DRAINAGE DESIGN MANUAL

CITY OF DALLAS

SEPTEMBER 2019

City of Dallas

DRAINAGE DESIGN MANUAL

SEPTEMBER 2019

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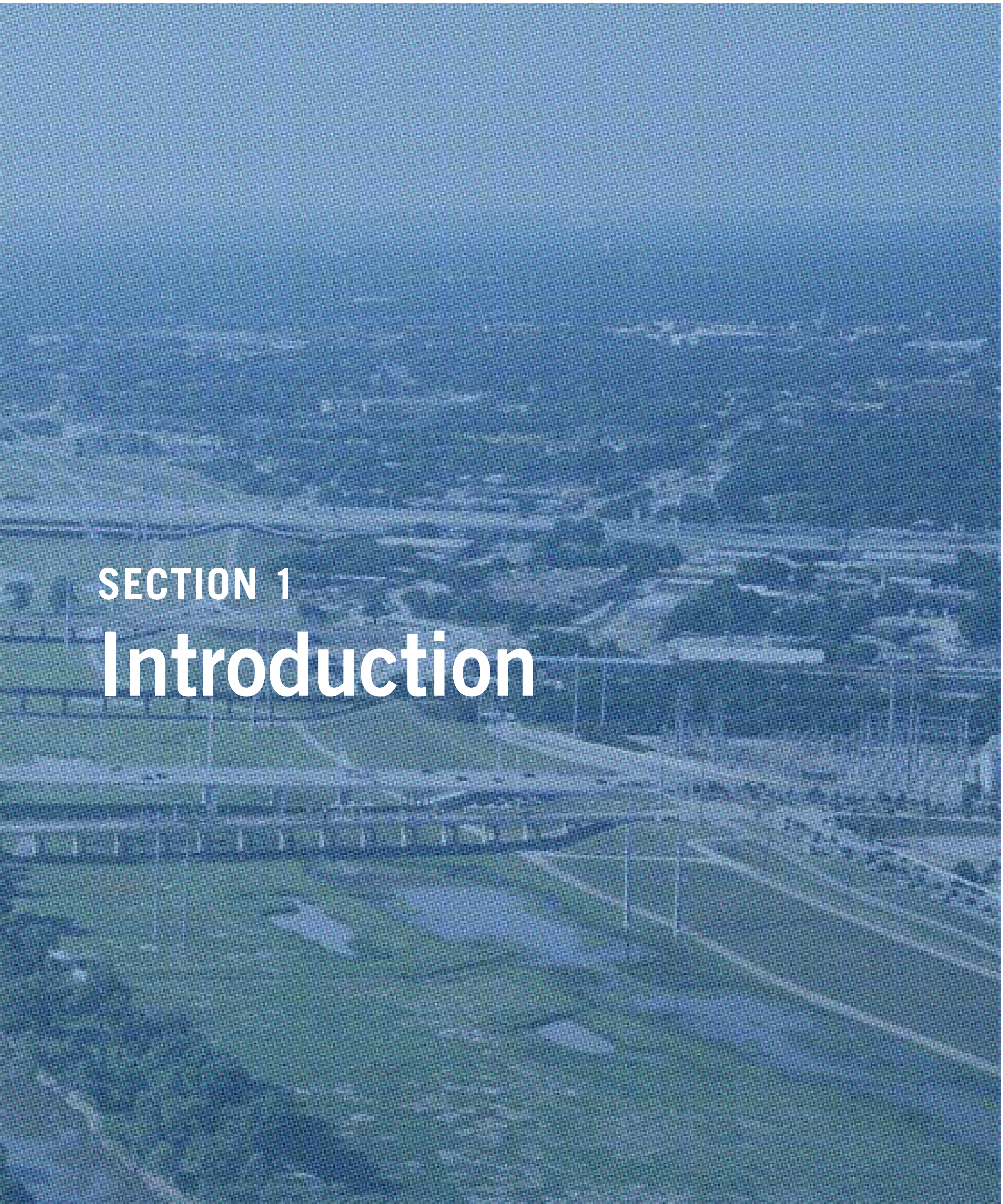


SECTION 1

Introduction

1.1

PURPOSE

The purpose of this Drainage Design Manual is to establish standard principles and practices for designing drainage facilities in the City of Dallas. This manual is for use by all City of Dallas departments, consultants employed by the City, and engineers for private development in the City. It is not intended to limit the design capabilities or engineering judgment of the design professional or the use of new technical developments in engineering. Responsibility for design remains with the design engineer.

1.2

SCOPE

The guidelines contained in this manual have been developed from a comprehensive review of basic design technology as contained in various engineering publications, and through the City's historical design and maintenance experience. This manual addresses storm drainage situations that are generally typical of the City of Dallas and its immediate geographical area. Accepted engineering principles are applied to these situations in detailed documented procedures. Criteria presented herein are intended to be consistent with federal and state regulations, including but not limited to the United States Army Corps of Engineers (USACE) authority concerning the Clean Water Act and the City's participation as a cooperating technical partner with the Federal Emergency Management Agency (FEMA) under the Community Rating System (CRS). The documentation is not intended to limit initiative but rather is included to provide a standardized format to aid in design and as a record source. Unusual circumstances or special designs requiring variances from the standards within this manual may be coordinated and approved by the applicable Department Director. All requests for a variance must be submitted in writing to the Department Director. Pre-development meetings to discuss strategies and concepts are recommended and may be required.

Additional information and development regulations regarding drainage design can be found in the Development Code - Article V Sec. 51A-5.100-5.105.

Additional federal and local regulations may apply to a project. The project and regulating documents should be reviewed in their entirety to determine applicability. All other applicable regulations and permit requirements must be fulfilled to obtain approval from the City of Dallas. A list of reference regulatory documents is included in Appendix A.1.

1.3

OVERVIEW

This manual is divided into the following 11 sections:

Section 1 Introduction is a general discussion of the intended use of the material and an explanation of its organization.

Section 2 Hydrology is a discussion of hydrologic methods and analysis criteria for the determination of design discharges, volumes, and hydrologic impacts of the development of a site.

Section 3 Roadway Drainage Design lists criteria and parameters for the design of roadway drainage elements, including inlets, storm drains, culverts, and outfalls, and outlines the design procedures and methodology used by the City of Dallas.

Section 4 Bridge Hydraulic Design includes methodology and criteria for the hydraulic design of bridges.

Section 5 Open Channel Design includes methodology, design procedures, and criteria for the hydraulic design of open channels.

Section 6 Drainage Storage Design includes methodology, design procedures, and criteria for drainage storage systems.

Section 7 Erosion and Sediment Control Measures includes temporary erosion control guidance, and methodology and criteria for permanent channel erosion and sediment control measures.

Section 8 Floodplain and Sump Design Requirements is a discussion of design criteria and regulatory requirements within areas defined by the City of Dallas as floodplain or sumps.

Section 9 Other Regulatory Requirements lists requirements for permitting, easements, and submittals to other regulatory agencies.

Section 10 Submittal Requirements provides City of Dallas requirements regarding construction plans, as well as helpful information to assist the design engineer and expedite the plan review process.

Section 11 Operation and Maintenance includes consideration and parameters for maintenance of stormwater infrastructure

Appendices contain applicable regulations, charts and tables to assist with computations, a list of reference documents, a list of acronyms and definitions, guidelines & procedures, and checklists & forms.

A list of design resources and approved software can be found on the City of Dallas website.

1.4

INTEGRATING DRAINAGE

1.4.1 Drainage Design Process

Drainage should be considered at the beginning of any design process. The following general process should be used for drainage design for a site. City capital projects must follow same process as private developments.

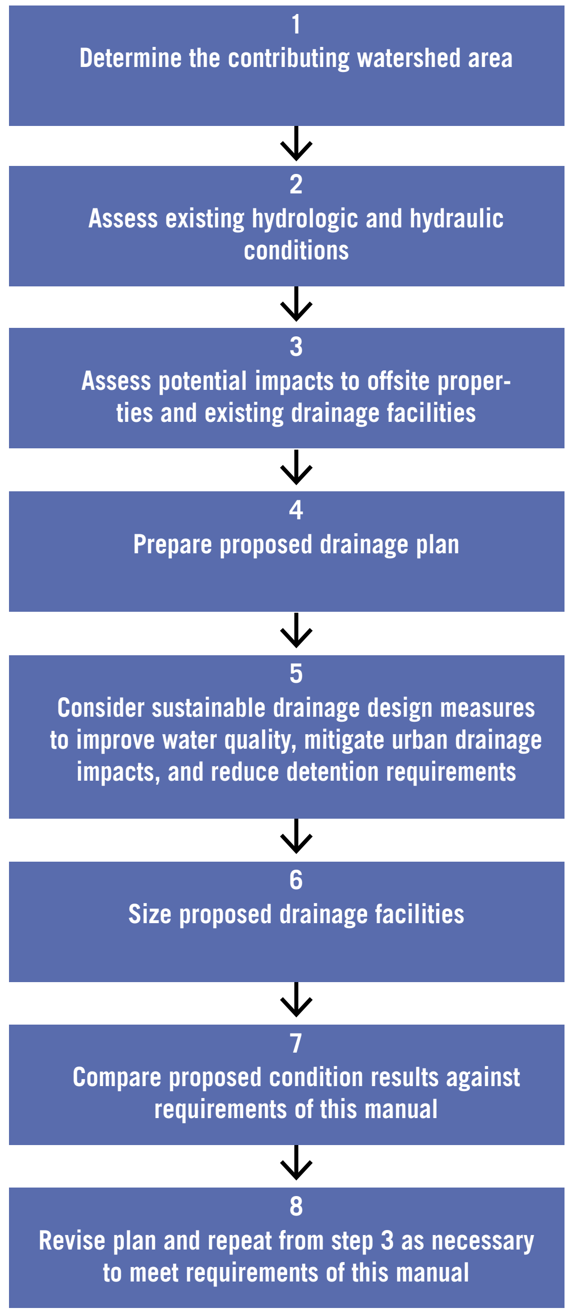
1.4.2 Drainage Design Considerations

A significant percentage of developed land within the City of Dallas is comprised of impervious area, including buildings, roads and sidewalks. The increased impervious area in the City impacts natural drainage and contributes to increased rates and volumes of runoff and processes such as infiltration, evapotranspiration, and groundwater recharge. Unmitigated development can lead to increased flooding, system exceedance, channel and bank erosion, and stormwater pollution. The City of Dallas encourages that both water quantity and quality be taken into account for any design both during and after construction. Construction plans must consider erosion control and have procedures in place to prevent stormwater pollution. Permits shall be acquired in accordance with federal, state, and local regulations.

Water quality and quantity is also important for street design. The City of Dallas encourages a holistic approach to street design, integrating the elements of multi-modal transportation, drainage, walkability, and aesthetics. Refer to the Street Process and Street Design Manuals for guidance on the street design process. By managing stormwater with sustainable drainage measures before it enters the conveyance system, positive water quantity and quality impacts can be achieved, especially for smaller storms. Additional benefits include reducing the heat island effect, improving air quality, and increasing the value and aesthetics of property.

Dallas is an urban environment, therefore aesthetic design must be considered. The City of Dallas encourages the preservation of natural channels and floodplains, tree canopy buffers, and natural drainage patterns where possible. Structural components should be designed to integrate with the project surroundings.

Figure 1.1 Drainage Design Process



1.4.3 Defining a Project

When considering the hydrologic impacts of a project, the entire contributing drainage area, adjacent property, and upstream and downstream conditions must be taken into account. The project size refers to the size of the entire developed project, not the individual phases of the project. The cumulative effects of a project with multiple phases should be taken into consideration.

1.4.4 Sustainable Design Guidelines

The methods and policies outlined in this document are the minimum requirements for drainage for the development of a site. The City of Dallas encourages innovation, sustainable drainage practices, and project review relative to more sustainable design by utilizing project rating systems such as LEED and Envision. Drainage requirements within these programs should be coordinated with other street and facility plans to meet the minimum requirements described in this manual. In addition to contributing towards greater community resilience, these guidelines are also requirements of the City's municipal separate storm sewer system permit under Texas Pollutant Discharge Elimination System (TPDES) and our participation in the FEMA CRS program.

1.4.5 Pre-Development Meeting

A pre-development meeting is encouraged for all projects within the City of Dallas. It is valuable to spend time in discussion with the City to address any concerns at the beginning of a project, verify an understanding of design criteria as it applies to the project, and discuss any unusual elements of the project. A pre-development meeting shall be required if any of the conditions below apply:

- The project is private and has a size greater than 5 acres
- The project site is within 1/4 mile of a defined sump area (refer to City of Dallas website for map of sump locations)
- The project site is within 100 feet of a stream
- The project lies within a floodplain according to the current floodplain ordinance. Consult Article V for floodplain application meeting requirements.
- The project is within the Escarpment Zone or a geologically similar area
- The downstream system does not have adequate capacity
- Innovative design approaches are being used

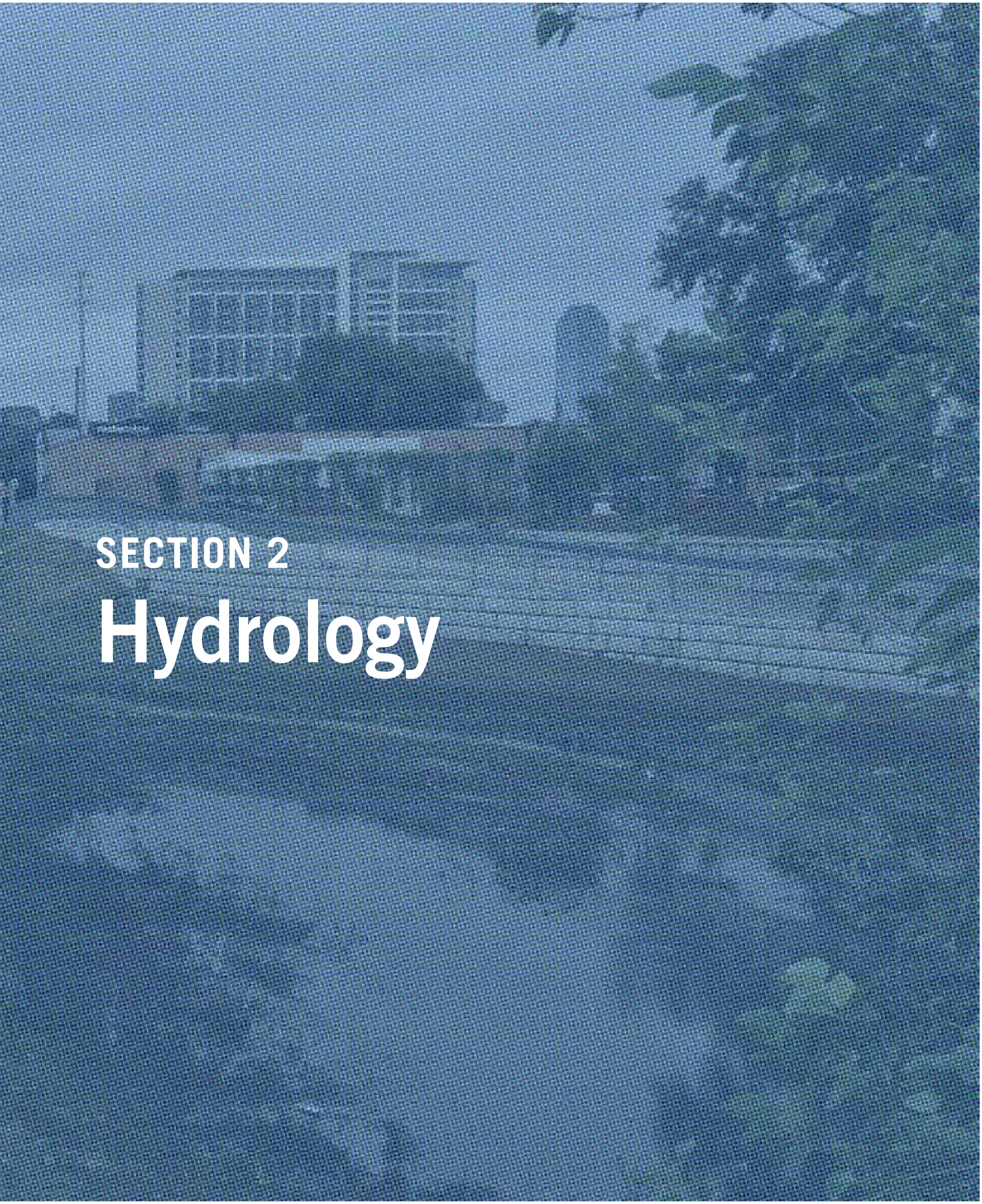
If a pre-development meeting is required or requested, the following items, at a minimum, must be reviewed and prepared by the designer prior to the meeting. Other items may be necessary depending on site and project conditions.

- Topographic workmap
- Conceptual layout
- Existing flood studies or models
- FEMA flood maps if applicable
- Upstream Drainage Area Delineation
- Upstream and downstream conditions and capacity

See Section 10 for project submittal requirements.

1.4.6 Design Software

Software that facilitates the design of drainage facilities can be found on the City of Dallas website. Depending on the analysis being performed by the design engineer, certain applications may require the use of specific software and modeling techniques. This should be discussed in the pre-development meeting.



SECTION 2
Hydrology

2.1

GENERAL

This section contains the minimum hydrologic design criteria to be followed in the design of storm drainage facilities and demonstrates the design procedures to be used on drainage projects in the City of Dallas.

All new systems will be designed to accommodate the 1% annual chance storm event or flood of record. See Table 2.1 for a comparison of annual chance storm events to annual recurrence intervals. For rainfall, this is also known as the annual exceedance probability (AEP).

Table 2.1 Probability of Exceedance

Annual Chance (as percent)	Annual Recurrence Interval
50%	2-year
10%	10-year
2%	50-year
1%	100-year
0.2%	500-year

See Table 2.2 for a summary of design storms for drainage elements.

Table 2.2 Design Storms for Drainage Elements

Design Element	Annual Chance Design Storm	Design Standard
Detention/ Retention	1%, 2%, 10%, 50%	No increase in peak discharge No increase in erosive velocity
Culverts	1%	Convey with a minimum of 2 feet of freeboard between top of curb and HGL
Bridges	1%	Convey with a minimum of 2 feet of freeboard between low chord of bridge and HGL
Open Channels	1%	Convey with a minimum of 2 feet of freeboard below top of bank, or in spreading creek or river areas, convey with 2 feet of freeboard below top of conveyance section covered by easement or regulated floodplain No increase in erosive velocity

2.2

HYDROLOGY

The Rational Method for computing storm water runoff design rates and volumes can be used for hydrologic analysis of facilities when the total contributing drainage area is less than 100 acres. For all other drainage areas, runoff is to be calculated using the Soil Conservation Service (SCS) Unit Hydrograph Method, the preferred method for design.

Coordination with the Director is required for the use of other methods, including Snyder's method. If modifying an existing regulatory model performed using a different methodology, continued use of that methodology may be allowed as approved by the Director.

2.2.1 Rational Formula

The Rational Formula for computing peak runoff rates is as follows:

$$Q = CIA \quad (Equation\ 2.1)$$

Q = Runoff rate (cfs)

C = Runoff coefficient

I = Rainfall intensity (in/hr)

A = Drainage area (ac)

2.2.1.1 Runoff "C" Value

Runoff coefficients for existing and proposed conditions are shown in Table 2.3 and 2.4. Existing land use shall be used for determining existing conditions. All off-site runoff calculations shall be based upon a fully developed watershed and proposed zoning. Larger coefficients may be used if considered appropriate by the Director. See Section 2.3.3 for potential reduction to these factors if the drainage plan incorporates sustainable drainage design measures.

Table 2.3 Runoff Coefficients for Zoned Areas

Zone	Zoning District Name	Runoff Coefficient
A (A)	Agriculture	0.30
R – 1ac (A)	Residential	0.45
R – 1/2ac (A)	Residential	0.45
R – 16 (A)	Residential	0.55
R – 13 (A)	Residential	0.55
R – 10 (A)	Residential	0.65
R – 7.5 (A)	Residential	0.70
R – 5 (A)	Residential	0.70
D (A)	Duplex	0.70
TH – 1 (A)	Townhouse	0.80
TH – 2 (A)	Townhouse	0.80
TH -3 (A)	Townhouse	0.80
CH	Clustered Housing	0.80
MF – 1 (A), MF - 2 (A), MF - 3 (A), MF - 4 (A)	Multifamily Residential	0.80
MH (A)	Mobile Home	0.55
NO (A)	Neighborhood Office	0.80
LO – 1, LO - 2, LO - 3	Limited Office	0.90
MO – 1, MO - 2	Midrange Office	0.90
GO (A)	General Office	0.90
NS (A)	Neighborhood Services	0.90
CR	Community Retail	0.90
RR	Regional Retail	0.90
CS	Commercial Service	0.90
LI	Light Industrial	0.90
MU	Industrial Research	0.90
IM	Industrial Manufacturing	0.90
CA – 1 (A), CA - 2 (A)	Central Area	0.95
MU – 1	Mixed Use - 1	0.80
MU – 2	Mixed Use - 2	0.80
MU - 3	Mixed Use - 3	0.90
MC - 1, MC - 2, MC - 3, MC - 4	Multiple Commercial	0.90
UC - 1, UC - 2, UC - 3	Urban Corridor	0.90
P (A)	Parking	0.95

Table 2.4 Runoff Coefficients for Non-Zoned Areas

Land Use	Runoff Coefficient "C"
Church	0.8
School	0.7
Park	0.4
Cemetery	0.4

For shared access development, a composite value based on pervious and impervious cover must be calculated.

The C value estimates may be adjusted to account for the impact of sustainable drainage measures on peak discharge rates for Rational Method calculations. C value adjustment factors should only be used for determining runoff for the 50% annual chance storm event, as sustainable drainage measures are only typically used to reduce runoff for smaller storm events and will usually have minimal effect on larger storms.

Rain Gardens & Bioretention

If the design water quality volume to be treated by the measure is determined and it is designed to treat 100% of the design water quality volume, the following equation can be used to determine an adjusted C factor (C_{adj}) for the 50% annual chance and smaller events.

$$C_{adj} = C * \left(1 - \frac{1}{2} P_{WQ}\right) \quad (Equation\ 2.2)$$

C_{adj} = Adjusted proposed runoff coefficient

C = Proposed runoff coefficient without consideration of sustainable drainage measures

P_{WQ} = Design water quality precipitation (1.0 - 1.5 inches)

See example in Appendix A.2.

Permeable Pavement & Pavers

For permeable pavement and pavers, the C value can be adjusted by treating the area of the permeable pavement as park area (C value of 0.4) in rational method calculations, for the 50% annual chance and smaller storm events only.

2.2.1.2 Rainfall Intensity (i)

The rainfall intensity value "i" is the intensity for a duration equal to the time of concentration (T_c). Rainfall intensities can be found online using National Oceanic and Atmospheric Administration's (NOAA) National Weather Service (NWS) Precipitation Frequency Data Server (PFDS).

2.2.1.3 Time of Concentration (T_C)

Design rainfall intensities are dependent upon the time of concentration contributing to the drainage facility.

The definition for time of concentration (T_c) considers the geometry, geology, soil type, land use, topography and other hydraulic characteristics of a watershed. T_c is defined as the time required for runoff to travel from the furthest point, hydraulically, within the delineated area (or watershed) to the outlet of the drainage area (or watershed).

In order to provide a uniform and versatile equation which can be applied to most situations encountered in the City of Dallas, the T_c shall be considered in four parts: 1) sheet flow; 2) shallow concentrated flow; 3) pipe flow; and 4) channel flow.

Combining time of concentration for sheet flow, shallow concentrated flow, pipe flow, and channel flow yields the following equation:

$$T_c = T_{cs} + T_{csc} + T_{cp} + T_{cc} \quad (Equation\ 2.3)$$

T_c = Time of concentration (hr)

T_{cs} = Time of concentration representing sheet flow (hr)

T_{csc} = Time of concentration representing shallow concentrated flow (hr)

T_{cp} = Time of the concentration representing pipe flow (hr)

T_{cc} = Time of concentration representing stream or channel flow (hr)

Detailed discussion of each component is provided in the following paragraphs. If the computed T_c is less than 10 minutes, then a minimum T_c of 10 minutes shall be used. This condition occurs often, though not always, when computing the T_c for an inlet in a residential or commercial area.

Sheet Flow

Sheet flow is flow over a planar surface with a depth up to 0.1 feet. The following equation can be used to estimate the sheet flow travel time. Ordinarily, sheet flow will occur for a distance of 50 to 100 feet. For sheet flow across impervious surfaces, the maximum sheet flow length shall be 50 feet.

$$T_{cs} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S_0)^{0.4}} \quad (Equation\ 2.4)$$

T_{cs} = Time of concentration representing sheet flow (hr)

n = Manning's roughness coefficient (reference Table 2.5)

L = Flow length (ft) (≤ 100 ft)

P_2 = 2-year (50% AEP) 24-hour rainfall (in)

S_0 = Slope of land surface (ft/ft)

Table 2.5 includes Manning's n values, or roughness coefficients, for sheet flow. These 'n' values are higher than typical open channel flow values to account for the impact of raindrops and drag forces on flow over a plane with depths less than 0.1 feet.

Table 2.5 Roughness Coefficients (Manning's n) for Sheet Flow

Surface Description	n[1]
Smooth surfaces	
(concrete, asphalt, gravel, or bare soil)	0.011
Fallow	
(no residue)	0.05
Cultivated soils:	
Residue cover < 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range	
Natural	0.13
Woods[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values composite information by Engman (1986)

[2] Includes species such as bluestem grass, buffalo grass, grama grass, and native grass mixtures

[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Shallow Concentrated Flow

The equation for shallow concentrated flow travel time is as follows:

$$T_{csc} = \frac{L_{sc}}{3600V_{sc}} \quad \textit{(Equation 2.5)}$$

T_{csc} = Time of concentration representing shallow concentrated flow (hr)

L_{sc} = Length of shallow concentrated flow (ft)

V_{sc} = Average velocity (ft/s)

These equations can be used to determine average velocities, including for slopes less than 0.005 ft/ft.

$$\textbf{\textit{Unpaved}} \quad V_{SC} = 16.13\,(S)^{0.5} \quad \textit{(Equation 2.6)}$$

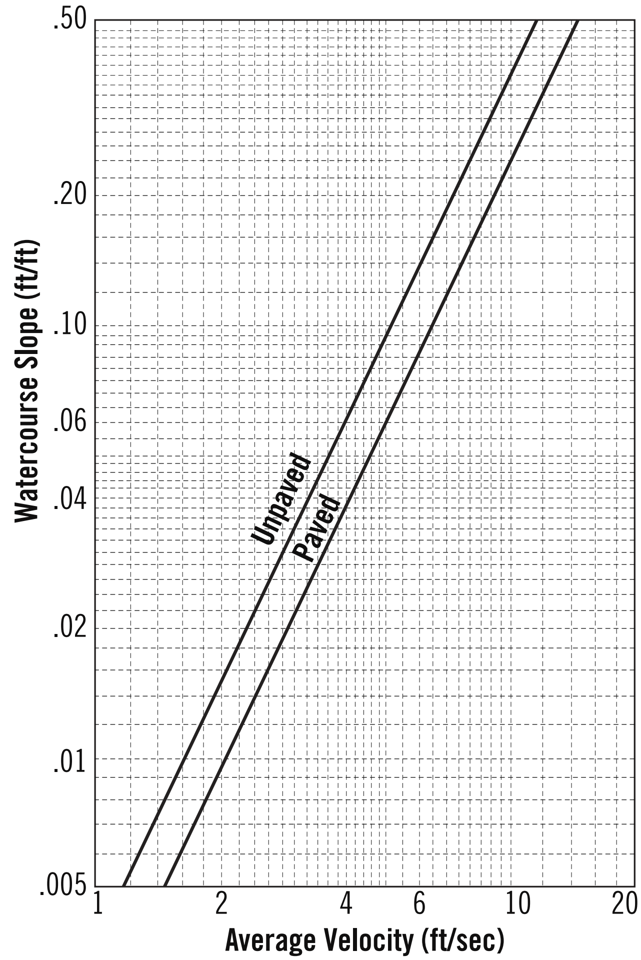
$$\textbf{\textit{Paved}} \quad V_{SC} = 20.33\,(S)^{0.5} \quad \textit{(Equation 2.7)}$$

V_{sc} = Average velocity for shallow concentrated flow (ft/s)

S = Slope of land surface (ft/ft)

The nomograph provided in Figure 2.1 may be used to estimate shallow concentrated flow velocity as it is a graphical representation of the unpaved and paved equations provided.

Figure 2.1 Shallow Concentrated Flow Average Velocity



(see enlarged figure in Appendix A.7)

The equation for travel time in a pipe is as follows:

$$T_{cp} = \frac{L_p}{3600V_p} \quad \textit{(Equation 2.8)}$$

T_{cp} = Time of concentration representing pipe flow (hr) (for simplification purposes, a full flow condition may be assumed in the pipe)

L_p = Length of pipe (ft)

V_p = Average velocity of the design discharge in the pipe, calculated per Manning's equation (ft/s)

Refer to Appendix A.3 for pipe 'n' values.

Channel Flow

The equation related to channel flow relates velocity, length, and time:

$$T_{cc} = \frac{L_c}{3600V_c} \quad (Equation\ 2.8)$$

T_{cc} = Time of concentration representing stream or channel flow (hr)

L_c = Effective hydraulic length of ditch or channel (ft)

V_c = Average velocity of ditch or channel flow (ft/s)

The velocity (V_c) in the channel can be obtained directly from the existing hydraulic model. When velocity results are not available for a reach, channel velocity can be estimated using Manning's formula:

$$V_c = (1.49/n)\,(R)^{2/3}\,(S)^{1/2} \quad (Equation\ 2.10)$$

V_c = Velocity in channel (ft/s)

n = Roughness coefficient (see Appendix A.3)

R = Hydraulic radius (ft) equal to A / P_W

A = Cross sectional flow area (ft^2)

P_w = Wetted perimeter (ft)

S = Slope of the hydraulic grade line (ft/ft)

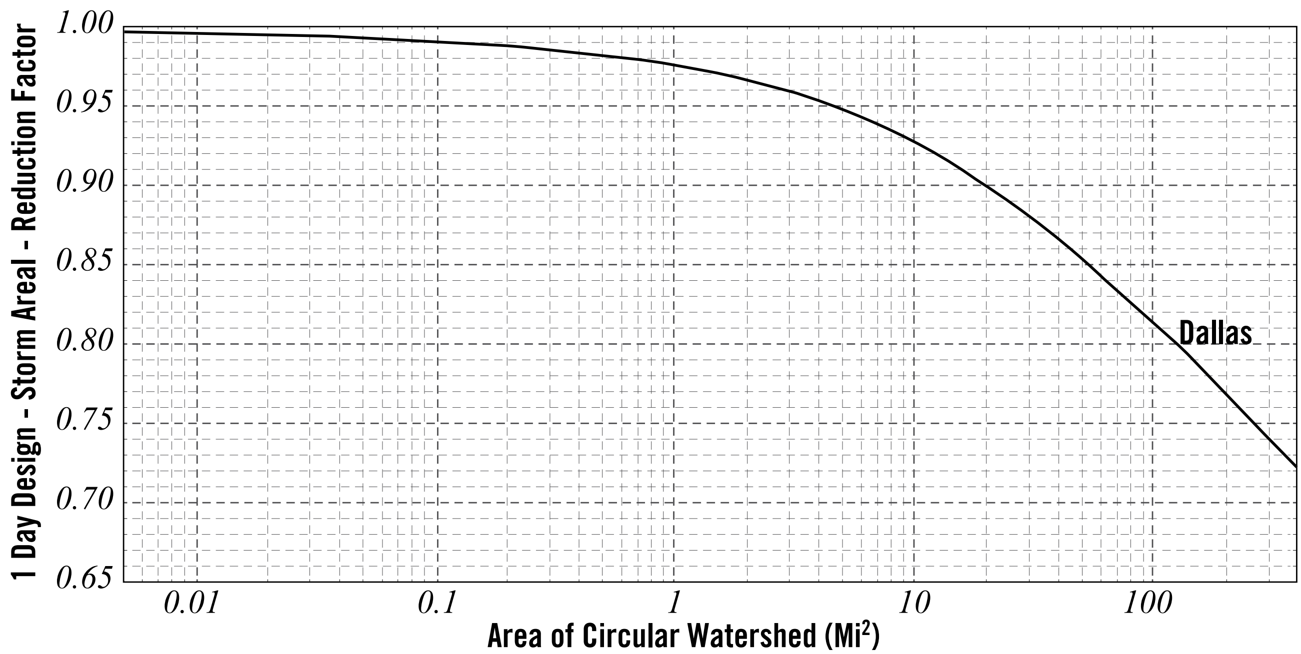
2.2.2 Hydrograph Methods

2.2.2.1 SCS Unit Hydrograph Method

When a drainage area under consideration is greater than or equal to 100 acres, the City of Dallas recognizes the Soil Conservation Service (SCS) Unit Hydrograph Method for determining rates and volumes of stormwater runoff. The City recognizes the use of approved software (a list of which can be found on the City of Dallas website.) The City of Dallas maintains hydrologic models on most primary and some secondary channels in Dallas. Where the City has an existing hydrologic model, it should be utilized unless the Director approves development of a new model. Models that are currently in use in an area should be utilized for future projects. Any variances must be approved by the Director.

A unit hydrograph is the time discharge relationship, defined at a given point along a water course, which results from a storm producing one inch of runoff uniformly distributed over the watershed. The SCS Unit Hydrograph Method computes the unit hydrograph for a given area and then converts it to the flood hydrograph theoretically resulting from a given rainfall event. The input required to compute the design flood hydrograph includes drainage area, lag time, runoff curve number, rainfall information for the design storm, and a dimensionless unit hydrograph.

Figure 2.2 Areal Reduction Factors for Small Watersheds for 50% AEP or Greater 1-day Design Storms



(see enlarged figure in Appendix A.7)

Rainfall

24-hour rainfall depths can be found online using NOAA's National Weather Service Precipitation Frequency Data Server. Areal Reduction Factors may be used to account for non-uniform distribution of rainfall in large watersheds. See Figure 2.2 for Dallas adjustment factors. Information pertaining to design storm duration and distribution will be provided by the Director where the City has an existing hydrology model. Where new models are being developed, SCS Type III 24-hour rainfall distribution will be used.

Drainage Area Delineation

Drainage area delineations must utilize the most accurate and recent ground data available for the area. The entire watershed draining to a property must be considered, not just the area within the project or property boundaries. The area of consideration shall be to the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 10 acres, the area of consideration ends at the point where the total drainage area is 100 acres or greater. The drainage area delineation must also contain any contributing storm sewer systems.

In some instances, drainage area subdivision may be appropriate. Examples of such scenarios include but are not limited to:

- Drainage areas with significantly varying C values / curve numbers
- Significantly different slope patterns
- A detention facility within a contributing area
- Intermediate points of interest for facility design
- Stream confluences
- Infrastructure such as dams and levees

Subdivision of drainage areas will require detailed routing in order to incorporate the effects of travel time and floodplain storage between the subdivided basins. The following sections include approved hydrograph routing methods.

Runoff Curve Number

Rainfall runoff potential, presented in the form of a runoff curve number (CN), is an expression of the runoff potential for a given land use and the runoff potential of the underlying soil. Higher CN values correlate to higher runoff potential. The National Engineering Handbook by the National Resource Conservation Service (NRCS, formerly Soil Conservation Service, SCS) provides the definitions for four hydrologic soil groups, based upon hydrologic conductivity characteristics of the soil (NRCS. 2009. Part 630, National Engineering Handbook, Chapter 7, Hydrologic Soil Groups, (210-VI-NEH)). These four groups are generally described as follows:

Group A Soils: generally consist of deep sands and other soils that have a low runoff potential when thoroughly wet, due to high hydraulic conductivity.

Group B Soils: generally consist of soils with moderately low runoff potential when thoroughly wet.

Group C Soils: generally have a moderately high runoff potential when thoroughly wet, because water transmission through the soil is somewhat restricted.

Group D Soils: generally have a high runoff potential when thoroughly wet.

Surficial soils data can be obtained from County Soil Maps prepared by the NRCS, however, NRCS soil data should not be substituted for geotechnical data for drainage infrastructure design. Additionally, hydrologic soil groups for borrow material, mixed and or disturbed site soils shall be determined through appropriate geotechnical field investigation. Curve numbers associated with hydrologic soil group and cover type are provided in Table 2.6.

Equation 2.11 can be used to estimate direct runoff from a 24-hour storm rainfall.

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \quad \textit{(Equation 2.11)}$$

$$S = \frac{1000}{CN-10} \quad \textit{(Equation 2.12)}$$

Q = Accumulated direct runoff (in)

P = Design precipitation (in)

CN = Runoff curve number

S = Maximum potential retention (in)

The values in Table 2.6 are based on average percent impervious area for different land uses. Composite curve numbers can be developed for drainage areas with multiple land uses. They can be developed in two ways:

1. An open space curve number can be applied for the entire basin area along with a percent impervious area. The composite curve number can be computed by hand.
2. A weighted curve number is developed for each land use based on the percentage of total land area that it encompasses. See Table 2.7 for the calculation of a composite curve number.

Table 2.6 Runoff Curve Numbers[1]

Cover Description	Avg % impervious area[2]	Curve numbers for hydrologic soil groups A	B	C	D
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
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; 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
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by avg lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
¼ acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
½ acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

[1] Average runoff conditions and initial abstraction = 0.2S

[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. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

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

Table 2.7 Composite Curve Number Example

Land Use	Fraction of Total Land Area	Curve Number	Weighted Curve Number (Fraction area x CN)
Residential 1/8 acre Soil Group B	0.80	85	68
Meadow Good condition Soil Group C	0.20	71	14
Total Composite Curve Number = 68 + 14 = 82			

The curve numbers in Table 2.8 can be used for applicable sustainable drainage measures for the 50% annual chance storm event only.

Table 2.8 Sustainable Drainage Measure Curve Numbers

Practice/Land Type	Curve Number
Permeable Paving, Permeable Unit Pavement System, Porous Pavers	40
Rain Gardens/Bioretention	35

Lag Time

Lag time is a required input for the SCS unit hydrograph method. Lag time is defined as difference in time between the peak of the rainfall event and the peak discharge of the resulting hydrograph. Lag time can be determined by the equation below.

$$T_{lag} = 0.6\,T_c \qquad (Equation\ 2.13)$$

T_{lag} = Lag time (hr)

T_c = Time of concentration (hr)

Discussion of methodology to estimate the time of concentration for a basin is provided in Section 2.2.1.3. If a recorded flood hydrograph is available, the lag time may be estimated from the hydrograph parameters rather being derived empirically.

2.2.2.2 Use of Other Hydrograph Methods

There may be the need for the use of the Snyder Unit Hydrograph method or the NUDALLAS method on a case by case basis, dependent on the methods used in the existing models.

2.2.2.3 Flood Routing

Flood routing is an iterative process that determines flood wave (flow) travel time and attenuation using hydrologic or hydraulic routing methods and is normally required when drainage areas are subdivided. Flood routing can be determined using approved hydrologic software, a list of which can be found on the City of Dallas website. Applicability and limitations for each of the methods indicated in the computer program's technical reference manual should be considered. The Modified Puls method is the City-preferred flood routing method and shall be used for all streams located within a FEMA-designated Zone AE floodplain. The Modified Puls method must also be used for any stream/floodplain where significant backwater is expected or significant overbank storage exists. Other approved methods suitable for streams where the Modified Puls is not required are also discussed in the following subsections. Methods not listed below may only be used with approval of the Director.

Channel Routing

Approved hydrologic channel routing methods include the Modified Puls, Muskingum-Cunge, Lag, and Kinematic Wave methods. See the descriptions below for guidance on appropriate use of methods. Refer to HEC-HMS and HEC-RAS user manuals for full description of methodology.

Modified Puls Method

This method requires detailed hydraulic modeling (HEC-RAS) to compute a storage-outflow relationship. Additionally, an input of routing steps or subreaches will need to be determined. The number of routing steps (n) can be computed by the following:

$$n = \frac{L / V_w}{\Delta t} \quad \textit{(Equation 2.17)}$$

n = Number of routing steps (dimensionless)

L = Reach length (ft)

V_w = Average flood wave velocity (ft/s)

Δt = Hydrograph time interval (min)

Alternatively, travel time can be pulled directly from HEC-RAS to replace L/V_w

To run a Modified Puls model, a steady flow model must be created first in HEC-RAS, with several different flow profiles ranging from low to higher than expected flows to create a rating curve. See the HEC-RAS User's Manual for guidance on how to run a Modified Puls model.

A storage-discharge table can be created for each reach, showing the storage loss associated with each flow profile for every reach that can be used as input data to the HMS model to properly model storage loss as runoff is routed through a reach.

Muskingum-Cunge Method

The Muskingum-Cunge method provides a simplified estimation of floodplain storage to reflect hydrograph attenuation for locations where detailed hydraulic modeling is not available for use in the Modified Puls method. The standard channel configuration may be used for man-made channels with simple prismatic cross-sections. The 8-point cross section method should be used for natural channels. Refer to HEC-HMS Technical Reference manual for guidance on this method.

Lag Method

This method may only be used if the following condition is met:

$$\frac{L}{V} < 2\,\Delta\,t \quad \textit{(Equation 2.18)}$$

L = Reach length (ft)

V = Channel velocity (ft/s)

Δt = Hydrograph time interval (min)

It should be noted that the Lag method is most appropriate for short reaches on small streams and does not account for flood volume attenuation. The Lag method simply translates the inflow hydrograph in time to account for the time it takes to travel through the reach.

Kinematic Wave Method

This method is a simplified hydraulic routing technique that can be computed using HEC-HMS, SWMM, or other approved hydrologic software. It is appropriate for channels that have been modified to have a regular shape and uniform slope.

Reservoir Routing

Hydrograph routing through a reservoir can be computed with the Modified Puls method. A detailed description of level-pool routing can be found in the HEC-HMS Technical Reference manual and other manuals from the approved software list found on the City of Dallas website. A storage-outflow relationship is required for this routing method. The storage-outflow relationship is dependent upon the characteristics of the storage facility and the outlet and spillway. For simple outflow structures, outlet configurations can be entered directly into most hydrologic modeling software. Data from a hydraulic program such as HEC-RAS can be used to run multiple steady-state water surface profiles to determine outflow values for corresponding storage volumes/elevations for complex outlet structures.

2.2.2.4 Hydrograph Time Interval Determination

For an SCS Unit Hydrograph model, computational time intervals must be less than 29% of the lag time.

2.2.2.5 Gage Data and Calibration Methods

When United States Geological Survey (USGS) gage data is available at a site, a statistical analysis of the data can be performed to determine hydrologic design information. If the site is near a gage station on the same stream and has similar hydrologic conditions, the data can be directly applied in a flood frequency analysis. If the site is on the same stream, but not near the gage station, an in-depth statistical analysis, described in the Texas Department of Transportation (TxDOT) Hydraulic Manual, can be used in the flood frequency analysis.

Refer to the City of Dallas website for a list of gage stations in the Dallas area. If applicable, gage data from these sites may be used in a stream flow analysis. Each of these gages is affected by regulation or diversion, so consideration must be given to upstream conditions when streamflow data is used for analysis.

If gage data is available for the site, models should be calibrated to best fit the observed data. The HEC-HMS manual provides detailed explanations of calibration parameters, which include discharge, pool elevation, and cumulative precipitation.

2.2.2.6 Storage Options

Storage within a drainage basin that acts to change the shape and peak value of a hydrograph is not always associated with a formalized detention facility. Natural storage from small depressions can impact peak runoff rates. This type of storage can be accounted for within unit hydrograph procedures as described in the HEC-HMS Technical Reference manual. Storage within most urban floodplains can be modeled with the Modified Puls routing method. Storage within a drainage basin is sometimes a result of undersized storm drain systems or a lack of adequate natural conveyance capacity within the watershed. The impact of this storage can be significant. Adequate assessment of the impact of this storage may necessitate the use of a 2-dimensional hydrodynamic model such as SWMM or InfoWorks. The use of a 2D hydrodynamic model is not only necessary in some cases to adequately assess existing conditions discharges, it can be critical to understanding the impact that stormwater conveyance improvement projects (which tend to eliminate these storage areas) can have upon downstream peak discharges. A list of approved 2D hydrodynamic modeling software can be found on the City of Dallas website. Pre-design meeting discussions of whether this type of modeling is appropriate on a specific project are encouraged.

Floodplain valley storage can also impact routing and the shape of hydrographs. Loss of valley storage can have an adverse downstream impact. Refer to Article V Section 51A-5.100 for valley storage requirements.

2.3 DETENTION / RETENTION ANALYSIS

2.3.1 Design Objectives

Stormwater detention is sometimes required to temporarily impound (detain) excess storm water, thereby reducing peak discharge rates.

The purpose of detention is to impound water for a certain period of time before it drains or overflows into a storm sewer system or channel. Detention ponds can be designed to remain dry in between storm events (Dry Detention pond) or maintain a permanent wet pool for aesthetics or recreation (Wet Detention pond). Retention ponds impound water while allowing infiltration through porous soils so that all runoff is retained and no runoff is passed downstream.

