time widths at 50 and 75 percent of peak flow.

Time Base

The time base of a Snyder's method synthetic unit hydrograph can be computed using the following equation. This equation provides reasonable estimates for large watersheds, but has a tendency to overestimate smaller areas. It is suggested to use three to five times the time to peak as the base time for smaller watersheds.

$$t_b = 3 + \frac{t_l}{8} \qquad 4\text{-}48$$

Where:

t_b = *Base time, in days.*

t_l = *Lag time, in hours.*

Duration

The duration of rainfall excess for Snyder's synthetic unit hydrograph method is a function of lag time. Since changes in lag time occur with changes in duration of the unit hydrograph, the following equation was developed to allow for these adjustments.

$$t_{lR} = t_l + 0.25(t_R - t_r) \qquad 4\text{-}49$$

Where:

t_{lR} = *Adjusted lag time, in hours.*

t_l = *Original lag time, in hours.*

t_R = *Desired unit hydrograph duration, in hours.*

t_r = *Original unit hydrograph duration, in hours.*

Peak Discharge

Snyder's method utilizes the assumption that if a duration rainfall will produce one inch of direct runoff, then the outflow volume is a relative constant percentage of the inflow volume. Therefore, the equation for peak discharge can be written as follows:

$$Q_p = \frac{640 C_p A}{t_{lR}} \qquad 4\text{-}50$$

Where:

Q_p = *Peak discharge, in cubic feet per second.*

C_p = *Coefficient of peak discharges.*

A = *Watershed area, in square miles.*

t_{lR} = *Adjusted lag time, in hours.*

Values of C_p range from 0.4 to 0.8 and generally indicate retention or storage capacity of the watershed. Larger values of C_p are generally associated with smaller values of C_t. Typical values are tabulated below.

Table 4-15: Snyder's Peak Discharge Coefficient Variance

Location	Range of C_p	Average C_P
Sewered Urban Areas	0.1-0.6	0.3
West Texas	0.4-0.94	0.61

Hydrograph Construction

Using the equations for lag time, base time, duration, and peak discharge, plot three points for the unit hydrograph, remembering the total direct runoff amounts to one inch. The USACE gives additional assistance in plotting time widths for points on the hydrograph located at 50 and 75 percent of peak discharges. As a general rule of thumb, the time width at W_{50} and W_{75} ordinates should be proportioned on each side of the peak in a ratio of 1:2 with the shorter time side on the left of the synthetic unit hydrograph peak. As noted earlier, for smaller areas the base time should be adjusted by similar factors.

$$W_{50} = \frac{830}{(Q_p/A)^{1.1}} \qquad 4\text{-}51$$

$$W_{75} = \frac{470}{(Q_p/A)^{1.1}} \qquad 4\text{-}52$$

The seven points formed through the use of these equations can be plotted and a smooth curve drawn. To assure a unit hydrograph, the curve shape and ordinates should be adjusted until the area beneath the curve is equivalent to one unit of direct runoff depth over the watershed area.

4.3.4.5. Hydrograph Routing

Hydrograph routing is a procedure by which a hydrograph at any downstream point is determined from a known hydrograph at some upstream point. As a flood hydrograph moves down a channel, its shape is modified due to flow resistance along the channel boundaries and the storage of water in the channel and floodplain. An example of inflow and outflow hydrographs from a stream reach is provided in Figure 4-10 and Figure 4-11. Note that the hydrograph is attenuated and translated as it moves downstream.

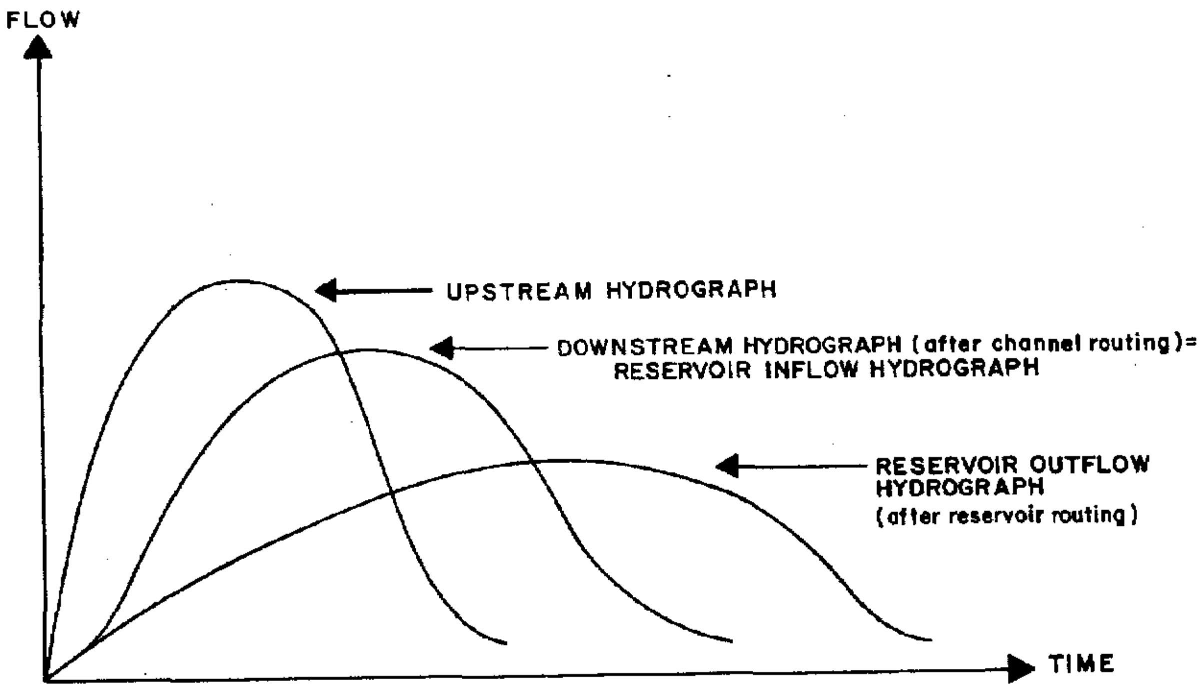


Figure 4-10: Effects of Hydrograph Routing to Defined Hydrograph

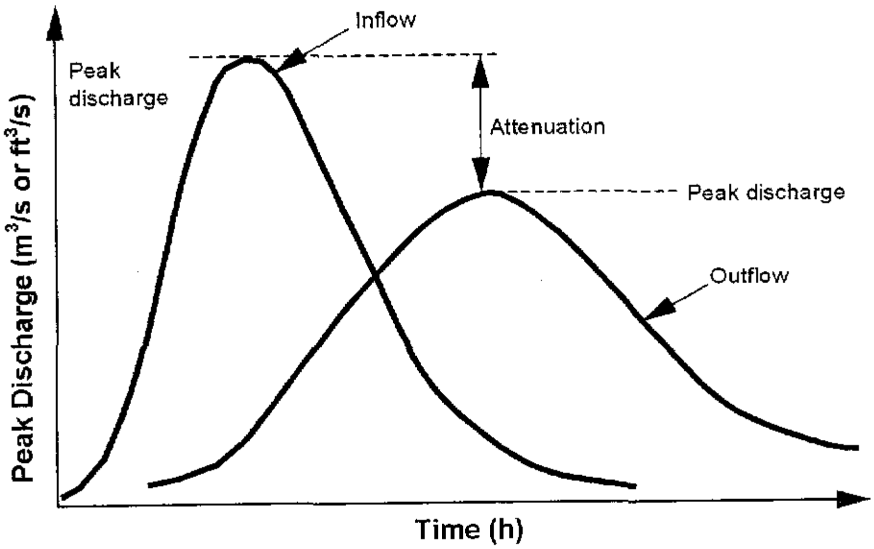


Figure 4-11: Inflow and Outflow Hydrographs from a Stream Reach

The general equation for hydrograph routing is based on continuity and represents an accounting of all flow within a reach. Mathematically, the continuity of mass can be written in terms of storage as:

$$\frac{dS}{dt} = I - O \qquad \text{4-53}$$

Where;

I = inflow, in cubic feet per second.

O = outflow, in cubic feet per second.

t = time, in seconds.

S = channel storage, in feet per second.

Clearly, the continuity of mass equation does not account for lateral or tributary inflow. A number of techniques are available for routing hydrographs through channels. Four commonly used methods are Muskingum, kinematic wave, Muskingum-Cunge, and the modified Att-Kin method. The method to choose for a given reach depends on the amount and type of data available, as well as the nature of the hydrograph to be routed. The most widely used method is the Muskingum, which is presented below.

The Muskingum routing method is based on two equations; the continuity equation and a relationship of the storage, inflow, and outflow of the reach shown below.

$$\frac{I1 + I2}{2} \times \Delta t - \frac{O1 + O2}{2} \times \Delta t = S2 - S1 \qquad \text{4-54}$$

$$S = K\{X1 + (1 - X)O\} \quad \text{4-55}$$

Where:

I1 and I2 = Inflow discharges at time 1 and time 2, in cubic feet per second.

O1 and O2 = Outflow discharges at time 1 and time 2, in cubic feet per second.

Δt = Time difference between time 1 and time 2, in seconds.

S1 and S2= Values of reach storage at time 1 and time 2, in cubic feet.

S = Reach storage, in cubic feet.

I = Inflow discharge, in cubic feet per second.

O = Outflow discharge, in cubic feet per second.

K = Storage constant, in seconds.

Combining and simplification of the two equation yields:

$$O2 = C1 \times I1 + C2 \times I2 + C3 \times O1 \quad \text{4-56}$$

Where:

$$C0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \quad \text{4-57}$$

$$C1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \quad \text{4-58}$$

$$C2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \quad \text{4-59}$$

$$C0 + C1 + C2 = 1 \quad \text{4-60}$$

The development of equations to estimate K and X is a trial and error process following these procedures:

1. For each point in time, compute the storage S2 by rearranging Equation 4-54.

$$S2 = S1 + \Delta t\left(\frac{I1 + I2}{2} - \frac{O1 + O2}{2}\right) \quad \text{4-61}$$

S1= Is assumed to be Zero for the initial condition

2. Using the value of X, compute [XI+(1-X)O] for each point in time.
3. Plot the computed storage S from Step 1 versus [XI+(1-X)O] from Step 2 for each point in time. This will result in a closed loop.
4. Revise the value of X and repeat Steps 1 to 3 until the plot shows a minimum amount of deviation from a straight line drawn through the center of the loop.
5. Use the slope of the line as the best estimate of K and the value of X that produced the minimum deviation line in Step 4 as the estimate of X.

When data is not available, K is estimated to be the average travel time through the reach. The discharge used in determining a value for K is the average discharge for the hydrograph. Using Manning's equation to derive an expression for wave speed (celerity)=dQ/dA and assuming a wide channel, then celerity, $c=\beta V$, where V=flow velocity and $\beta=5/3$. It must be noted that the value of X must be between 0 and 0.5.

Muskingum Method Example: Consider a river reach as shown in Figure 4-12. A hydrograph developed at point A is to be routed along the 15,750 ft reach of river. What effect will this hydrograph routing have on the peak discharge experienced at the roadway at point B?

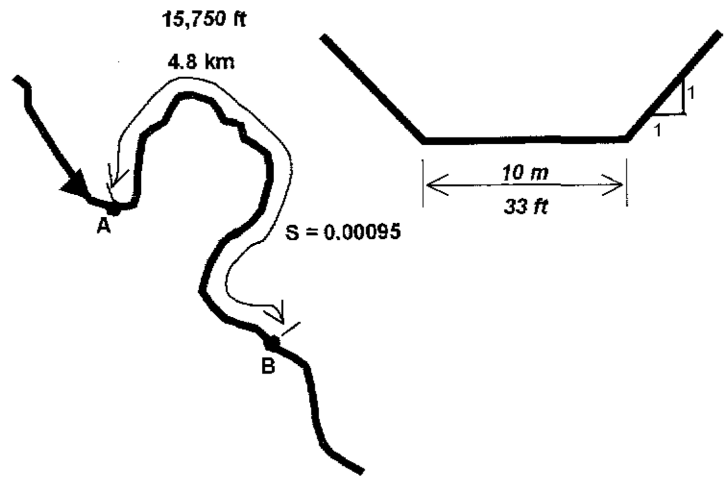


Figure 4-12: Muskingum Routing Example Problem Schematic

A synthetic hydrograph is developed at Point A using the procedures presented in Section 4 for a 25-year design discharge. The upstream hydrograph, which is used as the inflow, is given in Table 4-16. The peak discharge is 3,090 ft^3/s. It is assumed that the routing coefficients are constant and based on the reference discharge for this hydrograph at 1,200 ft^3/s.

Time (h)	Inflow (m^3/s)	Muskingum Outflow (m^3/s)	Kinematic Wave Outflow (m^3/s)	Muskingum-Cunge Outflow (m^3/s)	Modified Att-Kin Outflow (m^3/s)
0.0	0	0.0	0.0	0.0	0.0
0.5	7	1.4	0.0	0.4	0.0
1.0	13	6.5	6.2	6.2	4.3
1.5	23	13.0	11.9	12.4	9.6
2.0	32	21.9	21.7	21.7	17.8
2.5	49	32.4	30.3	31.2	26.4
3.0	68	47.9	46.6	47.0	40.2
3.5	76	63.7	66.1	64.9	57.1
4.0	84	74.0	74.9	74.6	68.6
4.5	78	80.0	83.8	82.1	78.0
5.0	71	77.3	78.8	78.3	78.0
5.5	60	70.8	72.2	71.6	73.7
6.0	52	61.7	61.3	61.5	65.4
6.5	46	53.7	53.0	53.3	57.2
7.0	40	47.1	46.8	46.9	50.4
7.5	36	41.4	40.7	40.9	44.1
8.0	32	36.8	36.6	36.6	39.2
8.5	28	32.7	32.5	32.6	34.8
9.0	24	28.6	28.5	28.5	30.7
9.5	20	24.6	24.5	24.5	26.6
10.0	16	20.6	20.5	20.5	22.6
10.5	13	16.8	16.5	16.6	18.6
11.0	11	13.7	13.4	13.5	15.2
11.5	7	11.0	11.4	11.2	12.6
12.0	6	8.0	7.3	7.6	9.2
12.5	3	6.0	6.3	6.1	7.3
13.0	0	3.3	3.4	3.4	4.7
13.5	0	1.0	0.2	0.6	1.8
14.0	0	0.3	0.0	0.1	0.7
14.5	0	0.1	0.0	0.0	0.3
15.0	0	0.0	0.0	0.0	0.1
15.5	0	0.0	0.0	0.0	0.0
16.0	0	0.0	0.0	0.0	0.0
16.5	0	0.0	0.0	0.0	0.0
17.0	0	0.0	0.0	0.0	0.0

Table 4-16: Muskingum Routing Example Problem Inflow Hydrograph

The given data of the trapezoidal cross-section of the reach can be calculated for the discharge:

depth = 6.6 feet

cross-sectional area = 261 ft^2

average velocity = 4.6 fps

wave velocity (celerity) = (5/3) V = 7.7 fps wave travel time = K=15,750 feet / [7.7 fps (3,600 s/h)] = 0.57 hour.

For the Muskingum method, the coefficients C0, C1, and C2 are first computed from equations below using $\Delta t = 0.5$ hour and assumed values of X = 0.2 and K = 0.57 hour as follows:

$$C0 = \frac{-0.57(0.2) + 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.193$$

$$C1 = \frac{0.57(0.2) + 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.516$$

$$C2 = \frac{0.57 - 0.57(0.2) - 0.5(0.5)}{0.57 - 0.57(0.2) + 0.5(0.5)} = 0.291$$

The outflow hydrograph ordinates can now be computed with t=0.5 hours.

$$O2 = C0 \times I2 + C1 \times I1 + C2 \times O1$$

=0.193(247) + 0.516(0) + 0.291 (0) = 48 cubic feet

At t=1

$O2$=0.193(459)+0.516(247)+0.291(48)=230 cubic feet per second

The results for this example problem are provided in Table 4-17.

Time (h)	Inflow (ft^3/s)	Muskingum Outflow (ft^3/s)
0.0	0	0
0.5	247	48
1.0	459	230
1.5	812	461
2.0	1,130	772
2.5	1,730	1142
3.0	2,401	1690
3.5	2,684	2250
4.0	2,966	2614
4.5	2,754	2825
5.0	2,507	2730
5.5	2,119	2500
6.0	1,836	2178
6.5	1,624	1897
7.0	1,412	1664
7.5	1,271	1460
8.0	1130	1300
8.5	989	1154
9.0	847	1011
9.5	706	868
10.0	565	727
10.5	459	592
11.0	388	485
11.5	247	389
12.0	212	282
12.5	106	212
13.0	0	117
13.5	0	34
14.0	0	10
14.5	0	3
15.0	0	1
15.5	0	0
16.0	0	0
16.5	0	0
17.0	0	0

Table 4-17: Muskingum Routing Example Problem Results

5. Pavement Drainage

This chapter describes the methodology that should be used for hydraulic design of pavement drainage.

5.1. Surface Drainage

A secondary use of the street network is the conveyance of storm runoff. This secondary use must always be subsidiary to the primary function of streets as the safe conveyance of people and vehicles. The goals of street hydraulic design are therefore:

- To provide an economical means of transporting storm runoff.
- To ensure that the safety and convenience of the public are preserved.
- To prevent storm runoff, once collected by the street system, from leaving the street right-of-way except at specially designated locations.

5.1.1. Hydroplaning

Hydroplaning conditions can develop for relatively low vehicle speeds and at low rainfall intensities for storms that frequently occur each year. The following are the factors of hydroplaning

- Vehicle speed.
- Tire conditions (pressure and tire tread).
- Pavement type.
- Roadway geometrics (pavement width, cross slope, grade).
- Pavement conditions (rutting, depressions, roughness).

Speed appears as a significant factor in the occurrence of hydroplaning, therefore, it is considered to be the driver's responsibility to exercise prudence and caution when driving during wet conditions. In many respects, hydroplaning conditions are analogous to ice or snow on the roadway.

Designers do not have control over all of the factors involved in hydroplaning. However, many remedial measures can be included in development of a project to reduce hydroplaning potential. The following is provided as guidance for the designer as practical measures to consider in accordance with the AASHTO *Policy on Geometric Design of Highways*:

Pavement Sheet Flow

- Maximize transverse slope.
- Maximize pavement roughness.

Gutter Flow

- Limit street spread (adequate inlet spacing).
- Maximize interception of gutter flow on the high side of superelevation transitions.

Sag Areas

- Limit pond duration and depth.

Overtopping

- Limit depth and duration of overtopping flow.

In the event that suitable measures cannot be implemented to address an area of high potential for hydroplaning or an identified existing problem area, consideration should be given to installing advance warning signs.

5.1.2. Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement, since it is susceptible to storm water spread. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge. Gutter grades should not be less than 0.5 percent. Minimum grades can be maintained in very flat terrain by use of a sawtooth profile. Minimum and maximum street grades and horizontal curves will conform to standards set forth in the City of El Paso Design Standards for Construction (DSC).

5.1.3. Cross (Transverse) Slope

The Design Standards for Construction (DSC) establishes the cross slopes for highways. Median areas should not be drained across traveled lanes. A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas. A uniform transverse slope of 2% is assumed throughout the roadway cross section.

Shoulders should generally be sloped to drain away from the pavement.

5.1.4. Curb and Gutter

Curb and gutter shall be installed according to the provisions of both the City of El Paso Subdivision Regulations and Design Standards for Construction (DSC).

Curbs are normally used at the outside edge of pavements for low-speed, highway facilities, and in some instances adjacent to shoulders on moderate to high-speed facilities. They serve to:

- Contain the surface runoff within the roadway and away from adjacent properties.
- Prevent erosion on fill slopes.
- Provide pavement delineation.
- Enable the orderly development of property adjacent to the roadway.

A curb and gutter combination forms a triangular channel that can convey runoff equal to or less than the design flow without interruption of the traffic. When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders and portions of the traveled surface.

5.1.5. Roadside Channel

For streets without curb and gutter, roadside ditches are recommended for control of surface drainage from the pavement and surrounding area. An open-channel ditch, cut into the natural terrain along the roadside, is the most economical method to produce a drainage channel. The most desirable channel, from the standpoint of hydraulic efficiency, is one with steep sides.

However, limitations on soil stability and roadside safety (clear zone principles) require flatter slopes. The effect of slope combinations on the safety of a vehicle traversing them is an important consideration. Slope combinations for channels can be selected to produce a cross-section that can be safely traversed by an errant vehicle. Additionally, the cost of right-of-way needs to be considered when selecting combinations of slopes for the roadside.

The depth of the roadside channel must be sufficient to remove surface water without saturation of the subgrade including the pavement base. The depth of water that can be tolerated in the channel, particularly on flat channel slopes, depends on the soil characteristics. Designers are encouraged to refer to the AASHTO *Roadside Design Guide* for clear zone requirements based on vehicle design speed.

The minimum desirable grade for a roadside channel is based on the drainage velocities needed to avoid sedimentation. The maximum desirable grade for roadside channels is based on a tolerable velocity for vegetation and shear on soil types. Refer to the Federal Highway Administration (FHWA) *Design of Roadside Channels with Flexible Linings HEC 15, 3rd Edition.* The channel grade does not have to follow the grade of the adjacent roadbed, particularly if the roadbed is flat. Not only can the depth and width of the channel be varied to meet different quantities of runoff, slopes of channel, types of lining, and the distance between discharge points, but the lateral distance between the channel and the edge of traveled way can also be varied. Care should be taken to avoid abrupt changes in the roadway section that produces a discontinuity of the roadside environment and violates driver expectancy. Care should also be taken to avoid major breaks in channel grade that could cause unnecessary scour or silt deposition.

5.2. Flow in Gutters

When estimating the total capacity of a roadway (curb to curb or sidewalk to sidewalk), Manning's equation as expressed in Equation (5-1) shall be used:

$$Q = A\left(\frac{1.49}{n}\right)R_H^{\ 0.67}S^{0.5} \qquad 5\text{-}1$$

Where:

Q = *Total flow, in cubic feet per second.*

n = *Manning's roughness coefficient. An n value of 0.016 is typically used for paved streets unless special conditions exist.*

A = *Flow area, in square feet.*

R_h = *Hydraulic radius, in feet.*

S = *Slope of energy grade line, assumed equal to longitudinal street slope, in feet per feet.*

Procedure:

1. Obtain the appropriate details for curb and gutter and cross slope requirements from both the City of El Paso Design Standards for Construction (DSC) and Subdivision Regulations.

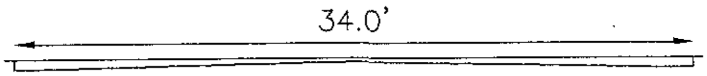
2. Either use the aid of CADD program to draw the section to obtain the area and the wetted perimeter or develop an equation.

3. Calculate the hydraulic radius (A/P)

4. Calculate the Q allowable from Manning's equation in respect to the longitudinal street slope.

Example:

Local Street Minimum Width of 34 feet face of curb to face of curb with 6 inch curbs.



Area= 12.19 feet squared
Wetted Perimeter= 35.00 feet
Hydraulic Radius= .35

$$Q = \frac{8.99\ S^{0.5}}{n}$$

To determine the parabolic crown, the general formula is:

$$C = \left(\frac{W^2}{160(W-10)}\right) \qquad 5\text{-}2$$

Where:

C = Crown, in feet.

W = Width, in feet of the parabolic section.

It is possible to get approximately the same results obtained with the above quadratic equation by using a linear equation. The deviation will not be more than 0.01 feet over the entire range of street widths. The use of this linear equation is acceptable and may be used instead of the above equation in determining the crown value. This formula is:

$$C = 0.006W + 0.10 \qquad 5\text{-}3$$

Example:

For a width (W) of 36 feet, the value of the crown (C) is 0.006(36) + 0.10 = 0.316 feet or approximately 0.32 feet. The value of the radius curve that approximates the parabolic curvature is determined by the formula R = 20W - 200, and the radius that may be used for laying out the parabolic curve for the above example would be R = 20(36) - 200 = 520 feet.

5.2.1. Capacity Relationships

Street hydraulic design criteria are as follows:

- Weighted Manning's roughness coefficient for a street section consisting of concrete gutter and asphalt pavement is 0.016.
- Conjugate and/or sequent depth in the event of the 100-year design discharge shall be contained within the street right-of-way.

- Flow depths in the event of the 10-year design discharge may not exceed 0.5 feet in any collector or arterial street. One lane free of flowing or standing water in each traffic direction must be preserved on arterial streets.
- The product of depth times velocity shall not exceed 6.5 at any location on any street (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet).
- Inverted crown streets are allowed for residential streets; however, an alley gutter is required at the inverted crown. The gutter at the curb is either not required or shall match the cross slope of the road.

For streets with more than two driving lanes in each direction:

- The product of depth times velocity may not exceed 6.5 at any location on any street (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet).
- Inverted crown streets are prohibited.
- The street capacity shall be calculated based on the actual street cross section. It should not be assumed that the street has equal flow distribution.

The following is the generalized capacity rating curves for individual street types. It is encouraged, however, that capacity calculations be performed specific to the project. The cross sections are obtained from the City of El Paso Subdivision Design Standards for Construction (DSC) and are based on a 2% cross (transverse) slope.

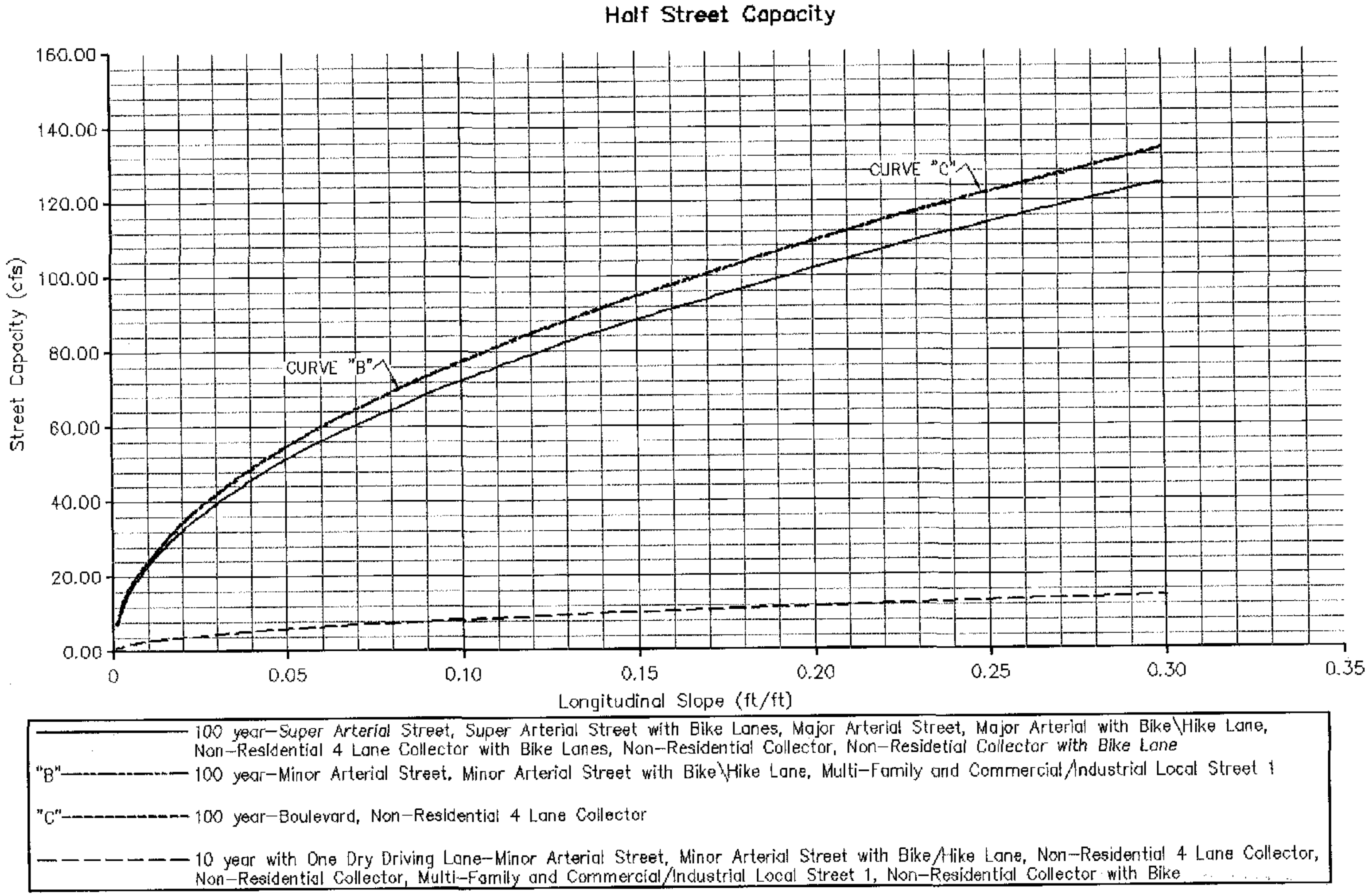


Figure 5-1: Arterial and Major Street Conveyance Capacity Curves

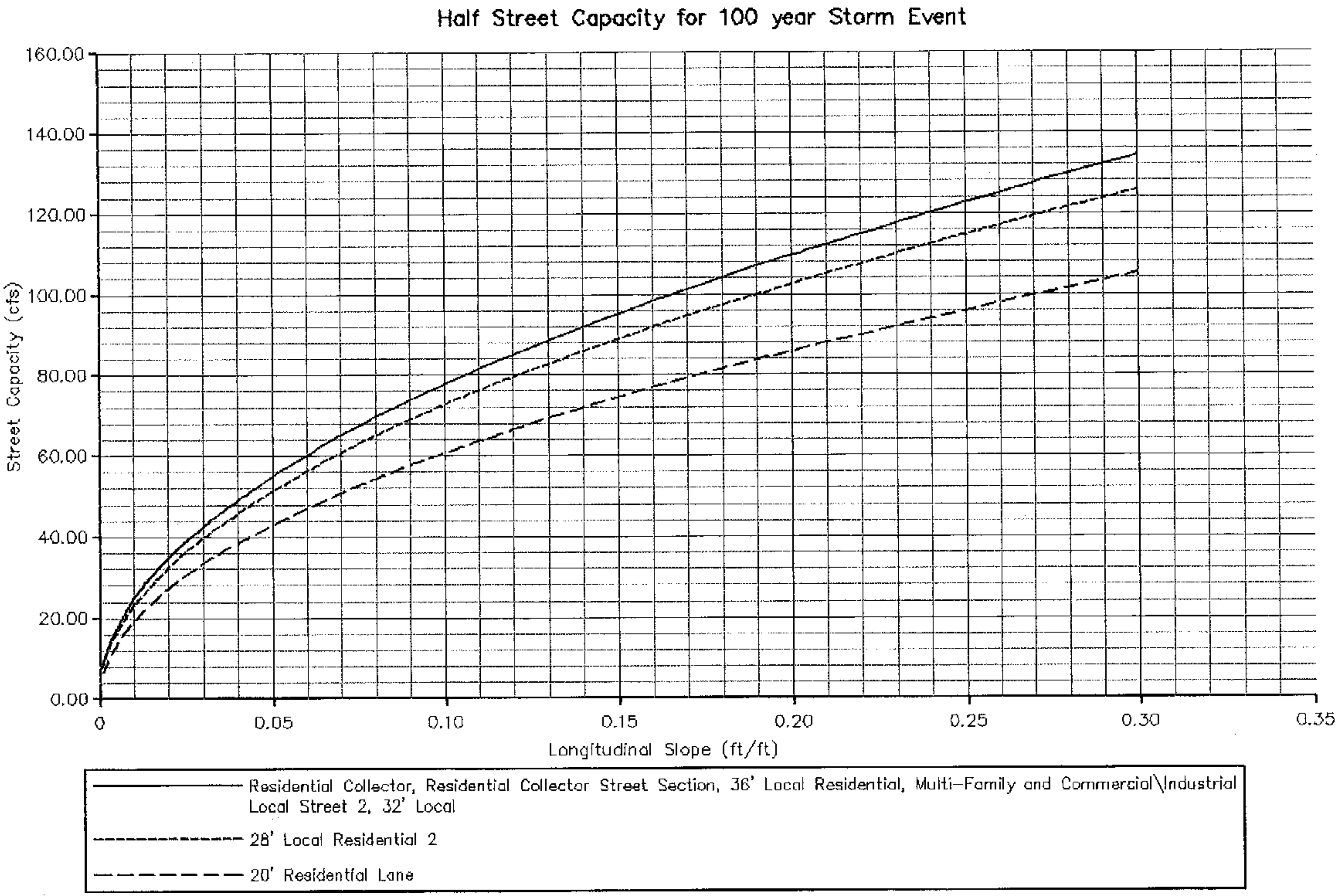


Figure 5-2: Residential Street Conveyance Capacity Curves

5.2.2. Hydraulic Jump

The procedure presented in the Capacity Relationships Section (Section 5.2.1) must be used in the hydraulic design of streets. T-intersections, radical slope changes, and intersections are potential locations for hydraulic jumps when upstream slopes are steeper than critical slope. The height of jump should not exceed curb height and shall be contained within the street right-of-way. To determine the length of hydraulic jump, refer to Section 12.1 Hydraulic Jump.

When conditions indicate that a hydraulic jump or that the effects of superelevation will allow runoff to exceed street hydraulic design criteria, provisions must be made for treatment of the problem. An example of acceptable provisions is to modify curb height to contain the jump. The warping of street sections and the construction of deflector walls for these purposes is prohibited.

Intersections and other radical changes in street cross section and slope require special consideration whenever the flow depth/street slope relationship results in flows occurring in the supercritical flow regime. In general, the slope at which supercritical to subcritical occurs is at 0.60%.

6 Storm Drain Systems

This chapter describes the methodology that should be used for the hydraulic design of a storm drain. When dealing with conflicts with existing utilities, the designer must use professional judgment. If the designer needs to deviate from the requirements herein in order to minimize utility conflicts, the Storm Water Utility and City Engineer should be contacted immediately for approval.

6.1 Hydraulics of Storm Drainage Systems

Closed conduit sections (pipe, box, or arch sections) will be designed as flowing full and, whenever possible, under pressure, except when the following conditions exist:

(A) In some areas of high sediment potential, there is a possibility of stoppage occurring in drains, resulting in storm drain failure. In situations where sediment may be expected, the City Engineer must be consulted for a determination of the appropriate bulking factor.

(B) In certain situations, open channel sections upstream of the proposed closed conduit may be adversely affected by backwater.

If the proposed conduit is to be designed for pressure conditions, the hydraulic grade line shall be one foot below ground or street surface.

6.1.1 Hydraulic Grade Line

Most procedures for calculating hydraulic grade line profiles are based on the Bernoulli equation, a derivation of the energy equation with the addition of minor head loss. This equation can be expressed as follows and is represented graphically in Figure 6-1:

$$\frac{V_1^2}{2g} + D_1 + S_o L = \frac{V_2^2}{2g} + D_2 + S_f L + h_{minor} \qquad 6\text{-}1$$

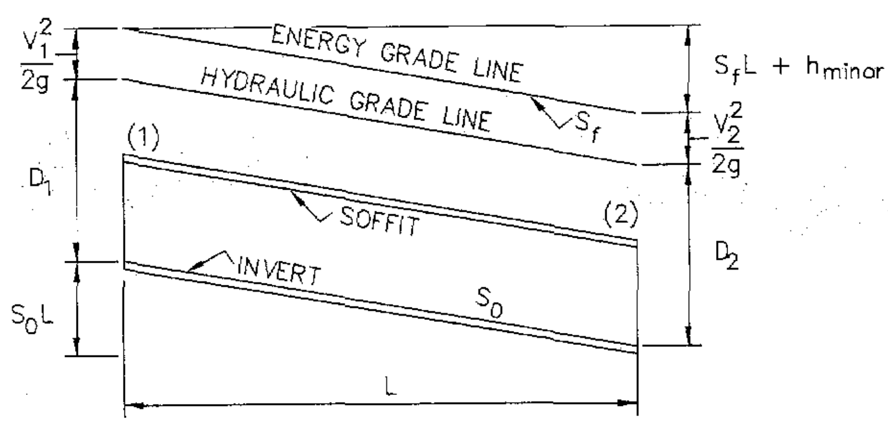


Figure 6-1: Hydraulic Grade Line Calculation

Where:

V = Average velocity (Q/A) at Sections 1 or 2, in feet per second.

g = Gravitational acceleration, 32.2 feet per second squared.

D = Vertical distance from invert to Hydraulic Grade Line (HGL) at Sections 1 or 2, in feet.

S_o *= Invert slope, unitless.*

L = Horizontal projected length of conduit, in feet.

S_f *= Average friction slope between Sections 1 and 2, unitless.*

h_{minor} *= Minor head losses, in feet (refer to subsequent sections).*

Minor losses have been included in the Bernoulli equation because of their importance in calculating hydraulic grade line profiles and are assumed to be uniformly distributed.

When specific energy (E) is substituted for the quantity ($V^2/2g + D$) in the above equation and the result rearranged, use:

$$L = \frac{E_2 - E_1 + h_{\min or}}{S_o - S_f} \qquad 6\text{-}2$$

The above is a simplification of a more complex equation and is convenient for locating the approximate point where pressure flow may become unsealed.

Table 6-1 provides an example hydraulic grade line computational spreadsheet recommended for use when computing the HGL by hand methods.

Table 6-1: Example Hydraulic Grade Line Computation Sheet

Project Example				Reach 1											Pipe Loss								Structure Losses								
Structure	Invert	From	To	Length L (ft)	Diam Pipe (in)	Area (ft^2)	Perimeter (ft)	R	Q in (cfs)	Vel (fps)	n	HGL dn	Hydraulic Slope S_f (ft/ft)	Friction Loss H_f (ft)	H_b	H_c	H_e	H_j	Total Pipe Loss	HGL up @ downstream structure	d_{aho}	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V^2/2g)$	HGL up @ upstream structure	Grate Elevation

Pipe Loss

H_l = Sf * L Pipe Friction Losses
h_b = Bend Losses
H_c = Transition Losses contraction
H_o = Transition Losses expansion
H_j = Junction Losses

Structure Loss

$K=K_oC_DC_dC_QC_pC_B$
K_o = Initial head loss coefficient hased on relative access hole size
C_D = Correction factor for pipe diameter (pressure Flow only)
d_{aho} = water depth in access hole above the outlet pipe invert
C_d = Correction factor for flow depth

C_Q = Correction factor for relative Flow
C_p = Correction factor for plunging Flow
C_B = Correction factor for benching (refer to 5-5)
V_o = Velocity of outlet pipe

$$S_f = \left[\frac{nQ}{1.486\ AR^{2/3}}\right]^2 \quad h_b = K_b\left[\frac{V^2}{2g}\right] \quad K_b = 0.20\sqrt{\frac{\Delta}{90°}} \quad h_{m.h.} = 0.05\left[\frac{V^2}{2g}\right] \quad K_o = 0.1\left(\frac{b}{D_o}\right)(1-\sin\theta)+1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta$$

$$H_f = LS_f \quad h_e = 0.2\left(\frac{v_1^2}{2g}-\frac{v_2^2}{2g}\right) \quad hj = \frac{V_2^2}{2g}-\frac{V_1^2}{2g}-\frac{2A_3}{A_2}*\frac{v_3^2}{2g}*\cos\theta \quad C_D = \left(\frac{D_o}{D_i}\right)^3 \quad C_Q = (1-2\sin\theta)\left(1-\frac{Q_i}{Q_o}\right)^{0.75}+1$$

$$h_c = 0.1\left(\frac{v_1^2}{2g}-\frac{v_2^2}{2g}\right) \quad C_d = 0.5\left(\frac{d_{aho}}{D_o}\right)^{0.6} \quad K = K_oC_DC_dC_QC_pC_B \quad C_p = 1+0.2\left(\frac{h}{D_o}\right)\left(\frac{h-d_{aho}}{D_o}\right) \quad HGL_{up} = K\frac{V^2}{2g}+Total\_Pipe\_Loss+HGL_{down}$$

HGL Example Problem:

In the storm drain layout, illustrated in figure 6-2, the system discharges into a retention basin. The captured discharge is given, all piping is reinforced concrete pipe (RCP) with a Manning's n value of 0.013, and the minimum design pipe diameter is 18 inches for maintenance purposes. Calculate the HGL profile from Structure 40 to 44.

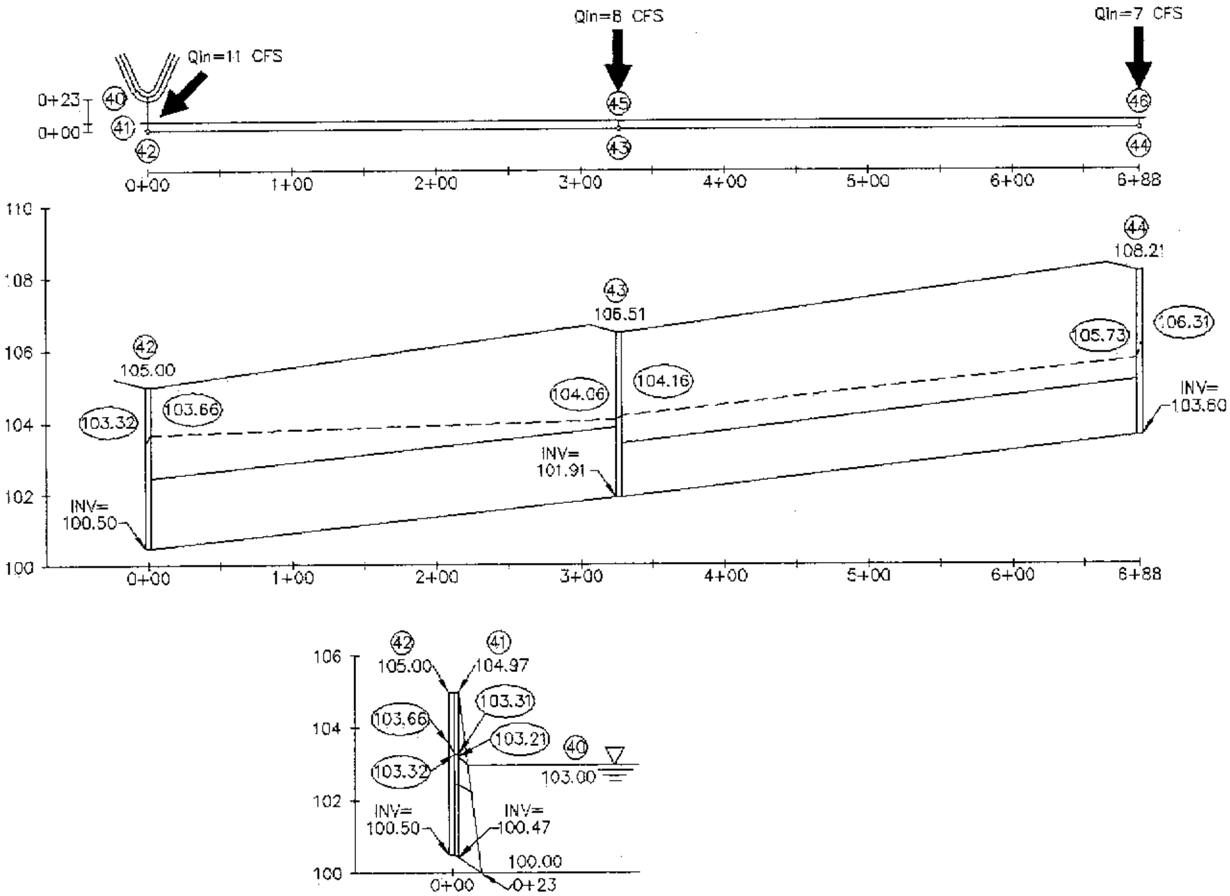


Figure 6-2: Hydraulic Grade Line Example

Table 6-2: Example Problem HGL Computation Sheet

Project Example			Reach 1												Pipe Loss								Structure Losses								
Structure	Invert	From	To	Length L (ft)	Diam Pipe (in)	Area	Perimeter	R	Q in	Vel (fps)	n	HGL dn	Hydraulic Slope S_f (ft/ft)	Friction Loss H_f	H_b	H_c	H_e	H_j	Total Pipe Loss	HGL up @ downstre	d_{aho}	K_o	C_D	C_d	C_Q	C_P	C_B	K	$K(V^2/2g)$	HGL up @ upstream structure	Grate Elevation
41	100.47	40	41	16	24	3.14	6.28	0.50	26	8.3	0.013	103.00	0.013	0.21					0.21	103.21	2.74	0.10	1	0.60	1.52	1	1	0.09	0.10	103.31	104.97
42	100.50	41	42	3.5	24	3.14	6.28	0.50	15	4.8	0.013	103.31	0.004	0.02					0.02	103.32	2.82	1.55	1	0.61	1.00	1	1	0.96	0.34	103.66	105.00
43	101.91	42	43	322	24	3.14	6.28	0.50	8	2.5	0.013	103.66	0.001	0.40					0.40	104.06	2.15	2.23	8	0.34	0.17	1	1	1.02	0.10	104.16	106.51
44	103.60	43	44	356	18	1.77	4.71	0.38	7	4.0	0.013	104.16	0.004	1.56					1.56	105.73	2.13	2.23	19	0.34	0.17	1	1	2.40	0.58	106.31	108.21
46	103.71	44	46	3.5	18	1.77	4.71	0.38	7	4.0	0.013	106.31	0.004	0.02					0.02	106.33	2.62	0.00	0	0.59	1.83	1	1	0.00	0.00	106.33	108.21
45	102.01	43	45	3.5	18	1.77	4.71	0.38	8	4.5	0.013	104.16	0.006	0.02					0.02	104.18	2.17	0.00	0	0.53	1.83	1	1	0.00	0.00	104.18	106.51

Table 6-3 - Example Problem Structure Information

structure	θ	Manhole Diameter b	Outgoing pipe diameter Do	Inflowing pipe diamter Di	Flow in the inflow pipe Qi	Flow in the outflow pipe Qo
41	180	2	2	2	15	26
42	90	4	2	2	15	15
43	90	4	2	1.5	7	15
44	90	4	1.5	1.5	7	7
46	0	2	0	1.5	7	7
45	0	2	0	1.5	7	7

1. Determine tailwater elevation, which is the surface elevation of the retention pond at 103.00 feet.
2. Determine each structure's minimum elevation. Insert elevations into Invert Column.
3. Determine pipe segment length. Calculate area, perimeter, and hydraulic radius of the pipe.
4. Determine velocity and the hydraulic slope.
5. For each pipe segment, determine losses (refer to subsequent sections).
6. For each structure, determine losses (refer to subsequent sections).
7. Calculate the HGL at the exit of the structure, total pipe loss plus tailwater elevation
8. Calculate the HGL at the entrance of the structure, total pipe loss plus tailwater elevation and structure loss.
9. Continue for each structure and pipe segment.

6.1.2. Storm Drain Outfalls and Inlets

6.1.2.1 Outfalls

A conduit to be designed for pressure conditions may discharge into one of the following:

A body of water such as a detention reservoir.

A natural watercourse or arroyo.

An open channel, either improved or unimproved.

Another closed conduit.

The water surface elevation at the point of discharge is commonly referred to as the control and is generally located at the downstream end of the conduit for pressure flow. If flow becomes unsealed, the control is at the point where the flow becomes unsealed.

Two general types of controls are possible: Control elevation above the soffit or below the soffit.

When the control elevation is above the soffit, the control must conform to the following criteria.

In the case of a conduit discharging into a detention/retention facility, the control is the design water surface elevation for that type of facility, as described in Chapter 11 – Storm Water Ponding.

In the case of a conduit discharging into an open channel, the control is the water surface elevation of the channel.

In the case of a conduit discharging into another conduit, the control is the design hydraulic grade line elevation of the outlet conduit at the confluence.

When the control elevation is at or below the soffit elevation, the control is the normal depth of the point of discharge. This condition may occur in any one of the three situations described above.

(A) Where a storm drain discharges into the detention reservoir, the designer should follow the City criteria for location and type of structure used.

(B) When a storm drain outlets into a natural channel or open area, an outlet structure will be provided in order to prevent erosion.

(C) Velocity of flow at the outlet should match as closely as possible with the existing channel velocity. Fencing and a protection barrier will be provided where deemed necessary by the City Engineer.

   (1) When the discharge velocity is low or sub-critical, the outlet structure will consist of a headwall, wingwalls, and an apron. The apron may consist of a concrete slab, grouted rock, concrete slope blankets, projecting end, or well designed dumped riprap depending on conditions.

   (2) When the discharge velocity is high or supercritical per the HEC-RAS modeling, the designer will design bank protection in the vicinity of the outlet and an energy dissipater structure.

6.1.2.2. Inlets

An inlet structure will be provided where the storm drain is receiving flow from a channel or open collection area. The structure should generally consist of a headwall, wingwalls to protect the adjacent banks from erosion, and a paved inlet apron. Other options include a concrete slope blanket or projecting end blanket. Depending on the intake velocity, additional erosion protection may be required beyond the inlet apron.

The apron bottom should be limited to a maximum of 2H:1V slope. Wall heights should extend at least to the height of the headwater upstream of the inlet, and be adequate to protect both the fill over the drain and the embankments. Headwall and wingwall fencing and a protection barrier to prevent public entry will be provided.

If trash and debris are prevalent, then a debris barrier will be included in the design.

6.1.3 Energy Losses

As captured drainage progresses through the storm drain network, it encounters a variety of hydraulic structures, such as manholes, bends, contractions, enlargements, and transitions, which all contribute to velocity head loss. The largest of the energy losses is the friction loss, known as the major loss, and all others are considered minor losses. The total velocity loss is the cumulative of all the major and minor losses presented.

6.1.3.1 Pipe Friction Losses

Friction losses for closed conduits carrying storm water, including pump station discharge lines, will be calculated from the Manning equation or a derivation thereof. The empirical version of the Manning equation is commonly expressed as follows:

$$Q = \frac{1.486}{n} A R^{2/3} S_f^{\ 1/2} \qquad \text{6-3}$$

Where:

Q = Discharge, in cubic feet per second.

n = Roughness coefficient.

A = Area of water normal to flow, in square feet.

R = Hydraulic radius, unitless.

S_f = Friction slope, unitless.

When rearranged into a more useful form, where:

$$K = \frac{1.486 A R^{2/3}}{n} \qquad \text{6-4}$$

Then:

$$Sf = \left[\frac{nQ}{1.486 A R^{2/3}}\right]^2 = \left[\frac{Q}{K}\right]^2 \qquad \text{6-5}$$

The loss of head due to friction (h_f) throughout the length of reach (L) is calculated by:

$$H_f = L S_f = L\left[\frac{Q}{K}\right]^2 \qquad \text{6-6}$$

The value of K is dependent upon only two factors: the geometrical shape of the flow cross section as expressed by the quantity ($AR^{2/3}$) and the roughness coefficient (n). The values of n are shown in Table 6-4.

Table 6-4: Values of Manning's Roughness Coefficient "n"

Material	n
Reinforced concrete pipe	0.013
PVC Pipe	0.010
HDPE	0.020
Poured concrete	0.013
No-joint cast in place concrete pipe	0.014
Reinforced concrete box	0.015
Reinforced concrete arch	0.015
Streets (asphalt pavement)	0.017
Flush grouted riprap	0.020
Corrugated metal pipe	0.025

Material	n
Grass-lined channels (sodded & irrigated)	0.025
Earth-lined channels (smooth)	0.030
Natural arroyos	0.030
Wire-tied riprap	0.040
Medium weight dumped riprap (Please refer to FHWA HEC-15 for varying D50)	0.045
Grouted riprap (exposed rock)*	0.045
Arroyo overbank	0.045
Jetty Type riprap (D50 > 24")	0.050

*Excess Grout defeats purpose of Rip-Rap to slow down velocities.
Table is an exert from HEC-15 Design of Roadside Channels with Flexible Linings.

6.1.3.2. Exit Losses

When a storm drain outfalls to a pond, lake, or open channel, additional head loss occurs due to the change in velocity at the outlet of the pipe and the changes in flow direction. The exit head loss at storm drain outlets is expressed as:

$$h_o = 1.0\left[\frac{V_o^{\,2}}{2g}\right] \qquad 6\text{-}7$$

Where:

h_o = *Head loss at outlet, in feet.*

V_o = *Average outlet velocity, in feet per second.*

6.1.3.3. Bend Losses

Bend losses will be calculated from the following equations. Bend losses should be included for all closed conduits, those flowing partially full as well as those flowing full.

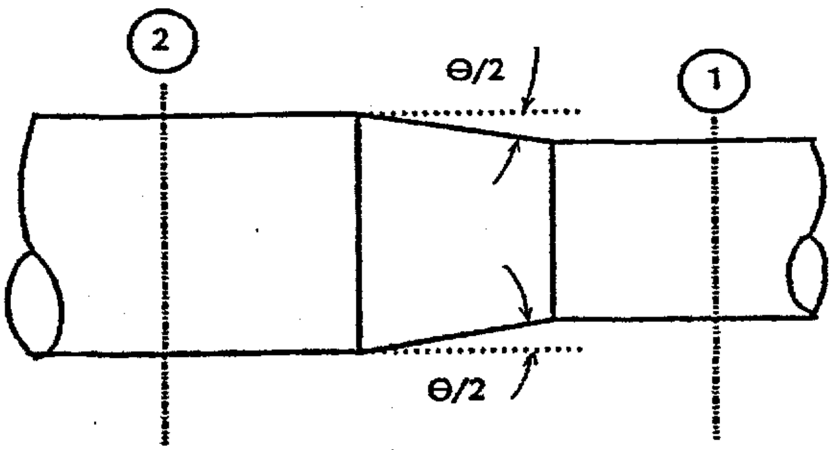
$$h_b = K_b\left[\frac{V^2}{2g}\right] \qquad 6\text{-}8$$

in which:

$$K_b = 0.20\sqrt{\frac{\Delta}{90°}} \qquad 6\text{-}9$$

6.1.3.4. Transition Losses

Transition losses will be calculated using the equations shown in Figure 6-3 below. If the transition is through a manhole, then both the transition loss and the manhole loss are applied. These equations are applicable when no change in Q occurs and where the horizontal angle of divergence or convergence (θ/2) between the two sections does not exceed 5 degrees 45 minutes.



For increasing velocities in the direction of flow from 2 to 1

$$h_t = 0.1\left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right]$$

For decreasing velocities in the direction of flow from 1 to 2

$$h_t = 0.2\left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right]$$

Figure 6-3: Transition Head Loss

When the angle of divergence or convergence (θ/2) is greater than 5 degrees 45 minutes, the above equation is not appropriate as it will give results for h_t that are too small. The use of more accurate methods, such as the Gibson method (see Figure 6-4), is more appropriate and acceptable.