For all projects, runoff must be detained to existing levels with no increase in peak discharge and no increase in erosive velocity downstream of the delineated project site for the 1%, 2%, 10%, and 50% annual chance storm events. An increase in discharge or erosive velocity is considered not to occur when the following conditions are met:

1. When a downstream analysis (See Section 2.3.2) has not been performed:
    a. Zero increase for the 1%, 2%, 10%, and 50% annual chance discharges
2. When a downstream analysis (See Section 2.3.2) has been performed:
    a. Zero increase for the 1%, 2%, and 10% annual chance discharges
3. Channel velocities do not exceed the permissible maximum velocity at any location within the downstream assessment for the 1%, 2%, 10%, or 50% annual chance events. Exceptions to these criteria require certified geotechnical / geomorphic studies that provide documentation that the higher velocities will not cause erosion.

Compliance with the threshold conditions used to determine whether a project creates a significant impact on downstream discharges and velocities does not relieve the project from compliance with floodplain regulations, Article V Section 51A-5.100-5.105.

Infill developments and redevelopments are exempt from detention requirements if the parcel is less than 1 acre in size and adds less than 5,000 square feet of additional impervious surface relative to existing conditions. However, the use of rain gardens, pervious pavements and other sustainable design measures are encouraged to minimize cumulative impacts from this type of development on a neighborhood scale.

For roadway projects, determination of whether detention is required shall be assessed for each roadway outfall. Roadway projects are exempt from detention requirements at an outfall if they add less than 10,000 square feet of additional impervious area draining to that outfall.

All detention requires an easement. For plat regulations and easement requirements, see Section 9.3.

2.3.2 Downstream Analysis

A downstream analysis is a tool for evaluating the impact that a project may have on downstream discharges, water surface elevations, and velocities. The assessment must extend downstream of a project outfall until the drainage area tributary to the project outfall comprises less than 10% of the downstream assessment area.

2.3.3 Design Methodology

Detention Storage – Basins without upstream detention areas and with drainage areas of 100 acres or less can be designed using the Modified Rational Method. This method estimates peak rates using the Rational Formula and storage requirements using inflow minus outflow hydrograph volume at the time of peak outflow. Basins with drainage areas greater than 100 acres or where the Modified Rational Method is not applicable are to be designed using the Unit Hydrograph Method and approved software, a list of which can be found on the City of Dallas website. Detention design that meets the requirements for multiple design events (1%, 2%, 10%, and 50% annual chance) often requires complex configurations for the outlet structure.

2.3.3.1 Modified Rational Method

The following steps are used to determine the minimum detention volume required for each design event:

Step 1

Determine the 1% annual chance event peak runoff rate on the site with existing conditions, which will be the maximum allowable release rate (QA). The intensity for the 1% annual exceedance probability (AEP) event can be found online using NOAA's National Weather Service Precipitation Frequency Data Server.

$I_{1\%}$ = Rainfall intensity for the 1% AEP storm event (in/hr)

T_c = Time of concentration (See Section 2.2.1.3) (min)

$$Q_{1\%} = C * I_{1\%} * A \quad (Equation\ 2.19)$$

$Q_{1\%}$ = Peak runoff for the 1% annual chance storm event (cfs)

C = Runoff coefficient

A = Contributing drainage area (acres)

Step 2

Determine inflow hydrographs for storms of multiple durations to determine the maximum volume required. Use 10-minute time increments and determine inflow rates. The last increment will be when the peak inflow is less than the release rate found in Step 1.

For each time increment, determine the intensity (I) and peak flow (Q) using the following equation.

$$Q_i = K * C_{prop} * I * A \quad (Equation\ 2.20)$$

I = Rainfall intensity (in/hr) (from online PFDS)

T_i = Time step duration (min)

Q_i = Rate of discharge for the time step duration (cfs)

K = 1.0 for 10% AEP storm event or more frequent storm event; 1.15 for 1% AEP storm event or less frequent storm event. Application of the 'K' factor provides a factor of safety to account for the inherent uncertainty associated with Modified Rational Method volume assessments.

C_{prop} = Proposed runoff coefficient

Step 3

Determine the storage volume required (V_{req}) for each time step duration using the following equations.

$$V_{req} = V_{in} - V_{out} \quad (Equation\ 2.21)$$

$$V_{in} = T_i * Q_i * 60\ sec/min \quad (Equation\ 2.22)$$

$$V_{out} = 0.5 * (T_i + T_c) * Q_A * 60\ sec/min \ (Equation\ 2.23)$$

V_{req} = Required storage volume (ft^3)

V_{in} = Volume in (ft^3)

V_{out} = Volume out (ft^3)

T_i = Time step duration (min)

Q_i = Rate of discharge for the time step duration (cfs)

T_c = Time of concentration in the basin (min)

Q_A = Maximum allowable release rate (cfs)

Step 4

Determine the storm duration that produces the largest amount of storage required. This volume will be the design storage, with the maximum release rate calculated in Step 1.

2.3.3.2 Adjustment for Sustainable Drainage Measures

Purpose

Sustainable drainage measures can have a significant impact on the overall hydrology of a site and are of most benefit when used close to the point source of runoff and distributed uniformly rather than centralized. When implemented at a high level, sustainable drainage measures can have a noticeable effect on smaller storms, by reducing peak runoff and detaining stormwater for a short period of time. The hydrologic impact of sustainable drainage measures can be calculated by adjusting curve numbers and storage volumes.

There is no place in Dallas where sustainable drainage design cannot be considered, and it is encouraged relative to compliance with United States Environmental Protection Agency (USEPA), Texas Commission on Environmental Quality (TCEQ), and FEMA regulatory requirements. Appropriate geochemical and geotechnical testing should be implemented as a part of the design process to determine appropriate design considerations for your particular location. Design processes for ponds are included in Section 6 of this Manual.

Water Quality Volume

The first 1.5 inches of rainfall, which is the average depth of the 85th percentile storm for North Texas, is considered to be the water quality protection volume. The first flush of a storm event contains the highest load of pollutants and sediment; therefore treating the first 1.5 inches of stormwater runoff can provide significant water quality benefits. Although recommended, not all sites may have the capability of treating the volume of runoff from the first 1.5 inches of rainfall. A minimum of 1 inch of rainfall must be treated if a sustainable drainage measure is used. A design water quality precipitation between 1.0 – 1.5 inches of rainfall can be chosen to determine the design water quality volume to be treated by the sustainable drainage measure.

The water quality protection volume can be determined using the following methods. The proposed runoff coefficient "C" can be determined from Table 2.3 and Table 2.4.

Using a range from 1.0 - 1.5 inches as the design precipitation for water quality in North Central Texas, the design water quality capture depth in inches (D_{WQ}) can be calculated using the following formula:

$$D_{WQ} = P_{WQ} * C \quad (Equation\ 2.24)$$

D_{WQ} = Design water quality capture depth (in)

P_{WQ} = Design water quality precipitation (1.0 – 1.5 in)

C = Proposed runoff coefficient

The design water quality volume in cubic feet (V_{WQ}) can be calculated using the following formula

$$V_{WQ} = D_{WQ} * A * (1\ ft/12\ in) \quad (Equation\ 2.25)$$

V_{WQ} = Design water quality volume (ft^3)

A = Contributing drainage area (ft^2)

Many sustainable drainage measures can be sized based on a design water quality volume. Some measures are designed based on a flow capacity. Discussions of appropriate design approach for various sustainable drainage measures are provided in Section 3 and Section 6.

Adjustment for Detention

If an adjusted C factor is calculated based on the implementation of sustainable drainage measures from Section 2.3.3.2 which yields project runoff using the rational method that does not result in an increase in runoff for the 50% annual chance relative to existing conditions, then detention for the 50% annual chance is not required.

If the post project runoff is higher than existing conditions for the 50% annual chance event, and sustainable drainage measures are designed to the specifications in this manual, the following method can be used to decrease the required detention storage computed from the Modified Rational Method for the 50% annual chance event:

Figure 2.3 Process for C Value Adjustment

1
Determine preliminary detention volume needed using the modified rational method with no adjustments for sustainable drainage measures.

↓

2
Design sustainable drainage measure with water quality volume computed from Equation 2.25

↓

3
Subtract the storage volume of the sustainable drainage measures from the preliminary detention volume to determine the adjusted detention volume needed.

2.3.4 Water Rights

Surface water in Texas is owned by the State and held in trust for the citizens of the State. The State grants the right to use this water to different people and entities such as farmers or ranchers, cities, industries, businesses, and other public and private interests.

The Texas Commission on Environmental Quality (TCEQ) is the State Agency responsible for the administration of the State's water rights. Information on water rights can be found on the TCEQ website.

State water may not be impounded and/or diverted without a water right. State water is defined by Texas Water Code §11.021 as

"(a) The water of the ordinary flow, underflow, and tides of every flowing river, natural stream, and lake, and of every bay or arm of the Gulf of Mexico, and the storm water, floodwater, and rainwater of every river, natural stream, canyon, ravine, depression, and watershed in the state is the property of the state.

(b) Water imported from any source outside the boundaries of the state for use in the state and which is transported through the beds and banks of any navigable stream within the state or by utilizing any facilities owned or operated by the state is the property of the state."

The City of Dallas has been granted water rights in the upper Trinity River Basin for municipal, domestic, industrial, mining, agricultural (livestock and irrigation), recreation and hydro-electric purposes. Dallas Water Utilities manages the majority of the City's water rights.

Texas Water Code Chapter 11 and Texas Administrative Code Title 30 Chapter 295 and 297 should be thoroughly reviewed to understand the regulatory impacts and permitting requirements of collecting and retaining storm water.

SECTION 3

Roadway Drainage Design

3.1

GENERAL

Streets or roadways shall be designed to convey the runoff which results from the 1% annual chance storm event between the curbs, or where no curb exists, within the roadside ditches. Alleys shall be capable of conveying runoff from the 1% annual chance storm event within the limits of the paved surface.

Inlets shall be placed at locations where the gutter capacity is exceeded by runoff from the 1% annual chance storm event. Likewise, inlets shall be placed at locations where alley capacity is exceeded by runoff from the 1% annual chance storm event. Consideration must be given to such factors as location relative to streets, schools, parks and other areas subject to frequent pedestrian use as well as basic economics. A street or alley adjacent to an open channel shall have the edge of pavement with a design elevation not lower than the drainage and floodway easement elevation with required minimum freeboard, or as directed by the Director.

Drainage design requirements for open and closed systems shall provide protection for property during the 1% annual chance storm event. The design flow will result from assuming fully developed conditions as projected by the City's current zoning maps, and this projected flow shall be carried in the streets and closed drainage systems in accordance with these guidelines. Runoff from paved areas shall be conveyed through enclosed storm drain systems if curb and gutter are used. Ditches are allowed subject to the requirements in Section 3.4. Throughout this section, nomographs are provided as design aids for convenience. Designers may use the nomographs, spreadsheet calculations, or software from the approved list on the City of Dallas website or other software approved by the Director.

3.2

IMPROVED ROADWAY DRAINAGE

3.2.1 Street Capacity

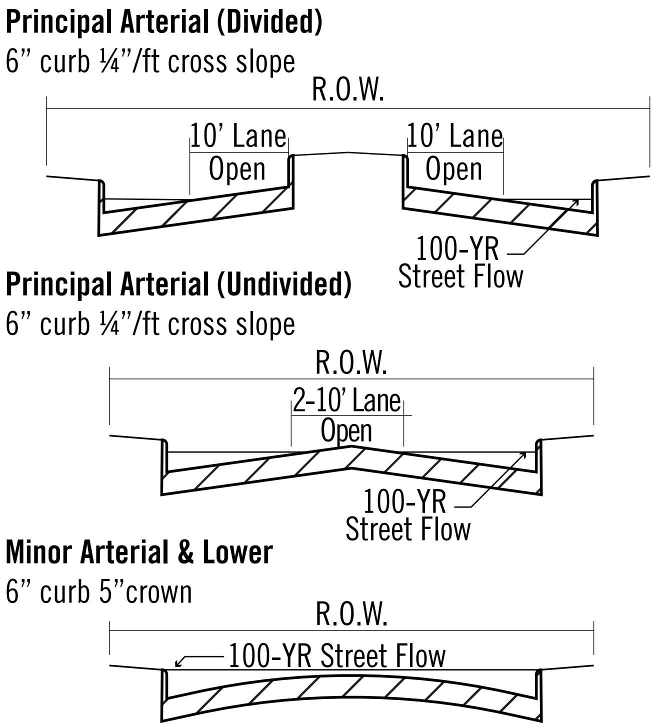
Streets may be used for stormwater drainage only if the calculated stormwater flow does not exceed the maximum flow depth as provided in Table 3.1. A Manning's roughness coefficient of 0.0175 shall be used in calculating street flow conditions. Where streets are not capable of carrying their design criteria stormwater discharge, inlets or curb openings are required. The inlets or openings will discharge into a drainage channel or storm drain system. The following table specifies the allowable encroachment limits for the different street types.

Table 3.1 Encroachment Limits

Street Classification	Allowable Encroachment (dry lane)
Minor Arterial and lower	Maximum depth of 6" or top of curb
Principal Arterial	One lane of traffic in each direction must remain open

All streets shall be capable of conveying the 1% annual chance storm event runoff without exceeding the top of curb, as shown in Figure 3.1.

Figure 3.1 Dry Lane (Allowable Lane Encroachment)



3.2.2 Straight Crown Street

Equation 3.1 may be used to determine the capacity of flow in a street having a straight cross-slope. See the Street Design Manual for limitations on street cross-slopes and longitudinal grades.

Figure 3.1 applies to all width streets having a straight cross slope. Stormwater flows in a street having a straight flow in a slope may be expressed as follows:

$$Q = \frac{0.56}{n} S_x^{1.67} S_L^{0.5} \left(\frac{d}{S_x}\right)^{2.67} \quad \textit{(Equation 3.1)}$$

Q = flow rate (cfs)

n = Roughness coefficient for street paving (a value of n = 0.0175 to be used)

S_x = cross slope (ft/ft)

S_L = longitudinal slope (ft/ft)

d = depth of flow (ft)

This formula is a reformulation of Manning's Equation for flow in triangular channels (“Manning's Equation, HEC-22, Third Edition, 2009, Equation 4.2 and 4.3.)

For determining the flow capacity in a street with a depressed gutter, refer to the current version of HEC-22.

3.2.3 Parabolic Crown Street

The following formulas can be used for determining the gutter capacity in streets with parabolic crowns:

$$Q = 1.486 \frac{A (R)^{2/3} (S_f)^{1/2}}{n} \quad \textit{(Equation 3.2)}$$

$$R = \frac{A}{P} \quad \textit{(Equation 3.3)}$$

$$A = \frac{W_O C_O}{6} \quad \textit{(Equation 3.4)}$$

Q = Flow (cfs)

A = Cross sectional area of flow (ft^2)

R = Hydraulic radius (ft)

S_f = Longitudinal slope of the street gutter (ft/ft)

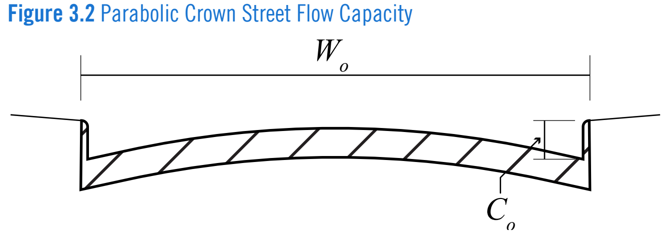
n = Roughness coefficient for street paving (a value of n = 0.0175 to be used)

P = Wetted perimeter (ft)

W_O = Width of the street (ft)

C_O = Crown height of the street (ft)

Figure 3.2 Parabolic Crown Street Flow Capacity



3.2.4 Valley Gutter Flow

The use of valley gutters to convey stormwater across a street intersection is subject to the following criteria:

1. Principal and minor arterials shall not· be crossed by a valley gutter.
2. At any intersection, perpendicular valley gutters will not be permitted; and parallel valley gutters may cross only the lower classified street.

The design shall achieve longitudinal street grades and cross slopes that provide proper flow of surface drainage toward and into inlets without sags and puddles along or within the street. Design street grades and cross slopes must consider the characteristics of the underlying soils, the engineered cuts and fills, and proposed pavement section(s) to achieve positive surface drainage. See the Street Design Manual for longitudinal grade design criteria for streets and minimum grade for valley gutters.

3.2.5 Flow in Alleys

All alley capacities shall be hydraulically designed using Manning's equation. The 1% annual chance storm event shall be contained within the edge of pavement. Where the standard alley section capacity is exceeded, storm sewer systems with inlets shall be provided.

Flow in alleys shall be calculated using Manning's equation.

$$Q = 1.49 \frac{A(R)^{2/3} (S_f)^{1/2}}{n} \qquad \textit{(Equation 3.5)}$$

Q = Flow (cfs)

A = Cross sectional area of the flow (ft^2)

R = Hydraulic radius
(area of flow divided by the wetted perimeter) (ft)

S_f = Longitudinal slope (ft/ft)

n = Roughness coefficient for alley pavement
(a value of n = 0.0175 to be used for alleys)

Inlets shall be located in alleys upstream from an intersection and where necessary to prevent water from entering intersections in amounts exceeding allowable street capacity requirements.

Examples of alley capacity calculations for conditions with, and without, a curb are provided in Appendix A.3. Alleys adjacent to a drainage channel shall be required to have curbs for the full length.

3.3 INLET DESIGN

3.3.1 Types of Inlets

Inlets collect excess storm water from the street, transition the flow into storm drains, and can provide maintenance access to the storm drain system. Storm water inlets are used to remove surface runoff and convey it to a storm drainage system. There are five major types of inlets: grate, curb, Y-inlet, slotted, and combination. Table 3.2 provides considerations in proper selection.

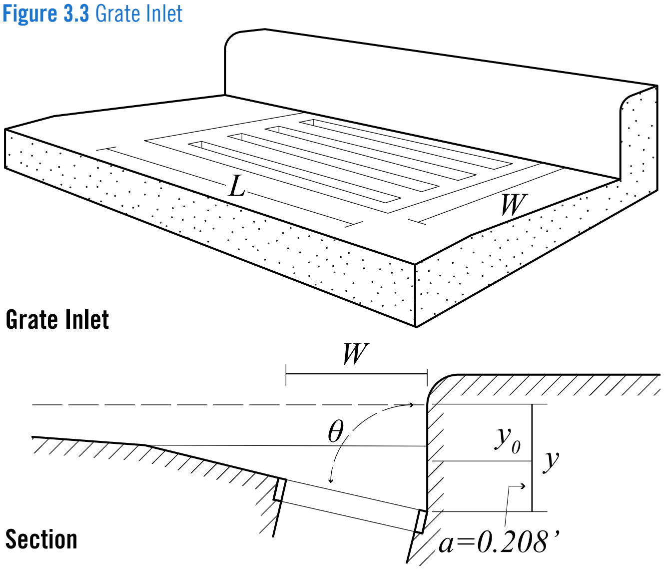
Table 3.2 Inlet Types

Inlet Type	Applicable Setting	Advantages	Disadvantages
Grate	Sags & continuous grades (should be made bike safe).	Perform well over wide range of grades.	Can become clogged.
Curb	Sags & continuous grades (but not steep grades).	Do not clog easily. Bicycle safe.	Not effective with steep grades.
Y-Inlet	Swales & sags	High Capacity. Do not clog easily.	Cannot be used in roadways.
Combination	Sags & continuous grades (should be made bike safe).	High Capacity. Do not clog easily.	More expensive than grate or curb-opening acting alone.
Slotted	Locations where sheet flow must be intercepted.	Intercept flow over wide section.	Susceptible to clogging.

3.3.1.1 Grate Inlet

Although grate inlets may be designed to operate satisfactorily in a range of conditions, they may become clogged by floating debris during 1% annual chance storm events. In addition, they can produce a hazard to wheel-chair and bicycle traffic and must be designed according to ADA guidelines. For these reasons, they may be used only at locations where space restrictions prohibit the use of other types of inlets and must be approved by the Director.

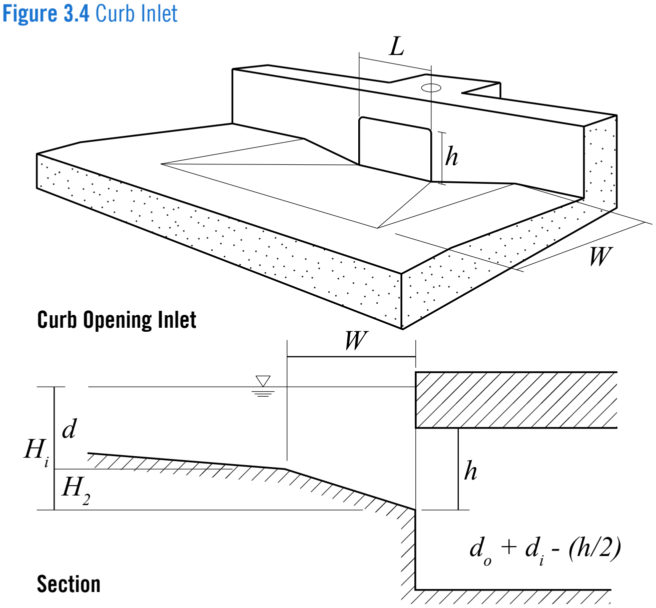
Figure 3.3 Grate Inlet



3.3.1.2 Curb Inlet

Curb inlets (both recessed and non-recessed) are the most effective type of inlet on slopes flatter than 3%, in sag locations, and with flows that typically carry large amounts of debris. Similar to grate inlets, curb inlets also tend to lose capacity as street grades increase, but to a lesser degree than grate inlets.

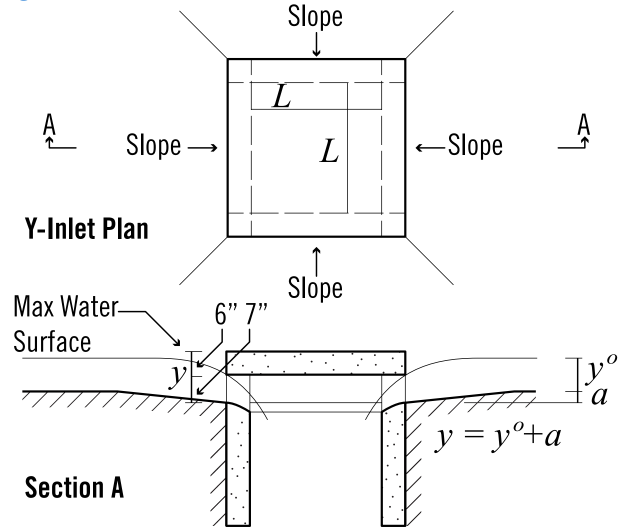
Figure 3.4 Curb Inlet



3.3.1.3 Y-Inlet

Y-inlets are most often used in swale and sag drainage.

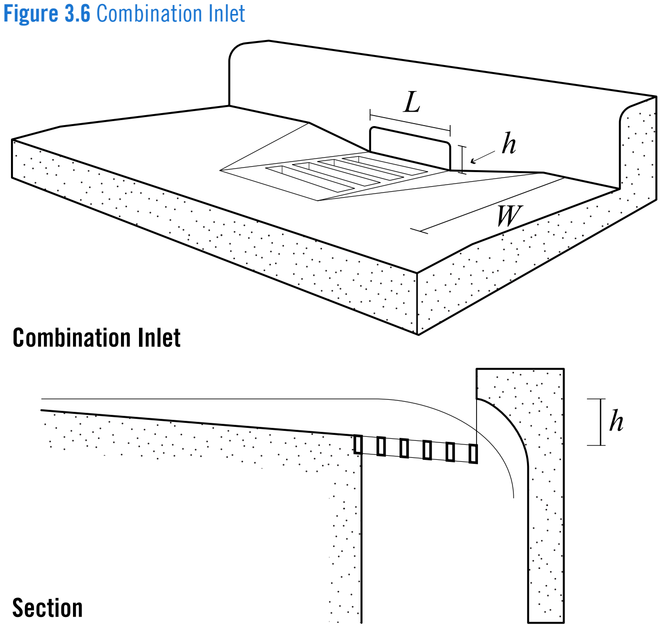
Figure 3.5 Y-Inlet



3.3.1.4 Combination Inlet

A combination inlet consists of both the grate inlet and the curb inlet. This configuration provides many of the advantages of both inlet types. The combination inlet also reduces the chance of clogging by debris with flow into the curb portion of the inlet. If a curb opening is extended on the upstream side of the combination inlet it will act as a "sweeper", and remove debris before it reaches the grate portion of the inlet. Due to the presence of the grate, a combination inlet can produce a hazard to wheel-chair and bicycle traffic and must be designed according to ADA guidelines. For these reasons, they may be used only at locations where space restrictions prohibit the use of other types of inlets and must be approved by the Director.

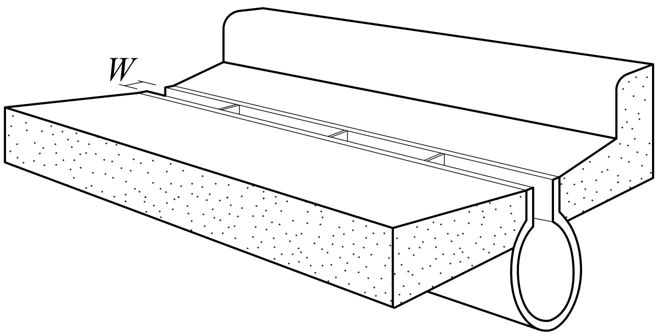
Figure 3.6 Combination Inlet



3.3.1.5 Slotted Drain Inlet

Slotted drain inlets can be used to intercept sheet flow, or flow in wide sections. They are not recommended for use in the City of Dallas since they are susceptible to clogging from debris. Use of slotted inlets may not be used unless approved by the Director.

Figure 3.7 Slotted Drain Inlet



3.3.2 Inlet Design Requirements

Inlets shall be placed to ensure that the flow in the 1% annual chance storm event in a street does not exceed top-of-curb elevation, and that encroachment into the travel way does not violate the minimum dry lane requirements of 10' of pavement available to drive on, in each direction during 1% annual chance storm events.

If in the judgment of the engineer the flow in the gutter is excessive, the storm drain shall be extended to a point where the gutter flow can be effectively intercepted by inlets. Guidelines for inlet placement include:

1. Placing multiple curb inlets at a single location is only permitted in areas with steep grades (4% or greater) to prevent flooding and avoid exceeding street capacity in flatter reaches downstream.
2. For all sag locations on a principal arterial street, flanking on-grade inlets shall be placed not more than 50 feet away on each side of the sag inlet.
3. To minimize water draining through an intersection, inlets shall be placed upstream from an intersection.
4. Inlets shall also be located in alleys upstream of an intersection and where necessary to prevent water from entering intersections in amounts exceeding allowed street capacity.
5. Inlets shall be placed upstream from right angle turns.
6. Inlet boxes designed more than 4.5 feet deep require a special detail.
7. All Y-inlets and inlets 10-foot or greater shall have a minimum 21-inch reinforced concrete pipe (RCP) lateral.
8. Inlets are required at all sag points.
9. The end of the recessed inlet box shall be at least 10 feet from a curb return or driveway wing; and the inlet shall be located to minimize interference with the use of adjacent property. Inlets shall not be located across from median openings where a drive may be added.
10. Inlets located directly above storm drain lines shall be avoided.
11. Data shown at each inlet shall include paving or storm drain stationing at centerline of inlet, size of inlet, type of inlet, top-of-curb elevation and flow line of inlet.
12. Inlet box depth shall not be less than 4 feet when the lateral is 21 inches.
13. Laterals conveying flow from an inlet must tie into a trunk line and shall not tie into another inlet, unless otherwise approved by the Director.
14. Grate type inlets shall be carefully considered and used only if justified due to site conditions. Grate inlets must be approved by the Director.

In situations where only the lower portion of an enclosed storm drain system is being built, stub-outs for future connections must be included. In this case, it is not necessary to capture all the street flow at the stub-out. At a minimum, there must be enough inlets to capture an amount equal to the total street flow capacity at the stub-out.

Inlets shall be appropriately placed to avoid excessive water draining through an intersection. Inlets shall be provided on all streets intersecting minor and principal arterials to capture the surface drainage in the street with no drainage bypass. A few considerations for designing drainage in conjunction with the new paving construction or reconstruction projects are listed below:

- Inlets shall not be placed at inconvenient locations for property owners such as near driveways and lead walks.
- Grate inlets may be used only where space restrictions prohibit the use of other types of inlets. If used, the inlet opening should be designed to be twice as large as the theoretical required area to compensate for clogging and must be approved by the Director.
- Combination curb inlets (with opening in curb and grate opening in gutter) may be used only where space behind the curb prohibits the use of other inlet types.
- Where recessed inlets are required, they shall not decrease the width of sidewalk or interfere with utilities.
- Recessed inlets must also be depressed, unless otherwise approved by the Director. Maximum allowable gutter depression for recessed inlets shall be seven inches.
- Recessed inlets may be used on divided thoroughfares.
- Non-recessed, depressed inlets shall have a maximum allowable gutter depression of five inches.

Refer to Figure 3.8 for standard storm drain inlets used in the City of Dallas and typical locations for each.

3.3.3 Inlet Capacity Calculations

Inlets and openings will be located and sized to meet the design criteria of the roadways they service, the ponds they drain to and from, and other connected drainage system conveyance.

The following inlet capacity formulas and design guidelines are based on inlets on grade and at sag points. Inlet capacities for on grade inlets are less than those of inlets in sag. The capacity of an on grade inlet depends on street grade, deflections, cross slope, depressions, etc. The capacity of an inlet in sag depends on the water depth at the curb opening and the height and length of the curb opening. Inlet capacity calculations may be computed from the equations provided in the following sections, or they may be performed using software from an approved list which can be found on the City of Dallas website. Appendix A.3 provides an approved spreadsheet template that may be utilized for computing inlet capacities.

3.3.3.1 Grate Inlet

Grate Inlet on Grade

The interception capacity of grate inlets on grade depends on the width and length of the grate and the width and velocity of the flow approaching the grate. When the approaching flow velocity is slow and the flow width does not exceed the grate width, the grate inlet might be able to intercept all of the approaching flow. In cases where the width of the approaching flow exceeds the grate width, very little of the approaching flow that exceeds the grate width will be intercepted by the inlet. When the velocity of the approaching flow is too high, the flow will “splash over” the grate. Both of these phenomena contribute to flow bypass of grate inlets, which is analogous to the bypass flow discussed in relation to curb opening inlets on grade.

The actual length (L_A) and width (W_A) of the grate opening is the overall dimension of the grate less the width of any bars or vanes. To account for the effects of clogging of a grated inlet on grade, the actual length and width of the grate opening are reduced by a factor of fifty percent ($C = 0.50$). A single grate inlet per 251D standards has an actual length of $L = 2.6$ ft and an actual width of $W = 1.3$ ft. Therefore, applying a clogging factor of fifty percent, the effective length and width of a single grate inlet are $L_E = 1.3$ ft and $W_E = 0.7$ ft, respectively.

The procedure to determine the interception capacity of a grated inlet on grade is as follows:

Step 1. Divide the flow approaching the inlet ($Q_{APPROACH}$) into frontal discharge (Q_W) and side flow (Q_S), (i.e., flow beyond the width of the grate).

$$Q_W = Q_{APPROACH}\left[1 - \left(\frac{W_E}{T}\right)^{2.67}\right] \quad \text{(Equation 3.6)}$$

$$Q_S = Q_{APPROACH} - Q_W \quad \text{(Equation 3.7)}$$

Q_W = Portion of approaching flow within the width of the grate (cfs)

$Q_{APPROACH}$ = Total flow approaching the grate (cfs)

W_E = Effective width of grate (ft)
(an effective width $W_E = 0.7$ ft)

T = Total spread of water in the roadway (ft)

Q_S = Side discharge (cfs)

Step 2. Compare the approach velocity to the splash-over velocity. The splash-over velocity for a single grate inlet is 2.0 ft/s. When the approach velocity is less than the splash-over velocity, it can be assumed that the grate intercepts all of the approaching frontal discharge. When the approach velocity exceeds the splash-over velocity, calculate the amount of frontal discharge intercepted by the grate.

$$Q_{INTERCEPT,FRONT} = (1.0 - 0.09\,(V - V_0))\,Q_W$$

$$\text{(Equation 3.8)}$$

$Q_{INTERCEPT,FRONT}$ = Frontal discharge intercepted by grated inlet (cfs)

V = Velocity of flow approaching inlet (ft/s)

V_O = Splash-over velocity (V_O = 2.0 ft/s)

Q_W = Frontal flow approaching grated inlet (cfs)

Step 3. Determine the amount of side flow ($Q_{INTERCEPT,\ SIDE}$) that is intercepted by the grate.

$$Q_{INTERCEPT,\ SIDE} = \frac{Q_{SIDE}}{\left[1 + \left(\frac{0.15V^{1.8}}{S_X L_E^{\ 2.3}}\right)\right]} \quad \text{(Equation 3.9)}$$

$Q_{INTERCEPT,\ SIDE}$ = Side discharge intercepted by grated inlet (cfs)

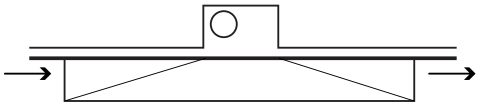
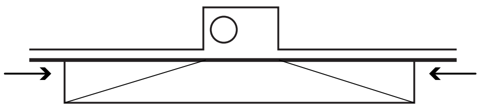
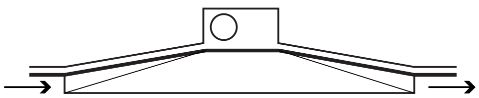
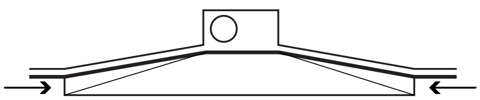
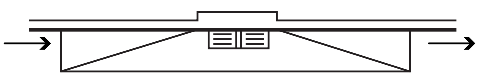
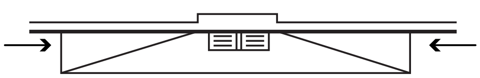
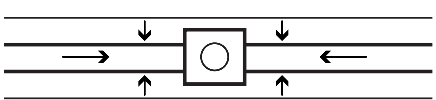
Q_{SIDE} = Side flow (i.e., flow outside width of grate) (cfs)

V = Velocity of flow approaching inlet (ft/s)

S_X = Street cross slope (not longitudinal slope of gutter) (ft/ft)

L_E = Effective length of grate (ft). Effective length of L_E =1.3 ft

Figure 3.8 Storm Drain Inlets

Inlet Description	Common Inlet size	Where Used
 Standard curb opening inlet on grade	**L** 5' 10' 14'	All Minor Streets
 Standard curb opening inlet at low point	**L** 5' 10' 14'	All Minor Streets
 Recessed curb opening inlet on grade	**L** 5' 10' 14'	All Divided Secondary and Major Streets
 Recessed curb opening inlet at low point	**L** 5' 10' 14'	All Divided Secondary and Major Streets
 Grate inlet (curb type) on Grade	**#** Single Double Triple	Combination inlets to be used only where space behind curb prohibits other inlet types and in alleys and with permission of the Director
 Grate inlet (curb type) at low point	**#** Single Double Triple	Do not use except with permission of the Director
 Grate inlets (Gutter type)	Special 2 Grate 4 Grate 6 Grate 8 Grate	Inlet location with no curb Alleys - Driveways with permission of the Director
 Y-inlet	"Y"	Open Channels - Ditches

The Federal Highway Administration's Urban Drainage Design Manual (HEC-22) provides guidance for other grate types and configurations.

Step 4. Calculate the amount of flow intercepted by the inlet and the bypass flow, and apply the bypass flow to the roadway flow calculations and inlet capacity calculations downstream.

$$Q_{BYPASS} = Q_{APPROACH} + Q_{INTERCEPT} \quad \text{(Equation 3.10)}$$

Grate Inlet on Sag

A grate inlet in a sag location operates as a weir at shallower depths, and as an orifice at larger depths. The transition from weir flow to orifice flow depends on factors such as grate size and bar configuration. The designer shall estimate the capacity of the inlet under both weir flow and orifice flow conditions, then adopt a design capacity equal to the smaller of the two results.

Step 1. Calculate the capacity of a grate inlet operating as a weir, using the weir equation with a length equivalent to perimeter of the grate. When the grate is located next to a curb, disregard the length of the grate against the curb.

$$Q = C_W \; P_E \; d^{1.5} \quad \text{(Equation 3.11)}$$

Q = Inlet capacity of the grated inlet (ft/s)

C_W = Weir coefficient (C_W = 3.0 for U.S. Traditional Units)

P_E = Effective grate perimeter length (ft)

d = Flow depth approaching inlet (ft)

To account for the effects of clogging of a grated inlet operating as a weir, a clogging factor of fifty percent (C_L = 0.50) shall be applied to the actual (unclogged) perimeter of the grate (P):

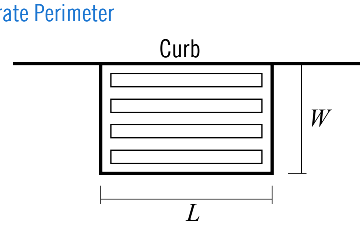
$$P_E = (1 - C_L) \; P_A \quad \text{(Equation 3.12)}$$

P_E = Effective grate perimeter length (ft)

C_L = Clogging factor (C_L = 0.50)

P_A = Actual perimeter of the grate (ft) as shown in Figure 3.9.

Figure 3.9 Grate Perimeter



$P_A = 2\,(W\text{- width of bars}) + L\,(\text{with curb})$
$P_A = 2\,(W + L\text{ - bars})\,(\text{without curb})$
$A = WL\text{ - area of bars}$

Step 2. Calculate the capacity of a grate inlet operating as an orifice. Use the orifice equation assuming the clear opening of the grate reduced by a clogging factor C_A = 0.50. A single grate inlet per 251D standards has an actual clear opening of A = 1.5 ft^2. The Federal Highway Administration's Urban Drainage Design Manual (HEC-22) provides guidance for other grate types and configurations.

$$Q = C_0 \; A_E \; (2gd)^{1/2} \quad \text{(Equation 3.13)}$$

$$A_E = (1 - C_A) \; A \quad \text{(Equation 3.14)}$$

Q = Inlet capacity of the grated inlet (cfs)

C_O = Orifice coefficient (C_O = 0.67 for U.S. Traditional Units)

A_E = Effective (clogged) grate area (ft^2)

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

d = Flow depth above inlet (ft)

C_A = Area clogging factor (C_A = 0.50)

A = Actual opening area of the grate inlet (i.e., the total area less the area of bars or vanes). The actual opening area for a single grate inlet per 251D standards is A = 1.5 ft^2.

Step 3. Use more conservative of the two results

3.3.3.2 Curb Inlet

Recessed and Standard Curb Opening Inlets on Grade

The length of inlet necessary to intercept gutter flow can be calculated using Equation 3.15.

$$L = \frac{Q(H_1 - H_2)}{0.70(H_1^{2.5} - H_2^{2.5})} \quad \text{(Equation 3.15)}$$

L = Length of inlet required to intercept the gutter flow (ft)

Q = Gutter flow (cfs)

H_1 = Depth of flow in the gutter approaching the inlet plus the inlet depression (ft)

H_2 = Inlet depression (ft)

Figure 3.10 can be used to determine the amount of flow intercepted by an inlet length that is less than the required inlet length computed from Equation 3.15.

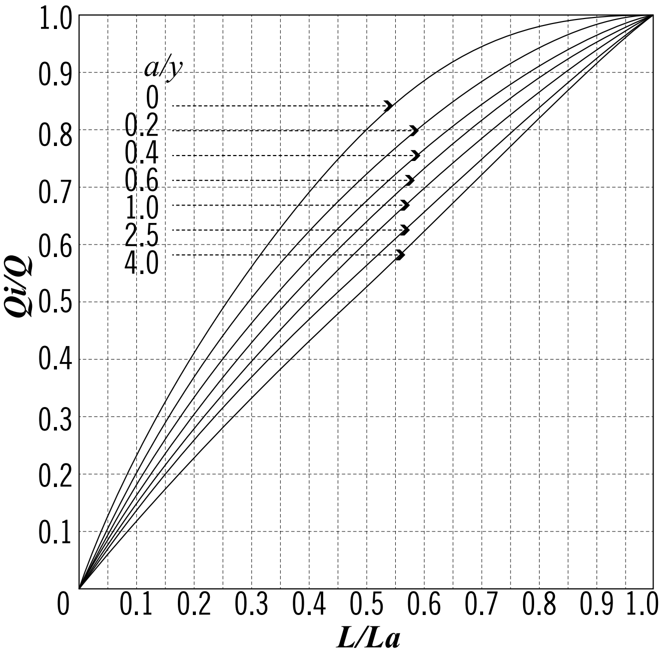
Q_I = Flow intercepted by inlet of length L

L_A = Length of curb opening for 100% interception

a = Gutter depression

y = Depth of flow in approach gutter

Figure 3.10 Curb Inlets on Grade: Ratio of Intercepted to Total Flow



(see enlarged figure in Appendix A.7)

Recessed/Standard Curb Inlets at Sag

Sag inlets operate as a weir up to a depth equal to 1.4 times the height of opening, and as an orifice for greater depths. See Figure 3.4 for dimensions. The corresponding equations for weir flow are:

$$Q = 2.1\,(L + 1.8W)\,d^{1.5} \quad \text{(Equation 3.16)}$$

Q = inlet capture (cfs)

W = width of depression (ft) (measured transversely from face inlet)

L = length of curb opening, (ft)

d = depth of water at curb (ft) (from the normal cross slope gutter flow line)

Orifice Flow

$$Q = 0.60\,hL\left(2g\left(d_i - \frac{h}{2}\right)\right)^{0.5} \quad \text{(Equation 3.17)}$$

h = height of inlet opening (ft)

L = length of curb opening (ft)

d_i = depth at lip of curb opening (ft)

h = height of curb opening orifice (ft)

Assuming that gutter flow is at the top-of-curb elevation for a 6-inch curb, a curb inlet with a 5-inch depression and a 6-inch height of inlet opening in a sag will require 0.5 feet of opening per each 1 cfs of gutter flow.

The coefficient in Equation 3.17 has been adjusted to accommodate a 10% loss in capacity due to clogging.

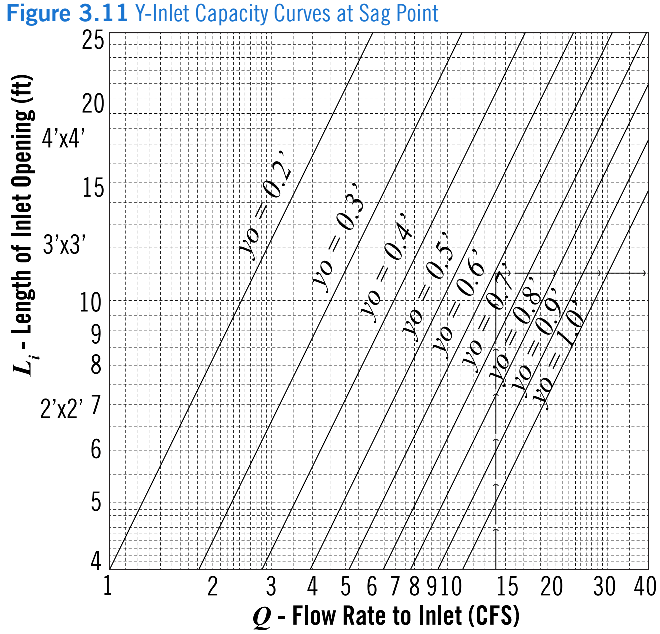
3.3.3.3 Y-Inlet

A Y-inlet performs similarly to a curb opening inlet in a low point. The inlet capacity process described above for recessed/standard curb inlets may be used for Y-inlets, with the length, L, modified to account for all 4 sides of the Y-inlet.

Alternatively, the following procedure for calculating the capacity of a Y-inlet using a nomograph may be used:

1. Determine the flow rate to the inlet.
2. Determine the maximum depth of the desired flow (y_O).
3. Enter Figure 3.11 at the flow rate and intersect with (y_O), read (L_I) to determine the length of the inlet opening.

Figure 3.11 Y-Inlet Capacity Curves at Sag Point



(see enlarged figure in Appendix A.7)

3.3.3.4 Combination Inlet

The procedure for calculating the capacity of a combination of a curb opening inlet and grate inlet assumes that the curb opening inlet is placed upstream of the grated inlet.

1. Determine the portion of flow intercepted by the curb-opening part of the inlet.
2. Determine the depth, width, and velocity of the flow that bypasses the curb opening part of the inlet (if on grade).
3. Determine the portion of bypassed flow intercepted by the grated inlet.

See Appendix A.3 for Sag Inlet Design Worksheet.

3.3.3.5 Slotted Drain Inlet

Slotted drain inlets shall only be specified with prior City approval. When located on a grade, slotted drain inlets function as a side-flow weir, much like curb-opening inlets.

Federal Highway Administration (FHWA) suggests that the hydraulic capacity of slotted drain inlets corresponds closely to the hydraulic capacity of curb-opening inlets when the slot openings are greater than 1.75 inches. Therefore, the designer may use the equations developed for curb opening inlets when the slot openings are greater than 1.75 inches.

When located in a sump, slotted drain inlets can function either as a weir or an orifice. As with grated inlets, the designer shall estimate the capacity of the inlet under both weir flow and orifice flow conditions, then adopt a design capacity equal to the smaller of the two results.

As with grate inlets, a clogging factor (C_L) shall be applied to the actual (unclogged) length of a slotted inlet (L):

$$L_E = (1 - C_L)\ L \quad \text{(Equation 3.18)}$$

L_E = Effective grate perimeter length (ft)

C_L = Clogging factor (C_L = 0.50)

L = Actual (unclogged) length of the slotted inlet (ft)

3.4

UNIMPROVED ROADWAY DRAINAGE

3.4.1 Bar Ditch Design

Bar ditches can be used on roads without curb and gutter. Bar ditches should have a maximum side slope of 3:1 with a bottom width dependent on required capacity. Bar ditches shall be designed to have a minimum velocity of 2 ft/s at design discharge. If project constraints dictate the need for an exception, discuss with the Director and provide the exception in writing.

Bar ditches shall be designed to contain the 1% annual chance storm event within the right-of-way and meet the design requirements discussed in Section 3.2.1. Refer to the Street Design Manual for minimum pedestrian clear zones.

3.4.2 Bioswales / Bioretention Swales

3.4.2.1 Bioswales

Bioswales are grass or vegetated channels that are designed to filter and convey stormwater runoff. Bioswales do not contain bioretention soil mix or engineered media, so do not provide the same pollutant removal capacity as bioretention, but they can remove sediment and improve water quality.

Site Considerations

The top of the side slope of bioswales shall be set back a minimum of 3 feet from property lines.

Design Considerations

Design should allow for a minimum hydraulic retention time of 5 minutes at design flow rate. Water depth, at design flow rate, should generally be below 4 inches.

Bioswales should have side slopes 3:1 or less, unless approved by Director, with a minimum bottom width of 2 feet.

Check dams can be used to slow velocities and provide additional treatment.

Longitudinal slopes should not exceed 4%. If channel slopes are above 2%, drop structures or check dams should be used.

At the inlet and outlet of the bioswale, vegetation that can withstand high velocities should be chosen.

See Appendix A.6 for landscaping requirements.

Sizing

Bioswales are designed for conveyance rather than storage. Using site conditions and design requirements above, open channel flow methods (see Section 5) can be used for design. When not used for primary roadway drainage, design is required to convey 1% annual chance event discharge.

3.4.2.2 Bioretention Swales

Bioretention swales are typically a linear channel with a parabolic or trapezoidal cross-section and may have a sloped bottom. They contain a filter bed of bioretention soil mix or engineered media and an underdrain system and are typically used along roadways and in parking lot islands. Bioretention swales can be effective for treating runoff, reducing sediment, and routing runoff to surface waters or storm drain systems. They can be used for retrofits, especially for existing ditch and swale systems. They can also be used within medians between curbs if space is available. Bioretention swales can be planted with grass but are more effective at improving water quality and slowing runoff velocities when vegetated with seeds or other plant material. A drain or overflow structure is typically located at one end. Check dams or other methods are typically employed to slow runoff and promote infiltration and treatment.

Site Considerations

Bioretention swales should have a maximum contributing drainage area of 5 acres.

Bioretention swales are not permitted in areas where there is seasonal high groundwater present less than 4 feet in depth from the bottom of the swale or where infiltration would contribute to soil or groundwater contamination.

The cross-section of the bioretention swale typically does not allow for erosive velocities.

The top of the side slope of bioretention swales should be located a minimum of 3 feet away from buildings and property lines. If located within 10 feet of a building, a liner must be used.

Design Considerations

Bioretention swales must be able to accomodate the 1% annual chance storm event through capture or flow diversion. If large events might cause washout, a bypass should be installed. Velocities shall be checked for the 1%, 2%, 10%, and 50% annual chance storm events.

Bioretention swales should have a maximum ponding drawdown time of 24 hours for the design flow. A longer ponding time may be approved by the Director. If not, an underdrain system should be used. See Section 6.10 for infiltration requirements.

Longitudinal slopes should not exceed 4%. If channel slopes are above 2%, drop structures or check dams should be used. A minimum design head of 3 to 5 feet from inflow to outflow is needed for positive drainage.

A gravel reservoir course can be provided under the bioretention soil mix for additional storage. A geotextile liner should be used to separate filter media and native soil.

Bioretention swales should have a minimum bottom width of 2 feet and a recommended side slope of 4:1. Bioretention swales should have a maximum side slope of 3:1 but can be steeper if located in a median and approved by the Director. Overflow ponding depth should be 6-12 inches. At the downstream end of the channel, the ponding depth should not exceed 18 inches.

Energy dissipators may be needed for concentrated inflows.

Overflow can be provided by adjacent inlets, vertical stand pipes, horizontal drainage pipes, or armored channels at the maximum ponding elevations which lead to surface waters, public storm drain systems, or storage reservoirs. Backwater calculations may be required depending on the capacity of the discharge system during large storm events. The overflow shall be sized for the 1% annual chance 24-hr storm event.

If an underdrain is installed, the gravel backfill shall have a minimum depth of 26 inches with a porosity of 0.35.

Aesthetics should be taken into consideration when the swale is placed in a public area.

Sizing

Bioretention swales can be sized similarly to rain gardens or bioretention areas (See Sections 6.8 and 6.9). See Section 2.3.3 for C value adjustments.

3.4.3 Culvert Crossing

Refer to Section 3.7 for culvert hydraulic design. Safety end treatments may be necessary for culverts under unimproved roadways as the culvert opening may constitute a safety hazard to the traveling public. TxDOT provides guidance on the use of guardrail or safety end treatments for culverts located within a defined "clear zone" adjacent to a roadway. Refer to the TxDOT Roadway Design Manual for guidance on determining whether guardrail or a safety end treatment may be needed for a culvert under an unimproved roadway.

3.5

ADDITIONAL ROADWAY DRAINAGE CONSIDERATIONS

3.5.1 Utility and Drainage Infrastructure Requirements

The top of storm drainage conduits must not conflict with the pavement subgrade preparation zone. Special caution is required for storm drainage laterals close to inlets in the pavement warp down area to the inlet throat.

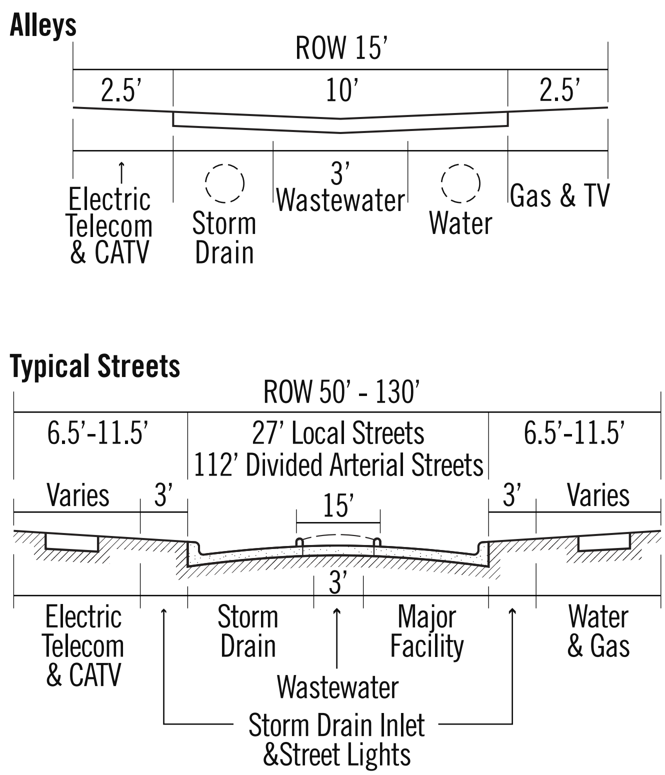
Generally, the top of all drainage pipes should be a minimum of 6 inches below the bottom of the paving subgrade in all locations to avoid damage during the construction process.

In practice, much greater depths are generally utilized over storm drainage pipes. Storm drainage mains and major storm drainage facilities normally are placed in the zone of the street between the back of curb and a plane 3 feet off of the pavement curbline. See Figure 3.12.

In general, curb inlets are to be used along paved streets and Y-inlets (drop inlets) are to be used in unpaved areas and drainage ditches. For ditches along roads, railroads, and in other applicable locations, provide section, profile with design water surface, and hydraulic computations.

Auxiliary drainage coming from off-site sources such as roof drain downspouts or private foundation drains should not be tied directly into the back of storm inlets.

Figure 3.12 Utility in Typical Streets



3.5.2 Alternative Pavements

3.5.2.1 General Design Considerations

Alternative pavements may be considered for design upon approval by Director. Alternative pavements include permeable pavement, permeable unit block systems, and porous pavers. Each of these alternative pavements are intended to provide the following benefits:

- Alternative pavement is considered as a sustainable design element. By proper application and defining appropriate design details, permeable pavement can have a long life, provide for groundwater recharge, lessen the quantity of stormwater runoff, and/or improve the quality of stormwater discharge from a site.
- The appearance of alternative pavement can improve the appearance and aesthetics of a site by softening the otherwise hard appearance of a conventional paved surface.

Required provisions for alternative pavement include:

- Permeable pavement is not allowed within public right-of-way unless specifically approved by the Director. It is not intended that permeable pavement will be used as a public street surface.
- A geotechnical investigation and evaluation is required to analyze existing soils and subsurface conditions for feasibility. This analysis will provide a basis for design of the pavement section.
- Proper and adequate drainage shall be provided where permeable pavement is used. Stormwater runoff shall not be diverted from other areas to permeable pavement. Drainage design shall include surface and subsurface conditions and considerations so that ponding does not occur and the pavement structure is sound.
- Conveyance must be provided for the 1% annual chance event runoff above the water quality capture volume of the alternative pavement. Drawdown of the design water quality volume should occur within 24-48 hours. The maximum ponding elevation should be 6 inches below the top of the wearing course. Inlets at the maximum ponding elevation or other methods of outflow must be provided for the conveyance of large storm events.
- If adjacent to a structure or roadway, there should be an impermeable barrier to prevent infiltration.
- Positive surface drainage should be established to prevent ponding, and slopes should direct runoff away from all structures.
- Permeable pavement is adequate for slopes of two percent or less. If slopes are greater, interceptor infiltration trenches or check dams should be used.
- When installed above clay soils, an impermeable layer may be installed between the subbase and subgrade to limit the potential of shrinking and swelling. Where soils do not allow infiltration, geomembrane liners are used to contain the reservoir area to allow for slow runoff release. Underdrains are required to protect infrastructure or utilities when clay or geomembrane liners are used.
- Design shall provide adequate support and surface for the intended use. The pavement sections shall be engineered assessing subsoil conditions, subgrade material and thickness, and the actual pavement material(s).
- Design for pedestrian uses shall meet any required accessibility criteria, especially if the pathway is a designated or expected route for pedestrians with disabilities.
- Accessory and ancillary elements necessary to accompany the permeable pavement shall be provided. These elements may include edge markings if the permeable pavement is a required fire lane, lighting where necessary for certain pedestrian or vehicular routes, etc.
- The property owner shall be responsible for all maintenance and upkeep of the permeable pavement. Sediments, debris / trash, mulch and lawn clippings should not be blown on the surface of the permeable unit paving.

3.5.2.2 Permeable Paving

Permeable paving consists of porous concrete or asphalt designed without fine aggregate. The additional void space allows for stormwater infiltration with a layer of gravel and filter fabric underneath.

Pervious Concrete

Pervious concrete pavement is a typical concrete mortar with minimal fine sands and particles. It has a high percentage of void space (typically 15-22%) that allows water to pass through the material. It is installed with an aggregate base with an underdrain system if infiltration is not possible. Pervious concrete pavement is suitable for walking paths, sidewalks, plazas, etc. and is not suitable for vehicular traffic.

Porous Asphalt

Porous asphalt consists of regular bituminous asphalt that has been screened to remove the fine material, creating void spaces in the material. When laid on top of an open-graded stone base, stormwater infiltrates through the pavement and underlying base. Porous asphalt is suitable for parking lots, walking paths, sidewalks, plazas, etc. Porous asphalt is not suitable for roads with heavy traffic (more than 25,000 vehicles/day) or for areas with high sediment and pollutant loading.

Porous asphalt is designed to be ADA compliant. The system must be designed with a base depth, typically 18-36 in, that will not allow water to rise through the asphalt. Potholes and cracking are less likely to occur in porous asphalt due to enhanced drainage, but conventional mix can be used if patching is required.

Design Considerations

Pervious pavement should be 4-6 inches thick. Porosity of the material should be no greater than 0.30.

Sealant should not be used on top of permeable paving.

The reservoir course must consist of single size aggregate with little or no fines. Reservoir course aggregate depth should be a minimum of 6-18 inches for pervious pavement. The reservoir course should have a minimum of 20 percent total void volume after compaction.

A minimum width of 8 feet of access should be provided for maintenance.

Sizing

Pervious pavement areas can be installed for any amount of area, as they are designed to treat only the precipitation which falls on it and not runoff routed from other areas. See Section 2.3.3 for C value adjustments.

Pervious concrete and asphalt can be assumed to have a porosity of 0.18. The gravel base course should be assumed to have a porosity of 0.32. The following equation can be used to determine the depth of the gravel layer based on a design water quality volume and other design parameters.

The total volume to be infiltrated should only include rainfall which falls on the surface on the permeable pavement area.

$$d_g = \frac{1}{n_g}\left[\left(\frac{V_{WQ}}{A}\right) - \frac{kT}{12} - n_p d_p\right] \quad (Equation\ 3.19)$$

d = Depth (ft) (g for gravel, p for concrete or asphalt)

n = Porosity (g for gravel, p for concrete or asphalt)

V_{WQ} = Design water quality volume (ft^3) (See Section 2.3.3)

A = Surface area (ft^2)

k = Percolation (in/hr)

T = Fill time (hrs)

Check that drain time of the total ponding depth is within 24-48 hours.

3.5.2.3 Permeable Unit Pavement System

Permeable unit paving consists of permeable interlocking concrete pavement, permeable interlocking brick pavers, or open jointed and open cell paving blocks. The units are designed with void spaces in between them that can be filled with aggregate that stormwater runoff can filter through. The system reduces impervious area and provides filtration for the stormwater before entering the storm sewer system or collected for reuse.

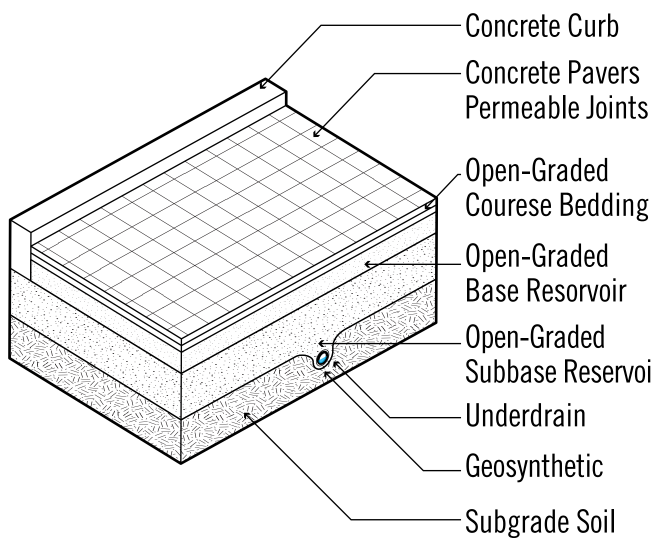
Permeable unit paving is suitable for parking lots, street parking areas, walking paths, sidewalks, driveways, plazas, and low traffic roads, and is not suitable for roads with heavy traffic (more than 25,000 vehicles / day) or for areas with high sediment and pollutant loading.

Permeable Pavers - Interlocking Concrete or Brick

Permeable interlocking concrete pavers consist of concrete units in a variety of interlocking patterns, shapes, colors, and finishes with small, stone filled joints. Permeable brick pavers similarly consist of brick units in an interlocking pattern, natural colors, and fired finishes. Both allow 100% of surface runoff to flow into a highly permeable reservoir area consisting of open-graded bedding base, base, and subbase aggregates. The system must be designed with a base depth of 9-18 inches. The spaces between the aggregates store water and enable infiltration to the underlying subbase soils.

Permeable interlocking pavers are suitable for sidewalks, walking paths, pedestrian plazas, parking lots, driveways, street parking areas, and low-traffic roads. Permeable interlocking pavers are designed to be ADA compliant.

Figure 3.13 Permeable Unit Pavement System



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APPENDICES

Design Considerations

Pavers should only be installed in areas with a slope less than 10%. Interceptor infiltration trenches or check dams must be used for slopes greater than 2%.

Reservoir course aggregate depth should be a minimum of 9-18 inches and sized for the storage area of runoff for the pervious unit pavement area. The reservoir course should have a minimum of 20 percent total void volume after compaction.

Some permeable unit pavers are not ADA compliant, so alternate pavement should be used for ADA parking stalls and access routes.

Sizing

Permeable unit pavers can be installed for any amount of area, as they are designed to treat only the precipitation which falls on them and not runoff routed from other areas. See Section 2.3.3 for C value adjustments for use in runoff calculations. The following equation can be used to determine the minimum allowable depth for the gravel base course. A porosity of 0.32 should be used for the gravel base course. The total volume to be infiltrated should only include rainfall which falls on the surface on the permeable paver area.

$$d = \frac{V}{A} * n \quad \textit{(Equation 3.20)}$$

d = gravel layer depth (ft)

V = total volume to be infiltrated (ft^3)
(can be set to V_{WQ} computed in Section 2.3.3)

A = surface area (ft^2)

n = porosity (typically 0.32)

3.5.2.4 Porous Pavers

Porous pavers are interlocking units of cellular grid systems such as plastics or concrete unit pavement with grooves, voids, or joints filled with small stones, gravel, or porous soil. The larger void spaces in the joints allow for stormwater to infiltrate through the surface layer and an open-graded stone or gravel base underneath.

Porous Pavers or Gravel Cellular Grid Systems

Porous pavers or cellular grid systems with small stones or gravel aggregate are suitable for head-in parking lots. The surface is not suitable for parking with heavy wheel turning movement, roads with vehicular traffic, or pedestrian access surfaces that require an ADA compliant surface route, as the joint openings exceed ½ inch.

Porous Grass Turf Blocks or Grass Cellular Grid Systems

Porous grass concrete turf blocks or cellular grid systems with soil openings for grass to grow allow water to drain through the porous soil mix. Porous grass pavers can be used in recreational or open spaces that have occasional traffic, fire lanes, service access routes, and infrequent and overflow parking (used no more than 2 times per week), as the grass would need to receive at least 5 days a week of sunlight and supplemental irrigation during periods of drought. Permeable grass pavers are not suitable for frequent use parking lots, street parking areas, walking paths, sidewalks, plazas, roads, or for areas with high sediment and pollutant loading.

Site Considerations

Porous pavers should not be used on slopes greater than 2%.

Porous pavers must be set back at least 10 feet from building foundations, basements, wellheads, and roadways to prevent adverse effects from infiltration.

Design Considerations

Porous pavers should be designed with a minimum of 40% void space. A 1-inch layer of sand with bridging stone underneath should be placed between the pavers and underlying gravel base course. The gravel layer should have a minimum depth of 9 inches.

Porous pavers should be filled with sandy loam topsoil mix. If no vegetation is desired, aggregate can be used to fill in the paver grid. Filter fabric should be placed between the filler material and the gravel base.

Porous pavers are not ADA compliant, so alternate pavement shall be used for ADA parking stalls and access routes.

Sizing

See Section 3.5.2 for guidance on sizing.

3.5.3 Sand and Organic Media Filters

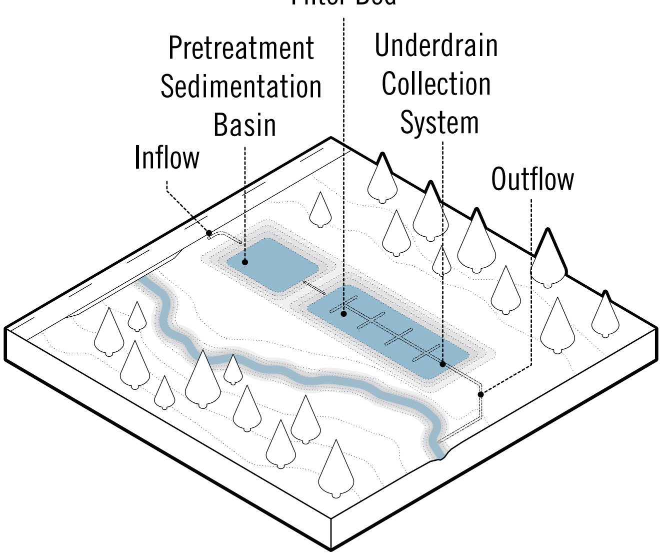
Sand and organic media filters contain either sand, leaf compost, or peat/sand mixture that serves as a filter media.

These materials provide enhanced removal of pollutants and can include the removal of heavy metals often found within runoff from roadway and parking surfaces.

Sand or organic media filters utilize a sedimentation basin, media bed for filtration, and underdrain collection system.

Underground filters can be used in space-limited areas.

Figure 3.14 Sand & Organic Media Filters



Sand or organic media filters should have a contributing drainage area no greater than 10 acres. Sand or organic media filters can be used in areas with higher pollutant loadings. However, sand filters should not be used for runoff with high sediment loadings. Areas with clay soils may require consideration of an underdrain system. Filters should not be located in an area with a slope greater than 6%.

Filters should be set back a minimum of 5 feet from buildings and roadways and 10 feet from property lines.

Design Considerations

Filters must be able to pass the 1% annual chance storm event through capture or flow diversion. If large events will cause washout, a bypass or overflow riser should be installed. Velocities shall be checked for the 1%, 2%, 10%, and 50% annual chance storm events.

Filters should be designed for intermittent flow rather than continuous. Grass cover can provide additional bacteria and pollutant removal as well as prevent clogging of the filter media.

Organic filters have two different configurations, a compost filter or a peat/sand filter. A compost filter has 18 inches of compost that is used as the filter media. A peat/sand filter uses a layer of 50/50 peat/sand mix over a 6-inch sand layer. If peat is used in the filter media, it should not contain a high percentage of decomposable material.

The pretreatment sedimentation basin should be sized to hold at least 25% of the design volume of the system with a recommended length-to-width ratio of 2:1. Inlet and outlet locations should be placed to avoid short-circuiting.

Filter beds should drain within 40 hours. A polyliner or concrete can be used to separate the filter bed from underlying soil.

Organic filters shall have a head of 5-8 feet. Sand filters shall have a head of 2-6 feet. A coefficient of permeability of 3.5 ft/day should be used for sand.

The media bed should have a depth of 18 inches (maximum of 24 inches) of sand and/or organic media with 3 inches of topsoil above and perforated pipe underdrain below and filter fabric placed between each layer. The underdrain must have a minimum slope of 1%.

Gravel should have a void space of approximately 40%.

Energy dissipators should be used at the inlets of surface sand filters. Inflow and outflow velocities should not be erosive. An emergency spillway is needed to convey overflow, and overflow should be routed away from downstream structures.

Maintenance access must be provided for inspection and media replacement.

Sizing

The following equation can be used to size the filtration basin chamber based on a design water quality volume:

$$A_f = V_{WQ} * d_f / [\, k*(h_f + d_f)*t_f] \quad \textit{(Equation 3.21)}$$

A_f = Surface area of filter bed (ft^2)

V_{WQ} = Design water quality volume (ft^3) (See Section 2.3.3)

d_f = Filter bed depth (ft)

k = Coefficient of permeability of filter media (ft/day) (use 3.5 ft/day for sand)

h_f = Average height of water above filter bed (ft) (1/2 h_{max})

t_f = Design filter bed drain time (days)

The following equation can be used to size the sedimentation chamber, which should be sized to hold at least 25% of the design water quality volume and have a length-to-width ratio of 2:1.

$$A_S = -\left(\frac{Q_O}{W}\right) * \ln 1 - E \quad \textit{(Equation 3.22)}$$

A_S = Sedimentation basin surface area (ft^2)

Q_O = Rate of outflow = V_{WQ} over a 24-hour period

V_{WQ} = Design water quality volume (ft^3) (See Section 2.3.3)

W = Particle settling velocity

E = Trap efficiency

The following assumptions can be made:

- 90% sediment trap efficiency ($E = 0.9$)
- Particle settling velocity, $W = 0.0033$ ft/s for imperviousness < 75%
- Particle settling velocity, $W = 0.0004$ ft/s for imperviousness ≥ 75%
- Average of 24 hour holding period

Using these assumptions,

$$A_s = 0.066 * V_{WQ} \quad for\ I < 75\% \quad \textit{(Equation 3.23)}$$

$$A_s = 0.0081 * V_{WQ} \quad for\ I \geq 75\ \% \quad \textit{(Equation 3.24)}$$

I = Imperviousness

Using the following equations, compute the minimum storage volume in the facility.

$$V_{min} = 0.75 * V_{WQ} = V_s + V_f + V_{f\text{-}temp} \quad \textit{(Equation 3.25)}$$

$$V_f = A_f * d_f * n \quad \textit{(Equation 3.26)}$$

$$V_{f\text{-}temp} = 2 * h_f * A_f \quad \textit{(Equation 3.27)}$$

$$V_s = V_{min} - V_f - V_{f\text{-}temp} \quad \textit{(Equation 3.28)}$$

V_{min} = Minimum storage volume in the facility (ft^3)

V_{WQ} = Design water quality volume (ft^3)

V_f = Water volume within filter bed/gravel/pipe (ft^3)

$V_{f\text{-}temp}$ = Temporary storage volume above filter bed (ft^3)

V_s = Volume within sediment chamber (ft^3)

n = Porosity (0.4)

A_f = Bottom surface area of filter bed (ft^2)

d_f = Filter bed depth (ft)

h_f = Average height of water above filter bed (ft) (1/2 H_{max})

The head in the sedimentation chamber can be calculated using the following equation:

$$h_s = V_s / A_s \quad \textit{(Equation 3.29)}$$

The head in the filter bed (H_f) and sedimentation chamber (H_s) should be confirmed to meet design requirements above. Iterate design as needed. The orifice from the sedimentation basin to the filter bed should be sized to release V_s within 24 hours with an average release rate of 0.5 H_s.

3.6

ENCLOSED STORM DRAIN SYSTEM DESIGN

After completing the hydrologic computations of the design storm runoff quantity entering each inlet, the size and gradient of pipe required to carry the design storm are determined. All enclosed systems shall be hydraulically designed using Manning's equation:

$$Q = 1.49 \frac{A R^{2/3} S_f^{1/2}}{n} \quad \text{(Equation 3.30)}$$

Q = flow (cfs)

A = cross sectional area of the conduit (ft^2)

n = roughness coefficient of the conduit

R = hydraulic radius (ft), the area of flow (A) divided by the wetted perimeter (P)

S_f = slope of the energy gradient (ft/ft)

P = wetted perimeter (ft)

The hydraulic gradient and velocity shall be calculated using the design flow, appropriate pipe size, and Manning's equation.

Enclosed storm drain systems shall be reinforced concrete pipe (RCP) or reinforced concrete box (RCB) sections. Other types of pipes will need prior approval from the Director.

Storm drain junction boxes are needed for access to underground storm sewers for inspection and cleanout. Junction boxes should be located at junctions with storm drain main lines, and at abrupt changes in alignment or grade.

Alignments of proposed storm drain systems shall utilize existing easements and right-of-way areas. Storm drainage systems are normally aligned so that the necessary trenching will not undermine existing surface structures, utilities or trees. No part of the proposed storm drain is to be designed within the improved subgrade of a proposed pavement.

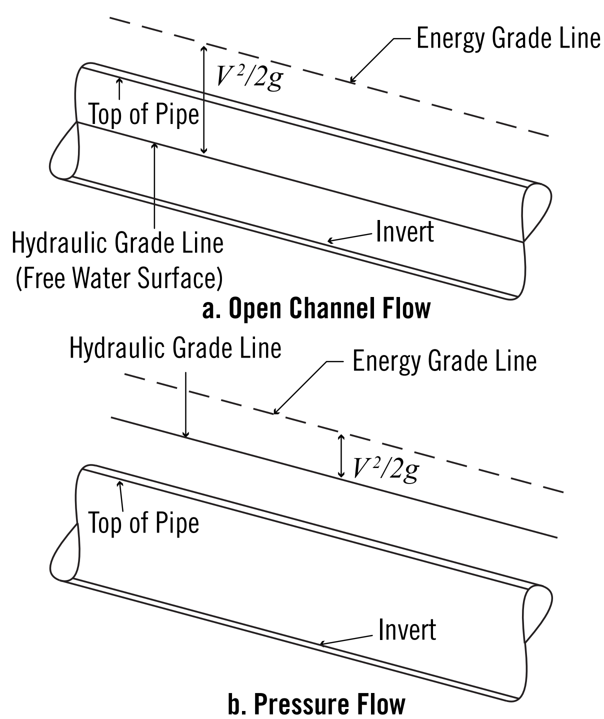
Storm drain line calculations spreadsheets may be used for design. A list of approved software that may be utilized for design of storm sewers in lieu of spreadsheet applications can be found on the City of Dallas website.

3.6.1 Starting Hydraulic Gradeline Elevation

The Hydraulic grade line (HGL) is the water surface of an open channel or the water surface of a conduit with partial flow. For a conduit with pressure flow, the HGL would be the level of water surface that would rise within a vertical tube at any point along the conduit.

The EGL is an imaginary line that is the measure of total energy along the open channel or conduit carrying water. This total energy includes elevation head, velocity head, and pressure head. The EGL is a velocity head ($V^2/2g$) above the HGL. The EGL is always increasing in the upstream conduit. The EGL should not be above the finished grade, or top of curb, at any point along the conduit.

Figure 3.15 Hydraulic Gradient



The City of Dallas requires that all hydraulic gradient calculations begin at the outfall of the system. In open channels, the water surface itself is the hydraulic grade line.

In some situations, generally at the upstream end of a pipe system, the inside top of the pipe may be above the HGL, which results in partial flow. In such cases, the pipe capacity and velocity shall be calculated at normal depth, neglecting minor losses. The HGL shall be shown in the profile on the plans for all storm drain lines, including inlet leads.

Table 3.3 can be used to establish appropriate design tailwater elevations for storm drainage systems based on the expected coincident storm frequency on the outfall channel and the ratio of the drainage area of the storm drain system and the receiving stream.

Table 3.3 Frequencies for Coincidental Occurrences

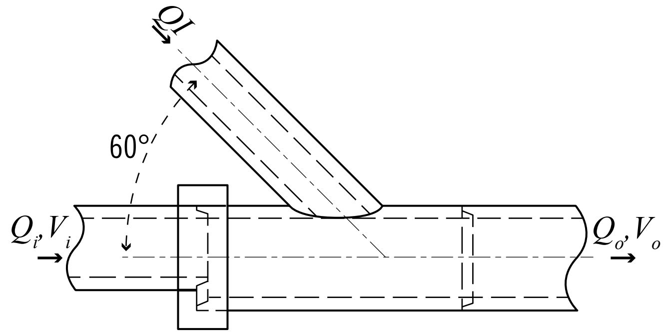
Area Ratio (Receiving Stream Area/Storm Drain Area)	Storm Drain Frequency		
	50% Annual Chance	10% Annual Chance	1% Annual Chance
10,000:1	1	1	2
1,000:1	1	2	10
100:1	2	5	25
10:1	2	10	50
1:1	2	10	100

3.6.2 Storm Drain Line Design

The following guidelines shall be followed for storm drainage design:

1. The minimum lateral storm drain pipe diameter shall be 21 inches.
2. Pipe diameters shall increase downstream, unless approved by the Director.
3. At points of change in storm drain size, pipe crowns shall be set at the same elevation.
4. A minimum grade of 0.3 percent will be maintained in the pipe unless approved by the Director.
5. Only standard pipe sizes will be used unless approved by the Director.
6. Laterals shall be connected to trunk lines using manholes or reinforced concrete wye connections. Special situations may require laterals to be connected to trunk lines by a cut-in (punch-in). However, such cut-ins must be approved by the Director.
7. Vertical curves in the storm pipe will not be permitted. Horizontal curve design for storm drains shall take into account joint closure. Half tongue exposure is the maximum opening permitted with tongue and groove pipe and horizontal curves must meet manufacturer's requirements for offsetting of the joints. Where horizontal alignment requires greater deflection, radius pipe on curved alignment should be used.

Figure 3.16 Wye Connection Detail



8. To prevent sedimentation in enclosed systems, minimum flow velocities with full pipe flow shall be 3 ft/s. Higher flow velocities may be associated with lower flow rates as may be the case with relatively steep pipes and low tailwater conditions. In such cases, full pipe flow velocities lower than 3 ft/s may be allowable as determined by the Director.
9. Where velocity exceeds 12 ft/s, a special lined RCP with a minimum of 1-1/2inch steel clearance on the inside surface shall be used. Maximum velocity in special lined RCP shall be 30 ft/s. Velocity rings shall be provided where grade of pipe exceeds 40% and length of pipe exceeds 100 feet. Velocity rings shall be cast-in-place type, not metal band type, and provide a low pass through notch.
10. Concrete pipe collars or manufactured transition pieces must be used at all pipe size changes on trunk lines and where the slope difference for the vertical grade change is less than 3%. Where the algebraic difference for vertical grade is greater than 3%, a manhole shall be provided.

In addition to the criteria listed above, the following general design guidelines will tend to alleviate or eliminate common problems of storm drain performance:

1. Select pipe size and slope so that the velocity of flow will increase progressively down the system or at least will not appreciably decrease at inlets, bends or other changes in geometry or configuration.
2. For all pipe junctions other than manholes and junction boxes, the angle of intersection shall be 60 degrees or as otherwise approved by the Director.
3. Inlet laterals will normally connect only one inlet to the trunk line. Special circumstances requiring multiple inlets to be connected with a single lateral shall require the Director's approval.
4. Storm drain pipes shall be reinforced concrete pipe, minimum class III, or stronger as determined by the design engineer.
5. Corrugated metal and plastic pipe will not be allowed beneath pavement in public easements and right-of-way areas.

6. The cover over the crown of circular pipe and box culverts shall be at least two feet from the bottom of proposed pavement and shall not encroach on the subgrade of the pavement, or from ground surface when no pavement is anticipated. As a general rule, the pipe cover should be based on the type of pipe used, the expected loads and the supporting strength of the pipe. It is the responsibility of the design engineer to calculate the required cover. Direct traffic box sections or less than required cover may be allowed in special situations with the approval of the Director.
7. Storm drain pipe type proposed for underground detention shall be submitted for review and approval by the Director prior to installation. See Section 6.4 for underground detention guidelines.
8. The flow regime shall be determined for all pipes in partial flow. If supercritical flow is anticipated, the location of any potential hydraulic jumps shall be determined and noted on the plans. (The procedure for this determination is not covered in this manual.) In areas where hydraulic jumps are anticipated, additional erosion control measures may be required as directed by the Director.
9. City approved software can be used to aid in design of storm drains. See the City of Dallas website for a list of approved software.

3.6.3 Lateral Design

Laterals are designed to convey the 1% annual chance storm event of the contributing area and meet dry lane requirements in Section 3.2.1.

Inlet laterals leading to storm sewers, where possible, shall enter the inlet and the storm drain main at a 60 degree (60°) angle from the street side. Laterals shall be four feet from top of curb to flow line of inlet, unless utilities or storm sewer depth require otherwise. Laterals shall not enter the corners or bottoms of inlets. All lateral profiles shall be drawn showing appropriate information including the HGL and utility crossings. Short laterals (12 feet or less) crossing utility lines shall be profiled.

The hydraulic grade line shall be calculated for all proposed laterals and inlets, and for all existing laterals being connected into a proposed drainage system. Connecting more than one lateral into a storm drain at the same joint localizes head losses; however, a manhole or junction structure must be provided unless each of the laterals are no more than ½ the diameter of the trunk line.

Laterals shall not outfall into downstream inlets unless approved by the Director.

3.6.4 Head Losses

From the time storm water first enters the storm drainage system at the upstream inlet until it discharges at the outlet, it will encounter a variety of structures such as inlets, manholes, junction, bends, and enlargements that will cause minor head losses. In general, these minor losses can be expressed as a function of velocity head.

In storm drain systems flowing full, all losses of energy through resistance of flow in pipes, by changes of momentum or by interference with flow patterns at junctions, must be accounted for by the cumulative head losses along the system from its initial upstream inlet to its outlet. The purpose of accurate determinations of head losses at junctions is to include these values in a progressive calculation of the hydraulic gradient along the storm drain system.

Calculations of the hydraulic grade line in the main begin from the downstream starting hydraulic grade line elevation and progress upstream using Manning's formula. Adjustments are made to the hydraulic grade line whenever the velocity in the main changes due to conduit size changes or discharge changes. Laterals in partial flow must be designed appropriately.

Hydraulic grade line "losses" wyes, pipe size changes, and other velocity changes will be calculated by the following formulas,

$$\text{Where } V_1 < V_2 \qquad \frac{V_2^2}{2g} - \frac{V_1^2}{2g} = H_L \quad \text{(Equation 3.31)}$$

$$\text{Where } V_1 > V_2 \qquad 0.1 = H_L \quad \text{(Equation 3.32)}$$

V_1 = upstream velocity (ft/s)

V_2 = downstream velocity (ft/s)

g = acceleration due to gravity (32.2 ft/s^2)

In determining the hydraulic gradient for a lateral, begin with the hydraulic grade of the trunk line at the junction plus the H_L due to the velocity change. Where the lateral is in full flow, the hydraulic grade is projected along the friction slope calculated using Manning's Equation. At an inlet where the lateral is in full flow, the losses for the inlet are:

$$\frac{1.5V^2}{2g} \quad \text{(Equation 3.33)}$$

V = velocity in lateral (ft/s)

g = acceleration due to gravity (32.2 ft/s^2)

The hydraulic grade line within the inlet should be a minimum of 0.5 feet below the top of the inlet.

Head losses for laterals, wyes, and enlargements shall be calculated using equations 3.31, 3.32, and 3.33, and will be applied as shown in Figure 3.17.

Head losses in bends, junction boxes, and manholes shall be calculated as shown in Figure 3.18

A minimum head loss of 0.1 foot is to be applied for all structures, bends, wyes, junctions, and enlargements. Head gains will not be recognized.

3.6.5 Manhole Placement and Design

All manholes shall be constructed in accordance with the Standard Details unless otherwise instructed by the Director. The following is a list of guidelines governing the placement and design of storm drain manholes to ensure adequate accessibility of the storm drainage system:

1. Storm drain lines 45 inches in diameter or less should have points of access no more than 500 feet apart. A manhole should be provided where this condition is not met.

2. Storm drain lines 48 inches in diameter or larger should be accessible at intervals of no greater than 1000 feet.

3. A manhole is required where two or more pipes connect into a main at the same joint. The exception to this would be the case in which the diameter of the main line is at least twice as large as the diameter of the largest adjoining pipe. A construction detail may be necessary at such locations.

4. In selecting a location for a manhole, pipe size changes and junctions are preferred sites. This will localize and minimize head losses.

5. All precast manhole structures will be built in conformity to the North Central Texas Council of Governments (NCTCOG) Standard Specification for Public Works Construction, latest addendum.

6. The size difference between a manhole and the largest adjoining pipe should be no less than 2 feet.

7. Maximum manhole size should be determined based upon a cost comparison between a manhole and junction box.

8. A minimum outside clearance of 1 foot should be provided between pipes connecting into a manhole.

9. Manholes on junction boxes, box culverts and horseshoes should be located toward the side of the structure such that the steps descending into the structure are aligned vertically. Steps should be made of either plastic or rubber coated steel.

10. The maximum allowable angle for the taper on a manhole riser is 3 vertical to 1 horizontal. Maintaining a minimum distance equal to ((3/2 x diameter) - 3) feet below the bottom of pavement will ensure that this angle is not exceeded. When there is insufficient clearance for proper taper, a flat top may be built over the manhole structure. Flat slab tops will be built in conformity with NCTCOG Standard Specifications for Public Works Construction.

11. Manhole covers on inlet boxes should be located at the same end of the inlet box as the lateral draining the inlet.

12. A more hydraulically efficient manhole design is shown in the 251D-1 Standard Construction Details. This design (which does not necessarily require brick or tile construction) should be utilized wherever possible. The design extends the pipe through the manhole but provides a leaveout in the top half of the pipe (above the "spring line").

3.6.6 Tree Box Filters

Tree box filters are small bioretention areas located on streetscapes or in parking lots. Stormwater runoff is diverted into the box by a curb cut where it filters through an engineered growing medium while also providing water for the tree. Suspended solids, nutrients, metals, oil and grease, and debris can be removed as the stormwater filters through the medium. The overflow from the box is connected to the storm sewer system or can be captured and used for further irrigation.

Tree-box filters should be located upstream of stormwater inlets to allow for bypass and underdrains to enter the downstream inlet and should be set back a minimum of 3 feet from buildings and property lines.

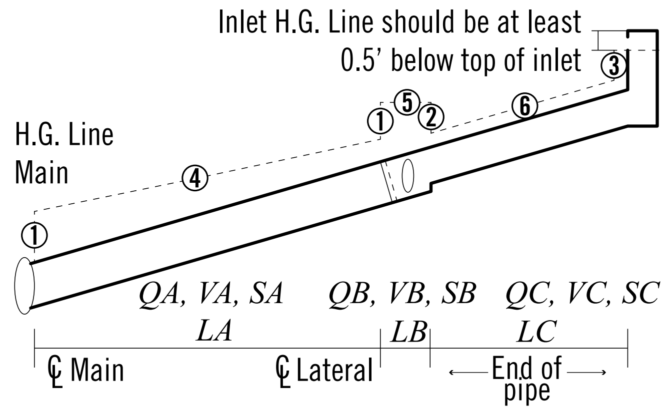
Figure 3.17 Head Losses in Laterals, Wyes, and Enlargements

Head Losses & Gains for Laterals

① $H_L = \frac{V_2^2}{2g} - \frac{V_1^2}{2g}$ ② $H_L = \frac{V_2^2}{4g} - \frac{V_1^2}{4g}$

③ $H_L = \frac{1.5\ V_2^2}{2g}$ ④ $H_L = LA * S_f A$

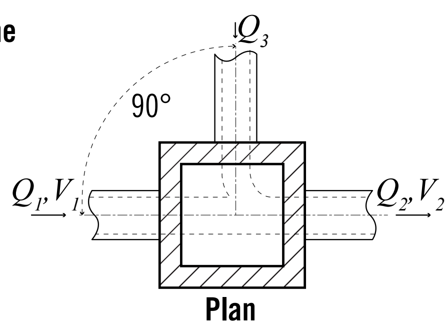
⑤ $H_L = LB * S_f B$ ⑥ $H_L = LC * S_f C$

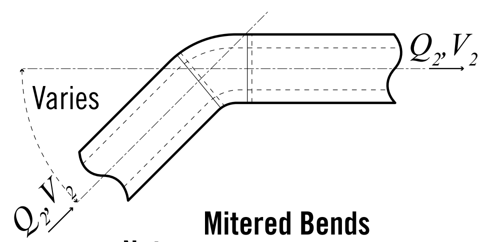


(see enlarged figure in Appendix A.7)

Figure 3.18 Head Losses for Junction Boxes, Manholes, and Bends

90° Lateral
Manhole on Main Line



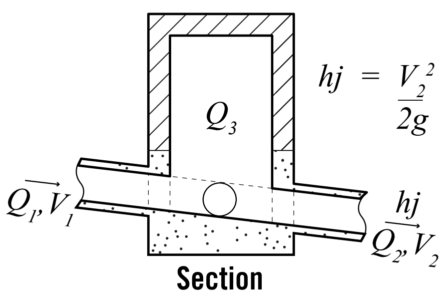


Note
60° Lateral

$$hj = \frac{V_2^2}{2g} - \frac{0.35\ V_1^2}{2g}$$

45° Lateral

$$hj = \frac{V_2^2}{2g} - \frac{0.5\ V_1^2}{2g}$$



$$hj = \frac{V_2^2}{2g} - \frac{0.25\ V_1^2}{2g}$$

Note

Head loss applied at beginning of bend. Bends to be used only with the permission of the drainage engineer.

90° Bend

$$hj = 0.80\ \frac{V_2^2}{2g}$$

60° Bend

$$hj = 0.60\ \frac{V_2^2}{2g}$$

45° Bend

$$hj = 0.50\ \frac{V_2^2}{2g}$$

30° Bend

$$hj = 0.45\ \frac{V_2^2}{2g}$$

(see enlarged figure in Appendix A.7)

3.7 CULVERT HYDRAULIC DESIGN

3.7.1 General

The function of a culvert is to convey surface water under a roadway, railroad, or other embankment. In addition to the hydraulic function, the culvert must carry construction, roadway, railroad, or other traffic and earth loads.

The material selected for a culvert depends upon various factors, including durability, structural strength, roughness, bedding condition, abrasion and corrosion resistance, and water tightness. The most common culvert material used is Class III reinforced concrete.

Another factor that significantly affects the performance of a culvert is its inlet configuration. The culvert inlet may consist of a culvert barrel projecting from the roadway fill or mitered to the embankment slope. Other culvert inlets have headwalls, wingwalls, and apron slabs or standard end sections of concrete.

A careful approach to culvert design is essential, both in new land development and retrofit situations, because culverts often significantly influence upstream and downstream flood risks, floodplain management, and public safety.