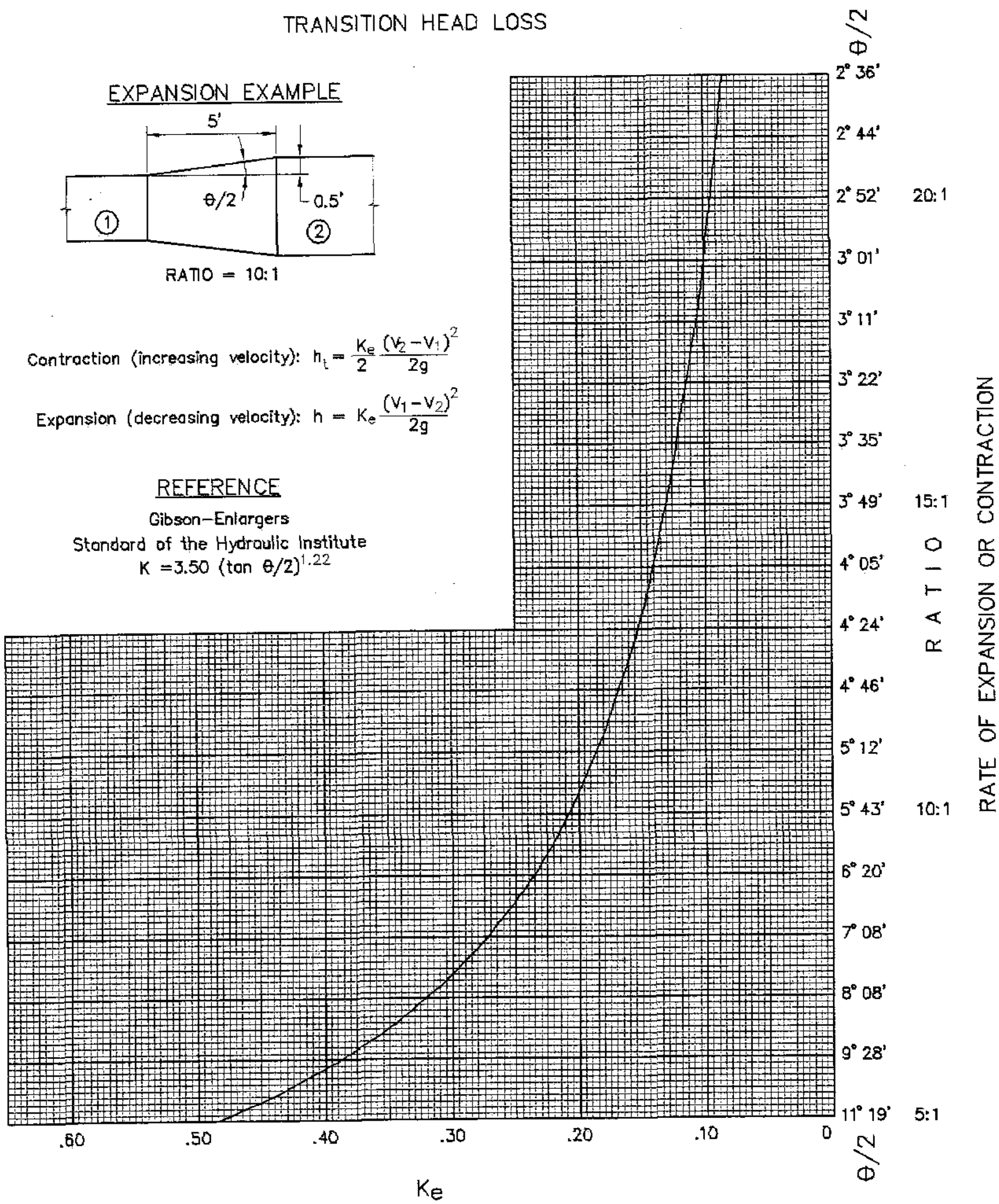


Figure 6-4: Gibson Method Transition Head Loss

As a general rule, storm drains will be designed with sizes increasing in the downstream direction. However, it may be advisable to decrease the size of a downstream section for increased velocities and increased sediment suspension. The conduit may be decreased in size in accordance with the following limitations:

For slopes of 0.0025 (0.25 percent) or less, conduit sizes may be decreased to a minimum diameter of 72 inches; however, the reduction is limited to a maximum of 6 inches.

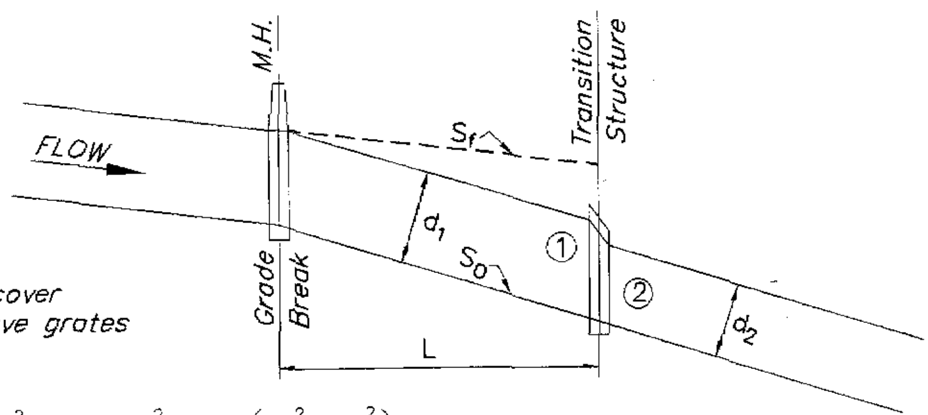
For slopes steeper than 0.0025, conduit sizes may be decreased to a minimum diameter of 30 inches; however, the reduction is limited to a maximum of 3 inches for pipe 48 inches in diameter or smaller, and to a maximum of 6 inches for pipe larger than 48 inches in diameter.

In any case the reduction in size must result in a more economical system.

Where conduits are to be decreased in size due to a change in grade, the criteria for locating the transition is shown on Figure 6-5.

Large to Small Conduit

NOTE:
Manhole cover should have grates



$$S_0L + d_1 + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g} + 0.1\left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) + S_fL + h_m \text{ , and}$$

$$S_0L - S_fL = d_2 - d_1 + 1.1\left(\frac{V_2^2}{2g} \quad \frac{V_1^2}{2g}\right) + h_m \text{ ..... therefore}$$

$$L = \frac{d_2 - d_1 + 1.1\left(\frac{V_2^2}{2g} \quad \frac{V_1^2}{2g}\right) + h_m}{S_0 - S_f}$$

where
S_o = slope of conduit
S_f = friction slope of larger conduit
d_1 = diameter or depth of larger conduit
V_1 = velocity in larger conduit flowing full
d_2 = diameter or depth of smaller conduit
V_2 = velocity in smaller conduit flowing full
h_m = other losses occurring between the transition and the grade break such as bend and confluence losses

EXAMPLE PROBLEM

Q=400 cfs

d_1 = 84" = 7'	d_2 = 78" = 6.5'
A_1 = 38.49 sq. ft.	A_2 = 33.18 sq. ft.
V_1 = 10.40 fps	V_2 = 12.0 fps
$\frac{V_1^2}{2g}$ = 1.68'	$\frac{V_2^2}{2g}$ = 2.24'

S =.00474
S = .00395

$$L = \frac{6.5 - 7.0 + 1.1(2.24 - 1.68)}{.00474 - .00395} = 147$$

Figure 6-5: Location of Transition Example Problem

6.1.3.5. Junction Losses

In general, junction losses are calculated by equating pressure plus momentum through the confluences under consideration. This can be done by using either the Pressure plus Momentum method or the Thompson equation (Equation 6-10). Both methods are applicable in all cases for pressure flow and will give the same results.

For the special case of pressure flow with A1 = A2 and friction neglected,

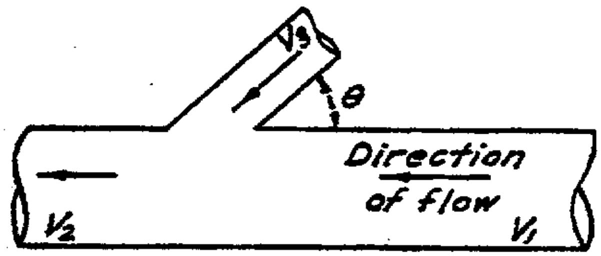


Figure 6-6: Junction Losses

$$hj = \frac{V_2^{\,2}}{2g} - \frac{V_1^{\,2}}{2g} - \frac{2A_3}{A_2} * \frac{v_3^{\,2}}{2g} * \cos\theta \qquad 6\text{-}10$$

The Momentum Equation for junctions is described by the following

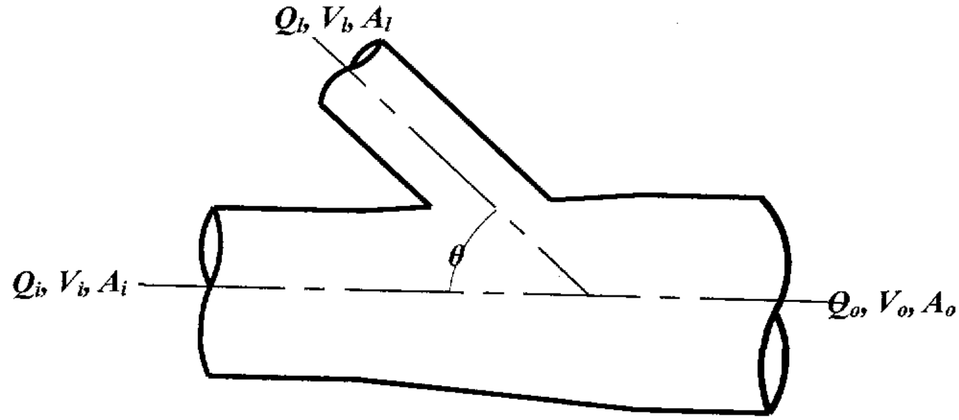


Figure 6-7: Momentum Equation for Junction Losses

$$hj = \frac{(Q_o V_o) - (Q_i V_i) - (Q_1 V_1 \cos\theta)}{0.5g(A_o + A_i)} + h_i + h_o \qquad 6\text{-}11$$

Where:

hj = difference in hydraulic gradient for the two end sections, in feet.

A_{avg} = *average area, in feet squared* = $1/6\ (A_1 + 4A_m + A_2)$.

A_m = *mean area of flow, in feet squared.*

The above equation is applicable only to circular conduits or channels. The friction force may be considered negligible or can be calculated and taken into account. It is recommended that the Thompson equation not be used when an open channel changes side slope going through a junction.

For greater detail of the above methods, refer to the FHWA *Urban Drainage Design Manual, Hydraulic Engineering Circular HEC-22, Second Edition.*

6.1.3.6. Drop Inlet and Manhole Losses

The energy loss encountered going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head of the outlet pipe. Using K to represent the constant of proportionality, the energy loss, H_{ah}, is approximated by equation 6-12. Experimental studies have determined that the K value can be approximated by the relationship in equation 6-13 when the inflow pipe invert is below the water level in the access hole.

Table 6-5: Head Loss Coefficient

Structure Configuration	K_{ah}
Inlet-Straight Run	0.50
Inlet-angled through	
90 degrees	1.50
60 degrees	1.25
45 degrees	1.10
22.5 degrees	0.07
Manhole – Straight run	0.15
Manhole – Angled through	
90 degrees	1.00
60 degrees	0.85
45 degrees	0.75
22.5 degrees	0.45

$$H_{ah} = K\left(\frac{V_o^2}{2g}\right) \qquad 6\text{-}12$$

$$K = K_o C_D C_d C_Q C_p C_B \qquad 6\text{-}13$$

Where:

K = Adjusted loss coefficient.

K_o = *Initial head loss coefficient based on relative access hole size.*

C_D = *Correction factor for pipe diameter (pressure flow only).*

C_d = *Correction factor for flow depth.*

C_Q = *Correction factor for relative flow.*

C_p = *Correction factor for plunging flow.*

C_B = *Correction factor for benching.*

V_o = *Velocity of outlet pipe.*

The initial head loss coefficient, K_o is estimated as a function of the relative access hole size and the angle of deflection between the inflow and outflow pipes. This deflection angle is represented in Figure 6-8.

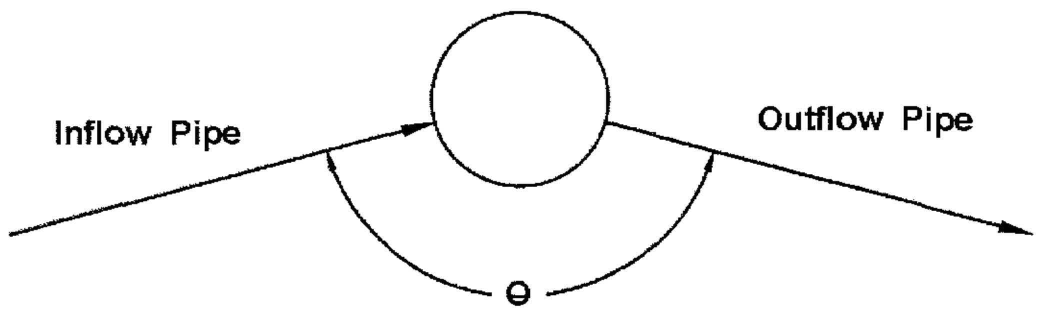


Figure 6-8: Deflection Angle

$$K_o = 0.1\left(\frac{b}{D_o}\right)(1-\sin\theta)+1.4\left(\frac{b}{D_o}\right)^{0.15}\sin\theta \qquad \text{6-14}$$

Where:

θ = *Angle between the inflow and the outflow pipes, in degrees.*

b = *Access hole or junction diameter, in feet.*

D_o = *Outlet pipe diameter, in feet.*

A change in head loss due to differences in pipe diameter is only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d_{aho}/D_o, is greater than 3.2. In these cases a correction factor for pipe diameter, C_D, is computed using the following equation. Otherwise C_D is set equal to 1.

$$C_D = \left(\frac{D_o}{D_i}\right)^3 \qquad \text{6-15}$$

Where:

D_o = *Outgoing pipe diameter, in feet.*

D_i = *Inflowing pipe diameter, in feet.*

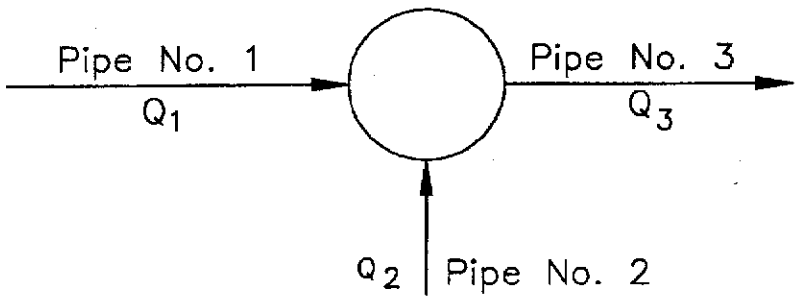


Figure 6-9: Relative flow effect

The correction factor for flow depth, C_d, is significant only in cases of free surface flow or low pressures, when the d_{aho}/D_o ratio is less than 3.2. In cases where this ratio is greater than 3.2, C_d is set equal to 1. To determine the applicability of this factor, the water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor is calculated using Equation 6-16.

$$C_d = 0.5\left(\frac{d_{aho}}{D_o}\right)^{0.6} \qquad 6\text{-}16$$

Where:

d_{aho} = *Water depth in access hole above the outlet pipe invert, in feet.*

D_o = *Outlet pipe diameter, in feet.*

The correction factor for relative flow, C_Q, is a function of the angle of the incoming Flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed using Equation 6-17. The correction factor is only applied to situations where there are 3 or more pipes entering the structure at approximately the same elevation. Otherwise, the value of C_Q is equal to 1.0.

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \qquad 6\text{-}17$$

Where

C_Q = *Correction factor for relative flow.*

θ = *The angle between the inflow and outflow pipes, in degrees.*

Q_i = *Flow in the inflow pipe, in cubic feet per second.*

Q_o = *Flow in the outflow pipe, in cubic feet per second.*

The correction factor for plunging flow, C_p, is calculated using Equation 6-18. This correction factor corresponds to the effect another inflow pipe, plunging into the access hole, has on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 6-9, C_p is calculated for pipe #1 when pipe #2 discharges plunging flow. The correction factor is only applied when h > d_{aho}. Additionally, the correction factor is only applied when a higher elevation flow plunges into an access hole that has both an inflow line and an outflow in the bottom of the access hole. Otherwise, the value of C_p is equal to 1.0. Flows from a grate inlet or a curb opening inlet are considered to be plunging flow and the losses would be computed using Equation 6-18.

$$C_p = 1 + 0.2\left(\frac{h}{D_o}\right)\left(\frac{h - d_{aho}}{D_o}\right) \qquad 6\text{-}18$$

Where:

C_p = *Correction for plunging flow.*

h = *Vertical distance of plunging flow from the flowline of the higher elevation inlet pipe to the center of the outflow pipe, in feet.*

D_o = *Outlet pipe diameter, in feet.*

d_{aho} = *Water depth in access hole relative to the outlet pipe invert, in feet.*

The correction for benching in the access hole, C_B, is obtained from Table 6-6. Figure 6-10 illustrates benching methods listed in Table 6-6. Benching tends to direct flow through the access hole, resulting in a reduction in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Table 6-6: Correction for Benching

Bench Type	Correction Factors, C_B	
	Submerged	**Unsubmerged**
Flat or Depressed Floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07

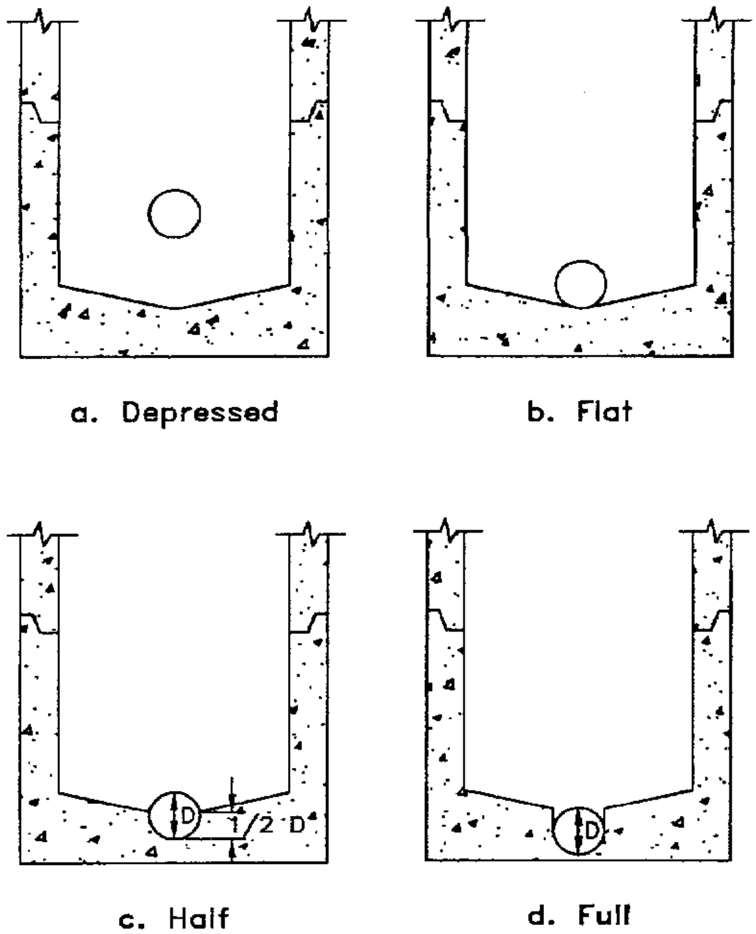


Figure 6-10: Access Hole Benching Methods

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe using the energy-loss method, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

6.2 Design Guidelines and Considerations

6.2.1 Velocity and Grade Constraints

The minimum slope for main line conduit is 0.001 (0.10 percent), so long as flow velocity for the 10-year design flow is at least 3 feet per second. For lateral pipes, the slope is derived from the minimum velocity of 3 feet per second. The aforementioned flow velocities for both lateral and trunk storm drain of 3 feet per second shall also be known as the minimally accepted **"Cleaning"** velocity.

The maximum velocity in storm drains are as follows:

1. Storm drain inlet laterals - no maximum permissible velocity.
2. Storm drain trunk - 20 feet per second.

6.2.2 Sizing

In cases where the conduit may carry significant amounts of sediment, the minimum diameter of main line conduit will be 24 inches. For all other situations the minimum diameter is 18 inches.

6.2.3 Location

Manholes should be located as close to changes in grade as feasible when the following conditions exist:

(A) When the upstream conduit has a steeper slope than the downstream conduit and the change in grade is greater than 10 percent, sediment tends to deposit at the point where the change in grade occurs.

(B) When transitioning to a smaller downstream conduit due to an abruptly steeper slope downstream, sediment tends to accumulate at the point of transition.

6.2.4 Access Spacing

Manholes, mainline inlets, or junction boxes should be spaced at intervals of approximately 450 feet. Where the proposed conduit is less than 30 inches in diameter and the horizontal alignment has bends or angle points, the manhole spacing should be reduced to approximately 300 feet.

The spacing requirements shown above apply regardless of design velocities.

Lateral pipe entering a main line pipe storm drain generally will be connected radially and exceptions may be permitted where it can be shown that the cost of bringing laterals into a main line conduit would be excessive. Where a manhole is used to connect later to main line, there shall be a minimum spacing of 5 feet from other manholes or laterals.

6.2.5 Alignment

In general, the angle of confluence between main line and lateral must not exceed 45 degrees, and as an additional requirement, must not exceed 30 degrees under any of the following conditions:

(A) Where the peak flow in the proposed lateral exceeds 10 percent of the main trunk line peak flow.

(B) Where the velocity of the peak flow in the proposed lateral is 20 feet per second or greater.

(C) Where the size of the proposed lateral is 60 inches or greater.

(D) Where hydraulic calculations indicate excessive head losses may occur in the main line due to the confluence.

Connector pipe may be joined to main line pipe at angles greater than 45 degrees up to a maximum of 90 degrees, provided none of the above conditions exist. If, in any specific situation, one or more of the above conditions does apply, the angle of confluence for connector pipes may not exceed 30 degrees. Connections must not be made to main line pipe that may create conditions of adverse flow in the connector pipes.

The above requirements may be waived only if calculations are submitted to the City Engineer showing that the use of a confluence angle larger than 30 degrees will not unduly increase head losses in the main line, using the equations for junction in this Chapter.

When the design flow in a pipe flowing full has a velocity of 20 feet per second or greater, or is supercritical in a partially-full pipe, the total horizontal angle of divergence or convergence between the walls of the manhole and its center line should not exceed 5 degrees 45 minutes.

A manhole shaft safety ledge will be provided in all instances when the manhole shaft is 20 feet or deeper. The location of the access opening and steps shall be placed at opposite locations for each ledge chamber.

The minimum angle for laterals connecting into manhole trunk line shall not be less than 90 degrees. For pipe junctions other than manhole, the angle of intersection between the two flow paths shall not be greater than 45 degrees. This includes discharges into box culverts and channels.

6.2. 6 Junction Boxes without Manhole (Prefabricated)

Junctions will only be permitted on main storm drain lines larger than 42 inches. Junction locations cannot be more that 24 feet from the downstream manhole. An exception to this requirement may be laterals with slopes of 5% or greater. The City Engineer's approval will be required for this exception and all other variances.

6.2. 7 Flap Gates

A flap gate must be installed in all laterals connecting to a main line storm drain whenever the potential water surface level of the main line is higher than the surrounding area drained by the lateral. The gate must be set back from the main line drain so that it will open freely and not interfere with the main line flow.

6.2. 8 Pipe Materials

Rubber-gasketed pipe will be used in all storm drain construction unless otherwise approved by the City Engineer. Asbestos and non-reinforced concrete pipe may not be used for storm drain applications. Pipe material should withstand soil corrosive nature, be cost effective, and withstand design loading. Table 6-7 provides a list of AASHTO pipe materials.

Table 6-7: AASHTO (AWWA) Pipe Materials

Category/Parameters	Pipe
Reinforced Concrete	
Round	M 170M/M 170
Arch	M 206M/M 206
Elliptically Shaped	M 207M/M 207
Reinforced D-Load	M 242M/M 242
Cylinder	C 303
Pressure	C 302

6.2.9 Access Protection Barrier

An access protection barrier is a means of preventing people from entering storm drains. Protection barriers will be provided wherever necessary to prevent unauthorized access to storm drains. The barrier may be a breakaway type. It will be the designer's responsibility to provide a protection barrier appropriate to each situation and to provide details of such on the construction drawings.

7. Drop Inlets

Proper surface drainage of streets and highways may require intercepting excess flows with storm water drop inlets. A storm water drop inlet is an opening into a storm drain system that permits entrance of surface storm runoff. The most upstream drop inlet in the system should be placed as far downstream as possible, because as soon as the runoff enters the pipe system, it is carried rapidly downstream which tends to reduce the Time of Concentration. The placing of drop inlets is dictated by street encroachment and flow depth criteria.

Submitted data should include complete cross sections between property lines of streets at the proposed drop inlets and of any streets that control the flow of water to the pertinent locations. Street cross sections should indicate the following:

- Dimensions from the street center line to the top of curb and property line.
- Gutter slope upstream of each drop inlet.
- Elevations for the top of curb, flow line, property line and street crown at each drop inlet center line.
- Curb batter.

7.1. Characteristics and Uses of Drop Inlets

- Curb opening drop inlets.
- Grated drop inlets.
- Combination drop inlets.
- Slotted drains

Curb opening drop inlets are best for use when a sump condition exists. Although a curb opening drop inlet will not guarantee against plugging by debris, it is the most desirable type of drop inlet.

A curb opening drop inlet is a vertical opening in a curb through which the gutter flow passes. For safety reasons, the vertical opening should not be greater than 6 inches. The gutter may be undepressed or depressed in the area of the curb opening. The capacity of the curb opening is significantly increased by depressing the opening.

The engineer shall demonstrate proper design of drop inlets through "bypass flow" calculations where applicable.

Grated or gutter drop inlet refers to an opening in the gutter covered by one or more grates through which the water falls. As with other drop inlets, grated drop inlets may be either depressed or undepressed and are more efficient than curb opening drop inlets when located on a continuous grade.

When grated drop inlets are used, the engineer should design them to optimize hydraulic efficiency, bicycle and pedestrian safety, structural adequacy, economy, and freedom from clogging. The adjusting gross capacity factor for inlets on a grade is 0.70 or 70% (30% partially clogged inlet) and the adjusting gross capacity factor for inlets on a sump is 0.50 or 50% (50% partially clogged inlet).

A combination drop inlet is composed of a curb opening and a grated drop inlet acting as a unit. Usually the gutter opening is placed adjacent to the curb opening. As with other drop inlets, a combination drop inlet may be either depressed or undepressed and located in a sump or on a continuous grade.

For the widest range of conditions, the combination drop inlet is the most efficient type of storm water drop inlet for hydraulic interception capabilities and eliminating debris clogging.

A slotted drain is a slot opening in the pavement that intercepts sheet flow and conveys it through a corrugated steel pipe. Slotted drains are most effective when street slopes are shallow.

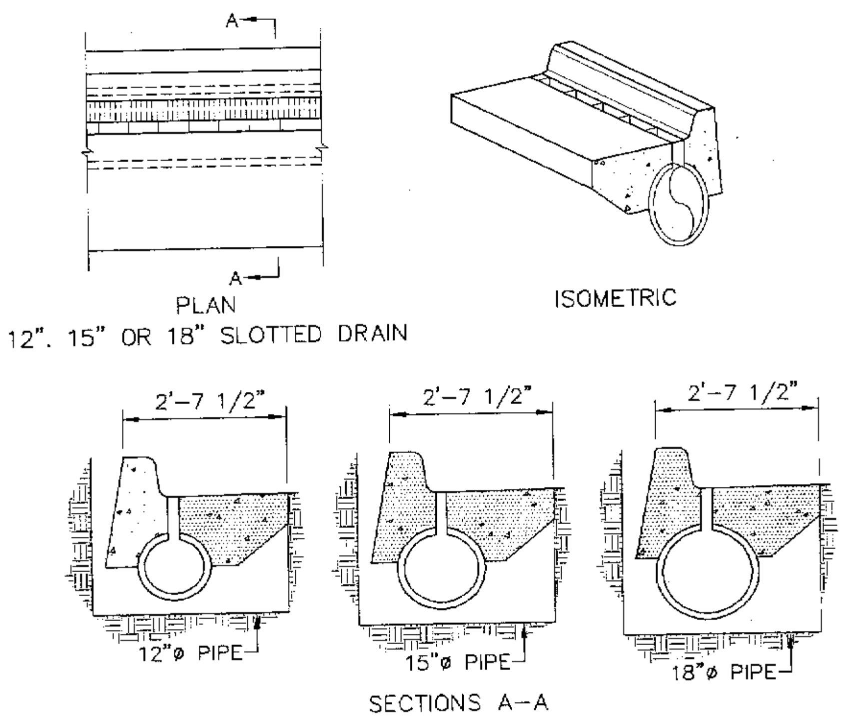


Figure 7-1: Slotted Drain

Drop inlets may be further classified as being on a continuous grade or in a sump. The continuous grade condition exists when the street grade is continuous past the drop inlet and the water can flow past. The sump condition exists whenever water is restricted to the drop inlet area because it is located at a low point. This may be due to a change in grade of the street from positive to negative or due to the crown slope of a cross street when the drop inlet is located at an intersection.

7.2. Drop Inlet Capacity

For capacities of the various inlet design configurations acceptable to the City of El Paso and outlined in the City of El Paso-Design Standards for Construction (DSC), the weir and orifice equations (when applicable) included in this Drainage Design Manual (DDM) shall be applied accordingly on case by case basis.

7.3. Interception Capacity of Drop Inlets on Grade

(Note; For inlets on a grade, net inlet capacity to be adjusted by factor of 0.70 or 70% (30% partially clogged inlet) to derive gross inlet capacity. Refer to Section 7.1.)

7.3.1. Grate Drop Inlets

On-grade grated drop inlets intercept all of the frontal flow until splash over (the velocity at which water begins to splash over the grate) is reached. At velocities greater than splash over, grate efficiency in intercepting frontal flow is diminished. Grates also intercept a portion of the flow along the length of the grate, or the side flow, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

The ratio of frontal flow to total gutter flow, E_o, for a straight cross slope is:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad 7\text{-}1$$

Where:

Q_W = Flow rate in width (W), in cubic feet per second.

Q = Total flow, in cubic feet per second.

W = Width of grate or gutter, in feet.

T = Spread of flow on the pavement, in feet.

The ratio of side flow, (Q_s) to total gutter flow (Q) is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \qquad 7\text{-}2$$

Where:

Q_s = *Flow rate outside of width (W), in cubic feet per second.*

Q_w = *Flow rate in width of grate or gutter (W), in cubic feet per second.*

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed:

$$R_f = 1 - 0.09(V - V_o) \qquad 7\text{-}3$$

Where:

R_f = *Ratio of frontal flow intercepted to total frontal flow.*

V = *Velocity of flow in the gutter, in feet per second.*

V_o = *Gutter velocity where splash over first occurs, in feet per second.*

This ratio is equivalent to frontal flow interception efficiency. Figure 7-2 is the solution to the ratio of frontal flow.

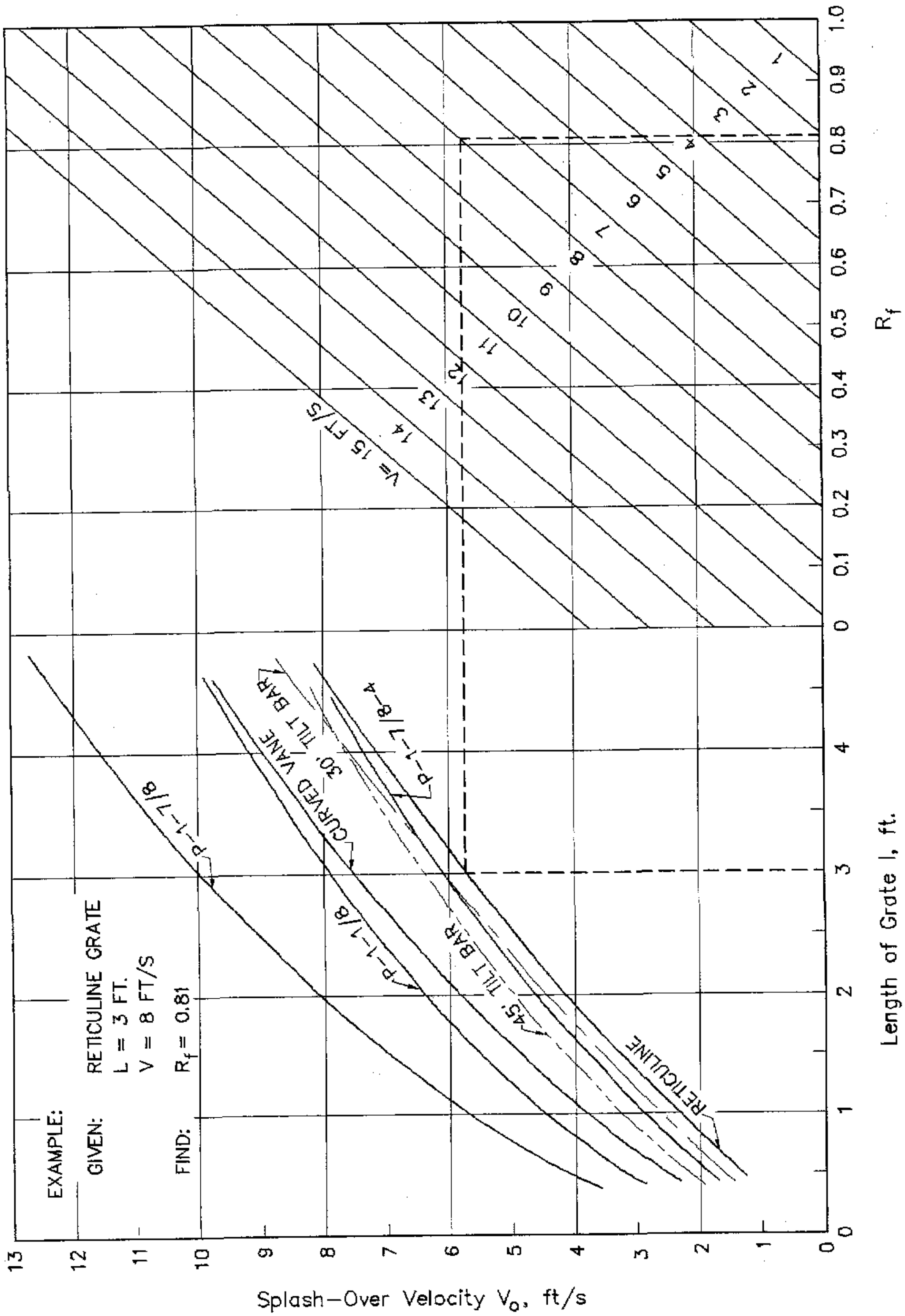


Figure 7-2: Ratio of Frontal Flow

The ratio of side flow intercepted to total side flow, R_s, or side flow interception efficiency, is expressed:

$$R_s = \frac{1}{1+\frac{0.15V^{1.8}}{S_x L^{2.3}}} \qquad 7\text{-}4$$

Where:

S_x = Pavement cross slope, in foot per foot.

L = Length of grate, in feet.

V = Velocity of flow in the gutter, in feet per second.

Figure 7-36 is the solution to the R_s equation.

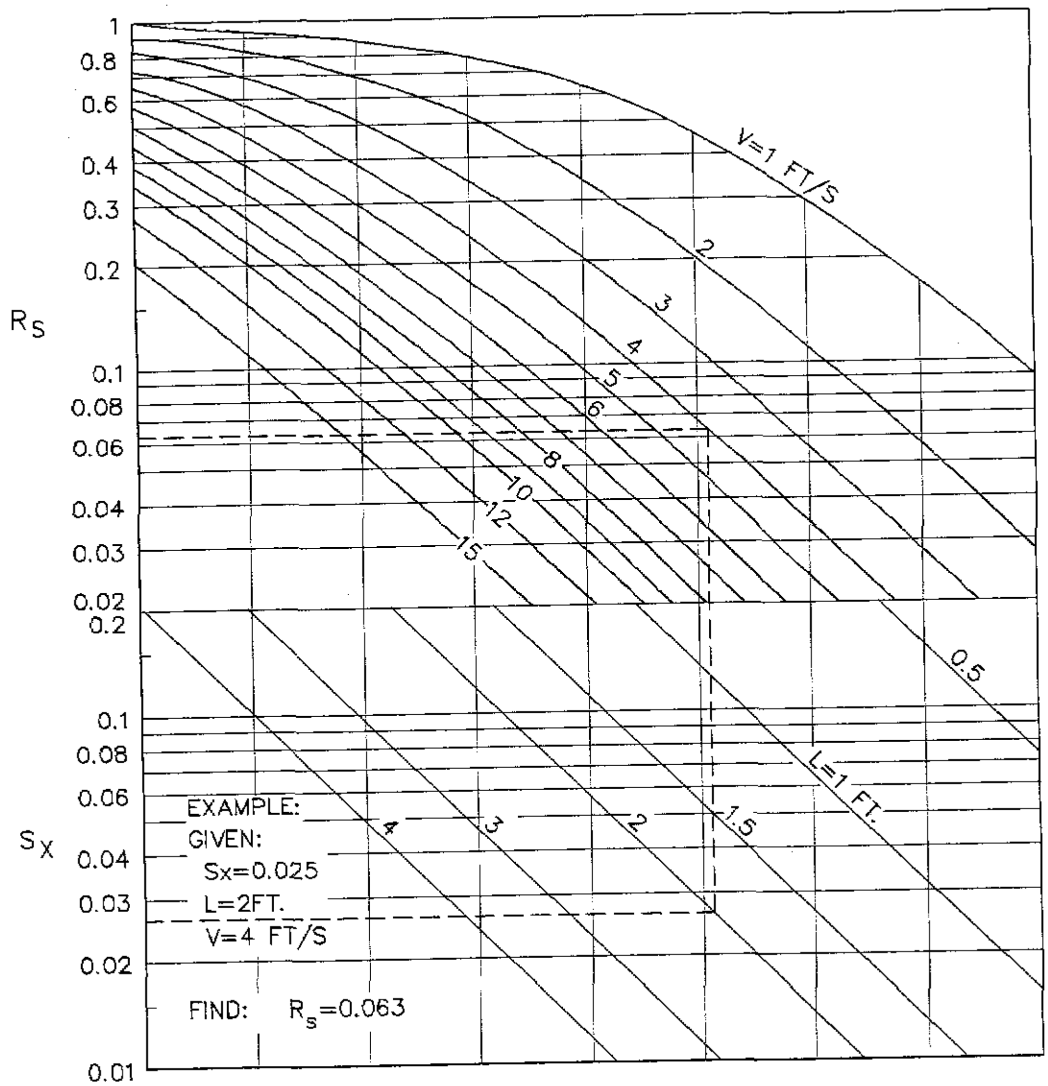


Figure 7-3: Ratio of Side Flow

The efficiency, E, of a grate is:

$$E = R_f E_o + R_s(1 - E_o) \quad 7\text{-}5$$

The first term on the right side of efficiency equation is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity, Q_i, of a grate drop inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\left[R_f E_o + R_s(1 - E_o)\right] \quad 7\text{-}6$$

7.3.2. Curb-Opening Drop Inlets

For the on grade condition the length of curb opening drop inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed as:

$$L_t = 0.6Q^{0.42}S^{0.3}a\left(\frac{1}{nS_x}\right)^{0.6} \quad 7\text{-}7$$

Where:

Q = *Total gutter flow rate, in cubic feet per second.*

S = *Longitudinal slope, in foot per foot.*

S_x = *Pavement cross-slope, in foot per foot.*

n = *Manning's roughness coefficient.*

The efficiency, E, of curb-opening drop inlets shorter than the length required for total interception is:

$$E = 1 - \left(1 - \frac{L}{L_t}\right)^{1.8} \quad 7\text{-}8$$

Where:

L = *Length of curb opening, grate or slot, in feet.*

L_t = *Curb opening length required to intercept 100% of the gutter flow, in feet.*

The length of drop inlet required for total interception by depressed curb-opening drop inlets or curb openings in depressed gutter sections can be found by using an equivalent cross slope, S_e, calculated by the following:

$$S_e = S_x + S_w' E_o \quad 7\text{-}9$$

Where:

S'_w = *Cross slope of the gutter (at the inlet) measured from the cross slope of the pavement, in foot per foot ($S'w=a/12W$).*

E_o = *Ratio of flow in the depressed section to total gutter flow.*

S_x = *Pavement cross-slope, in foot per foot.*

E_o is the ratio of flow in the depressed section to the total gutter flow, and S'_w is the cross slope of the gutter measured from the cross slope of the pavement, S_x.

The length of curb-opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Using the equivalent cross slope, S_e becomes:

$$L_t = 0.6Q^{0.42}S^{0.3}\left(\frac{1}{nS_e}\right)^{0.6} \qquad \text{7-10}$$

7.3.3. Slotted Drains

Wide experience with the debris handling capabilities of slotted drains is not available. Deposition in the pipe is the problem most commonly encountered; however, the slotted drain is accessible for cleaning with a high pressure water jet.

Flow interception by slotted drain for on-grade drop inlets and curb-opening drop inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Analysis of data from the HEC-12 tests of slotted drains with slot widths greater than or equal to 1.75 inches indicates that the length of the slotted drain required for total interception can be computed using curb opening length equation. Similarly, the efficiency of curb opening equation is also applicable to slotted drains. It should be noted, however, that it is much less expensive to add length to a slotted drain to increase interception capacity than it is to add length to a curb-opening drop inlet.

7.3.4. Combination Inlets

A combination drop inlet consists of a grated drop inlet and an adjacent curb opening inlet. A combination drop inlet has a net interception capacity equal to the sum of the grated drop inlet and of the curb opening inlet. This net interception capacity shall be adjusted to account for debris / clogging by the applicable factor for the design scenario.

7.4. Interception Capacity of Drop Inlets in Sump

(Note; For inlets in a sump, net inlet capacity to be adjusted by factor of 0.50 or 50% (50% partially clogged inlet) to derive gross inlet capacity. Refer to Section 7.1.)

7.4.1. Grate Drop Inlets

The efficiency of drop inlets in passing debris is critical in sump locations because all runoff that enters the sump must be passed through the drop inlet. Total or partial clogging of drop inlets in these locations can result in hazardous ponding conditions. Grate drop inlets alone are not recommended for use in sump locations because of the tendencies of grates to become clogged. Combination drop inlets or curb-opening drop inlets are recommended for use in these locations. A grate drop inlet in a sump location operates as a weir to depths dependent on the bar configuration and size of the grate and as an orifice at greater depths. Grates of larger dimension and grates with more open area, that is, with less space occupied by lateral and longitudinal bars, will operate as weirs to greater depths than smaller grates or grates with less open area. The capacity of grate drop inlets operating as weirs is:

$$Q_i = C_w P d^{1.5} \qquad 7\text{-}11$$

Where:

C_w = *Weir coefficient = 3.0*

P = *Perimeter of the grate, disregarding bars and side against curb, in feet.*

d = *Depth of flow at curb, in feet.*

The capacity of a grate drop inlet operating as an orifice is:

$$Q_i = C_o A_g (2gd)^{0.5} \qquad 7\text{-}12$$

Where:

C_o = *Orifice coefficient = 0.67*

A_g = *Clear opening area of the grate, in square feet.*

d = *Depth of flow at curb, in feet.*

g = *Gravity, 32.2 feet per second, squared.*

Use of the orifice equation requires the clear opening area of the grate. Tests of grates showed that for flat bar grates, the clear opening is equal to the total area of the grate less the area occupied by longitudinal and lateral bars.

Figure 7-4 shows the solution for the weir and orifice equation.

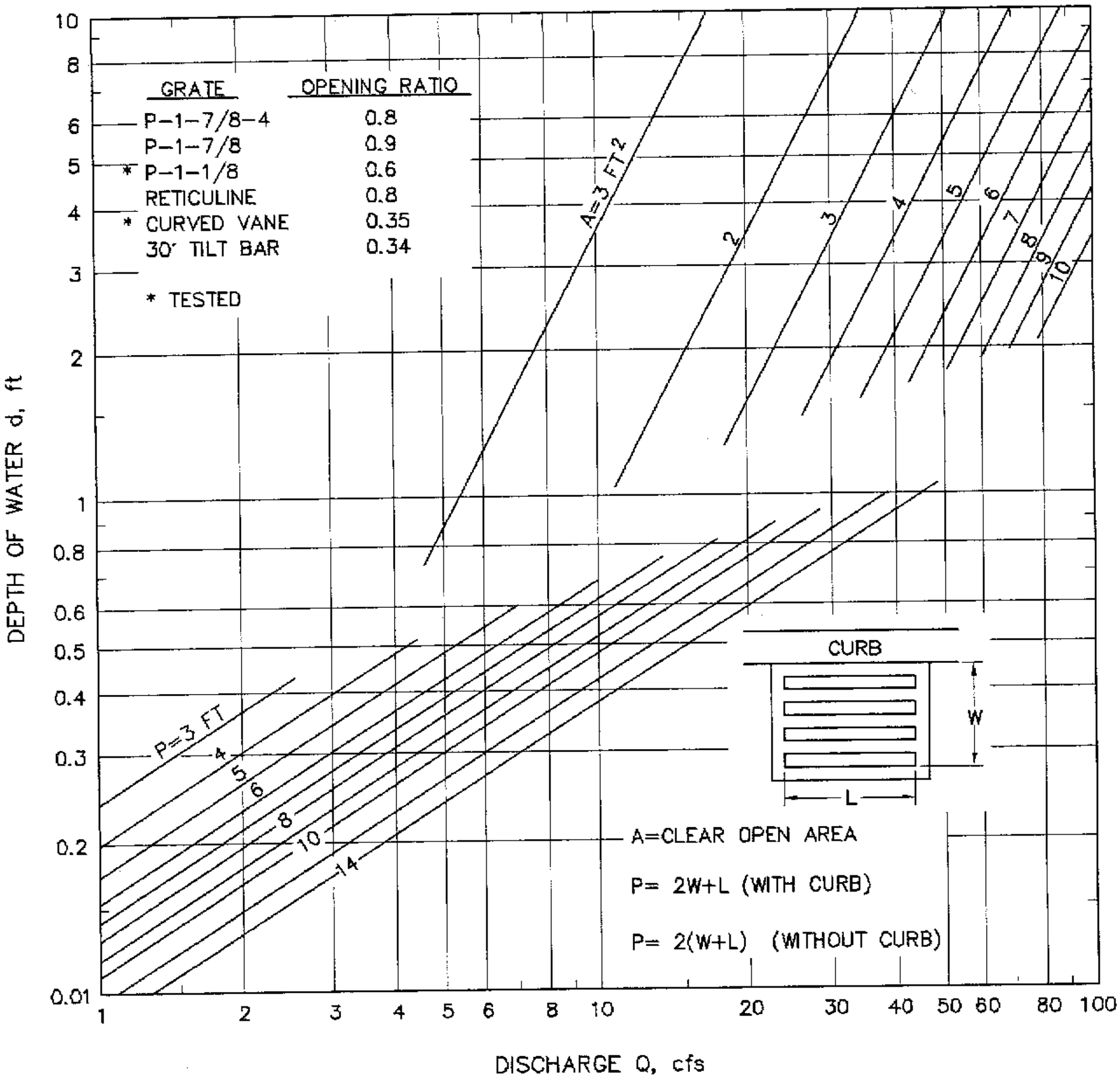


Figure 7-4: Grate Capacity

7.4.2. Curb-Opening Drop Inlets

The capacity of a curb-opening drop inlet in a sump depends on water depth at the curb, the curb opening length, and the height of the curb opening. The drop inlet operates as a weir for depths of water up to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At water depths between 1.0 and 1.4 times the opening height, flow is in a transition stage. The weir location for a depressed curb-opening drop inlet is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb-opening. The weir location for a curb opening drop inlet that is not depressed is at the lip of the curb-opening, and its length is equal to that of the curb-opening drop inlet. The equation for the interception capacity of a depressed curb-opening drop inlet operating as a weir is:

$$Q_i = C_w(L + 1.8W)d^{1.5} \qquad 7\text{-}13$$

Where:

Q_i = *Amount of street flow intercepted by inlet, in cubic feet per second.*

C_w = *Weir coefficient = 2.3.*

W = *Width of grate or depressed gutter, in feet.*

d = *Depth of flow, in feet (measured from water surface to projected cross slope).*

L = *Length of curb opening, or slot, in feet.*

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of a depressed curb opening drop inlet is:

$$d \le \left(h + \frac{a'}{12}\right) \qquad 7\text{-}14$$

Where:

h = *Height of curb opening drop inlet, curb opening orifice, or orifice throat width, in feet.*

a = *Gutter depression, in inches.*

The weir equation for curb opening drop inlets without depression ($W = 0$) becomes:

$$Q_i = C_w L d^{1.5} \qquad 7\text{-}15$$

Where:

$C_w = 3.0.$

d = *Depth of flow, in feet.*

L = *Length of curb opening or slot, in feet.*

The depth limitation for operation as a weir becomes: $d \le$ h

Curb opening drop inlets operate as orifices at depths greater than approximately 1.4*h*. The interception capacity can be computed by the following equation and is applicable to depressed and undepressed curb opening drop inlets, and the depth at the drop inlet includes any gutter depression:

$$Q_t = C_o h L (2 g d_o)^{0.5} \qquad 7\text{-}16$$

Where:

C_o = Orifice coefficient = 0.67.

g = Gravity, 32.2 feet per second squared.

d_o = Effective depth at the center of the curb opening orifice, in feet.

h = Height of curb opening drop inlet, curb-opening orifice, or orifice throat, in feet.

L = Length of curb opening, in feet.

Height of the orifice assumes a vertical orifice opening. As illustrated in Figure 7-5, other orifice throat locations can change the effective depth on the orifice and the dimension (d_l - $h/2$). A limited throat width could reduce the capacity of the curb-opening drop inlet by causing the drop inlet to go into orifice flow at depths less than the height of the opening.

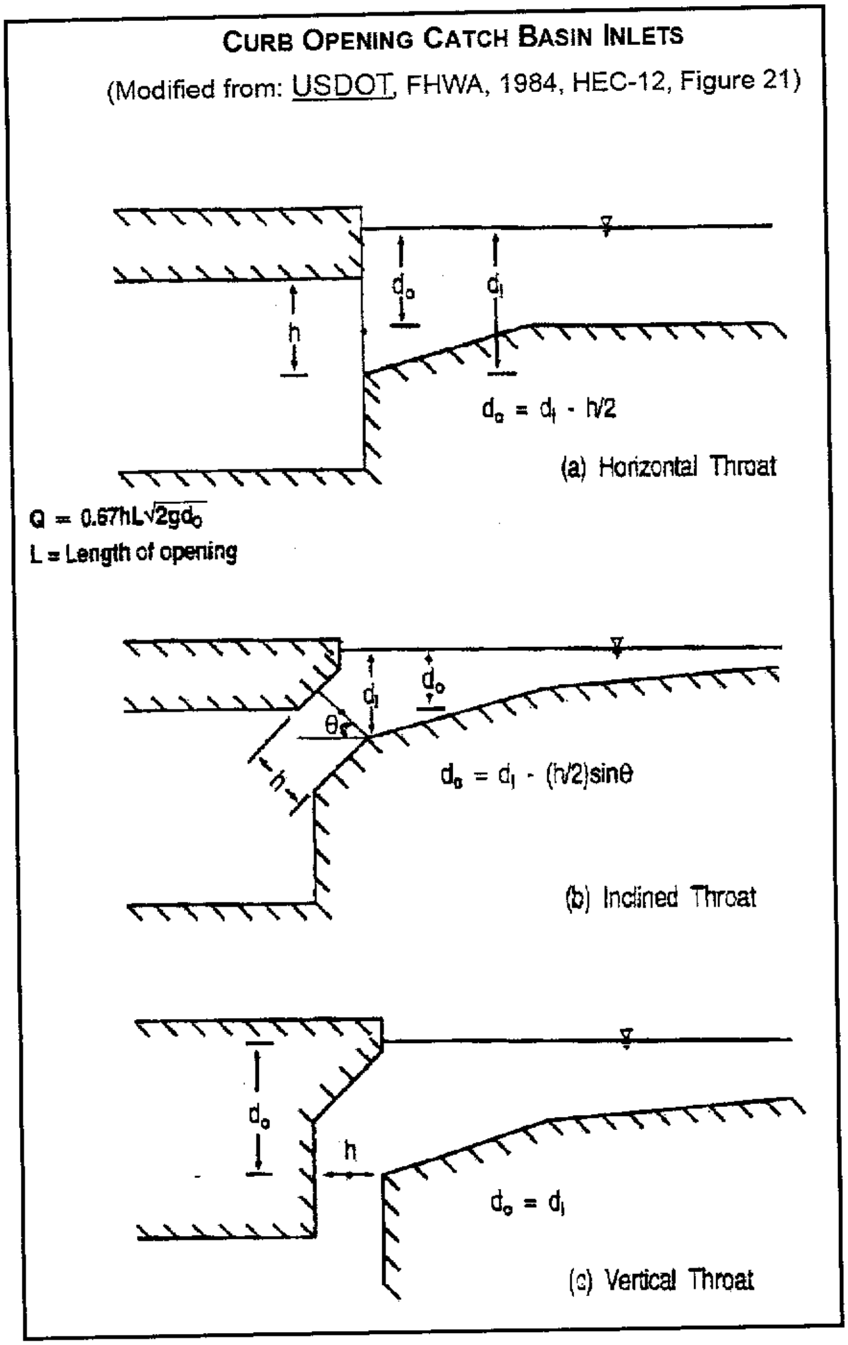


Figure 7-5: Throat Configurations for Curb-Opening Drop Inlets

Figure 7-6 is provided for use for curb openings with inclined or vertical orifice throats.