3.7.2 Design Frequency

The culvert(s) shall be designed for the 1% annual chance storm event. Channels upstream and downstream of culverts must contain the design storm and freeboard. Culverts shall not increase the water surface elevation of the 1% annual chance storm event flow.

Freeboard, the vertical clearance between the design water surface and the top-of-curb elevation, is included as a safety factor in the event of clogging of the culvert. Two feet of freeboard above the 1% AEP storm event water surface is required.

Culverts shall be designed using the approved list of programs provided on the City of Dallas website or Federal Highway Administration (FHWA) nomographs in Hydraulic Design of Highway Culverts, Third Edition (HDS 5). The remainder of this section provides information necessary for the hydraulic design of culverts.

If designing a culvert using HEC-RAS, please refer to Section 4.1 for guidance on modeling structures.

3.7.3 Hydraulic Controls

3.7.3.1 Inlet Control

Inlet or entrance control occurs when a culvert is capable of carrying more flow than the inlet will accept and the culvert is hydraulically steep (critical depth is greater than normal depth).

When the culvert is under inlet control, the control section is just inside the entrance of the culvert. Hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity. The inlet geometry, including barrel shape, cross-sectional area, and the inlet edge condition represent the major flow controls. If the flow of the culvert is a free surface flow, then critical depth will occur at or near the control section. Downstream of the control section and a free surface flow, the flow will be supercritical and a hydraulic jump may occur within the culvert.

The computation of headwater depth, the depth from the culvert inlet invert to the energy grade line, is complex and dependent upon the type of culvert and the ratio of the headwater depth to the culvert height. The TxDOT Hydraulic Design Manual provides a detailed description of inlet control computations for various conditions.

3.7.3.2 Outlet Control

Outlet or exit control occurs when a culvert is not capable of carrying as much flow as the inlet will accept.

When the culvert is under outlet control, the hydraulic grade line inside the culvert at the entrance exceeds critical depth. The headwater of a culvert with outlet control is determined by the frictional slope, entrance and exit geometry, and tailwater level.

$$HW_{OC} = h_e + h_{vi} + \sum h_f - S_O L + H_O - h_{va}$$

(Equation 3.34)

HW_{OC} = headwater depth due to outlet control (ft)

h_e = entrance head loss (ft)

h_{vi} = velocity head in the entrance (ft)

h_f = frictional head losses (ft)

S_O = culvert slope (ft/ft)

L = culvert length (ft)

H_O = depth of hydraulic grade line just inside the culvert at the outlet (ft)

h_{va} = velocity head of flow approaching the culvert entrance (ft)

$$h_v = \left[\frac{V^2}{2g}\right] \quad \textit{(Equation 3.35)}$$

h_v = velocity head (ft)

v = velocity (ft/s)

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

When tailwater controls, the following formula includes the exit loss.

$$H_O = TW + h_{TW} + h_o - h_{VO} \quad \textit{(Equation 3.36)}$$

H_O = water surface at outlet (ft above datum)

TW = tailwater depth (ft above datum)

h_{TW} = velocity head in tailwater (ft)

h_O = exit head loss (ft)

h_{VO} = velocity head inside culvert at outlet (ft)

The outlet depth, H_O is the hydraulic grade line inside the culvert outlet. The conditions in Table 3.4 will determine the outlet depth.

Table 3.4 Outlet Depth Conditions

If	And	Then
Tailwater depth (TW) exceeds critical depth (d_c) in the culvert at outlet	Slope is hydraulically mild	Set H_o using Equation 3.36, using the tailwater as the basis
Tailwater depth (TW) is lower than critical depth (d_c) in the culvert at outlet	Slope is hydraulically mild	Set H_o as critical depth
Uniform depth is higher than top of the barrel	Slope is hydraulically steep	Set H_o as the higher of the barrel depth (D) and depth using Equation 3.36.
Uniform depth is lower than top of the barrel & tailwater exceeds critical depth	Slope is hydraulically steep	Set H_o using Equation 3.36
Uniform depth is lower than top of the barrel & tailwater is below critical depth	Slope is hydraulically steep	Ignore, as outlet control is not likely

3.7.4 Energy Gradient Analyses

3.7.4.1 Energy Loss Through Culvert

There are four (4) different flow conditions that occur within the culvert:

- Type A – Free Surface Flow
- Type B – Full Flow in Conduit
- Type BA – Full Flow at outlet and free surface flow at inlet, and
- Type AB – Free Surface Flow at outlet and full flow at inlet.

These conditions are further explained on the following pages and in Figures 3.19 – 3.22.

Type A – Free Surface Flow

With a free surface flow occurring in the culvert a standard step backwater analysis can be used to calculate the water surface through the culvert to the entrance. With this condition the backwater profile is based on the outlet depth. Normal depth occurs in the culvert.

The headwater may be affected only when the culvert is in subcritical flow, backwater from the culvert outlet is present, and if the culvert is on a steep slope with a tailwater higher than critical depth and lower than the soffit of the culvert outlet.

Figure 3.19 Outlet Control Headwater for Culvert with Free Surface

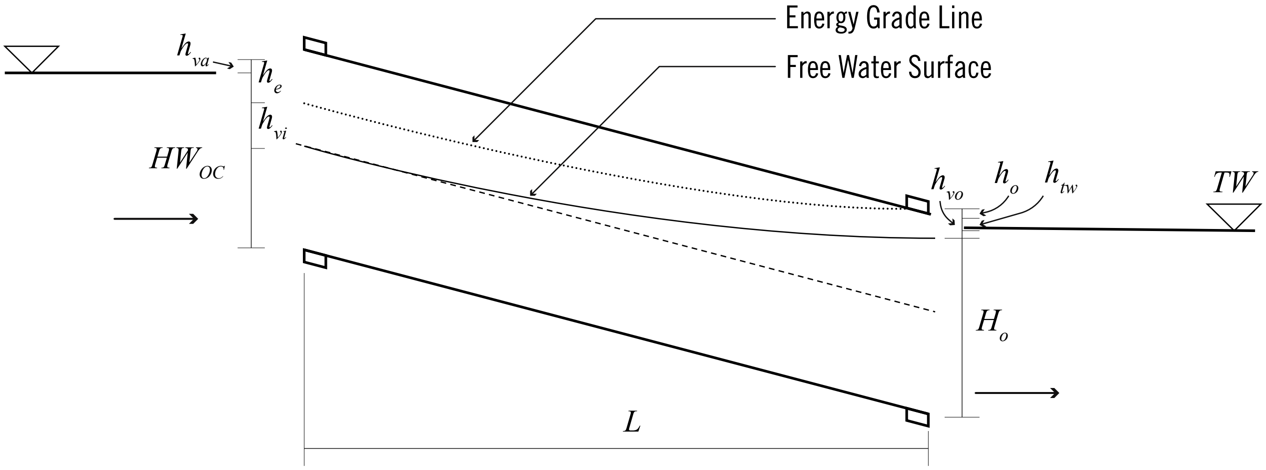


Figure 3.20 Outlet Control, Fully Submerged Flow

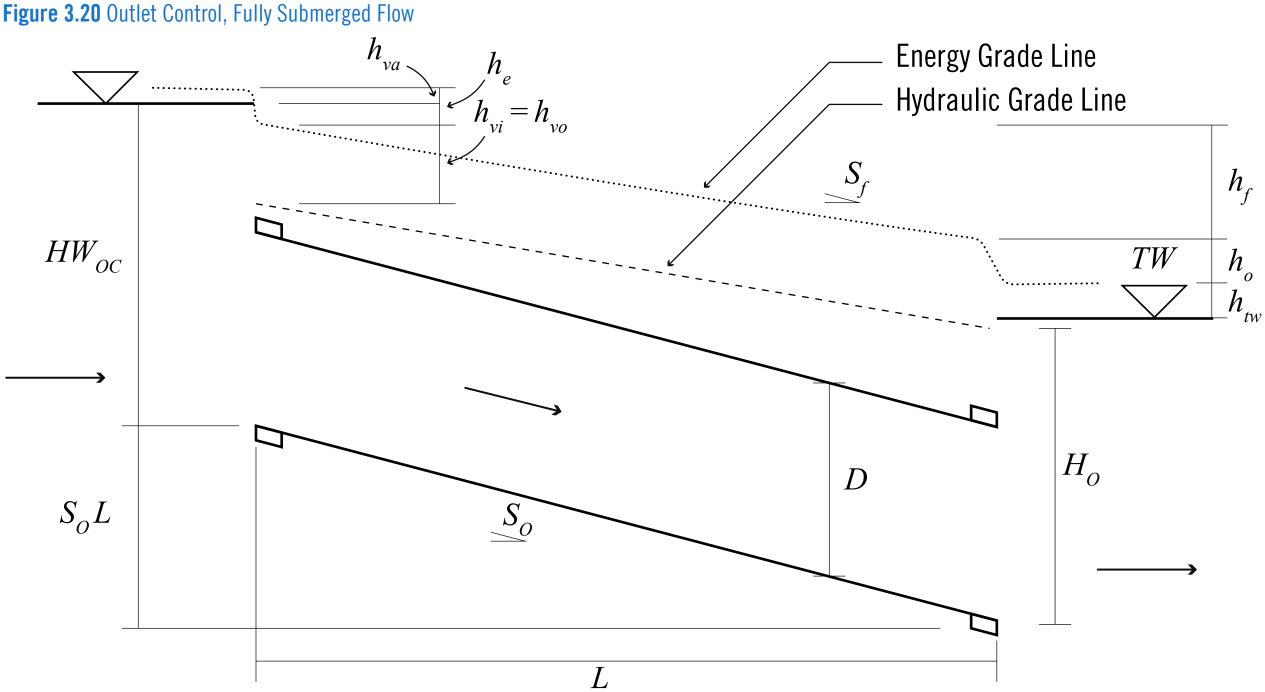


Figure 3.21 Point at Which Free Surface Flow Begins

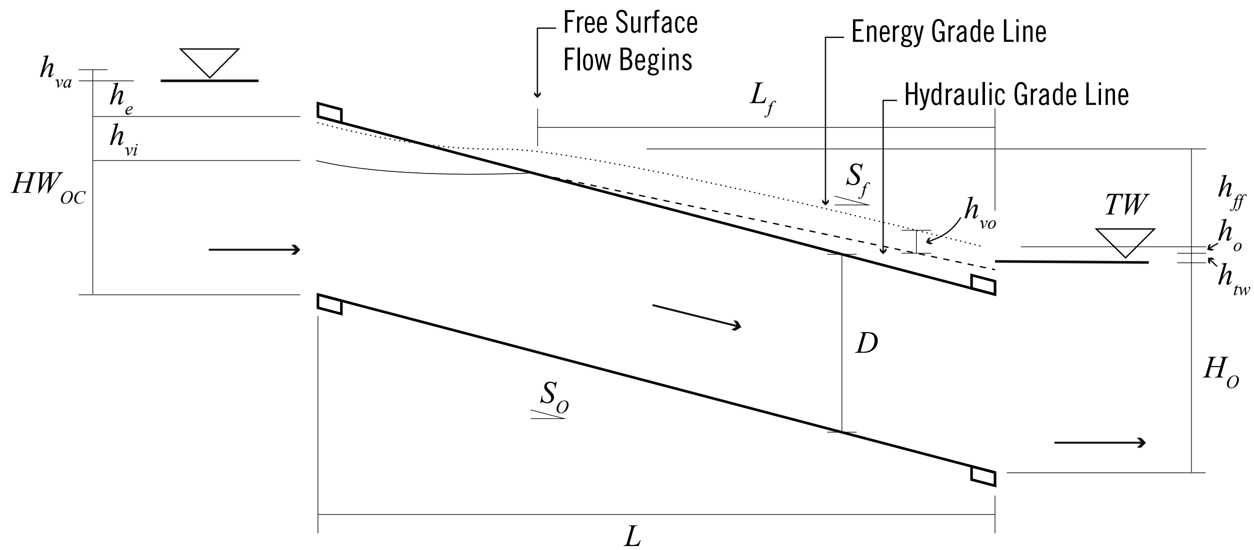
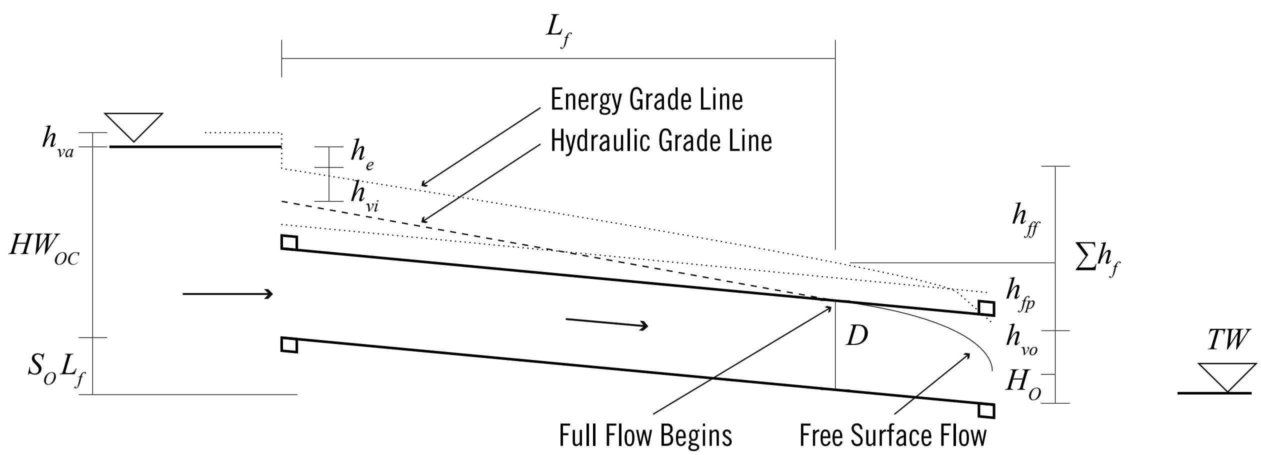


Figure 3.22 Headwater due to Full Flow at Inlet and Free Surface at Outlet



Type B – Full Flow in Conduit

If the full flow condition exists within the length of the culvert then the hydraulic grade line will be at or above the soffit. The hydraulic grade line at the culvert outlet is based on the outlet depth (H_O) being at or above the soffit at the outlet.

Use Equation 3.39 to calculate the frictional slope of the culvert. If the friction slope is less than the culvert slope, the hydraulic grade line may drop below the soffit of the culvert. If this condition occurs, then the culvert flow may be Type BA, discussed in the following section.

The frictional loss through the culvert is determined by Equation 3.37. To determine the hydraulic grade line at the upstream end of the culvert, at the inlet use Equation 3.38. To obtain the headwater elevation the entrance loss will need to be calculated.

$$h_f = S_f L \quad \textit{(Equation 3.37)}$$

h_f = head loss due to friction in the culvert barrel (ft)

S_f = friction slope (ft/ft)

L = length of culvert containing full flow (ft)

$$H_i = H_O + h_f - S_O L \quad \textit{(Equation 3.38)}$$

H_i = depth of hydraulic grade line at inlet (ft)

H_O = outlet depth (ft)

h_f = friction head losses (ft)

S_O = culvert slope (ft/ft)

L = culvert length (ft)

$$S_f = \left(\frac{Qn}{1.486 R^{2/3} A} \right)^2 \quad \textit{(Equation 3.39)}$$

S_f = friction slope (ft/ft)

Q = flow in pipe (cfs)

n = Manning's 'n'-value

R = hydraulic Radius (A / wetted perimeter) (ft)

A = area of the pipe (ft^2)

Type BA – Full Flow at Outlet and Free Surface Flow at Inlet

If the frictional slope is less than the culvert slope and the outlet depth (H_O) is greater than the soffit of the culvert at the outlet then the culvert may flow full for a portion of its length.

First determine the length of full flow using Equation 3.40.

$$L_f = \frac{H_O - D}{S_O - S_f} \quad \textit{(Equation 3.40)}$$

L_f = length over which full flow occurs (ft)

H_O = outlet depth (ft)

D = conduit barrel height (ft)

S_O = culvert slope (ft/ft)

S_f = friction slope (ft/ft)

Should the length L_f be greater than the culvert length, the culvert is flowing full for its entire length, see Type B calculations. If the length of L_f is less than the culvert length, a free surface flow begins a point along the culvert at a distance of L_f from the culvert outlet. From this point up to the culvert inlet the water surface can be calculated using the standard step backwater method.

With the water surface (H_i) or d_i (shown on Figure 3.22 below) at the culvert inlet, the headwater elevation at the entrance can be calculated.

Type AB – Free Surface at Outlet and Full Flow at Inlet

If the frictional slope is greater than the culvert slope and the outlet water surface H_o is less than the culvert soffit at the outlet, calculate the H_i using the following steps.

Step 1 – Start with the outlet depth H_o.

Step 2 – Use a standard step backwater analysis to determine the point along the conduit where the water surface will intersect the soffit.

Step 3 – At this point along the culvert length, the remaining culvert length L_f is substituted for L in the Equation 3.37 to determine h_{ff} in Figure 3.22.

Step 4 – With the hydraulic grade line at the culvert inlet, the headwater elevation at the entrance can be calculated.

3.7.4.2 Energy Balance at Inlet

The culvert inlet headwater (HW_{OC}) can be calculated using Equation 3.41. With the hydraulic grade line at the culvert entrance, H_i, the velocity head at the entrance (h_{vi}) can be calculated.

$$HW_{OC} = H_i + h_{vi} + h_e - h_{va} \quad \textit{(Equation 3.41)}$$

HW_{OC} = headwater depth due to outlet control (ft)

H_i = depth of hydraulic grade line immediately inside the culvert at inlet (ft)

h_{vi} = velocity head in the entrance (ft)

h_e = entrance head loss (ft)

h_{va} = velocity head of flow approaching the culvert entrance (ft)

Generally the approach velocity of the upstream channel to the culvert inlet can be assumed to be zero (0), thus the headwater and energy grade line are equal. This is a conservative approach for a headwater depth. The design engineer can calculate the approach velocity and determine the appropriate headwater.

The entrance loss should be calculated using Equation 3.42.

$$h_e = C_e\left[\frac{V_i^2}{2g}\right] \quad \textit{(Equation 3.42)}$$

h_e = entrance loss

C_e = entrance loss coefficient (see Table 3.5)

V_i = flow velocity inside culvert inlet (ft/s)

g = the acceleration due to gravity (32.2 ft/s^2)

Table 3.5 Entrance Loss Coefficients

Type of Structure and Design of Entrance	**Coefficient** C_o
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.3
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square edge	0.5
Rounded (radius = D/12)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tape red inlet	0.2
Pipe or Pipe-Arch Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tape red inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

3.7.4.3 Determination of Outlet Velocity

The outlet velocity is based on the discharge and the cross sectional area at the outlet.

$$V_O = \frac{Q}{A_O} \quad \textit{(Equation 3.43)}$$

V_O = outlet velocity (ft/s)

Q = culvert discharge (cfs)

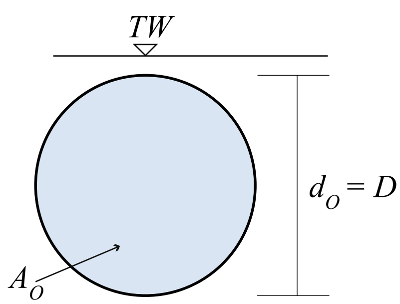
A_O = cross-sectional area of flow at outlet (ft^2)

There are a few conditions to consider for determining the depth (d_O) at the outlet.

If the tailwater at the outlet is above the culvert outlet soffit or the culvert is flowing full because the culvert capacity is less than the discharge, then the depth (d_O) is equal to the barrel rise (D) and the full cross sectional area of the culvert is used. See Figure 3.23.

If the tailwater at the outlet is below the culvert outlet soffit, determine the critical depth of the culvert. Set the depth (d_O), to the higher of tailwater or critical depth.

Figure 3.23 Cross Sectional Area Based on Full Flow



3.7.4.4 Depth Estimation Approaches

For inlet control under steep slope conditions, estimate the depth at the outlet using one of the following approaches:

Use a standard step backwater method starting from critical depth (d_C) at the inlet and proceed downstream to the outlet. If the tailwater is lower than critical depth at the outlet, calculate the velocity resulting from the computed depth at the outlet. If the tailwater is higher than critical depth, a hydraulic jump within the culvert is possible. Section 3.7.5 discusses a means of estimating whether the hydraulic jump occurs within the culvert. If the hydraulic jump does occur within the culvert, determine the outlet velocity based on the outlet depth, $d_O = H_O$.

Assume uniform depth at the outlet. If the culvert is long enough and tailwater is lower than uniform depth, uniform depth will be reached at the outlet of a steep slope culvert. For a short, steep culvert with tailwater lower than uniform depth, the actual depth will be higher than uniform depth but lower than critical depth. This assumption will be conservative; the estimate of velocity will be somewhat higher than the actual velocity. If the tailwater is higher than critical depth, a hydraulic jump is possible and the outlet velocity could be significantly lower than the velocity at uniform depth.

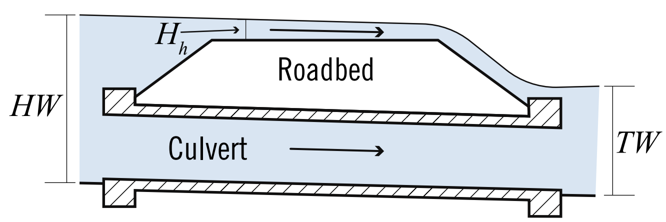
3.7.4.5 Direct Step Backwater Method

The free flow water surface water within a culvert can be determined with the Direct Step Method described in TxDOT Hydraulic Design Manual. An incremental water depth (d) is chosen and the corresponding distance over which the depth of change is computed. This method can be used for either supercritical or subcritical flow within a culvert.

3.7.4.6 Roadway Overtopping

When the calculation of the culvert headwater, assuming the total discharge passes through the culverts, is above the low point of the roadway, a weir condition will develop and roadway overtopping will occur, illustrated in Figure 3.24.

Figure 3.24 Culvert with Overtopping Flow



The calculation of the amount of flow that passes through the culvert and the remaining portion of flow that overtops the roadway is an iterative process.

Use Equation 3.44 to determine the average depth between headwater and low roadway elevation (H_h) for the roadway.

$$Q = k_t \, C L H_h^{1.5} \quad \textit{(Equation 3.44)}$$

Q = discharge (cfs)

k_t = over-embankment flow adjustment factor (see Fig 3.25)

C = discharge coefficient (3.0 for roadways)

L = horizontal length of overflow (ft)
(this length should be perpendicular to the overflow direction)

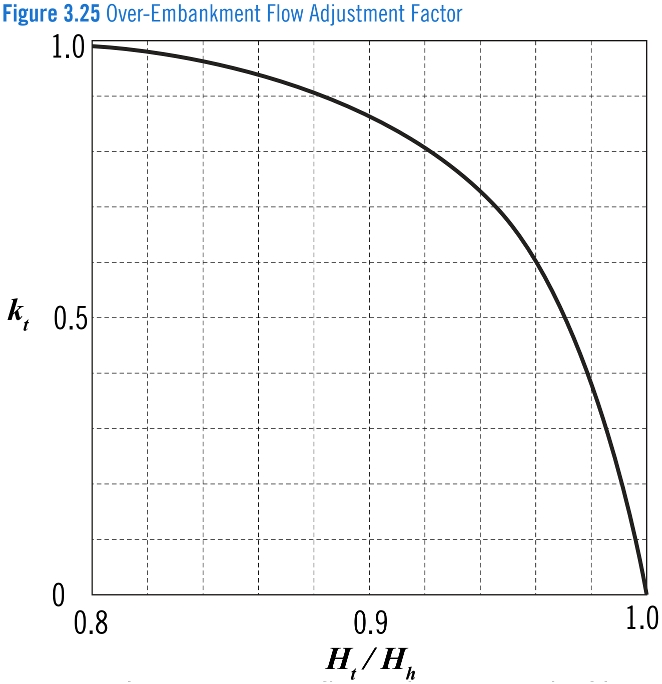
H_h = average depth between headwater and low roadway elevation (ft)

H_t = average depth between tailwater and low roadway elevation (ft)

If the tailwater is sufficiently high, the adjustment factor k_t, determined by Figure 3.25, would reduce the discharge over the roadway. For values of H_t / H_h below 0.8, the adjustment factor k_t is one (1). Roadway embankments, as shown in Figure 3.26, may need to be broken down into segments for the computation of the weir flow.

The use of HEC-RAS or other approved models can be used to determine the flow through the culvert and over the roadway.

Figure 3.25 Over-Embankment Flow Adjustment Factor



(see enlarged figure in Appendix A.7)

Figure 3.26 Roadway Overtopping with High Tailwater

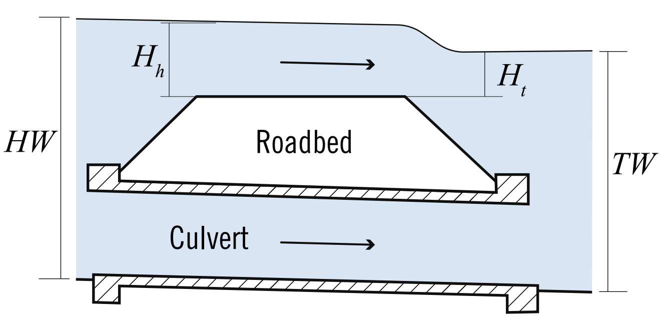
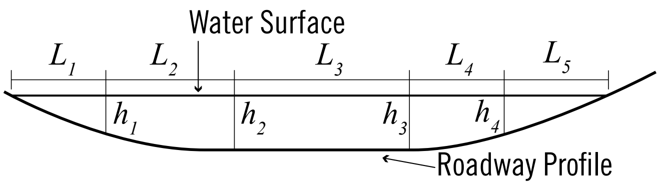


Figure 3.27 Cross Section of Flow Over Roadway



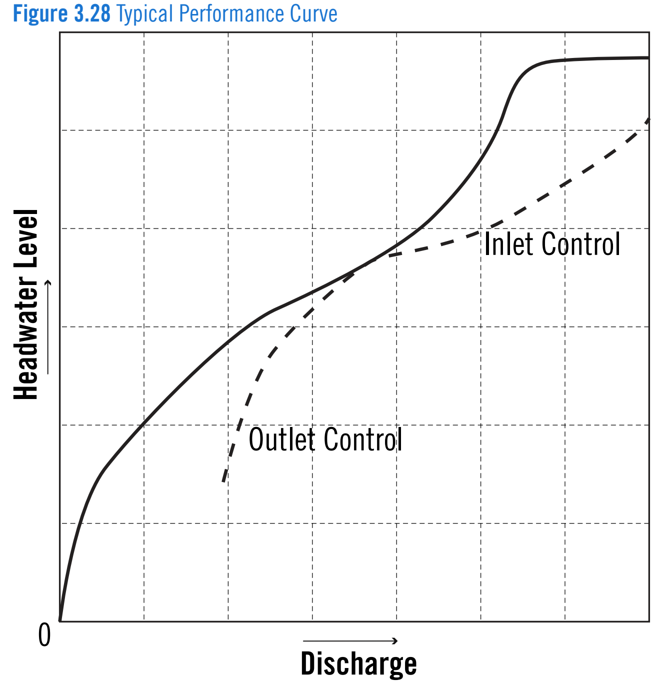
3.7.4.7 Performance Curves

The performance curve is a combination of inlet and outlet control that will vary with the discharge.

A sample plot of the headwater versus discharge for inlet and outlet control of a culvert is shown in Figure 3.28.

With varying discharge the culvert system may change from inlet control to outlet control. This information is useful for a risk assessment or routing a hydrograph through a detention basin with a culvert outlet.

Figure 3.28 Typical Performance Curve



3.7.4.8 Exit Loss Considerations

An exit loss should be considered at the hydraulic interface between the tailwater and the culvert outlet. The exit loss coefficient varies from 0.5 to 1. The starting hydraulic grade line elevation (H_o) at the interface between the outside and inside of the culvert outlet is based on Equation 3.45.

$$H_O = TW + \frac{V_{TW}^2}{2g} + h_O - \frac{V_O^2}{2g} \quad \textit{(Equation 3.45)}$$

H_O = outlet depth - depth from the culvert flow line to the hydraulic grade line inside the culvert at the outlet (ft)

V_O = culvert outlet velocity (ft/s)

V_{TW} = velocity in outfall (tailwater velocity) (ft/s)

h_O = exit loss (ft)

$$h_O = K \frac{V_O^2 - V_{TW}^2}{2g} \quad \textit{(Equation 3.46)}$$

K = loss coefficient (1.0 for exit losses)

g = acceleration due to gravity (32 ft/s^2)

3.7.5 Flow Regime Determination

3.7.5.1 Subcritical Flow and Steep Slope

If the culvert has a free water surface with a subcritical flow at the outlet and the culvert has a hydraulically steep slope, then the water depth d is negative in the computation (decrement). If the depth of flow reaches critical depth before reaching the culvert entrance, then the culvert is under inlet control. A hydraulic jump may occur in the culvert. If the depth of flow calculated at the culvert entrance is higher than the culvert critical depth, use Equation 3.47.

3.7.5.2 Supercritical Flow and Steep Slope

If the culvert has supercritical flow and a steep slope, then begin the computation starting at the culvert entrance with critical depth and proceed downstream for the water surface computation. Use a decrement water depth d in the computation. If the tailwater is higher than the culvert critical depth a backwater may occur within the culvert.

3.7.5.3 Hydraulic Jump in Culverts

An example of a momentum and energy plot is shown in Figure 3.29. For a given discharge there are two possible depths; the first is less than critical depth (supercritical flow) and the other is greater than critical depth (subcritical flow) at a sequent (or conjugate) depth. Both depths will have the same momentum with different specific energy. If you have a supercritical flow in a culvert, the possibility of hydraulic jump can occur with the proper configuration. There will be a loss in energy, ΔE as a result of the hydraulic jump.

$$M = \frac{Q^2}{gA} + Ad \quad (Equation\ 3.47)$$

M = momentum function (ft^2)

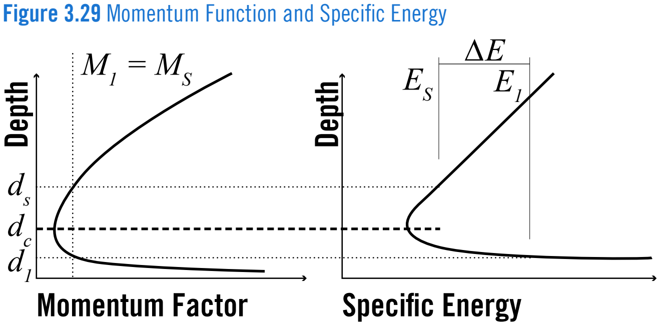
Q = discharge (cfs)

g = acceleration due to gravity (32.2 ft/s^2)

A = cross-sectional flow area (ft^2)

d = distance from water surface to centroid of flow area (ft)

Figure 3.29 Momentum Function and Specific Energy



1 = flow depth

C = critical depth

S = sequent depth

3.7.5.4 Sequent Depth

If the culvert has a free surface flow and is supercritical, sequent depth can be calculated. For slopes greater than ten percent (10%) a more complex solution is required and is provided in FHWA HEC-14 “Hydraulic Design of Energy Dissipators”.

To determine sequent depth within a rectangular culvert, use Equation 3.48.

$$d_s = 0.5\, d_1\left(\sqrt{1 + \frac{8 v_1^{\,2}}{g d_1}} - 1\right) \quad (Equation\ 3.48)$$

d_s = sequent depth (ft)

d_1 = depth of flow (supercritical) (ft)

v_1 = velocity of flow at depth d_1 (ft/s)

For a circular culvert the calculation to determine sequent depth is not a direct solution. Flowmaster or other approved methods shall be used to determine the depth of flow, d. Figure 3.30 and the following equations and nomograph in Figure 3.31 can be used to determine the sequent depth, y_2.

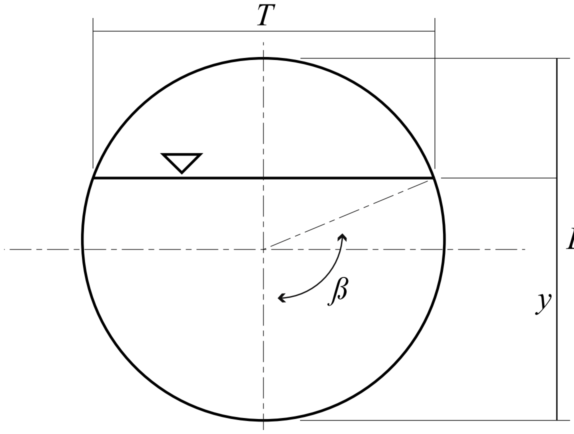
3.7.6 Other Culvert Design Considerations

A minimum height of 3 feet shall be maintained in box culverts for maintenance purposes. Access should be available for maintenance at the upstream and downstream ends of culverts. If the length of the culvert is greater than 100 feet, the minimum height shall be increased to 6 feet, or access manholes shall be placed such that access is provided at a maximum spacing of 500 feet.

To minimize the undesirable backwater effects and erosive conditions produced where the total width of box culverts is less than the bottom width of the channel, a transition upstream and downstream of the culverts must be provided. The transition should have a minimum bottom width transition of 2 to 1 and include warping of side slopes as required. The 2 to 1 transition is 2 along the centerline of the channel and 1 perpendicular to the centerline.

Where multiple box configurations of 3 or more boxes are utilized, sedimentation often occurs within some of the boxes. This typically occurs because during low discharges, flow spreads out and slows down relative to upstream channel velocities. To prevent this occurrence, the 50% annual chance storm event velocity through the box culvert shall be a minimum of 3 ft/s. Additionally, the 50% annual chance storm event velocity within the culvert shall not be less than 50% of the velocity within the channel upstream of the culvert prior to the beginning of channel transition to the culvert. Deviations from this criterion that may be dictated

Figure 3.30 Determination of Sequent Depth



$$A=\frac{D^2}{8}\left[2\cos^{-1}\left(1-\frac{2d}{D}\right)-\sin\left(2\cos^{-1}\left(1-\frac{2d}{D}\right)\right)\right]$$ (Equation 3.49)

$$A'=\frac{A}{D^2}$$ (Equation 3.50)

$$T = D\sin\left(\cos^{-1}\left(\frac{2d-D}{D}\right)\right)$$ (Equation 3.51)

$$Fr_1=\frac{Q}{\sqrt{gD^5A_1'^3/T_1}}$$ (Equation 3.52)

$$d' = \frac{d}{D}$$ (Equation 3.53)

A = section area of flow (ft^2)

D = pipe diameter (ft)

d = depth of flow (ft)

Fr = Froude number

T = width of water surface (ft)

Q = discharge (cfs)

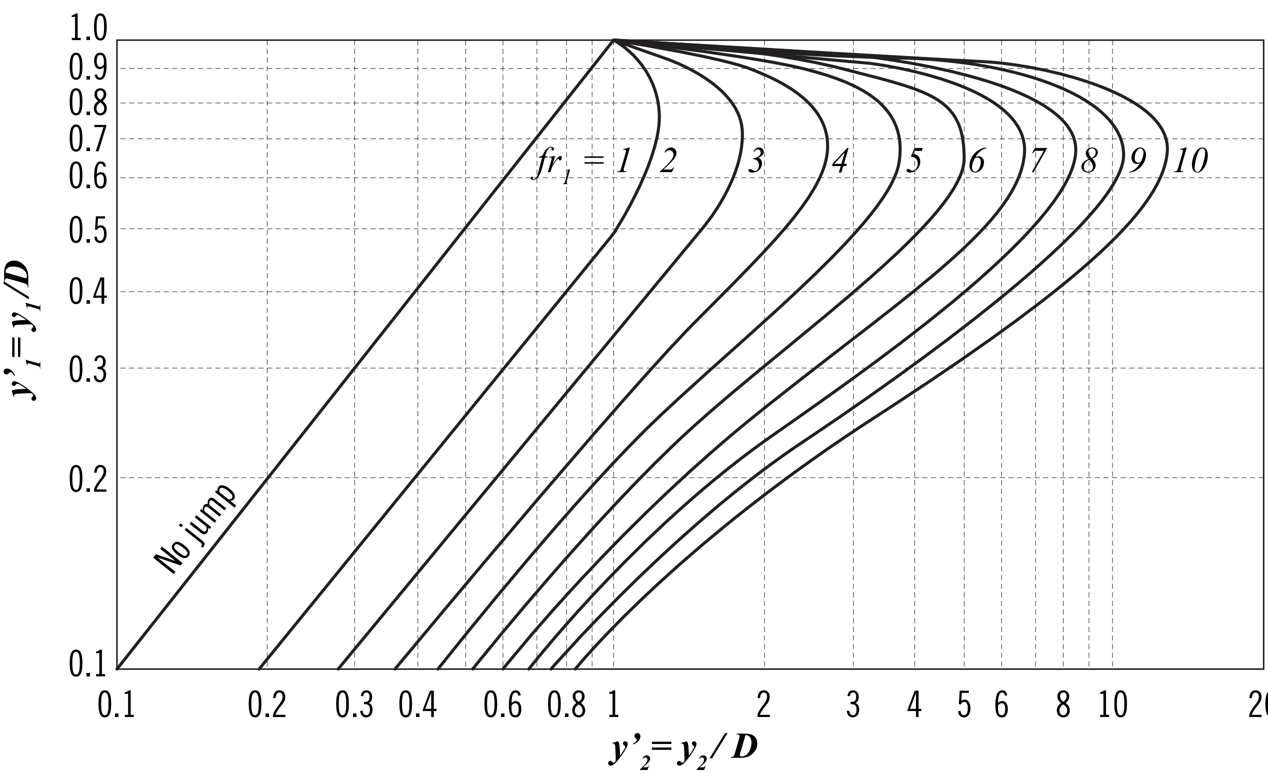
g = gravitational constant (32.2 ft/s^2)

Fr_1 = Froude number at supercritical flow depth

A_1 = area of flow at supercritical flow depth (ft^2)

T_1 = width of water surface at supercritical flow depth (ft)

Figure 3.31 Solution for Circular Conduits



by local site conditions must be approved in writing by the Director. Common methods for achieving the 50% annual chance storm event velocity requirement include placing only one box invert as low as the channel invert so that only one box is utilized during low flows, or placing a short training wall upstream of the boxes to direct low flows into a single box.

Culvert installations under high fills may present an opportunity to use a high headwater or ponding to attenuate flood peaks. The possibility of catastrophic failure should be investigated prior to considering deep ponding because a breach in fill could be quite similar to a dam failure. See Section 6 for detention design requirements.

Headwalls, with or without wingwalls and aprons, shall be designed to fit the conditions of the site and constructed according to the City of Dallas Standard Construction Details, 251D-1, or the TxDOT Details. The following are general guidelines governing the use of various types of headwalls:

1. Straight headwalls (Type A) should be used where the approach velocity in the channel is below 6 ft/s.
2. Headwalls with wingwalls and aprons (Type B) shall be used where the approach velocity is from 6 to 12 ft/s and downstream channel protection is recommended.
3. Special headwall and wingwall configurations will be required where approach velocities exceed 12 ft/s, and where the flow must be redirected in order to enter the culvert more efficiently.

While the immediate concern in the design of headwalls, wingwalls, and endwalls is hydraulic efficiency, consideration should also be given to the safety aspect of culvert end treatments. The use of flared and / or sloped end sections may enhance safety significantly, since the end section conforms to the natural ground surface. For any headwall adjacent to pedestrian traffic, either 6-gauge galvanized steel fencing (251D-1) or a guard rail shall be installed. Fence poles shall be set in concrete at the time of construction. For any headwall adjacent to vehicular traffic, a guard rail must be installed or safety end treatment designed per TxDOT standards.

3.8 OUTFALL DESIGN

3.8.1 General

Each outfall situation shall be considered individually. The following are examples of conditions to be considered when determining the need for energy dissipation:

1. Presence of, and elevation of, competent rock assessed by a geotechnical engineer as being capable of resisting erosion
2. Normal water surface elevation
3. Channel lining
4. Alignment of pipe to channel
5. Erodibility of channel
6. Downstream channel degradation that may undermine the proposed outfall

Creative approaches to engineering design of outfalls are encouraged in order to produce the most cost effective and environmentally acceptable system. As an example, if there is stable rock in a creek bottom, the system could outfall at the rock line. Or, if there is concrete channel lining, the pipe could be brought to the concrete at a reasonable grade.

The outfall for a storm drain system should discharge into a natural low, existing storm drainage system, or a channel. The start of the HGL for the storm drain system begins at the outfall. The design engineer should determine the tailwater for the downstream drain to find the impact on the proposed outfall.

A low tailwater elevation is typically the worst-case scenario for providing outfall protection for the prevention of erosion. Therefore, assumption of a 1% annual chance event HGL on the receiving stream may lead to an underestimate of the range of velocities expected at an outfall. For this reason, regardless of whether a coincident peak analysis is used for determining the starting HGL for an enclosed system hydraulic grade line calculation, Table 3.4 should be used for establishing the tailwater HGL for use in outfall design.

The discharge velocity from the outfall should not cause erosion to the downstream channel. Velocity controls should be used when erosion might occur in the downstream channel. The outfall of the storm drain should be positioned in the downstream channel in the downstream direction to reduce the turbulence and erosion. The design engineer should be aware of the types of soils at the outfall location and design accordingly.

Outfall protection is required for all outfalls to open channels where the outfall velocity exceeds the maximum allowable velocity per Table 5.1.

Provide headwalls for all storm sewer outfalls. Riprap is generally necessary around headwalls and is required where embankment slopes exceed 3:1.

3.8.2 Headwalls / Pipe End Treatment

The normal functions of properly designed headwalls are to anchor the culvert in order to prevent movement due to hydraulic and soil pressures, to control erosion and scour resulting from excessive velocities and turbulence and to prevent adjacent soil from sloughing into the waterway opening. All headwalls shall be constructed of reinforced concrete and may be either straight-parallel, flared or warped. They may or may not require aprons, as determined by site conditions. Headwalls should be aligned with the direction of the receiving flow when discharging into a waterway. Precast headwalls may be used if all other criteria are satisfied.

Safety end treatments should be considered. Refer to the TxDOT Roadway Design manual for a discussion of clear zone requirements. If safety end treatments are to be used, appropriate clogging factors must be applied. TxDOT Hydraulic Design Manual provides some guidance for the selection of clogging factors.

3.8.3 Energy Dissipators

Energy dissipators are used to eliminate the excess specific energy of flowing water. Effective energy dissipators must be able to slow the flow of fast moving water without damaging the structure or the channel below the structure. The City of Dallas recognizes the Bureau of Reclamation's publications on the Hydraulic Design of Stilling Basins and Energy Dissipators (1984) as an accepted reference for the design of energy dissipators for which design guidelines are not given in this manual.

Impact-type energy dissipators direct the water into an obstruction (baffle) that diverts the flow in many directions to reduce energy. A baffled outlet is one type of impact-type energy dissipator that can be used at outlet transitions with high velocities. Energy is dissipated through the impact as well as the resulting turbulence. Other impact-type energy dissipators include baffled aprons, splash shields, roughness rings, and outlet weirs. Impact-type energy dissipators should include a low-flow outfall. Impact-type energy dissipators are generally considered to be most effective for outfalls of enclosed storm drainage systems, spillways, or drop structures. They also tend to be smaller and more economical structures.

Other energy dissipators use the hydraulic jump to dissipate excess energy. In this type of structure, water moving in supercritical flow is forced into a hydraulic jump when it encounters a tailwater condition equal to the conjugate depth. Stilling basins are structures of this type where the flow plunges into a pool of water created by a weir or sill placed downstream of the outfall. Riprap basins are a common form of energy dissipation that consists of a riprap-lined scour hole. Riprap aprons, which also dissipate energy by hydraulic jump, are an economical option for energy dissipation and are typically used at transitions from culverts to overland sheet flow or channels.

All energy dissipators should be designed to facilitate maintenance. The design of outlet structures in or near parks or residential areas must give special consideration to appearance.

Design guidance is provided below for several of the more common types of energy dissipation/outfall protection. Creativity and use of other types of protection is encouraged. If other methods are utilized, calculations demonstrating the expected effectiveness of the methods are to be provided to the City of Dallas for review, along with any design references used in the design.

3.8.3.1 Riprap Aprons / Basins

Riprap Aprons

Flat riprap aprons are used for transitions from pipes or culverts to natural channels. Energy is dissipated by the roughness of the material and expansion of flow as the water hits and moves across the apron. The length, width, depth, and riprap size must be sufficient to fully dissipate the flow rather than move the concentrated flow and potential for erosion downstream.

The apron size and median riprap diameter, d_{50}, can be determined by the following methods. The appropriate nomograph (Figure 3.32 and Figure 3.33) should be used for minimum or maximum tailwater conditions, respectively. The riprap apron can be designed to minimum tailwater conditions if the tailwater flow elevation is lower than the centerline of the pipe, and if not, maximum tailwater conditions should be used. If this information is not known, the riprap apron should be sized for whichever is greater.

For pipes with full flow, the pipe diameter, d, and design discharge can be used to find the median riprap diameter, d_{50}, from the lower curves. Apron length, L_a, can be determined from the upper curves and using the appropriate nomograph (Figure 3.32 and Figure 3.33) apron width can be calculated.

For rectangular conduits or culverts, the depth of flow, d, and velocity, v, can be used to find the median riprap diameter, d_{50}, using the intersection of the lower set of curves and the apron length, L_a, from reading up to the upper curves on the appropriate nomograph and calculating apron width using conduit width for diameter in the equation.

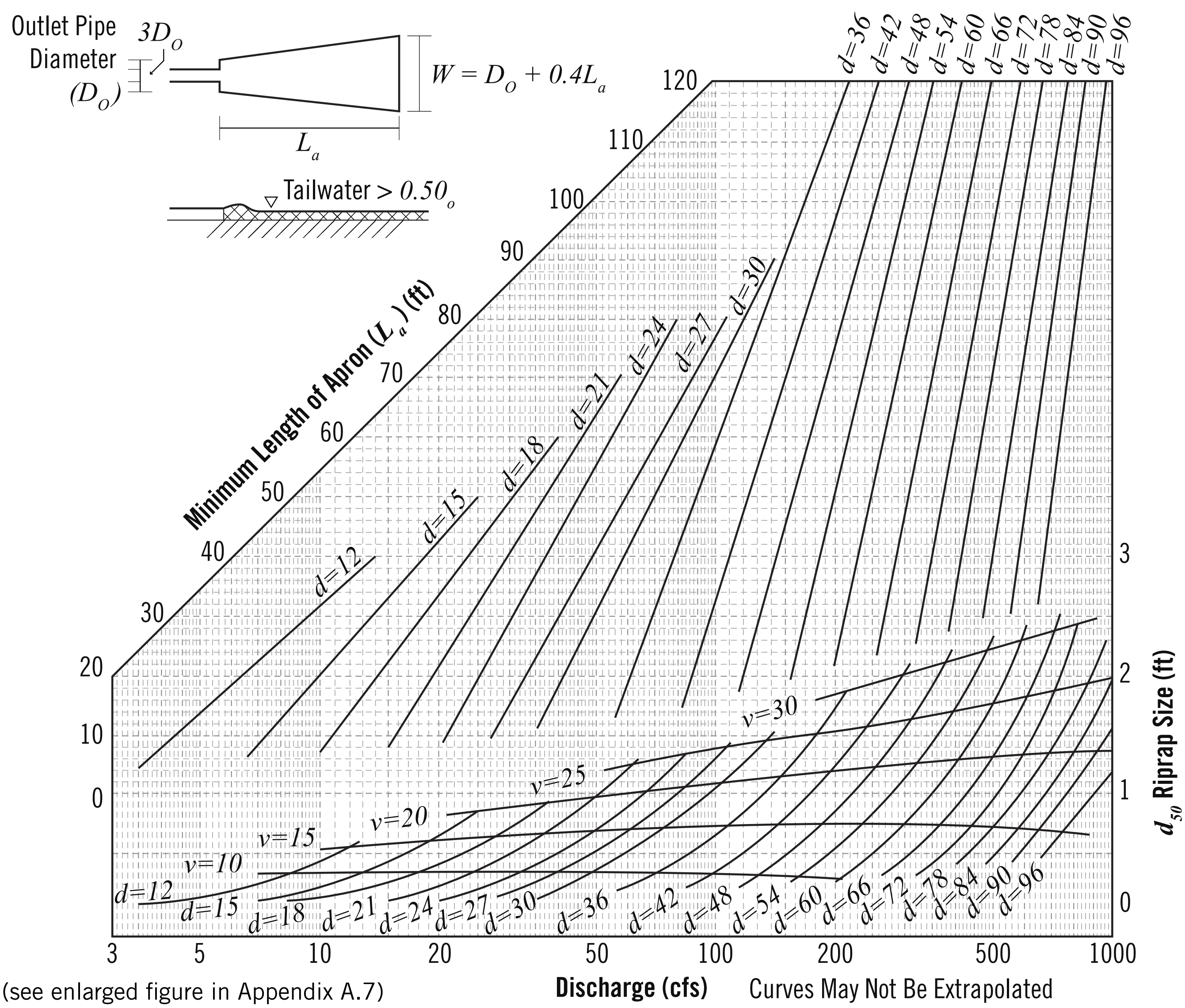
The maximum stone diameter should be 1.5 times the median riprap diameter. The thickness of the riprap should be a minimum of 1.5 times the maximum stone diameter or 6 inches, whichever is greater.

The width of the apron at the outlet should have a minimum width of the pipe or culvert at the outlet, and if connecting to an existing channel, the width at the end of the apron should match the channel width. Riprap should be placed on the sides of the apron and around the outlet at a maximum slope of 2:1. Protection should extend to stable slopes if steep slopes are present.

Riprap Basins

Riprap basins are pre-shaped depressions lined with riprap used to dissipate energy at stormwater outlets by creating hydraulic jumps. The mounded material at the downstream

Figure 3.32 Design of Riprap Apron under Minimum Tailwater Conditions



(see enlarged figure in Appendix A.7)

end of the basin serves as additional energy dissipation and protection against the scour hole enlarging.

Design factors include median riprap size, d_{50}, and the elevation of the basin below the culvert inlet, h_s, which is set to the approximate depth that a riprap pad would be scoured by the design discharge. The elevation difference between the outlet pipe and riprap pad, h_s, should be 2-4 times the median riprap size, d_{50}. The length of the energy dissipating pool should be the larger of $10*h_s$ or $3*W_0$, where W_0 is the diameter of the stormwater outlet. The overall length of the basin should be the larger of $15*h_s$ or $4*W_0$.

The ratio of tailwater depth to brink depth should be greater than 0.75. Figure 3.34 and Figure 3.35 can be used to determine the ratio of the brink depth, Y_0, to the height or diameter of the culvert, D.

Then, the velocity, V_0, can be found using the following equation. If the outlet is on a steep slope, V_0 should be determined using Manning's equation.

$$V_0 = Q/A \quad (Equation\ 3.53)$$

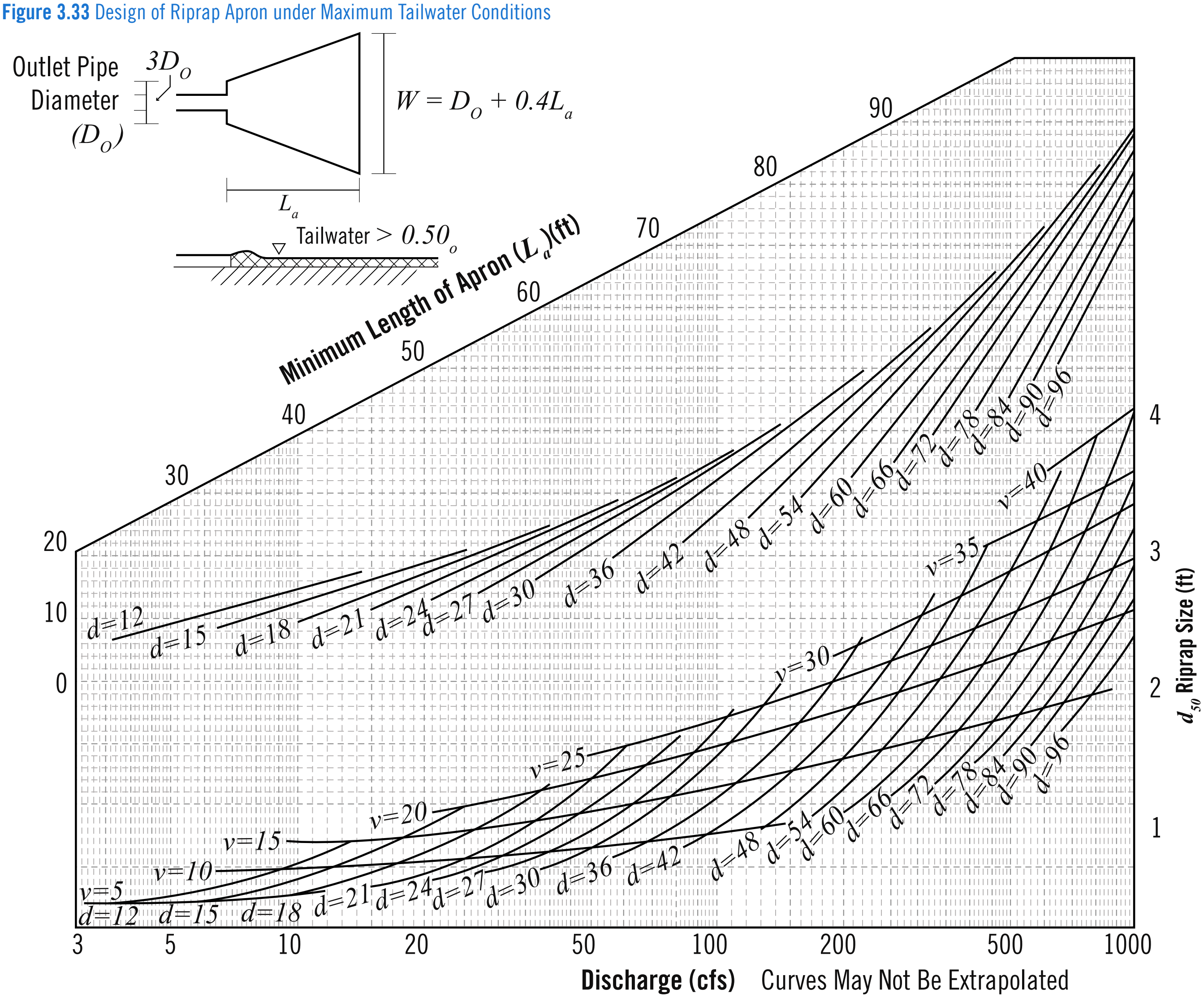
V_0 = velocity (ft/s)

Q = design discharge (cfs)

A = flow area associated with brink depth, Y_0 (ft^2)

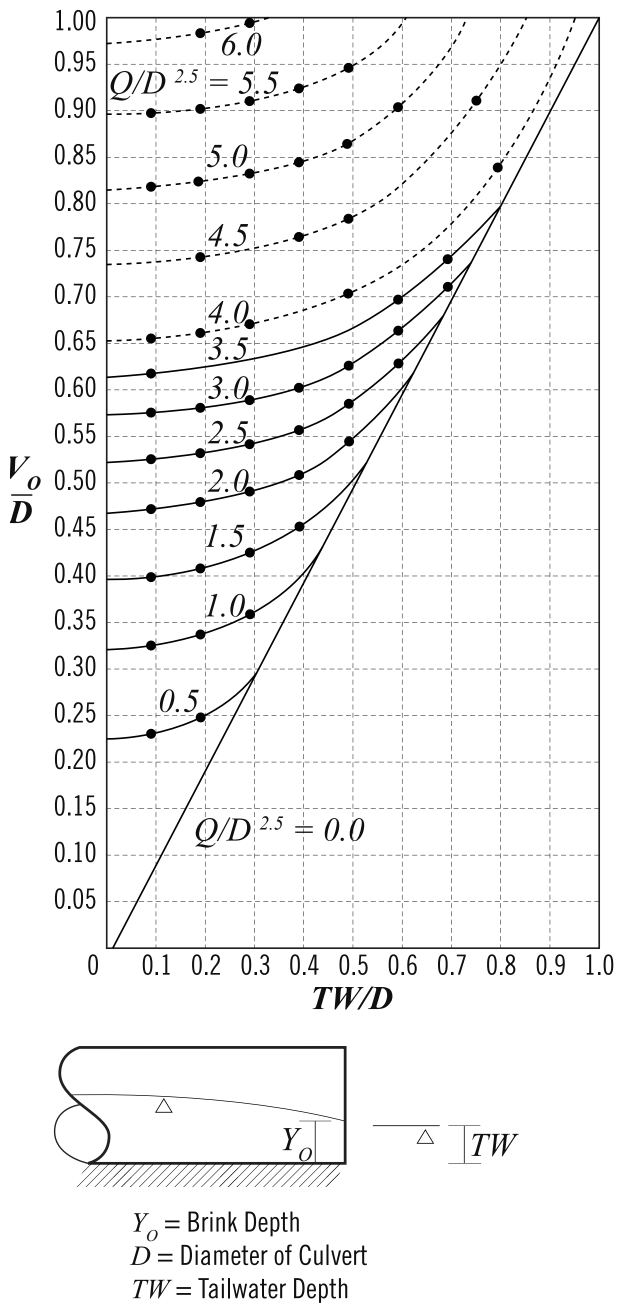
The Froude number can be computed using $y_e = y_0$ for box culverts and $y_e = (A/2)^{1/2}$ for circular pipes. Select an appropriate d_{50}/y_e based on what is available in the area. Using Figure 3.36, determine h_s/y_e. Check that $2 < h_s/d_{50} < 4$.

Figure 3.33 Design of Riprap Apron under Maximum Tailwater Conditions



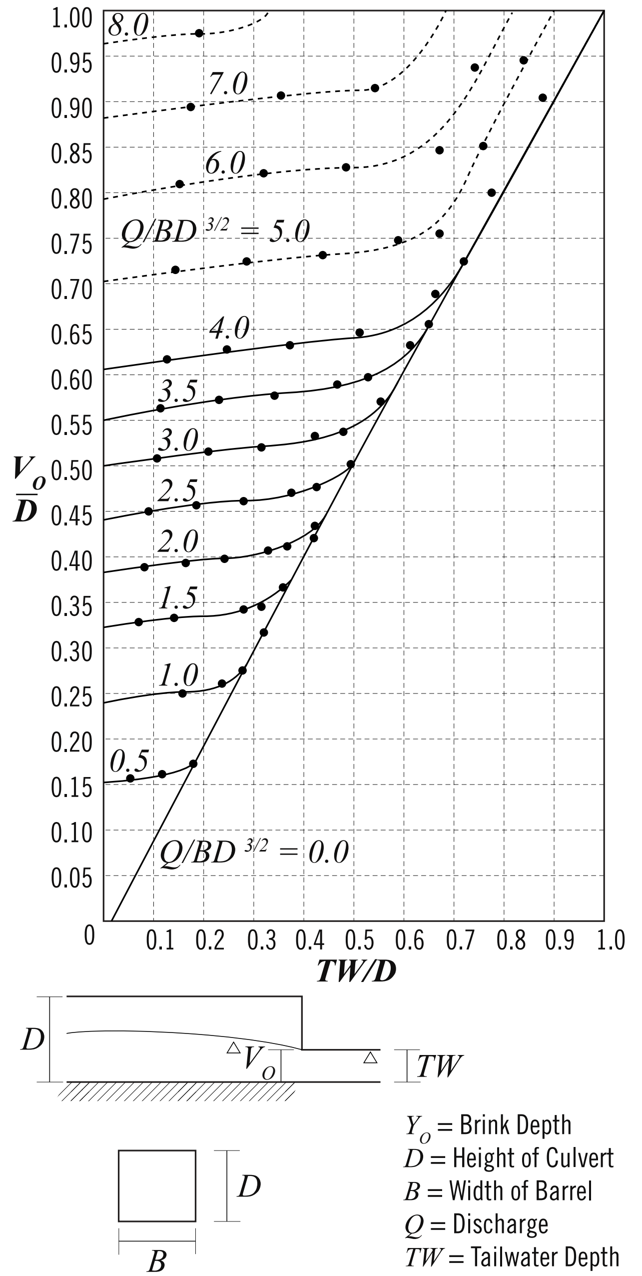
(see enlarged figure in Appendix A.7)

Figure 3.34 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes



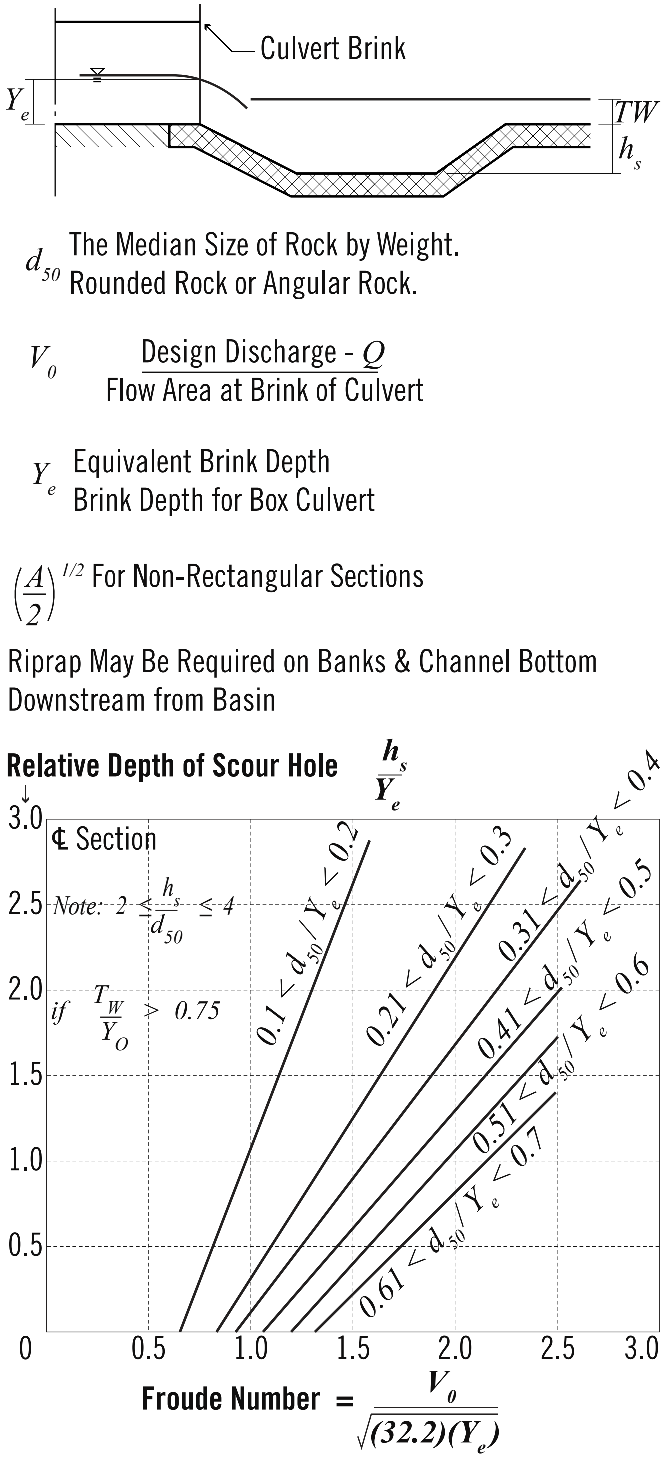
(see enlarged figure in Appendix A.7)

Figure 3.35 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes



(see enlarged figure in Appendix A.7)

Figure 3.36 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable



(see enlarged figure in Appendix A.7)

Determine dimensions of riprap basin based on Figure 3.36.

If an allowable exit velocity is required, determine the basin exit depth and exit velocity. Extend the length of the basin if needed. A smaller basin may be achieved by selecting a larger size of riprap.

If erosion is expected downstream, a riprap cutoff wall or sloping apron should be used. The walls and apron of the basin should be shaped to the natural channel at the outlet. If high tailwater conditions are expected $(TW / y_0 \geq 0.75)$, additional riprap should be used downstream to protect against high velocities. The scour hole may be shallower and longer if the stormwater outlet is discharging into high tailwater.

Flow downstream of an energy dissipator is generally more turbulent than in a typical channel reach. In lieu of the typical stone riprap design methodology presented in Section 7.4.4, the following equation (Searcy 1967) can be used for sizing riprap downstream of energy dissipators that have exit velocities exceeding the allowable channel velocity.

$$d_{50} = \frac{0.692}{S - 1}\left(\frac{V^2}{2g}\right) \quad \text{(Equation 3.54)}$$

d_{50} = median rock size (ft)

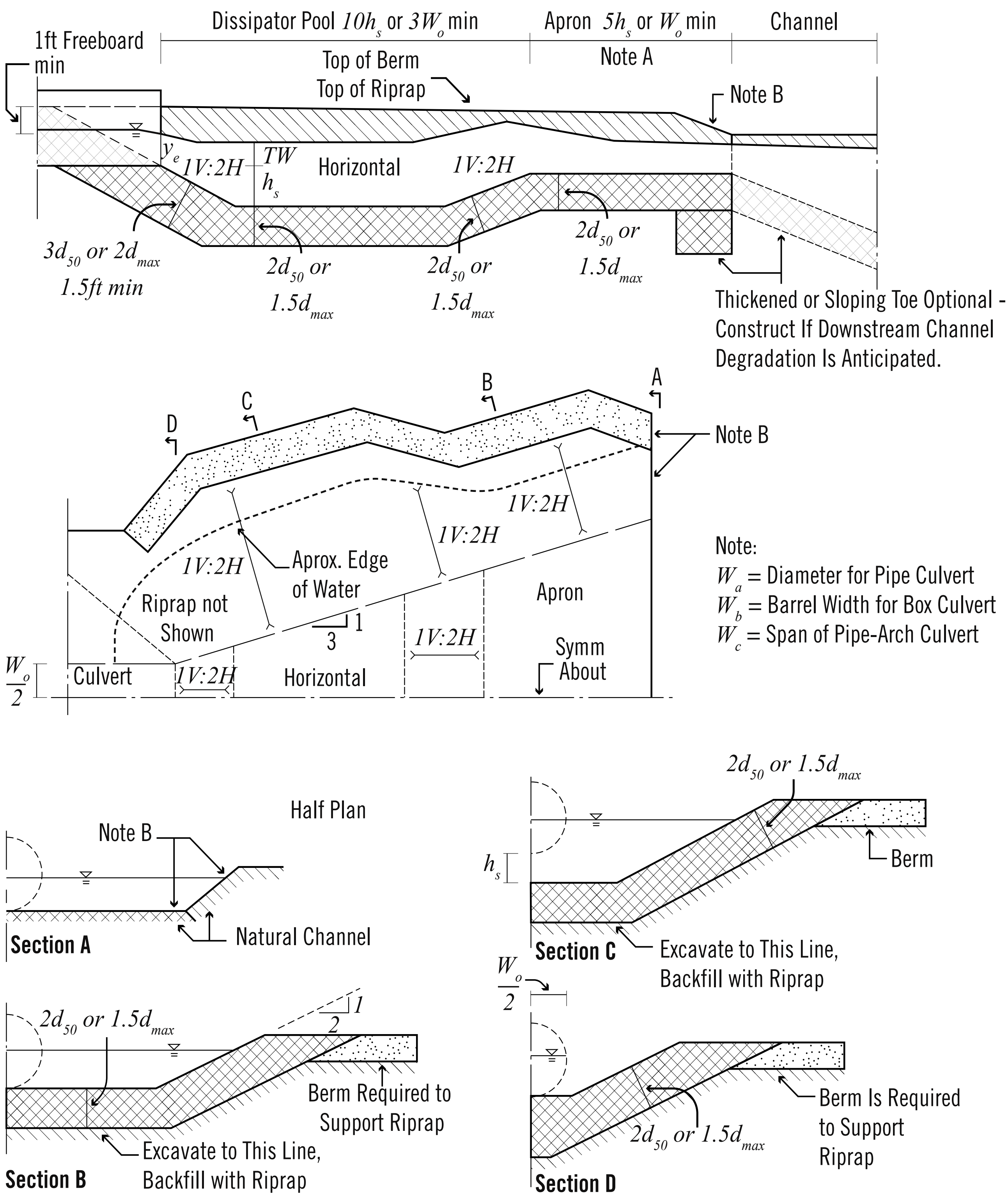
V = velocity at exit of dissipator (ft/s)

S = riprap specific gravity

D_{30} is typically 15 percent smaller than D_{50}. D_{85} shall lie between $1.8D_{15}$ and $4.6D_{15}$.

Filter fabric can be placed in between the structure and underlying soil. This layer would prevent the movement of soil, distribute weight more evenly, and limit erosion. Filter fabric is required when riprap is placed on non-cohesive soils. A gravel bedding layer should be placed between larger riprap and the filter fabric to prevent tears. Fabric layers should overlap and placement should allow for stretching as riprap is placed on top. The material should be wrapped around the toe of the revetment. See Figure 3.37 for additional design details.

Figure 3.37 Details of Riprap Outlet Basin



Note A

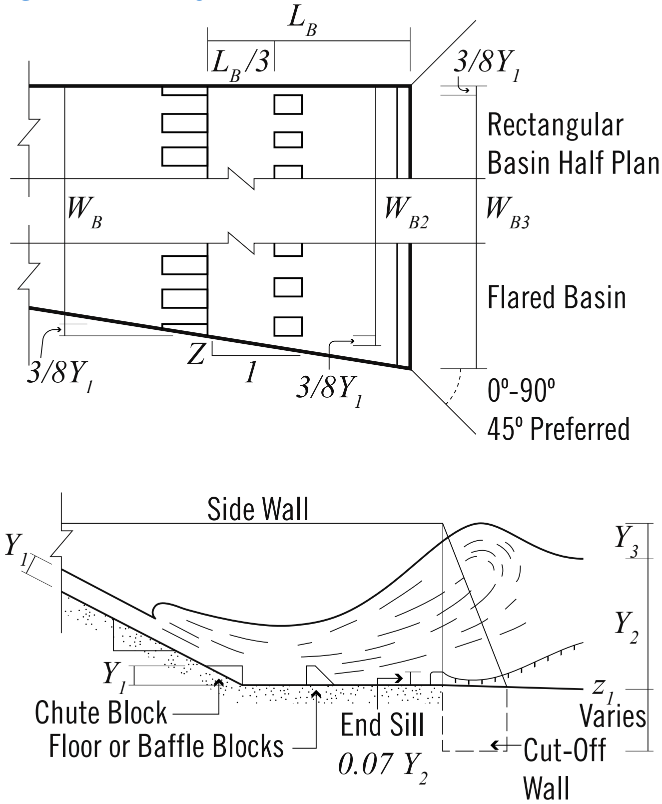
If Exit Velocity of Basin Is Specified, Extend Basin as Required to Obtain Sufficient Cross-Sectional Area at Section a Such That Qds / (Cross Section Area at Section A) = Specified Exit Velocity.

3.8.3.2 Concrete Aprons / Basins

Stilling Basin

The stilling basin is used as an energy dissipator to trigger a hydraulic jump within the basin. The basin requires a tailwater condition. These stilling basins normally operate within Froude numbers from 1.7 to 17. The Saint Anthony Falls (SAF) stilling basin is shown in Figure 3.38.

Figure 3.38 SAF Stilling Basin

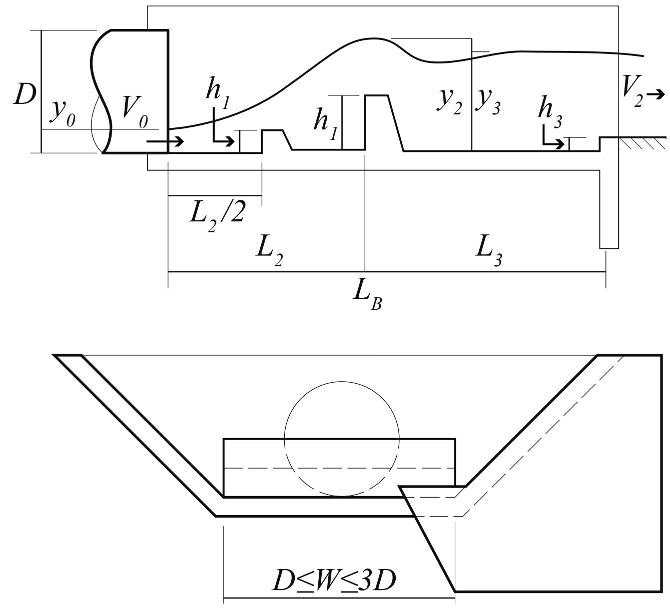


For the design of stilling basins, refer to FHWA HEC-14.

Contra Costa Basin

The Contra Costa Basin can be used for a culvert outlet with some tailwater. The Contra Costa Basin was developed at the University of California, Berkeley, in conjunction with Contra Costa County, California. See Figure 3.39 for details. This basin is best suited to situations in which the depth of flow at the outlet is equal to one-half the culvert height.

Figure 3.39 Contra Costa Basin



For the design of stilling basins, refer to FHWA HEC-14.

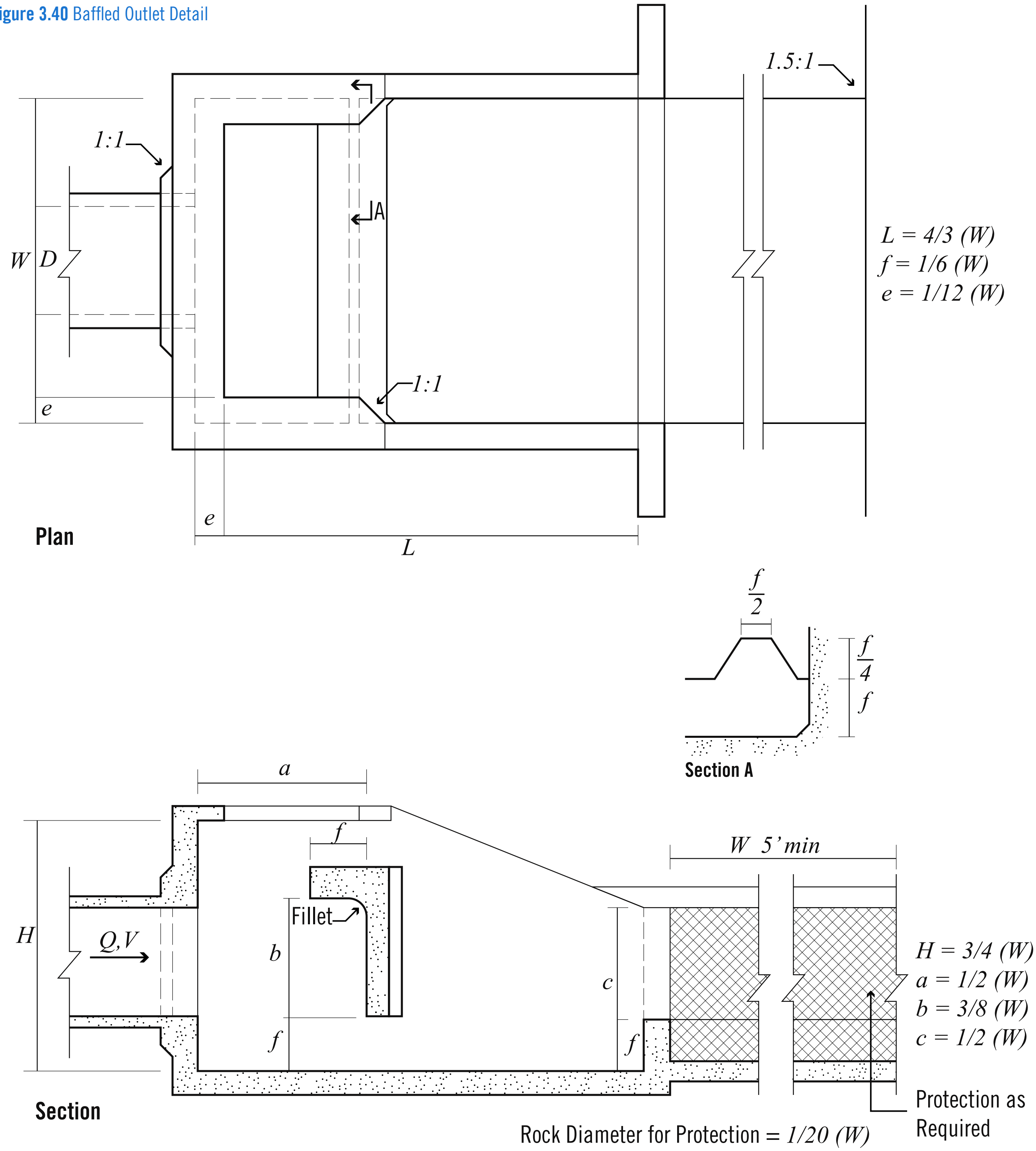
3.8.3.3 Baffle Blocks

Baffle blocks should be used to reduce the subcritical velocity to 6 feet per second or less. A minimum of 2 rows of block should be used. The distance from the culvert to the first row of blocks should be a minimum of the culvert height. The height of blocks should be a minimum of 1 foot or critical depth (d_c). The width of the block and spacing of blocks should match the height of block. The second row of blocks should be offset so the block lines up with the spacing of the first row of blocks. The blocks should extend across the total bottom width of the culvert outlet structure.

3.8.3.4 Baffled Outlets

A baffled outlet is a box-like structure that utilizes a vertical hanging baffle and end sill to increase turbulence and slow velocities. A baffled outlet is also known as a USBR Type VI impact basin. It can be used at outlet of culverts or in open channels. Runoff hits the baffle, creating eddies. At higher discharges, water will overflow over the top of the baffle as well as below it. The notches in the baffle serve to create concentrated flow to wash out any built up sediment. Baffled outlets will work in all tailwater conditions but will perform best when maximum tailwater is no greater than half of the baffle. Riprap should be placed downstream of the basin. See Figure 3.40 for details. See FHWA HEC-14 for detailed design methods.

Figure 3.40 Baffled Outlet Detail



3.8.3.5 Drop Structures

Drop structures are typically used in channels to mitigate the effects of steep slopes by creating gentle slopes and vertical drops that dissipate and slow velocities. The stilling basin below the drop should be armored to prevent erosion. A drop structure should have a maximum drop of 4 feet. Baffles and sills can be placed within the stilling basin to provide additional energy dissipation if desired. See FHWA HEC-14 for detailed design methods.

Several factors should be taken into consideration when designing drop structures. A geotechnical and structural analysis should be done to confirm the structural stability at the site. There is potential for erosion upstream of the drop structure, as drawdown will occur. A 10 foot long riprap apron should be placed upstream of the drop to limit velocity and turbulence as the water approaches the drop. The channel sides adjacent to the drop structure should be protected against erosion due to the steep hydraulic gradient. In addition, the toewalls of the channel must be protected upstream and downstream of the drop structure to prevent scour and undermining. Downstream of the drop structure, the channel should be armored with reinforced concrete or riprap to protect against the erosion and scour from increased turbulence. Table 3.6 should be used for determining the minimum length of riprap apron downstream of a drop structure.

Table 3.6 Riprap Apron Length Downstream of Drop Structures

Maximum Unit Discharge, q (cfs/ft)	Length of Downstream Apron, L_B (ft)
<15	10
15	15
20	20
25	20
30	25

3.8.3.6 Other Energy Dissipators

Other energy dissipation methods may be used provided that the selected design reduces velocities to below 6 ft/s.

3.8.4 Sedimentation Basin

A sedimentation basin is designed to collect and retain runoff and allow for the settling of suspended solids so that they do not enter natural bodies of water. Sedimentation basins can be used as a temporary measure on a construction site or as a permanent water quality measure. They can be used for drainage areas up to 100 acres for a permanent engineered basin and 20 acres for a temporary basin during construction. Sedimentation basins may not be effective with soils that have a high percentage of clay or silt.

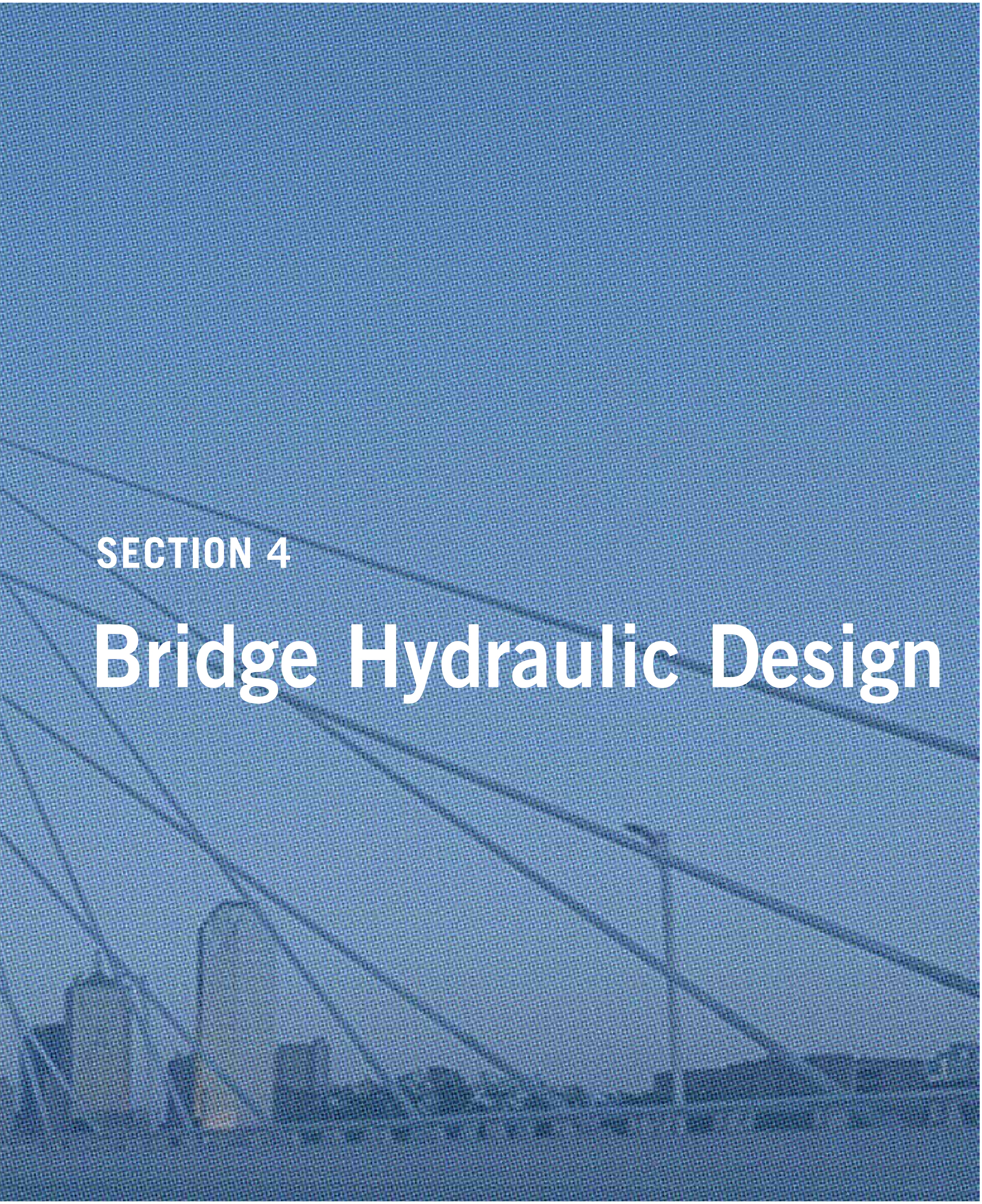
Sedimentation basins should be sized to have a length four times greater than the width, or employ other methods (e.g. baffle walls) to have a long flow path length. They should have a minimum depth of two feet with side slopes no steeper than 2:1. Basins should be sized to hold at least 1 inch of runoff from the contributing drainage area.

Unless designed with a permanent pool, drawdown should be a minimum of 6 hours and a maximum of 24 hours.

Sedimentation basins should not be located in streams or wetlands. The sedimentation basin should be accessible for maintenance. Measures to prevent vector attraction should be used.

Emergency spillways should be designed to pass the 1% annual chance storm event. All dams are subject to TCEQ regulations, as applicable.

Temporary detention basins are designed for 2-year storm, unless they are online to a stream. Online temporary sedimentation basins are required to address the 1% annual chance storm event for upstream contributing area.



SECTION 4

Bridge Hydraulic Design

4.1

GENERAL

The function of a bridge is to convey surface water under a roadway, railroad, or other embankment. Impact from floating debris must be considered for the structural design of the bridge components. Where proposed streets cross existing or proposed watercourses, all-weather crossings are required.

The design engineer will analyze both existing and proposed bridge conditions. The U.S. Army Corps of Engineers' HEC-RAS modeling software is recommended to analyze a bridge. Other software may be used with the approval of the Director. A bridge may increase the depth of flow upstream. Modifications of the channel downstream and upstream of the proposed bridge may be needed to mitigate the upstream impact.

Figure 4.1 displays the four typical model cross-section locations necessary to assess the hydraulic performance of a bridge. Section 1 should be located downstream of the bridge at a point where the expansion of flow from the bridge is expected to be complete. Section 2 should be located a short distance downstream of the bridge. Section 3 should be located a short distance upstream of the bridge. Section 4 should be located upstream of the bridge at a point where the start of contraction is expected to occur.

Typical contraction and expansion values at bridge sections are 0.3 and 0.5 respectively. Abrupt transitions will have higher values. The contraction and expansion values should typically be applied to Sections 2, 3, and 4. Refer to the HEC-RAS Hydraulic Reference Manual for additional guidance related to hydraulic modeling of bridges.

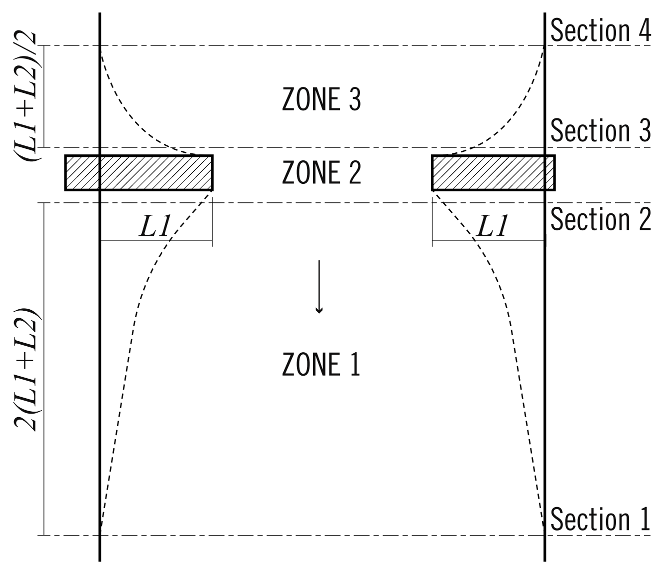
4.2

DESIGN FREQUENCY AND FREEBOARD

4.2.1 Design Frequency

The design frequency for bridges is the 1% annual chance storm event.

Figure 4.1 Model Cross-Section Locations



* Adapted from HEC-RAS User's Manual

4.2.2 Freeboard / Minimum Clearance

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.

Bridges should be designed with a minimum of 2 feet of freeboard from the low chord to the 1% annual chance flood elevation unless a low water crossing is approved by the Director.

For bridges over aesthetic waterways that do not serve a drainage conveyance function (for example, a pond that is located off-stream and serves no detention function), there is no freeboard requirement, but overtopping should not occur in the 1% annual chance storm event.

The design engineer should consider the minimum clear height from the channel bottom to the low chord to be 6 feet. Additional height should be considered for passage of maintenance vehicles under the bridge to minimize the number of channel access ramps. Low water crossings must be approved by the Director.

4.3

FLOW REGIMES

4.3.1 Low Flow Considerations

For the purpose of flow regimes, low flow exists when the water surface is below the low chord of the bridge opening. There are three classes of flow for low flow conditions. See Figure 4.2.

Class A low flow exists when the water surface is subcritical from sections 1 to 4.

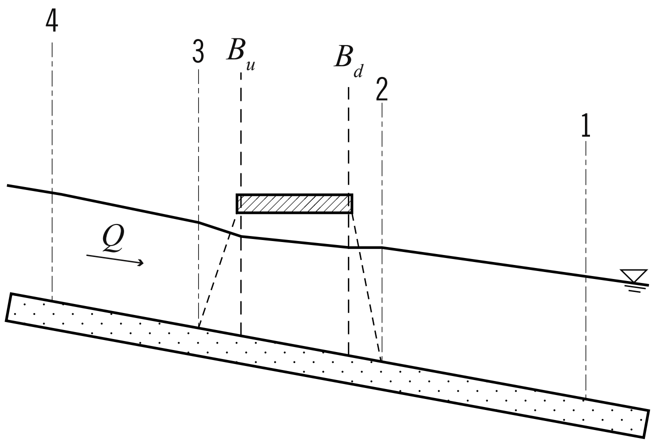
There are four methods available for assessing hydraulic performance between Sections 2 and 3 as follows:

- energy equation
- momentum balance
- Yarnell equation, and
- FHWA WSPRO method.

Class B low flow exists when the water surface passes through critical depth within the bridge constriction between Sections 2 and 3. The flow upstream and downstream of the bridge can be either subcritical or supercritical.

Class C low flow exists when the water surface is supercritical from Sections 1 to 4.

Figure 4.2 Bridge Profile with Cross Section Location



4.3.2 High Flow Considerations

High flow exists when the water surface comes into contact with the maximum low chord of the bridge opening. The computation would be by the Energy equation or by hydraulic equations for pressure and/or weir flow.

4.3.3 Supercritical Flow

For supercritical flow conditions in a stream or channel, the design engineer should confirm that the bridge opening is clear of bridge piers or other projections and does not impact the flow. If bridge piers or other projections are within the bridge opening, then hydraulic jumps within the bridge structure should be considered and the impacts should be addressed in the bridge design.

4.3.4 Erosive Velocities

Velocities within bridge sections are often higher than velocities within the channel upstream of the bridge. Erosion protection as discussed in Section 7 is often required for bridges, as restrictions to open channel velocities also apply to channels under bridges.

4.4 CHANNEL CONSTRICTION

Where sloping abutments are utilized, slopes shall be designed per requirements in Section 7.2 for natural channel setback. The Director may approve a steeper slope if a slope stability analysis is provided by a Licensed Geotechnical Engineer that demonstrates stability at the steeper slope and the abutment slope is lined with hard armoring such as concrete, concrete block mats, stone riprap, or gabions.