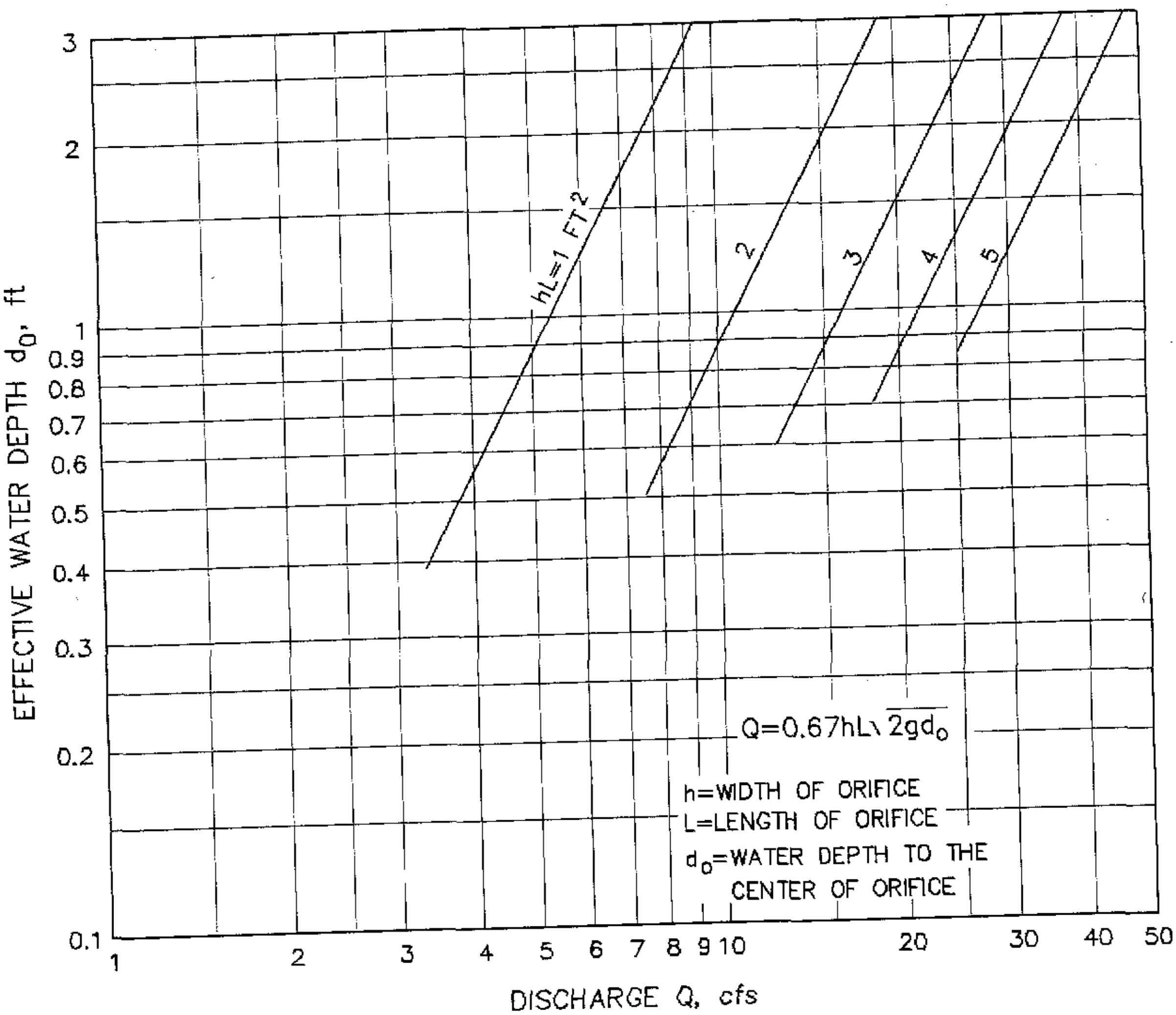


Figure 7-6: Capacity of Inclined or Vertical Orifice Throat Curb-Opening Drop Inlets

7.4.3. Slotted Drains

Slotted drains in sump locations perform as weirs to depths of about 0.2 foot, dependent on slot width and length. At depths greater than about 0.4 foot, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted drain operating as an orifice can be computed by:

$$Q_i = 0.8LW(2gd)^{0.5} \qquad \text{7-17}$$

Where:

Q_i = *Amount of street flow intercepted by slotted inlet, in cubic feet per second.*

L = *Length of slotted inlet, in feet.*

W = *Width of slot, in feet.*

d = *Depth of water at slot, $d \geq 0.4$ foot.*

g = *Gravity, 32.2 feet per second squared.*

$$Q_i = 0.94Ld^{0.5} \qquad (7\text{-}18)$$

Where:

W = 0.15 foot (1.75 inches)

The interception capacity of slotted drains at depths between 0.2 and 0.4 feet can be computed by using the orifice equation. The orifice coefficient varies with depth, slot width, and the length of the slotted drain.

Figure 7-7 provides the solutions for weir flow, transition flow, and orifice flow.

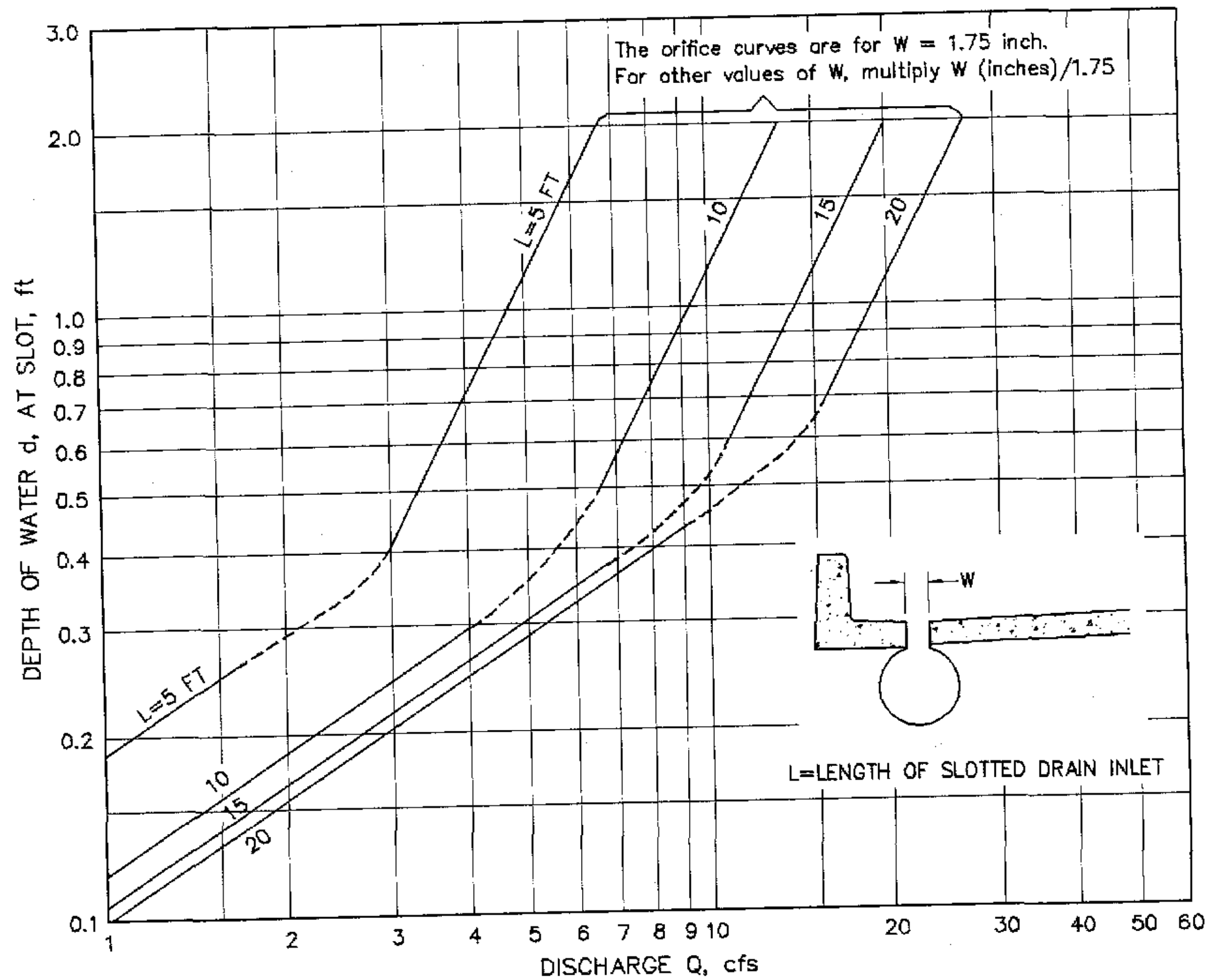


Figure 7-7: Capacity of Slotted Drain

7.4.4. Combination Drop Inlets

Combination drop inlets consisting of a grate and a curb opening are considered advisable for use in sumps where hazardous ponding can occur. The interception capacity of the combination drop inlet is essentially equal to that of a grate alone in weir flow unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the capacity of the grate plus the capacity of the curb opening.

The weir flow in combination drop inlets in sump locations can be used, assuming complete clogging of the grate the curb opening equation is applicable.

$$Q_i = 0.67A_g(2gd)^{0.5} + 0.67hL(2gd_o)^{0.5} \qquad \text{7-19}$$

Where:

Q_i = *Amount of street flow intercepted by inlet, in cubic feet per second.*

A_g = *Clear opening area of the grate, in square feet.*

g = *Gravity, 32.2 feet per second squared.*

d = *Depth of flow at curb, in feet.*

h = *Height of curb opening portion of drop inlet, curb-opening orifice or orifice throat, in feet.*

L = *Length of curb opening, in feet.*

d_o = *Effective depth at the center of the curb opening orifice, in feet.*

7.5. Drop Inlet Placement

Drop inlets will be located within street rights-of-way. Drop inlets must be placed to intercept storm water under existing conditions, which may include placement outside street right-of-way. Right-of-way or an easement for such drop inlets must be acquired. Drop inlets to be located outside dedicated streets to accommodate future street widening and those that will not intercept storm water under existing conditions will be marked for easy detection.

Drop inlets to be constructed off the paved portion of the roadway, but within the street property lines, must be made operable by grading the roadway to permit storm water to flow to the basin. Street remodeling of this nature will be performed during construction.

If a project is to have one or more cutoff points in phased construction, each cutoff point should have a battery of drop inlets at the upstream terminus sufficient to collect the flow carrying capacity of the storm drain at that point. Each battery of drop inlets should be designed with sufficient data regarding types and sizes of drop inlets, connector pipe sizes and D-loads, local depressions, and any other information may be necessary to construct the system. Drop inlets should be designed for the following criteria in the subsequent sections.

7.5.1. Inlet Geometric Controls

There are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or flow bypass. Examples of such locations are:

- At all low points in the gutter grade.

- Immediately upstream of median breaks, entrance/exit ramp gores, crosswalks, and street.
- At intersections (i.e., any location where water could flow onto the travel way).
- Immediately up grade of bridges (to prevent pavement drainage from flowing onto bridge decks).
- Immediately downstream of bridges (to intercept bridge deck drainage).
- Immediately up grade of cross slope reversals.
- Immediately up grade from pedestrian crosswalks.
- At the end of channels in cut sections.
- On side streets immediately up grade from intersections.
- Behind curbs, shoulders, or sidewalks to drain low area.

In addition to the areas identified above, runoff from areas draining towards the highway pavement should be intercepted by roadside channels or inlets before it reaches the roadway. This applies to drainage from cut slopes, side streets, and other areas alongside the pavement. Curbed pavement sections and pavement drainage inlets are inefficient means for handling extraneous drainage.

At Arterial-Arterial intersections with significant pedestrian traffic, drop inlets should be placed to capture 100 percent of flow at corners. For Arterial-Collector intersections, an allowable 75 cubic feet per second can be conveyed through the intersection along the Arterials. For Local-Collector intersections, flow bypass of 50 cubic feet per second is allowed.

7.5.2. Drop Inlet Spacing on Continuous Grade

Design spread is the criterion used for locating storm drain drop inlets between those required by geometric or other controls. The interception capacity of the upstream drop inlet will define the initial spread. As flow is contributed to the gutter section in the downstream direction, spread increases. The next downstream inlet is located at the point where the spread in the gutter reaches the design spread. Therefore, the spacing of drop inlets on a continuous grade is a function of the amount of upstream bypass flow, the tributary drainage area, and the gutter geometry. For a continuous slope, the designer may establish the uniform design spacing between inlets of a given design if the drainage area consists of pavement only or has reasonably uniform runoff characteristics and is rectangular in shape. In this case, the Time of Concentration is assumed to be the same for all drop inlets.

7.5.3. Flanking Drop Inlets

It is good engineering practice to place flanking drop inlets on each side of the low point inlet when in a depressed area that has no outlet except through the system. The purpose of flanking drop inlets is to act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

Flanking drop inlets can be located so they will function before water spread exceeds the allowable spread at the sump location. Flanking drop inlets should be located so that they will receive all of the flow when the primary drop inlet at the bottom of the sag is clogged. They should do this without exceeding the allowable spread at the bottom of the sag. If flanking drop inlets are the same dimension as the primary drop inlet, they will each intercept one-half the design flow when they are located so that the depth of ponding at the flanking drop inlets is 63 percent of the depth of ponding at the low point. If the flanking drop inlets are not the same size as the primary drop inlet, it will be necessary to either develop a new factor or test a trial and error solution using assumed depths with the weir equation to determine the capacity of the flanking drop inlet at the given depths.

7.6. Drop Inlet Design Discharge

The design discharge for drop inlet design should be determined based on the following procedures:

1. Outline the drainage area on a map with a scale of not less than 1 inch = 500 feet.
2. Outline the drainage area tributary to each proposed drop inlet, designating this area with the corresponding sub-area number and with a letter (2A, 2B, 2C, etc.). Drainage areas should be differentiated by color or line type.
3. Calculate the tributary area in acres for each drop inlet or battery of drop inlets.
4. Assuming satisfactory drainage area relationships, the drop inlet design discharge will be calculated as follows:

$$Q_{DES} = \frac{Q_p}{A_T} A \qquad \text{7-20}$$

Where:

A = Drop inlet tributary drainage area, in acres.

A_T = Total drainage area in acres of the appropriate sub-area.

Q_p = Peak discharge from appropriate sub-area calculated by methods presented in Chapter 4 of this manual, in cubic feet per second.

In cases where the main line design discharge is reduced because of a restricted outlet, the drop inlet design discharge must be reduced by the same percentage.

If, during the design of a project, it is determined that the proposed drop inlet interception points will change the interception points assumed in the main line hydrology, then the main line discharge should be adjusted accordingly.

7.6.1. Computer Software

It is encouraged that designers use computer models capable of inlet capacity analysis. Computer models acceptable to the City of El Paso include:

- Haestad Methods FlowMaster.
- Haestad StormCAD.
- Other programs that follow HEC 22 methodology.

If the designer desires to compute drop inlet capacity using manual calculations, the methods should follow the Hydraulic Engineering Circular No. 22 Second Edition and this manual.

7.6.2. Inlet Flow Calculation Sheet

The following is a procedure on calculation of inlet flow design (See Table 7-1).

Column 1. Inlet number. All inlets are classified with a designated number.

Column 2. Drainage area number. List all numbers of the drainage areas that drain storm water into inlet number in Column 1.

Column 3. The corresponding discharge from the drainage areas in Column 2.

Column 4. The carry-over flow (Q_{pass}) in this column is the quantity of water that has passed by the last preceding inlet to the inlet under consideration.

Column 5. The total run-off, Q_a, is the run-off from Column 3 plus the carry-over from preceding drainage areas.

Column 6. The slope, S, expressed in percentage, is obtained from established grade lines as shown on the plan-profile sheets or from specified data.

Column 7. Gutter depression.

Column 8. The water depth, y_o, in the gutter is expressed in feet.

Column 9. The value of the ponded width is the product of the water depth (in Column 7) and the reciprocal of the cross slope. The ponding width must be kept within the maximum permissible ponded limit of the streets.

Column 10. The adjusting gross capacity factor for inlets on a grade is 0.70 or 70% (30% partially clogged inlet) and the adjusting gross capacity factor for inlets on a sump is 0.50 or 50% (50% partially clogged inlet).

Column 11. Q_a/L_a is read from Figure 7-7 by the gutter depression and gutter flow depth.

Column 12. L_a is calculated from Q_a, divided by the value in Column 11. L_a represents the length of an inlet for 100 percent interception.

Column 13. Length of the inlet L.

Column 14. The ratio of L/L_a.

Column 15. The ratio of gutter depression (in feet) to water depth in the gutter (in feet).

Column 16. The ratio of Q/Q_a. The value is read from Figure 7-8.

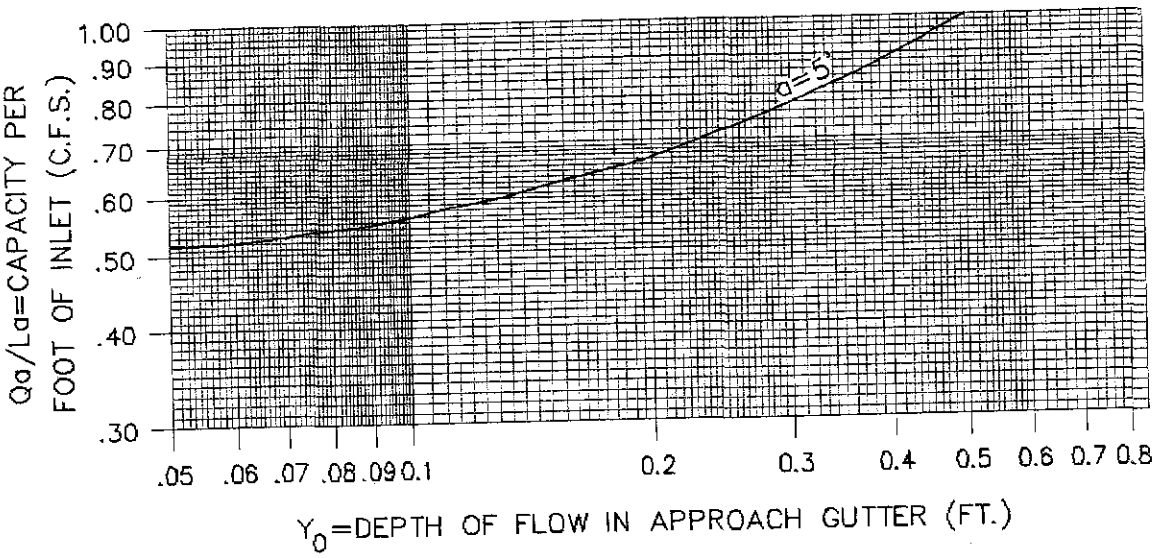
Column 17. Q is the flow intercepted by the inlet of length L.

Column 18. The carry-over flow (Q_{pass}) is the result of Q_a-Q.

Column 19. This column is used to specify the inlet information.

Table 7-1 Example Inlet Computation Sheet

1 Inlet Number	2 Drainage Area number	3 Q (cfs)	4 Q pass	5 Q total Qa	6 Slope (S)	7 Gutter Depression	8 Water Depth (feet)	9 Ponded Width	10 Clogging Factor	11 Qa/La	12 La	13 Length (feet)	14 L/La	15 a/yo	16 Q/Qa	17 Q (cfs)	18 Qpass	19 Remarks

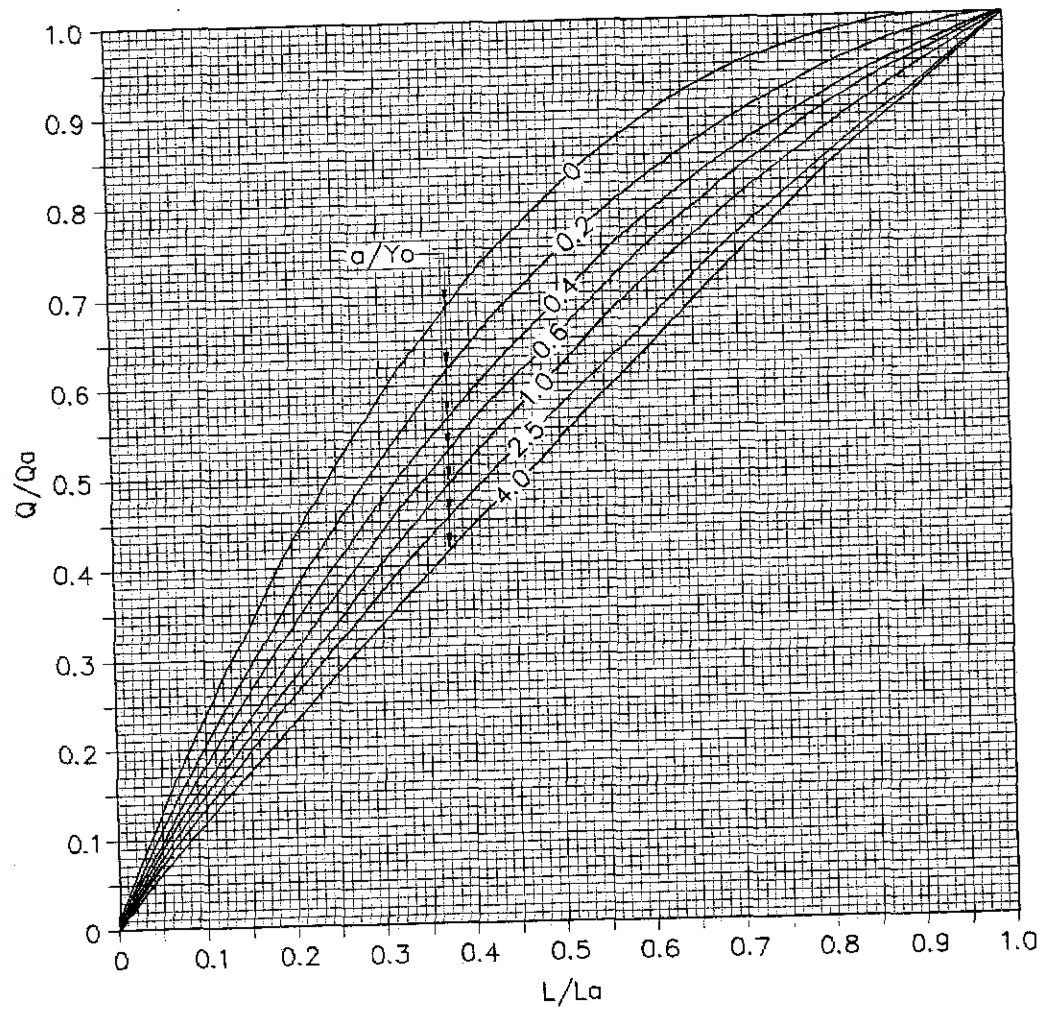


$$Q_a/L_a = 0.7\left[\frac{1}{H_1-H_2}\right]\left[(H_1)^{\frac{5}{2}}-(H_2)^{\frac{5}{2}}\right]$$

$H_1 = a + y_0$

$H_2 = a =$ GUTTER DEPRESSION

Figure 7-8: Water Depth, y_o



L = LENGTH OF CURB OPENING (FT.)
La = LENGTH OF CURB OPENING FOR 100% INTERCEPTION (FT.)
Q = FLOW INTERCEPTED BY INLET OF LENGTH "L" (C.F.S.)
Qa = TOTAL FLOW IN APPROACH GUTTER (C.F.S.)
a = GUTTER DEPRESSION (FT.)
Yo = DEPTH OF FLOW IN APPROACH GUTTER

Figure 7-9: Ratio of Q/Qa

7.6.3. Connector Pipe and "V" Depth Calculations

Connector Pipe

- The minimum diameter of connector pipe is 18 inches.
- The horizontal alignment of connector pipes must not contain angle points or bends.
- Connector pipes outletting into a pipe from both sides of a street should be offset 8 feet or more at the main line. Connections at manholes or junction structures are preferred.
- The drop inlet spacing shall be a minimum of 25 feet between curb transitions.

- Drop inlet connector pipes shall outlet at the downstream end of the drop inlets, unless prevented by field conditions. Downstream, in this paragraph, refers to the directions of the gutter slope at the drop inlet in question.
- Where feasible, connector pipes should be located so as to avoid, as much as possible, cutting into existing cross gutters and spandrels.
- The conversions of inlet types will not be permitted. If the drop inlet is in conflict with a driveway, the drop inlet will be removed and replaced with another inlet outside of the driveway. To avoid conflicts with driveways, the engineer should identify the proposed driveways on the grading plan when drop inlets front the lots.

Single Drop inlets

Given the available head (H), the required connector pipe size can be determined from culvert equations, such as those given in King and Brater, *Handbook of Hydraulics, Section Four, 5th Edition.*

Figure 7-10 provides a schematic of variables for a single drop inlet configuration. The minimum drop inlet "V" depth should be determined as follows:

$$V = C.F. + 0.5 + 1.2 * \frac{V^2}{2g} + \frac{d}{\cos(S)} \qquad \text{7-21}$$

Where:

V = Depth of the drop inlet, or "V" depth, measured from the invert of the connector pipe to the top of the curb, in feet.

C.F. = Vertical dimension of the curb face at the drop inlet opening, in feet.

v = Average velocity of flow in the connector pipe, in feet per second, assuming a full pipe section.

d = Diameter of connector pipe, in feet.

S = Slope of connector pipe in feet per feet.

The term $1.2(V^2/2g)$ includes an entrance loss of 0.2 of the velocity head.

Assuming a curb face at the drop inlet opening of 10 inches, which is the value normally used, and cosine S = 1, the above equation may be simplified to the following:

$$V = 1.33 + 1.2 * \frac{V^2}{2g} + d \qquad \text{7-22}$$

To determine the required supporting strength of concrete pipe installed under asphalts, other flexible pavements, or relatively shallow earth cover, it is necessary to evaluate the effect of live loads, such as highway truck loads, in addition to dead loads imposed by soil and surcharge loads. Refer to *Concrete Pipe Design Manual*, American Concrete Pipe Association (ACPA), June 2000, for design methodology. For expedited calculations, ACPA has developed the PipPac 2000 computer program.

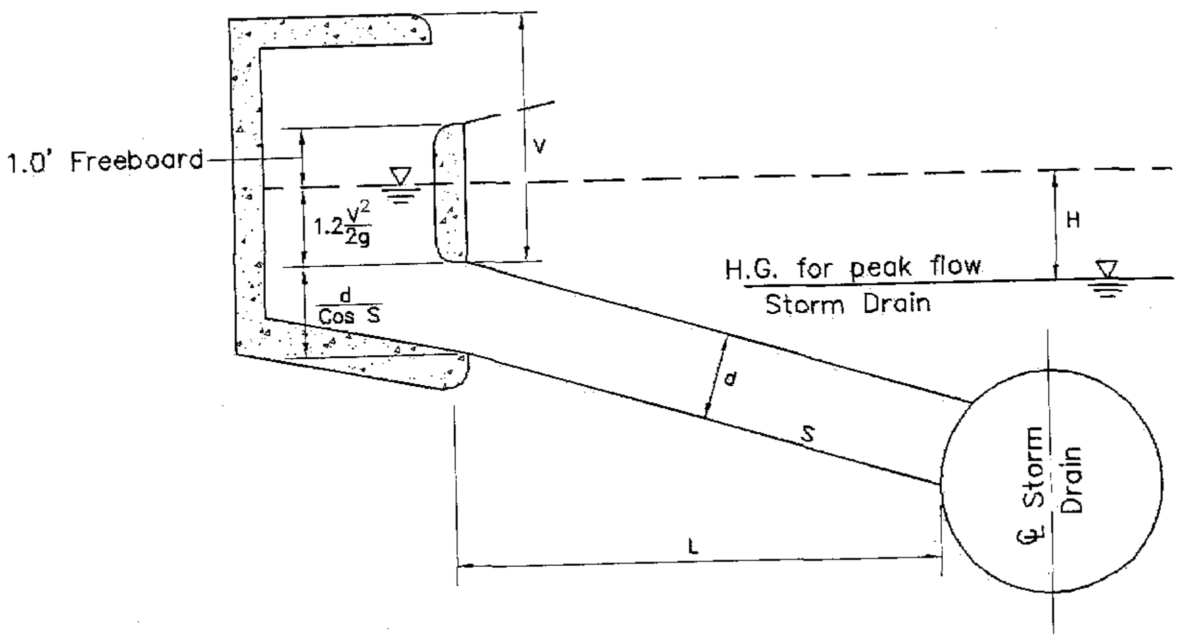


Figure 7-10: Single Drop Inlet

Figure 7-11 provides a schematic of variables for drop inlets in series.

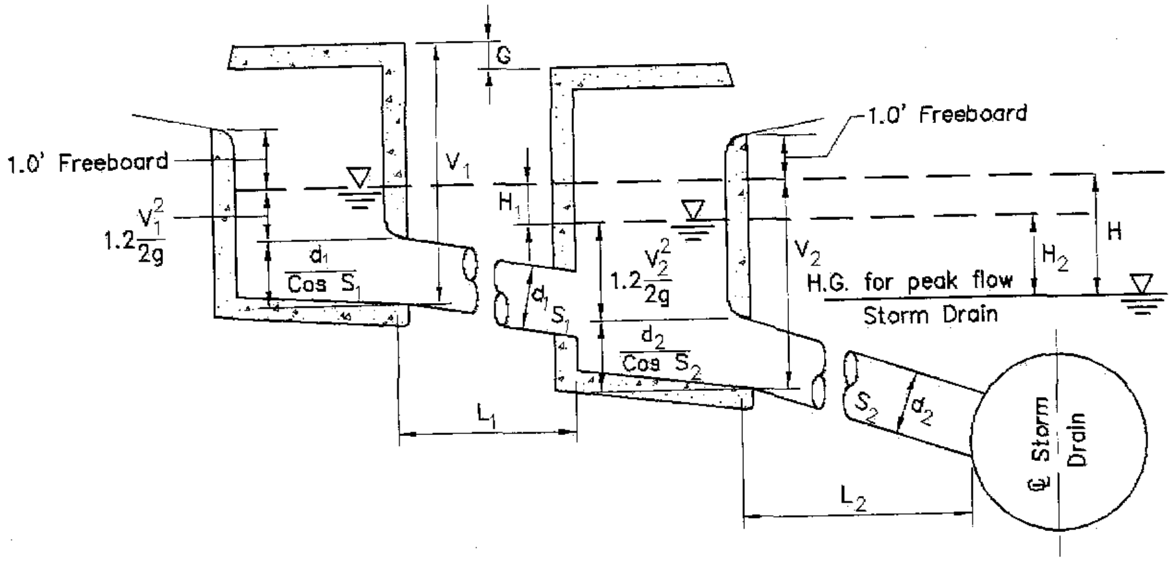


Figure 7-11: Drop inlets in Series

To determine the "V" depth for drop inlets in series, first select a connector pipe size for each drop inlet and determine the related head loss (H1, H2) by means of a culvert equation. The sum of head losses in the series should not exceed the available head, i.e.:

$$H_1 + H_2 + ...H_n \leq H \quad \text{7-23}$$

The minimum drop inlet "V" depths are determined in the following manner. The first drop inlet "V" depth is calculated as for a single drop inlet:

$$V_1 = 1.33 + 1.2 * \frac{V_1^2}{2g} + d_1 \quad \text{7-24}$$

The second drop inlet "V" depth is determined as follows:

$$V_2 = C.F._{\cdot 1} + 0.5 + H_1 + 1.2 * \frac{V_2^2}{2g * \cos(S_2)} + d_2 - G \quad \text{7-25}$$

Assuming again that C.F. = 0.83 and Cos S_2 = 1,

$$V_2 = 1.33 + H_1 + 1.2 * \frac{V_2^2}{2g} + d_2 - G \quad \text{7-26}$$

The freeboard provided for the second drop inlet generally should not be less than 0.5 feet and shall be checked as follows:

$$FB_2 = V_2 - \frac{d_2}{\cos(S_2)} - 1.2 * \frac{V_2^2}{2g} - C.F._{\cdot 2} \quad \text{7-27}$$

If C.F.$_{\cdot 2}$=0.83 and Cos S_2=1

$$FB_2 = V_2 - d_2 - 1.2 * \frac{V_2^2}{2g} - 0.83 \quad \text{7-28}$$

Where especially "tight" conditions prevail, the 0.5 feet freeboard requirement referred to above may be omitted. In such cases the difference between the gutter elevation and the hydraulic grade line elevation of the main line will be accepted as the available head.

Connector pipes between drop inlets in series should be checked for adverse slope by the following relationship. The value of 0.5 shown below is the standard 6-inch cross slope of the drop inlet floors.

$$V_2 - 0.5 > V_1 - G \quad \text{7-29}$$

8. Open Channel

8.1. Hydraulics of Open Channel Flow

An open channel is a conveyance system in which water flows with a free surface at the water atmosphere interface. The channel may be either a natural watercourse or an artificial, "engineered" conveyance. Natural streams typically consist of a main flow channel, often termed the thalweg, and adjacent floodplains. Artificial channels are used for a wide variety of applications varying in scale from modest roadside ditches to large conveyance facilities that can be up to several hundred feet wide. Design guides are provided for the analysis of both natural and engineered channels.

The state of open channel flow is governed by the effects of viscosity and gravity relative to the inertial forces of the flow. The effect of gravity on the state of flow is represented by a ratio of inertial forces to gravity forces. This ratio is given by the Froude number, *Fr*, defined as:

$$Fr = \frac{V}{\sqrt{gd}} \quad \text{8-1}$$

Where:

V = Mean velocity, in feet per second.

g = Acceleration of gravity, 32.2 feet per second squared.

d = Hydraulic depth, in feet (which is the cross sectional area of the water, A [square feet], divided by the width of the free surface, T [in feet].

When F_r is equal to 1, the flow is in the critical state. This flow condition is unstable and flow depths at or near critical depth should be avoided. If F_r is less than 1, the flow is subcritical and gravity forces dominate. When F_r is greater than 1, the flow is supercritical and inertial forces predominate.

8.1.1. Energy

Newton's Second Law of Thermodynamics states that energy can neither be created nor destroyed; it can only be transformed. Thus, in the case of an open channel carrying a steady flow, the total energy at any two points must be equal. At a given cross section, the total energy at any point is the sum of kinetic and potential energy at that point, as illustrated in Figure 8-1.

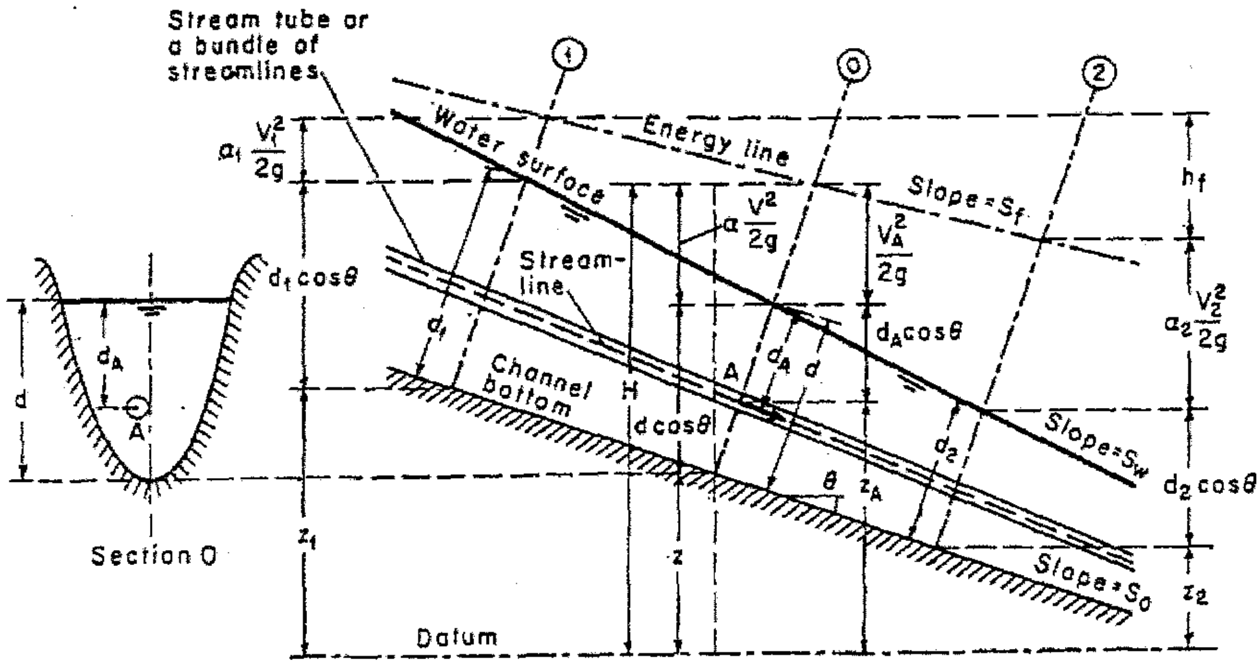


Figure 8-1: Energy in Gradually Varied Open-Channel Flow

The following relationship is readily deduced from Figure 8-1:

$$E_T = \alpha_1 \frac{V_1^2}{2g} + y_1 + z_1 = \alpha_2 \frac{V_2^2}{2g} + y_2 + z_2 + h_f \qquad 8\text{-}2$$

Where: $y_1 = d_1 \cos\theta$

The velocity head coefficient, α, is a correction to account for the non-uniformity of the velocity in the channel. Experimental data indicates this value varies between 1.03 and 1.36 for fairly straight, prismatic channels. The value is generally higher for small channels and lower for larger streams of considerable depth. For channels of regular cross section and fairly straight alignment, the effect of non-uniform velocity distribution on the computed velocity head is small, especially when compared to other uncertainties involved in the computation. Therefore, α is often assumed to be 1.0. Additionally, experience indicates that using the average velocity often gives satisfactory accuracy for usual open channel flow conditions. However, in some cases it may be desirable to use the computed value of the energy coefficient α. Kinetic energy (KE) is estimated by Equation 8-3.

$$KE = \alpha \frac{\bar{v}^2}{2g} \qquad a \geq 1.0 \qquad 8\text{-}3$$

Where:

$$a = \frac{\sum v^3 \Delta A}{\bar{v}^3 A} \qquad 8\text{-}4$$

Example: The flow in a river downstream of a bridge constriction is as shown. Calculate the energy coefficient.

$$Q_T = (.5)(1)(200) + (10)(15)(20) + (.5)(1)(200) = 3200 \text{ cfs}$$

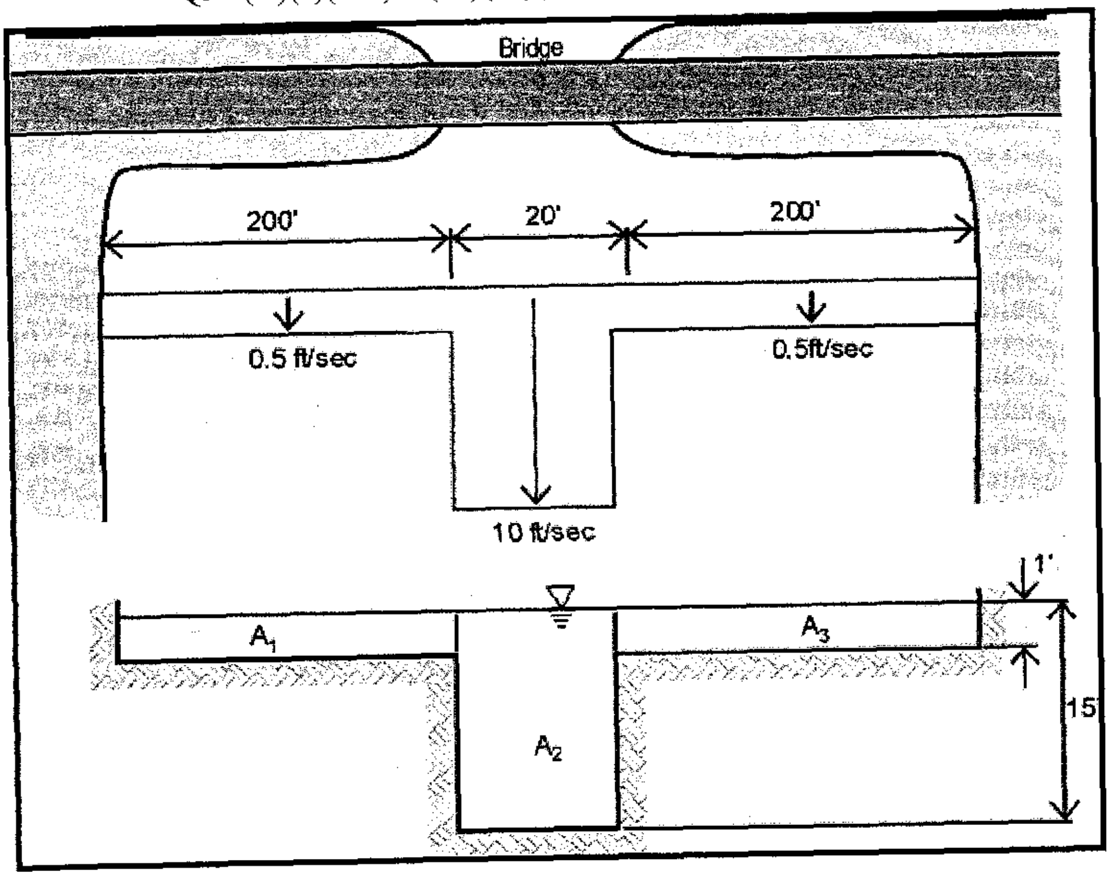


Figure 8-2: Energy Coefficient Example

$$A = A_1 + A_2 + A_3 = 200 + 300 + 200 = 700 ft^2$$

$$\bar{v} = \frac{Q}{A} = \frac{3200}{700} = 4.57 ft/\sec$$

$$\alpha = \frac{\sum v^3 \Delta A}{\bar{v}^3 A} = \frac{(0.5)^3(200) + (10)^3(300) + (0.5)^3(200)}{(4.57)^3(700)}$$

$$\alpha = 4.49$$

If the datum is the invert of the channel at Section 2, $z_2 = 0$ and $z_1 = S_o l$, where l is the channel length between Sections 1 and 2. The energy lost due to friction is represented as $h_f = S_f l$. Making these substitutions, Equation 8-2 reduces to the following:

$$E = \alpha \frac{V_1^2}{2g} + y_1 + s_{o1} = \alpha \frac{V_2^2}{2g} + y_2 + S_{f1} \qquad \text{8-5}$$

Equation 8-5 is the basis for calculating water surface profiles, which will be discussed in more detail in Section 8.1.5.

8.1.2. Specific Energy

Specific energy in a channel section is defined as the energy per pound of water at any section of a channel measured with respect to the channel bottom and may be expressed as:

$$E = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2} \quad 8\text{-}6$$

Where:

V = Mean velocity, in feet per second.

g = Acceleration of gravity, 32.2 feet per second squared.

y = Flow depth, in feet.

When the depth of flow is plotted against the specific energy for a given channel section and discharge, a specific energy curve is obtained. The specific energy curve, as shown in Figure 8-3, has two limbs, AC and BC. The limb AC approaches the horizontal axis asymptotically toward the right. The limb BC approaches the line OD as it extends upwards and to the right. The line OD has an angle of inclination equal to 45°. At any point, P, on this curve, the ordinate represents the depth of flow, and the abscissa represents the specific energy that is equal to the sum of the pressure head, y, and the velocity head, $V^2/2g$. The curve shows that, for a given specific energy, there are two possible depths, the low stage, y_1, and the high stage, y_2. The low stage is called the alternate depth of the high stage and vice versa.

At point C, the specific energy is a minimum and the stage is at critical depth. When the depth of flow is greater than the critical depth, the velocity of flow is less than the critical velocity and the flow is subcritical. When the depth of flow is less than the critical depth, the flow is supercritical. Inspection of channels in the vicinity of critical depth reveals that a small change in the energy will result in a relatively large change in the depth of flow. For this reason, it is strongly recommended that flow depths producing Froude numbers between 0.87 and 1.13 be avoided.

Due to safety concerns resulting from excessively high velocities and intractable hydraulic forces, the desirable design of a channel has a Froude number of just under 2.0. In areas within the City of El Paso, this is not always possible because of steep terrain. If the Froude number exceeds 2.0, any small disturbance to the water surface is amplified in the course of time and the flow tends to proceed as a series of "roll waves."

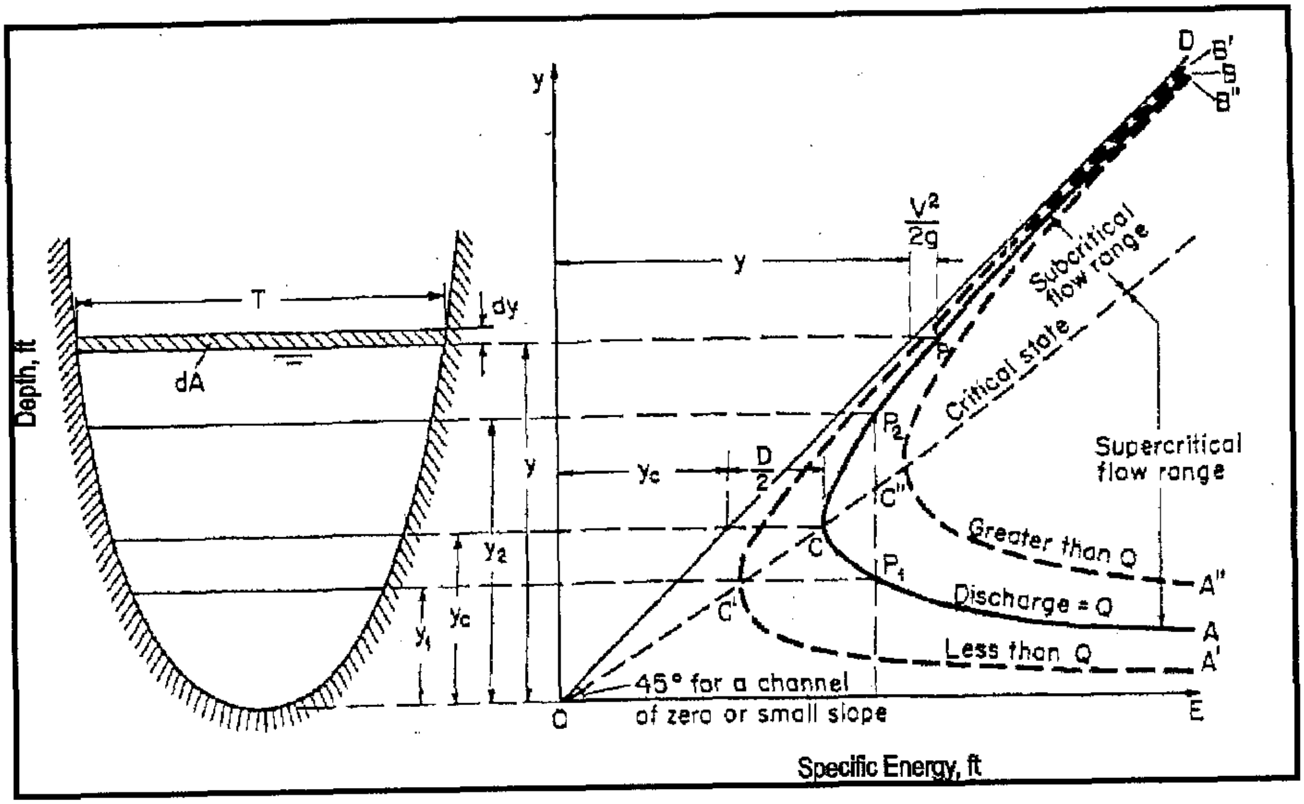


Figure 8-3: Specific Energy Curve

8.1.3. Flow Classification

Open channel flow is classified into many types and described in various ways based upon how the flow varies spatially and temporally. A steady flow is one in which all conditions at any point in a stream remain constant with respect to time (Daugherty and Franzini, 1977). Steady flow is often more simply defined as a constant flow rate producing a constant depth of flow at a given point in a channel for the time period under consideration. Conversely, the flow is unsteady if the flow conditions such as depth change with time. Thus, time is the criterion in the determination of steady and unsteady flow. In most open channel design problems, only steady flow conditions are considered.

Space is the criterion in the determination of uniform and varied flow. A truly uniform flow is one in which the velocity is the same in both magnitude and direction at a given instant at every point in the fluid (Daugherty and Franzini, 1977). Open channel flow is often considered uniform if the flow depth is the same at every point along the channel. Flow is nonuniform where it is spatially varied or discontinuous; that is, discharge varies or other flow conditions change along the course of flow. Uniform flow may be steady or unsteady, depending on whether or not the flow conditions change with time. Uniform flow is also called normal flow and the flow depth under uniform flow conditions is referred to as normal depth. Refer to Section 8.1.5 for more detailed information in regard to the computation of normal depth.

Flow is varied if the flow conditions, such as depth, change along the length of the channel. If the depth varies at points along the channel, it will do so either rapidly or gradually, depending upon the channel geometry and flow constraints. The flow is rapidly varied if the depth changes abruptly over a relatively short distance. Examples of rapidly varied flow include local phenomena, such as hydraulic jumps and hydraulic drops. Under steady flow conditions, if the depth of flow along the length of the channel gradually increases or decreases, it is gradually varied. This is the usual condition in open channel flow. Gradually varied flow occurs under either subcritical or supercritical flow regimes. Water surface profile computations are required to estimate the depth of flow for varied flow conditions at any given location as described in Section 8.1.5.2.

Critical depth in an open channel has the following characteristics:

- For a given flow rate, the specific energy is at a minimum.
- The discharge is at a maximum for a given specific energy.
- The velocity head is one half of the flow depth.
- The Froude number is 1.0.

By substituting $V^2 = Q^2/A^2$ into Equation 8-1 and rearranging, we can obtain a general expression for critical depth that is applicable to any channel cross section:

$$\frac{Q^2}{g} = \frac{A^3}{T} \qquad 8\text{-}7$$

Example: What is the critical depth of flow for 400 cfs flowing in a rectangular channel 10.0 feet wide?

$$\frac{400^2}{32.2} = \frac{(10y_c)^3}{10}$$
$$y_c = 3.68\,ft$$

Flows producing Froude numbers less than 1.0 are subcritical and have the following general characteristics relative to critical depth:

- Slower velocities.
- Greater depths.
- Lower hydraulic losses.
- Less erosive power.
- Less sediment carrying capacity.
- Behavior easily described by relatively simple mathematical equations.
- Surface waves propagate upstream.

Flows with Froude numbers greater than 1.0 are supercritical and have the following general characteristics relative to critical depth:

- Higher velocities.
- Shallower depths.
- Higher hydraulic losses.

- More erosive power.
- More sediment carrying capacity.
- With few exceptions, behavior cannot be easily predicted mathematically.
- Surface waves propagate downstream only.

For any flow, the discharge, Q, at a channel section is expressed by:

$$Q = AV \qquad 8\text{-}8$$

Where V is the mean velocity (feet per second) and A is the cross sectional area of the flow measured normal to the direction of flow (square feet). Under steady flow conditions, the discharge is constant and:

$$Q = A_1V_1 = A_2V_2 \qquad 8\text{-}9$$

The subscripts denote different channel sections. Equation 8-9 is known as the Continuity Equation and is applicable to the flow conditions addressed in this chapter. Obviously, Equation 8-9 is invalid for unsteady flow conditions in which discharge increases nor decreases along the course of flow. Examples of unsteady flow are flood waves, bores, roadside gutters, side-channel spillways, wash water troughs in filters, and effluent channels around sewage treatment tanks. Precise treatment of unsteady flow is mathematically complicated and beyond the scope of this chapter.

8.1.4. Uniform Flow

8.1.4.1. Manning's Equation

The most commonly used equations for analysis of open channel flow express mean velocity of flow as a function of the roughness of the channel, the hydraulic radius, and the slope of the energy gradient. They are empirical equations in which the values of constants and exponents have been derived from experimental data. Manning's equation is one of the most widely accepted and commonly used of the open channel equations:

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2} \qquad 8\text{-}10$$

Substituting Equation 8-8 and rearranging yields the familiar form of Manning's equation:

$$Q = \frac{1.486}{n} AR^{2/3} S_f^{1/2} \qquad 8\text{-}11$$

The Manning's roughness coefficient (n value) is a measure of the frictional resistance exerted by a channel on the flow. The n value can also reflect other energy losses such as those resulting from unsteady flow, extreme turbulence, and transport of suspended material and debris that are difficult or impossible to isolate and quantify. The reader is referred to Barnes (1967) and Thomsen and Hjalmarson (1991) for discussion of the estimation of n values for natural and composite channels. Table 8-1 provides commonly used Manning's n values.

The most common error in the application of Manning's equation is to substitute the bed slope of the channel, S_o, for the slope of the energy gradient, S_f. This substitution is correct only when the two gradients are parallel, as in the case of uniform flow. For a given condition of n, Q, and S_o, uniform flow is maintained only at normal depth. Normal depth rarely occurs in nature, and it is primarily a theoretical concept that simplifies the computation and analysis of uniform flow. Table 8-2 lists the algebraic expressions for computing the hydraulic geometry for typical channel sections.

Table 8-1: Manning's Roughness Coefficients

Channel Material	Roughness Coefficient (n)		
	Minimum	Normal	Maximum
Concrete:			
Trowel finish	0.011	0.013	0.015
Float finish	0.013	0.015	0.016
Unfinished	0.014	0.017	0.020
Shotcrete, good section	0.016	0.019	0.023
Shotcrete, wavy section	0.018	0.022	0.025
Soil Cement	0.018	0.020	0.025
Constructed channels with earthen bed:			
Clean earth; straight	0.018	0.022	0.025
Earth with grass and forbs	0.020	0.025	0.030
Earth with sparse trees and shrubs	0.024	0.032	0.040
Shotcrete	0.018	0.022	0.025
Soil cement	0.022	0.025	0.028
Concrete	0.017	0.020	0.024
Riprap	0.023	0.032	0.036

Source: Simons, Li and Associates, 1988. Adapted from Chow (1959) and Aldridge and Garret (1973)

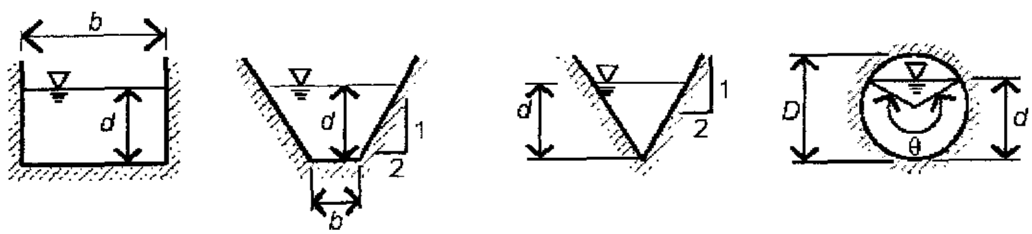
Table 8-2: Typical Channel Sections

Channel Section	**Area**	Wetted Perimeter	Hydraulic Radius	Top Width
Rectangle	bd	$b + 2d$	$\dfrac{bd}{b+2d}$	b
Trapezoid	$bd + zd^2$	$b+2d\sqrt{z^2+1}$	$\dfrac{bd+zd^2}{b+2d\sqrt{z^2+1}}$	$b+2zd$
Triangle	zd^2	$2d\sqrt{z^2+1}$	$\dfrac{zd}{2\sqrt{z^2+1}}$	$2\ zd$
Circular < 1/2 full (2)	$\dfrac{D^2}{8}\left(\dfrac{\pi\theta}{180}-\sin\theta\right)$	$\dfrac{\pi D\theta}{360}$		$D\sin\theta$ *or* $2\sqrt{d(D-d)}$
Circular > 1/2 full (3)	$\dfrac{D^2}{8}\left(2\pi-\dfrac{\pi\theta}{180}+\sin\theta\right)$	$\dfrac{\pi D(360-\theta)}{360}$	$\left(\dfrac{45D}{\pi(360-\theta)}\right)^{*}\left(2\pi-\dfrac{\pi\theta}{180}+\sin\theta\right)$	$D\sin\theta$ *or* $2\sqrt{d(D-d)}$

(1) After USDA Soil Conservation Service ES-33

(2) $\theta = 4\sin^{-1}\sqrt{d/D}$ Insert θ in degrees

(3) $\theta = 4\cos^{-1}\sqrt{d/D}$ Insert θ in degrees



Rectangle Trapezoid Triangle Circular

Example:

A trapezoidal channel (as shown in Table 8-2) has the following characteristics:

S_o= 0.01
b = 2.62 ft
z = 3
d = 1.64 ft

Find the channel capacity and flow velocity for the following concrete channel lining:

n=0.013

A = bd + 2(1/2)(d)(zd)
= bd + zd2
= (2.62)(1.64) + (3)(1.64)2
= 12.4 ft2

P = b + 2[(zd)2 + d2)]1/2
= b + 2d(z2+1)0.5
= (2.62) + (2)(1.64)(32+1)0.5
= 13.0 ft

R = A/P
= 12.4/13.0
= 0.95 ft

Qn= 1.49 A R0.67 So0.5

= (1.49)(12.4)(0.95)0.67(0.01)0.5
= 1.79 ft3/s

Q = Qn / n
= 1.79/0.013
=137.7 ft3/s

V = Q/A
= 137.7/12.4
= 11.1 ft/s

8.1.4.2. Composite Channels

The cross section of a natural or artificial watercourse or a street right-of-way may be composed of several distinct subsections, with each subsection having different hydraulic characteristics, such as hydraulic roughness and average flow depth. For example, a natural alluvial channel may have a primary, sand-bed channel that is bounded on both sides by densely-vegetated, overbank floodplains, or an urban flooded street section may be bounded on both sides by landscaped front yards having shallower flow depths and slower flow velocities. In composite channels like these, the discharge is computed for each subsection having distinct and different hydraulic characteristics, and the total computed discharge is set equal to the sum of the individual discharges. Similarly, the mean velocity for the entire flow cross section is assumed to be equal to the total discharge divided by the total water area. Open Channel Hydraulics (Chow, 1959) provides an example of computing flow in channels having composite roughness. In the urban setting, it is not unusual for buildings and other structures to occupy a significant portion of any given hydraulic cross section. Under these circumstances, it is often difficult to estimate both the effective width of the cross-section and the Manning's roughness coefficient for the overbank areas. Given this situation, the engineer should eliminate the portion of the cross section occupied by the building. Where only an estimate of the computed water surface elevation is needed, a second option may be selected. An adjusted urban roughness coefficient, n_u, may be computed and applied to the total cross-sectional area (Hejl, 1977). See Figure 8-4.

$$n_u = n_o\left(1.5\left(\frac{W_T}{\sum W_o}\right)+\left(1-\frac{W_T}{\sum W_o}\right)\frac{\sum L_o}{L_T}-0.5\right) \qquad 8\text{-}12$$

where all coefficients are defined in Table 8-1.