The width of the main channel between the toe of slope from each abutment shall not be less than the typical channel width upstream and downstream of the bridge.

4.5

SCOUR

Consideration of the scour of soil around a bridge is critical to the longevity of the structure. The total scour at a bridge crossing is comprised of three components: long term aggradation and degradation, contraction scour, and local scour at piers and abutments. The long term aggradation and degradation should be assessed and accounted for in the bridge scour analysis.

Bridge scour analysis for contraction scour and local scour at piers must be performed. Contraction and pier scour shall be evaluated and results incorporated into structural design. Guidance on contraction and pier scour can be found in FHWA's HEC-18. A reduction factor of 0.5 for soils with 11% or more clay can be applied to the computed pier scour. Abutment riprap shall be designed in accordance with the design guidelines in Section 7.4.4 in lieu of performing abutment scour analysis. The riprap abutment toe must be designed to account for any anticipated contraction scour. Scour analysis will not be needed if the channel is concrete lined.

4.6

BRIDGE DECK DRAINAGE

Bridge deck drains should:

- Minimize the spread of water into the traffic lanes and, at a minimum, meet the street capacity requirements described in Section 3.2.1
- Prevent the accumulation of significant depth of water to reduce hydroplaning
- Be integrated into the structural deck
- Reduce drain hazards to bicyclists
- Be easy to maintain
- Provide sufficient longitudinal grade
- Avoid zero longitudinal grade and sag vertical curves on the bridge
- Intercept all flow from curbed street before it reaches bridge
- Not cause erosion within the channel below the bridge

4.7

ROADWAY OVERTOPPING

Ideally, the public vehicular roadway adjacent to the bridge should not overtop during the 1% annual chance storm event. In order to prevent roadway overtopping, the profile grade of the roadway, at all points within the floodplain, shall be a minimum of 1 foot above the energy grade elevation or hydraulic grade line for the 1% annual chance storm event, whichever is greater. All exceptions must be approved by the Director.

4.8

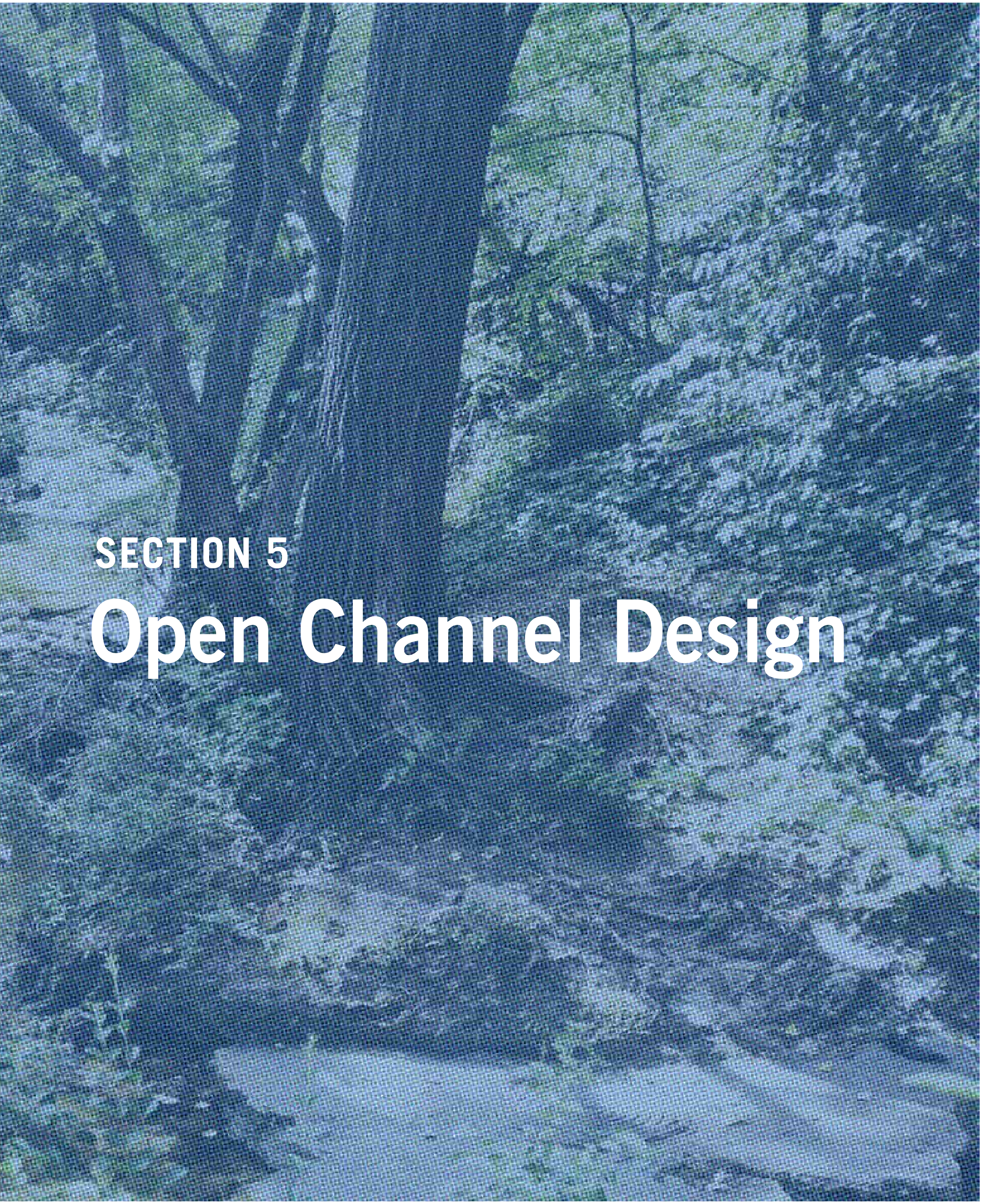
OTHER DESIGN CONSIDERATIONS

Maintenance agreements are required for private vehicular / pedestrian bridges over creeks and channels.

The following guidelines pertain to the hydraulic design of bridges:

1. Side swales in a main bridge opening or relief channels with overflow structures may be used to provide additional conveyance downstream of and through bridges. (Swales are subject to the criteria contained in the Floodplain Ordinance Article V Section 51A-5.100-5.105.)

2. Bents should not be located in the main channel when possible. Bents should be aligned approximately parallel to flow.



SECTION 5

Open Channel Design

5.1

GENERAL

This section addresses proposed improvements or modifications to drainage channels and watercourses that convey storm water runoff.

Preservation of creeks in their natural condition is preferred. In the event that it is necessary to utilize a constructed open channel, it shall be designed to convey the full design discharge, the 1% annual chance storm event. Hydraulic characteristics of open channels can vary greatly and have a significant impact on the performance of open channels. Section 5.2 provides discussion of open channel hydraulics relative to channel design. Open channel design guidelines are provided in Section 5.3. Refer to Section 3.4 for bar ditch and bioswale design.

See Appendix A.3 for further guidance on energy conservation equations and Froude numbers.

5.2

OPEN CHANNEL HYDRAULICS

Hydraulic calculations can be performed using software from the City's approved list (a list of approved software can be found on the City's website).

Certain watersheds have hydrologic and hydraulic models that are available through the City. Projects proposed within the limits of these watersheds must have the models updated by the design engineer to reflect changes in flow, channel configuration (including alterations to vegetation) and channel structures.

5.2.1 Flow Classification

5.2.1.1 Types of Flow in Open Channels

Open channel flow can be characterized in many ways. Types of flow are commonly characterized by variability with respect to time and space. The following terms are used to identify types of open channel flow:

Steady flow—flow conditions at any point in a stream remain constant with respect to time (Daugherty and Franzini 1977).

Unsteady flow—flow conditions (e.g., depth, velocity) vary with time.

Uniform flow—the magnitude and direction of velocity in a stream are the same at all points in the stream at a given time (Daugherty and Franzini 1977). If a channel is uniform and resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel for uniform flow.

Varied flow—discharge, depth, or other characteristics of the flow change along the course of the stream. For a steady flow condition, flow is termed *rapidly varied* if these characteristics change over a short distance. If characteristics change over a longer stretch of the channel for steady flow conditions, flow is termed *gradually varied*.

5.2.1.2 Subcritical Flow

Flows with a Froude number less than 1.0 are subcritical flows. Subcritical flows are characterized by relatively tranquil, low velocity flows compared to supercritical flow, though they can still be erosive. Water surface elevations for subcritical flow are computed from the downstream to the upstream direction as flow control is from the downstream direction.

Most stable natural channels have subcritical flow regimes. Consistent with the philosophy that the most successful artificial channels utilize characteristics of stable natural channels, major drainage design should seek to create channels with subcritical flow regimes.

5.2.1.3 Supercritical Flow

Design solutions that incorporate supercritical flow are to be avoided where possible. If transitioning from supercritical to subcritical flow cannot be avoided, see following design guidance for supercritical flow and hydraulic jumps.

The influence of gravity on fluid motion in open channel flow can be expressed in a dimensionless quantity called a Froude Number (*Fr*). The Froude Number is expressed in the following equation.

$$Fr = \frac{V}{\sqrt{gd}} \quad \textit{(Equation 5.1)}$$

V = Mean velocity (ft/s)

g = Acceleration due to gravity = 32.2 ft/s^2

d = Hydraulic depth (ft)

The hydraulic depth is defined as the cross sectional area of the channel perpendicular to the direction of flow divided by the free water surface topwidth.

Flows with a Froude number greater than 1.0 are supercritical flows. Supercritical flows have higher velocities, shallower depth, higher hydraulic losses, and greater erosive power than subcritical flows.

For supercritical flow, water surface computations occur from the upstream to the downstream direction. Supercritical flow in an open channel in an urban area creates hazards that the designer must consider. Special considerations for the design of channels with. supercritical flows are provided in Section 5.3. The minimum design depth of a channel shall be the frictional depth plus freeboard, or sequent depth without freeboard, whichever is greater. Supercritical flow conditions shall only be allowed in lined channels or at drop structures. The design engineer must demonstrate that the drop structure and downstream channel is designed to withstand the turbulence from the drop, or lining shall be provided. Subcritical flow conditions may be achieved by using energy dissipators in unlined channels in areas where the existing topography will not allow subcritical flow conditions to occur naturally. In all cases, the channel improvements shall be designed to avoid the unstable transitional flow conditions that occur when the Froude number is between 0.9 and 1.1 Therefore, channel Froude numbers must be below 0.9 or greater than 1.1 unless analyses demonstrate that no adverse effects will occur as a result.

5.2.1.4 Critical Flow

Critical flow in an open channel or covered conduit with a free water surface is characterized by the following conditions:

1. The specific energy is a minimum for a given discharge.
2. The velocity head is equal to half the hydraulic depth in a channel of small slope.
3. The Froude number is equal to 1.0.

Channels rarely flow at critical depth for long reaches. Flow passes through critical depth as it transitions from subcritical to supercritical flow, or vice versa. These transitions often occur at rapid changes in cross-section shape, significant changes in lining resistance and slope such as from a natural channel to a concrete lined channel, and where significant obstructions occur within a supercritical channel.

A slope less than critical slope will cause subcritical flow, and a slope greater than critical slope will cause supercritical flow. A flow at or near the critical state may not be stable. In design, if the depth is found to be at or near critical, with a Froude number near 1.0, the shape or slope should be changed to achieve greater hydraulic stability.

To simplify the computation of critical flow, dimensionless curves have been given for rectangular, trapezoidal, and circular channels located in Appendix A.7.

5.2.1.5 Hydraulic Jump

The hydraulic jump is a natural phenomenon that occurs when supercritical flow changes to subcritical flow. This often occurs due to an obstruction to the flow, a rapid decrease in channel slope, a contraction of the floodway, or a significant increase in Manning's 'n' value. This abrupt change in flow condition is accompanied by considerable turbulence and loss of energy. The hydraulic jump can be illustrated by use of a specific energy diagram as shown in Figure 5.1. The flow enters the jump at supercritical velocity, V_1, and depth, y_1, that has a specific energy of $E = y_1 + V_1^2/(2g)$. The kinetic energy term, $V_2/(2g)$, is predominant. As the depth of flow increases through the jump, the specific energy decreases. Flow leaves the jump area at subcritical velocity with the gravitational potential energy or depth, y, predominant.

The length and the conjugate depth of a hydraulic jump are generally of interest to channel designers. If a hydraulic jump is being modeled with hydraulic software, cross-sections should be spaced closer together than usual so that the model can accurately identify the expected location of the jump. To compute hydraulic jump parameters for rectangular channels by spreadsheet, the following equations and charts may be used.

$$\frac{y_2}{y_1} = 1/2\left(\sqrt{1 + 8Fr_1^2} - 1\right) \quad (Equation\ 5.2)$$

y_2 = Conjugate depth (ft)

y_1 = Depth approaching the jump (ft)

Fr_1 = Approach Froude number

If $y_2 / y_1 = J$, the expression for a hydraulic jump in a horizontal, rectangular channel becomes

$$J = 1/2\left(\sqrt{1 + 8Fr_1^2} - 1\right) \quad (Equation\ 5.3)$$

Then, the conjugate depth and length of the hydraulic jump can be determined from Figures 5.2 and 5.3, respectively.

5.2.1.6 Uniform Flow

Manning's Equation describes the relationship between channel geometry, slope, roughness, and discharge for uniform flow:

$$Q = \left(\frac{1.486}{n}\right) A \ R^{2/3} S^{1/2} \quad (Equation\ 5.4)$$

Q = Discharge (cfs)

n = Roughness coefficient

A = Area of channel cross section (ft^2)

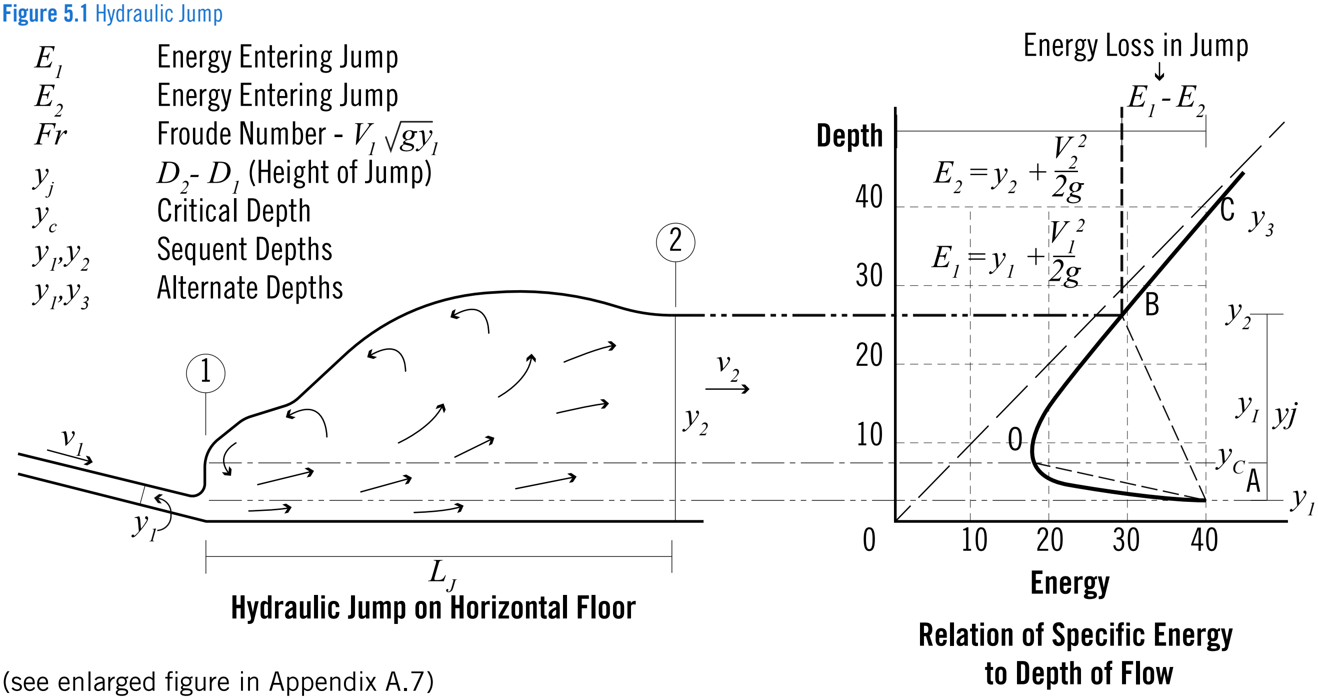
P = Wetted perimeter (ft)

R = Hydraulic radius = A/P (ft)

S = Channel bottom slope (ft/ft)

Manning's 'n' values are provided in Appendix A.3. Manning's Equation can also be expressed in terms of velocity by employing the continuity equation, $Q = VA$, as a substitution in the above equation, where V is velocity (ft/s).

Figure 5.1 Hydraulic Jump



(see enlarged figure in Appendix A.7)

Figure 5.2 Hydraulic Jump Conjugate Depth – Horizontal, Rectangular Channel

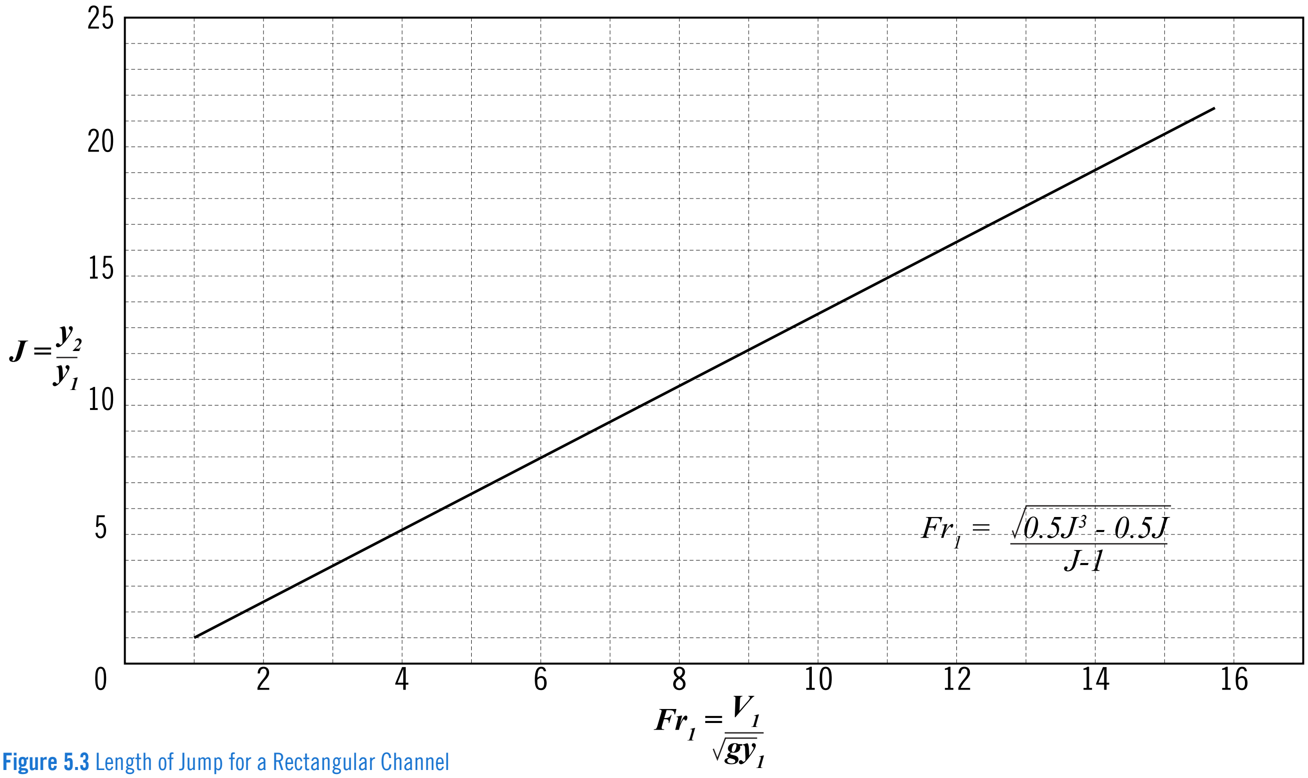
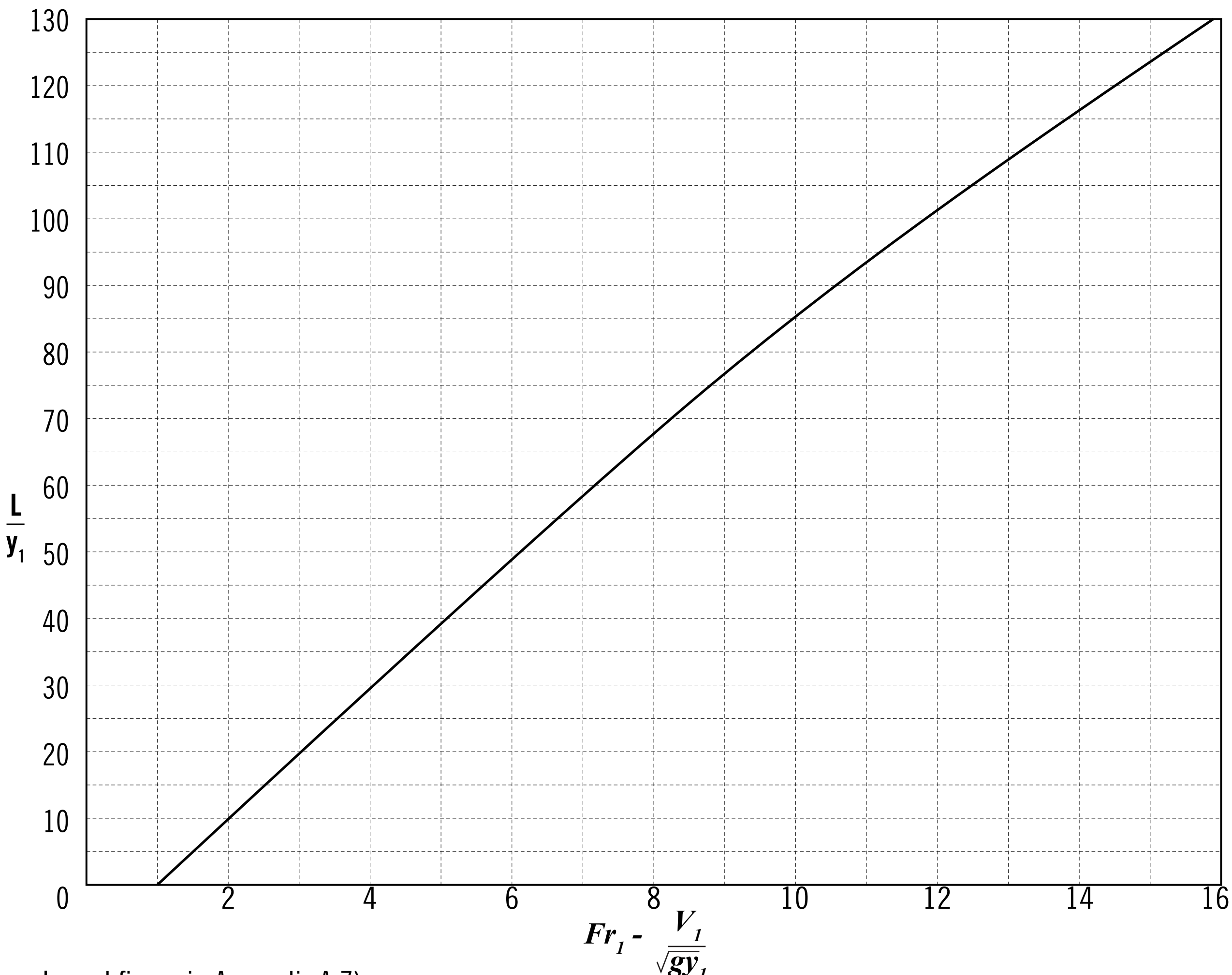


Figure 5.3 Length of Jump for a Rectangular Channel



(See enlarged figure in Appendix A.7)

5.3

DESIGN GUIDELINES

5.3.1 Manning's ‘n’ Values

Manning's roughness coefficients ('n' values) for use in hydraulic calculations shall be consistent with the values listed in Appendix A.3. Care should be taken when sub-dividing roughness coefficients in a channel cross-section. Refer to TXDOT Hydraulic Design Manual for guidance on how to correctly subdivide a channel for conveyance.

5.3.2 Design Frequency and Freeboard

All open channels require a minimum freeboard of 2 feet below top of bank for the 1% annual chance flood. See Section 5.3.8 for additional freeboard required at channel bends.

Obstructions that may enter a stream during a storm event may cause supercritical flows to experience a hydraulic jump and become subcritical flows. Therefore, channels that are designed for supercritical conditions must have a freeboard equal to the conjugate depth plus 1 foot.

5.3.3 Channel Velocity

Table 5.1 shall be used to determine maximum permissible channel velocity.

Where velocities are in the supercritical range, allowance shall be made in the design for the proper handling of the stormwater.

Table 5.1 Maximum Channel Velocities

Channel Type	Maximum Side Slopes	Maximum Velocities
Earth vegetated soils	3:1	6 fps
Partially lined	3:1 above lining	12 fps
Fully lined	2:1	15 fps (NA at drop structures)

At transitions from lined to unlined channels, velocities must be reduced to 6 fps or less. Velocities must be reduced before the flow reaches the natural channel using either energy dissipators and / or a wider channel. Unlined, non-vegetated swales are not allowed.

5.3.4 Channelization

All channelization projects are subject to the City of Dallas floodplain regulations, Article V Section 51A-5.100-5.105 of the Dallas Development Code. For the channelization of any natural creek, an Erosion Control Plan and a Revegetation Plan will be required as well as a 404 or nationwide permit if applicable. See Section 9.2 for applicability of additional requirements.

5.3.5 Channel Geometry

The constructed channel geometry may be triangular, rectangular or trapezoidal in shape. The side slopes should not exceed the requirements in 5.3.3. In areas where traffic safety may be of concern, the channel side slope should be $4H:1V$ or flatter or other vehicular protection devices may be required.

For natural channels, the channel geometry may be irregular in shape. The channel sections should be checked for areas of erosion and provide corrective measures with the natural channel design.

Open channels with narrow bottom widths shall be avoided, as they are characterized by high velocities and difficult maintenance. Minimum channel bottom widths are recommended to be equal to twice the depth. Any permanent open channel with a depth of flow greater than 3 feet shall have a bottom width of at least 5 feet. If maintenance vehicles are required to travel the channel bottom, the width shall be increased to 8 feet. Parallel access roads can also be used to facilitate maintenance.

If the channel cannot be maintained from the top of the bank, a maintenance access ramp shall be provided with a 15% maximum longitudinal grade and 2% minimum cross slope towards the channel and a minimum width of 12 feet. Ramps shall be installed as needed to provide continuous access but in no case shall ramp spacing be greater than 1 mile. Access is normally required for but not limited to the following:

- Sedimentation basins
- Between grade stabilizers
- Between drop structures
- Where access under bridges, culverts, etc. is restricted
- All channels with a bottom width greater than 12 feet

The type of equipment needing access is dependent on the channel. Large channels may need access for dump trucks and loaders. For small ditches, foot or pick-up truck access may suffice.

5.3.6 Channel Slope

The channel slope for constructed channels shall meet the requirements of 5.3.3. For earthen channels, the design engineer should consider the channel stability of the design slope to determine if additional protection will be needed to protect the bottom and side slopes.

Slope that achieves subcritical flow conditions are recommended for all channel designs. Supercritical flow conditions shall only be allowed in lined channels or at drop structures. The design engineer must demonstrate that the drop structure is designed to withstand the turbulence from the drop or lining shall be provided. Subcritical flow conditions may be achieved by using energy dissipators in unlined channels where the existing topography will not allow subcritical flow conditions to occur naturally.

In all cases, channel improvements will be designed with slopes necessary to avoid the unstable transitional flow conditions that can occur when the Froude number is between 0.9 and 1.1 unless analysis demonstrates that no adverse effects will occur as a result.

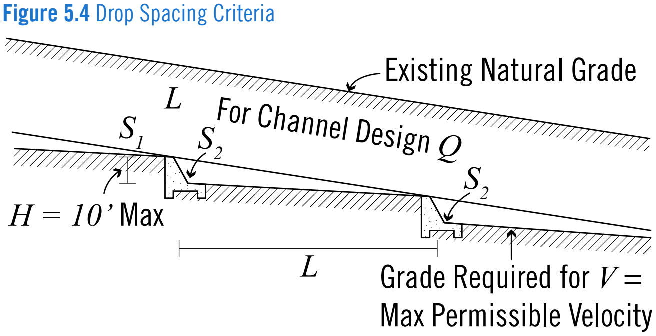
5.3.7 Bed Controls

The potential vertical degradation of a stream shall be assessed and channel protection provided where needed. If drop structures are utilized, shear stresses associated with the drop shall be computed and adequate channel lining provided in accordance with Section 7.4.

5.3.7.1 Channel Drops

Drop spacing shall be computed as follows. See Figure 5.4 and the following equations for spacing criteria.

Figure 5.4 Drop Spacing Criteria



$$L = \frac{H}{S_1 - S_2} \quad (Equation\ 5.5)$$

L = Distance required between drops (ft)

H = Height of drop (ft) (Maximum of 3 ft unless approved by Director)

S_1 = Actual slope of channel (ft/ft)

S_2 = Slope of proposed channel for maximum permissible velocity established from Table 5.1 (ft/ft)

$$S_2 = \frac{NV^2}{(1.486R^{2/3})^2} \quad (Equation\ 5.6)$$

V = Maximum permissible velocity established from Table 5.1 (ft/s)

N = Manning's 'n' value

R = Area/wetted perimeter (ft)

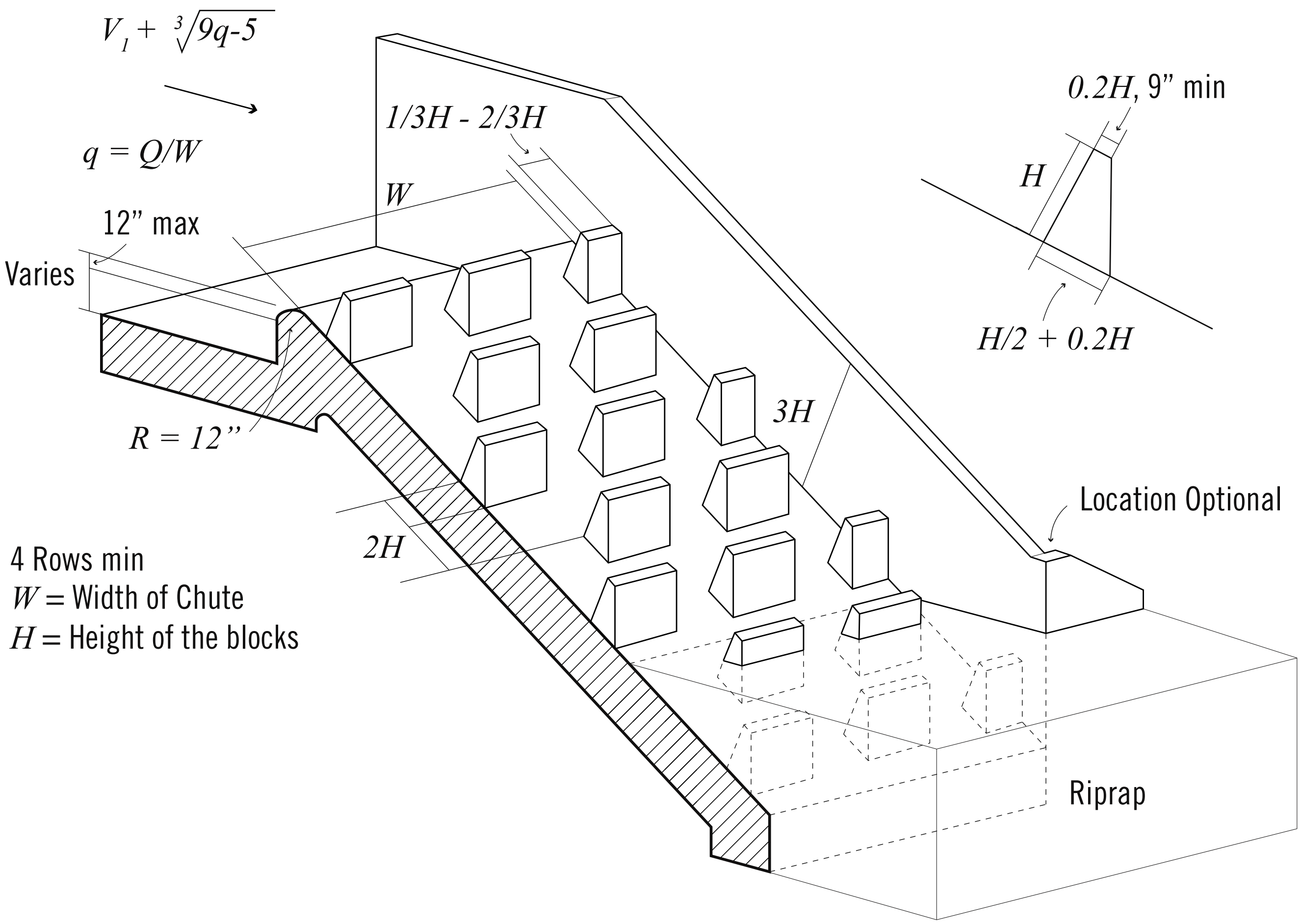
The design engineer should analyze channel drops to determine if the flow is or will become supercritical along the channel. If the channel becomes supercritical, the depth of the channel should contain the sequent depth (see Figure 5.1).

5.3.7.2 Baffle Chutes

For concrete chutes on earthen side slopes, the following shall be used for the design of baffle blocks on the chute drop. The approach velocity to the chute shall be less than critical velocity. The chute slope shall fall between $2H{:}1V$ to $4H{:}1V$. The maximum flow shall be 60 cfs per foot of chute width.

The height of the blocks, H, shall range from 0.8 times the critical depth to 0.9 times critical depth. The width and spacing of the baffle block shall be $1.5H$. The chute blocks are to extend across the total width of the chute. The subsequent rows of blocks shall be offset so the blocks line up with the spacing of the upstream block. The spacing of the row of blocks shall be $2H$.

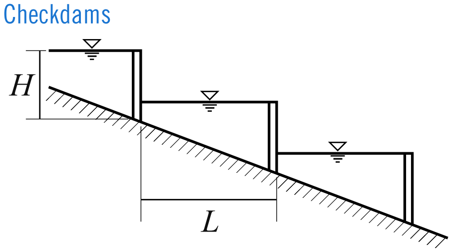
Figure 5.5 Baffle Chute



5.3.7.3 Checkdams

Checkdams may be used similarly to drop structures, but they maintain a normal water surface elevations between drops. Erosion protection shall be provided downstream of each drop structure.

Figure 5.6 Checkdams



Refer to Equation 5.5 for spacing of checkdams.

5.3.8 Channel Bends

Head loss resulting from channel bends shall be incorporated into channel hydraulic modeling. Shear stress shall be evaluated. See Section 7.4 for guidance on erosion protection. The following equation can be used to predict maximum scour depth at bends.

$$\frac{d_{max}}{d_m} = 1.5 + 4.5 \left(\frac{R_c}{W_i}\right)^{-1} \quad \text{(Equation 5.7)}$$

d_{max} = Maximum scour depth in bendway pool (ft)

d_m = Mean depth (cross-sectional area / W) (W = reach average bankfull width) (for the 1% annual chance storm event, or smaller event if it would result in a higher scour depth estimate)

R_c = Radius of curvature (degrees)

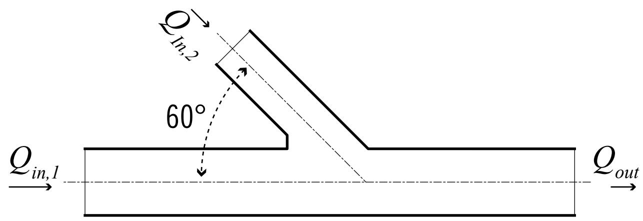
W_i = Width at meander inflection point (ft)

Allowance for extra freeboard shall be made when the centerline radius of the channel is less than three times the bottom width or for supercritical flow regime. See Section 5.3.11 for guidance on superelevation.

5.3.9 Channel Junctions

Channel junctions shall have a maximum angle of 60 degrees. Refer to Figure 5.7 for example geometry.

Figure 5.7 Example Geometry for Channel Junction



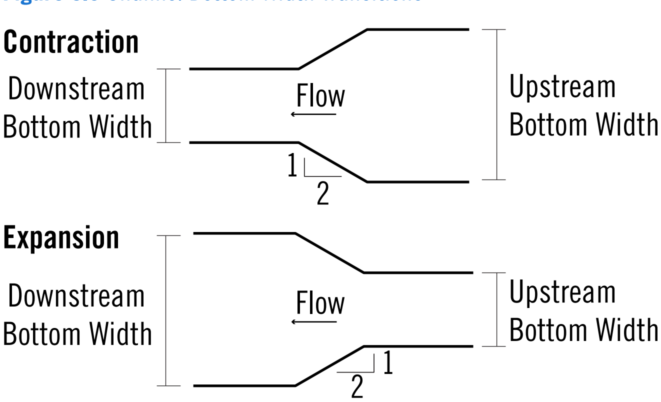
Designer shall assess potential for erosion on both banks of the junction and assess whether erosion protection is needed. This assessment shall consider the relative size and timing of the peak flows and the potential for the adjoining stream peak discharge to enter the main channel without the presence of significant main channel flow that may prevent erosion of the opposite bank. See Section 7.4 for guidance on erosion protection. Junction headloss shall be considered in open channel hydraulic modeling.

Refer to the HEC-RAS technical manual for further guidance on modeling junction headloss.

5.3.10 Bottom Width Transition

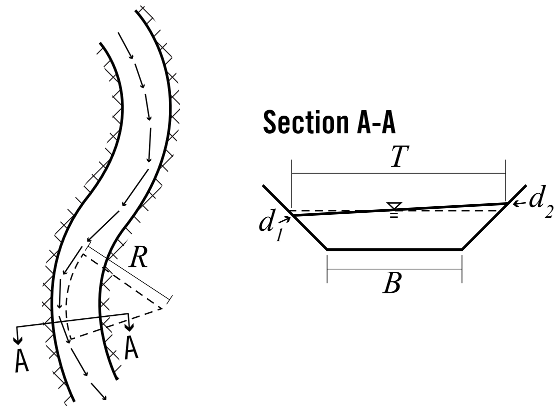
For all designed bottom width transitions greater than a 1:3 W:H ratio, per side of channel, the designer shall provide an assessment of the impact of the transition on shear stress and demonstrate that adequate lining has been provided per Section 7.4. See Figure 5.8 for guidance. Potential headloss due to bottom width transitions shall also be accounted for. For supercritical channels, the transition must be modeled using City-approved software and assessed to determine whether a hydraulic jump may occur. Bottom width transitions due to culverts and bridges shall be assessed if they meet the above requirements.

Figure 5.8 Channel Bottom Width Transitions



5.3.11 Superelevation

Figure 5.9 Superelevation



Allowance for extra freeboard shall be made when the centerline radius of the channel is less than 3 times the bottom width or for supercritical flow regime. For supercritical flow, see Section 5.2.1.3 and refer to Section 7 for erosion protection measures. Where bends or high velocities are involved and the flow regime is subcritical, the applicant shall use the following formula for computing the extra freeboard (refer to Figure 5.9):

$$d_2 - d_1 = V^2(T + B)/\ 2gR \quad (Equation\ 5.8)$$

d_2 = Depth of flow at the outside of the bend (ft)

d_1 = Depth of flow at the inside of the bend (ft)

B = Bottom width of channel (ft)

V = Average approach velocity in channel (ft/s)

T = Width of flow at the water surface (ft)

g = Acceleration due to gravity (32.2 ft/s^2)

R = Centerline radius of the bend (ft)

a. The quantity $d_2 - d_1$ divided by 2 shall be added to the normal depth of flow before adding the required freeboard in calculating required right-of-way widths.

b. Where sharp turns are used without curved sections, the depth required shall be large enough to provide for all head losses. Allowance shall be made for any backwater that may result.

c. For supercritical flow regimes, the extra freeboard calculated with the above formula shall be doubled.

SECTION 6

Drainage Storage Design

6.1

GENERAL

Stormwater detention is sometimes required to temporarily impound (detain) excess stormwater, thereby reducing peak discharge rates and velocities, as well as providing water quality benefits. Refer to Section 2.3 for guidance on when detention is required.

The four primary types of basins or ponds are:

Dry Detention Basins

Dry detention basins are open space areas designed to remain dry most of the time. They fill up during large storm events and drain completely through an outlet structure once the storm event has passed.

Wet Detention Basins

Wet detention basins are ponds designed to have a permanent pool, sometimes with a water feature. During a large storm event, detention storage is provided above the normal permanent pool level; after the storm event, the detention storage drains through an outfall and the water surface level returns to the permanent pool/normal water level. Normal ponded storage below the permanent pool (dead storage) does not count towards the active detention volume.

Retention Basins

Retention basins are basins that do not have an outfall and drain completely by infiltration into the underlying soil and evaporation. During a large storm event, retention basins fill and provide storage. After the storm event, the water drains from the site and no water is released downstream. Retention basins in Dallas require analyses of site specific percolation/infiltration rates, water balance analyses, and potential underdrain design to be an effective approach in Dallas.

Bioretention Basins

Bioretention is a sustainable drainage storage option that can be used to provide water quality benefits in addition to detention storage. Individual bioretention areas can be used to serve highly impervious areas less than 2 acres in size, and typically feature a filter bed which may be tied to an underdrain and returned to the storm drainage system. In areas with NRCS Classified A or B soils, with a low groundwater table and low risk of groundwater contamination, a bioretention facility can be used to infiltrate runoff directly into native soils. In NRCS Classified C or D soils, an underlying hard-pan, or rocklayer, or other impediments to percolation will require an appropriately designed underdrain system. Section 6.9 and the methodology for bioretention outlined by the Commonwealth of Virginia may be used to develop appropriate design.

Design guidance is provided for dry and wet detention basins (Section 6.2), retention and bioretention basins (Section 6.3), underground detention basins (Section 6.4), stormwater ponds (Section 6.5), and constructed wetlands (Section 6.6). Sediment forebays are required for detention or retention basins with upstream drainage areas of 100 acres or more, stormwater quality ponds, and constructed wetlands. The sediment forebay should be sized to hold 0.1 inches per impervious acre of the contributing drainage area and should generally be 4-6 feet deep. The volume in the forebay can count towards the total design volume of the stormwater pond. A sediment marker shall be installed in the forebay for maintenance and upkeep.

In compliance with the current City Stormwater Permit and FEMA CRS Program, water quality measures should be incorporated wherever practicable as part of prudent and holistic drainage system design. There are several sustainable drainage measures that provide varying degrees of detention while also providing water quality benefit. Guidelines for the implementation of these measures are also provided in this section.

6.2

DETENTION BASIN

6.2.1 Design Guidance

Detention is encouraged to be considered as part of site design and a multi-objective facility such as a park or nature area. The following are minimum criteria for detention basins within the City of Dallas. Criteria established by the State of Texas, as regulated by TCEQ, for dam safety and impoundment of state waters shall apply where required by the state, and where, in the engineer's judgment, the potential hazard requires these more stringent criteria. These design criteria apply to dry detention basins and wet detention basins.

Site Considerations – Dry detention basins shall not be located in areas where seasonal high groundwater is present within 2 feet of the bottom of the basin. If detention ponds are to be located in waters of the U.S., a USACE 404 permit and any other applicable permits must be obtained. At a minimum, the top of slope of detention basins must be set back 5 feet from buildings and roadways and 10 feet from property lines for private development. Designer should determine if a geotechnical analysis is needed to determine if more stringent requirements are necessary based on site-specific conditions.

Design Storm Event – The 1%, 2%, 10%, and 50% annual chance storm events shall be used for design.

Outflow Velocity – The outflow structure will discharge flows at a nonerosive rate. This rate is specified as 3 ft/s for areas above the Escarpment Zone and 5 ft/s for Geologically Similar Areas below the Escarpment Zone. In areas not regulated by the Escarpment Regulations, the allowed velocity shall not exceed velocities described in this manual. If channel lining or energy dissipators are needed, see Sections 5 and 7).

Detention Storage – Basins without upstream detention areas and with drainage areas of 100 acres or less can be designed using the Modified Rational Method (see Section 2.3.3).

Basins with drainage areas greater than 100 acres or where the Modified Rational Method is not applicable are to be designed using the Unit Hydrograph Method. The design hydrograph routings through the detention basin are to be performed using approved software, a list of which can be found on the City of Dallas website.

Drawdown of the detention storage for the 1% annual chance storm event must occur within 48 hours after design storm rainfall unless appropriate water rights are obtained.

Freeboard and Emergency Spillway – Where earth embankments are used to temporarily impound the required detention, the top of the embankment will be a minimum of 2.0 feet above the maximum 1% annual chance flood level. In addition, an emergency spillway will be provided at the maximum 1% annual chance flood level to ensure that the undetained 0.2% annual chance flood event does not overtop the embankment.