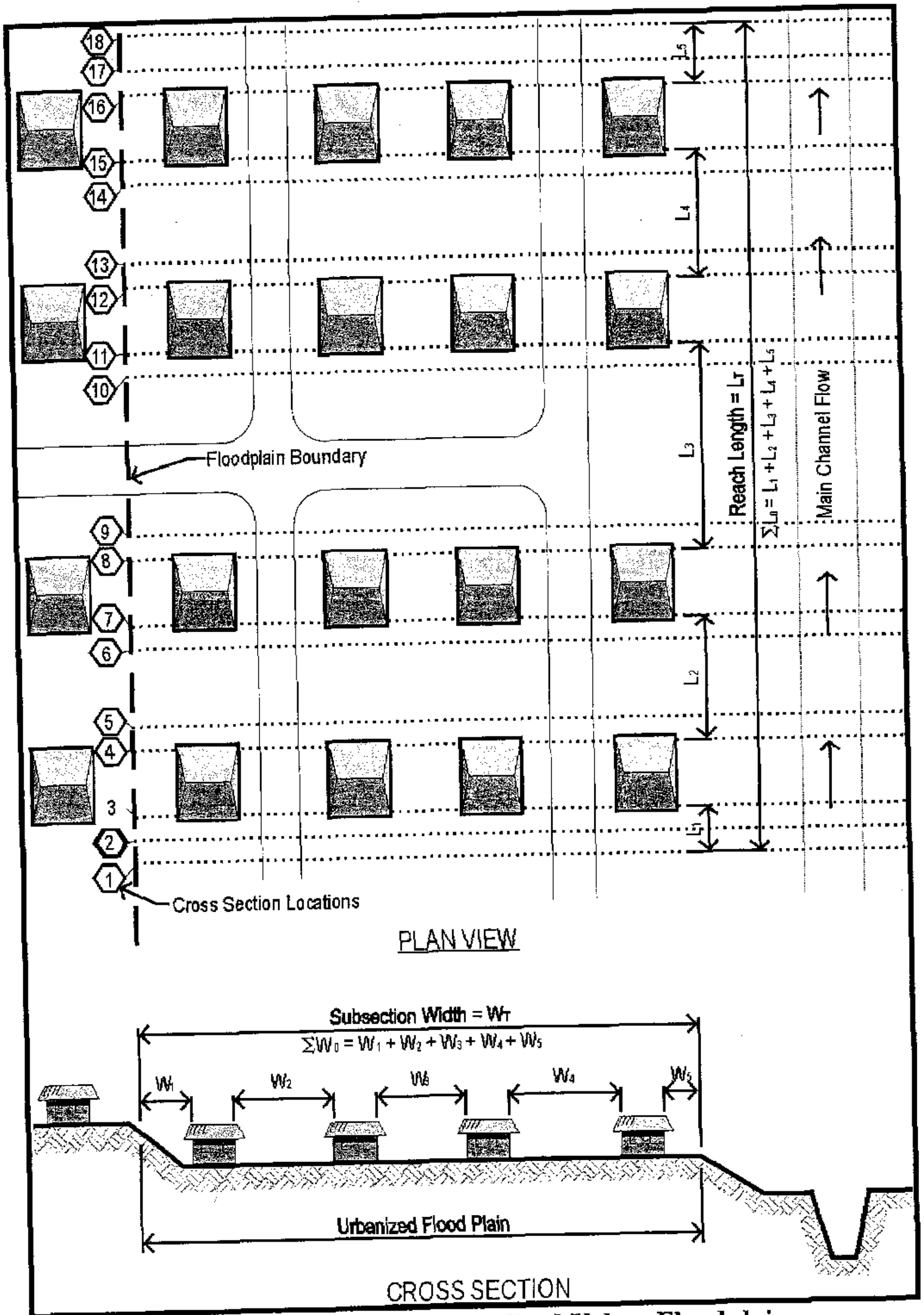


Figure 8-4: Diagram of Idealized Urban Floodplain

The following examples illustrate the concept of normal depth and the selection of the roughness coefficient.

Example: What is the hydraulic capacity of a shotcrete lined channel with a 20-foot bottom width, 2:1 side slopes, an invert gradient of 0.0016 ft/ft, and a uniform flow depth of 4.0 feet?

Select the appropriate expressions from Table 8-2 for the cross section area and the hydraulic radius:

$$A = bd + zd^2 = (20)(42) = 112\,ft^2$$

$$R = \frac{bd + zd^2}{b + 2d\sqrt{(2^{2+1})}} = \frac{(20)(4) + (2)(4^{2)}}{20 + (2)(4)\sqrt{(2^2+1)}} = \frac{112}{37.98} = 2.96\,ft$$

Select the appropriate Manning's roughness coefficient from Table 8-1. Substitute these values in Equation 8-11:

$$Q = \frac{1.49}{n} AR^{2/3} S_f^{1/2}$$

$$Q = \frac{1.49(112.0)(2.96)^{2/3}(0.0016)^{1/2}}{0.022} = 625.5$$

Example: What is the normal depth of flow in a shotcrete lined channel with a 20-foot bottom width, 2:1 side slopes, an invert gradient of 0.0016 ft/ft, and a steady flow rate of 625 cfs?

Rearrange Equation 8-11:

$$AR^{2/3} = \frac{Q_n}{1.49S^{1/2}} = \frac{(625)(0.022)}{(1.49)(0.0016^{1/2})} = 230.70 = \left(20d + 2d^2\right) \times \left[\frac{\left(20d + 2d^2\right)}{20 + (2d)\sqrt{(2^2+1)}}\right]^{2/3}$$

By trial and error solution, d = 4.0 ft

8.1.5. Gradually Varied Flow

8.1.5.1. Classification of Water Surface Profiles

Chow (1959) describes the classification of these flow profiles into fifteen different types according to the nature of the channel slope and the zone in which the flow surface for a given discharge lies. These water surface profile types are designated according to an alphanumeric protocol, as follows:

- The letter is descriptive of the slope (i.e., H for horizontal, M for mild, C for critical, S for steep (supercritical), and A for adverse slope)
- The numeral represents the zone number, where:
  - Zone 1 – Water surface above both normal and critical depths.
  - Zone 2 – Water surface between normal and critical depths.
  - Zone 3 – Water surface below both normal and critical depths.

These types are designated as H1, H2, H3; M1, M2, M3; C1, C2, C3; S1, S2, S3; and A1, A2, A3, as shown in Figure 8-5.

Flow profile analysis enables the designer to predict the general shape of the flow profile for a given channel layout. This step is a significant part of the open channel design process and it should not be omitted. Flow profile analysis will serve to identify control sections and to provide a work plan for more detailed design calculations.

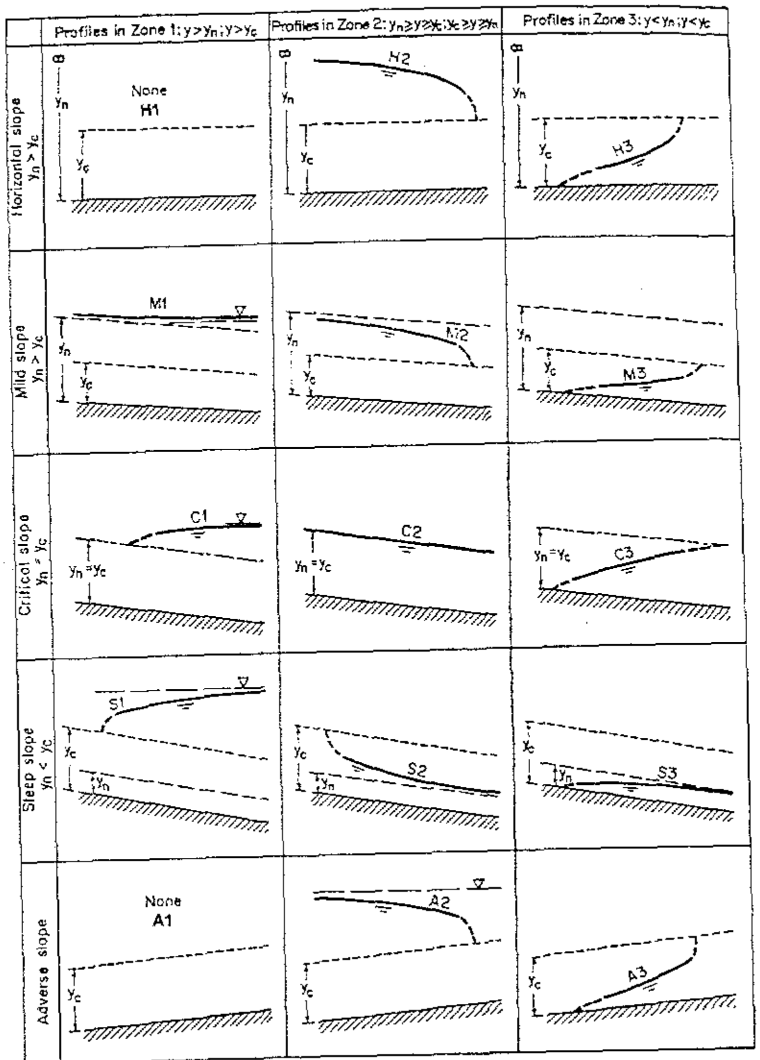


Figure 8-5: Classification of Flow Portion of Gradually Varied Flow

8.1.5.2. Calculation of Water Surface Profiles

Section 8.1.4 presents methods for calculation of normal depth that assume uniform flow. However, sudden changes in discharge, bed slope, and cross sectional area and/or form will produce additional energy losses that are not accounted for in Manning's equation. This may be particularly true in cases of sudden contractions and expansions of the channel cross section.

In those instances where an upstream or downstream hydraulic control section exists, the Standard Step Method should be used for evaluating water surface profiles. The procedure used for Standard Step calculations is presented in several of the technical references listed at the end of this DDM. The designer can perform the Standard Step calculations either manually using standard forms, or digitally using readily available and well-documented computer programs such as HEC-2 (USACE, 1990) or HEC-RAS (USACE, 2001a & b). (These programs were developed by the USACE and are available through the Corps website at http://www.hec.usace.army.mil).

One advantage of the Standard Step Method is the ability to converge an actual water surface profile for the study reach without needing to know the precise starting water surface elevation. If the computation is started at an assumed elevation that is incorrect for the given discharge, the resulting flow profile will approach the correct water surface elevation with each succeeding cross section evaluated within a study reach. If no accurate elevation is known within or near the reach under consideration, an arbitrary elevation may be assumed at a cross section far enough away from the "starting" cross section in the study reach to compensate for any initial error.

The step computations should be carried upstream if the flow is subcritical, and downstream if the flow is supercritical. Otherwise, step computations carried in the wrong direction will result in a profile that diverges from the actual water surface profile.

For natural streams flowing under supercritical conditions, critical depth should be used for determining the water surface profile. Using the critical depth will produce higher, and thus more conservative, water surface elevations for design purposes. Velocities computed for the supercritical profile will be higher and more conservative and, therefore, should be used to evaluate scour potential and other velocity critical design features such as superelevation and freeboard.

The reader is referred to the technical references listed in Chapter 19 for more information regarding application of the Standard Step Method and/or use of computer models, such as HEC-2 and HEC-RAS, for computation of water surface profiles. Specific references most instructive in this subject include Chow (1959) and USACE (1990, 2001a, 2001b), among others.

8.2. Design Considerations for Open Channel

8.2.1. Channel Geometric Controls

Good design practice requires that several issues be addressed. Unless exempted by the governing agency, water surface profiles must be computed for all channels during final design and must be clearly shown on a copy of the final drawings. Computation of the water surface profile should use Standard Step backwater methods (see Section 8.1.5). These computations must account for all losses due to changes in velocity, drops, bridge openings, and other factors. Computations should begin at a known point and extend in an upstream direction for subcritical flow regimes, and in a downstream direction for supercritical regimes. Concrete lined channels with supercritical flow regimes should be analyzed as described in Section 8.1.5. The energy gradient must be shown on all preliminary drawings to help check for errors; however, it is optional for final drawings. Open channel flow in urban drainage is usually non-uniform due to bridge openings, channel curves, and hydraulic structures; therefore, backwater computations must be used for all final channel design work.

The design of a safe and economical drainage system should be one of the first steps in the land development process. Drainage system requirements may determine the character of the development and often dictate the layout of streets and lots. Attention to drainage requirements during the first phases of planning will result in better land use decisions and lower maintenance costs.

A drainage system that is well planned and designed incorporates several features. The proposed drainage system should be aligned with any existing and proposed structures, such as bridges and culverts, and should be designed in such a manner that subcritical flow is maintained throughout (except at designed drop structures). The design should incorporate uniform channel properties, such as gradient and cross sectional geometry, as much as possible. Sharp and closely spaced curves should be avoided. Uncontrolled local runoff should not be allowed to enter the channel; rather, it should be collected and discharged into the channel through a structure specifically designed for that purpose. In all cases, the issue of wet and dry weather safety should be a paramount consideration in route and right-of-way determinations.

In general, all open channels should be designed with the tops of the channel walls at or below the adjacent ground to allow for interception of surface flows. If it is unavoidable to construct the channel wall at or below the adjacent ground, this will create a low spot on the landward side of the channel and also possibly create a levee condition. If a levee condition is created, then the levee must be designed in accordance with the requirements provided herein and in 44 CFR 65.10 (for example, freeboard requirements are higher for riverine levees than for channels). If a low spot is created on the landward side of the channel, then a means of draining this low spot must be provided on the drawings. All local drainage should be completely controlled. External flows must enter the channel at designated locations and through designated inlets, unless specifically otherwise authorized by the City Engineer.

The following sidewall slopes are generally the maximum values used for channels on at least one side of the concrete-lined channel. The road should be sloped away from the channel, and roadway runoff carried in a controlled manner to the channel.

Table 8-3: Maximum Sidewall Slopes

Lining Material	Maximum Slope (H:V)
Soil Cement	2:1
Concrete Lined	2:1
Concrete Lined	1:1 (*)
Vertical Concrete Wall	0:1 (*)
Grouted Rock Riprap	2:1
Dumped Rock Riprap	2:1
Earth Lined	3:1
Grass Lined (sodded)	4:1

(* = Used only with the prior approval of the City Engineer on a case by case basis)

Although concrete lined channels can be constructed at slopes steeper than 2:1, steeper slopes are not recommended due to structural and safety concerns.

8.2.1.1. Roll Waves

Roll waves, sometimes known as slug flow, are intermittent surges on steep slopes that will occur when the Froude number, *Fr,* is greater than 2.0 and the channel invert slope (S_0) is greater than the quotient, twelve divided by the Reynolds Number. When they do occur, it is important to know the maximum wave height at all points along the channel so that appropriate wall heights may be determined, based on the experimental results of roll waves by Richard R. Brock, and that maximum wave height can be estimated.

The height of roll waves can be approximated using the model of positive surges that have an advancing front with the profile similar to a moving hydraulic jump. When the height of the surge is small, the surge appears undular like an undular jump. When the height is increasing, the undulation will eventually disappear and the surge will have a sharp and steep front. Such an unsteady flow can be converted to a steady pattern by adding the wave speed to the flow field. Let the subscript 2 represent the design flow condition determined by Manning's formula for the selected channel cross section, and let the subscript 1 represent the section without roll waves defined by the limiting Froude number. Solving the continuity and momentum principles simultaneously yields:

$$V_2 = \frac{(V_1 - V_w)A_1 + V_w A_2}{A_2} \qquad \text{8-13}$$

and the wave speed in a moving jump is:

$$V_w = V_1 + \sqrt{\frac{(A_2 y_2 - A_1 y_1)g}{A_1(1 - A_1 / A_2)}} \qquad \text{8-14}$$

$$h = y_2 - y_1 \qquad \text{8-15}$$

Which

V_w = *Wave velocity, in feet per second.*
V = *Flow velocity, in feet per second.*
A = *Flow area, in square feet.*
g = *Gravitational acceleration, 32.2 feet per second squared.*
h = *Height of roll waves, in feet.*
y = *Distance to centroid of flow area, approximated by 0.5y, in feet.*

Considering that the roll waves near the center of the channel section are similar to those in a rectangular channel, the height of roll waves can be derived as:

$$h = \frac{C^2}{g}\left(\frac{2y_1}{y_1 - y_2}\right)(F_2 - F_1) \qquad \text{8-16}$$

$$C = V_w - V_2 \qquad \text{8-17}$$

Where:

F_2 = *Froude number for design discharge.*
F_1, = *Limiting Froude number.*
C = *Celerity of wave.*

When the roll depth is significantly less than the flow depth Equation 8-16 can be simplified to:

$$h = \frac{C^2}{g}(F_2 - F_1) \qquad \text{8-18}$$

and for a rectangular channel:

$$F_1 \le \frac{3}{2}(2Y^* + 1) \qquad \text{8-19}$$

Where:

$Y^* = y/b$.
y = *flow depth, in feet.*
b = *channel bottom, in feet.*

for a trapezoidal channel:

$$F_1 \leq \frac{3}{2}\left[\frac{(1+2kY^*)(1+2zY^*)}{1+2zY^*+2kzY^{*2}}\right] \tag{8-20}$$

Where:

z = *side slope*
$k = \sqrt{1+z^2}$

Example:

Determine the roll wave height of a rectangular channel with flow depth of 5 feet and 10-foot bottom. The channel slope is 8.0%. The roll wave is measured to be 65 feet per second. Assume that the roll wave height is significantly less than the flow depth.

Entering the data above into Manning's equation, discharge is 2977 cfs and velocity is 59.11 feet per second with a Froude Number of 4.70.

$$F_1 = \frac{3}{2}(2Y^*+1) = \frac{3}{2}\left(2\left(\frac{5}{10}\right)+1\right) = 3$$

$$C = 61\ ft/s - 59.55\ ft/s = 1.45\ ft/s$$

$$h = \frac{C^2}{g}(F_2 - F_1) = \frac{1.45^2}{32.2}(4.70-3) = 0.11 ft$$

8.2.2. Grade Control

Regardless of the size of watershed, a key design element, including conceptual layout, is establishing whether or not grade control exists below the design section. General degradation and aggradation are beyond the scope of this manual; however, references are provided in Chapter 18.

Grade control is a critical factor in the long-term behavior of non-rigid channels. By definition, grade control is any natural or man-made structure within a channel that limits or prevents vertical movement of the channel bed, either by degradation or aggradation. Examples include rock outcroppings, culverts under embankments, drop structures, and bridges; however, not all drop structures, culverts, or bridges can be considered as grade control structures.

Grade control and channel slope are interrelated. In the design of grade control structures, the stability of the study reach must be assessed in context of the equilibrium of the entire system. The benefits of establishing grade control within a specific channel reach are minimal when the adjacent channel reach is either in a degradational or aggradational mode. When designing artificial channels, the designer needs to assess the stability of the reach immediately downstream from the segment under design. If there is evidence of ongoing downstream degradation, a grade control structure may be required. At a minimum, the grade control structure should extend to a depth sufficient enough to prevent upstream migration of the headcut. For each alternative investigated, the longitudinal spacing of grade control structures and the design slope of the channel should result in a stable channel.

8.2.3. Minimum Velocity

The minimum permissible velocity is two (2) feet per second. Very low velocities encourage sedimentation and undesirable plant growth, which decreases channel carrying capacity and promotes nuisance ponding. Channels must be designed with respect to sedimentation issues based on considerations and specialized analysis by a qualified engineer.

8.2.4. Maximum Velocity

The maximum permissible velocity is six (6) feet per second and includes all transitions to or from channels and waterways with similar or different materials. In all cases, the velocity must be nonerosive. For earthen or grass lined channels, maximum permissible velocities should be governed by Table 8-4 and Table 8-5, respectively. If the natural channel slope would cause excessive velocity, employ drop structures, checks, riprap (FHWA, Hydraulic Engineering Circular HEC-11), or other suitable velocity control design features. Channel lining type shall be designed per HEC-11 and HEC-15.

Table 8-4: Maximum Permissible Velocities For Roadside Drainage Channels

Soils Type of Lining (Earth, No Vegetation)	Permissible Velocity (1)(2), ft/sec
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (noncolloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

(1) For sinuous channels, multiply permissible velocity by:

- 0.95 for slightly sinuous.
- 0.90 for moderately sinuous.
- 0.80 for highly sinuous.

(2) Higher velocities may be allowed for design of unlined channels, for the 100-year design event in particular, based on sediment balance considerations and specialized analysis by a qualified engineer. However, sufficient setback allowance should be provided for expected bank erosion during the 100-year event, or a series of annualized events over a 60-year period. Higher velocities may also be acceptable for 100-year peak flow design with approved engineering justification based on a tractive force analysis (FHWA HEC-11).

Table 8-5: Roadside Channels
With Uniform Stand Of Grass Cover and Well Maintained

Cover	Permissible Velocity, ft/sec
Bermuda Grass	6.0
Desert Salt Grass Vine Mesquite	5.0
Lehman Lovegrass Big Galleta Purple Threeawn Sand Dropseed	3.5

(1) Use velocities over 5 ft/sec only where good cover and proper maintenance can be obtained.
(2) Grass is accepted only if an irrigation system is provided.
(3) Grass lined channels are not recommended for slopes greater than 5%.
(4) (Adapted from FHWA 1961 and 1988)(1)(2)(3)

8.2.5. Freeboard

Freeboard is the distance between the calculated water surface and the top of the channel lining or bank. The minimum freeboard is calculated as follows:

$$FB = 0.25\left(y + \frac{V^2}{2g}\right) \qquad \text{8-21}$$

In subcritical channels, the minimum required freeboard is 1 foot or that calculated using Equation 8-21, whichever is greater. In supercritical channels, the required freeboard is 2 feet or the results of Equation 8-21, whichever is greater. For channels with levees, the required freeboard is 3 feet or the results of 8-21, whichever is greater. In all instances, the freeboard required is additive to any increases in water surface due to superelevation or channel curvature. Freeboard for levees must meet Federal Emergency Management Agency (FEMA) freeboard requirements (3, 3.5, or 4 feet minimum depending on location relative to end of levee and to other structures).

In no case should the freeboard be less than 1 foot.

8.2.6. Channel Curvature

In making preliminary layouts for the routing of proposed channels, it is desirable to avoid sharp curvatures, reversed curvatures, and closely-spaced series of curves. If this is unavoidable, the design considerations in subsequent subsections must be followed to reduce superelevations and to eliminate initial and compounded wave disturbances.

The minimum radius of a curved channel, measured to the channel centerline, carrying subcritical flows is recommended to be three times greater than the width of the water surface. Equation 8-22 represents this statement mathematically.

$$r_c \geq 3T \qquad \text{8-22}$$

If the channel is carrying supercritical flows, the recommended minimum radius is:

$$r_c = \frac{4V^2T}{gy} \qquad \text{8-23}$$

Where:

r_c = *Minimum radius of curvature, in feet.*

V = *Velocity, in feet per second.*

T = *Water surface width, in feet.*

g = *Gravity, 32.2 feet per second squared.*

y = *Flow depth, in feet.*

8.2.6.1. Easement Curves

Easement curves are alignment transition curves, employed upstream and downstream of circular curves, when supercritical flow exists in open channels. The purpose of the easement curve is to alter the transverse slope of the water surface and keep the water prism in constant static equilibrium against centrifugal force throughout the entire length of the easement curve and central circular curves, thus achieving minimum heights of superelevation with avoidance of cross-wave disturbances.

Circular easement curves are recommended in lieu of spiral transition curves for ease of design and construction. Also very little hydraulic advantage is gained by the use of the spiral. The circular easement curve consists of curved sections upstream and downstream of the main curve having a radius (2R), twice the main curve radius (R).

Conditions requiring easement curves are:

- When the freeboard, above superelevated water surface (as calculated without an easement curve), is less than two feet.
- In reverse curves or on alignments where curves follow one another closely.
- For any case where elimination of cross-wave disturbances is required (if easement curves are not used, additional freeboard downstream of the curve may be necessary).
- In trapezoidal channels for all cases of supercritical velocity.

For rectangular channels, the length of easement curve (L_E) is given by the following equation:

$$L_E = 2X = 0.32\left(\frac{bV}{\sqrt{D}}\right) \qquad \text{8-24}$$

For trapezoidal and associated channel types, the length of easement curve (L_E) can be calculated as follows:

$$L_E = 0.32(b + 2zD)\left(\frac{V}{\sqrt{D}}\right) \qquad \text{8-25}$$

8.2.7. Superelevation

Curves in a channel cause the maximum flow velocity to shift toward the outside of the bend. Along the outside of the curve, the depth of flow is at a maximum. The consequent rise in the water surface is referred to as superelevation. The equations below are recommended to estimate the magnitude of the superelevation for each particular type of channel.

Readers are cautioned to avoid curves in channels with supercritical flows. The shift in the velocity distribution may cause cross-waves to form, which will persist downstream and could severely limit the hydraulic capacity of the channel. Advanced design criteria or physical model studies beyond the scope of this chapter may be required.

8.2.7.1. Rectangular Channels

For subcritical velocity, or for supercritical velocity where a stable transverse slope has been attained by an upstream easement curve, the superelevation (S) can be calculated from the following equation:

$$S = \frac{V^2 b}{2gr} \qquad \text{8-26}$$

For supercritical velocity in the absence of an upstream easement curve, the superelevation (S) is given by the following equation:

$$S = \frac{V^2 b}{gr} \qquad \text{8-27}$$

Where:

V = Velocity of the flow cross section, in feet per second.

b = Width of the channel, in feet.

g = Acceleration due to gravity, 32.2 feet per second squared.

r = Radius of channel center line curve, in feet.

x = Distance from the start of the circular curve to the point of the first S, in feet.

D = Depth of flow for an equivalent straight reach, in feet.

β = Wave front angle, in degrees.

$$x = \frac{\pi b V}{\sqrt{12gD}} = \frac{0.16bV}{\sqrt{D}} = \frac{0.908b}{\sin\beta} \qquad \text{8-28}$$

$$\sin\beta = \frac{\sqrt{gD}}{V} = \frac{1}{F} \qquad \text{8-29}$$

"S" will not be uniform around the bend, but will have maximum and minimum zones that persist for a considerable distance into the downstream tangent.

8.2.7.2. Trapezoidal Channels

For subcritical velocity, the superelevation (S) can be calculated from the following equation:

$$S = 1.15\ V^2(b + 2zD)/2gr \qquad \text{8-30}$$

Where:

z = cotangent of bank slope.

b = channel bottom width, in feet.

For supercritical velocity, curving alignments shall have easement curves with a superelevation (S) given by the following equation:

$$S = 1.13 \ \ V^2(b + 2zD)/2gr \qquad 8\text{-}31$$

8.2.7.3. Unlined Channels

Unlined channels will be considered trapezoidal insofar as superelevation calculations are concerned. However, this does not apply to calculations of stream or channel cross sectional areas.

8.2.8. Toe Protection

Toe protection failures result when the foundation of the bank protection measure is undermined by scour at the toe resulting from local scour and/or general channel bed degradation. Proper design of protection from toe scour involves two parameters. First, an estimate must be made of the maximum scour expected to occur over the design life of the structure. Second, a means of protection must be provided for the maximum scour. The first parameter, scour depth estimation, requires specialized analysis techniques by a qualified engineer. Mitigation measures for providing protection for the maximum scour are presented in this section.

The two methods of providing toe protection in erodible channels are:

1. To extend protection to the maximum estimated depth of scour.
2. To provide protection that adjusts to the scour as it occurs.

The first method is the preferred technique because the protection is initially placed to a known depth and the designer does not have to depend on uncertainties associated with the method that adjusts to the scour. This method requires extension of the bank protection into the excavated channel bed and is primarily used for placement in dry conditions because of the expense and uncertainties of deep excavation that can frequently encounter groundwater.

The main advantage of the second method is the elimination of relatively deep excavation and related water control. The most frequently used material for providing adjustable toe protection is riprap placed at the toe of the bank in a weighted riprap configuration. The riprap moves downslope, as scour occurs, to form a protective cover. Figure 8-6 shows the desirable configuration for a weighted riprap toe. Other materials utilized are gabion mattresses (see Figure 8-6). These mattresses are anchored to the bank protection and their riverward ends are allowed to lower as scour occurs. Studies by Linder (1976) and the USACE (1981) on riprap toe protection have arrived at the following conclusions:

1. Volume of rock in the weighted riprap toe is probably the most significant factor in determining the success of the weighted riprap toe.
2. Toe shape has a definite influence on performance. Thin toes do not release rock fast enough, which results in poor slope coverage. Thick toes release rock at a greater rate than is needed. The thickness of the recommended toe ranges from two to three times the thickness of the riprap bank protection. The recommended toe shape is shown in Figure 8-6a.

3. Complex toe designs that are difficult to construct are not necessary.
4. Downslope rock movement occurs without significant movement in the down-stream direction.
5. Results from modeling and the subsequent prototypes show that the recommended weighted toe designs launch at a slope slightly steeper than 2:1.
6. Toe volume in the physical model is approximately equal to the volume needed to extend the bank protection to the maximum scour depth at a 2:1 slope. Linder (1976) recommends a toe volume equal to 1.5 times the volume of extending the bank protection to the maximum scour depth.

Weighted riprap toes have been used successfully for many years. However, success has not been universal. A common factor among the failures appears to be the presence of impinged flow on the bank. Therefore, the guidelines herein apply chiefly to flow conditions parallel to the bank. Where impinged flow is likely, analyses must be made to determine an appropriate additional level of protection for such flow conditions.

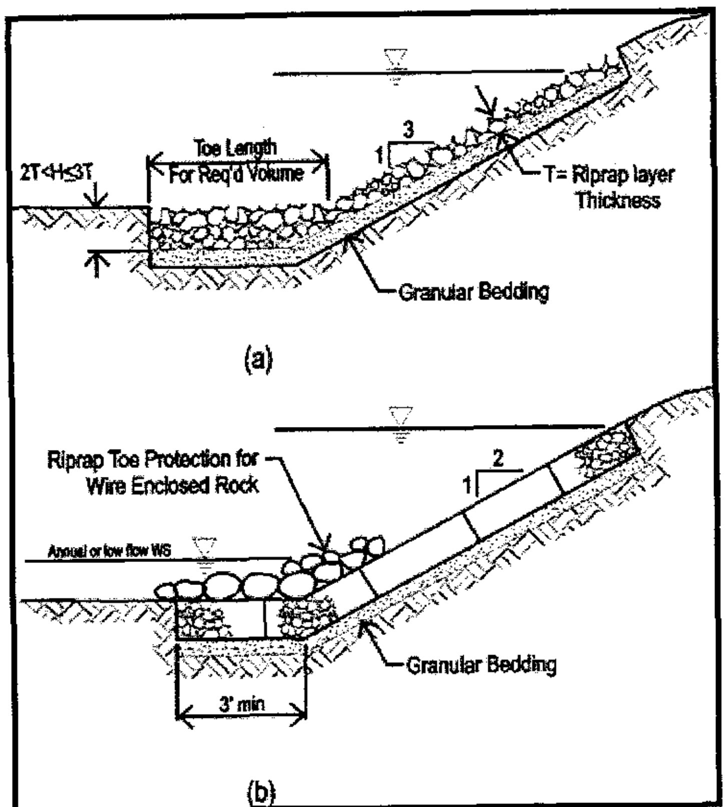


Figure 8-6: Toe Protection Channel Lining (A and B)

8.2.9. Safety

Deep channels with steep side-slopes and high flow velocities can be a hazard to the health, safety, and welfare of the general public. Therefore, the design engineer should always consider the safety aspects of any design. Fencing should be provided for all supercritical channels, regardless of depth. Depending upon velocity, shallower subcritical channels may require fencing. Concrete, shotcrete, or smooth sided soil cement channels meeting certain criteria should have emergency escape ladders or equivalent. In instances where open channels connect conduits that meet the geometric and hazard requirements previously listed, safety devices are recommended to restrict access by the general public along the entire reach of that channel. An example would be a concrete lined channel with 1(H):1(V) side slopes.

8.2.10. Maintenance

The design engineer must also consider maintenance issues associated with any design. At a minimum, a 12-foot maintenance access lane with access ramps is recommended to be provided on at least one side of a channel for publicly maintained channels. In some cases, the City Engineer may require additional width. Turnouts and access ramps will be provided as necessary for complete access to the channel throughout its entire length with a maximum of ½ mile intervals. Turnarounds must be provided at all access road dead ends. Ingress and egress from public right-of-way and/or easements to the channel maintenance and access road must be provided. Channel maintenance and access roads shall be surfaced with 6 inches of gravel base course.

Ramps shall be constructed of 8-inch thick reinforced concrete and will not have slopes greater than 10 percent, and they shall not enter the channel at angles greater than 15 degrees from a line parallel to the channel centerline.

Ramps will be constructed on the same side of the channel as the maintenance and access road. The maintenance and access road shall be offset around the ramp to provide for continuity of the road full length of the channel. The downhill direction of the ramp should be oriented downstream.

To minimize maintenance, paths, walkways, play areas, and irrigation systems should be located in less frequently inundated levels of channels. Bottom widths of channels should be designed in consideration of maintenance requirements for the channel lining and will be no narrower than 8 feet, unless otherwise approved by the jurisdictional entity.

The selection of a channel treatment type should include analyses of both short and long term maintenance. While maintenance efforts will vary between treatment types, all facilities should be able to function through one runoff event with no maintenance, through one flood season with very little maintenance, and from season to season with regular, but minimal maintenance requirements.

8.2.11. Channel Transitions

A flow transition is a change of the open channel flow cross section designed to be accomplished in a short distance with a minimum amount of flow disturbance. Types of transitions are illustrated in Figure 8-7. Of these, the abrupt (headwall) and the straight line (wingwall) are the most common.

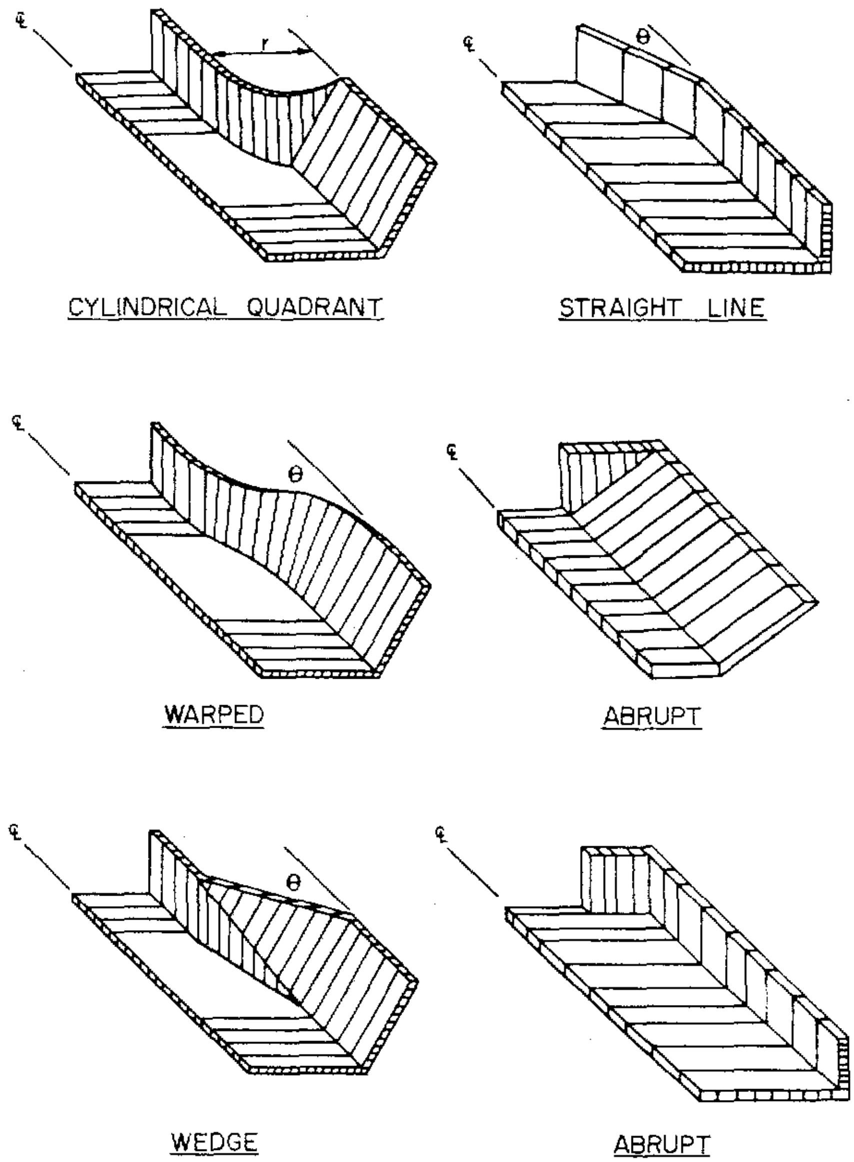


Figure 8-7: Channel Transition Types

8.2.11.1. Contractions

Specially designed open channel flow transitions (contractions) are normally not required for highway culverts. A culvert is normally designed to operate with an upstream headwater pool which dissipates the channel approach velocity and, therefore, negates the need for an approach flow transition.

Special inlet transitions are useful when the conservation of flow energy is essential because of allowable headwater consideration, such as an irrigation structure in subcritical flow, or where it is desirable to maintain a small cross section with supercritical flow in a steep channel. Furthermore, special transitions should be considered at locations where channel geometry changes and at bridges, chutes, and other structures.

Converging walls should be avoided when designing channels in supercritical flow; however, if this is impractical, the converging transition will be designed to minimize wave action. The walls of the transition should be straight lines.

With the initial Froude number and the contraction ratio fixed, and with the continuity equation giving trial curves can produce the geometry of the construction suggested above. The curves represent the equation:

$$\frac{W_1}{W_2} = \left(\frac{Y_3}{Y_1}\right)^{3/2} = \frac{Fr_3}{Fr_1} \tag{8-32}$$

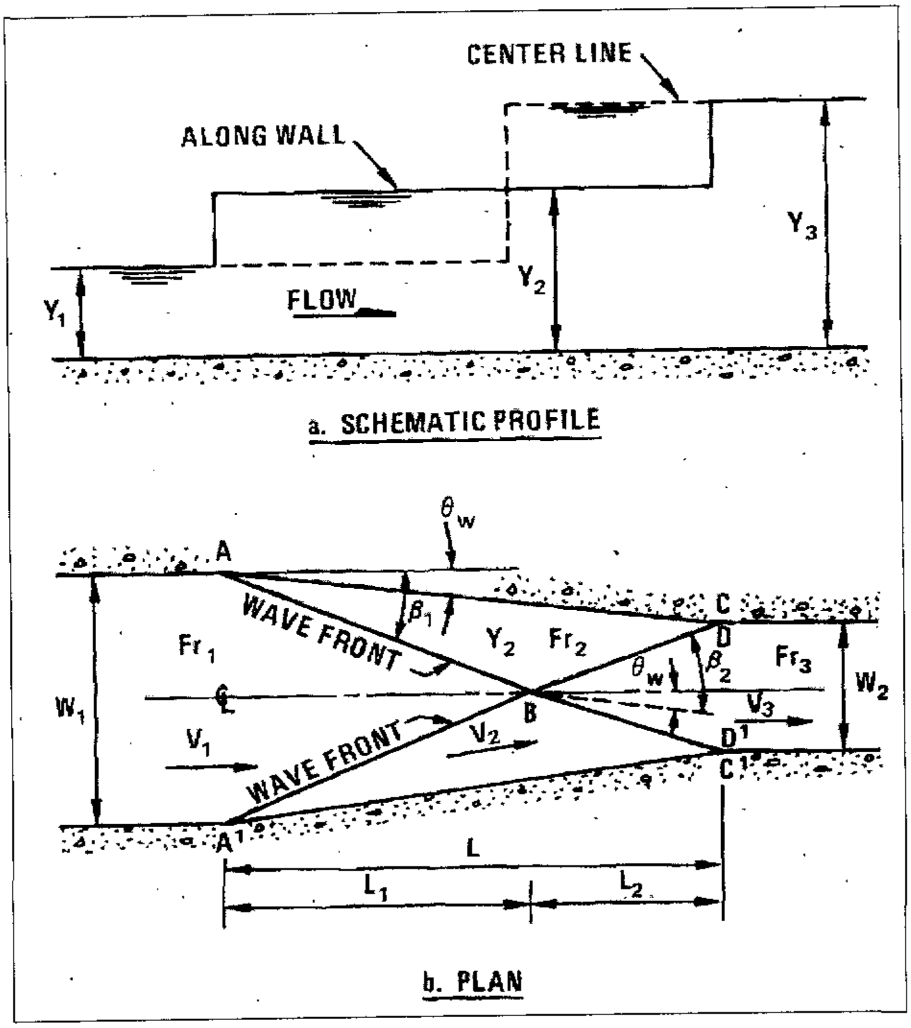


Figure 8-8: Convergent Walls

Figure 8-9 provides a graphical solution to equation 8-30

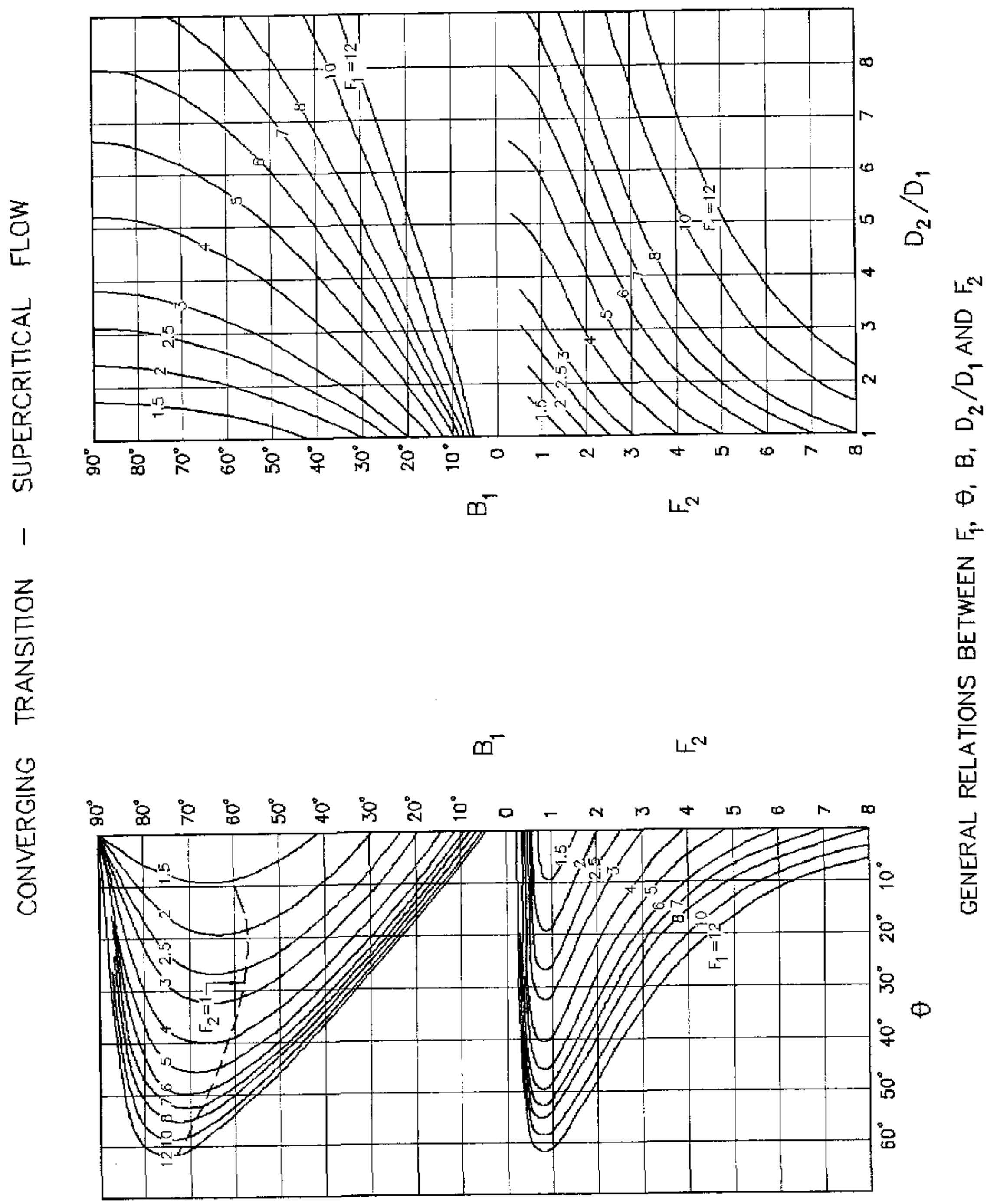


Figure 8-9: Converging Transition - Supercritical Flow

8.2.11.2. Expansions

Outlet transitions (expansions), changes in Q, right-of-way, channel geometry, bridges, chutes, and other structures must be considered in the design of all culverts, channel, protection, and energy dissipators. Design considerations for subcritical channel transitions are presented in

Hydraulic Design of Energy Dissipators for Culverts and Channels Hydraulic Engineering Circular HEC-14 (FHWA, 2006)

The angle of divergence between the center line of the channel and the divergent wall must not exceed 5 degrees 45 minutes or arctangent of F/3; whichever is smaller.

The length of the transition (L) is the longest length determined from the following equations:

$$L \geq 5.0\Delta B \qquad \text{8-33}$$

Where:

F = *Upstream Froude number based on depth of flow*

ΔB = *Difference in channel width at the water surface*

8.2.11.3. Side Channel Spillways

Side channel spillways offer a unique design consideration since the channel energy grade varies parallel to the spillway. Thus, weir equations are not always applicable. The hydraulic analysis of side channel spillways should be pursued with consultation with City of El Paso staff.

When the main channel is relatively narrow and when the peak discharge of side inflow is in the range between 3 and 6 percent of the main channel discharge, high waves are usually produced by the side inflow and are reflected downstream for a long distance, thus requiring additional wall height to preclude overtopping of the channel walls. This condition is amplified when the side inflow is at a greater velocity than the main channel.

To eliminate these wave disturbances a side channel spillway inlet may be used. The City may require this type of structure when outletting into one of their facilities, and its use should be considered for City channels if high waves above the normal water surface cannot be tolerated.

Surface-type inlets shall be constructed of concrete having a minimum thickness of 6 inches and shall be reinforced with the same steel as 6-inch concrete lining. The upstream end of the surface inlet shall be provided with a concrete cutoff wall having a minimum depth of three feet and the downstream end of the inlet shall be connected to the channel lining by an isolation joint. Side slopes of a surface inlet shall be constructed at slopes no greater than 10H:lV to allow vehicular passage across the inlet where a service road is required.

Drainage ditches or swales immediately upstream of a surface inlet shall be provided with erosion protection consisting of concrete lining, rock riprap, or other nonerosive material.

Surface inlets shall enter the channel at a maximum of 90° to the channel centerline, i.e., they may not point upstream.

8.2.11.4. Channel Junctions

Flow rates of 25 percent or more of the main channel flow must be introduced to the main channel by a side channel hydraulically similar to the main channel.

Special design considerations are needed for channel junctions as follows:

- The design water-surface elevations and velocities immediately upstream of the confluence should be matched to within 1 foot of velocity head and within 20 percent of the flow depth for both the 10-year and 100-year design discharges and the four combinations of side inlet and main channel flows that result.

- The centerline radius of any channel cannot be less than 3 times the top-width at the water surface.
- For subcritical velocities less than 12 feet per second, the angle of convergence or divergence between the center line of the channel and the wall must not exceed 12 degrees 30 minutes. The length of the transition (L) is determined from the following equation:

$$L \geq 2.5\Delta B \tag{8-34}$$

Where:

Δ B = the difference in channel width at the water surface.

- For subcritical velocities equal to or greater than 12 feet per second, the angle of convergence or divergence between the center line of the channel and the wall must not exceed 5 degrees 45 minutes. The length (L) is determined from the following equation:

  $$L \geq 5.0\Delta B$$

- The design depth of the main channel below the junction should be the same (or virtually so) as the main channel upstream of the confluence.
- For supercritical flow regime, a momentum analysis as outlined in the USACE document *EM 1110-2-1601* (USACE, 1991) must be undertaken. On a case by case basis, model testing will be required.
- Channels designed with Froude numbers between 0.9 and 1.13 will not be allowed.
- The centerline radius of the side channel may not be less than the quantity (QV/100) in feet.

Energy and momentum balance type calculations must be provided to support all designs involving side channels. The use of HEC-RAS computer program can expedite manual calculations.

8.2.11.5. Transitions between Channel Treatment Types

8.2.11.5.1. Earth Channel to Concrete Lining Transition

The mouth of the transition from earth channels (upstream) to concrete-lined channels (downstream) should match the earth channel section as closely as practicable. Wing dikes and/or other structures must be provided to positively direct all flows to the transition entrance.

The upstream end of the concrete-lined transition will be provided with a cutoff wall having a depth of 1.5 times the design flow depth, but at least 3.0 feet and extending the full width of the concrete section. Erosion protection directly upstream of the concrete transition consisting of grouted or dumped rock riprap at least 12 feet in length and extending full width of the channel section must be provided. Grouted riprap must be at least 12 inches thick (or refer to *Design of Rip-Rap Revetment HEC-11* for project specific design) and tied to the concrete lining and cutoff wall. Dumped riprap must be properly sized, graded, and projected with gravel filter blankets. The maximum allowable rate of bottom width transition is 1:7.5, meaning for every foot of width increase should transition over 7.5 feet of length.

8.2.11.5.2. Concrete Lining to Earth Channel Transition

The transition from concrete-lined channels (upstream) to earth channels (downstream) will include an energy dissipator as necessary to release the designed flows to the earth channel at a relatively non-erosive condition.

Since energy dissipator structures are dependent on individual site and hydraulic conditions, detailed criteria for their design can be found in *Hydraulic Design of Energy Dissipators for Culverts and Channels HEC-14* (FHWA, 2006) and only minimum requirements are included herein for the concrete to earth channel transition.

On this basis, the following minimum standards govern the design of concrete to earth channel transitions:

Table 8-6: Recommended Convergence and Divergence Transition Rates

Mean Channel Velocity (feet per second)	Wall Flare for Each Wall (horizontal to longitudinal)
10-15	1:10
15-30	1:15
30-40	1:20

Maximum rate of bottom width transitions:

- For every foot of horizontal increase (increase in width), the transition should occur along the channel alignment over the longitudinal length shown in the table above.
- The downstream end of the concrete transition structure will be provided with a cutoff wall having a minimum depth of 4 feet and extending the full width of the concrete section.

Directly downstream of the concrete transition structure, erosion protection shall be provided, consisting of rough, exposed surface, grouted rock riprap, and extending the full width of the channel section. The grouted rock riprap should be a minimum of 12 inches thick and tied to concrete structure and the cutoff wall. The length of riprap shall be determined by engineering analysis in accordance with *Design of Riprap Revetment HEC-11* and Section 8.2.12.5 of this DDM.

8.2.12. Channel Linings

Artificial channel linings vary with the shape of the section and with the velocity of the water. Typical channel linings include concrete, soil cement, rock, earth (natural), and grass. These linings can be used alone or in combination with other linings. Typical linings and sections are shown in Figure 8-10.

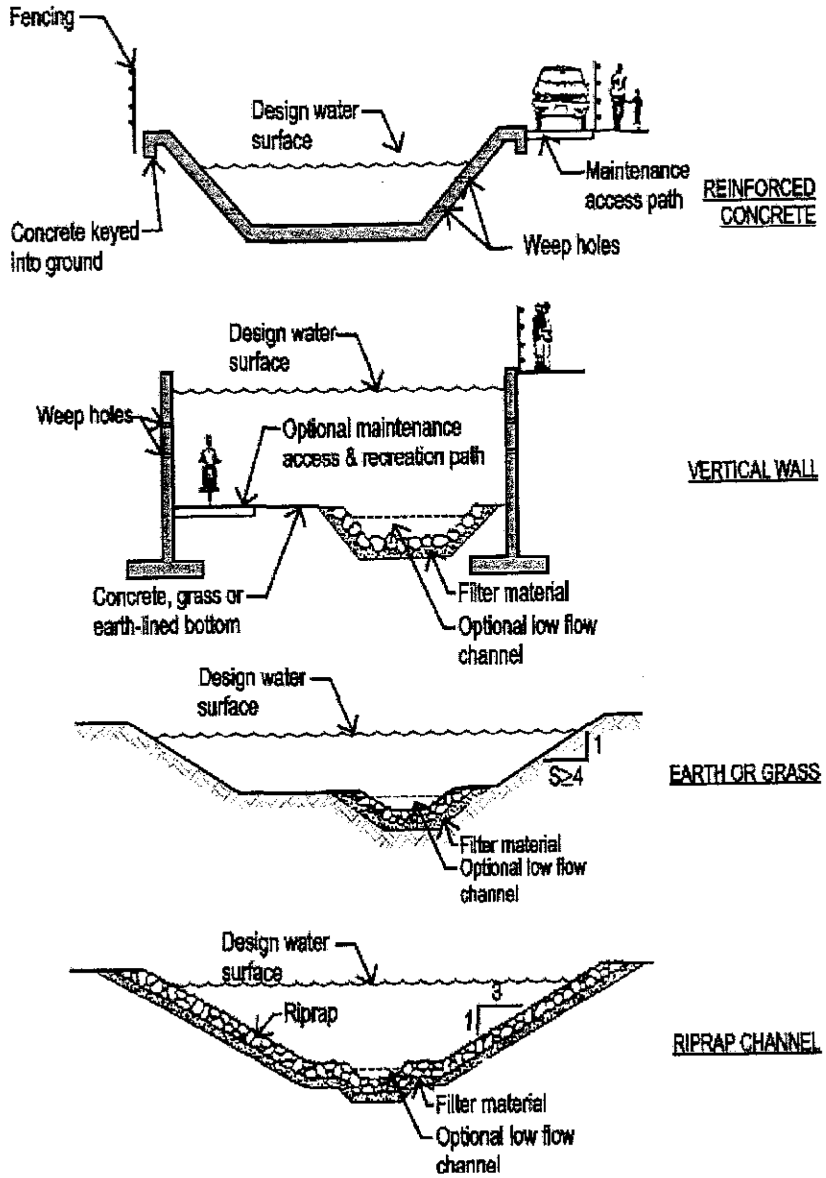


Figure 8-10: Typical Channel Sections

The type of stabilization that may be best suited for a particular purpose will depend upon a variety of factors, including hydraulic conditions, economic factors, soil conditions, material availability, aesthetics, maintenance, and compatibility with existing improvements. The order of preference for subcritical flow conditions is natural channels with periodic grade-control structures, channels with vegetal linings, compound channels, channels lined with riprap or its variations, channels lined with soil cement, and concrete-lined channels. Where supercritical flow conditions occur, only acceptable structurally sound channel linings, such as concrete and shotcrete, are recommended.

The magnitude of the flood control requirements and the consequences of a system failure should be considered foremost in the treatment selection process.

The cost of land and the availability of rights-of-way or easements should be considered in the channel treatment selection process. Rights-of-way and easements should be appropriately located, aligned, and sized for the particular treatment type. Some treatment types may require significant construction easements, but much smaller permanent rights-of-way or easements. The likelihood of replacement or reconstruction should be considered when channel treatment selection is balanced against the configuration of permanent rights-of-way and easements.

The selection of a channel treatment type should be based on any special safety considerations dictated by adjacent or nearby land uses. Whenever a required channel treatment is not compatible with adjacent land uses, adequate safety hazard mitigation measures should be incorporated into the design and construction of the facilities. Channels with vertical walls of 30 inches or greater will require a barrier or fence. Minimum fence or barrier height shall be 72 inches per the City of El Paso standards.

The treatment selection process for each channel reach should include an analysis of the impacts of existing and planned upstream and downstream treatment types.

The initial construction costs of various channel treatment types is and will always be one of the most heavily weighted factors in the selection process. However, when viewed on a larger scale, maintenance and replacement costs can be more important to the total costs of providing adequate levels of protection over time, and therefore must be considered in the planning, design, and construction of channel treatment measures.

The opportunities for including other uses such as transportation and utility corridors, open space, or recreation in the design should be considered when selecting a treatment type and establishing rights-of-way and easements. The inclusion of any other uses must be self-supporting financially and in no way impair or delay the implementation of the drainage and flood control function of the facilities

8.2.12.1. Earth Lined Channels

After full consideration has been given to the soil type, velocity of flow, desired life of the channel, economics, availability of materials, maintenance, and any other pertinent factors, an unlined earth channel may be approved for use.

This category includes both bare earth and naturally vegetated channels in El Paso. Subsequent to construction, some revegetation will naturally occur, or landscaping practices may be used to establish growth of indigenous plant materials. For El Paso, this growth will be desert-like, with few grasses and a sparse spacing of other plants.

Earth lined channels are to be designed for subcritical flow regimes. Normally, these channels are relatively small and do not require low flow channels. If earth lining is used for larger channels, an armored low flow channel is required to control meandering and sediment deposition during low flow events. The low flow design should be checked for the effect that less frequent storms may have on sediment or scour, in terms of maintenance and aesthetic implications.

Generally, earth lined channels use is only acceptable where erosion is not a factor and where the mean velocity does not exceed 3 feet per second. Old and well-seasoned channels will stand higher velocities than new ones; and with other conditions the same, deeper channels will convey water at a higher non-erosive velocity than shallower ones. Maximum side slopes are determined pursuant to an analysis of soil reports. However, in general, slopes should be 3H:lV or flatter.

8.2.12.2. Grass Lined Channels

In a desert environment such as El Paso County, there is not enough natural rainfall to maintain a grass lined channel without irrigation. Therefore, only those channels where an irrigation system is provided and maintenance can be performed are candidates for grass lining.

8.2.12.3. Compound Channels with Multi-Use Opportunities

A channel with a compound or contoured cross section typically contains a smaller, interior channel that isolates frequent low-flows from upper portions of the channel. The upper portions of the channel, which are only inundated during the less frequent storm events (typically, 100-year event), may then be utilized for landscaping and recreation opportunities (such as trails and bike paths). See Figure 8-11. Bank protection can extend from the channel bottom to the top of the low flow channel, or it can extend the full height of the channel sides to the top of the high-flow portion of the channel, depending on the hydraulic characteristics of the channel.

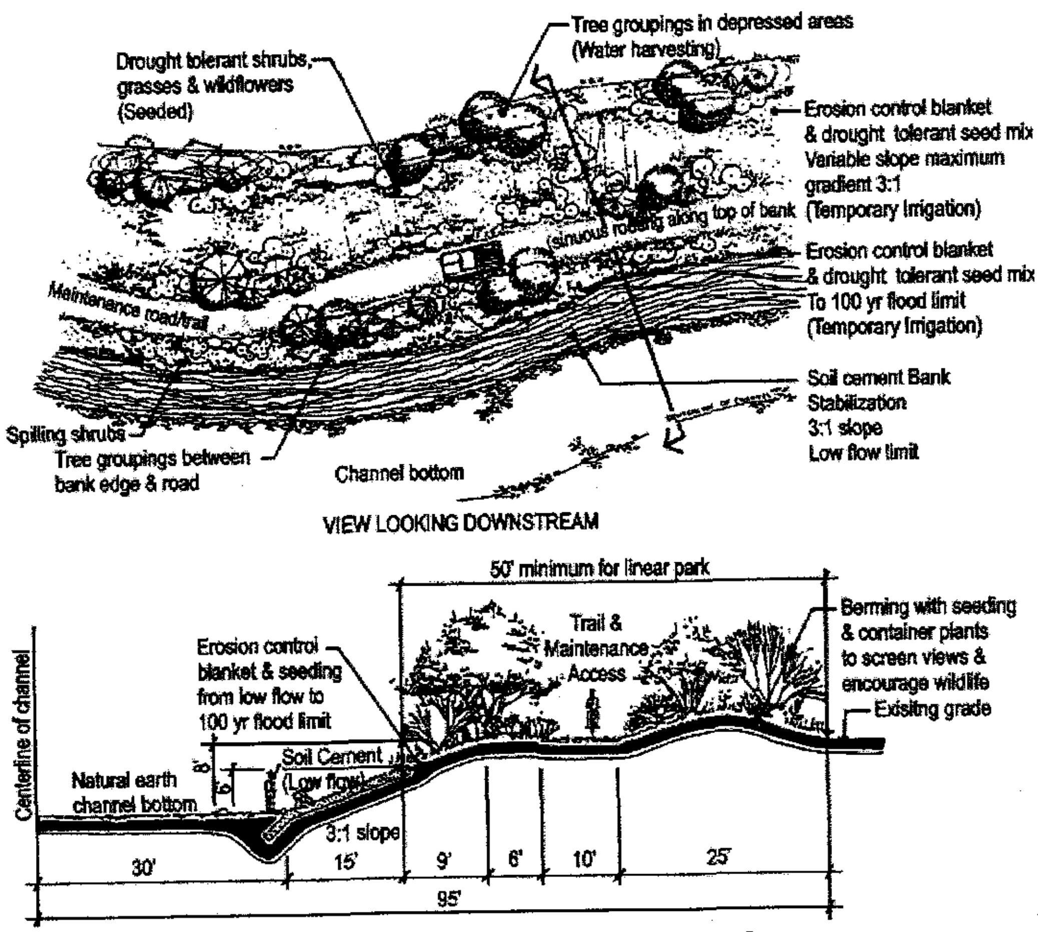


Figure 8-11: Compound Channel

8.2.12.4. Gabion Lined Channels

Gabions refer to rocks that are confined by a wire basket so that they act as a single unit. The wire mesh enclosed rock units are also known as gabion baskets or gabion mattresses. One of the major advantages of wire enclosed rock is that it provides an alternative in situations where available rock sizes are too small for common riprap. Another advantage is the versatility that results from the regular geometric shapes of wire enclosed rock. The rectangular blocks and mats can be fashioned into almost any shape that can be formed with concrete. The durability of wire enclosed rock is generally limited by the service life of the galvanized binding wire which, under normal conditions here in the arid southwest, is considered to be about 35 years. In applications where the gabions are subjected to frequent wet conditions, the life span diminishes to about 15 years (Myers, 2000). Water carrying silt, sand, or gravel can reduce the service life of the wire. Also, water that rolls or otherwise moves cobbles and large stones breaks the wire with a hammer and anvil action and considerably shortens the life of the wire. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified. The designer should verify site specific conditions and coordinate with a qualified manufacturer to properly specify gabion wire. See ASTM A-974 and ASTM A-975.

Gabions are not maintenance free and must be periodically inspected to determine whether the wire is sound. If breaks are found while they are still relatively small, they may be patched by weaving new strands of wire into the wire cage. Wire enclosed rock installations have been found to attract vandalism. Flat mattress surfaces seem to be particularly susceptible to having wires cut and stones removed. It is recommended that, where possible, mattress surfaces be buried, where they are less prone to vandalism. Wire enclosed rock installations should be inspected at least once a year under the best circumstances and may require inspection every three months in vandalism prone areas in conjunction with a regular maintenance program. They should also be inspected after high flow events. Under high flow velocity conditions, mattresses on sloping surfaces must be securely anchored to the surface of the soil as discussed previously.

8.2.12.4.1. Materials

Rock and Wire Enclosure Requirements

Rock filler for the wire baskets should meet the rock property requirements for common riprap. Rock sizes and basket characteristics should meet ASTM A-974 and ASTM A-975. The minimum rock size, d_o, should be equal to the size of the gabion mesh opening. The maximum rock size, d_{100}, should be less than the gabion thickness.

Bedding Requirements

Long term stability of gabion (and common riprap) erosion protection is strongly influenced by proper bedding conditions. A large percentage of all riprap failures are directly attributable to bedding failures, which is particularly disturbing in light of the fact that over half of all riprap installations experience some degree of failure within 10 years of construction. Refer to Section 8.2.12.4.5 for gravel bedding or filter design. Nonwoven, 8-ounce filter fabric has been found acceptable in many applications. The design engineer should check with the manufacturer for its given application.

8.2.12.4.2. Design Considerations

The geometric properties of gabions permit placement in areas where common riprap is either difficult or impractical to place. Proper design and construction is important to successful operation and lifetime performance. Twisted wire mesh has been found to be more tolerant to settlement than welded wire mesh (See ASTM A-975).

Slope Mattress Lining

Figure 8-12 shows a typical configuration for a gabion slope mattress channel lining. The long side of the gabion basket should be aligned parallel with the channel for applications on banks steeper than 2:1. Channel linings should be tied to the channel banks with gabion counterforts (thickened gabion sections that extend into the channel bank) at the upstream edge of the lining. Counterfort spacing shall be per manufacturer's recommendations.

Mattresses and flat gabions on channel side slopes need to be tied to the banks. The ties should be metal stakes no less than 4 feet in length (sandy soils warrant longer lengths). These should be located at the inside corners of basket diaphragms along an upslope (highest) basket wall, so that the metal stakes are an integral part of the basket. The exact spacing of the stakes depends upon the configuration of the baskets, however, the following is the suggested minimum spacing: stake every 6 feet along and down the slope for 2:1 slopes or steeper. Channel linings should be tied to the channel banks with gabion counterforts at the upstream edge of the lining. For most applications, mattresses should be a minimum of 9 inches thick.

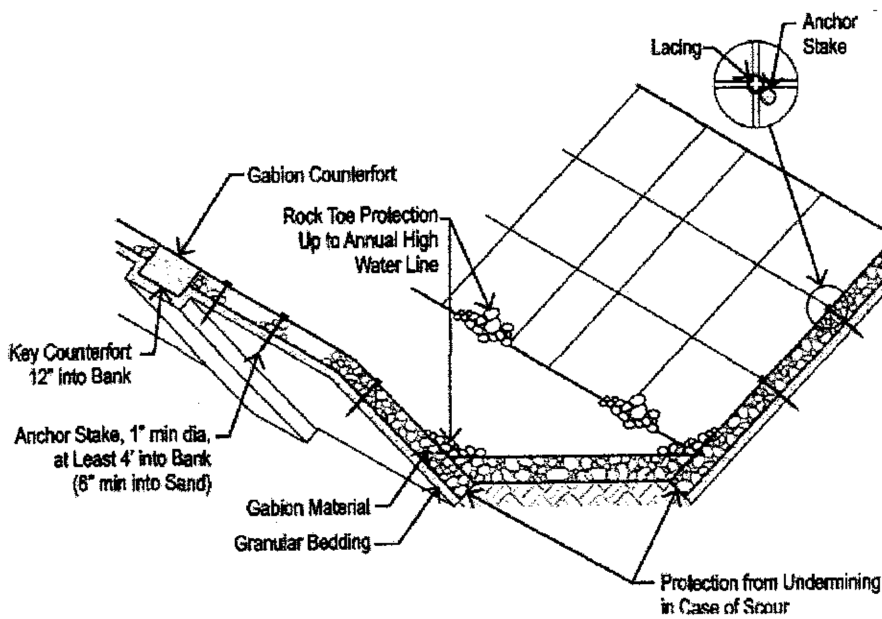


Figure 8-12: Slope Mattress Lining

8.2.12.4.3. Riprap Lined Channels

Common riprap can be an effective lining material if properly designed and constructed. The choice of riprap usually depends on the availability of graded rock with suitable material properties and at a cost that is competitive with alternative lining systems.

Riprap design involves the evaluation of five performance areas:

- Riprap quality.

- Riprap layer characteristics.
- Hydraulic requirements.
- Site conditions.
- River conditions.

In El Paso, site requirements and river conditions are important factors in the protection of bridge structures and flood control channels.

8.2.12.4.4. Riprap Quality

Riprap quality determination refers to the physical characteristics of the rock particles that make up the bank protection. Qualities determined to be most important include density, durability, and shape. Requirements for each of these properties are summarized in this section.

Specific Gravity (Density)

The design stone size for a channel depends on the particle weight, which is a function of the density or specific gravity of the rock material. All stones composing the riprap should have a specific gravity equal to or exceeding 2.4, following the standard test ASTM C127.

Durability

Durability addresses the in-place performance of the individual rock particles and also the transportation of riprap to the construction site. In-place deterioration of rock particles can occur due to cycles of freezing and thawing, or can occur during transportation to the site. The rock particles must have sufficient strength to withstand abrasive action without reducing the gradation below specified limits. Qualitatively, a stone that is hard, dense, and resistant to weathering and water action should be used. Rocks derived from igneous and metamorphic sources provide the most durable riprap.

Laboratory tests should be conducted to document the quality of the rock. Specified tests that should be used to determine durability include the durability index test and absorption test (see ASTM C127). Based on these tests, the durability absorption ratio (DAR) is computed as follows:

$$DAR = \frac{DurabilityIndex}{PercentAbsorption + 1} \qquad 8\text{-}35$$

The following specifications are used to accept or reject material:

1. DAR greater than 23, material is accepted.
2. DAR less than 10, material is rejected.
3. DAR 10 through 23:
   a) Durability index 52 or greater, material is accepted.
   b) Durability index 51 or less, material is rejected.

Shape

There are two basic shape criteria. First, the stones should be angular. Angular stones with relatively flat faces will form a mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. Second, not more than 25 percent of the stones should have a length more than 2.5 times the breadth. The shape of the riprap stone should be cubical, rather than elongated. Cubical stones nest together, and are more resistant to movement. The length is the longest axis through the stone, and the breadth is the shortest axis perpendicular to the length. Angularity is a qualitative parameter that is assessed by visual inspection. No standard tests are used to evaluate this specification. If the engineer is faced with a supply of rounded river rock without a crusher to create angular rock, stone size should be increased 25% and side slopes decreased (USACE, 1995).

The major characteristics of the riprap layer include characteristic size, gradation, thickness, and filter-blanket requirements.

Characteristic Size

The characteristic size in a riprap gradation is the d_{50}. This size represents the average diameter of a rock particle for which 50 percent of the gradation is finer, by weight.

Gradation

To form an interlocked mass of stones, a range of stone sizes must be specified. The object is to obtain a dense, uniform mass of durable, angular stones with no apparent voids or pockets. The recommended maximum stone size is 2 times the d_{50} and the recommended minimum size is one-third of the d_{50}.

The gradation coefficient, G, should equal 1.5.

$$G=0.5(d_{84}/d_{50} + d_{50}/d_{16})$$

Table 8-7 provides design gradations for riprap. As a practical matter, the designer should check with local quarries and suppliers regarding the classes and quality of riprap available near the site.

Table 8-7: Riprap Gradation Limits

Stone Size Range (ft)	Stone Weight Range (lb)	Percent of Gradation Smaller Than
1.5 d_{50} to 1.7 d_{50}	3.0 W_{50} to 5.0 W_{50}	100
1.2 d_{50} to 1.4 d_{50}	2.0 W_{50} to 2.75 W_{50}	85
1.0 d_{50} to 1.15 d_{50}	1.0 W_{50} to 1.5 W_{50}	50
0.4 d_{50} to 0.6 d_{50}	0.1 W_{50} to 0.2 W_{50}	15

Thickness

The riprap-layer thickness shall be the greater of 1.0 times the d_{100} value, or 1.5 times the d_{50} value. However, the thickness need not exceed twice the d_{100} value. The thickness is measured perpendicular to the slope upon which the riprap is placed.

Filter Blanket Requirements

The purpose of granular filter blankets underlying riprap is two-fold. First, they protect the underlying soil from washing out, and second, they provide a base on which the riprap will rest.

The need for a filter blanket is a function of particle-size ratios between the riprap and the underlying soil that comprise the channel bank. The inequalities that must be satisfied are as follows:

$$\frac{(d_{15})_{filter}}{(d_{85})_{base}} < 5 < \frac{(d_{15})_{filter}}{(d_{15})_{base}} < 40 \qquad 8\text{-}36$$

$$\frac{(d_{50})_{filter}}{(d_{50})_{base}} < 40 \qquad 8\text{-}37$$

In these relationships, "filter" refers to the overlying material and "base" refers to the underlying material. The relationships must hold between the filter blanket and base material and between the riprap and filter blanket (USDOT, 1988 and 1989).

If the inequalities are satisfied by the riprap itself, then no filter blanket is required. If the difference between the base material and the riprap gradations are very large, then multiple filter layers may be necessary. To simplify the use of a gravel filter layer, Table 8-8 outlines recommended standard gradations.

The Type-I and Type-II bedding specifications shown in Table 8-8 are developed using the criteria given in Equation 8-36 and Equation 8-37, considering that very fine grained, silty, non-cohesive soils can be protected with the same bedding gradation developed for a mean grain size of 0.045 mm. The Type-I bedding in Table 8-8 is designed to be the lower layer in a two-layer filter for protecting fine grained soils. When the channel is excavated in coarse sand and gravel (i.e., 50 percent or more by weight retained on the No. 40 sieve), only the Type-II filter is required. Otherwise, two bedding layers (Type-I topped by Type-II) are required. For the required bedding thickness, see Table 8-9.

Table 8-8: Gradation for Gravel Bedding

Standard Sieve Size	Type I (1)	Type II (1)
3 inches	-	90 to 100
1-1/2 inches	-	-
3/4 inch	-	20 to 90
3/8 inch	100	-
#4 (4.75 mm)	95 to 100	0 to 20
#16 (1.18 mm)	45 to 80	-
#50 (0.30 mm)	10 to 30	-
#100 (0.15 mm)	2 to 10	-
#200 (0.075 mm)	0 to 2	0 to 3

(1) Percent passing by weight

Table 8-9: Thickness Requirements For Gravel Bedding

Riprap Size Classification, inches	Minimum Bedding Thickness, inches		
	Fine Grain Native Soils		Coarse Grain
	Type I	Type II	Type III
6,8	4	4	6
12	4	4	6
18	4	6	8
24	4	6	8
30	4	8	10
36	4	8	10

Filter Fabric Requirements

The design criteria for filter fabric are functions of the permeability of the fabric and the effective opening size. The permeability of the fabric must exceed the permeability of the underlying soil, and the apparent opening size (AOS) must be small enough to retain the soil.

The criteria for apparent opening size are as follows:

1. For soil with less than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.6 mm (a No. 30 sieve).
2. For soil with more than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.3 mm (a No. 50 sieve).

Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface that provides less resistance to stone movement. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care must be exercised during construction. The site conditions and specific application and installation procedures must be carefully considered in evaluating filter fabric as a replacement for granular bedding material. Filter fabric can provide adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric.

Numerous failures have occurred because of the improper installation of filter fabric. Therefore, when using filter fabric it is critical that the manufacture's guidelines for installing it be followed.

8.2.12.5 Hydraulic Design Requirements

General

Channel linings constructed of placed, graded riprap or gabions to control channel erosion have been found to be cost effective where channel reaches are relatively short and where a nearby source of quality rock is available.

Situations where rock riprap or gabion basket linings may be appropriate are:

1. Major flows that produce channel velocities in excess of allowable non-eroding values.
2. Channel side slopes at 3:1 for rock riprap and 2:1 for gabion mattresses.
3. Where rapid changes in channel geometry occur, such as channel bends and transitions.

This section presents design requirements for common rock riprap, while Section 8.2.12.4 contains additional design considerations specifically related to gabions. Both sections are valid only for subcritical flow conditions where the Froude number is 0.86 or less.

Riprap Sizing

Several reference sources are available for design procedures. Two recommended sources are:

1. *Design of Riprap Revetment* (FHWA, HEC-11, Publication No. FHWA-IP-89-016, March 1989)
2. *Hydraulic Design of Flood Control Channels* (USACE, EM-1110-2-1601, 1991)

The riprap sizing method presented here is from HEC-11 (for a complete discussion on this method the designer is referred to the above referenced documents). This method is based on tractive force (shear stress) theory, but with velocity as its primary design parameter. This is a blend between the two approaches of permissible velocity and permissible tractive force. The hydraulic assumptions are uniform, steady, subcritical flow. However, adjustments to the design equation are provided for other regimes and conditions, such as gradually varying flow and approaching rapidly varying flow. In this method, the riprap size is selected such that the flow induced tractive force does not exceed the critical shear stress of the riprap. The critical shear is based on Shield's relationship, a function of specific weight of water, specific weight of the riprap material, the median rock size (d_{50}), Shield's parameter, and a factor that is a function of the bank angle and riprap's material angle of repose. The average shear stress or tractive force exerted by flowing water is the product of unit weight of water, energy grade line slope, and hydraulic radius. These two equations are combined to develop the design tractive force relationship in terms of a stability factor (*SF*). The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap materials critical shear stress. Therefore, if the stability factor is greater than 1.0, the critical shear stress is greater than the flow induced tractive stress and the riprap is considered stable.

For the HEC-11 method, the d_{50} (feet) is determined by:

$$d_{50} = \frac{0.001V_a^3}{d_{avg}^{0.5}K_1^{1.5}} \qquad \text{8-38}$$

Where:

V_a = *Average* velocity in the main channel, in feet per second.

d_{avg} = Average flow depth in the main channel, in feet.

K_1 = Bank angle correction factor.

The bank angle correction factor is determined using Equation 8-39:

$$K_1 = \left[1 - \frac{\sin^2\theta}{\sin^2\phi}\right]^{0.5} \qquad \text{8-39}$$

Where θ is the bank angle with the horizontal and Φ is the riprap material's angle of repose. The bank angle correction factor can also be determined using Figure 8-13. The riprap material's angle of repose can be determined using Figure 8-14.

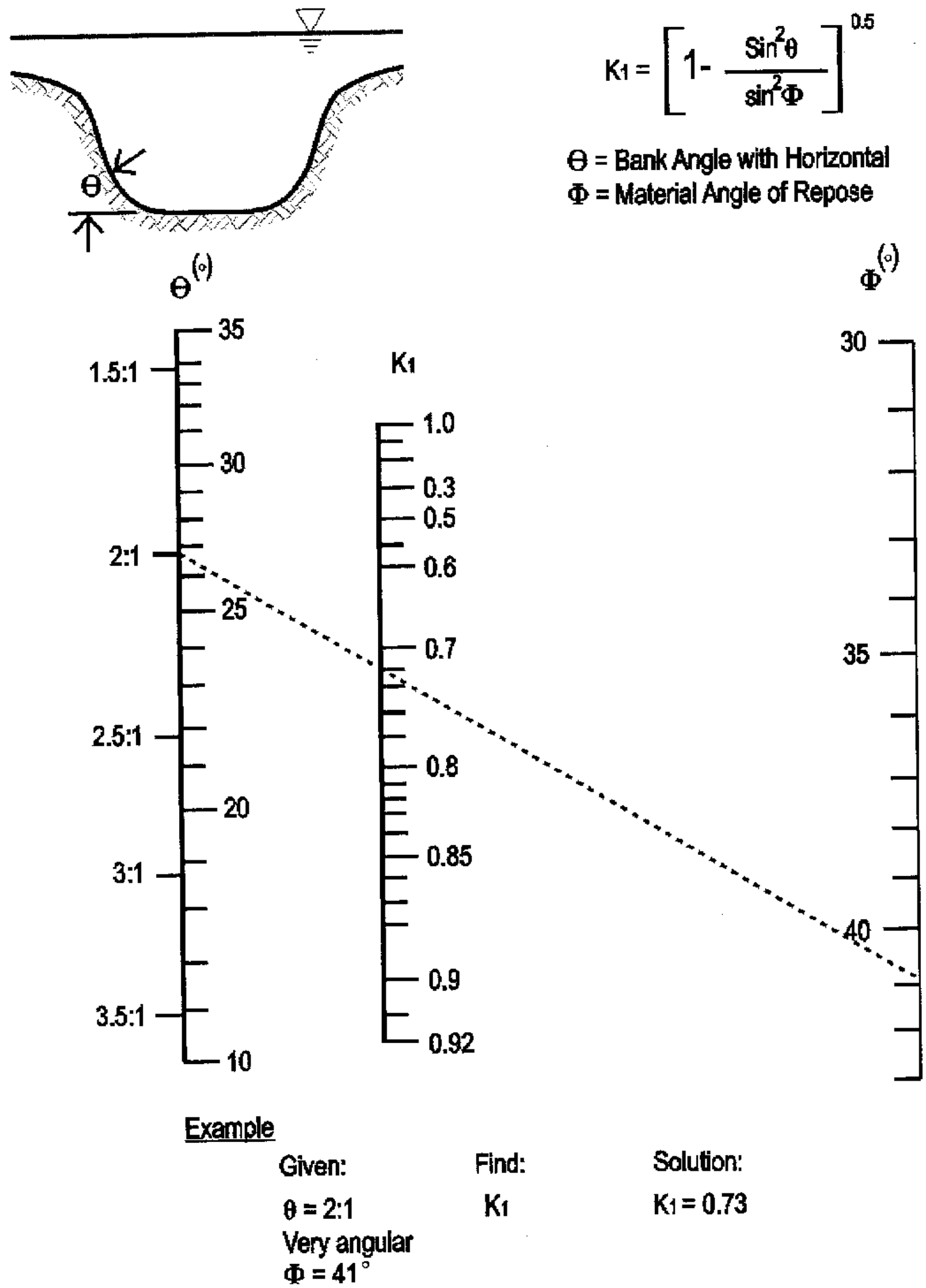


Figure 8-13: Bank Angle Correction Factor, K1

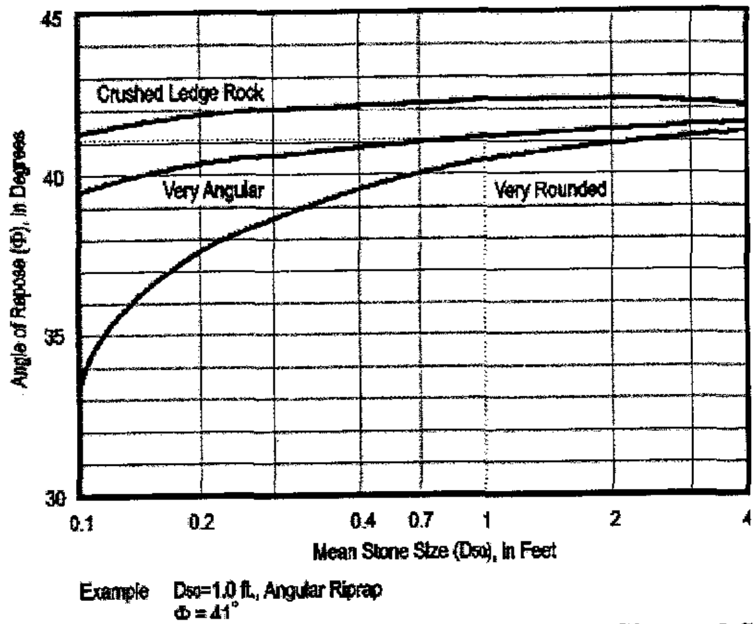


Figure 8-14: Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone

Equation 8-38 is based on a rock riprap specific gravity of 2.65 and a stability factor of 1.2. Equations 8-40 and 8-41 present correction factors for other specific gravities and stability factors.

$$C_{sg} = \frac{2.12}{(S_S - 1)^{1.5}} \qquad \text{8-40}$$

where S_s is the specific gravity of the rock riprap.

$$C_{sf} = \left(\frac{SF}{1.2}\right)^{1.5} \qquad \text{8-41}$$

where (SF) is the stability factor to be applied.

Table 8-10 presents guidelines for the selection of an appropriate value for the stability factor.

The correction factors computed using Equations 8-39 and 8-40 are multiplied together to form a single correction factor *C*. This correction factor is then multiplied by the riprap size computed from Equation 8-38 to arrive at a stable riprap size.

The stability factor is used to reflect the uncertainty in the hydraulic conditions at a particular site. Equation 8-38 is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear. For example, debris and/or ice impacts, or the cumulative effect of high shear stresses and forces from wind and/or boat generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. Typically, the minimum thickness of riprap linings should be the greater of $1 \times d_{100}$ or $1.5 \times d_{50}$.

Table 8-10: Stability Factors

Condition	Stability Factor Range
Uniform Flow: Straight or mildly curving reach (curve radius/channel width > 30); Impact from wave action and floating debris is minimal; Little or no uncertainty in design parameters.	1.0 - 1.2
Gradually Varying Flow: Moderate bend curvature (30 > curve radius/ channel width > 10); Impact from wave action and floating debris is moderate.	1.3 - 1.6
Approaching rapidly varying flow: Sharp bend curvature (10 > curve radius/channel width); Significant impact potential from floating debris and/ or ice; Significant wind and/or boat generated waves (1-2 ft); High flow turbulence; Turbulently mixing flow at bridge abutments; Significant uncertainty in design parameters.	1.6 - 2.0

8.2.12.6 Grouted Rock

Grouted rock is a structural lining composed of a blanket of rock that is interlocked and bound together by means of concrete grout injected into the void spaces to form a monolithic revetment. The grout must extend the full thickness of the rock blanket, with the face rocks exposed for a maximum of one-fourth to one-third of their depth.

This lining type is often suggested as a substitute for adequately sized riprap. It is not an equivalent product because it is neither rigid nor flexible. Any movement or settlement of the subgrade immediately results in cracks in the matrix that, in turn, allow water to enter behind the lining and greatly accelerate the lining's destruction. Some jurisdictions do not accept this alternative and its use is discouraged with two exceptions: riprap designed by the guidelines contained herein can be grouted to 1) minimize vandalism and/or 2) to inhibit the growth of volunteer vegetation and to aid in maintenance.

8.2.12.7 Soil Cement

Soil cement linings are composed of a thick layer (4-foot minimum) of unreinforced soil cement. Soil cement is subject to weathering and abrasion, and thus, may not function satisfactorily long-term when used in the bottom of channels. Soil cement can withstand relatively high velocities for short periods of time, and therefore, is most appropriate for channels with limited right-of-way or as a bank lining near bridges and culverts where local velocities tend to be high.

8.2.12.7.1 Materials

A wide variety of soils can be used to make durable soil cement. For maximum economy and most efficient construction, it is recommended that:

1. The soil contains no material retained on a 3-inch (75 mm) sieve.
2. Between 40 percent and 80 percent pass the No. 4 (4.75 mm) sieve.
3. Between 2 percent and 10 percent pass the No. 200 (0.074 mm) sieve.
4. The Plasticity Index (PI) of the fines should not exceed 10.

If the onsite material does not meet these guidelines, the addition of import material may be necessary. Standard laboratory tests are available to determine the required proportions of cement and moisture to produce durable soil cement. The design of most soil cement for water control projects is based on the cement content indicated by ASTM testing procedures and increased by a suitable factor to account for direct exposure, erosion, or abrasion forces.

The Portland cement should comply with one of the following specifications: ASTM C150, CSA A5, or AASHTO M85 for Portland cement of the type specified; or ASTM C595 or AASHTO M240 for Portland blast-furnace slag or Portland pozzolan cement, excluding slag cements Types S and SA.

It is important that testing to establish required cement content be done with the specific cement type, soil, and water that will be used in the project.

Typically, soil cement linings are constructed by the central-plant method, where selected onsite soil materials, or soils borrowed from nearby areas, are mixed with Portland cement and water and transported to the site for placement and compaction.

8.2.12.7.2 Design Considerations

Figure 8-15 shows a composite channel consisting of an earth bottom with soil cement stabilization along the banks. On side slopes, the soil cement is often constructed by placing and compacting the material in horizontal layers stair-stepped up the slope. The rounded step facing results from ordinary placement and compaction methods. Generally, an 8 to 9 foot minimum working width is required for placement and compaction of the soil cement layers by standard highway construction equipment. A width of 9-feet is preferred for maintenance and safety reasons. Figure 8-16 shows the relationship between slope of facing, thickness of compacted horizontal layer, horizontal layer width, and minimum facing thickness measured normal to slope. For a horizontal working width of 9 feet, a side slope of 2:1, and 6-inch thick layers, the resulting minimum thickness of facing would be about 4 feet, measured normal to the slope. The side slope can vary from 1:1 to 3:1 depending on the soil type and natural angle of repose. Side slopes steeper than 2:1 are not recommended, due to safety issues, but may be allowed when right-of-way is a problem. Soil cement may be placed on slopes 3:1 or flatter at a minimum thickness of eight to twelve inches, depending upon the mixing technique. This would be done without the stair-step layer approach, where a lesser level of protection is permissible.

An important consideration in the design of the soil cement facing is to provide that all extremities of the facing are tied into non-erodible sections or abutments. The upstream and downstream ends of the facing should terminate smoothly into the natural channel banks. A buried cutoff wall normal to the slope or other measures may be necessary to prevent undermining of the soil cement facing by flood flows.

The top of the lining should be keyed into the ground to protect against erosion of the backside of the soil cement layer by lateral inflows, as shown in Figure 8-17. As with any impervious channel lining system, seepage and related uplift forces should be considered and, if required, appropriate counter-measures provided, such as weep holes or subdrains. Tributary storm drain pipelines can normally be accommodated by placing and compacting the soil cement by hand, using small power tools or by using a lean mix concrete. For earthen channels with soil cement side slope protection, the lining should be designed to extend to the anticipated depth of total scour. Further design information may be found in ACI 230.1, *State Of The Art Report on Soil*

Cement. Additional information on design and construction is available from the Portland Cement Association, Skokie, IL.

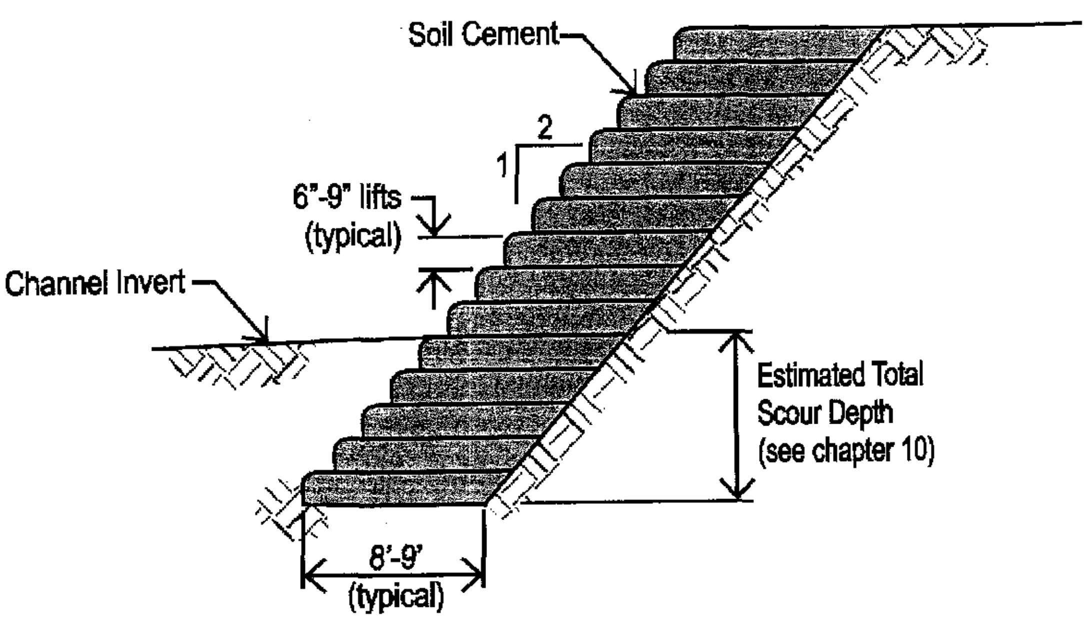


Figure 8-15: Soil Cement Placement Detail (Not To Scale)

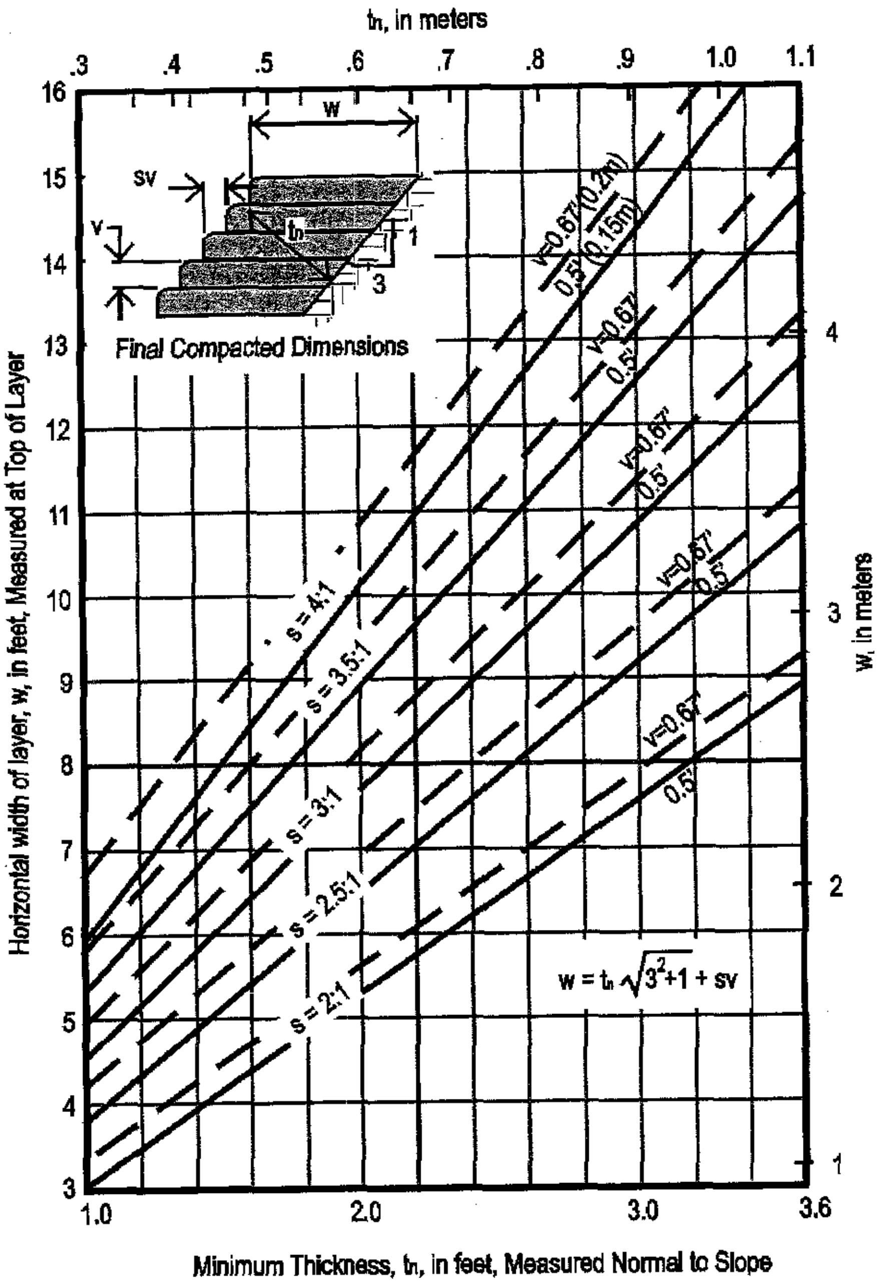


Figure 8-16: Relationships for Soil Cement Lining Slope, Facing Thickness, Layer Thickness, and Horizontal Layer Width

8.2.12.8 Concrete Lined Channels

Concrete lined channels may be constructed of reinforced concrete or shotcrete. They are used primarily where right-of-way is limited and may be designed for either subcritical or supercritical flow. Concrete lined channels generally have steep side slopes because of the limited right-of-way. Inherently, these channels present public safety problems both in wet and dry weather. The anticipated structural loads and the clearance requirements of the reinforcing steel will dictate the thickness of the concrete lining. Weep holes and subdrains are required to prevent uplift pressures from hydrostatic force in saturated conditions. Reinforced tie-ins are required at the top of the lining. These concepts are illustrated in Figure 8-10. Designers are cautioned against copying these details directly without first evaluating the design conditions for their specific project. Concrete and shotcrete lined channels are discouraged in residential and recreational areas. If concrete channels are needed in these areas, the designer should contact the technical staff of the appropriate jurisdiction.

The most common problems of concrete lined channels are due to bedding and liner failures. Typical failures are: 1) liner cracking due to settlement of the sub-grade; 2) liner cracking due to the removal of bed and bank material by seepage force; and 3) liner cracking and floating due to hydrostatic back pressure from high groundwater.

Lack of maintenance can result in vegetation growth through the concrete lining and sediment deposition in the channel that will increase the flow resistance. This reduction in channel capacity can cause overflow at design discharges, and consequently, permit the erosion of overbank material and failure of concrete lining.

Concrete lined channels are usually designed for supercritical flow conditions and/or when velocities exceed five feet per second for earth lined channels. Froude numbers for supercritical flow shall be greater than 1.13. Unstable flow conditions occur when the Froude number falls between 0.86 and 1.13 and must be avoided.

Supercritical flow in an open channel in an urbanized area creates certain hazards that the designer must take into consideration. From a practical standpoint it is generally unwise to have any curvature in a supercritical channel. Careful attention must be taken to prevent or control excessive oscillatory waves that may extend the entire length of the channel from only minor obstructions upstream. Imperfections at joints may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can readily occur. High velocity flow can enter cracks or joints and create uplift forces by the conversion of velocity head to pressure head causing damage to the channel lining. It is evident that when designing a lined channel with supercritical flow, the designer must use utmost care and consider all relevant factors.

All concrete lined channels must have continuous reinforcement extending both longitudinally and laterally. For channels carrying supercritical flow, there shall be no reduction in cross sectional area at bridges or culverts, or any obstructions in the flow path.

Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load that might be imposed upon the structure in the event of major debris blockage. Tributary storm drain pipelines must not protrude into the channel flow area.

Generally, if side slopes steeper than 2:1 are used, then safety and structural requirements become a primary concern. To determine the thickness of the lining, refer to the TXDOT *Drainage Manual*. Design of the lining should also include consideration of anticipated vehicular loading from maintenance equipment. Joints in the lining should be designed in accordance with standard structural analysis procedures with consideration of the size of the channel, thickness of the lining, and anticipated construction techniques. The concrete lining must be keyed into the adjacent overbanks as shown in Figure 8-17.

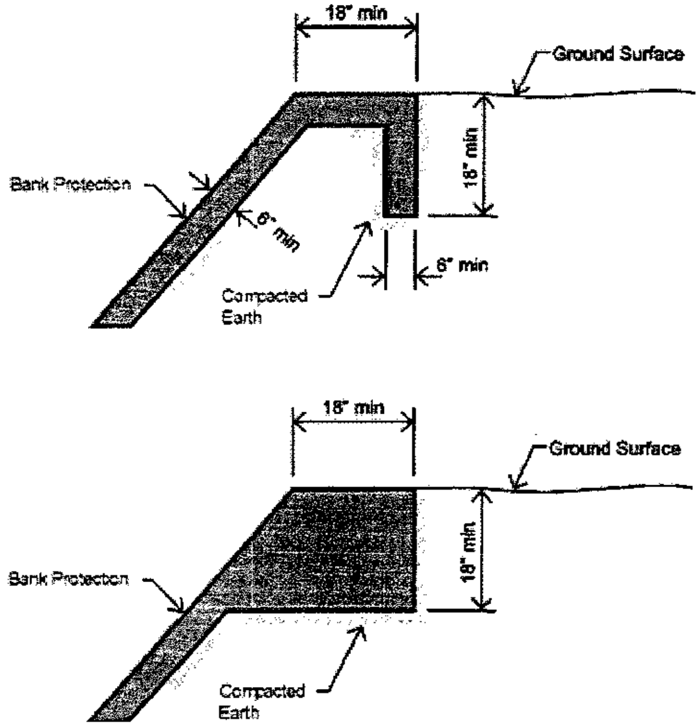


Figure 8-17: Typical Bank-Protection Key-Ins (Not To Scale)

The roughness coefficient for a concrete lining can vary from 0.011 for a troweled finish to 0.020 for a very rough or unfinished surface. For shotcrete, roughness coefficients can vary from 0.016 to 0.025. The accumulation of sediment and debris must be taken into account when determining the roughness coefficient.

Long-term stability of concrete lined channels depends in part on proper bedding. Undisturbed soils often are satisfactory for a foundation for lining without further treatment. Expansive clays are usually an extreme hazard to concrete lining and should be avoided. A filter underneath the lining is recommended to protect fine material from creeping along the lining. A well-graded gravel filter should be placed over the channel bed prior to lining the channel with concrete.

Since concrete-lined channels are often used at locations where excessive seepage exists or smaller channel cross sections are required, transitions will be required both upstream and downstream of the concrete lined channel. Such transitions are intended to prevent undermining of the lining and to reduce turbulence. Transitions should be lined with concrete or other scour resistant material to reduce scour potential.

Cutoff walls should be incorporated with transitions at both the upstream and downstream end of the concrete lined channel to reduce seepage forces and prevent lining failure due to scour, undermining, and piping. The depth of cutoff walls should extend below the expected scour depth. Determination of expected total scour depth requires analyses as discussed in Chapter 14.

The probability of damaging the concrete lining due to hydrostatic back pressure and subgrade erosion can be greatly reduced by providing underdrains. There are two types of artificial drainage installations. One type consists of 4- or 6-inch diameter perforated pipelines placed in gravel-filled trenches along one or both toes of the inside slopes. These longitudinal drains are either connected to transverse cross drains, which discharge the water below the channel or to pump pits, or extend through the lining and connect to outlet boxes on the floor of the channel. The outlet boxes are equipped with one-way flap valves that prevent backflow and relieve any external pressure that is greater than the water pressure on the upper surface of the channel bottom. The second type consists of a permeable gravel blanket of selected material or sand and gravel pockets, drained into the channel at frequent intervals (10 to 20 feet) by flap valves in the channel invert. Figure 8-18 shows a drawing of a flap valve for use without tile pipe and in a fine gravel and sand subgrade. Both the tile and pipe system and the unconnected flap valve type must be encased in a filter that will prevent piping of subgrade material into the pipe or through the valve. For detailed information on underdrains refer to *Lining for Irrigation Canals* (USBR, undated).

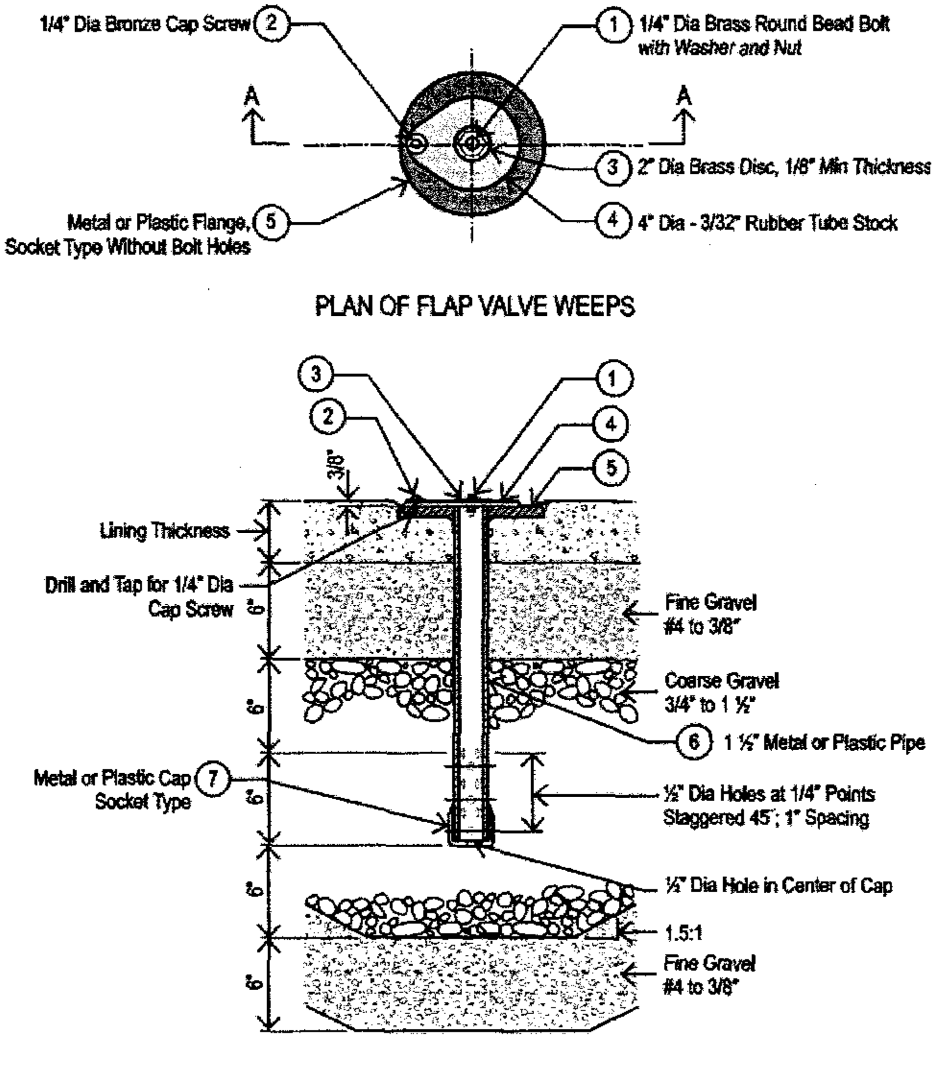


Figure 8-18: Flap Valve Installation for a Channel Underdrain

Where a lesser degree of seepage control is warranted, weep holes spaced at appropriate intervals may be used. When embankment stability may be compromised or when ground water levels may be raised by back drainage from the lined channel, weep holes may be equipped with flap valves or other measures that allow seepage relief but prevent backflow or introduction of surface water behind the lining.

The shotcrete process has become an important and widely used technique. Shotcrete is mortar or concrete pneumatically projected at high velocities onto a surface. In the past, the term "gunite" was commonly used to designate dry-mix mortar shotcrete. The term is currently outdated and "shotcrete" has become the trade name for all pneumatically applied dry-mix or wet-mix concrete or mortar.

ACI 506R (1985) discusses the properties, applications, materials, reinforcement, equipment, shotcrete crews, proportioning, batching, placement, and quality control of the shotcrete process.

As a channel lining, shotcrete is an acceptable method of applying concrete with a general improvement in density, bonding, and decreased permeability. The same design considerations discussed for concrete channels apply in the design of shotcrete channels. Shotcrete linings are to be designed to the same thickness and reinforcement as required for concrete linings. Given the limitations of construction, the minimum slope for concrete and shotcrete channels is 0.0015 foot per foot.

8.2.13 Low Flow Channels

Some of the sections shown in Figure 8-10 have an optional low flow channel. Low flow channels are provided to minimize lateral meandering and sedimentation during low flow events. They also permit the incorporation of recreational amenities by preventing these facilities from being flooded during high frequency, low discharge flow events in compound channels. Many large drainage basins have small base flows resulting from irrigation returns, treatment plant effluent, or urban cooling water. In addition, the most frequent runoff events are considerably smaller in magnitude than the storm for which the channel was designed. In the long term, these high frequency, low magnitude flows will deposit considerable amounts of sediment in the channel. Sediment deposition can cause redirection of flow into the channel banks resulting in erosion and/or a meandering low flow channel in the channel bottom. Earth and grass lined channels are particularly susceptible to this problem. It is recommended that low flow channels be provided whenever the following condition exists:

$$\frac{b}{V_y} \geq 1.40 \qquad \text{8-42}$$

8.2.14 Irrigation Structures

All irrigation structures within the State of Texas fall under the jurisdiction of public or quasi-public organizations. Modifications to existing structures must conform to the design guidelines and regulations of the jurisdictional body. Information regarding irrigation structures can be found on the El Paso County Water Improvement District No.1 website at http://www.epc/wid1.org.

The selection of a treatment type or of a combination of treatment types for a channel within the El Paso area should be based on a rational assessment of the needs of the City. Design for erosion protection shall adhere to this section of this DDM.

9. Culverts

Culverts are primarily used for conveying runoff through a roadway embankment. They are normally aligned with a watercourse or engineered drainage channel. Culverts are typically used for smaller drainageways. They may also serve as outfall structures for stormdrain systems. Bridges are generally used for larger drainageways such as large washes and rivers. For design of bridge crossings, please reference Section 10 of this DDM.

One series of popular culvert analyses and design manuals are the *Hydraulic Engineering Circulars (HEC)*, HEC-5 titled *Hydraulic Charts for the Selection of Highway Culverts,* HEC-10 titled *Capacity Charts for the Hydraulic Design of Highway Culverts, and* HEC-13 titled *Hydraulic Design of Improved Inlets for Culverts.* In September 1985, the FHWA published Hydraulic Design Series (HDS) No. 5, "Hydraulic Design of Highway Culverts" (FHWA, 2001) that combined the information contained in HEC-5, HEC-10, and HEC-13 into one comprehensive document on culvert design.

The charts and procedures for culvert design used in this section are based on the principles in the FHWA's, *Hydraulic Series Number 5, Hydraulic Design of Highway Culverts* (USDOT, FWHA, HDS-5. 1985). The designer is referred to this document for an in-depth discussion and derivation of the equations presented herein. Culvert designers use this reference liberally as it is the result of years of research and experience in culvert design and at this time represents the state of the art.

9.1. Culvert Flow Control

In the context of this DDM, a culvert is defined as a short conduit used to convey stream flow and/or storm water beneath a roadway. Culverts are often referred to as "hydraulically short," meaning they usually flow partially full, although they can flow full for a short part of the overall culvert length. A "hydraulically long" culvert is one in which full flow occurs over the majority of the overall culvert length.