If the emergency spillway capacity is to be provided over the embankment, the spillway will be structurally designed to prevent erosion and consequent loss of structural integrity of the dam embankment. If the capacity is to be provided in a vegetated earth spillway separate from the embankment, the required width for a trapezoidal spillway with a control section can be estimated by the equation:

$$Bw = \frac{0.36Q - 0.7ZD}{D^{3/2}} \quad (Equation\ 6.1)$$

Bw = bottom width (ft)

Q = emergency spillway capacity (cfs)

D = design depth above spillway crest (ft)

Z = side slope, i.e., horizontal distance to 1 foot vertical (ft/ft)

The minimum width for a vegetated earth spillway is 4.0 feet.

Outflow Structure – Multiple-stage outflow structures are encouraged as a method for meeting the requirements of Section 2.3. Where the outflow structure conveys flow through the embankment in a conduit, the conduit shall be reinforced concrete designed to support the external loads with an adequate factor of safety. It shall withstand the internal hydraulic pressures without leakage under full external load or settlement. It must convey water at the design velocity without damage to the interior surface of the conduit.

Earth Embankment Design – The steepest side slope permitted for a vegetated earth embankment is 4:1 and 2:1 for rock dam or as determined by geotechnical investigation. The minimum crown width is shown in Table 6.1.

Table 6.1 Minimum Crown Width of Embankments

Total Height of Embankment (ft)	Minimum Crown Width (ft)
14 or less	8
15 - 19	10
20 - 24	12
25 - 34	14

Detention basins to be excavated must provide positive drainage with a minimum grade of 0.3%. The steepest side slope permitted for an excavated slope not in rock is 4:1.

Earth Embankment Specifications – Embankment specifications for earth embankments for dry detention basins must, at a minimum, be adequate for levee embankments and be based on NCTCOG Standard Specifications for Public Works Construction for embankment, topsoil, sodding, and seeding. For wet detention basins, findings from geotechnical investigations of the site may result in more stringent specifications.

Maintenance Provisions – Access must be provided in detention basin design for periodic desilting and debris removal. A 12 foot maintenance access strip shall be maintained around the perimeter of any pond that will be maintained by the City. Wet detention basins must include dewatering devices to provide for maintenance.

A vehicle access ramp shall be provided if the bottom width of the detention pond is 15 feet or greater and the height of the embankment or wall is greater than 4 feet. The access ramp should be graded no steeper than 15% and be a minimum of 15 feet wide.

Fencing around detention – Security fencing with a minimum height of 4 feet shall encompass the basin area when required due to potential safety hazards created by prolonged storage of floodwater. Design shall be such as not to restrict the inflow or outfall of the basin. In basins to be used for recreation areas during dry periods, pedestrian access may be provided with the approval of the Director. Access points from streets will be controlled by a locked chain or gate.

Slope stability, erosion control, and maintenance should be considered for landscaping in detention areas. See Appendix A.6 for guidance on appropriate landscape materials for stormwater detention.

The developer will be responsible for maintenance of the basin until all construction on adjacent lots is complete (i.e., the basin is not to be used for disposal of waste building materials), required vegetation is established, and it has been accepted by the City. Maintenance deeds shall be defined on the plat.

Concrete aprons and wingwalls will be used at all outlet structures. See Section 3.8 for outlet condition requirements.

All pipes discharging into a detention basin will be discharged at the basin's flowline with adequate erosion control. However, pipes may be discharged above the bottom of the detention basin with adequate erosion control and approval from the Director.

Refer to Section 11 for guidance on operation and maintenance plans.

6.2.2 Low Flow Channels / Pilot Channels

Basins with longitudinal slopes less than 0.5 percent shall have concrete pilot channels. The minimum bottom width of the pilot channel shall be equivalent to the width of the low-flow outfall structure. The minimum [earthen] slope draining toward the pilot channel shall be one percent.

6.3

RETENTION BASIN

Retention ponds impound water as it infiltrates through existing porous soils. Retention ponds must be designed to fully infiltrate runoff within 48 hours from the end of a 24-hour design storm. A detailed geotechnical analysis is needed for proposed retention areas confirming that the site characteristics are suited for retention, and approval is needed from the Director. See Section 6.10 for infiltration requirements of underlying soils.

If meeting the needs of required detention due to proposed development, retention ponds shall be designed to the requirements discussed in Section 6.2.

6.4

UNDERGROUND DETENTION

Underground detention is an option for sites with dense urban development, sites where above-ground storage is not feasible, or sites that wish to capture runoff for beneficial reuse. Underground detention can be achieved by vaults, multiple large-diameter pipes, etc. Designer should verify that design is compliant with all current water rights regulations.

The bottom of underground detention shall be at least 2 feet above seasonal high groundwater. Underground detention shall not be located underneath any structure unless approved by the Director. Relocate underground utilities as necessary; no underground utilities shall cross through the underground detention area.

If underground detention is located underneath an area with vehicular traffic, the structural design must meet the requirements for emergency vehicle loading.

Maintenance access is required at both ends of the underground detention facility. Spacing between access openings shall not exceed 50 feet unless approved by the Director. Covers, grates, and hatches shall be bolt locking. If the vault or pipe contains cells, one access per cell minimum is required. Confined space entry regulations shall be followed if applicable.

For underground detention pipe systems, there should be a minimum cover of 2 feet above the top of the pipe. The maximum fill depth to the top of the pipe shall be governed by geotechnical/structural considerations. Calculations must be provided to the City for review.

Operation and maintenance plans are required for all underground detention facilities in accordance with Section 11.

6.5

STORMWATER PONDS

Stormwater ponds are wet detention ponds that also provide water quality benefits. Refer to Section 6.1. They retain and treat runoff by gravitational sedimentation and biological uptake. Stormwater ponds also provide opportunities for wildlife habitation and aesthetic design.

Pretreatment or a sediment forebay shall be provided upstream. Permanent pools prevent the re-suspension of sediment. Several different options of stormwater ponds are available:

A wet stormwater pond has a permanent pool only storing the design water quality volume and can provide temporary storage for larger flows above the permanent pool elevation if desired.

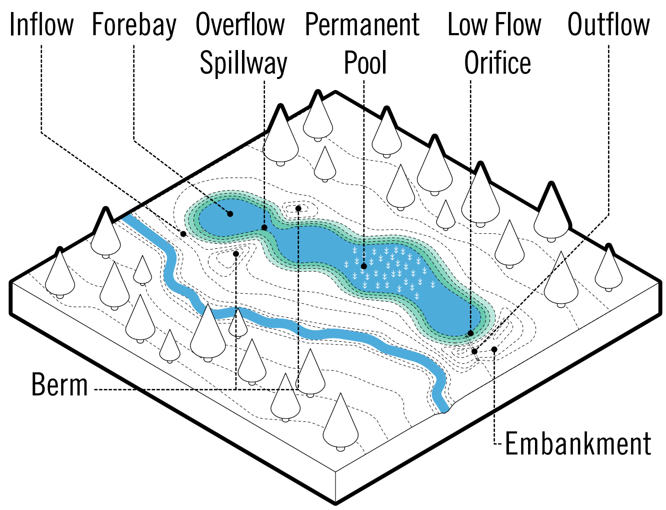
A wet extended detention pond splits the water quality volume between the permanent pool and the extended detention storage above. The extended detention is designed to be released over a minimum of 24 hours. The removal efficiency is similar to wet ponds, but it requires less space. If serving as required detention for proposed developments, it must meet the requirements in Section 6.2.

A micropool extended detention pond functions similarly to a wet extended detention pond, detaining the water quality volume for 24 hours. Only a micropool remains permanently in the pond, preventing resuspension of sediment and clogging. If serving as required detention for proposed development, refer to Section 6.2 for additional design requirements.

Multiple pond systems can consist of several ponds, storing the volume required for water quality volume and additional storage in two or more cells.

If space is available, stormwater ponds can provide required detention for development sites.

Figure 6.1 Stormwater Ponds



Site Considerations

Stormwater ponds should have a minimum contributing drainage area of 25 acres or 10 acres for an extended detention micropool pond.

Stormwater ponds should not be located on sites with a slope greater than 15% or in areas where seasonal high groundwater is present within 2 feet of the bottom of the pond.

If stormwater ponds are to be located in a navigable water or wetland of the U.S., a 404 permit and any other applicable permits must be obtained (See Section 9.2).

At a minimum, stormwater ponds must be set back 5 feet from buildings and roadways and 10 feet from property lines. Designer should determine if a geotechnical analysis is needed to decide if more stringent requirements are necessary.

If NRCS soil types A and B are present, a pond liner may be necessary to maintain a permanent pool.

Design Considerations

To achieve efficient sediment and pollutant removal, the following design measures should be observed.

Stormwater pond design should include a permanent pool, overlying zone if additional storage is desired, and a shallow littoral zone that acts as a biological filter. Other design features include an emergency spillway, maintenance access, safety bench, pond buffer, and landscaping.

Stormwater ponds should have a minimum length to width ratio of 1.5:1 with a permanent pool depth of 3-8 feet. Side slopes should be no steeper than 3:1. Safety benches with a maximum slope of 6% are required for any slopes steeper than 4:1.

An aquatic bench, which provides a shallow area for aquatic vegetation that acts as a biological filter, typically has a length of 15 feet and extends into the water at a maximum depth of 18 inches below the permanent pool level.

A sediment forebay should be present at every inlet to the pond unless the inlet provides less than 10% of the total design inflow. The forebay should be separated from the main pond. Velocities in the pipes or channels exiting the forebay should not be erosive.

Outlets can consist of risers, barrels, or weirs and can have multiple outlets at different heights. Energy dissipators should be placed at each outlet to prevent erosion.

When stormwater ponds are designed to serve as detention ponds to meet the requirements of Section 2.3, all detention basin requirements presented in Section 6.2 shall be met.

A drain must be provided that is capable of draining the pond within 48 hours unless water rights are attained. Maintenance access should be provided from a public road or easement and should have a minimum width of 15 feet and slope no greater than 15%.

Landscaping plays an important role in pollutant removal, safety, wildlife habitat, and aesthetic design. Wetland plants should be established within 6 inches of the normal pool elevation. The dam embankment and spillway are to remain free of large vegetation, such as bushes and trees.

Sizing

The permanent pool volume can be sized for a design water quality volume. See Section 2.3.3 for design water quality volume and Section 6.2 for detention pond design.

Orifices should be sized to release the streambank protection storage volume or extended detention volume in 24 hours. Embankment and spillways should be designed for the 1% annual chance storm event with the 0.2% annual chance storm event not overtopping the embankment.

6.6

CONSTRUCTED WETLANDS

Constructed wetlands are man-made wetland systems designed to treat and manage stormwater. They provide nutrient and pollutant removal as well as a habitat for wildlife.

Several different design options are available.

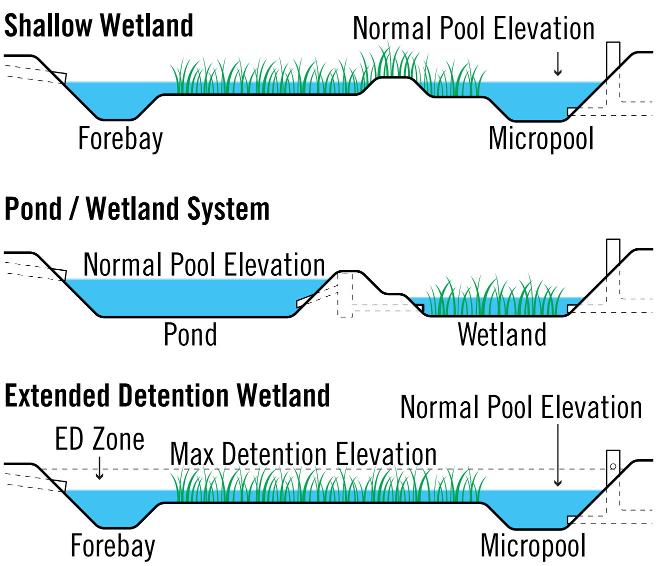
- A shallow wetland provides the majority of water quality treatment in the marsh area, which is relatively shallow. The forebay and micropool at the outlet are the only deeper portions.

- A pond/wetland system consists of two separate cells, a wet pond and a shallow marsh. Sedimentation occurs in the wet pond before the runoff enters the marsh for additional treatment.

- An extended detention shallow wetland is similar to a shallow wetland but also provides extended detention above the marsh which is released over 24 hours.

- A pocket wetland, which serves a drainage area of 5-10 acres, requires a groundwater supply to maintain constant aquatic life.

Figure 6.2 Constructed Wetlands



Site Considerations

Constructed wetlands should have a minimum contributing drainage area of 25 acres or 5 acres for pocket wetlands. Constructed wetlands should not be located on a slope greater than 8%.

Except for pocket wetlands, there should be a minimum of 2 feet between the bottom of the wetland and seasonal high groundwater.

If stormwater ponds are to be located in a navigable water or wetland of the U.S., a 404 permit and any other applicable permits must be obtained.

At a minimum, constructed wetlands must be set back 5 feet from buildings and roadways and 10 feet from property lines. Designer should determine if a geotechnical analysis is needed to decide if more stringent requirements are necessary.

If soil types A and B are present, a liner may be necessary to maintain a permanent micropool.

Design Considerations

The surface areas of constructed wetlands require a space that is approximately 30% of the contributing drainage area.

A constructed wetland should consist of a sediment forebay, a shallow marsh area with aquatic vegetation, a permanent micropool, and a detention zone to manage runoff volumes. Other design features include an emergency spillway, maintenance access, safety bench, wetland buffer, and landscaping.

A sediment forebay should be provided at each inlet unless the inflow is less than 10% of the total design storm inflow.

A minimum dry weather flow path should be provided from inflow to outflow with a length-to-width ratio of 2:1. This path should prevent short-circuiting. A constructed wetland requires a continuous baseflow to support vegetation. A 3-5 foot elevation difference (2-3 foot for pocket wetlands) from inflow to outflow is needed for positive drainage.

A minimum of 35% of the total area should have a depth of 6 inches or less, and 10-20% should have a depth of 1.5-6 feet.

Four different zones should be present.

- The deepwater zone should be 1.5-6 feet deep and includes the outlet micropool and deepwater channels. It can support submerged or floating vegetation.
- The low marsh zone is 6-18 inches below the normal pool elevation and can support several different wetland species.
- The high marsh zone is from the normal pool elevation to 6 inches below it. It should have a higher surface area to volume ratio. It can support a greater density of wetland plants compared to the low marsh zone.
- The semi-wet zone is located above the permanent pool and is only wet during large storm events. Plants in this zone must be flood tolerant.

A micropool with a depth of 4-6 feet should be designed at the outlet to prevent resuspension of sediment.

No permanent pool areas should exceed a depth of 6 feet. Extended detention should not exceed a water surface elevation of 3 feet above the normal pool.

If the constructed wetland is not designed to handle the 1% annual chance storm event, a bypass system should be constructed.

Outlets can consist of risers, barrels, or weirs and can have multiple outlets at different heights. Energy dissipators should be placed at the outlet to prevent erosion.

An emergency spillway shall be provided that meets all local and state dam safety requirements and does not impact downstream structures. A minimum of one foot of freeboard must be provided from the design water surface elevation to the lowest point of the embankment.

The designer should also review and consider inundation requirements for selected plants and perform water balance to ensure range in water depths over wet and dry periods so that the design:

a) does not require make up water

b) meets plant inundation requirement

Maintenance access should be provided from a public road or easement and should have a minimum width of 15 feet and slope no greater than 15%.

Landscaping plays an important role in pollutant removal, safety, wildlife habitat, and aesthetic design. The landscaping plan should encourage habitat for wildlife and waterfowl. Wooded vegetation should be present within 15 feet of the embankment or 25 feet from the main spillway. A wetland buffer of 25 feet from the maximum water surface elevation should be provided. Reference USACE/EPA constructed wetlands guidance for detailed design guidance.

Sizing

Constructed wetlands should have a minimum length-to-width ratio of 2:1. For efficient pollutant removal, it is recommended that it be designed based on the design water quality volume split between the marsh, pool, and extended detention areas as defined in Table 6.2.

Table 6.2 Constructed Wetland Volume Ratios

Allocation of V_{wq} in %	Shallow Wetland	ED Shallow Wetland	Pond/ Wetland	Pocket Wetland
Pool	25	25	70	25
Marsh	75	25	30	75
Extended Detention	0	50	0	0

In addition, the recommended ratios for surface areas of depth zones are defined in Table 6.3.

Table 6.3 Constructed Wetland Area Ratios

Allocation of Surface Area in %	Shallow Wetland	ED Shallow Wetland	Pond/ Wetland	Pocket Wetland
Deepwater	20	10	45	10
Low Marsh	35	35	25	45
High Marsh	40	45	25	40
Semi-wet	5	10	5	5

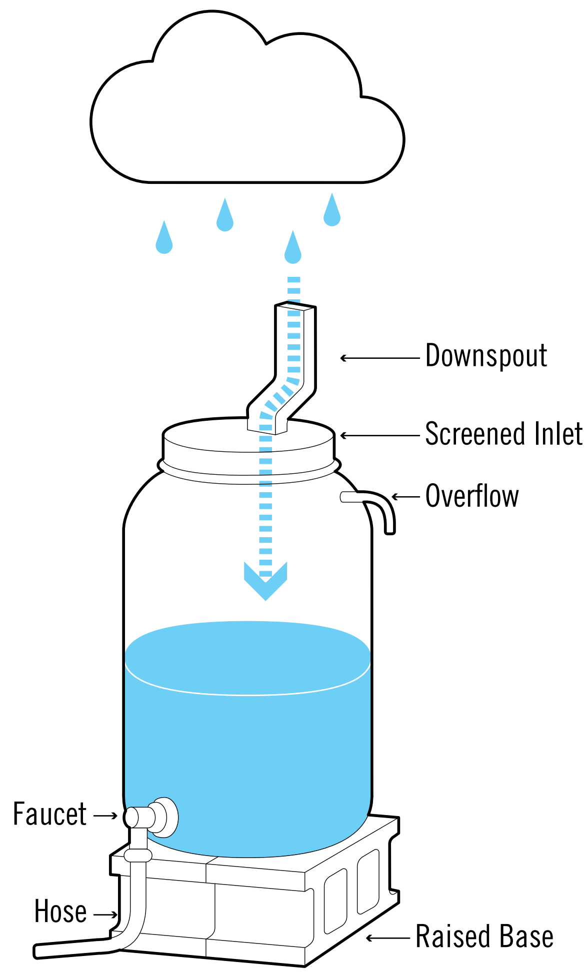
For extended detention shallow wetlands, orifices should be sized to release the streambank protection storage volume or extended detention volume in 48 hours, and the invert should be located at the maximum elevation for the extended detention water quality volume. If the extended detention wetland is being used to comply with the requirements of Section 2.3, all detention basin requirements of Section 6.2 shall be met.

6.7

RAINWATER HARVESTING

Rainwater harvesting consists of a system of capturing runoff from rooftops in barrels or cisterns for beneficial reuse. The system contains a downspout, a storage tank, and a method of dispersion. Rainwater can be captured for beneficial use, reducing the usage of potable water for non-potable water requirements and can be used for any non-potable application such as landscape irrigation, toilet flushing, and vehicle washing.

Figure 6.3 Rainwater Harvesting



Site & Sizing Considerations

Rainwater catchment tanks and cisterns can be used in any type of application and can be located above or below ground, when excavation is possible. There are many different configurations for aboveground rainwater storage with different size footprints. Besides the basic cylindrical cistern shape, rainwater can be stored in vertical boxes against a structure, in horizontal boxes below a deck, or even use the boxes to create a cosmetic fence or wall. Rain barrels or cisterns can be used in series to provide additional storage.

Gutters and downspouts from the roof can easily be diverted into a rain barrel, which when placed uphill of landscaping, can provide irrigation by gravity flow.

Design Considerations

Downspouts, piping, or other methods of conveyance can be used to runoff into storage tanks. Underground cisterns must be designed by a licensed qualified engineer. Design of rain barrels shall be consistent with TAC Title 30 Part 1 Chapter 210 Subchapter F.

Rain barrels should be opaque or painted if above ground to discourage algae growth and covered to reduce vector attraction. First-flush diverters or hydrodynamic separators should be used to prevent contaminants from entering the supply. Leaf screens and roof washers may be desired to prevent debris from entering the storage tank. Storage tanks must drain within 48 hours unless a cover is provided to prevent mosquito and vector attraction.

For new development, roof materials may be selected based on effectiveness for rainwater harvesting for optimization. Disinfection methods can also be implemented if potable use is desired. Backflow preventers are necessary to keep rainwater and municipal potable supply separate. Storage tanks should be located at a higher elevation than desired area of use to utilize gravity flow if possible. Pumps can be installed if needed.

Overflows should be designed to divert excess rainwater to additional storage, sustainable drainage measures, or storm drainage systems and should not discharge runoff toward structures or downstream properties.

If rainwater harvesting is used for landscape irrigation, the amount of supplemental irrigation necessary can be significantly decreased.

The following equation can be used to determine rain barrel volume.

$$V = A * R * 1\ ft/12\ in * 0.90 * 7.5\ gal/ft^3 \quad \text{(Equation 6.2)}$$

V = volume of rain barrel (gal)

A = surface area of roof (ft^2)

R = rainfall (in)

0.90 = system efficiency

6.8

RAIN GARDENS

Rain gardens are small bioretention areas that contain amended soil and native vegetation for receiving and temporarily storing stormwater runoff. Rain gardens can be designed for full, partial, or no infiltration into underlying soil, dependent on site and soil conditions. See Section 6.10 for infiltration requirements. Rain gardens provide hydrologic and water quality benefits as well as aesthetic benefits.

Site Considerations

Due to their ability to be designed for small sites or retrofits, rain gardens can be utilized for several different types of sites, including parking lot islands, roadway bulb-outs, and landscaping features. They should have a maximum contributing area of 5 acres per treatment cell. Pre-treatment may be required in areas with increased sediment, debris, and pollutants to limit maintenance needs and is required for facilities that have a contributing area of more than 2 acres.

Design Considerations

Rain gardens should have a ponding depth no greater than 9 inches and have a drawdown time of the full ponding depth of less than 24 hours (3-12 hours recommended). Rain gardens should not be built on slopes greater than 15% with side slopes of the ponded area no steeper than $2.5H{:}1V$ unless structural walls are used.

If the native underlying soil does not have a drawdown time of less than 24 hours for the entire ponding depth of the rain garden, but can still support some infiltration, a partial infiltration system can be used. In a partial infiltration system, outflow occurs through both a raised outlet pipe and infiltration into underlying soil. If the native soil supports a drawdown time of less than 24 hours for the full ponding depth, a full infiltration rain garden can be used. Otherwise, an underdrain must be installed that can convey the full design water quality volume. Underdrains are required when clay or geomembrane liners are used to protect infrastructure or utilities, or if the soils do not allow infiltration.

Runoff may enter the rain garden through sheet flow, curb cuts, curb cut forebays, or inlets. Where sheet flow enters the rain garden, a maximum side slope of 4:1 shall be used.

Method of inflow should prevent leaves and debris from entering the system. Energy dissipators or forebays may be needed for point concentrated inflows. Grass filter strips adjacent to the rain garden can slow inflow velocities. Inflow and outflow locations should be designed to prevent short-circuiting, when flow travels straight from inflow to outflow without the retention time in the system for infiltration, to maximize water quality benefits and reduce erosive effects. An overflow outlet shall be provided for runoff to bypass once the maximum ponding depth is reached. Inflow and outflow velocities shall not exceed 2 ft/s. When placing an overflow inlet next to a rain garden, grades should be set to establish preferential flow to the rain garden.

Overflow can be provided by adjacent inlets, vertical stand pipes, horizontal drainage pipes, or armored channels at the maximum ponding elevations which lead to surface waters, public storm drain systems, or storage reservoirs. Backwater calculations may be required depending on the capacity of the discharge system during large storm events. The overflow shall be sized for the 1% annual chance 24-hr storm event.

Overflow bypass can be provided by using off-line curb cuts or openings at the low side of the rain garden to allow runoff to enter the ponding area of the rain garden until full and then bypass to the storm inlet. Off-line curb openings prevent erosion and scouring impact to a rain garden; whereas, on-line openings may cause major erosion through the rain garden and wash mulch, soils, and plant materials into gutters or onto adjacent pavement surfaces.

Wood mulch shall be applied to the surface of the rain garden. Rain garden planting soil mix or soil amendments shall be installed over the ripped/tilled subbase soil. If no underdrain is required, then the mix may be stratified into the existing subbase.

Bioretention soil mix (BSM) shall be installed over the subbase soil, gravel storage reservoir, or underdrain and should be placed to a minimum depth of 18 inches and have an infiltration rate of 50 in/hr. Underdrain shall be placed at least 6 inches above bottom for incidental infiltration and to prevent clogging.

Additional storage can be provided by a gravel reservoir course underneath the rain garden or bioretention soil mix.

Impermeable or low permeability liners are required if a rain garden is located within 10 feet of a building or ROW. They may be necessary for no infiltration systems dependent on site conditions.

Rain gardens must be accessible from the right-of-way for maintenance using standard equipment.

Sizing

The following section contains two methods for sizing rain gardens and contributing drainage areas. The first set of equations is for determining the filtration area of the rain garden based on a known contributing area and a specified water quality volume. The second set determines the maximum allowable contributing drainage area for a known drainage area. A design water quality precipitation of 1 inch is chosen as the minimum amount of runoff treated that would still result in water quality benefits. If the designer can demonstrate significant infiltration at the project site that would result in a smaller required rain garden area for a partial or full infiltration system, pre-approval must be given by the Director.

Rain Garden Sizing Based on Known Contributing Drainage Area and Design Water Quality Volume:

The filtration area required, the surface area at the bottom of the rain garden basin above the soil mix, can be determined using the following equation.

$$A_f = V_{WQ} / (H + n_1 * d_1 + n_2 * d_2) \quad \textit{(Equation 6.3)}$$

A_f = filtration area (ft^2)

V_{WQ} = design water quality volume (ft^3) (See Section 2.3.3)

H = maximum head over the growing medium (ft)

n_1 = effective porosity of bioretention soil mix (typ. 0.30)

d_1 = depth of the bioretention soil mix (ft)

n_2 = effective porosity of the gravel reservoir course (typ. 0.35)

d_2 = depth of the gravel reservoir course (ft)

Sizing Contributing Drainage Area Based on Known Rain Garden Size and Design Water Quality Volume:

The following equation can be used to determine the contributing drainage area that can be routed to a rain garden and treat a design water quality volume. To find the maximum allowable contributing drainage area to a rain garden that will still provide water quality benefits, use the design water quality volume associated with a design water quality precipitation (P_{WQ}) of 1 inch of rainfall. A greater design water quality precipitation will provide increased water quality benefits.

$$A_{max} = \frac{12 * A_f (H + n_1 * d_1 + n_2 * d_2)}{D_{WQ}} \quad \textit{(Equation 6.4)}$$

A_{max}= contributing drainage area for specified rain garden (ft^2)

A_f = filtration area (ft^2)

H = maximum head over the bioretention soil mix (ft)

n_1 = effective porosity of bioretention soil mix (typ. 0.30)

d_1 = depth of the bioretention soil mix (ft)

n_2 = effective porosity of the gravel reservoir course (typ. 0.35)

d_2 = depth of the gravel reservoir course (ft)

D_{WQ}= design water quality capture depth(in) (See Section 2.3.3)

6.9

BIORETENTION

Bioretention areas collect, retain, and temporarily store surface water in soil void space and surface ponding from small drainage areas as well as providing water quality benefits. The water passes through a mulch layer, filter bed of soil or engineered soil media, plant root systems, and then infiltrates into native soils through a granular drainage base, or is collected in an underdrain system. Bioretention areas are similar to rain gardens but can be applied in different and larger geometries and support larger vegetation. Pollutant removal can be greatly increased by landscaping plant material selection, specific mulch type, and media design and depth within the bioretention area.

A bioretention planter, a small bioretention area confined to a typical street planter area, is typically designed with an underdrain, gravel, bridging stone and slotted pipe system, inspection ports or cleanouts, impervious walls, and an impervious bottom to prevent infiltration. A bioretention planter with infiltration is typically called an "infiltration planter," and a bioretention planter without is referred to as a "flow-through planter." Infiltration or soakage trenches are also a form of bioretention and are typically designed for full infiltration. See Section 6.10 for infiltration requirements.

Site Considerations

Bioretention areas can receive runoff from a contributing drainage area of up to 5 acres but should be limited to 0.5 acres if designed as an on-line system, where all runoff passes through the system before overflow continues downstream. Off-line systems can utilize a flow splitter to divert only the design volume to the bioretention area and send the overflow to other treatment areas or downstream. Multiple bioretention areas can serve a larger drainage area. Bioretention should not be used in areas with a slope greater than 6%.

Flow–through planters can receive runoff from roofs and residential, commercial, and mixed-use sites. Infiltration planters can also receive runoff from roadways, parking lots, and other paved surfaces.

Design Considerations

Method of inflow should prevent leaves and debris from entering the system. Energy dissipators may be needed for point concentrated inflows. Grass filter strips adjacent to the bioretention area can slow inflow velocities. Inflow and outflow locations should be designed to prevent short-circuiting to maximize water quality benefits and reduce erosive effects.

Ponding depth should be between 6-12 inches with a drawdown time of 24 hours and a minimum freeboard of 4 inches.

Bioretention soil mix (BSM) shall be installed over the subbase soil, gravel storage reservoir, or underdrain and should be placed to a minimum depth of 2.5 feet or 4 feet if large trees are planted and have an infiltration rate of 50 inches/hour. Additional BSM requirements can be found in Section 6.8 and the Materials Matrix in Appendix A.6.

Overflow can be provided by adjacent inlets, vertical stand pipes, horizontal drainage pipes, or armored channels at the maximum ponding elevations which lead to surface waters, public storm drain systems, or storage reservoirs. Backwater calculations may be required depending on the capacity of the discharge system during large storm events. The overflow shall be sized for the 1% annual chance 24-hr storm event.

Wood chips shall not be used as mulch to prevent clogging of the outlet.

Additional storage can be provided by a gravel reservoir course underneath the bioretention area or bioretention soil mix.

Impermeable or low permeability liners are required if a bioretention planter is located within 10 feet of a building. They may be necessary for no infiltration systems dependent on site conditions.

Sizing

A length to width ratio of 2:1 is recommended. Bioretention areas can be sized with the same methods as rain gardens. See Section 6.8.

6.10

INFILTRATION REQUIREMENTS AND TESTING

There is no place in Dallas where sustainable drainage design cannot be considered, and it is encouraged relative to compliance with current USEPA, TCEQ and FEMA regulatory requirements. Appropriate geochemical and geotechnical testing should be implemented as a part of the design process to determine appropriate design considerations for your particular location.

For sustainable drainage measures that use infiltration into native underlying soils, the design infiltration rate must be between 0.5 – 3 inches per hour. The design infiltration rate should be determined by applying a factor of 0.5 to the measured steady state saturated infiltration rate using the methods described in this section. If these requirements are not met for infiltration, an underdrain must be installed. The methodology outlined by the Commonwealth of Virginia may be used to develop appropriate underfilter design.

Methods for desktop study and field sampling shall be left to the discretion of the Designer. In-situ testing using hydraulic conductivity methods using appropriate ASTM standard test methods must be performed if infiltration into native underlying soils is proposed.

Media used in sustainable drainage measures including underdrain must meet the same design infiltration rate as described above to prevent ponding and clogging.

Desktop Study

Resources such as soil survey maps, reports, geotechnical, and other available data should be utilized to determine the feasibility of infiltration. Hydraulic conductivity can be determined based on location and soil type using data from the U.S. Department of Agriculture National Resources Conservation Service Soil Survey (NRCS.)

Field Sampling

Field sampling is done to determine depth and soil conditions at the proposed project location and should be done under the supervision of a qualified professional. Test pits, probes, borings, or similar methods should be carried out with a frequency of at least one test per 500 square feet. Test holes must extend to the minimum depth required for the sustainable drainage measure and corresponding requirements. Soil samples should be collected at the depth of expected infiltration and analyzed to determine an infiltration rate for the site.

In-situ Testing

In-situ infiltration or percolation testing will provide the most accurate infiltration rate. Several in-situ testing methods are available for determining the infiltration capacity of the soil and should be done under the supervision of a qualified professional. Infiltration testing methods shall be used above the water table and slug testing methods shall be used below the water table. Tests should be carried out with a frequency of at least one test per 2,000 square feet. Infiltration tests should be conducted as close as possible to the bottom elevation of the water quality control, but test location and depths should be adjusted in the field as necessary. If infiltration facilities are serving an area greater than one acre or there are other site conditions of concern, the City may require verification testing.

Determine if the observed infiltration rate is acceptable for the proposed measure. If not, an underdrain system must be provided.

Sustainable drainage measures shall not be placed in areas with industrial or hazardous runoff or high sediment loadings. Full or partial infiltration sustainable drainage measures are not permitted in areas that are within 2 feet of historical high groundwater from the bottom of the growing medium or where infiltration would contribute to soil or groundwater contamination. When full or partial infiltration systems are used, they must meet the infiltration requirements as stated in this section and must be set back a minimum of 5 feet from buildings and property lines to prevent adverse effects from infiltration. Designer should determine if a geotechnical analysis is needed to decide if more stringent requirements are necessary.

6.11

STORMWATER PUMP STATION DESIGN

The purpose of a pump station is to lift storm water runoff from a wet well to a receiving stream or outfall. The gravity system should be the primary and preferred means of discharging flow from a storm drain system. A pump station may be necessary when gravity outfalls are not economically or engineeringly feasible. A pump station may also be used in a water quality basin to discharge treated water into a receiving system.

The design guidance in this section is intended for stormwater pump stations only. The City of Dallas has a larger interior drainage system that utilizes sump lift stations. A pre-development meeting is required for projects relating to the city-wide interior drainage system, and for projects that require pump stations.

The pump station or pump wet well should be protected with fences, gates, and locks to prevent illegal entry. Adequate access should be available for service and maintenance vehicles during a storm event. Standard OSHA rules and other industrial standards of safety shall be followed for installation as well as maintenance of a pump station.

Figure 6.4 Sump Area with Drywell

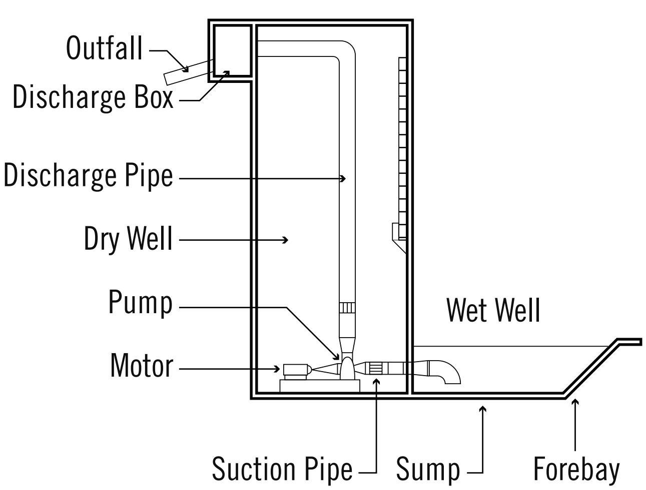
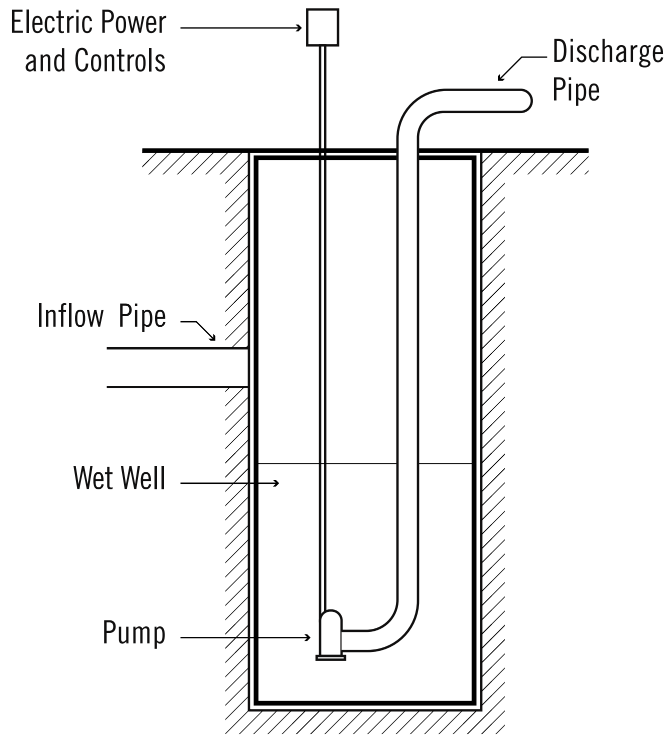


Figure 6.5 Wet Well



6.11.1 Design Guidance

Refer to FHWA Hydraulic Engineering Circular No. 24 (HEC-24), TxDOT Hydraulic Manual, and Hydraulic Institute standards for design and specifications of a pump station.

6.11.1.1 Pump Station Hydrology

In order to design a pump station effectively, the inflow hydrology must be known. The hydrology developed for the associated storm drain system usually will not serve as a firm basis for discharge determination into the pump station. A hydrograph is required because the time component is critical in understanding the inflow which governs the sizing of the wet well. The designer needs to know not only the peak inflow, but the timing and volume. The difference between the input and the output hydrographs is the storage requirements of the pump station wet well. The hydrograph should consider the storage abilities of the storm drain system, which may reduce the required size of the wet well. Governmental regulations or the physical limitations of the receiving waters determine the output discharge from the pump station.

The design frequency for a pump station will be the 1% annual chance storm event.

A mass inflow curve represents the cumulative inflow volume with respect to time. In order to determine a mass inflow curve, the hydraulic designer must first develop an inflow hydrograph based on a design storm. The most typical design method is the NRCS Dimensionless Unit Hydrograph and the procedure can be found in the FHWA Hydraulic Engineering Circular No. 24 (HEC-24).

6.11.1.2 Pump Station Hydraulics

The storage volume of the wet well should be less than the total volume of the wet well because allowances should be made for a sump and for freeboard. The sump is the volume of the wet well below the required minimum water level, which is the pump cutoff elevation. The wet well must maintain water above the pump inlet to keep the pump from attempting to pump dry or sucking air. The sump must also have room below the pump intake level for sedimentation and heavy trash that wash into the system.

The top of the storage volume determines the maximum water level, the level in the wet well above which the water should not be allowed to exceed. Any freeboard above the maximum water level is not included in the calculated storage volume. Pumping is initiated at or below the maximum water level, and is stopped when the water drops to the minimum water level.

Other spaces outside of the wet well which store storm water before flooding occurs can also be considered part of the available storage volume. These include sumps, pipes, boxes, inlets, manholes, and ditches of the storm drain system. The storm drain system can represent a significant storage capacity.

The selected rate of discharge from the pump station determines the number and size of pumps required for the facility. However, pump selection is a matter of economic analysis by the designer. A decision must be made using several manufacturers' technical data and whether a single or number of pumps would be necessary. A backup pump is required, and will be of the same discharge rating. A slightly lower pump rate than the allowable discharge is fine but the lower rate requires a larger wet well volume.

The designer must also consider the cost of construction and physical restrictions for the wet well. Enlarging the wet well and using fewer pumps might be a reasonable alternative to a larger wet well. In situations where one pump may be able to supply the entire discharge necessary, a minimum of two smaller pumps is recommended for reliability and maintenance. Multiple pumps also offer the opportunity for a staggered startup of pumps. Manufacturer's printed technical data and a sales or technical representative can be invaluable sources at this stage of the design in selecting the right pumps. The final design and pump selection must be based on all considerations.

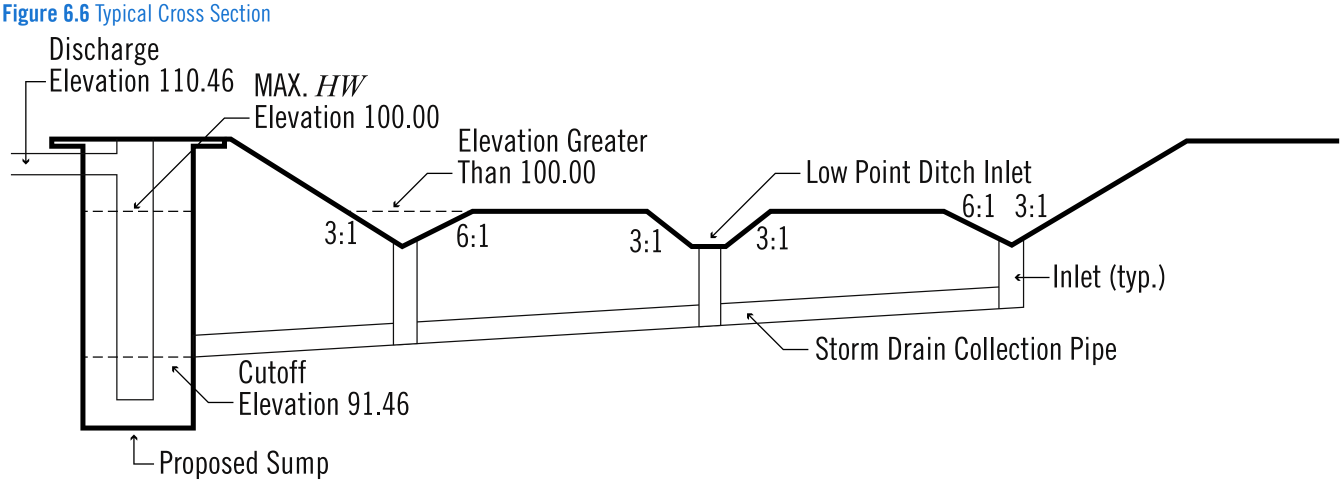
6.11.1.3 Operation and Maintenance

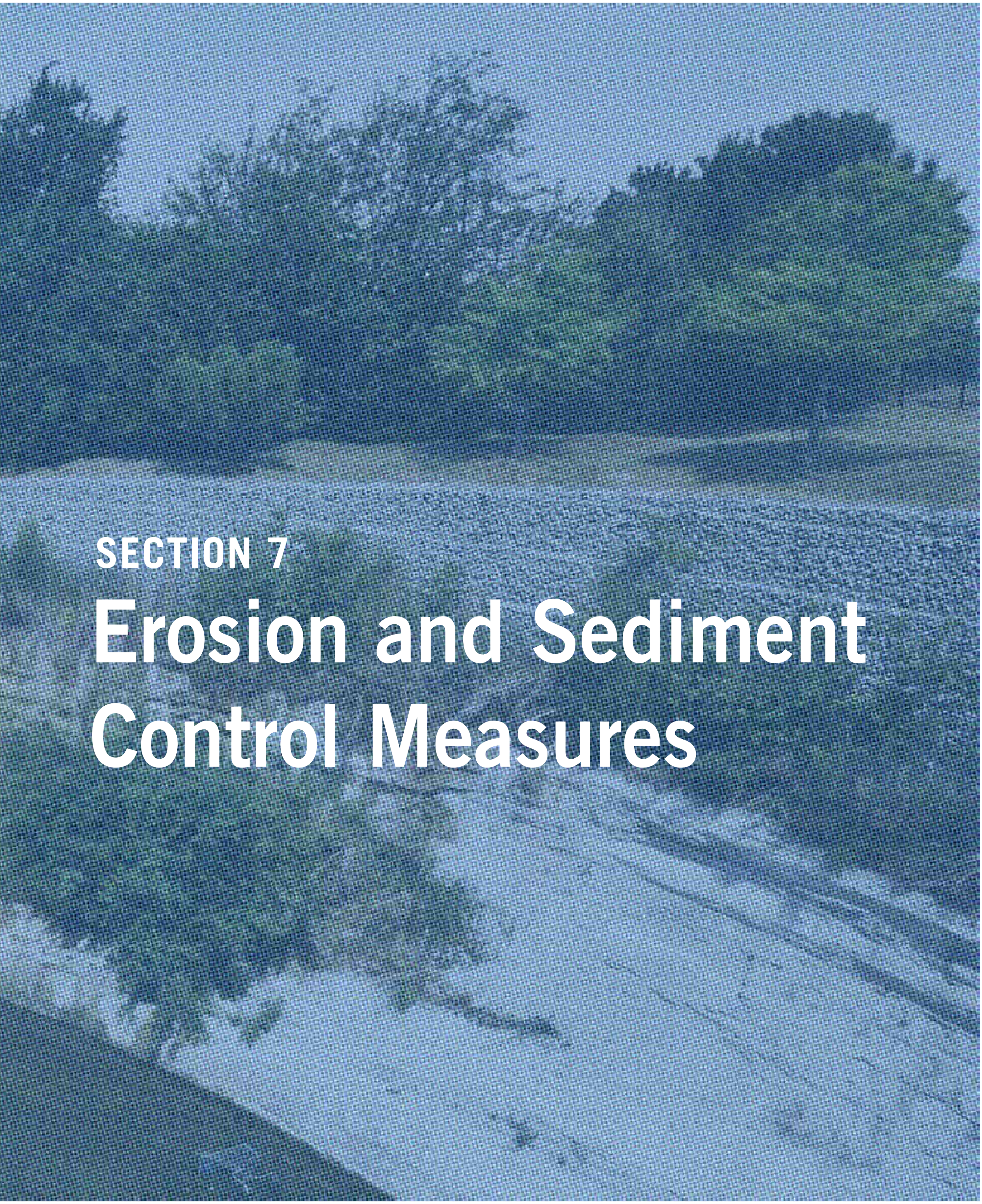
During pump station operation, all OSHA and local safety requirements must be adhered to. An entry plan should be developed as part of the operation and maintenance procedures for the pump station. The plan should identify measures to be taken prior to and during any visit to the pump station, including monitoring of environmental conditions, especially air quality.