Culverts come in a variety of shapes and materials and the selection of which to use is generally based on hydraulics and economics. Several different types of culverts can be used for any one situation, but there is usually one that is less expensive, stronger, more resistant to corrosion, or hydraulically more efficient than the others, which makes the selection more prudent. Culverts are generally constructed of concrete, steel, or aluminum, and to a lesser but growing degree, plastic. Shapes are usually circular, box (rectangular), or arch.

Regardless of the size of the culvert, street crossings shall be designed to convey the 100-year storm runoff under the roadway to an area downstream of the crossing. Flows up to and including the 100-year frequency event <u>should not</u> cause increased flooding to adjacent property or buildings, unless a drainage easement is acquired for those areas.

A culvert's performance is affected by the type of flow control. Flow can be controlled by the inlet conditions, known as inlet control, or by outlet and culvert conditions, known as outlet control. In the case of hydraulically long culverts, the culvert cross-sectional shape and roughness may control the flow, termed barrel control.

9.2. General Culvert Design

The design procedures in HDS-5 are based on the principles of the control section and minimum performance.

The control section is the location where there is a unique relationship between the flow rate and the upstream water surface elevation. Inlet control is governed by the inlet geometry. Outlet control is governed by a combination of the culvert geometry, the barrel characteristics, and the tailwater. Minimum performance is assumed by analyzing both inlet and outlet control and using the highest headwater condition as the flow control. The culvert may operate more efficiently at times (more flow for a given headwater level), but it will not operate at a lower level of performance than calculated.

There are computer culvert design programs that the City of El Paso accepts in lieu of manual calculations, including:

- CulvertMaster.
- Hydrain HY-8.

These programs are user friendly and are preferred over manual calculations. Input information required to run these programs are:

- Upstream and downstream channel configuration and section properties.
- Required and design flow rates.
- Culvert type, size, length, and inverts.
- Roadway configuration and section properties.
- Headwall information, if any.

It is recommended that the users of these programs be experienced with these programs if used to determine the culvert design. The use of these programs to determine the headwater elevations, discharge or size of single or multiple culvert alignments with various flow conditions is approved and their use and input values should be further documented in engineering submittals to the City of El Paso.

It should be noted that CulvertMaster is listed on FEMA's approved Hydraulic Models list, and should be used above Hydrain when analysis shows that the project will require additional submittal to FEMA.

9.3. Inlet Control

Inlet control occurs when the culvert barrel is able to convey more water than the inlet conditions will allow. In this case the inlet geometry and headwater conditions control the flow at the inlet. Inlet geometry includes the culvert shape, the inlet edge shape, and the culvert size. Critical flow depth occurs at the entrance to the culvert and flow though the culvert is supercritical. Figure 9-1 shows types of inlet control and is adapted from the HDS-5.

Under low headwater conditions, when the entrance is unsubmerged, the culvert entrance will perform as a weir, as shown in Figure 9-1 A and B. When high headwaters submerge the inlet as shown in Figure 9-1, C and D, the entrance performs as an orifice.

For inlet control, the control section is at the upstream end of the barrel (the inlet). The low passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet. Inlet edge configuration describes the entrance type. Some typical inlet edge configurations are thin edge projecting, mitered, square edges in a headwall, and beveled edge.

Inlet shape is usually the same as the shape of the culvert barrel. Typical shapes are rectangular, circular, elliptical, and arch. Check for an additional control section, if the inlet shape is different than the barrel shape. Tapered inlets are discussed later in this section.

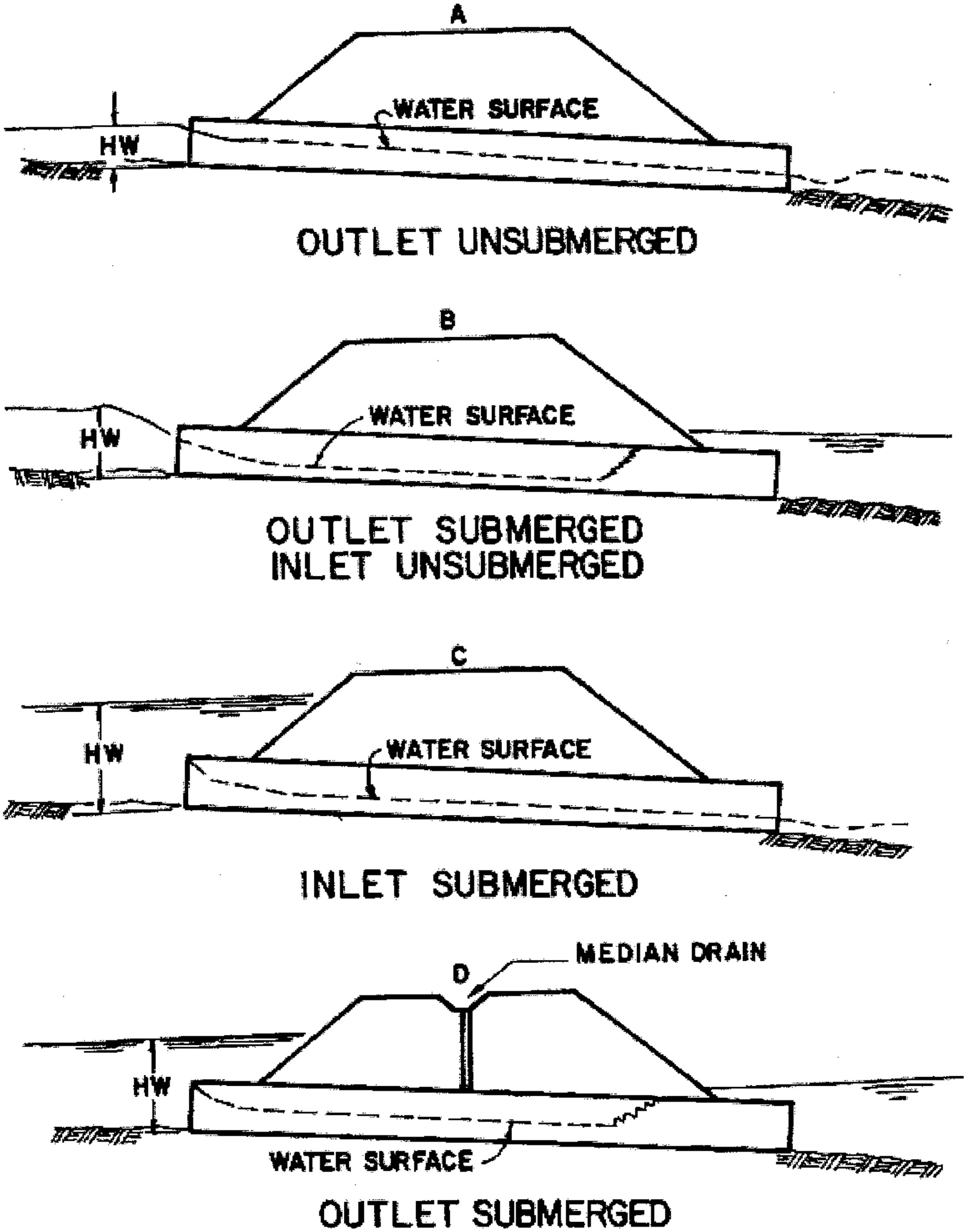


Figure 9-1: Types of inlet control

Three conditions of flow occur under inlet control: unsubmerged, transition, and submerged. The unsubmerged condition occurs when the headwater is below the inlet crown and the entrance operates as a weir. The submerged condition occurs when the headwater is above the inlet and the culvert operates as an orifice. The transition zone is located between the unsubmerged and the submerged flow conditions where the flow is poorly defined. This zone is approximated by plotting the unsubmerged and submerged flow equations and connecting them with a line tangent to both curves. The inlet control design equations are:

9.3.1. Unsubmerged Inlet

Form (1) $$\frac{HW_i}{D} = \frac{H_c}{D} + K\left[\frac{Q}{AD^{0.5}}\right]^M - 0.5S \qquad 9\text{-}1$$

Form (2) $$\frac{HW_i}{D} = K\left[\frac{Q}{AD^{0.5}}\right] \qquad 9\text{-}2$$

9.3.2. Submerged Inlet

$$\frac{HW_i}{D} = c\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S \qquad 9\text{-}3$$

Where:

HW_i = Headwater depth above inlet control section invert, in feet.

D = Interior height of culvert barrel, in feet.

H_c = Specific head at critical depth ($d_c + V^2_c/2g$), in feet.

Q = Discharge, in cubic feet per second.

A = Full cross sectional area of culvert barrel, in square feet.

S = Culvert barrel slope, in feet per feet.

K, M, c, Y = Constants from Table 9-1.

NOTES:

1. Equation 9-1 and 9-2 (unsubmerged) apply up to about $Q/AD^{0.5} = 3.5$
2. Equation 9-1 and 9-3, for mitered inlets, use +0.7S instead of -0.5S as the slope correction factor.
3. Equation 9-3 (submerged) applies when $Q/AD^{0.5}$ is approximately 4.0 or greater.

Note that there are two forms of the unsubmerged equation. Form (1) is based on the specific head at critical depth, adjusted with two correction factors. Form (2) is an exponential equation similar to a weir equation. Form (1) is preferable from a theoretical standpoint, but Form (2) is easier to apply and is the only documented form of equation for some of the inlet control monographs. Either form of unsubmerged inlet control equation will produce adequate results.

Table 9-1 provides the constants for the equation and the unsubmerged and submerged equation coefficients for each shape, material, and edge configuration. For the unsubmerged, the form of the equation is also noted within the table.

Table 9-1: Constants for Inlet Control Design Equations

Chart No.	Shape and Material	Nomograph Scale	Inlet Edge Description	Equation Form	Unsubmerged K	Unsubmerged M	Submerged c	Submerged Y	References
1	Circular Concrete	1	Square edge w/headwall	1	.0098	2.0	.0398	.67	56/57
		2	Groove end w/headwall		.0018	2.0	.0292	.74	
		3	Groove end projecting		.0045	2.0	.0317	.69	
2	Circular CMP	1	Headwall	1	.0078	2.0	.0379	.69	56/57)
		2	Mitered to slope		.0210	1.33	.0463	.75	
		3	Projecting		.0340	1.50	.0553	.54	
3	Circular	A	Beveled ring, 45° bevels	1	.0018	2.50	.0300	.74	57
		B	Beveled ring, 33.7° bevels*		.0018	2.50	.0243	.83	
8	Rectangular Box	1	30° to 75° wingwall flares	1	.026	1.0	.0347	.81	56
		2	90° and 15° wingwall flares		.061	.75	.0400	.80	56
		3	0° wingwall flares		.061	.75	.0423	.82	8
9	Rectangular Box	1	45° wingwall flare d = .043D	2	.510	.667	.0309	.80	8
		2	18° to 33.7° wingwall flare d = .083D		.486	.667	.0249	.83	
10	Rectangular Box	1	90° headwall w/3/4" chamfers	2	.515	.667	.0375	.79	8
		2	90° headwall w/45° bevels		.495	.667	.0314	.82	
		3	90° headwall w/33.7° bevels		.486	.667	.0252	.865	
11	Rectangular Box	1	3/4" chamfers; 45° skewed headwall	2	.545	.667	.04505	.73	8
		2	3/4" chamfers; 30° skewed headwall		.533	.667	.0425	.705	
		3	3/4" chamfers; 15° skewed headwall		.522	.667	.0402	.68	
		4	45° bevels; 10°-45° skewed headwall		.498	.667	.0327	.75	
12	Rectangular Box 3/4" chamfers	1	45° non-offset wingwall flares	2	.497	.667	.0339	.803	8
		2	18.4° non-offset wingwall flares		.493	.667	.0361	.806	
		3	18.4° non-offset wingwall flares 30° skewed barrel		.495	.667	.0386	.71	
13	Rectangular Box Top Bevels	1	45° wingwall flares - offset	2	.497	.667	.0302	.835	8
		2	33.7° wingwall flares - offset		.495	.667	.0252	.881	
		3	18.4° wingwall flares - offset		.493	.667	.0227	.887	
16-19	C M Boxes	2	90° headwall	1	.0083	2.0	.0379	.69	57
		3	Thick wall projecting		.0145	1.75	.0419	.64	
		5	Thin wall projecting		.0340	1.5	.0496	.57	

29	Horizontal	1	Square edge w/headwall	1	.0100	2.0	.0398	.67	57
	Ellipse	2	Groove end w/headwall		.0018	2.5	.0292	.74	
	Concrete	3	Groove end projecting		.0045	2.0	.0317	.69	
30	Vertical	1	Square edge w/headwall	1	.0100	2.0	.0398	.67	57
	Ellipse	2	Groove end w/headwall		.0018	2.5	.0292	.74	
	Concrete	3	Groove end projecting		.0095	2.0	.0317	.69	
34	Pipe Arch	1	90° headwall	1	.0083	2.0	.0379	.69	57
	18" Corner	2	Mitered to slope		.0300	1.0	.0463	.75	
	Radius CM	3	Projecting		.0340	1.5	.0496	.57	
35	Pipe Arch	1	Projecting	1	.0300	1.5	.0496	.57	56
	18" Corner	2	No Bevels		.0088	2.0	.0368	.68	
	Radius CM	3	33.7° Bevels		.0030	2.0	.0269	.77	
36	Pipe Arch	1	Projecting	1	.0300	1.5	.0496	.57	56
	31" Corner		No Bevels		.0088	2.0	.0368	.68	
	Radius CM		33.7° Bevels		.0030	2.0	.0269	.77	
41-43	Arch CM	1	90° headwall	1	.0083	2.0	.0379	.69	57
		2	Mitered to slope		.0300	1.0	.0463	.75	
		3	Thin wall projecting		.0340	1.5	.0496	.57	
55	Circular	1	Smooth tapered inlet throat	2	.534	.555	.0196	.90	3
		2	Rough tapered inlet throat		.519	.64	.0210	.90	
56	Elliptical	1	Tapered inlet-beveled edges	2	.536	.622	.0368	.83	3
	Inlet Face	2	Tapered inlet-square edges		.5035	.719	.0478	.80	
		3	Tapered inlet-thin edge projecting		.547	.80	.0598	.75	
57	Rectangular	1	Tapered inlet throat	2	.475	.667	.0179	.97	3
58	Rectangular	1	Side tapered-less favorable edges	2	.56	.667	.0446	.85	3
	Concrete	2	Side tapered-more favorable edges		.56	.667	.0378	.87	
59	Rectangular	1	Slope tapered-less favorable edges	2	.50	.667	.0446	.65	3
	Concrete		Slope tapered-more favorable edges		.50	.667	.0378	.71	

9.3.3. Design Nomographs

Nomographs for culvert design under inlet control can be found in HDS-5. The inlet control nomographs are based on the plotted curves for unsubmerged and submerged inlet conditions.

The partly full flow outlet control nomographs were developed based on numerous backwater calculations performed by FHWA staff. They found that the hydraulic grade line pierces the plane of the culvert outlet at a point one-half way between critical depth and the top of the barrel or (dc + D)/2 above the outlet invert. TW should be used if higher than (dc + D)/2.

9.4. Outlet Control

Outlet and barrel control occur when the culvert barrel can not convey as much flow as the culvert inlet allows entry. The geometry of the culvert, as well as the hydraulic characteristics of the culvert, control the flow conditions through the culvert for outlet control. In addition, headwater and tailwater conditions, as well as slope, length, and roughness of the culvert barrel can control flow conditions. Control of flow is at the outlet end of the culvert or further downstream in the receiving channel and flow may be subcritical or pressure flow through the culvert. Figure 9-2 shows types of outlet control.

Outlet control has depths and velocity that are subcritical. The control of the flow is at the downstream end of the culvert (the outlet). The tailwater depth is either assumed to be critical depth near the culvert outlet or the downstream channel depth, whichever is higher.

Full flow in the culvert barrel is assumed for the analysis of outlet control hydraulics. Outlet control flow conditions can be calculated based on an energy balance from the tailwater pool to the headwater pool.

The performance of a culvert may change from inlet to outlet control at different flow rates. Therefore, a culvert is designed for the worst case (minimum performance) with the assumption that it may perform more efficiently in some situations, but never will it perform less efficiently than at design flow rates.

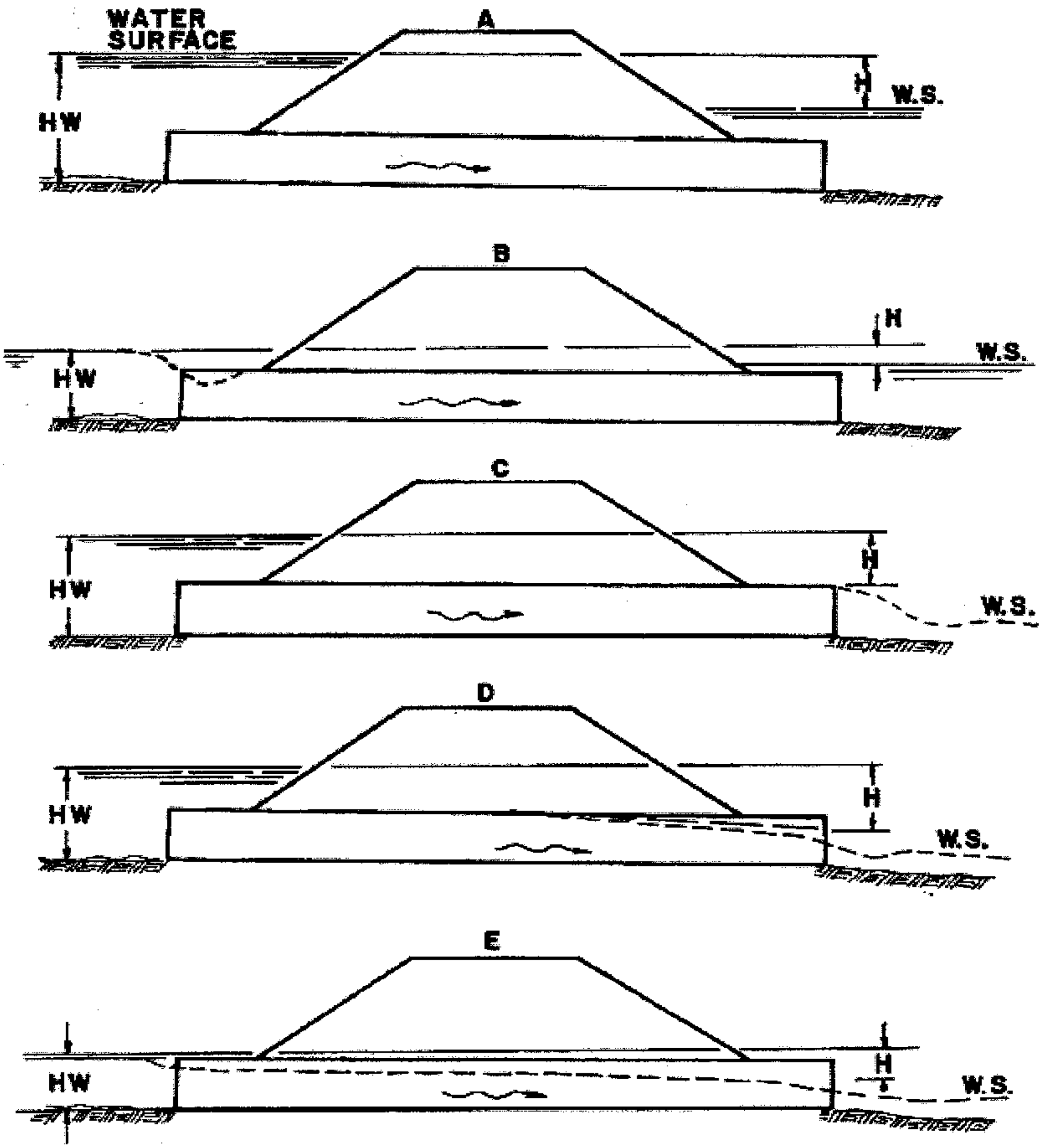


Figure 9-2: Types of Outlet Control

9.5. Design Nomographs

Nomographs for culvert design under inlet and outlet control can be found in HDS-5 are included in Appendix A. The inlet control nomographs are based on the plotted curves for unsubmerged and submerged inlet conditions.

The full flow outlet control nomographs were developed assuming that the culvert barrel is flowing full and the following:

- Tailwater (TW) > culvert diameter (D).
- Critical depth (d_c) > D.
- Upstream velocity is small and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert.
- Downstream velocity is small and its velocity head can be neglected.

The partly full flow outlet control nomographs were developed based on numerous backwater calculations performed by FHWA staff. They found that the hydraulic grade line pierces the plane of the culvert outlet at a point one-half way between critical depth and the top of the barrel or (dc + D)/2 above the outlet invert. TW should be used if higher than (dc + D)/2.

9.6. Minimum Velocity

Culverts should be designed to provide adequate velocity to self-clean during partial depth flow events. Debo and Reese (1995) suggest a minimum velocity of 2.5 feet per second for partial flow depths. Greater velocities are recommended for installations where sediment loads are heavy. Alternatively, a sediment trap can be utilized where culvert velocities are lower or excessive sediment deposition is expected.

9.7. Maximum Velocity

As a practical limit, outlet velocities should be kept below 15 feet per second unless special conditions exist. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. As outlet velocities increase, the need for channel stabilization at the culvert outlet increases. If culvert outlet velocities exceed permissible velocities for the outlet channel lining material, suitable outlet protection must be provided. Outlet velocities may exceed permissible downstream channel velocities by up to 10 percent without providing outlet protection if the culvert tailwater depth is greater than the culvert critical depth of flow under design flow conditions. Table 9-2 outlines the permissible velocities for several channel lining materials.

Table 9-2: Maximum Permissible Velocities for Roadside Drainage Channels with Erodible Linings

Soils Type of Lining (Earth, No Vegetation)	Permissible Velocity (1) (2), ft/sec
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (noncolloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0

(1) For sinuous channels multiply permissible velocity by: 0.95 for slightly sinuous, 0.90 for moderately sinuous, and 0.80 for highly sinuous

(2) Higher velocities may be allowed for design of unlined channels, for the 100-year design event in particular, based on sediment balance considerations defined using the guidelines in Chapter 14. However, sufficient setback allowance should be provided for expected bank erosion during the 100-year event or a series of annualized events over a 60-year period. Higher velocities may also be acceptable for 100-year peak flow design with approved engineering justification based on attractive force analysis (USDOT, FHWA HEC-11).

9.8. Outlet Velocity

Culvert outlet velocities should be calculated to determine the need for erosion protection at the culvert exit. Culverts usually result in outlet velocities that are higher than the natural stream velocities. These outlet velocities may require flow readjustment or energy dissipation to prevent downstream erosion. If outlet erosion protection is necessary, the flow depths and Froude number may also be needed. Additional calculations should be prepared in agreement with the FHWA's, *Hydraulic Engineering Circular HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels, July 2006*, if resulting outlet flow velocity and channel materials require flow readjustment or energy dissipation.

For culverts flowing full, use the Manning equation to determine flow velocity. For culverts flowing partially full, Figure 9-3 can be used to find the flow velocity.

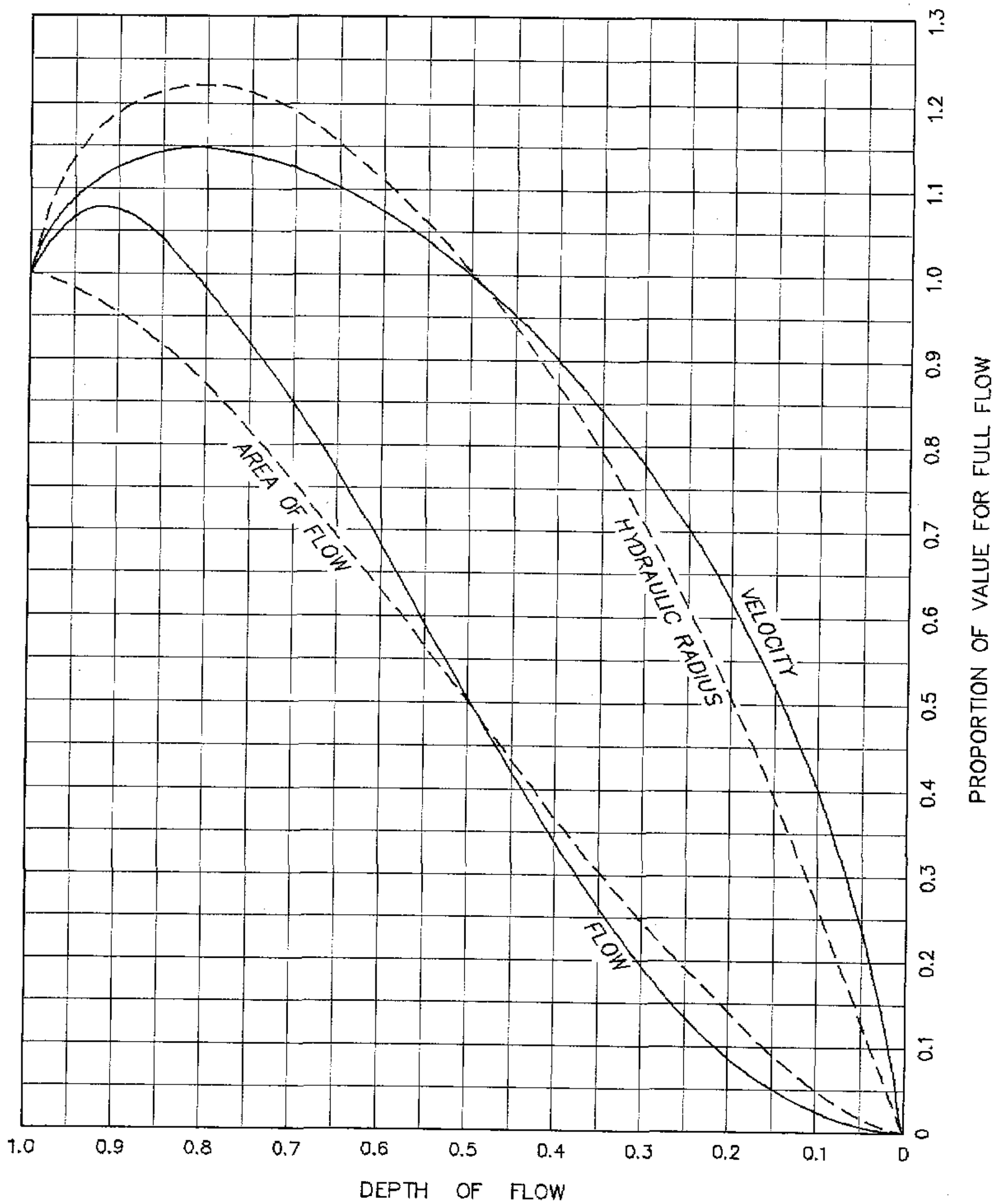


Figure 9-3: Hydraulic Elements Nomograph for Circular Pipe

9.9. Head Loss in Culverts

The total loss of head in a culvert is the sum of the entrance and outlet losses, which vary with the design of the ends, plus the loss of head due to friction, which varies directly with the length. It is convenient to treat culverts flowing full, with both ends submerged, as submerged tubes, and to include all losses of head in the coefficient of discharge C (Equation 9-4). To obtain the maximum discharge, the intake must be submerged to a depth equal to at least the sum of the velocity head and the loss of head at the entrance. The discharge is entirely independent of the slope of the tube. It depends only on h, the difference in elevation of the water surfaces at the intake and the outlet.

$$Q = CA\sqrt{2gh} \qquad \text{9-4}$$

Where:

Q = Flow through and orifice discharging into the atmosphere.

C = Coefficient of discharge.

A = Area of the orifice opening, in square meters.

g = Gravitational acceleration, in meters per second squared.

h = Height of water above orifice centerline, in meters.

The results of 1,480 experiments on concrete-, vitrified-clay-, and corrugated-metal-pipe culverts and 1,821 experiments on concrete-box culverts were published by Yarnell (1924). Experiments were conducted on pipes 30, 45, 60, and 75 cubic centimeters in diameter and on box culverts 0.6 by 0.6, 0.9 by 0.9, 1.2 by 1.2, 1.2 by 0.9, 1.2 by 0.7, 1.2 by 0.6, 1.2 by 0.3, 1.2 by 9.1, and 11.0 meters. If d is the diameter of pipe culverts, R the hydraulic radius of box culverts, and l the length of the culvert, the following are expressions for determining C in Equation 9-4. Equations 9-5 through 9-10 are in metric units.

For concrete pipe, beveled-lip entrance:

$$C = \left(1.1 + \frac{0.025l}{d^{1.2}}\right)^{\frac{1}{2}} \qquad \text{9-5}$$

For concrete pipe, square-cornered entrance:

$$C = \left(1 + 0.5d^{0.5} + \frac{0.025l}{d^{1.2}}\right)^{-\frac{1}{2}} \qquad \text{9-6}$$

For vitrified-clay pipe, bell and upstream:

$$C = \left(1 + 0.22d^{1.9} + \frac{0.022l}{d}\right)^{-\frac{1}{2}} \qquad \text{9-7}$$

For corrugated-metal pipe:

$$C = \left(1 + 0.33d^{0.6} + \frac{0.084l}{d^{1.2}}\right)^{-\frac{1}{2}} \qquad 9\text{-}8$$

For concrete-box culverts with rounded-lip entrance:

$$C = \left(1.05 + \frac{0.0033l}{R^{1.25}}\right)^{-\frac{1}{2}} \qquad 9\text{-}9$$

For concrete-box culverts with square-cornered entrance:

$$C = \left(1 + 0.57R^{0.3} + \frac{0.0033l}{R^{1.25}}\right)^{-\frac{1}{2}} \qquad 9\text{-}10$$

The experiments that led to the development of these equations were performed on culverts with lengths that ranged from 23 to 36 feet. For culverts that are longer than 36 feet, it is recommended that the equations only be applied to the first 36 feet, and the head losses associated with the remaining length of conduit is computed using open channel or pipe equations.

9.10. Roadway Overtopping

Roadway overtopping will begin when the headwater rises to the elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad-crested weir. Overtopping flow rate can be computed with the following equation:

$$Q_r = C_d L (HW_r)^{1.5} \qquad 9\text{-}11$$

Where:

Q_r = Overtopping flow rate, in cubic feet per second.

C_d = Overtopping discharge coefficient = $k_t C_r$ (see Figure 9-4 or use Table 9-3).

k_t = Submergence coefficient.

C_r = Discharge coefficient.

L = Length of the roadway crest, in feet.

HW_r = The upstream depth, measured above the roadway crest, in feet.

The charts in Figure 9-4 provide estimates of the correction factors k_t and C_r.

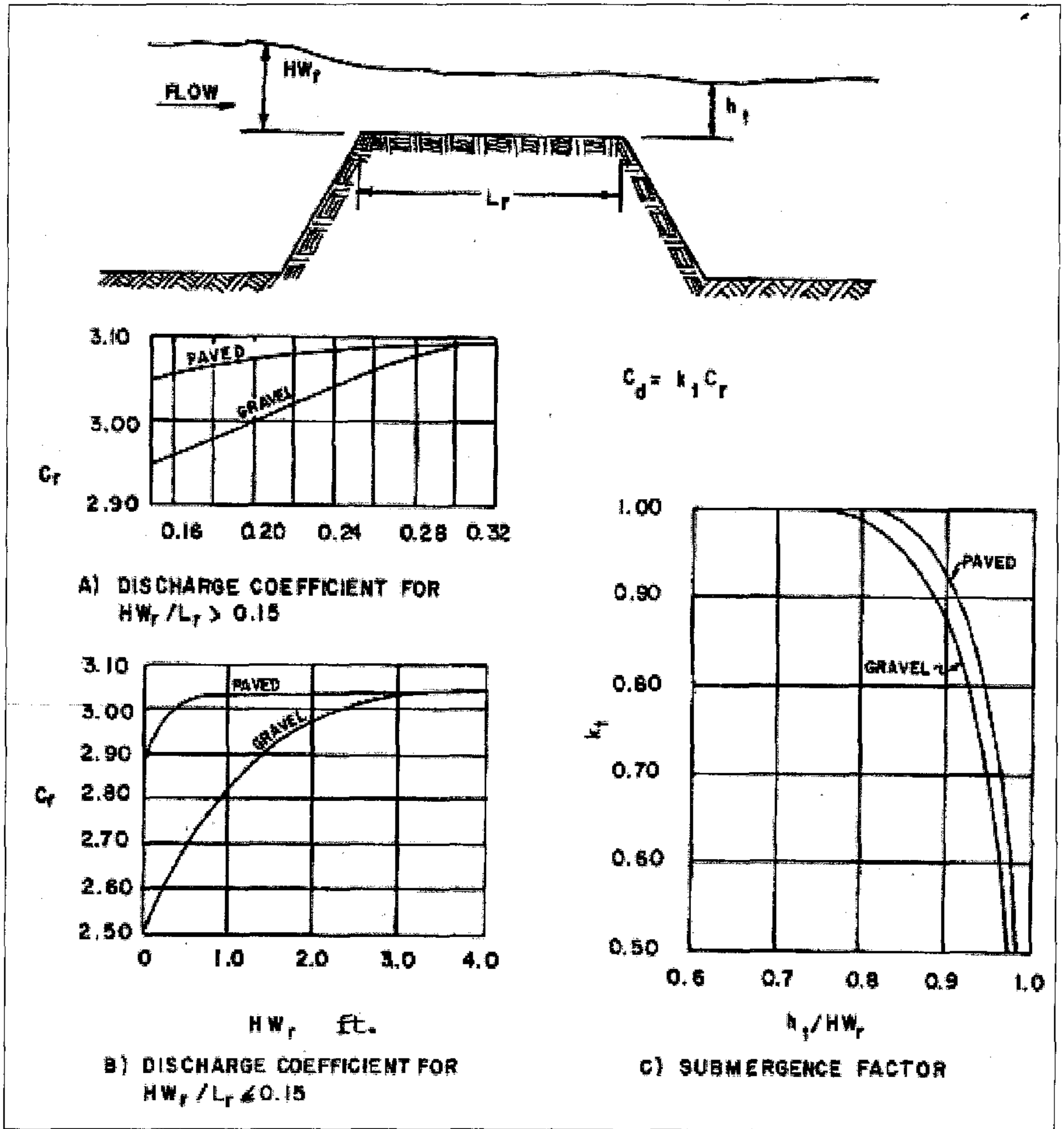


Figure 9-4: Discharge Coefficients for Roadway Overtopping

Table 9-3: Values of C_d for Weirs of Trapezoidal Cross Sections with Both Faces Inclined*

Slope of upstream face, horizontal: vertical	Slope of downstream face, horizontal: vertical	Width of crest, cm	Head H, cm							
			5	10	15	20	25	30	40	50
1:2	1:1	20	1.49	1.56	1.67	1.76	1.85	1.94	2.02	2.11
1:2	2:1	20	1.50	1.54	1.61	1.71	1.81	1.87	1.93	1.99
1:2	3:1	20	1.49	1.52	1.61	1.68	1.73	1.81	1.86	1.91
1:2	4:1	20	1.50	1.51	1.59	1.67	1.72	1.77	1.81	1.85
1:2	5:1	20	1.50	1.55	1.59	1.64	1.70	1.75	1.78	1.80
1:2	2:1	40		1.50	1.55	1.56	1.59	1.65	1.70	1.78
1:2	4:1	40		1.52	1.56	1.57	1.59	1.62	1.66	1.71
1:2	6:1	40			1.55	1.57	1.59	1.62	1.65	1.70
2:1	2:1	20	1.56	1.62	1.73	1.78	1.83	1.90	1.94	1.99
1:1	2:1	20	1.51	1.58	1.67	1.74	1.82	1.89	1.95	2.02
1:3	2:1	20	1.38	1.45	1.59	1.68	1.76	1.85	1.91	1.96
Vertical	2:1	20	1.41	1.43	1.53	1.62	1.71	1.80	1.87	1.94

*Table indicates that values of C increase slightly for heads above 50 cm.

If the elevation of the roadway crest varies, for instance where the crest is defined by a roadway sag vertical curve, the vertical curve can be approximated as a series of horizontal segments. The flow over each is calculated separately and the total flow across the roadway is the sum of the incremental flows for each segment (Figure 9-5).

If the assumption of horizontal segments is invalid ($HW_{ra} > 1.5\ HW_{rb}$), the following formula may be used, assuming the value of C_r remains constant:

$$Q_o = \frac{2k_t C_d L(HW_{ra}^{5/2} - HW_{ra}^{5/2})}{5(HW_{ra} - HW_{ra})} \qquad \text{9-12}$$

Where:

Q_o = *Overtopping flow rate, in cubic feet per second.*

k_t = *Submergence factor.*

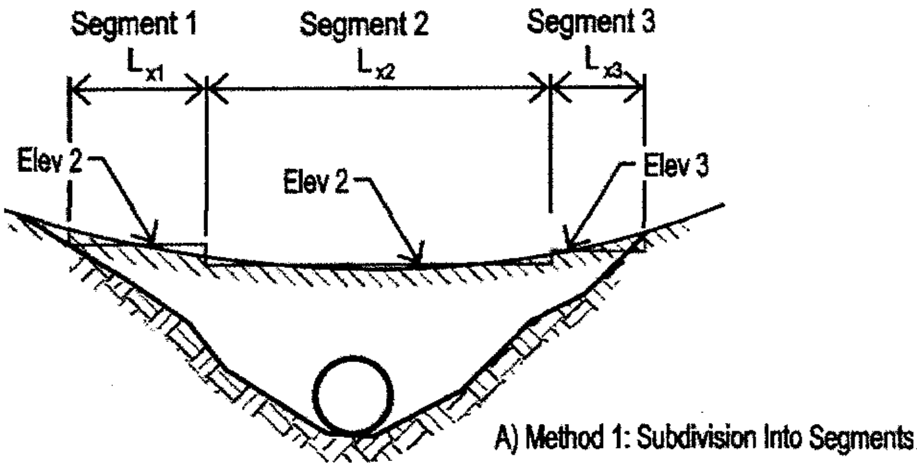
C_r = *Discharge coefficient.*

L = *Length of the roadway crest, in feet.*

HW_{ra} = *Flow depth above the roadway at the high end of the weir segment, in feet.*

HW_{rb} = *Flow depth above the roadway at the low end of the weir segment, in feet.*

The total flow across the roadway then equals the sum of the roadway overflow plus the culvert flow. A performance curve must be plotted including both culvert flow and road overflow. The headwater depth for a specific discharge, such as the 100-year discharge can then be read from the curve.



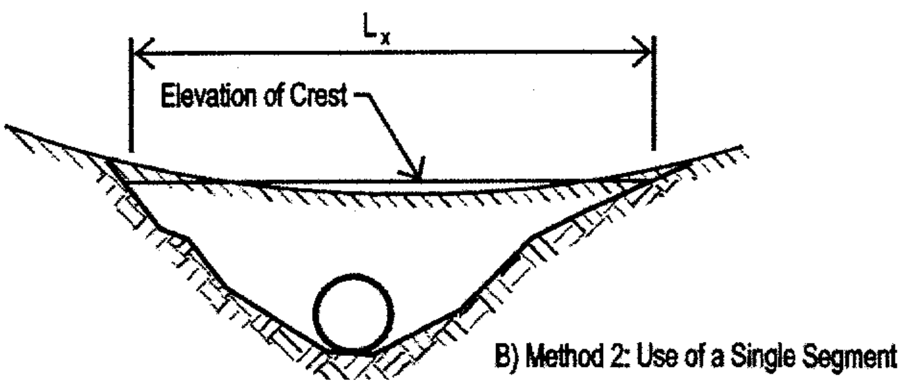


Figure 9-5: Weir Crest Length Determinations for Roadway Overtopping

9.11. Performance Curve – Roadway Overtopping

A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. The performance curve is a plot of flow rate versus headwater depth, elevation or velocity. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed. The performance curve depicts the sum of the flow through the culvert and the flow across the roadway.

The culvert performance curve is made up of the controlling portion of the individual performance curves for each of the following control sections (see Figure 9-6). The performance curve data is also used in retention pond analysis to model outflow structure.

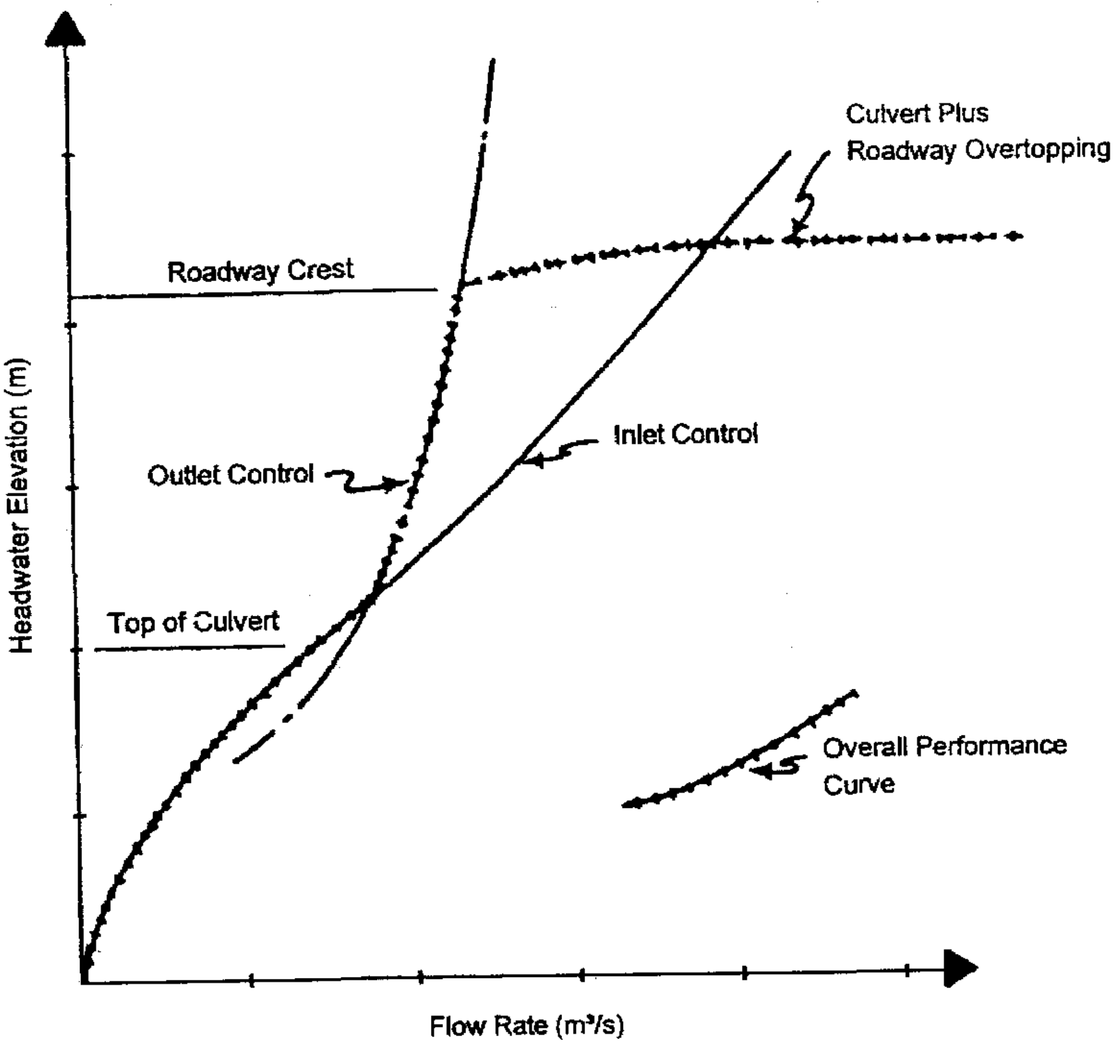


Figure 9-6: Overall Performance Curve

- Inlet – The inlet performance curve is developed using the inlet control nomographs in HDS-5, 1983.
- Outlet – The outlet performance curve is developed using the equations presented in HDS-5, the outlet control nomographs in HDS-5, 1983, or backwater calculations.
- Roadway – The roadway performance curve is developed using Equation 9-13.
- Overall – The overall performance curve is the sum of the flow through the culvert and the flow across the roadway and can be determined by performing the following steps:
    1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated. It is recommended

that the 2-, 10-, 50- and 100-year flow rates be included in the range of flow rates considered.

2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert based on the controlling stage for each discharge.
3. When the culvert headwater stages exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 9-11 or 9-12 to calculate flow rates across the roadway.
4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

Using the combined culvert performance curve, it is an easy matter to determine the headwater stage for any flow rate, or to visualize the performance of the culvert installation over a range of flow rates. When roadway overtopping begins, the rate of headwater increase will diminish. The headwater will rise very slowly from that point on. Figure 9-7 depicts an overall culvert performance curve with roadway overtopping. The 100-year discharge should be identified on the performance curve and the corresponding depth of flow over the roadway.

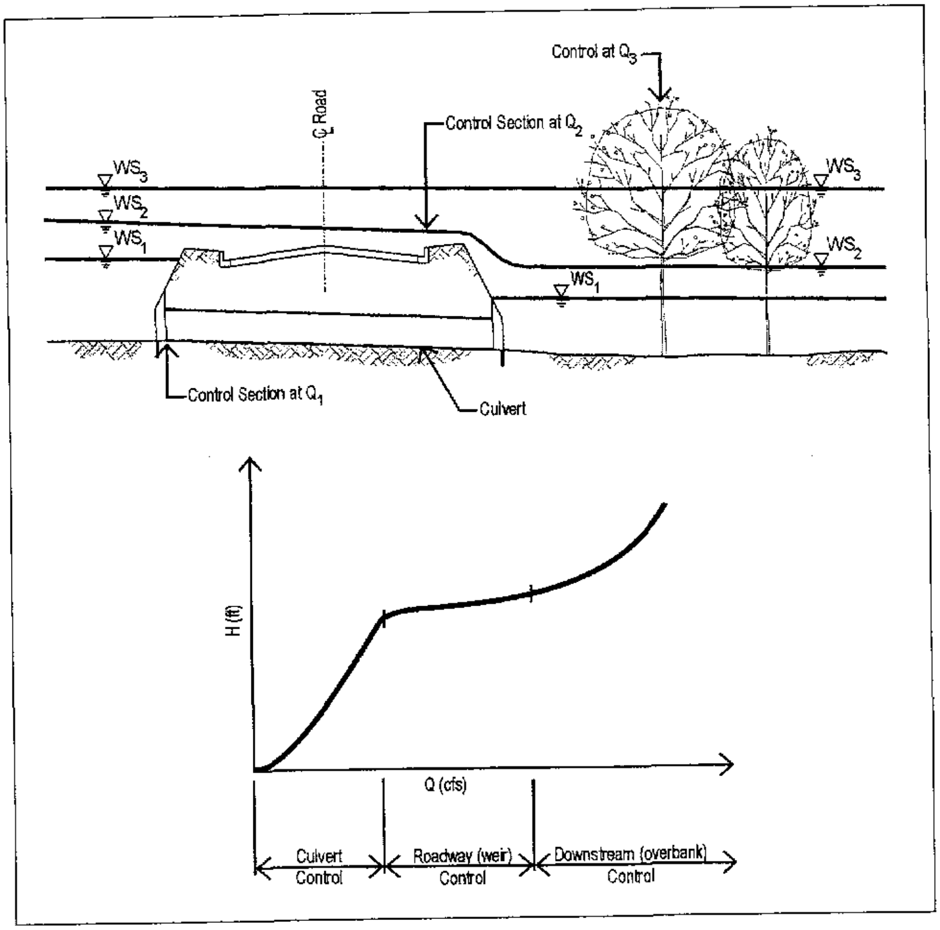


Figure 9-7: Culvert Performance Curve with Roadway Overtopping

The FHWA's computer program, HY8 (FHWA, 1999), can be used in the development of performance curves. HY8 automates the design methods described in HDS-5 (FHWA, 1985), and HEC-14 (FHWA, 1983). The USACE HEC-2 (USACE, 1990) and HEC-RAS computer programs (USACE, 2001) are also capable of analyzing culverts. The use of HY8 is preferred for design of culverts that are not subject to backwater conditions. HEC-RAS is preferred for modeling and design of culverts in river systems where backwater effects are of concern.

9.12. Materials

The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance. The culvert materials that should be considered are concrete (reinforced and non-reinforced), corrugated aluminum, corrugated steel, and PVC. Culverts may also be lined with other materials to inhibit corrosion and abrasion. Linings are not recommended to reduce hydraulic resistance because culvert linings have a short life span and are seldom reapplied as part of normal culvert maintenance. When linings are applied, the culvert sizing should neglect the reduced roughness from the lining material.

9.13. Minimum Cover

Minimum cover of fill over culverts must be provided to maintain the structural integrity of the structure under anticipated loading conditions. Culvert manufacturers provide minimum cover requirements for prefabricated pipe. A general rule of thumb for estimating minimum cover requirements is to provide one-eighth of the barrel diameter or span, with a minimum of 1 foot. The top of culverts should not extend into the roadway subgrade. Minimum cover should be measured from the top of subgrade, which is the bottom of the pavement structural section.

When a manufacture's minimum cover requirements dictate cover larger than the one-eighth of the barrel diameter or span calculated, the larger cover requirement shall dominate.

9.14. Flotation and Anchorage

Flotation is the term used to describe the failure of a culvert due to the uplift forces caused by buoyancy. The buoyant force is produced from a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet caused by flow separation. As a result, a large bending moment is exerted on the end of the culvert. This problem has been noted in the case of culverts under high head, with shallow cover, on steep slopes, and with projecting inlets. The phenomenon can also be caused by debris blocking the culvert end or by damage to the inlet. The resulting uplift may cause the inlet ends of the barrel to rise and bend. Occasionally, the uplift force is great enough to dislodge the embankment. Generally, flexible barrel materials are more vulnerable to failure of this type because of their light weight and lack of resistance to longitudinal bending. Large, projecting, or mitered corrugated metal culverts are the most susceptible.

A number of precautions can be taken by the designer to guard against flotation. Steep slopes (1 to 1 or steeper) of adequate height, which are protected against erosion by slope paving or headwalls, help inlet and outlet stability. When embankment fill heights are less than 1.5 times the pipe diameter or fill slopes are flatter than 1 to 1, the designer may consider other applications such as concrete encasement, concrete headwalls, and tie bars to guard against failures caused by flotation. Limiting headwater buildup also helps prevent flotation. It is desirable to limit design headwater depths to 1.5 times the culvert height.

9.15. Skewed Channels

A good culvert design is one that limits the hydraulic and environmental stress placed on an existing natural watercourse. This stress can be minimized by designing a culvert that closely conforms to the natural stream in alignment and grade. Often the culvert barrel must be skewed with respect to the roadway centerline to accomplish this goal. Alterations to the normal inlet alignment are often necessary as well.

The alignment of a culvert barrel with respect to a line perpendicular to the roadway centerline at the point of crossing is referred to as the barrel skew angle. A culvert aligned normal to the roadway centerline has a zero skew angle. Directions (right or left) must accompany the barrel skew angle (Figure 9-8). Some advantages of following a natural stream alignment include reduction of entrance losses, equal depths of scour at the footings, less sedimentation, and less excavation for installation.

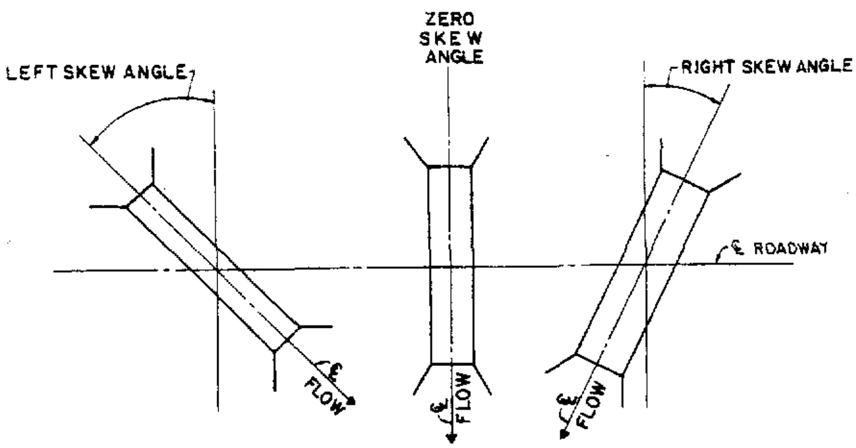


Figure 9-8: Barrel Skew Angle

The angle from the culvert face to a line normal to the culvert barrel is referred to as the inlet skew angle (Figure 9-9). The structural integrity of circular sections is compromised when the inlet is skewed due to the loss of a portion of the full circular section where the culvert barrel extends beyond the full section. Although concrete headwalls help stabilize the pipe section, structural considerations should not be overlooked in the design of skewed inlets. Culverts that have a barrel skew angle often have an inlet skew angle as well. This is because headwalls are generally constructed parallel to a roadway centerline to avoid warping of the embankment fill.

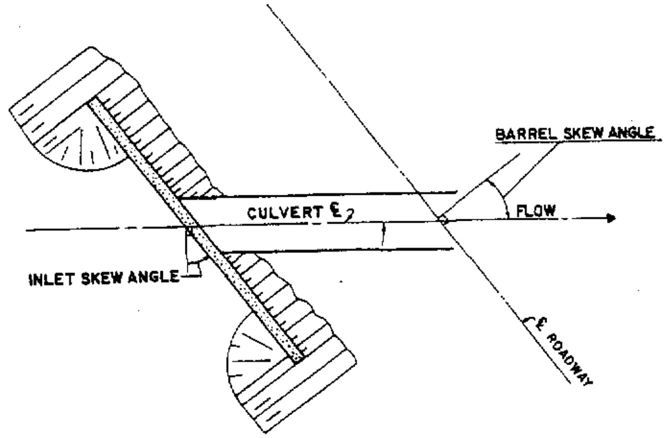


Figure 9-9: Inlet Skew Angle

In cases where the culvert barrel cannot be aligned with the channel flowline, such as when runoff is directed along a roadway embankment to a suitable crossing location, the flow enters the culvert barrel at an angle. The approach angle should be limited to a maximum of 90 degrees. When high velocities exist, inlet losses resulting from turning the flow into the culvert should be considered. If backwater computations are not employed and the approach channel velocity is 6 feet per second or greater, the following equation should be used to estimate the loss. The loss should be added to the other inlet losses in the culvert design computation, if they are not included in the appropriate nomographs.

$$H_t = \left(\frac{V_a^2}{2g}\right)\sin a \qquad \text{9-13}$$

Where:

H_t = Head loss due to turning flow at a headwall, in feet.

V_a = Approach channel velocity, in feet per second.

g = Acceleration due to gravity, 32.2 feet per second squared.

a = Angle of approach, in degrees.

Figure 9-10 presents typical Inlet configurations for skewed channels.

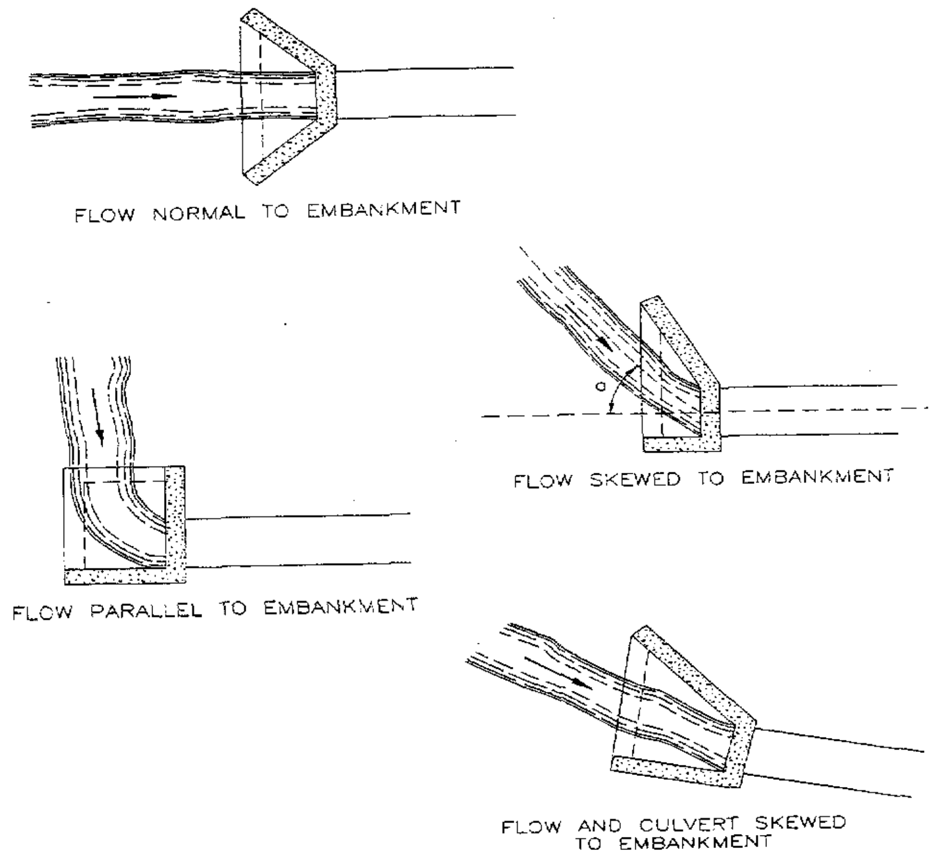


Figure 9-10: Typical Headwall/Wingwall Configurations for Skewed Channels

9.16. Bends

A straight culvert alignment is desirable to avoid clogging, increased construction costs, and reduced hydraulic efficiency. However, site conditions may require a change of alignment, either horizontally or vertically. When considering a nonlinear culvert alignment, particular attention should be given to erosion, sedimentation, and debris control. Vertical bends are permitted when they transition from a flatter to a steeper slope, but should not transition from steeper to flatter slopes because of the potential for sediment deposition in the flatter reach.

In designing a nonlinear culvert, the energy losses due to bends must be considered. If the culvert operates in inlet control, no increase in headwater occurs unless the bend losses cause the culvert to flow under outlet control. If the culvert operates in outlet control, an increase in energy losses and headwater will result due to the bend losses. To minimize these losses, the culvert should be curved or have bends not exceeding 15 degrees at intervals of not less than 50 feet. Under these conditions, bend losses can be ignored. If these conditions cannot be met, analysis of bend losses is required. Bend losses are a function of the velocity head in the culvert barrel. To calculate bend losses, use Equation 9-14:

$$H_b = K_b\left(\frac{V^2}{2g}\right) \qquad \text{9-14}$$

Where:

H_b = *Head loss through a bend of a culvert, in feet.*

V = *Velocity, in feet per second.*

g = *Acceleration due to gravity, 32.2 feet per second squared.*

$$K_b = 0.0033\Delta \qquad (9\text{-}15)$$

Where:

K_b = *Bend headloss coefficient.*

Δ = *Angle of curvature or deflection, in degrees.*

H_b is added to the other outlet losses. The loss coefficients (K_b) for bend losses in conduits flowing full will need to be determined as well with Equation 9-15. The broken back culvert, shown in Figure 9-11, has four possible control sections: the inlet, the outlet, and the two bends. The upstream bend may act as a control section, with the flow passing through critical depth just upstream of the bend. In this case, the upstream section of the culvert operates in outlet control and the downstream section operates in inlet control. Outlet control calculation procedures can be applied to the upstream barrel, assuming critical depth at the bend, to obtain a headwater elevation. This elevation is then compared with the inlet and outlet control headwater elevations for the overall culvert. The controlling flow condition produces the highest headwater elevation. Control at the lower bend is very unlikely. That possible control section can be ignored except for the bend losses in outlet control.

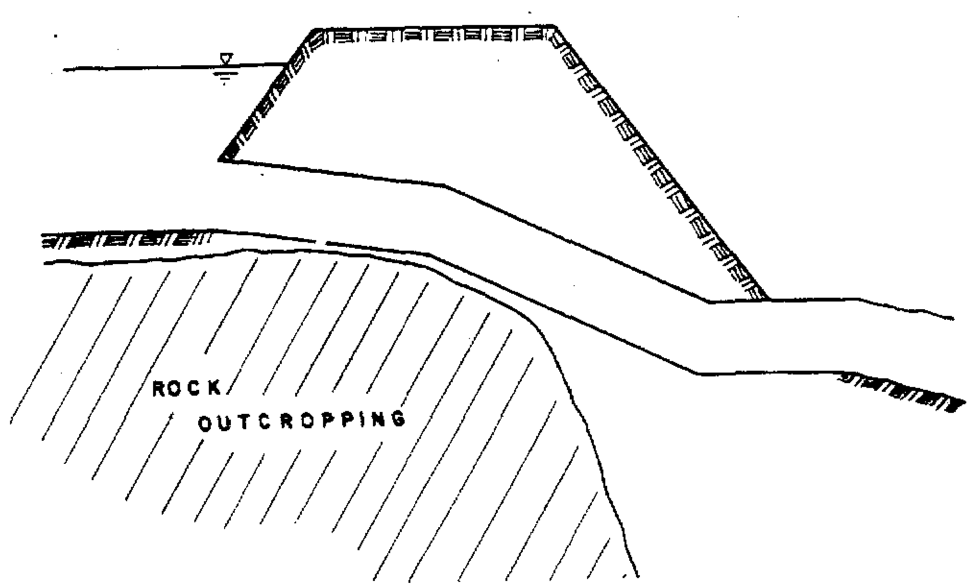


Figure 9-11: "Broken Back" Culvert

9.17. Junctions

Flow from two or more separate culverts or stormdrains may be combined at a junction into a single culvert barrel. For example, a tributary and a main stream intersecting at a roadway crossing can be accommodated by a culvert junction (Figure 9-12).

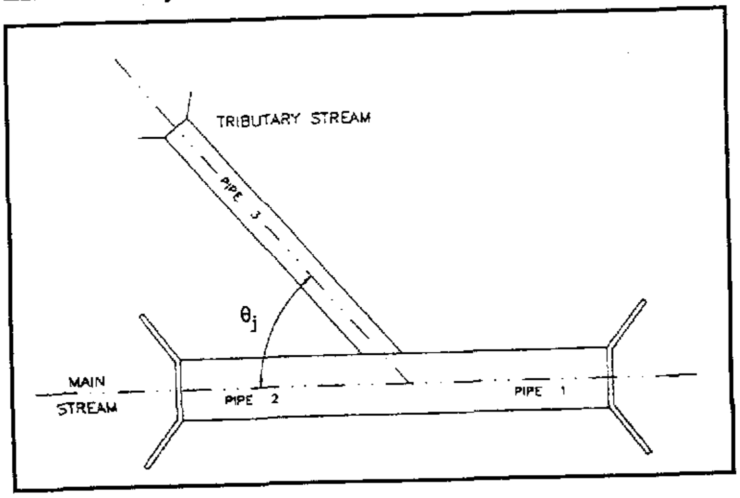


Figure 9-12: Culvert Junction

Loss of head may be important in the hydraulic design of a culvert containing a junction. Attention should be given to streamlining the junction to minimize turbulence and head loss. Also, timing of peak flows from the two branches should be considered in analyzing flow conditions and control. When possible, the tributary flow should be released downstream of the culvert barrel. When this is not practical, the following procedure should be used to estimate the losses. For a culvert barrel operating in outlet control and flowing full, the junction loss is calculated using Equations 9-16 and 9-17 below. The loss is then added to the other outlet control losses.

$$H_j = y' + H_{V1} - H_{V2} \qquad \text{9-16}$$

Where:

H_j = *Head loss through a junction, in feet.*

y' = *Change in hydraulic grade line through the junction, in feet.*

H_{V1} = *Velocity head of outlet pipe, in feet.*

H_{V2} = *Velocity head of upstream pipe, in feet.*

The equation for y' is based on momentum considerations and is as follows:

$$y' = \frac{Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta_j}{0.5(A_1 + A_2)g} \qquad \text{9-17}$$

Where:

Q = *Rate of flow, in cubic feet per second. (1 – outlet, 2 – upstream, 3 – lateral pipe).*

V = *Velocity, in feet per second (1 – outlet, 2 – upstream, 3 – lateral pipe).*

θ_j = *Angle between outfall and lateral at a junction, in degrees.*

A = *Area of the barrel, in square feet.*

g = *Acceleration due to gravity, 32.2 feet per second squared.*

9.18. Tapered Inlets

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. A tapered inlet may have a depression, or fall, incorporated into the inlet structure or located upstream of the inlet. The depression is used to exert more head on the throat section for a given headwater elevation. Therefore, tapered inlets improve culvert performance by providing a more efficient control section (the throat). Tapered inlets with fall also improve performance by increasing the head on the throat.

Tapered inlets are not recommended for use on culverts flowing under outlet control because a simple beveled edge is of equal benefit. Design methods have been developed for two basic tapered inlets: the side-tapered inlet and the slope-tapered inlet with site conditions determining the use of each type. Tapered inlet design nomographs are available for rectangular box culverts and circular pipe culverts and can be found in HDS-5.

9.18.1. Side-Tapered Inlet

The side-tapered inlet has an enlarged face section with transition to the culvert barrel accomplished by tapering the side walls (Figure 9-13). The face section is about the same height as the barrel height and the inlet floor is an extension of the barrel floor. The inlet roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than ten percent. The intersection of the tapered sidewalls and the barrel is defined as the throat section.

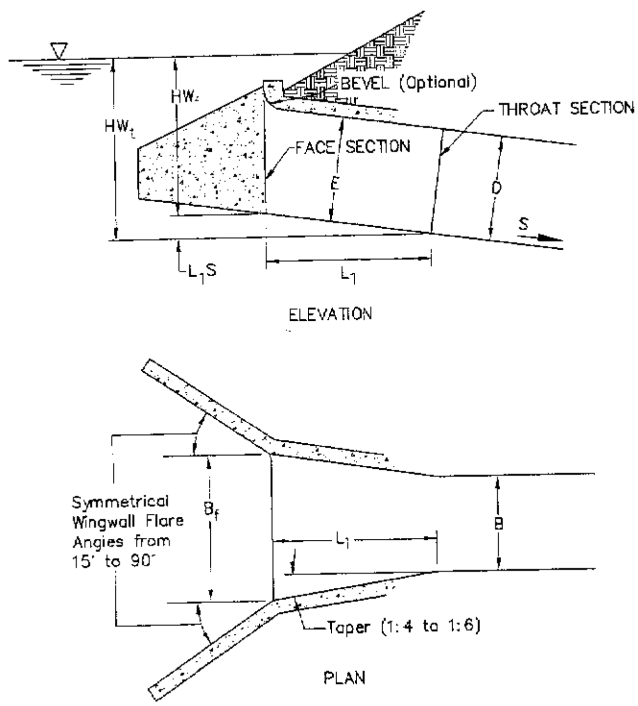


Figure 9-13: Side-Tapered Inlet

There are two possible control sections, the face and the throat. HW_f, shown in Figure 9-13, is the headwater depth measured from the face section invert and HW_t is the headwater depth measured form the throat section invert. The throat of a side-tapered inlet is a very efficient control section. The flow contraction is nearly eliminated at the throat. In addition, the throat is always slightly lower than the face so that more head is exerted on the throat for a given headwater elevation.

The beneficial effect of depressing the throat section below the streambed can be increased by installing a depression upstream of the side-tapered inlet. Figure 9-14 depicts a side-tapered inlet with the depression contained between wingwalls. For this type of depression, the floor of the barrel should extend upstream from the face a minimum distance of D/2 before sloping upward more steeply. The length of the resultant upstream crest, where the slope of the depression meets the streambed, should be checked to assure that the crest will not control the flow at the design flow and headwater. If the crest length is too short, the crest may act as a weir control section.

The side-tapered inlet throat should be designed to be the primary control section for the design range of flows and headwaters. Since the throat is only slightly lower than the face, it is likely that the face section will function as a weir or an orifice with downstream submergence within the design range. At lower flow rates and headwaters, the face will usually control the flow.

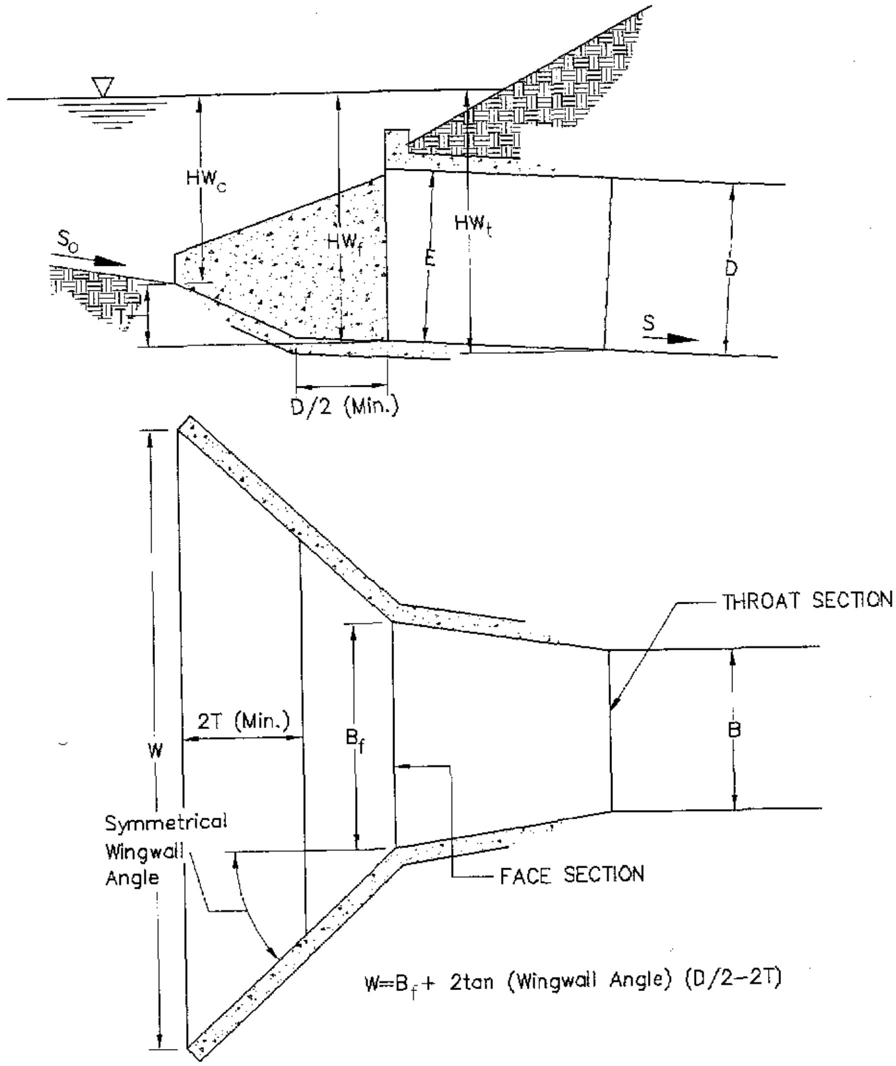


Figure 9-14: Side-Tapered Inlet with Upstream Depression Contained between Wingwalls

9.18.2. Slope-Tapered Inlet

The slope-tapered inlet, like the side-tapered inlet, has an enlarged face section with tapered sidewalls meeting the culvert barrel walls at the throat section (Figure 9-15). In addition, a vertical fall is incorporated into the inlet between the face and throat sections. This fall concentrates more head on the throat section. At the location where the steeper slope of the inlet intersects the flatter slope of the barrel, a third section, designated the bend section, is formed.

A slope-tapered inlet has three possible control sections, the face, the bend, and the throat. Of these, only the dimensions of the face and the throat section are determined by the design procedures of this manual. The size of the bend section is established by locating it a minimum distance upstream from the throat so that it will not control the flow.

The slope-tapered inlet combines an efficient throat section with additional head on the throat. The face section does not benefit from the fall between the face and the throat; therefore, the face sections of these inlets are larger that the face sections of equivalent depressed side-tapered inlets. The required face size can be reduced by the use of bevels or other favorable edge configurations. The vertical face slope-tapered inlet design is shown in Figure 9-15.

The slope-tapered inlet is the most complex inlet improvement recommended in this manual. Construction difficulties are inherent, but the benefits in increased performance can be great. With proper design, a slope-tapered inlet passes more flow at a given headwater elevation than any other configuration. Slope-tapered inlets can be applied to both box culverts and circular pipe culverts. For the latter application, a square to round transition is normally used to connect the rectangular slope-tapered inlet to the circular pipe.

The slope-tapered inlet throat can be the primary control section with the face section submerged or unsubmerged. If the face is submerged, the face acts as an orifice with downstream submergence. If the face is unsubmerged, the face acts as a weir, with the flow plunging into the pool formed between the face and the throat. As previously noted, the bend section will not act as the control section if the dimensional criteria of this publication are followed. However, the bend will contribute to the inlet losses that are included in the inlet loss coefficient, k_e. Values of k_e are given in Table 9-4.

When a culvert with a tapered inlet performs in outlet control, the tapered inlet entrance loss coefficient (k_e) is 0.2 for both side-tapered and slope-tapered inlets. This loss coefficient includes contraction and expansion losses at the face, increased friction losses between the face and the throat, and the minor expansion and contraction losses at the throat.

Tapered inlet design begins with the selection of the culvert barrel size, shape, and material. The calculations are performed using the Culvert Design Form, which can be found in HDS-5. The design nomographs contained in HDS-5 are used to design the tapered inlet. The design procedure is similar to designing a culvert with other control sections (face and throat). The result will be one or more culvert designs, with and without tapered inlets, all of which meet the site design criteria. The designer must select the best design for the site under consideration.

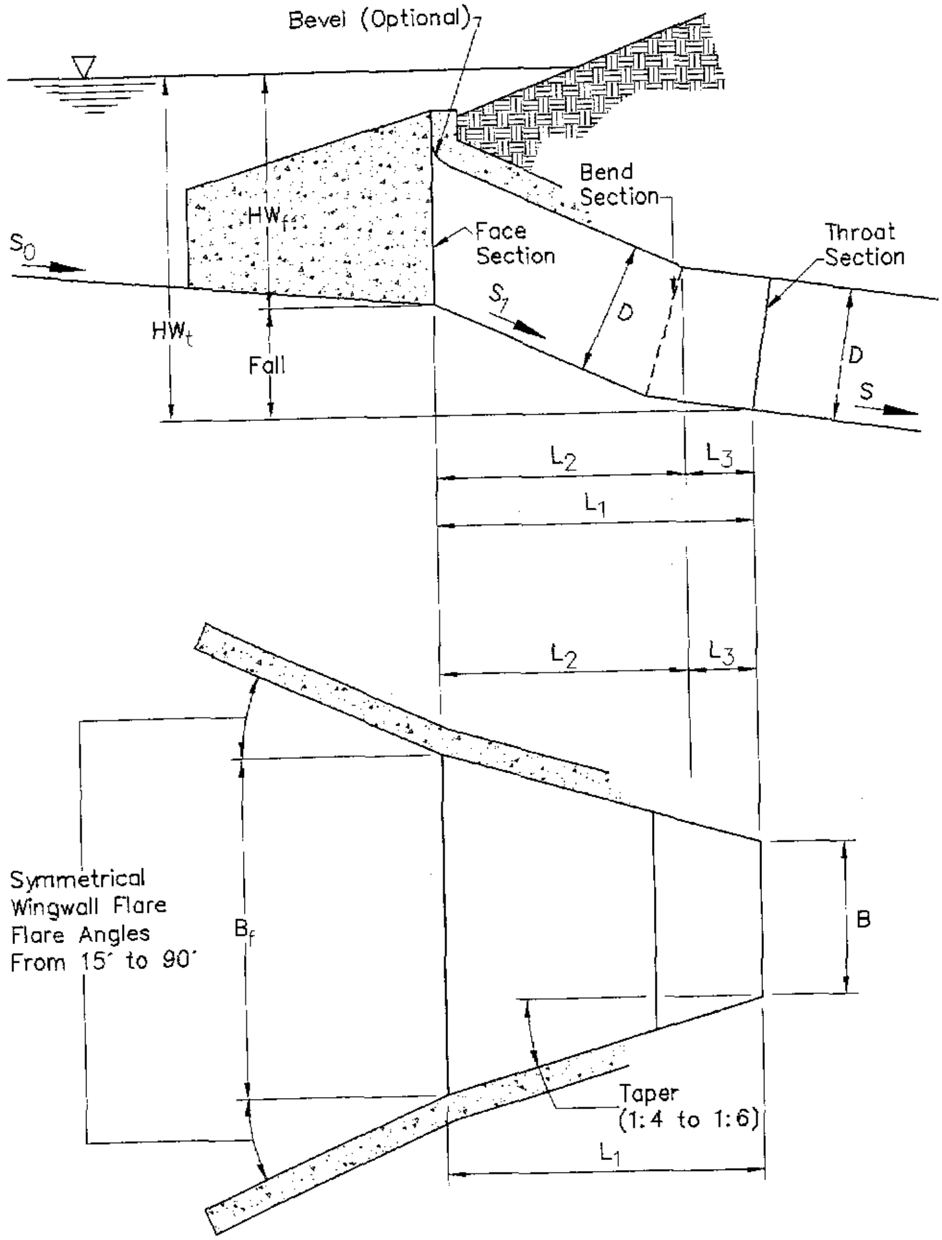


Figure 9-15: Slope-Tapered Inlet with Vertical Walls

9.18.2.1. Hydraulic Design (Inlet Control)

Tapered inlets have several possible control sections including the face, the bend (for slope-tapered inlets), and the throat. In addition, a depressed side-tapered inlet has a possible control section at the crest upstream of the depression. Each of the inlet control sections has an individual performance curve. The headwater depth for each control section is referenced to the invert of the section. One method of determining the overall inlet control performance curve is to calculate performance curves for each potential control section, and then select the segment of each curve that defines the minimum overall culvert performance.

Table 9-4: Outlet Control, Full or Partly Full Entrance Head Loss

$$H_e = K_e\left[\frac{V^2}{2g}\right]$$

Type of Structure and Design of Entrance	Coefficient K_e
• **Pipe, Concrete**	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end	0.2
Square-edge	0.5
Rounded (radius = D/12	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7⁰ or 45⁰ bevels	0.2
Side- or slope-tapered inlet	0.2
• **Pipe, or Pipe-Arch, Corrugated Metal**	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7⁰ or 45⁰ bevels	0.2
Side- or slope-tapered inlet	0.2
• **Box, Reinforced Concrete**	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30⁰ to 75⁰ to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10⁰ to 25⁰ to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

*Note: "End Sections conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

In the design of tapered inlets, the goal is to maintain control at the efficient throat section in the design range of headwater and discharge. This is because the throat section has the same geometry as the barrel, and the barrel is the most costly part of the culvert. The inlet face is then sized large enough to pass the design flow without acting as a control section in the design discharge range. Some slight oversizing of the face is beneficial because the cost of constructing the tapered inlet is usually minor compared with the cost of the barrel.

9.18.2.2. Performance Curves – Tapered Inlets

Performance curves are of utmost importance in understanding the operation of a culvert with a tapered inlet. Each potential control section (face, throat, and outlet) has a performance curve, based on the assumption that particular section controls the flow. Calculating and plotting the various performance curves results in a graph similar to Figure 9-16 containing the face control, throat control, and outlet control curves. The overall culvert performance curve is represented by the hatched line. In the range of lower discharges, face control governs; in the intermediate range, throat control governs; and in the higher discharge range, outlet control governs. The crest and bend performance curves are not calculated since they do not govern in the design range. To ensure that no overtopping occurs the designer should realize that the total head should be less than roadway crossing elevation.

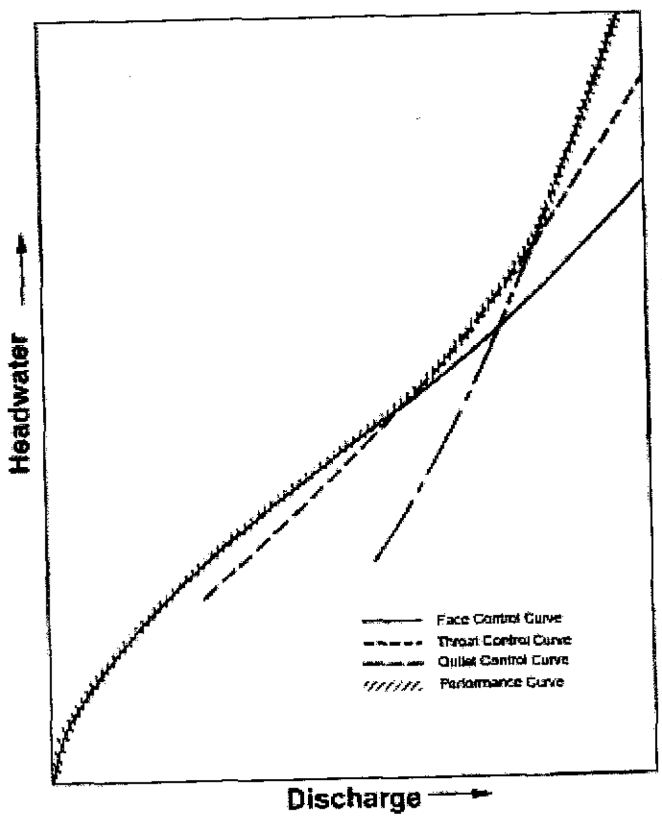


Figure 9-16: Culvert Performance Curve (Schematic)

10. Bridges

This section presents a brief overview of the hydraulic analyses for bridge crossings over open channels. A general discussion of scour is also presented. Comprehensive guidelines and criteria for hydraulic analyses of bridge crossings are beyond the scope of this manual. The reader should refer to appropriate texts and technical handbooks for further information on this subject.

Roadways must often cross open channels in urban areas; therefore, sizing the bridge openings is of paramount importance. In general, bridges should be designed to have as little effect as possible upon the flow passing beneath them. If possible, bridges over natural or man-made channels should be designed so that there is no disturbance to the flow whatsoever. Whenever piers are used, they need to be oriented parallel to flow. Impacts upon channels and floodplains created by bridges usually take the form of increased flow velocities through and downstream of the bridges, and increased scour and upstream ponding due to backwater effects. These impacts can cause flood damage to the channel, to adjacent property and to the bridge structure itself.

A new or replacement bridge <u>should not</u> be permitted to create a rise in the existing water surface elevation, to cause an increase in lateral extent of the floodplain, or to otherwise worsen existing conditions for discharges up to and including the 100-year discharge.

10.1. Hydraulic Analysis

The hydraulic analyses of pre- and post-bridge conditions can be performed using a computerized step-backwater model. The HEC-RAS program developed by the USACE (2001) is the most common backwater computation software available and is used nationwide. HEC-RAS is the preferred computer software for one-dimensional hydraulic analyses for studies of this type in the City of El Paso. USACE's older HEC-2 program may also be used for analyzing bridges, but is not preferred.

Bridge analysis requires meticulous input preparation for proper analysis, and care should be taken to review input data and to examine results thoroughly for reasonableness. Analyses of this type should only be undertaken by an engineer with a solid understanding of hydraulic fundamentals.

If there is a good possibility of debris collecting on the piers, it may be advisable to use a value greater than the physical pier width to account for debris blockage. Some agencies require the pier width to be modeled as twice its width while others require 1 foot added to each side of the pier. Thus, modeling requirements of debris blockage should be reviewed with the Storm Water Utility, the City, or other appropriate jurisdictional agency.

10.2. Hydraulic Design Considerations

Additional factors to be considered in the design of a bridge crossing include freeboard, flow regime (i.e., subcritical or supercritical flow), and anticipated scour effects.

10.2.1. Freeboard

Freeboard at a bridge is the vertical distance between the design water surface elevation and the low-chord of the bridge. The bridge low-chord is the lowest portion of the bridge deck superstructure. The purpose of freeboard is to provide room for the passage of floating debris, extra area for conveyance in the event that debris build-up on the piers reduces hydraulic capacity of the bridge, and a factor of safety against the occurrence of waves or floods larger than the design flood. A minimum freeboard of 2 feet for the 100-year event is recommended. The structural design of the bridge should take into account the possibility of debris and/or flows impacting the bridge. In certain cases, site conditions or other circumstances may limit the amount of freeboard at a particular bridge crossing. An example would be the replacement of a "perched" bridge across a natural watercourse where major flows overtop the roadway approaches. In general, variances to the minimum freeboard requirement will be evaluated on a case by case basis by the Storm Water Utility or City Engineer.

10.2.2. Supercritical Flow

For the special condition of supercritical flow within a lined channel, the bridge structure should not affect the flow at all. That is, there should be no projections, piers, etc. in the channel area. The bridge opening should be clear and permit the flow to pass unimpeded and unchanged in cross section.

10.2.3. Scour

The issue of scour analysis at a bridge is beyond the scope of this chapter. The following discussion touches upon the subject matter to provide the interested designer an indication of the issues. Local pier and abutment scour, contraction scour, and long-term scour must be investigated when designing a bridge.

General scour from a contraction usually occurs when the normal flow area of a stream is decreased by a bridge. The contraction of the flow by the bridge can be caused by a decrease in flow area of the stream channel by the abutments projecting into the channel and/or the piers taking up a large portion of the flow area. Also, the contraction can be caused by approaches to the bridge that cut off the overland flow that normally go across the floodplain during high flow. This latter case also can cause clear-water scour at the bridge section because overland flow normally does not transport any significant bed material sediments. This clear-water picks up additional sediment from the bed when it returns to the bridge crossing. In addition, if floodwater returns to the stream channel at an abutment, it increases the local scour there. A guide bank at an abutment decreases the risk from scour of that abutment from returning overbank flow. Also, relief bridges in the approaches reduce general scour by decreasing the amount of flow returning to the natural channel, which then decreases the scour problem.

The estimation of scour is an engineering application that requires both specific expertise and experience. Every application of scour technology is unique because of the wide variability of hydrologic, hydraulic and geologic/geomorphic factors. It is not possible to compile a comprehensive methodology in a drainage design manual that would be adequate to address all aspects of scour estimation. In addition, the knowledge of erosion and sedimentation is continually expanding because of the need to provide better technology in this field of engineering. Often, newer methodologies are presented in the engineering literature that should be considered and used, if appropriate. Therefore, the following are general guidelines for estimating scour along with currently used references that are considered applicable in the City of El Paso.

Scour calculations for bridges should consider the following components of scour:

- Long-term degradation of the bed of the watercourse.
- General scour through a specific reach of the watercourse.
- Local scour.

- Scour induced due to a bend in the watercourse.
- Scour associated with bedform movement through the watercourse.
- Scour due to low-flow incisement.

10.2.3.1. Determination of Scour

The procedure in *Evaluating Scour at Bridges, HEC-18* (FHWA, 2001) should be consulted when estimating scour at bridges. Usually the largest component of scour is from local scour at the pier or abutment. Certain scour equations include the angle of attack of the flow, and therefore, bend scour is not normally added because it can be accounted for in the local scour.

General Scour

General scour is that component of total scour that would occur during the passage of a design flood. This type of scour involves the removal of material from the bed and banks across all or most of the width of a channel. The scour is caused by increased velocities and shear stresses dictated by the local area geometry (such as at constrictions) and water surface controls. For major watercourses, general scour would often be estimated by a sediment transport model study, such as the use of *Scour and Deposition in Rivers and Reservoirs, Users Manual, HEC-6* (USACE, 1991).

Contraction Scour

Contraction scour occurs when the flow area of the watercourse is reduced because of natural conditions or because of the bridge approaches encroaching into the watercourse. Two equations are provided in HEC-18 (USDOT, 2001b) for contraction scour. One is for live bed conditions; that is, when there is bed material transport from upstream of the bridge. For that condition, a modified version of Laursen's live-bed contraction scour equation (Laursen, 1960) is used. The second is for clear water conditions; that is, when there is little or no sediment transport from upstream of the bridge. For that condition, Laursen's clear-water contraction scour equation (Laursen, 1963) is used. The HEC-18 publication (USDOT, 2001b) should be consulted when estimating contraction scour.

Pier Scour

The commonly used pier scour equations are the Colorado State University equation (Richardson et al., 2001) and Froehlich (1988). Both of those equations are considered in the HEC-RAS program for bridge pier scour (USACE 2001 and 2001b); however, only the Colorado State University equation is recommended in HEC-18 (USDOT, 2001b). The Froehlich equation has been shown to compare well with observed data. Those references should be consulted when estimating pier scour.

Abutment Scour

The commonly used abutment scour equations are the HIRE equation (Richardson et al., 2001) and Froehlich (1989). Those equations are provided both in HEC-18 (USDOT, 2001b) and the HEC-RAS program (USACE, 2001a and 2001b). Those references should be consulted when estimating abutment scour.

Watercourse Stability at Highways

The stability of the watercourse at and near highway structures should be considered if channel instability is suspected. Procedures to investigate watercourse stability are provided in HEC-20 (USDOT 2001c).

Bridge Scour Countermeasures

Procedures to provide bridge scour countermeasures should be provided in accordance with the FHWA's, *Bridge Scour and Stream Instability Countermeasures, Hydraulic Engineering Circular HEC-23, 2001.*

10.2.4. Additional Design Considerations

Drainage structures over the concrete-lined channel should be of the all-weather type, e.g., bridges, culvert pipe, or concrete box culverts. Crossing structures should conform to the channel shape in order that they disturb the flow as little as possible.

It is preferred that the channel section be continuous through crossing structures. However, when this is not practicable, hydraulic disturbance shall be minimized, and crossing structures should be suitably isolated from the channel lining with appropriate joints.

10.2.5. Minimum Clear Height

Channel lining transitions at bridges and box culverts should conform to the provisions for transitions hereinafter provided. Drainage structures having a minimum clear height of 8 feet and being of sufficient width to pass maintenance vehicles may result in minimizing the number of required channel access ramps. Unless otherwise specifically authorized by the City Engineer, all crossing structures must have at least 6 feet of clear height.

11. Storm Water Ponding

11.1. General

In developing areas where storm water peak runoff rates increase due to the creation of more impervious land area than historic conditions, storm water storage is a necessary component to flood hazard mitigation. Storm water ponding reduces the developed condition peak runoff rate and can be sized to help runoff rates match the historic rates. In a situation where a dam is utilized to create a storm water storage facility, coordination with the Texas Commission on Environmental Quality (TCEQ) and the Texas State Engineer Office must occur where dam heights exceed 6 feet.

The purpose of this section is to provide uniform minimum criteria for the design of safe, effective, and easily maintained ponds. Ponds may be required in newly developing areas where the capacity of the downstream conveyance system is inadequate to serve developed runoff from the developable area draining to it. Small ponds are discouraged in favor of larger regional ponds. The City may require new developments to provide adequate off-site downstream conveyance systems to a regional pond, instead of on-site ponds, to avoid excessive maintenance costs of numerous small ponds.

11.1.1. Definitions

The terms detention and retention ponding are often used interchangeably. However, each represents a very different type of storm water ponding. The following definitions from American Society of Civil Engineers (ASCE, 1992) for each type of facility will be used in this manual.

- Detention – The temporary storage of flood water that is usually released by a measured uncontrolled outlet. Detention facilities typically flatten and spread the inflow hydrograph, lowering the peak.
- Retention – Storage provided in a facility without a positive outlet, or with a specially regulated outlet, where all or a portion of the inflow is stored for a prolonged period. Infiltration ponds are a common type of retention facility.

11.1.2. Objectives

The objective for managing storm water quantity by storage facilities are typically based on limiting peak runoff rates to match one or more of the following values:

- Historic rates for a specific design condition.
- Non-hazardous discharge capacity for the downstream drainage system.
- A specified value for allowable discharge set by the local drainage authority.

For a watershed with no positive outfall or where delayed outlet hydrograph peaks may cause downstream problems, the total volume of runoff is critical and retention storage facilities are used to store the increases in volume and control discharge rates.

11.2. Stage-Storage-Discharge Curve

A Stage-Storage-Discharge curve represents the relationship of the stage (depth) of water and the associated storage (volume or pond capacity at that stage) and discharge (outflow of water through the outlet structures with the stage as the headwater elevation). The stage-discharge curve should take into account the discharge characteristics of both the principal and emergency spillways. Tailwater influences and structure losses must be considered when developing discharge curves. If a combination of outlet structures is used, backwater effects of one structure may affect the discharge of the combination of structures. Refer to Chapter 12 – Other Hydraulic Elements for Weir and Orifice spillway equations.

For the retention analysis, the volume under the inflow hydrograph is the total volume of runoff entering the retention pond, generally calculated from HEC-1 or other similar computer models. For detention analysis, the volume provided by the pond in conjunction with an outlet structure must be sufficient enough to lower the post-development hydrograph (100-year outflow hydrograph) to the pre-development hydrograph (100-year inflow hydrograph). This procedure is referred to as routing. It is recommended that the retention and detention pond modeling for volume calculations and routing procedures be performed using computer modeling program, as manual calculation is time consuming and repetitive. Examples of publicly available computer modeling programs are:

- TR-20.
- HEC-1.
- HEC-HMS.

The basic concept involved in storm water detention analysis is referred to as routing or the Storage-Indication method, which transforms the inflow hydrograph through the basin storage for a resulting outflow hydrograph. To assist with these calculations, there are many available reservoir routing computer programs as previously mentioned.

To perform the calculations in these programs one must have the following data:

- Inflow hydrograph for all selected design storms. Generally derived by use of HEC-1 or HEC-HMS.
- Stage-storage curve for proposed storage facility.
- Stage-discharge curve for all outlet control structures.

11.2.1. General Routing Procedure (PULS Method)

The general routing procedure is presented in detail for those who wish to do routing calculations manually:

1. Develop an inflow hydrograph, stage-discharge curve and stage-storage curve for the proposed storage facility.
2. Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < Tc/5$).
3. Use the storage-discharge and stage-storage data from Step 1 to develop storage characteristics curves that provide values of $S+(O/2)\Delta t$ versus stage.
4. I1 and I2 are known. Given the depth of storage or stage, H1, at the beginning of that time interval, $S1-(O1/2)\Delta t$ can be determined from the appropriate storage characteristics curve.

5. Determine the value of $S2 + (O2/2)\,\Delta t$ from the following equation:

$$S2 + (O2/2)\,\Delta t = [S1 - (O1/2)\,\Delta t] + [(I1 + I2)/2]\Delta t \quad \text{11-1}$$

Where:

$S2$ = Storage volume at time 2, in cubic feet.

$O2$ = Outflow rate at time 2, in cubic feet per second.

Δt = Routing time period, in seconds.

$S1$ = Storage volume at time 1, in cubic feet.

$O1$ = Outflow rate at time 1, in cubic feet per second.

$I1$ = Inflow rate at time 1, in cubic feet per second.

$I2$ = Inflow rate at time 2, in cubic feet per second.

Other consistent units are equally appropriate.

6. Enter the storage characteristics curve at the calculated value of $S2+(O2/2)\Delta t$ determined in Step 5 and read off a new depth of water, H2.
7. Determine the value of O2, which corresponds to a stage of H2 determined in Step 6, using the stage-discharge curve.
8. Repeat Steps 1 through 7 by setting new values of I1, O1, S1 and H1 equal to the previous I2, O2, S2 and H2, and using a new I2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

11.3. Interaction with Other Components of a Drainage System

Storm water ponding facilities are components of an overall storm water management system that also comprises natural and man-made channels, storm sewers, inlets, streets, and other drainage structures. Their purpose is to provide temporary storage of the storm water runoff from developed areas and control the increased peak rates of runoff. Proper planning and design of storm water ponding facilities must consider the interaction of storage with the other components of the drainage system. The greater the number of ponding facilities in a system, the more complex is the analysis of the interaction of the various discharges.

As part of the design process, the engineer must verify that the releases from the storm water ponding facility will not adversely impact downstream conditions in terms of both manner and quantity of flow. Conditions such as peak flow, velocity, flow concentration, prolongation of flow, and quality of discharge are factors to be considered.

11.4. Guidelines for Storm Water Ponding

The following general guidelines apply to the design of storm water ponding facilities.

11.4.1. Storage Capacity

11.4.1.1. Retention Facilities

A retention pond shall be designed to have storage capacity to retain the following components:

- 100 percent of the 100-year design storm volume using procedures outlined in Chapter 4 of this manual.

11.4.1.2. Detention Facilities

A detention pond shall be designed to have storage capacity to retain the following components:

- Basins to be designed utilizing good engineering practices and accepted methods (HEC-1) whereby 100% of the runoff volume to be properly managed though the use of channels and basins.

11.4.2. Pond Geometric Controls

With respect to siting, storm water ponding facilities that utilize a method of subsurface disposal shall be located such that the infiltration surface will be a specific distance, both horizontal and vertical, from any functioning water well. The El Paso Storm Water Utility should be contacted regarding regulations governing the siting of such facilities near wells and near the static groundwater table.

Basic requirements regarding facility shape, side slopes, depth, and bottom configuration are provided below and in subsequent sections.

11.4.2.1. Shape

As a general rule, curvilinear, irregularly shaped facilities will have the most natural character. A wide range of shapes can be considered and utilized to integrate the storm water ponding facility with the surrounding site development. Smooth curves should be used in the plan layout of the grading for the facility.

11.4.2.2. Side Slopes

Side slopes shall not exceed 3 horizontal to 1 vertical (3H:1V). Slopes steeper than 3H:1V may be allowed on the recommendation of a Licensed Professional Geotechnical Engineer, as determined by a soils investigation report. The soils investigation report shall contain the following minimum requirements:

- Soils analysis describing engineering properties and suitability of the existing soils to support the proposed slope.
- Erosion control measures.
- Slope stability analysis showing the failure plane and a minimum factor of safety of 1.25 for non-supporting structural slope and 1.5 for slope supporting structure.

Ponds with side slopes greater than 20 percent (5 horizontal to 1 vertical) shall be enclosed with a minimum 6 foot high screening wall. The height of the wall shall be measured from the ground inside or outside the wall, whichever is higher. The screening wall may consist of rock-wall, Concrete Masonry Unit (CMU) wall, combination of rock-wall and wrought iron, wrought iron, or other materials as approved by the City Engineer. For commercial or industrial areas, a minimum of 6 inches wide by 24 inches high stem-wall shall be required with a minimum 4-foot high chain-link fence, for a total combined height of 6 feet from highest side. The 6 feet screening wall may be waived if landscaping or other design applications are approved by the Storm Water Utility or City Engineer.

If a screening wall is required, one 18-foot minimum wide double gate accessible from public right-of-way with gate location is to be approved by the Storm Water Utility or City Engineer. Gates shall be set back a minimum of 50 feet from Arterial and Collector streets so equipment does not have to park in the street. The gate shall be wrought iron. The gate shall swing inward and not obstruct the maintenance road or other materials or opening, variances to composition and dimensions may be used upon approval of the Storm Water Utility or City Engineer.

11.4.2.3. Depth and Bottom Configuration

Deep facilities should be avoided, if possible. Maximum ponding depths shall be limited to 20 feet unless greater depths have approval of the Storm Water Utility or City Engineer. The allowable clearance at the bottom of the pond shall be 25 feet diameter minimum. For a detention pond, the bottom shall be designed to drain to a low flow channel.

11.4.3. Drain Time

The minimum design storm for public ponds will be the 100-year storm and may be increased for dam safety purposes where required by separate regulations. The pond must empty within 72 hours, either by infiltration or by means of an outlet. If the entire 24-hour storm volume can not be infiltrated in 72 hours, the duration of the design storm shall be increased to exceed the duration of the total infiltration of the design volume. The hydrology procedures in Chapter 4 shall be used to calculate run-off flows and capacities (volume).

11.4.4. Lining/Surface Treatment

In keeping with the Open Space Master Plan for the City of El Paso and the overall goal to utilize storm water facilities as amenities that incorporate multiple use concepts where possible, grass and/or landscape plantings are preferred surface treatments. As a general rule, grass and plant species used for landscaping should be native to the City of El Paso. Permanent irrigation systems are required for grass areas and most landscaping. A registered landscape architect should prepare the landscape design with consideration toward use of plant species appropriate for the level and frequency of inundation of the ponding facility.

The use of inert materials is appropriate for stabilization and erosion control where steep slopes are unavoidable, including along channels, at inflow points, at the outlet control structure, and any other location where flowing water may threaten stability. Use of these materials should be properly engineered and should respond to aesthetic considerations. Inert materials for erosion control include:

- Loose rock riprap with a specific, engineered gradation.
- Loose or grouted boulders (minimum dimension 18 inches or larger).
- River stone.
- Gabions.
- Soil cement and concrete.

Designs that combine both landscape planting with the use of inert materials is recommended.

11.4.5. Low Flow Channels

A low flow channel is required in the bottom of a detention pond to provide positive routing of drainage to the primary outlet structure. Low flow channels shall have a 0.5% maximum longitudinal slope.

11.4.6. Subsurface Disposal

The primary method for underground disposal of storm water runoff at retention ponds are engineered floors and drywells. Infiltration rates of pond floors or drywells shall not be used in determining outflow rates in flood-routing procedures.

11.4.6.1. (Intentionally left blank for Future Use)

11.4.6.2. Drywell

Drywells may be used for subsurface disposal of storm water if criteria such as soil permeability, groundwater levels, and maintenance can be satisfactorily addressed. Soil profiling shall be provided to a minimum of five (5) feet below the pond invert. The bottom of the pond shall be a minimum of 24 inches above the high groundwater table. Percolation test shall be performed at time of pond excavation according to ASTM D 5126 and the report submitted for review and approval by the Storm Water Utility or City Engineer. Storm water shall percolate within 72 hours under wet conditions.

The main cause of drywell failure is clogging of the transmission media by silt and debris. Failure can be avoided by utilizing proper design and installation and by performing routine maintenance.

11.5. Spillways

A typical detention pond facility has two spillways: principal and emergency. A typical retention pond facility will only have a emergency spillway to help direct flood events in a planned manner.

11.5.1. Primary

The outlet structure allows flows to discharge from the detention pond at a controlled rate. Examples of outlet structures are spillways and culverts. The principal outlet structure, called the primary spillway, is intended to convey the design storm without requiring flow to enter an emergency spillway. Outlets can be designed in a wide variety of configurations depending on the desired release rates. This may be accomplished using multi-stage control structures. If a multi-stage control structure satisfies the upper and lower discharge requirements, discharges from intermediate storm return periods can usually be assumed to be adequately controlled.

11.5.2. Emergency

The emergency spillway should be capable of passing flows up to 120% of the 100-year storm and/or meet TCEQ requirements. An emergency spillway shall be provided with a capacity equal to the peak discharge of the design storm with a maximum high water elevation of 1 foot above spillway invert. The spillway shall have a minimum freeboard of 6 inches.

11.6. Maintenance Considerations

Proper design should focus on the reduction of maintenance requirements by addressing the potential for problems to develop.

- Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers.
- Sedimentation may be controlled by constructing traps to contain sediment for easy removal or by low-flow channels to reduce erosion and sediment transport by frequent or continuous flows.
- Outlet structures should be selected to minimize the possibility of blockage.

An access ramp meeting the following criteria will be provided for all storm water pond facilities:

- Maximum slope of 15%.
- Minimum width of 15 feet.
- Ramp material will consist of a maximum P.I. of 8 with no loose material and a compacted drivable surface.
- Compaction will be at a minimum 95% per ASTM D-1557.

A depth gauge shall be required for ponds 5 feet or deeper.

11.7. (Intentionally left blank for Future Use)

12. Other Hydraulic Elements

12.1. Hydraulic Jump

A hydraulic jump is a phenomenon where the flow regime changes from supercritical to subcritical flow, and it is often abrupt (particularly if the Froude number (Fr) is larger than 2.0). The flow passes through critical depth in a hydraulic jump.

A hydraulic jump occurs when flow changes rapidly from low stage supercritical flow to high stage subcritical flow. Hydraulic jumps can occur: 1) when the slope of a channel abruptly changes from steep to mild; 2) at sudden expansions or contractions in the channel section; 3) at locations where a barrier, such as a culvert or bridge, occurs in a channel of steep slope; 4) at the downstream side of dip crossings or culverts; and 5) where a channel of steep slope discharges into other channels.

As described in *Hydraulic Design Series Number 6 - River Engineering for Highway Encroachments (HDS-6)* (FHWA, 2001), when the flow velocity V_1 is rapid or supercritical, the surge dissipates energy through a moving hydraulic jump. When V_1 equals the celerity, c, of the surge the jump is stationary, see Figure 12-1. Equation 12-1 is the equation for a hydraulic jump on a flat slope.

$$c = \left\{ g y_1 \left[\frac{y_2}{2y_1} \left(\frac{y_2}{y_1} + 1 \right) \right] \right\}^{1/2} \qquad 12\text{-}1$$

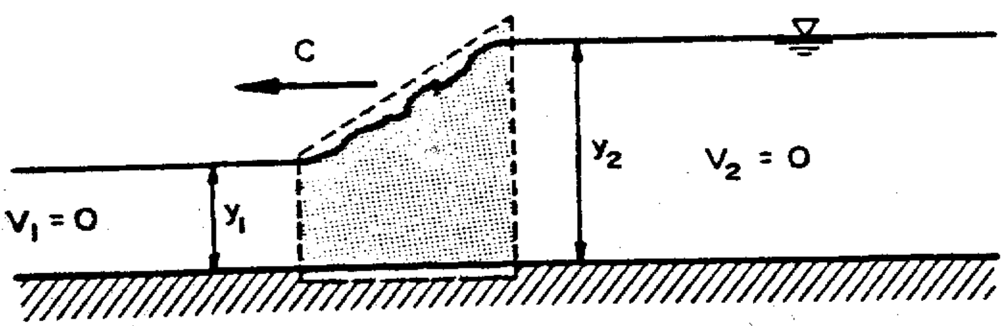


Figure 12-1: Sketch of Surge in a Channel

Solutions for a hydraulic jump on a sloping channel are given in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (HEC-14, 2006). Equation 12-1 can be rearranged to the form:

$$\frac{V_1}{\sqrt{g y_1}} = Fr_1 = \left[\frac{y_2}{2y_1} \left(\frac{y_2}{y_1} + 1 \right) \right]^{1/2} \qquad 12\text{-}2$$

Or

$$\frac{y_2}{y_1} = \frac{1}{2} \left[\left(1 + 8 Fr_1^2 \right)^{1/2} - 1 \right] \qquad 12\text{-}3$$

The corresponding energy loss in a hydraulic jump is the difference between the two specific energies. It can be shown that this head loss is:

$$h_L = \frac{(y_2 - y_1)^3}{4y_2y_1} \qquad 12\text{-}4$$

As discussed in HDS-6, Equation 12-4 has been experimentally verified along with the dependence of the jump length, L_j, and energy dissipation (head loss, h_L) on the Froude number of the approaching flow. The results of these experiments are given in Figure 12-2. When the Froude number for rapid flow is less than 1.7, an undulating jump with large surface waves is produced. The waves are propagated for a considerable distance downstream. In addition, then the Froude number of the approaching flow is less than three, the energy dissipation of the jump is not large and jets of high velocity flow can exist for some distance downstream of the jump. These waves and jets can cause erosion a considerable distance downstream of the jump. For larger values of the Froude number, the rate of energy dissipation in the jump is very large and Figure 12-2 is recommended. The U.S. Bureau of Reclamation (Chow 1959) classifies the hydraulic jump on a flat slope into various types as illustrated in Figure 12-3.

The designer is referred to *Hydraulic Design Series Number 4 Introduction to Highway Hydraulics (HDS-4)*, HDS-6, HEC-14, *Open Channel Hydraulics* by Chow (1959) and *Engineering Hydraulics* by Rouse (1950) for further discussion of hydraulic jumps.

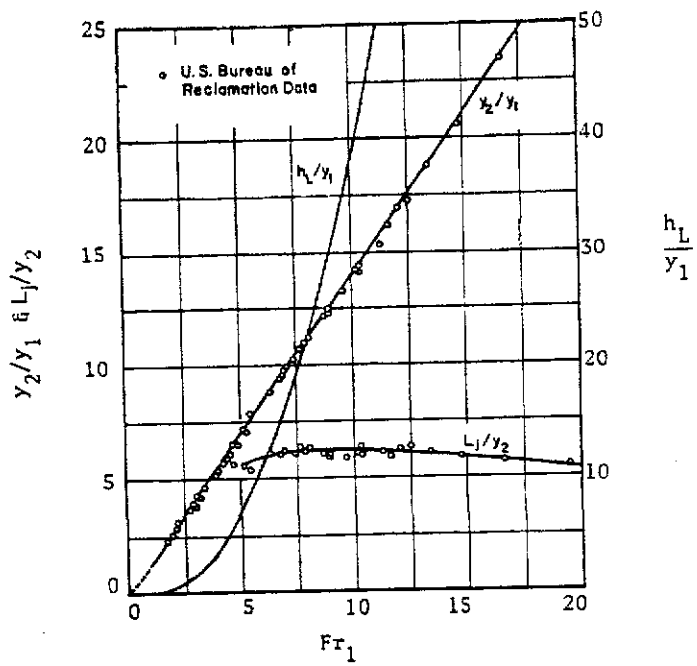


Figure 12-2: Hydraulic Jump Characteristics as a Function of the Upstream Froude Number

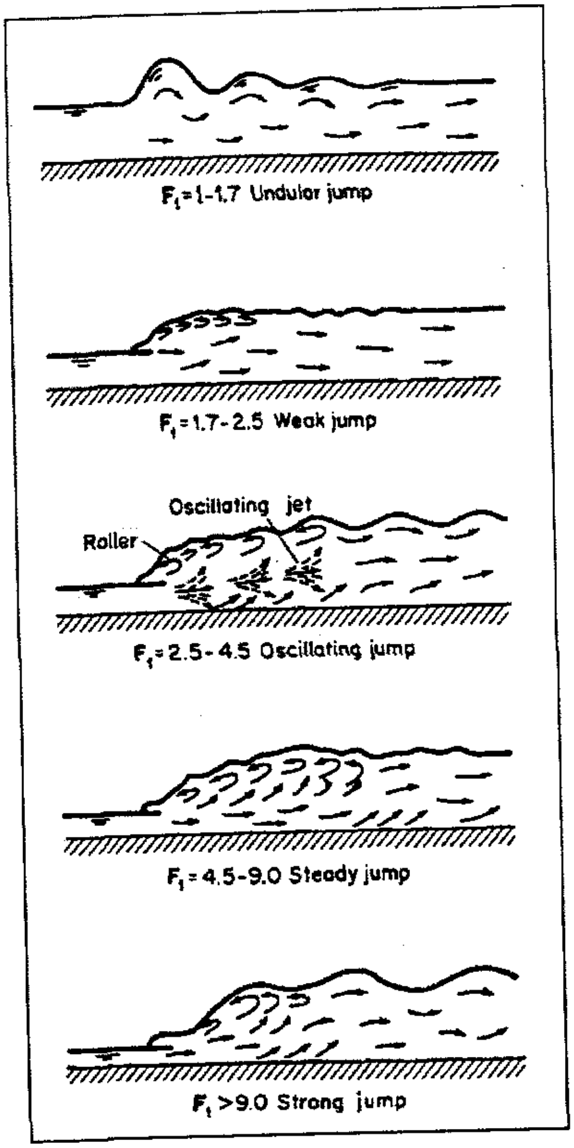


Figure 12-3: Various Types of Hydraulic Jumps

12.1.1. Application of Hydraulic Jumps

Hydraulic jumps can occur in pavement drainage, storm drains, channels, transitions, culverts, bridges, drop structures, flow in bends, chutes, pools, and other hydraulic structures. The designer should be aware of instances where hydraulic jumps are unwanted and may result in erosion.

For applications of hydraulic jumps, including energy dissipation to control erosion at hydraulic structures, the designer is referred to HEC-14.

12.2. Weirs

A weir is any control or barrier placed in an open channel to permit measurement of water discharge. Weirs are generally used as measuring and hydraulic control devices. The most common applications of weirs are emergency spillways and at-grade roadway crossings of channels. Application of weir equations can be applied to certain storm drain inlets, channel drop structures, outlet structures for detention facilities and other hydraulic structures. Special care must be exercised when selecting weir coefficients in the following cases:

- Submerged weirs.
- Broad-crested weirs.
- Weirs with obstructions (i.e., guardrails, piers, etc.).

12.2.1. Broad Crested

The equation generally used for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \qquad (12\text{-}5)$$

Where:

Q = Discharge, in cubic feet per second.

C = Broad-crested weir coefficient refer to Brater and King (1976) for common coefficients (2.61 to 3.08).

L = Broad-crested weir length, in feet.

H = Head above weir crest, in feet.

As discussed in the HEC-22 circular, if the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest. This gives the maximum C value of 3.08. For sharp corners on the broad crested weir, a minimum value of 2.61 should be used.

12.2.2. Sharp Crested

A weir with a sharp upstream corner or edge such that the water springs clear of the crest is called a sharp-crested weir. These weirs are to be analyzed using the following equation:

$$Q = CLH^{3/2} \qquad (12\text{-}6)$$

Where:

Q = Discharge, in cubic feet per second.

C = Discharge coefficient from Handbook of Hydraulics, 5th Edition *(King and Brater, 1976).*

L = Effective length of crest, in feet.

H = Depth of flow above elevation of crest, in feet (approach velocity shall be disregarded in most applications).

A sharp-crested weir with no end contractions is illustrated in Figure 12-4 below. The discharge equation for this configuration is (Chow, 1959):

$$Q = \left[\left(3.27 + 0.4\left(\frac{H}{H_c}\right)\right)\right] LH^{1.5} \qquad (12\text{-}7)$$

Where:

Q = *Discharge, in cubic feet per second.*

H = *Head above weir crest excluding velocity head, in feet.*

H_c = *Height of weir crest above channel bottom, in feet.*

L = *Horizontal weir length, in feet.*

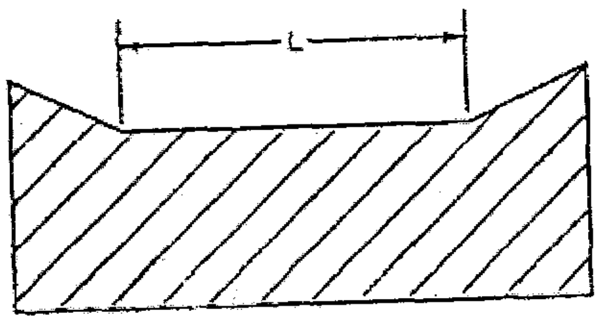


Figure 12-4: Sharp-Crested Weir- No End Contractions

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_s = Q_r\left(1-\left(\frac{H_2}{H_1}\right)^{1.5}\right)^{0.385} \qquad 12\text{-}8$$

Where:

Q_S = *Submergence flow, in cubic feet per second.*

Q_f = *Free flow, in cubic feet per second.*

H_1 = *Upstream head above crest, in feet.*

H_2 = *Downstream head above crest, in feet.*

12.2.3. V-Notch Weir

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5\tan\left(\frac{\theta}{2}\right)H^{2.5} \qquad 12\text{-}9$$

Where:

Q = *Discharge, in cubic feet per second.*

θ = *Angle of v-notch, in degrees.*

H = *Head on apex of notch, in feet.*

12.3. Orifices

An orifice is a submerged opening with a closed perimeter through which water flows. Orifices are analyzed using the following equation:

$$Q = CA\sqrt{2gh} \qquad \text{12-10}$$

Where:

Q = Discharge, in cubic feet per second.

C = Coefficient of discharge from Handbook of Hydraulics, 5th Edition *(King and Brater, 1976).*

A = Area of opening, in square feet.

g = Gravitational acceleration, 32.2 feet per second squared.

h = Depth of water measured from the center of the opening, in feet (approach velocity shall be disregarded in most applications).

Orifices are generally used as measuring and hydraulic control devices. Orifice hydraulics controls the function of many "submerged inlet - free outlet" culverts, primary spillways, manholes, and drop inlets.

For spillway design, the following orifice equation should be used for pipes smaller than 12 inches if H/D is greater than 1.5. For square edged entrance conditions:

$$Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5} \qquad \text{12-11}$$

Where:

Q = Discharge, in cubic feet per second.

A = Cross-section area of pipe, in square feet.

g = Acceleration due to gravity, 32.2 feet per second squared.

D = Diameter of pipe, in feet.

H = Head on pipe, from the center of pipe to the water surface, in feet.

12.4. Channel Drop Structure

As discussed in HDS-4, drop structures are commonly used for flow control and energy dissipation. Drop structures may be used to reduce the effective slope of a natural or artificial channel. Changing the channel slope from steep to mild, by placing drop structures at intervals along the channel reach, changes a continuous steep slope into a series of gentle slopes and vertical drops. Instead of slowing down and transferring high erosion producing velocities into low non-erosive velocities, drop structures control the slope of the channel in such a way that the high, erosive velocities never develop. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by a specially designed apron or stilling basin.

The Drainage Design Manual for Maricopa County, Arizona, indicates that a drop structure typically extends across the entire width of the channel and provides grade control for a full range of flows. Check structures are similar in concept, but their objective is to stabilize and control the channel bed or low flow zone. During a major flood, portions of the flow circumvent the structure, but erosion is maintained at an acceptable level. Overall stability is maintained by control of the low flow area, which would otherwise degrade downward. A series of check structures can be an economical interim grade control measure for natural channels in urbanizing areas or for artificial channels where funding is inadequate for construction of drop structures.

12.4.1. Application of Drop Structures

As discussed in HDS-6, drop structures or check dams may be constructed of rock, gabions, concrete, timber, sacked concrete, filled fences, sheet piling, or combinations of any of the above. They are most suited to locations where bed materials are relatively impervious; otherwise, underflow must be prevented by cutoffs. The designer is referred to HDS-6 for further discussion on the use of drop structures.

Additional resources for the designer for vertical drop structures or sloping structures are available in Rouse (1950), Chow (1959), Peterson (1986), and Simons and Senturk (1992). HEC-23 (Lagasse et al., 2001) provides design guidelines for a vertical drop structure and stilling basins for drop structures as well as river training devices such as check dams.

12.5. Energy Dissipation Structures

Concrete energy dissipation or stilling basin structures are required to prevent scour damages caused by high exit velocities and flow expansion turbulence at conduit outlets. Outlet structures can provide a high degree of energy dissipation and are generally effective even with relatively low tailwater control. Rock protection at conduit outlets is appropriate where moderate outlet conditions exist; however, there are many situations where rock basins are impractical even at low to moderate flow conditions. Concrete outlet structures can be designed easily and are suitable for a wide variety of site conditions. In some cases, they are more economical than large rock basins, particularly where long term costs are considered.

It is recommended that designers refer to HEC-14 and HDS-4 for further discussion of energy dissipation structures.

12.6. Debris Barriers

A debris barrier or deflector is a means of preventing large debris or trash, such as tree limbs, logs, boulders, weeds, and refuse, from entering a storm drain and possibly plugging the conduit. The debris barrier should have openings wide enough to allow as much small debris as possible to pass through, yet narrow enough to protect the smallest conduit in the system downstream of the barrier. One type that has been used effectively in the past is the debris rack. This type of debris barrier is usually formed by a line of posts, such as steel pipe filled with concrete or steel rails, across the line of flow to the inlet. Other examples of barriers are presented in *Hydraulic Engineering Circular No. 9*, "Debris-Control Structures," published by the FHWA, which is available at the FHWA website (www.fhwa.dot.gov). It will be the designer's responsibility to provide a debris barrier or deflector appropriate to the situation.

12.7. Sediment Basins

Sediment basins, check dams, and similar structures are a means of dropping out sediment held in suspension. Sediment basins constructed upstream of storm drain conduits help trap such material before it reaches the conduit. Sediment basins must be cleaned out on a regular basis in order to function effectively.

12.8. Berms and Levees

For levees to be recognized by FEMA, evidence of adequate design, operation and maintenance must be provided in order to provide reasonable assurance that protection from the 100-year flood exists. In addition to meeting the following design requirements, the owner of the levee must submit and comply with operation and maintenance procedures in accordance with 44 CFR 65.10.

Berms, dams, levees, and diversions of certain magnitudes and nature may fall within the jurisdiction of the TCEQ. The design professional is expected to be aware of and comply with regulations promulgated by the TCEQ.

12.8.1. Freeboard

Riverine levees must provide a minimum freeboard of three feet above the water-surface level of the 100-year flood. An additional one foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.

Occasionally, exceptions to the minimum riverine freeboard requirement described above may be approved. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated 100-year flood elevation profile and include, but not necessarily be limited to an assessment of statistical confidence limits of the 100-year discharge; changes in stage-discharge relationships; and the sources, potential, and magnitude of debris, sediment, and ice accumulation. It must be also shown that the levee will remain structurally stable during the 100-year flood when such additional loading considerations are imposed. Under no circumstances will freeboard of less than two feet be accepted.

12.8.2. Closures

All openings must be provided with closure devices that are structural parts of the system during operation and designed according to sound engineering practice. The USACE provides guidance on the selection of closure types within the *Engineering and Design – Structural Design of Closure Structures for Local Flood Protection Projects*, EM 110-2-2705, dated March 31, 1994. Examples of closures include swing gates, rolling gates, trolley gates, flap gates, and check valves.

12.8.3. Embankment Protection

Engineering analyses must be submitted that demonstrate that no appreciable erosion of the levee embankment can be expected during the 100-year flood, as a result of either currents or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability. The factors to be addressed in such analyses include, but are not limited to:

- Expected flow velocities (especially in constricted areas).
- Expected wind and wave action.
- Ice loading.
- Impact of debris.

- Slope protection techniques.
- Duration of flooding at various stages and velocities.
- Embankment and foundation materials.
- Levee alignment, bends, and transitions.
- Levee side slopes.

12.8.4. Embankment and Foundation Stability

Engineering analyses that evaluate levee embankment stability must be submitted. The analyses provided shall evaluate expected seepage during loading conditions associated with the 100-year flood and shall demonstrate that seepage into or through the levee foundation and embankment will not jeopardize embankment or foundation stability. An alternative analysis demonstrating that the levee is designed and constructed for stability against loading conditions for Case IV as defined in the USACE manual, *Design and Construction of Levees* (EM 1110-2-1913, Chapter 6, Section II), may be used. The factors that shall be addressed in the analyses include:

- Depth of flooding.
- Duration of flooding.
- Embankment geometry and length of seepage path at critical locations.
- Embankment and foundation materials.
- Embankment compaction.
- Penetrations.
- Other design factors affecting seepage (such as drainage layers).
- Other design factors affecting embankment and foundation stability (such as berms).

12.8.5. Settlement

Engineering analyses must be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement and demonstrate that freeboard will be maintained within the minimum standards set forth herein. This analysis must address embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods. In addition, detailed settlement analysis using procedures such as those described in the USACE manual, *Soil Mechanics Design – Settlement Analysis* (EM 1100-2-1904) must be submitted.

12.8.6. Interior Drainage

An analysis must be submitted that identifies the source(s) of such flooding, the extent of the flooded area, and if the average depth is greater than one foot, the water-surface elevation(s) of the base flood. This analysis must be based on the joint probability of interior and exterior flooding and the capacity of facilities (such as drainage lines and pumps) for evacuating interior floodwaters.

12.8.7. Levee Certification Requirements

Data submitted to support that a given levee system complies with the structural requirements set forth herein must be certified by a Professional Engineer registered by the State of Texas. Also, certified as-built plans of the levee must be submitted. Certifications are subject to the definition given in 44 CFR 65.2. In lieu of these structural requirements, a federal agency with responsibility for levee design may certify that the levee has been adequately designed and constructed to provide protection against the 100-year flood.

13. Pump Stations

The installation of a pump station will be considered when an area to be drained is so low that construction of a gravity drain is not feasible. Pumping stations are used to drain depressed sections of urban roadways and paved areas and for discharge of water from retention basins when other means of gravity drainage are not available.

The design of pumping stations involves many different disciplines and the design approach is dependent upon the size and purpose of the facility and the consequences of system failure. This chapter provides general requirements and guidelines for planning and analysis of pumping facilities. For a more rigorous discussion of the design of storm water pump stations, refer to *Highway Stormwater Pump Station Design, HEC-24* (USDOT, 2001).

13.1. Design Criteria

Gravity drainage of retention basins and other low-lying areas is preferred. Only under special circumstances with prior Storm Water Utility approval should pump stations be used.

13.2. Design Frequency

It is recommended practice to develop a design capable of accommodating at least a 50-year flood event because the pump station is generally used when drainage by gravity from a low point is inadequate or impractical. Pump stations must provide sufficient capacity to discharge the volume of storm runoff generated by the design storm while maintaining the water level below the maximum allowable elevation in the ponding area.

13.3. Drainage of Basins and Ponding Areas

Retention basins and ponding areas shall be drained within 72 hours following the storm, although less time may be required by the City. Retention basin pump stations are required for publicly maintained basins where the following conditions exist:

- The depth of water retained exceeds 1 foot.
- A gravity flow bleedoff system is not possible.
- The use of dry wells is not a viable option.

13.4. Bleedoff Lines

- The minimum pipe diameter for pressure bleedoff lines is 8 inches and 24 inches for gravity lines. A restrictor plate may be used to limit maximum rate of flow.
- All bleedoff lines shall have a method to shut off flows.
- A control valve or gate shall be installed such that they are readily available for inspection by City staff.
- Water cannot be discharged onto a City street or street gutter or alley.

The following, listed in order of preference, are methods of discharging from pump station facilities:

1. Discharge to an open channel either natural or man-made.

2. Discharge directly to a nearby stormdrain system with a maximum discharge limited to the available capacity of the system as approved by the City.
3. Discharge to the surface of a stormdrain system if pumped water can be discharged directly into a drop inlet or other inlet.

13.5. Pump Station Design Requirements

- If a pump station is to be used, the rate at which the pump station discharges must not overload downstream drainage systems.
- Storm water pump stations are classified by size, smaller or larger than 60 cubic feet per second. The larger stations will have additional requirements such as flow recording equipment.
- Pump stations shall be located so that the equipment is accessible when the basin or sump is full. Pumping facilities (excluding components whose design requires submersion) will be set at an elevation at or above the anticipated level of the design storm event, considering that a total power failure may occur.
- Pumps shall be capable of handling solids up to a maximum of 3 inches. Consideration for handling smaller solids can be made for pumping facilities that serve storage facilities.
- An inflow hydrograph for the design of the storage reservoir shall be determined in accordance with the procedures in Chapter 4 of this design manual.
- Plugging factors will be used on inlets of pipe systems that are tributary to pump stations. Fifty percent of the area of the trashrack shall be assumed plugged. If a pump station is designed with divided wet wells, only the trashracks associated with the active wet well or wells will be used in calculating plugged area for pump station operation.
- Maximum use of surface storage, instead of underground storage, is desirable for minimizing storage costs. Volumes of cross pipes, inlets, manholes, or drop inlets should not be considered as part of the available storage reservoir volume.

The engineer shall provide the following design information:

- Headloss calculations for the entire system, including maximum and minimum Total Dynamic Head (TDH) and flow rate.
- Net positive suction head (NPSH) and pump level settings for on, off, and alarm positions.
- Stage-storage curves (total storage that is available within the system at any stage between the inlet elevation of the pump station and the maximum allowable elevation in the wet well).
- Inflow and outflow hydrographs and accumulated inflow and outflow curves (mass flow curves). The use of HEC-1 is not appropriate for the design of pumping stations. A real-time procedure that routes the design inflow hydrograph using pump on and off elevations and actual pump performance curves must be used.

- Specifications for the model and type of pump(s) proposed including pump curves (single pump and parallel operation). Overloading the pump anywhere on the pump curve is not permitted.

13.6. Pump Station Facility Requirements

- Pump station structures should meet requirements for public safety, local extreme weather conditions, site security, and maintenance operation. Aesthetics and the possible need for future expansion should also be considered.
- The site layout should consider adequate access for maintenance vehicles to refill fuel tanks and remove pumps, generators, and accumulated debris and silt. The site layout should also be designed to mitigate on-site noise.
- The collection system shall discharge into a separate sump that screens the water before entering the pump sump.
- The wet well shall be a minimum of 6 feet by 10 feet inside dimensions and shall be provided with a means to drain it when the pump is not running.
- A pump shall be provided with an automatic control switch and a water level sensor. Larger stations shall provide communications equipments to permit transmission of failure signals to designated reporting locations and to allow remote operation of the pumps.
- A potable water supply with backflow prevention and hose bibs shall be provided to aid in removal of silt and trash.
- A ventilation system will provide intermittent ventilation of wet-wells and the pump room. The ventilation system should be designed to prevent accumulation of dust that will damage electrical and mechanical equipment.
- A redundant pumping system may be required, particularly at small installations.

13.7. Discussion

13.8. Pump Selection Study

Information required for the pump selection study includes:

- Proposed station capacity and pertinent water surface elevations for maximum, average, and minimum flow conditions.
- Points of discharge.
- Proposed station locations and their soil conditions.
- Proposed piping system.
- Pertinent information on terrain, locations of utilities, and power source availability.
- Proposed method of operation.

A complete hydraulic and economic analysis is necessary for any considered configuration of pumping capacity. Possible requirements for future expansion must be considered in the planning. Expansion may be accomplished by modification of the existing units (such as increasing the pump impeller diameter), by adding pumps or by replacement of the original pumps. Allowances for any of the alternatives should be made in the original design, as should the provision of a large enough motor if the pump impeller diameter is to be increased at a later time or the provision of space and foundation size for an additional pump along with proper sizing of piping.

13.9. System Analysis

A system analysis for a pump station will aid in the selection of the best pumping units. For the analysis, system head curves for the proposed system are calculated for critical conditions and combined with the characteristic curves of pumps that are being considered for the installation. A preliminary plan for the piping system is needed for this purpose.

A system head curve is a plot of total system head against flow rate where the total system head at any flow rate is the head to be supplied by the pump to produce the given flow rate at the discharge point of the piping system. The total system head is the static head plus the head losses in the piping system, which includes the pipe friction losses and the minor losses at entrance and exit and the fittings such as valves, bends, expansions, and contractions. See Figure 13-1.

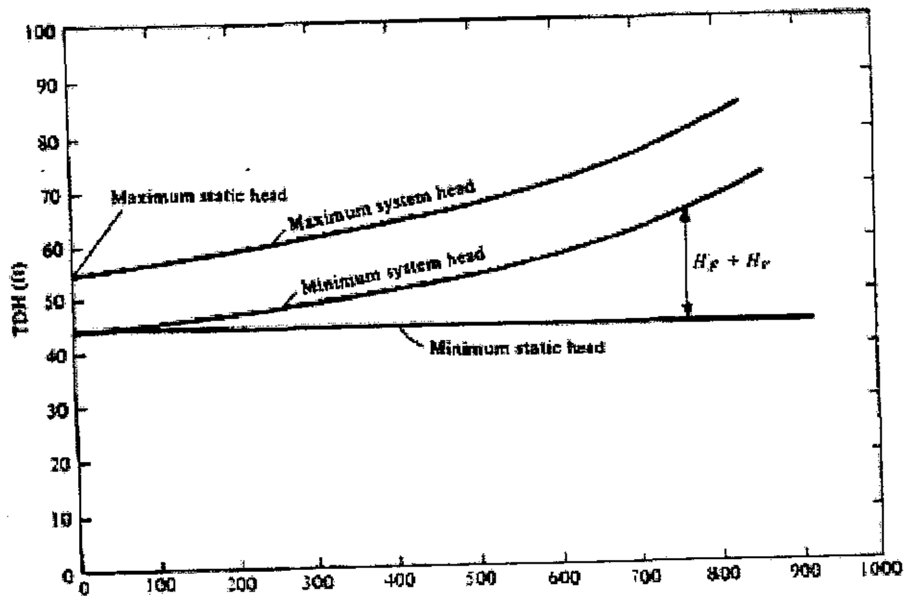


Figure 13-1: System-Head Curves for Fluctuating Static Pumping Head

Pipe friction losses are calculated by the Manning's equation. For this calculation the equation may be expressed as follows:

$$h = 2.87n^2 \frac{LV^2}{D^{4/3}} \qquad \text{13-1}$$

Where:

h = *Head loss, in feet.*

L = *Pipe length, in feet.*

D = *Pipe diameter, in feet.*

V = *Flow velocity, in feet per second.*

n = *Manning's roughness coefficient.*

Commonly used values of n range from 0.010 to 0.041. Wherever possible, local experience with different pipe materials should be used in choosing a value of n.

Minor losses are most often expressed as:

$$H_L = \frac{KV^2}{2g} \qquad 13\text{-}2$$

Where:

H_L = *Head loss, in feet.*

K = *Coefficient for the particular fitting.*

$V^2/2g$ = *Velocity head in the pipe, in feet.*

Alternatively, minor losses may be expressed as "equivalent length of pipe."

Static head is the difference in elevation between the water surfaces in the set well and that at the discharge point. Storm water pumping systems are usually designed to discharge into a channel, conduit, or receiving body at atmospheric pressure. In this case, the appropriate elevation to use is the centerline elevation of the effluent pipe (see Figure 13-2). Since the level in the wet well may vary, system head curves for maximum and minimum static heads are usually plotted.

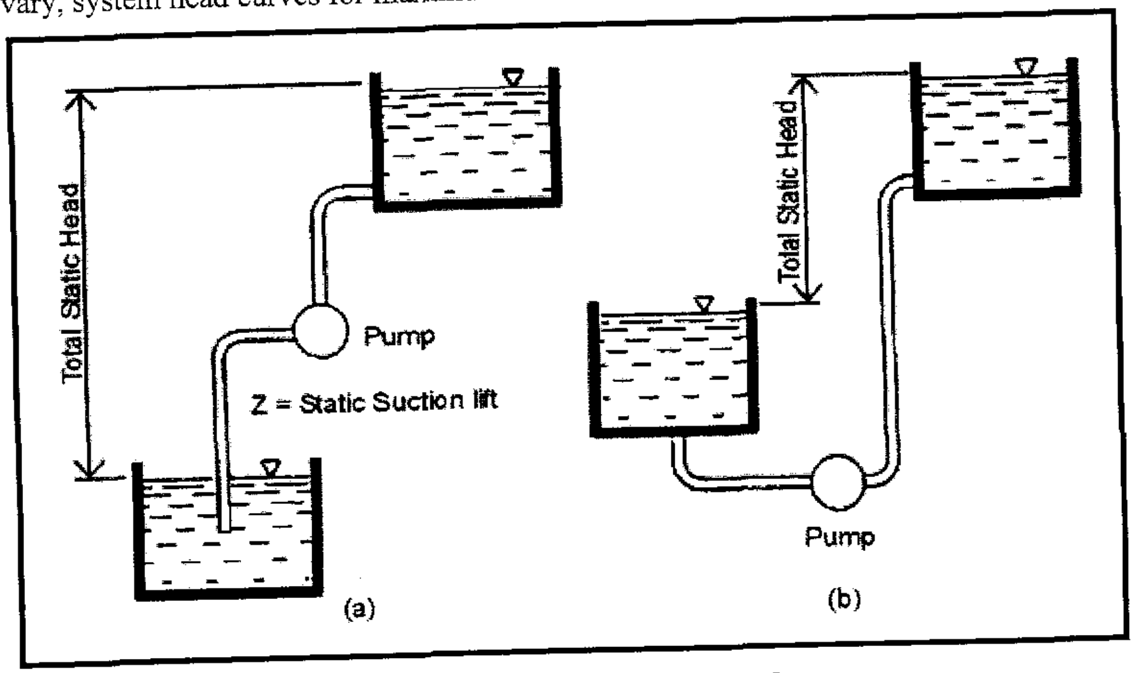


Figure 13-2: Total Static Head

The analysis proceeds with the plotting of the head-discharge curve for the selected pump(s). Intersection of the pump curve with a system head curve represents an operating point of the pump under the assumed conditions of static head and head loss. Efficiency and input power curves should be also plotted. The operating point (see Figure 13-1) should be at or near to the best efficiency point for the selected pump(s) in order to reduce noise, power costs, and excessive wear resulting in lower maintenance costs.

13.10. Pump Types

Storm water pumps may be vertical, wet-pit or dry-pit, submersible or horizontal, depending upon the individual conditions of each situation. Dry-pit pumps are usually volute pumps, while vertical wet-pit pumps generally have diffusion vanes to save space. Some advantages of the wet-pit type are no priming requirement, relatively high available NPSH and economy of space. Dry-pit pumps allow easier accessibility and require less maintenance. In small installations submersible pumps may be used. In such a pump, the motor is close-coupled to the pump and is submerged.

Storm water pumps may be radial, axial, or mixed flow types. In general, radial flow pumps are adapted to high-head, low-flow situations, while axial flow pumps are used for large flows at low heads. The specific speed can serve as a guide to choosing the proper pump type.

13.11. Pump Selection

It is typical in storm water pumping stations to require at least two pumps for reliability and redundancy, with either one capable of handling the station capacity. When more than one pump is used, the station capacity should be met when the largest pump is out of service.

Often, a small utility pump (e.g., a capacity of 1,200 gallons per minute or 4 cubic meters per minute) is used in the combination of pumps. Such a pump then would serve to cycle on and off during small runoff events while the larger capacity pumps would be reserved for the larger runoff events. This pump would also be used to drain the wet well to a level lower than would be capable by the larger pumps. This is often referred to as a sump pump.

Pump speeds used in storm water pumping area usually limited to 1,750 revolutions per minute to minimize maintenance problems and provide greater reliability. Generally, constant speed control is used.

13.12. Piping and Valves

The suction inlet (suction bowl) for dry pit pumps should be flared to avoid the formation of vortices that cause air to be drawn into the pump.

Vertical wet pit pumps are very sensitive to intake conditions within the wet well. Follow the recommendations of the pump manufacturer.

Suction piping should be kept as short as possible to minimize head losses and should not contain any unnecessary fittings. The line should contain a flanged gate valve and an eccentric flanged reducer with the flat side up, as the suction pipe should be one or two sizes larger than the pump suction nozzle. Velocities in the suction pipe should not exceed 7 feet per second with lower velocities desirable.

Discharge piping size should be based upon velocities not exceeding 8 feet per second with lower values preferred. The pipe size should be at least one size larger that the pump discharge nozzle with the transition made by a concentric increaser. The increaser should be followed by a check valve and gate in a horizontal section of pipe. The possible occurrence of water hammer should be analyzed and appropriate surge control measures provided.

13.13. Location

The following points related to the location of a pump station should be taken into account:

- Accessibility for maintenance and removal of equipment for repair.
- Parking space for personnel and emergency equipment, such as generators.
- Local drainage and protection from flooding.
- Availability of suitable electrical power supply.
- Soil conditions at proposed site and information on ground water levels, particularly maximum heights.
- Safety and environmental requirements of local codes should be met, including those concerning exhaust of internal combustion engines, if used.

13.14. Wet Well Design

The principal purposes of the wet well are to provide a sump for the pump suction intakes and to supply storage to minimize on-off cycling of the pumps. For small stations a common requirement is to have a volume (in gallons) between on and off levels in the well of 2.5 times the pump flow rate in gallons per minute. This requirement is based upon the criteria that the minimum on-off cycle for a pump should be 10 minutes and for a uniform pumping rate is twice the inflow. The cycling time for a single pump is given by:

$$t = \frac{V_w}{P-Q} + \frac{V_w}{Q} \qquad \text{13-3}$$

Where:

T = Cycling time, in minutes.

P = Pumping rate, in gallons per minute.

Q = Inflow, in gallons per minute.

V = Wet well volume, in gallons, between the levels of the on and off controls.

Then, the required wet well volume for minimum cycling time, t_m, is:

$$V_w = \frac{t_m P}{4} \qquad \text{13-4}$$

or for a 10 minute minimum:

$$V_w = 2.5P \qquad \text{13-5}$$

If two pumps are used alternately, the cycling time is increased by a factor of two.

Other details of wet well design that should be considered are the following:

- The wet well floor design should minimize deposition of solids.

- Access to the wet well should be from the outside and means of ventilation should be provided.
- Divided wet wells should be provided for larger pumping stations so that one part may be used while the other is shut down.
- A high water alarm should be provided.
- Screening may be required if the storm water is expected to be carrying debris that would damage the pumps.

13.15. Pump Room or Drywell

Besides space for pumps, motors, and control equipment, the pump station must provide space and facilities so that maintenance work on all equipment can be done effectively and safely. One rule for spacing between pumps is at least 3 feet from each outside pump to the wall and 4 feet between each pump discharge casing. The pump room should be adequately and safely lighted and ventilated. Floor drainage should be provided. Openings for removing equipment should provide ample space for doing so. Power operated crane hoists should be installed over basket screens, submersible pumps, other pumps or motors, and other locations where it is necessary to lift heavy pieces of machinery or equipment.

13.16. Pump Settings

The pump setting is the location in elevation of the impeller or propeller centerline with respect to the water level. The setting needed is related to the following requirements:

- Provision of sufficient available NPSH at extreme operating conditions.
- Sufficient submergence of the suction inlet to prevent air from being drawn into the pump by vortices.
- Priming or the need for a centrifugal pump to be filled with water when started.

For dry pit pumps, it is recommended that the high point of the casing be set below the minimum water level in the wet well to ensure proper priming.

13.17. Pump Station Controls

Storm water pump controls may include the following elements:

- Water level sensors for pump activation and deactivation.
- Manual on-off switches to operate one or more constant speed pumps.
- Step or stepless variable speed control units.
- Mechanical or electrical alternators for pump sequencing of two or more pumps.
- Activation of the standby generator when necessary.
- Operation of security lighting, cameras, or other equipment.
- Automatic communication with the central office.

Sensing elements for maximum and minimum water levels should be separated by at least 3 feet, and individual controls should be at least 12 inches apart.

In addition to the on-off controls, a high water alarm, low water alarm, and cut-out switch should be installed. The maximum water level should be such that undesirable surcharging of the incoming storm water is prevented and the high water alarm should be 6 inches above this level. The low water alarm should be 12 inches below the minimum wet well level.

13.18. Electrical Power Supply

The power authority should be consulted to determine what voltage the type of electrical power is available at the station site. For small stations 240 or 480 volt three phase is usually used. For large motors (over 400 horsepower), higher voltages may be preferred, if available, for economy.

For reliability in large pumping stations, two independent incoming power lines should be available at the pumping station and provisions for supplying emergency generator power to the pumps should be considered.

13.19. Emergency Power Supply

In situations where electrical failure would result in prolonged inundation and cause undue hardship such as arterial roadway dip sections under bridges, an emergency back up generator is necessary. Placement of the generator above flood levels or proper flood proofing is mandatory. This applies to fuel storage as well. Standby generators are usually powered by diesel, propane, or natural gas. Fuel type depends on the size of the standby unit, the fuel source and availability, the site layout, and local economics. Proper exhaust ventilation must be planned. The need for emergency power supply will be considered on a case-by-case basis and will be dependent upon the threat to public health and welfare.

13.20. Safety and Security

All pump stations should have adequate controls to prevent unlawful access to the pump station. These controls may include steel doors, locks, gates, exterior lighting, security cameras, unauthorized entry alarms, surrounding walls, and fencing. When planning fencing or surrounding walls, provide adequate access for service and maintenance vehicles. The security requirements will be evaluated based on the location and surround areas of the pump station.

13.21. Aids

The following table provides a general checklist for the design and hydraulic analysis of pump stations.

Table 13-1: Design Checklist for Pump Stations

General
Initial Data
Contributing Drainage Basin
Location of Outfall
Capacity of Outfall
Probable Growth in the Contributing Basin
Inflow Hydrographs
Possible Components
Source of Power (primary and emergency)

General
Pumps
Intakes and Drop Inlets
Controls
Storage
Debris Handling
Potable Water Supply
Testing
Hoisting Equipment
Ventilation
Controls of Hazardous Materials
Hydrology
Economic and Alternative Analysis
Designation of Significantly Different Concepts
Hydrologic and Hydraulic Detailing of Alternatives
Cost Evaluation
Extreme Event Evaluation of Components and Alternatives
Environmental Considerations
Documentation and Comprehensive Evaluation

Hydraulic Analysis
Mass Curve Routing
Outflow Hydrograph
Pump Characteristics
Pipe Losses
Miscellaneous Losses
Sediment Transport
Additional Considerations

14. Sediment Transport

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15. Floodplain Management

The purpose of proper floodplain management is to provide measures to protect lives and property. The City of El Paso participates in the National Flood Insurance Program (NFIP) and has adopted a Flood Damage Prevention (FDP) Ordinance (Chapter 18.60 of the City of El Paso Municipal Code) to ensure proper floodplain management development occurs within the identified Special Flood Hazard Areas (SFHA). By participating in the NFIP and enforcing the FDP ordinance flood insurance is made available to anyone within the City.

The date of the effective Flood Insurance Rate Maps (FIRMs) is January 3, 1997. The City, in conjunction with the County, has taken steps with FEMA to update the current effective FIRMs into digital FIRMs (DFIRMS) with revised floodplains.

The City of El Paso has both detailed floodplains with base flood elevations (BFEs) and floodways identified and unnumbered A-zones where no BFEs have been determined.

The City has also established a permitting process for any floodplain development that must be adhered to.

15.1. FEMA Defined Floodplains and Floodways

FEMA has identified SFHAs and established floodways that are shown on the effective FIRMs. The policies related to implementation of the NFIP Rules and Regulations (Section 60.3) along with those of the City ordinance are as follows:

15.1.1. Policy 1: Best Available Data

New or updated data for SFHAs are constantly being developed by the City and other entities. It is the City's policy, in conformance with the NFIP Rules and Regulations (Section 60.3[a] and [b]) and FDP Ordinance, to use this information for regulatory purposes when there are no identified floodplains but has the potential to flood, is more restrictive than what is published on the effective FIRMs or is identified as an unnumbered A zone of the effective FIRMS. It will be considered as "Best Available Data" for floodplain management purposes and will be used for all new development. The data should establish floodplain information such as BFEs and floodway delineations for any proposed structure or development.

For proposed subdivisions that are 5 acres or 50 lots in size, which ever is lesser, the developer is required to provide technical data developed by a Texas registered professional engineer that has established BFEs and floodway delineations prior to issuance of any permits.

15.1.2. Policy 2: CLOMR Requirement Prior to Issuance of a Grading Permit

Any revisions to a floodplain with detailed data and/or encroachments into the designated floodway that may cause increases to the BFE, must receive the City of El Paso and FEMA's approval by submitting for a Conditional Letter of Map Revision (CLOMR). The approved CLOMR request must be received before a grading and drainage permit will be issued by City of El Paso for the development. A Letter of Map Revision (LOMR) will then be required once the proposed development has been completed.

15.1.3. Policy 3: LOMR Requirement Prior to Final Development Approval

Any proposed development that has submitted a CLOMR to FEMA for revisions of the FEMA-designated floodplain and/or floodway must receive a FEMA-approved LOMR before final approval by City of El Paso is granted for building occupancy for the development.

15.1.4. Policy 4: Location of Structures

The developer should locate proposed structures outside of SFHAs when possible. The floodplain administrator shall work with the developer to determine the building placement prior to issuance of any building or grading permits. Relevant factors that will be considered in the development of adequate structure placement are:

- The danger to life and property due to flooding or erosion damage.
- The susceptibility of the proposed facility and its contents to flood damage and the effect of such damage on the individual owner.
- The danger that materials may be swept onto other lands to the injury of others.
- The compatibility of the proposed use with existing and anticipated development as determined by the director of planning, research, and development.
- The safety of access to all buildings in the time of flood.
- The expected heights, velocity, duration, rate of rise, and sediment transport of the floodwaters at the site.
- The justification of the proximity of the facility to the abutting floodway, where applicable.
- The availability of alternate locations, not subject to flooding or erosion damage, for the proposed use.
- The relationship of the proposed use to the comprehensive plan for that area.

15.1.5. Policy 5: Public and Private Roads Affecting Effective SHFAs

A CLOMR and LOMR must be submitted to the City of El Paso and FEMA for approval for any proposed roadways that affect a designated floodplain and/or floodway.

In the event that a proposed project is located within a floodplain with BFEs established, a Texas licensed professional engineer shall submit, at a minimum, a grading plan, drainage control plan, and temporary and permanent erosion control plans to the City of El Paso.

The grading plan shall include existing contours, proposed contours, finished grade elevations, and finished floor elevations. Detailed information of the grading is presented in Chapter 18.44 of the City of El Paso Municipal Code. It is unacceptable that the proposed grading scheme adversely impact adjoining properties or existing structures.

The drainage control plan shall provide tabulated calculations of 100-year storm event for retention and detention basins, and the 50-year storm event for streets, channels, and underground storm drains, which will include drainage basin areas, flows, and drain pipe capacities. Proposed drainage infrastructure is to be congruent with upstream and downstream conditions. The proposed method of ponding shall contain the difference between the post development and predevelopment runoff.

The control plan shall depict flow patterns and high and low points, and locations of existing and proposed drainage systems.

The temporary and permanent erosion control plan shall show location and installation of erosion control elements and will be in accordance with Chapter 15 of this manual.

15.2. Floodplain Management Standards for SFHAs without BFEs

For development in areas of the floodplain without BFEs established, Policy 1 and the following standards must be complied with:

15.2.1. Anchoring

All new construction or substantial improvements shall be designed or modified and adequately anchored to prevent flotation, collapse, or lateral movement of the structure resulting from hydrodynamic and hydrostatic loads, including the effects of buoyancy. As defined in the FDP ordinance, new construction is defined as structures for which the start of construction commenced on or after the effective date of the floodplain management regulation adoption by the City of El Paso and includes any subsequent improvements to such structures. Substantial improvement is defined as any reconstruction, rehabilitation, addition, or other improvement of a structure, the cost of which equals or exceeds fifty percent of the market value of the structure before start of construction of the improvement. Please refer to the FDP ordinance for a complete listing of relevant definitions.

15.2.2. Construction Materials and Methods

All new construction and substantial improvements shall be constructed:

(A) With materials and utility equipment resistant to flood damage.

(B) Using methods and practices that minimize flood damage.

15.2.3. Utilities

(A) All new and replacement water supply systems shall be designed to minimize or eliminate infiltration of floodwaters into the system.

(B) New and replacement sanitary sewerage systems shall be designed to minimize or eliminate infiltration of floodwaters into the systems and discharge from the systems into floodwaters.

(C) On-site waste disposal systems shall be located to avoid impairment to them or contamination from them during flooding.

All new construction or substantial improvements shall be constructed with electrical, heating, plumbing, ventilation and air conditioning equipment, and other service facilities that are designed and/or located so as to prevent water from entering or accumulating within the components during conditions of flooding.

15.2.4. Subdivision Proposals in SFHAs

Subdivision proposals in SFHAs shall conform to Policy 1 and the following:

(A) All subdivision proposals shall be consistent with the need to minimize flood damage.

(B) All subdivision proposals shall have public utilities and facilities such as sewer, gas, electrical, and water systems located and constructed to minimize flood damage.

(C) All subdivision proposals shall have adequate drainage provided to reduce exposure to flood damage.

(D) Base flood elevation data shall be provided for subdivision proposals and other proposed developments that contain at least 50 lots or five acres, whichever is less.

15.2.5. Standards of Areas of Shallow Flooding (AO and AH Zones)

These SFHAs have associated base flood depths of one to three feet where a clearly defined channel does not exist and where the path of flooding is unpredictable and indeterminate. Therefore, the following provisions apply:

(A) All new construction and substantial improvements of residential structures must have the lowest floor, including the basement, elevated above the highest adjacent grade at least as high as the depth number specified in feet on the FIRM (at least two feet if no depth number is specified).

(B) All new construction and substantial improvements of nonresidential structures must:

(1) Have the lowest floor, including the basement, elevated above the highest adjacent grade at least as high as the depth number specified in feet on the county FIRM (at least two feet if no depth is specified); or

(2) Be designed together with attendant utilities and sanitary facilities so that the structure is watertight below the base flood level with walls substantially impermeable to the passage of water and with structure components having the capability of resisting hydrostatic and hydrodynamic loads of effects of buoyancy.

(C) A Texas licensed professional engineer shall submit a certification to the City floodplain administrator that the standards of this section are satisfied.

(D) Require adequate drainage paths around structures on slopes within zones AH and AO to guide floodwaters around and away from proposed structures.

15.3. Floodplain Management Standards for SFHAs w/ Established BFEs

For development in SFHAs where BFEs have been established, the following specific standards apply:

15.3.1. Residential Construction

New construction and substantial improvements of any residential structure shall have the lowest floor, including the basement, elevated at or above the BFE. An elevation certificate will be required to document this.

15.3.2. Nonresidential Construction

New construction and substantial improvement of any commercial, industrial, or other nonresidential structure shall either have the lowest floor, including the basement, elevated at or above the BFE, or it, together with attendant utility and sanitary facilities shall:

(A) Be flood proofed so that the structure is watertight below the base flood level with walls substantially impermeable to the passage of water.

(B) Have structural components capable of resisting hydrostatic and hydrodynamic loads and effects of buoyancy.

(C) Be certified by a Texas licensed professional engineer that the standards of this subsection are satisfied.

15.3.3. Manufactured Homes

(A) All manufactured homes to be placed within zone A shall be installed using methods and practices which minimize flood damage. For the purpose of this subsection, manufactured homes must be elevated and anchored to resist flotation, collapse, or lateral movement. Methods of anchoring may include, but are not limited to, use of over-the-top or frame ties to ground anchors. This requirement is in addition to applicable state and local anchoring requirements for resisting wind forces.

(B) All manufactured homes shall be in compliance with section 15.3.3a.

(C) All manufactured homes are to be placed or substantially improved within zones A1-30 and AH on the community's FIRM. They must be elevated on a permanent foundation such that the lowest floor of the manufactured home is at or above the base flood elevation, and must be securely anchored to an adequately anchored foundation system in accordance with the provision of section 15.3.3a.

15.3.4. Floodways

Floodways are an extremely hazardous area due to the velocity of the floodwaters, which carry debris and potential projectiles, along with a significant erosion potential. It must be kept clear of all obstructions to allow the flow of the 100-year storm event; therefore, the following provisions shall apply:

(A) Encroachments are prohibited, including fill, new construction, substantial improvements, and other developments, unless certification by a Texas licensed professional engineer is provided demonstrating that encroachments shall not result in any increased flood levels during occurrence of the base flood discharge. A "No-Rise" certification must be submitted to the City for review and approval prior to any permits being issued.

(B) If subsection 15.2.5a of this section is satisfied, all new construction and substantial improvements shall comply with all applicable flood hazard reduction provisions of this section.

(C) Prohibit placement of any mobile homes, except those that meet the requirements of section 38-102(4).

15.4. Areas Outside the SFHAs

There are many flood prone areas in the City of El Paso that do not have identified SFHAs. The City's mission is clear: to provide regional flood hazard identification, regulation, remediation, and education for residents so that they can reduce their risks of injury, death, and property damage from flooding, while still enjoying the natural and beneficial values served by floodplains.

The City of El Paso's policies pertaining to areas outside of SFHAs or erosion prone areas are provided in Policy 1 and the following:

15.4.1. Requirement to Delineate 100-year Flood Hazard Area or Establish Minimum Finished Floor Elevation

In locations where development is proposed and a SFHA designation does not exist, but has the potential to be a flood prone area, the delineation of the 100-year flood hazard area may be required. Elevations determine and must be in compliance with FDP ordinance, or they must establish the minimum finished floor elevation to be at 3 feet about the highest adjacent grade to the proposed structure.

15.4.2. Erosion Protection

Building pads and foundations may be required to have an additional setback or be protected from erosion and scour.

15.4.3. Lot Grading

Lots are to be graded to drain so as not to adversely affect adjacent property owners. Runoff redirected from its natural flow location may drain onto or through an adjacent property if a written agreement is in place with the affected property owner or a drainage easement or tract is provided. Such agreements, easements, or tract must be recorded against the deed of the affected properties. A legal description and exhibit drawing of every easement must be included as a part of the recorded documents. Refer to Chapter 16 of this document for more information on drainage right-of-ways and easements.

16. Rights-of-Way and Easements

When a subdivision is traversed by a watercourse, drainage way, channel, and underground facilities, there shall be provided either a storm water easement or drainage right-of-way conforming substantially to the lines of such a watercourse, and of such width and construction as outlined in this section. When a proposed drainage system routes storm water across private land outside the subdivision, appropriate drainage rights shall be secured by the sub-divider and indicated on the plat. All existing and proposed drainage easements and rights-of-way shall be shown on the plat.

Rights-of-way and permanent easements required for drainage, flood control, and erosion control facilities will conform to the following criteria:

16.1. (Intentionally left blank for Future Use)

16.2. (Intentionally left blank for Future Use)

16.3. (Intentionally left blank for Future Use)

16.4. (Intentionally left blank for Future Use)

17. Drainage Submittal Requirements and Checklist

17.1. Deliverables

- Drainage Report with supporting Calculations
- Grading and Drainage Plan

17.2. Drainage Report Checklist

☐ General project location

☐ Development concept for the site

☐ Drainage concept for the site (include relevant #'s as appropriate)

☐ How off-site flows will be handled

☐ How on-site flows will be handled and discharged

☐ Downstream capacity and how determined

☐ Impacts on or requirements of other jurisdictions

☐ Identify all approvals being requested in conjunction with this submittal, such as:

- Zone Change
- Subdivision Plat
- Site Plan for Subdivision
- Site Development Plan for Building Permit
- Building Permit
- Grading Permit
- Paving Permit
- Design Variance
- Conditional Letter of Map Revision (CLOMR), Letter of Map Revision (LOMR), or Letter of Map Amendment (LOMA)

☐ Previously approved drainage management plans, drainage reports, plans, or studies including watersheds, basins, drainageways, etc. as defined therein

☐ Legal description

☐ Identify proximity of site to a designated FEMA Flood Hazard Zone

☐ Include a copy of the relevant FEMA FIRM or Flood Boundary and Floodway Map with the site clearly identified along with all affected Flood Zones

☐ Identify portion of designated Flood Hazard Zone to be revised or amended when CLOMR, LOMR, or LOMA approval requested

17.3. Calculations Checklist

☐Fully developed watershed, ability to accept and safely convey runoff generated from design storm

☐Hydrologic Analysis

- ☐*Watershed delineation, existing and proposed*
- ☐*Time of Concentration calculations (where applicable)*
- ☐*Runoff coefficients development calculations or supporting information*
- ☐*Development of modeling input parameters*

☐Hydraulic Capacity

- ☐*Channel*
- ☐*Crossing structure*
- ☐*Storm inlet and/or entrance conditions*
- ☐*Storm drain*
- ☐*10- and 100-year Hydraulic Grade Line*
- ☐*Street and/or alley*
- ☐*Storage capacity*
- ☐*Detention pond/reservoir*
- ☐*Retention pond*
- ☐*Flood zone impacts of changes*
- ☐*Stability*
- ☐*Channel/Arroyo*
- ☐*Natural slope*
- ☐*Cut/Fill slope*
- ☐*Sediment bulking*

17.4. Grading and Drainage Plan Checklist

☐Professional Engineer's stamp with signature and date

☐North Arrow

☐Scales - recommended engineer scales:

1" = 20' for sites less than 5 acres

1" = 50' for sites 5 acres or more

☐Legend

☐Plan drawings size: 24" x 36" or 11"x17"

☐ Notes defining property line, asphalt paving, sidewalks, planting areas, ponding areas, project limits, and all other areas whose definition would increase clarity

☐ Top of curb elevations, gutter elevations, and finished ground elevation

☐ Proposed and existing contours

☐ Finish floor and pad elevations

☐ Grade change and grade break locations

☐ Slope arrow and slope percentage

☐ Vicinity Map

☐ Benchmark - location, description, and elevation in State Plane Coordinates

☐ Permanently marked temporary benchmark located on or very near site

☐ Flood Hazard Boundary Map (FHBM) or FIRM

☐ Legal description

18. Bibliography

Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA). *Sediment and Erosion Design Guide*, RCE Ref. No. 90-5560, November 1994.

AMAFCA. *Draft Design guide for Trapezoidal Concrete Flood Control Channels*, Revised April 1982.

American Society of Civil Engineers (ASCE). *Pipeline Design for Water and Wastewater*, 1975.

Chow, Ven Te. *Open Channel Hydraulics*. McGraw-Hill Book Company, Inc., New York, 1959.

Chow, Ven Te, Maidment, David R., Mays, and Larry W. *Applied Hydrology*. McGraw-Hill Book Company, Inc., New York, 1988.

Clark, John W., Viessman, Warren, and Hammer, Mark J. *Water Supply and Pollution Control*, Third Edition, Harper & Row, 1977.

Code of Federal Regulations, Title 44, Volume 1, Sec. 65.10 – Mapping of Areas Protected By Levee Systems, U.S. Government Printing Office via GPO Access, Revised October 2002.

City of El Paso. *City of El Paso, Texas - Title 19: Subdivision Regulations*, Draft April 2007.

Flood Control District of Maricopa County. *Hydraulics Drainage Design Manual for Maricopa County, Arizona*, Phoenix, Arizona, January 2004.

King, Horace W. and Brater, Ernest F. *Handbook of Hydraulics for the Solution of Hydrostatic and Fluid-Flow Problems*, Fifth Edition, McGraw-Hill Book Company, Inc., New York, 1963.

Los Angeles County Flood Control District. *Design Manual – Hydraulics*, Los Angeles, California, Revised 1973.

Merritt, Frederick S. *Standard Handbook for Civil Engineers*, McGraw-Hill Book Company, Inc., New York, 1968.

Morris, H.M. and Wiggert, James M. *Applied Hydraulics in Engineering*, Second Edition, The Ronald Press Company, 1972.

Natural Resources Conservation Service Web Soil Survey <http://websoilsurvey.nrcs.usda.gov/app/>, Accessed June 1, 2007.

Petersen, M. *River Engineering*, Prentice Hall, New York, 1986.

Rouse, H., (ed). *Engineering Hydraulics*, John Wiley and Sons, Inc., New York, 1950.

Simons, Li and Associates. *Design Guidelines and Criteria for Channels and Hydraulic Structures on Sandy Soil*, prepare for Urban Drainage and Flood Control District and City of Aurora, Colorado, June 1981.

Simons, Daryl B. and Senturk, Fuat. *Sediment Transport Technology*, Revised edition, Water Resources Publications, Littleton, Colorado, 1992.

Streeter, Victor L. *Fluid Mechanics*, Fifth Edition, McGraw-Hill Book Company, Inc., New York, 1971.

Texas Department of Transportation (TXDOT). *Hydraulic Design Manual*, March 2004.

US Army Corps of Engineers (USACE). *Engineering and Design – Structural Design of Closure Structures for Local Flood Protection Projects*, EM 1110-2-2705, Office of the Chief of Engineers, Washington, D.C., 1994.

USACE, *Hydraulic Design of Flood Control Channels*, EM 1110-2-1601, Office of the Chief of Engineers, Washington, D.C. 1970.

US Department of Homeland Security, Federal Emergency Management Agency (FEMA) Region VI, Flood Frequency Determination – El Paso County and Incorporated Communities, Texas, Disaster Response Recovery Data, FEMA-1658-DR, March 2007.

US Department of the Interior, Bureau of Reclamation. *Hydraulic Design of Stilling Basins and Energy Dissipaters*, Fourth Printing, Office of Water Data Coordination, Reston, Virginia, Revised September 1981.

US Geologic Survey (USGS). *Guidelines for Determining Flood Flow Frequency*, U.S. Government Printing Office, Washington, Revised 1973.

US Department of Transportation Federal Highway Administration (FHWA). *Hydraulic Design of Energy Dissipators for Culverts and Channels*, Hydraulic Engineering Circular No. 14, FHWA-NHI-06-086, Washington, DC., July 2006.

FHWA. *Debris Control Structures Evaluation and Countermeasures*, Hydraulic Engineering Circular No. 9, Third Edition, FHWA-IF-04-016, Washington, DC., September 2005.

FHWA. *Hydraulic Design of Highway Culverts*, Hydraulic Design Series No. 5, FHWA-NHI-01-020, Washington, D.C., Revised May 2005.

FHWA. *River Engineering for Highway Encroachments*, Hydraulic Design Series No. 6, FHWA-NHI-01-004, Washington, D.C., December 2001.

FHWA. *Introduction to Highway Hydraulics*, Hydraulic Design Series No. 4, FHWA-NHI-01-019, Washington, D.C., August 2001.

FHWA. *Urban Drainage Design Manual,* Second Edition, Hydraulic Engineering Circular No. 22, FHWA-NHI-01-021, Washington, D.C., July 2001.

FHWA. *Evaluating Scour at Bridges*, Fourth Edition, Hydraulic Engineering Circular No. 18, FHWA-NHI-01-001, Washington, DC., May 2001.

FHWA. *Bridge Scour and Stream Instability Countermeasures – Experience, Selection, and Design Guidance,* Second Edition, Hydraulic Engineering Circular No. 23, FHWA-NHI-01-003, Washington, D.C., March 2001.

FHWA. *Stream Stability at Highway Structures*, Third Edition, Hydraulic Engineering Circular No. 20, FHWA-NHI-01-002, Washington, DC., March 2001.

FHWA. *Highway Stormwater Pump Station Design*, Hydraulic Engineering Circular No. 24, FHWA-NHI-01-007, Washington, DC., February 2001.

FHWA. *Design of Riprap Revetment*, Hydraulic Engineering Circular No. 11, FHWA-IF-04-016, Washington, DC., March 1989.

US Department of Agriculture (USDA) Soil Conservation Service (SCS, now the National Resource Conservation Service). *National Engineering Handbook, Section 3 – Sedimentation*, Soil Conservation Service, Washington, D.C., 1983.

USDA SCS. *National Engineering Handbook, Section 4 – Hydrology, Chapter 15 – Travel Time, Time of Concentration, and Lag,* Soil Conservation Service, Washington, D.C., 1972.

USDA SCS. *Soil Survey El Paso County, Texas*, Soil Conservation Service Cartographic Division, Washington, D.C., November 1971.

USDA SCS. *Planning and Design of Open Channels*, Technical Release No. 25, Soil Conservation Service, Washington, D.C., October, 1971.

Viessman Jr., Warren and Lewis, Gary L. *Introduction to Hydrology*, Fourth Edition, HarperCollins College Publishers, New York, 1996.