All semantics, product information, and operational manuals should be provided by the contractor or engineer to the owner upon completion and acceptance of the pump station.

An operation and maintenance schedule should identify the frequency of inspection in regards to the need for debris and sediment removal, as the build-up may cause a reduction in pump efficiency and possible failure. A provision should be added for future pump maintenance and/or removal due to failure or reduced durability of the pump due to possible unforeseen circumstances. FHWA Hydraulic Engineering Circular No. 24 (HEC-24) has a general list of problems, causes, and solutions available. A full performance test should be periodically performed to check the continued operating efficiency of the pumping station.

Figure 6.6 Typical Cross Section





SECTION 7

Erosion and Sediment Control Measures

7.1

GENERAL

All projects shall be designed so that erosion is minimized during construction as well as after the construction is completed. The volume, rate and quality of storm water runoff originating from development must be controlled to prevent soil erosion. Specific efforts shall be made to keep sediment out of street and water courses. BMPs (best management practices) for construction should be coordinated with permanent erosion control measures.

Creek bank erosion and sedimentation are natural processes, but can be aggravated by human causes. Erosion is typically a concern along the outside curves for curved channel banks, and areas located within the Dallas escarpment and geologically similar areas (the Dallas escarpment areas are shown on the City of Dallas Zoning website). Typically, soils that are not protected by vegetation or armoring are more susceptible to erosion processes. As a result, sediment can be carried and deposited in a stream, and may have negative impacts on aquatic life. Severe erosion can also lead to bank failure, which can be caused by removal of vegetation or alteration of the creek channel. Whenever possible, vegetated buffer zones shall be used between developed areas and creeks. Barren slopes or disturbed areas of creeks shall be stabilized and revegetated. Planted erosion control mats or other slope stabilization methods shall be used on earthen slopes steeper than 2:1 or in areas of high velocities.

7.2

NATURAL CHANNEL SETBACK

Natural channel setbacks are delineated buffers around natural channels in which no development can occur. The setback prevents damage from large flood events and preserves natural riparian habitats.

The natural channel setback shall be included in the Drainage Easement for the channel.

The natural channel setback shall be computed by each of the two following methods. The method that results in the larger setback shall be used to define the minimum natural channel setback. See Figure 7.1 for an illustration of the two methods. For the purpose of defining the natural channel setback, the following terms & definitions shall apply.

Crest means the line at the top of the bank where the channel slope becomes flatter than four to one. Toe means the line at the bottom of the bank where the slope becomes steeper than four to one.

Method 1

A line that is formed by the intersection of the surface of the land and the vertical plane located a horizontal distance of 20 feet beyond the crest.

Method 2

A line formed by the intersection of the surface of the land beyond the crest and a four to one sloping plane for clay or shale soils that begins at the horizontal location of the toe, but at an elevation equal to the flowline of the stream. For other soils, a three to one sloping plane can be used.

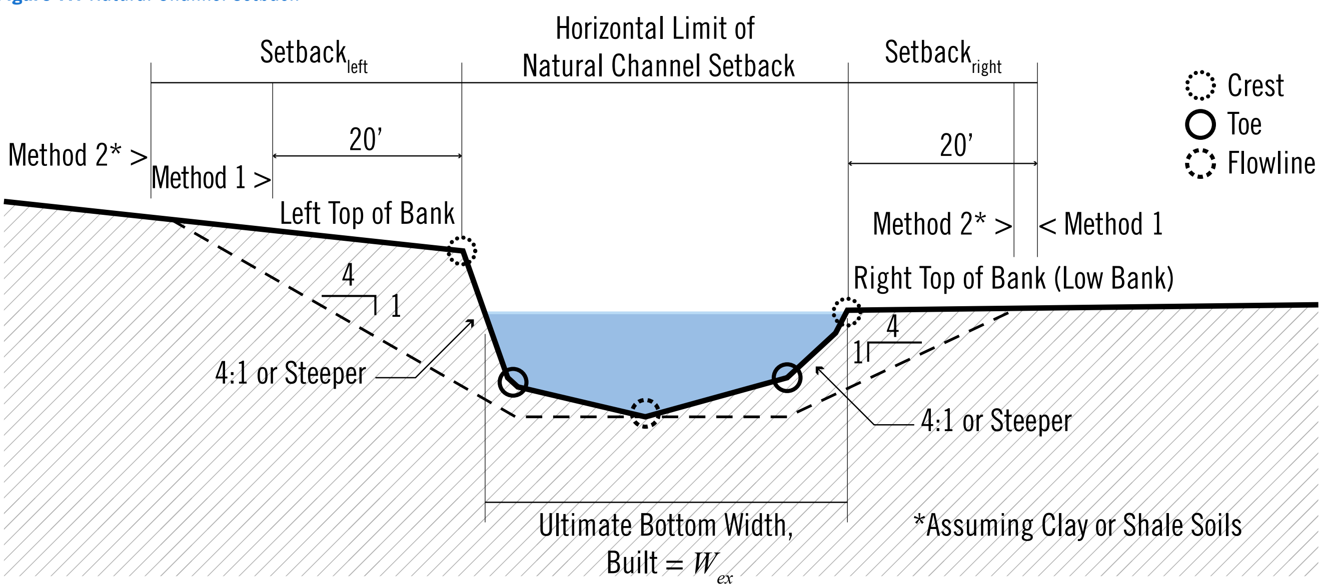
Establishment of a smaller natural channel setback may be approved by the Director under either of the following scenarios:

- A structurally engineered retention system is designed to prevent erosion of the channel slope.
- A geomorphic assessment is prepared and sealed by a professional engineer providing justification for a smaller setback.

Establishing, and maintaining project setbacks within this area can help to maintain these natural buffers.

For additional requirements for natural buffer during construction, refer to TPDES General Construction Permit 150000.

Figure 7.1 Natural Channel Setback



7.3

TEMPORARY CONSTRUCTION EROSION CONTROL

Where an NPDES (National Pollutant Discharge Elimination System) Construction General Permit is required for construction of a project (under regulations contained in 30 TAC, under the authority of the Clean Water Act), a Storm Water Pollution Prevention Plan (SWPPP) meeting the permit requirements must be prepared, included with the plans and specifications, and posted onsite during construction.

SWPPP must be prepared and implemented on the site before any construction activities begin, including grading, and must be continuously updated.

In addition, all projects shall comply with the requirements for storm water management at construction sites as set forth in the Construction General Permit, TXR150000 as well as Texas Water Code Chapter 26 and TAC Chapter 30. Construction plans and specifications must include the types of management, structural, and source control measures, known as BMPs.

Technical guidance for the preparation of a Storm Water Pollution Prevention Plan and implementation of BMPs for construction sites may be referenced in the iSWM Construction Controls Technical Manual.

The owner and operator are responsible for maintenance of erosion and sedimentation control measures and must remove sediment resulting from construction from City right-of-way or storm drainage systems.

7.4

PERMANENT EROSION CONTROL DESIGN

7.4.1 Grass / Vegetation

Vegetation in channels should have grasses as described in Appendix A.6 with a good, deep root structure to stabilize the soil from erosive velocities.

7.4.2 Turf Reinforcement

There are a number of turf reinforcement mats (TRM) and high performance turf reinforcement mats (HPTRM) that are available to the design engineer. Selection and installation of the TRM or HPTRM is critical to the stability of the earthen channel. The TRM will provide scour protection and enhance the vegetative root and stem development. Design aids for the design of TRM and HPTRM can be found in FHWA HEC-15, Design of Roadside Channels with Flexible Linings.

7.4.3 Bioengineering

Bioengineering methods can be used instead of rock armoring structural methods in appropriate scenarios. While bioengineering techniques may be cheaper than other methods, they should only be used in channels where the methods can withstand high-flow velocities. Many bioengineering methods can be used in combination with, or in addition to hard armoring. Designers are encouraged to reference the NRCS publication NEH-634, “Stream Restoration Design” for detailed discussions of bioengineering design methods. Several bioengineering options are presented in Figure 7.2.

Figure 7.2 Bioengineering

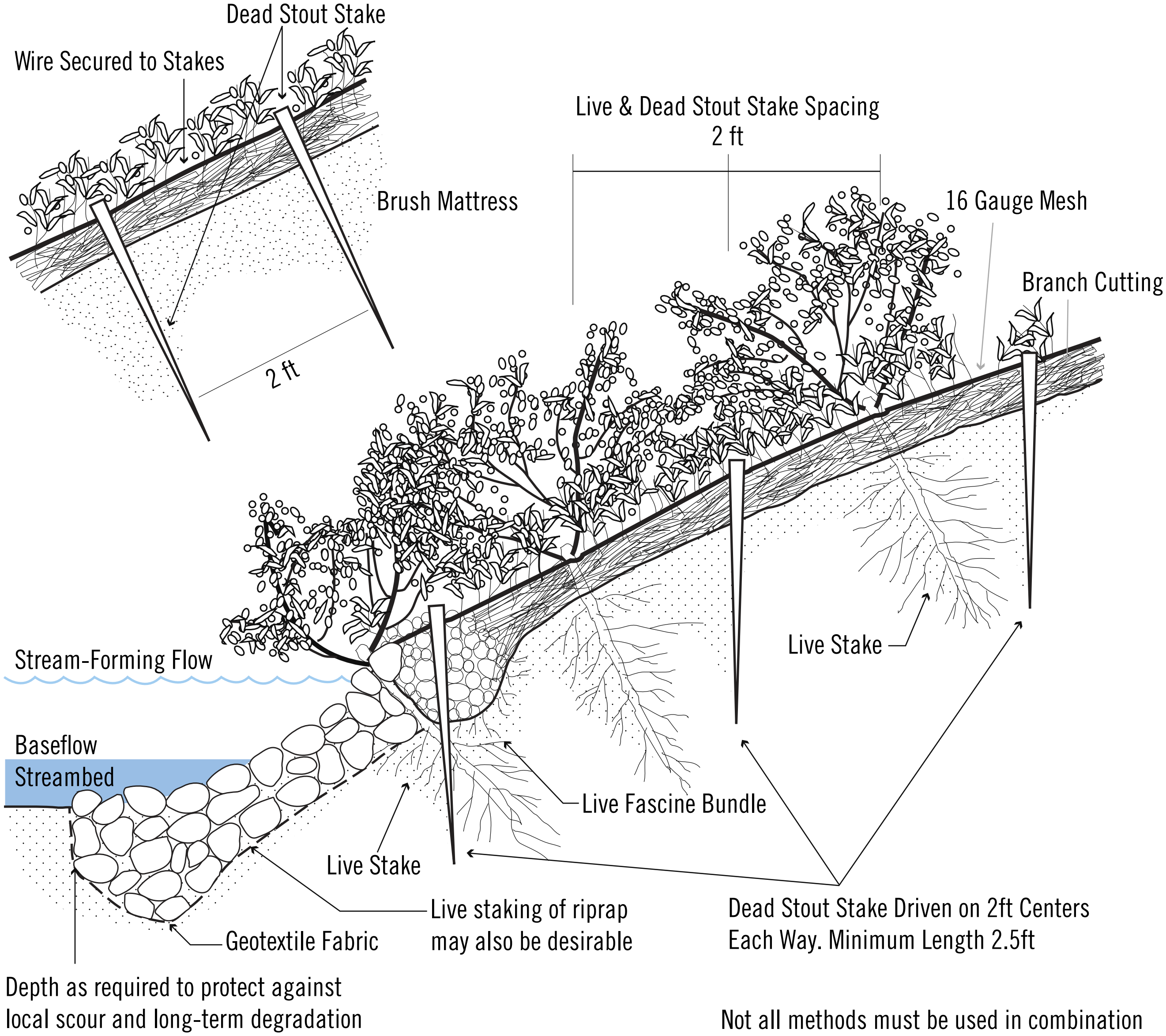
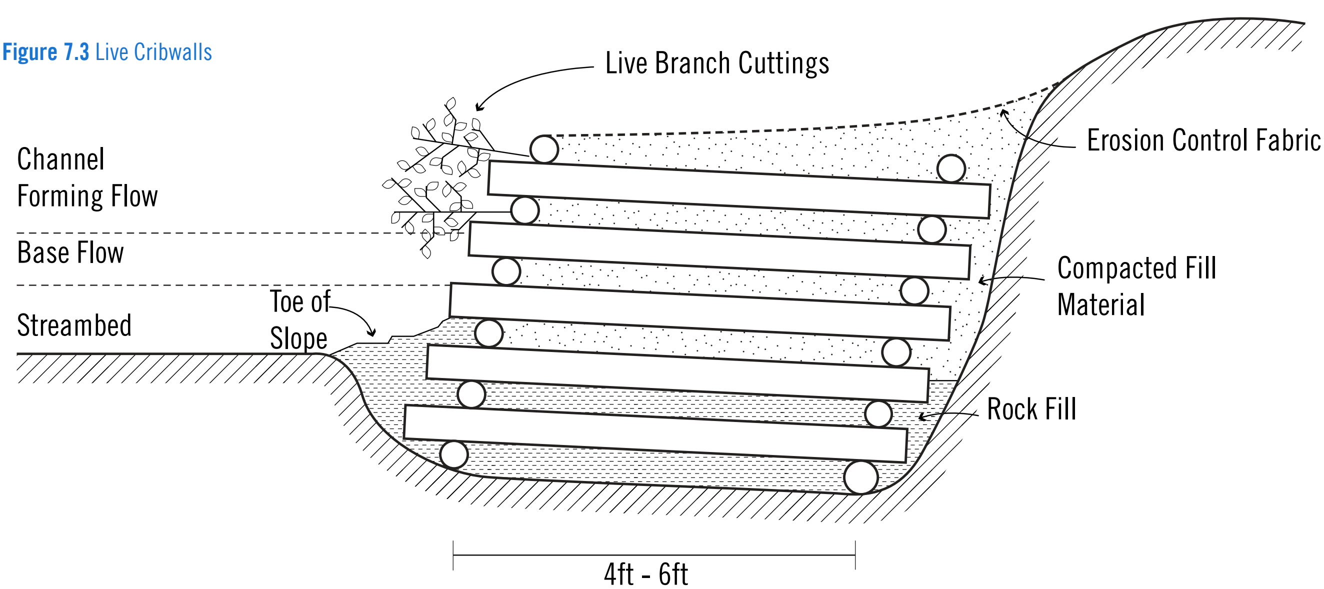


Figure 7.3 Live Cribwalls



In bioengineered vegetation lining, living plants are used to provide protection from streambank erosion, and the roots provide stability and hold the soil in place. After high flows, the roots provide for natural release of water, pulling the water from the saturated soil. This method will require time to integrate into the existing soil matrix before being fully established. Growing season of plants utilized in the design shall be considered and temporary irrigation provided where appropriate.

Live Staking

Live staking consists of using live vegetative cuttings either to stabilize the soil or secure mattresses or other erosion control elements. As they grow, the strength of the system increases. The benefits of live staking include low cost and easy installation.

Wattles

Wattles, or live fascines, are branch cuttings that are bundled, tied, and staked in shallow excavated trenches. Once the wattle is placed in the trench, it is back-filled until only the top of the wattle is showing.

Brush Layering

Brush layering consists of creating a series of benches with live branches covered with soil and compacted. The branches stabilize the slope, especially as they take root and grow. The edges of the branches that stick out of the slope help reduce surface erosion.

Brush Mattressing

In brush mattressing, a mattress-like branch layer is secured to the surface of the bank to reduce erosion.

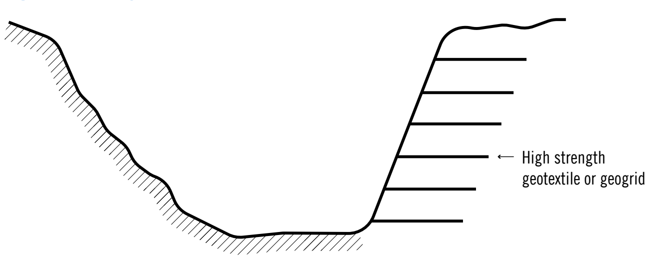
Live Cribwalls

A live cribwall is built using logs or timber tied with live branch cuttings. See Figure 7.3. They can be used for slope protection either along the streambank or at the toe of the slope.

Geogrids

Geogrids are geotextiles wrapped around lifts of soil and compacted to provide structural stability and protection against erosion. Materials range from plastics to natural geotextile such as coir.

Figure 7.4 Geogrid



7.4.4 Stone Riprap Design

Stone riprap is an effective tool for preventing erosion in many different circumstances. Riprap can be effective in circumstances such as: around headwalls or pipe outfalls, at the toe of a slope or on a steep slope in a channel, around structures in the channel, and areas subject to wave action.

Design requirements and uses vary greatly. National Engineering Handbook (NEH) Technical Supplement 14C provides design guidance for sizing riprap. The median size of stone, thickness of riprap layer, gradation of size, angularity, and length of protection should all be designed on a site-by-site basis. The following design method can be used for riprap in low turbulence areas. For high turbulence areas, riprap placed in impinging flow, or other special conditions, refer to NEH Technical Supplement 14C for other design methods.

The USACE – Maynord method should be used for design of riprap along channel banks. Use of this method satisfies the requirement to verify that the proposed lining have a permissible shear stress that is higher than the expected shear stress. This method is not applicable for high-turbulence areas. The equation calculates D_{30}, the stone size for which 30% of the stone mix is finer than by weight. D_{30} is typically 15 percent smaller than D_{50}. D_{85} shall lie between $1.8D_{15}$ and $4.6D_{15}$. The equation to determine D_{30} is as follows:

$$D_{30} = FS\, C_s\, C_v\, C_T\, d\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5}\left(\frac{V}{\sqrt{K_1 g d}}\right)\right]^{2.5}$$

(Equation 7.1)

D_m = stone size (ft); percent finer by weight

FS = factor of safety (usually 1.1 to 1.5), suggest 1.2

C_s = stability coefficient for 2:1 slope or flatter $C = 0.3$, (0.3 for angular rock, 0.375 for rounded rock)

Cv = velocity distribution coefficient (1.0 for straight channels or inside of bends, calculate as below for outside of bends)

CT = thickness coefficient
(use 1.0 for D_{100} or $1.5D_{50}$, whichever is greater)

d = water depth (ft)

Y_w = specific weight of water (62.4 lb/ft^3)

Y_s = specific weight of stone (lb/ft^3)

V = local velocity; if unknown use $1.5V_{average}$

K_1 = side slope correction as computed in Equation 7.3

g = acceleration due to gravity = 32.2 ft/s^2

The outside bend velocity coefficient and the side slope correction can be calculated:

$$C_v = 1.283 - 0.2\log\left(\frac{R}{W}\right) \quad \textit{(Equation 7.2)}$$

R = centerline bend radius

W = water surface width (ft)

The side slope correction can be calculated:

$$K_1 = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \quad \textit{(Equation 7.3)}$$

(equation only applicable for angles of rock smaller than the repose angle)

θ = angle of rock from the horizontal

ϕ = angle of repose (typically 40°)

The proper placement of riprap is just as important for effectiveness at erosion prevention as the correct sizing. Angular stone shall be used to promote interlocking. Minimum riprap bed thickness shall be 1.5 times the D_{50} size. Stones shall be placed using a method where the stone sizes are not segregated to prevent voids that leave the subgrade unprotected. A geotechnical fabric or filter bed shall be placed underneath the riprap to prevent erosion or migration of bank material and undermining of the underlying material. The area where riprap is placed shall be clear of obstructions, debris, and tree roots that may damage or dislodge the riprap.

7.4.5 Gabions

Gabions are rock filled wire baskets or mattresses. Gabions can be used similar to rock riprap, but usually the size of rocks are of a smaller diameter. A filter blanket below the basket or mattress is required to keep the subgrade soil from migrating into the gabions. The gabions shall be installed per the manufacturer's instructions and proper anchoring and toe downs are required.

Gabion mattresses are shallow and can be used to protect the bed or banks of a stream against erosion.

Gabion baskets can be used to stabilize slopes where mattresses are not adequate to stabilize slopes and can also be used to construct drop structures, pipe outlet structures, or protect other areas with erosive velocities. The baskets are made from welded or woven wire mesh that is galvanized or coated in plastic to prevent corrosion. While vegetation can grow within the baskets, large woody vegetation shall not be allowed to grow within the gabions to prevent structural damage to the baskets. Design methodology and guidance for gabion products are available from most manufacturers. Design aids for the design of gabion mattresses and gabion baskets can be found in FHWA HEC-15, Design of Roadside Channels with Flexible Linings.

7.4.6 Concrete

The lining of a channel with concrete may be necessary for erosive velocities, or confined channel areas.

7.4.7 Concrete Block Mats

Concrete block mats are flexible, interlocking systems of uniform concrete blocks that can be used for hard armor erosion control. They allow for flexibility, quick installation, and minor changes in the bank geometry. While providing hard armoring, they also allow for drainage through the system and can allow for vegetation. Concrete block mats do not act as retaining walls and must be placed on stable slopes with a filter layer. Refer to manufacturer guidelines for design information.

7.4.8 Retaining Walls in Waterways

This section discusses only retaining walls within a creek environment. For retaining walls in other environments, refer to Section 4 of the Street Design Manual.

All retaining structures/walls located within a 1% annual chance floodplain in the City of Dallas shall be designed for the specific onsite conditions and approved by the Director. Special structural designs, including modifications of the 251D-1 Standard Construction Details, shall be submitted with supporting calculations.

Retaining walls shall be designed to achieve a minimum factor of safety of 2 against overturning and 1.5 against sliding, unless otherwise approved by the Director.

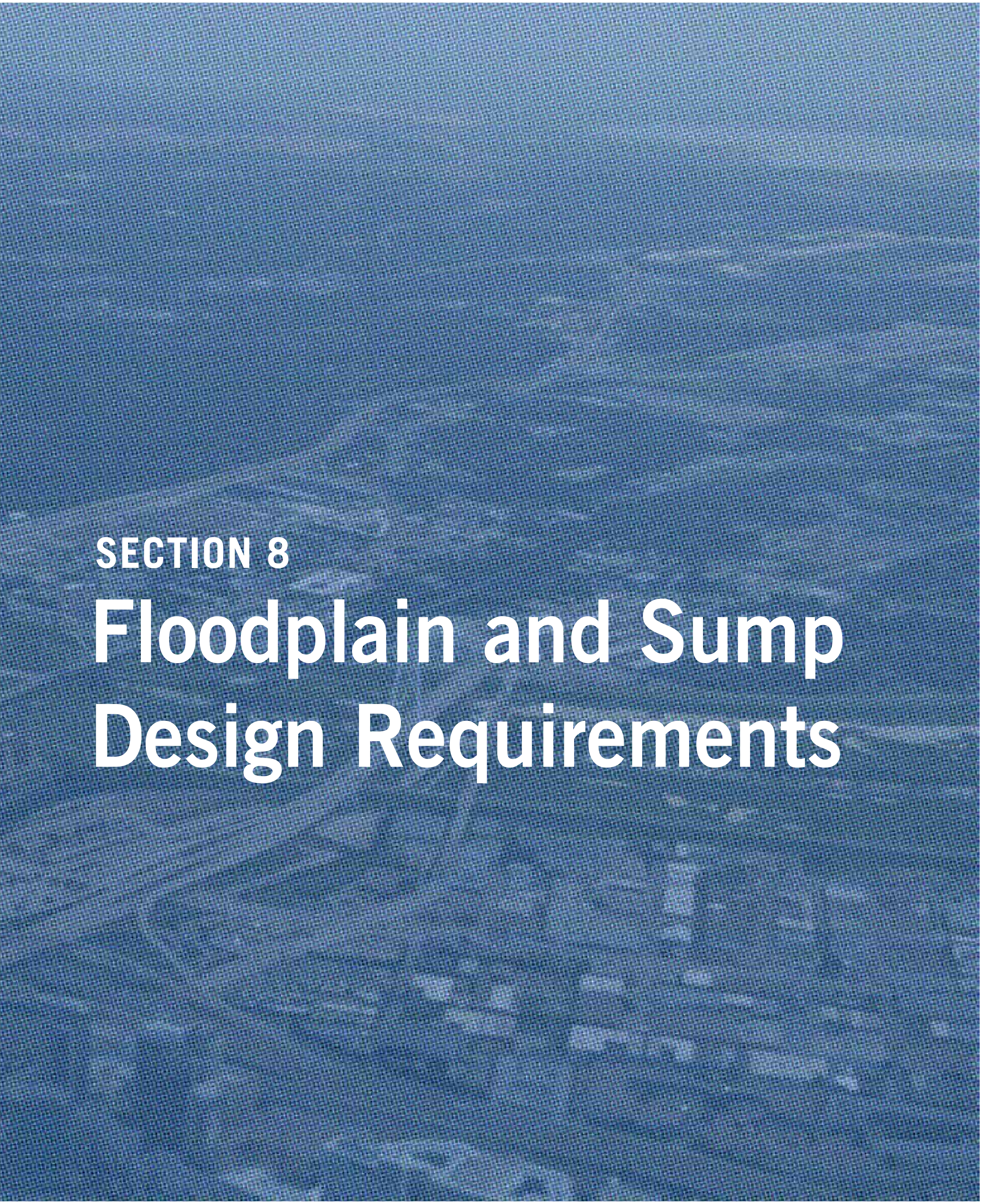
Retaining wall design shall consider the following parameters/criteria:

1. Allowable soil and/or rock bearing capacity
2. Surcharge loadings, existing and future
3. Hydrostatic pressure due to stormwater, groundwater, irrigation, etc.
4. Backfill drainage (perforated pipe or weep holes)
5. Uplift if applicable
6. Resistance to sliding (The potential for future deterioration of materials at the toe of the structure, and the subsequent decrease in passive resistance pressures should be considered)

7. Location of slip plane for proposed conditions (must ensure that plane is not located below wall footing)
8. Erosion at the ends of the wall over top of wall and undermining at the toe
9. Adequate room or right-of-way for construction of the footing
10. Placement of construction and expansion joints
11. Potential for impact or abrasion (Gabions and similar materials should be avoided in areas subject to direct impact from debris or falling water)
12. Maintenance requirements
13. Lateral loads due to onsite material or select fill
14. Proper compaction of backfill
15. Sustained velocity and shear stress

Base layers of gabions should be placed below expected scour depth. If retaining wall does not extend above the expected water surface elevation for the design flood, measures such as tiebacks to protect against flanking should be installed.

Any wall taller than 4 feet in height will require a building permit and an engineer's certification that the wall is structurally sound and built as per plan specifications. Any wall taller than 6 feet in height must comply with OSHA requirements during construction and future maintenance. Any retaining wall that extends beyond the natural creek bank, blocking the natural flow of water in any way, must be analyzed hydraulically per Article V Section 51A-5.100-5.105 to verify it has no impact.



SECTION 8

Floodplain and Sump Design Requirements

8.1

GENERAL

Development within floodplain or sump areas requires special consideration. The City of Dallas is susceptible to flooding and has regulations in place to minimize loss of life and property. Increasing development throughout the City and within the floodplain areas can lead to increased runoff and flooding. No development may occur that raises flood water levels within floodplain areas or adversely impacts runoff storage capacity. Valley storage requirements in Article V, Section 51A-5.100 shall be met. Special consideration should be given to ensure that there is no increase in water surface elevation within a project, upstream or downstream.

8.2

FLOODPLAIN HYDRAULIC ANALYSIS

Development within or adjacent to a floodplain must comply with the floodplain regulations, Article V, Section 51A-5.100-5.105, and coordinate with the appropriate regulatory agencies. A floodplain / floodway study must be completed if a project site is in or adjacent to a mapped floodplain area to determine that there is no negative impact from development. Major floodplain / floodway studies must conform to FEMA regulations described in Part 65 of 44 Code of Federal Regulations, and the City of Dallas Development Code.

The designer is responsible for the collection of all existing data with regard to flooding in the study area. This search could include published reports, specific information on past flooding, drainage structures in the area, available topographic maps, available Flood Insurance Rate Maps (FIRM), and photographs of past flood events.

If the floodplain is impacted by encroaching development, proposed floodplain exhibits should include (but not be limited to):

- Existing and proposed contours
- Location, size, and elevations of proposed structures
- Filled and excavated areas with proposed contours and grades
- Location and elevation of roads & utility lines
- Existing and proposed floodplain boundaries
- Cross-section locations
- Property boundaries
- Survey information, including benchmarks and control points

See Article V Sections 51A-5.100-5.105 of the Dallas Development Code and Procedures for Filling in a Floodplain Under the Flood Plain Management Guidelines for additional floodplain regulations.

8.2.1 Hydraulic Modeling

Floodplains and floodways should be modeled using approved software, a list of which can be found on the City of Dallas website. Further explanation of the different types of modeling techniques and appropriate uses can be found in the HEC-RAS Hydraulic Reference Manual.

Cross sections used in the hydraulic analysis shall be representative of current channel and floodplain conditions obtained by surveying. When cross section data is obtained from other studies, the data shall be confirmed to represent current channel and floodplain conditions, or new channel cross section data shall be obtained by field survey. All information should be referenced directly to the most updated vertical datum. If other datum was used for the source data, conversion information is available through the National Geodetic Survey.

8.2.1.1 1D Steady Flow Modeling

1D steady flow modeling can be used when flow is steady, gradually varied, one-dimensional, and the slope of the river is less than 1:10. 1D flow assumes that the velocity components are only in the direction of flow; that all flow is perpendicular to the modeled cross-sections. Velocity in directions other than the direction of flow are not accounted for. Steady flow modeling assumes that conditions at a given point in the stream remain constant with respect to time. Steady flow modeling should not be used when flow is significantly influenced by other factors, such as dam breaching, levee overtopping, flow reversals, or very flat areas where significant floodplain storage may impact flow.

8.2.1.2 1D Unsteady Flow Modeling

1D unsteady flow modeling allows the user to model the change of depth over time during a storm event as discharge increases and then recedes. The assumption of one dimensional velocity is maintained. 1D unsteady flow modeling is often utilized where significant floodplain storage is expected to have an impact on modeling results. In an unsteady flow model, an upstream boundary condition is required where a flow hydrograph for the reach is applied.

8.2.1.3 2D Unsteady Flow Modeling

2D unsteady flow modeling eliminates the assumption that velocity is one-dimensional. Rather than using a cross-section to estimate one-dimensional velocity perpendicular to the cross-section, the terrain is subdivided into a grid, where depth and velocity are computed for each grid element across the terrain. Velocity in any horizontal direction is thus accounted for in 2D modeling and the water surface elevation is not assumed to be constant across a floodplain. 2D unsteady flow modeling is often used where significant flow is expected to occur that would not be perpendicular to a traditional 1D cross-section. Examples of such instances are wide floodplains, roadway crossings with multiple opening bridges, or other floodplain obstructions that invalidate an assumption of one-dimensional flow. Another common application is the modeling of shallow floodplains in urban settings where it is desirable to understand the path, depth, velocities, and timing of runoff that traverses through urban areas.

While 2D unsteady flow modeling can require more effort than 1D modeling, the 2D modeling can often provide additional information and a more detailed understanding of flooding conditions. In previously developed areas where an undersized storm system results in localized flooding, a 2D unsteady model can aid assessment of potential downstream impacts that may result from upgrading a system to eliminate the localized flooding and the existing storage associated with the inundated areas. For more guidance on 2D modeling, reference the HEC-RAS River Analysis System 2D Modeling User's Manual (Version 5.0 February 2016 or latest version) or other approved software reference manuals.

8.3

REGULATORY FLOODPLAIN MAPS

Floodplain maps may be found through FEMA, but coordination is required with the Director to ensure that the latest updated floodplain maps are used for design.

8.4

FLOODPLAIN APPROVAL PROCESS

If a project is within 100 feet of a stream, the designer shall check with the City to determine whether the project is impacted by a floodplain.

A fill permit shall be used to remove an area from a designated floodplain. A floodplain alteration permit shall be used when there is alteration on the site, but it is not removed from the floodplain. Fill or floodplain alteration permits are pre-cursors to a building permit. Work may not begin after the approval of a fill or floodplain permit until all other building permits are approved.

If storage is provided to offset fill in accordance with Article V Section 51A-5.100-5.105, it must be within property boundaries. City right-of-way or easements shall not be used for storage requirements.

The following sections shall be met for both fill and floodplain alteration permits as per the current floodplain regulations at the time of the adoption of this manual.

8.4.1 Fill Permits

Fill is prohibited within a floodplain unless it is demonstrated that all criteria are met and a fill permit is approved. If a fill permit is approved, property can be developed to a specified elevation. Refer to Article V Section 51A-5.100-5.105 of the Dallas Development Code for guidance on regulations and permitting within floodplains. A FEMA Letter of Map Revision is needed to re-designate an area that was previously in the floodplain. A Corridor Development Certificate (CDC) may be needed before a fill permit can be approved if located within the Trinity River Corridor.

Building pads must be a minimum of two feet and the finished floor a minimum of three feet above the design flood elevation. The water surface elevation of a creek or river may not be increased upstream, downstream, or throughout the project unless in a detention area. Post-project velocities downstream, if above 6 ft/s, shall not exceed existing velocities.

8.4.2 Sump Fill Permits

A fill activity within a sump area requires a fill permit, using the method outlined herein, consistent with the Floodplain Ordinance. If fill action for a project is located within the critical zone for a levee system (see Figure 8.1), it will require a fill permit. In addition, the following items shall be reviewed for applicability: ROW/easement abandonment, 404 and 408 permitting (See Section 9.2), and USACE Section 10 Harbors Act approval. All floodplain permits require an application meeting to support full understanding of project requirements.

8.4.3 Floodplain Alteration Permits

A floodplain alteration is defined as "the construction of buildings or other structures, alterations, mining, dredging, filling, grading, or excavation in the floodplain which does not remove a floodplain designation." Floodplain alteration permits are needed for structures such as swimming pools, tennis courts, fences, and significant landscaping that lie within a floodplain. A Corridor Development Certificate (CDC) may be needed before a floodplain alteration can be approved if located within the Trinity River Corridor.

8.4.4 Other City Permits

If a project includes moving more than 50 cubic yards of earthen material, then a grading permit is required.

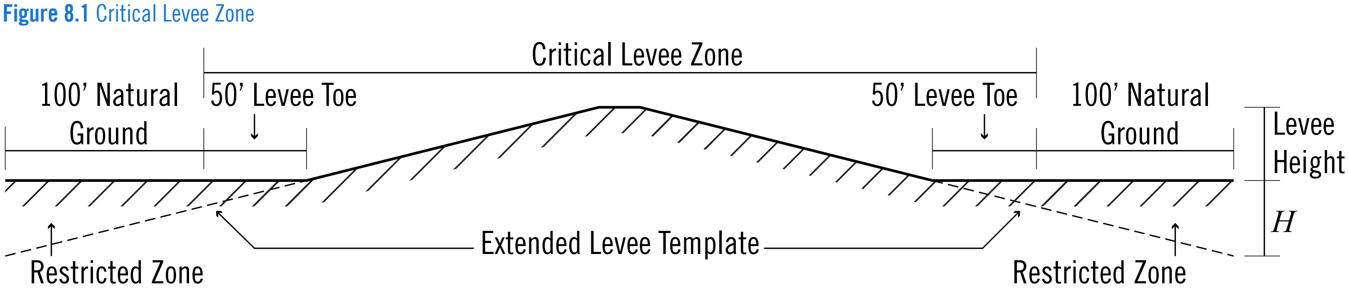
8.5

ENHANCEMENTS AND RECREATIONAL USES

Multi-purpose detention areas can be sized to temporarily store a portion or all of the volume of runoff required to control the flood mitigation storm, if required. Multi-purpose areas should not be used for extended detention or water quality treatment. Routing calculations must be used to demonstrate that the storage volume is adequate. See Section 6 for detention storage design. Emergency overflows are to be provided for storm events larger than the design storm. The overflow must have no adverse impact to downstream properties or the conveyance system. Damage of property and conveyance systems should be minimized. Maintenance plans after flooding events are required.

Parks, sports fields, etc. may be built within the floodplain if a floodplain alteration permit is approved, but consideration should be given to the probable risk of flooding. Sports fields can serve as detention basins by constructing berms around the facilities. Outflow can be controlled through the use of an overflow weir or other appropriate control structure. Proper grading must be performed to ensure complete drainage of the facility. Recessed public common areas can also be utilized for stormwater detention but should be designed to flood infrequently. Maintenance considerations should be taken into account during design.

Figure 8.1 Critical Levee Zone



8.6

FEMA COORDINATION

Coordination with FEMA may be necessary if development is proposed within an existing floodplain.

A Letter of Map Amendment (LOMA) shall be submitted to FEMA for a property that has been inadvertently mapped in the floodplain but is on higher ground than the base flood elevation.

A Letter of Map Change (LOMC) shall be submitted to FEMA if the floodplain area or designation is revised or amended based on the Flood Insurance Rate Map (FIRM).

A Letter of Map Revision (LOMR) shall be submitted to FEMA if physical measures affect the hydrologic or hydraulic characteristics of a flooding source and thus result in the modification of the existing regulatory floodway, the effective Base Flood Elevations (BFEs), or the Special Flood Hazard Area (SFHA). A LOMR officially revises the FIRM and and sometimes the Flood Insurance Study (FIS).

A Letter of Map Revision based on Fill (LOMR-F) is FEMA's modification of the SFHA shown on the FIRM based on the placement of fill outside the existing regulatory floodway.

A Conditional Letter of Map Revision (CLOMR) can be submitted on a proposed project before construction but will not be added to the FIRM until it is confirmed to have been built as proposed.

See FEMA's online Flood Map Service Center for more information about flood maps and submittals. The City's approval is required prior to any submittal to FEMA for FIRM map revisions.

8.7

SUMP AND LEVEE AREAS

Pre-development meetings with Director are required for any project in sump and levee areas.

8.7.1 Sumps

Sumps are drainage features of levee systems that temporarily store stormwater runoff before it is conveyed to a river system by pumping over or draining through a levee. These areas must have a method of drainage to prevent standing water and allow for periodic maintenance.

8.7.2 Levee Setback

The levees are designed as a safety measure to prevent frequent flooding in areas near the floodplain. The levees must be maintained and periodically inspected for structural integrity.

Refer to USACE's EM 1110-2-1913 for guidelines on design and construction in and around levees and ETL 1110-2-583 for guidelines on landscape planting and vegetation management at levees. The Critical Levee Zone is the area of highest concern and subject to the more stringent requirements set forth by USACE for justifying any proposed modifications. The Restricted Zone is defined as the outer boundary of the 150-foot area from each levee toe for which reviews are required. Projects within this defined zone may require 404 and/or 408 permitting. Refer to Section 9.2 for further guidance.

SECTION 9

Other Regulatory Requirements

9.1

GENERAL

The design engineer is responsible for obtaining approvals and maintaining compliance with any State or Federal regulations, permits, and programs including TCEQ, EPA, U.S. Army Corps of Engineers (USACE), FEMA or other agencies. The developer is required to provide the City with copies of all approved permits and associated submittals required by any State or Federal agency for developments within City of Dallas. Refer to Article V Section 51A-5.100-5.105 for critical facility requirements.

9.2

ENVIRONMENTAL AND CONSTRUCTION PERMITTING REQUIREMENTS

9.2.1 Water Quality

Refer to TPDES, which is under the provision of Section 402 of the Clean Water Act and Chapter 26 of the Texas Water Code and Chapter 19, Article IX, Section 19-118 of the Dallas City Code for water quality and pretreatment requirements.

These regulatory requirements set forth water quality standards for surface water, requiring permits for point discharges of pollutants into the Waters of United States.

9.2.2 Tree Mitigation

Trees reduce stormwater runoff and are a natural filtration system. Care should be taken to protect the root system of a tree to maintain healthy trees and receive the full benefit of decreasing stormwater runoff and improving water quality.

New earthen facilities and alterations to existing facilities shall be planted with drought resistant, low growth, native species grasses which will allow unobstructed passage of floodwaters. Existing vegetation shall be preserved where possible. Even if existing above ground vegetation must be removed, remaining root systems may help stabilize soil while new vegetation is established.

Refer to Article X Sec. 51A-10.101 for landscape and tree preservation regulations. Natural vegetative buffers shall be maintained along riparian corridors if feasible in accordance with the TPDES Construction General Permit TXR150000.

A mitigation plan will be required if the above minimum preservation requirements are not met.

9.2.3 Section 404 Permitting

Section 404 of the Clean Water Act regulates the discharge of dredged fill materials into the Waters of the United States, including, but not limited to jurisdictional wetlands. Many projects fall under a general permit at the nationwide, regional, or state level. As the administrator of Section 404 permits, USACE provides guidance on when a nationwide or individual permit is needed for fill within the Waters of the United States. Permit applications can be found on the USACE website.

9.2.4 Section 408 Permitting

Section 408 of U.S. Code Title 33 Chapter 9 Subchapter I, regulates the alteration, occupation, or use of USACE civil works project. Permission is granted by the Secretary of the Army Corps if the use or modification "will not be injurious to the public interest and will not impair the usefulness of such work." Permit applications can be found on the USACE website.

The area comprising the federal project through Dallas is loosely defined as the areas within 1/4 mile of Dallas floodway and the Dallas floodway extension.

9.2.5 Aquatic Relocation

Endangered fresh water mussels and several species of concern may be identified in the Trinity River and tributaries through Dallas. In addition to CWA Section 404/408 permit requirements, construction or maintenance activities in or near public waters may require a Biologic Assessment and Aquatic Relocation Plan by a certified biologist in accordance with Texas Parks & Wildlife Code 66.015. These permits are implemented by the Texas Department of Parks & Wildlife in coordination with City Staff. The Biological Assessment may also be used to support Section 404 permit requirements, as coordinated with the USACE, through City of Dallas.

9.2.6 Construction Permit Requirements

9.2.6.1 TPDES Construction General Permits

Construction activity must be performed in compliance with the Texas Pollutant Discharge Elimination System (TPDES) Construction General Permit as authorized by TCEQ and in accordance with City of Dallas Code Section 19-118.

9.2.6.2 City Grading and Fill Permits

City of Dallas grading and fill permits are often required for construction activities. The City of Dallas website should be consulted to determine whether a specific project will be required to obtain grading and fill permits. Questions regarding permit requirements should be discussed in a predevelopment meeting.

9.2.6.3 Floodway Access Permits

All activity on the stream side of the levee toes within the Trinity River Floodway requires a Floodway Access Permit available from the City. Contact the Director for additional information.

9.3 EASEMENTS

Drainage easements will be required for all storm water management facilities accepting runoff from properties other than the lot on which the facility exists or will be constructed.

Additionally, drainage easements are typically necessary when storm water is to be conveyed across private property from public property, public right-of-way areas and easements, or public infrastructure to an established channel, creek, or other public drainage system.

Easements shall be defined as discussed in Article VIII Section 51A-8.100.

Public conveyance systems should be located within the public right-of-way when possible. If the public drainage system is located on private property, a public drainage easement shall be provided. Expenses associated with easement acquisition are the responsibility of the Owner / Developer.

All easements shall be shown on project plans and identified as private or public.

9.3.1 Storm Sewer Line Easements

Storm sewer easements shall be a minimum of 15 feet wide. Additional width may need to be dedicated for the easement based on the size of the storm drain pipe or width of a box structure in accordance with Table 9.1. Storm sewers are to be located within the center of the drainage easement. Retaining walls are not permitted within or adjacent to a drainage easement in order to reduce the easement width.

Table 9.1 Easement Widths

Total Structural Width (in)	Easement Width (ft)
39” and under	15’ wide
42” – 54”	20’ wide
60” – 66”	25’ wide
72” – 102	30’ wide

For storm drain facilities that are deeper than 20 feet, increase storm drain easement width by 4 feet for each additional foot of depth.

9.3.2 Detention / Retention Area Easements

All detention and retention facilities shall be located within drainage easements. Detention basins constructed through Private Development Activities shall be maintained by the developer/owner or neighborhood association. The City of Dallas provides maintenance only of regional detention facilities or detention basins constructed for the City. Detention area easements must be dedicated on the plat when detention facilities are on-site and dedicated by separate instrument when detention facilities are off-site.

9.3.3 Access Easements

All Floodway Management Areas, Floodway Easements, Detention Easements, and Drainage Easements shall include provisions for adequate maintenance such as dedicated and maintained Access Easements. These shall be sufficient to provide ingress and egress for maintenance. Access Easements are needed only when the area to be maintained does not border a public right-of-way. The access easement conditions shall prohibit the property owner from installing any structures, improvements, retaining walls, etc. which would hinder access to the drainage facility or necessitate restoration of access easement area. Access easements are to be provided outside of slope easement.

9.3.4 Slope Easements

A slope easement shall be provided along all natural and constructed channels where the depth from top of bank to flowline is greater than 5 feet. For constructed channels, the slope easement shall extend a minimum of 10 feet horizontally from the top of bank. For natural channels, the slope easement shall extend a minimum of 10 feet from the edge of the natural channel setback as determined by Section 7.2.

9.3.5 Subsurface Easements (Tunnels / Tieback Anchors)

Infrastructure placed underground (tiebacks, tunnels, etc.) requires a subsurface easement. Easement language for subsurface easement is to be discussed with the City Attorney.

Some erosion control projects for streams <10 feet from structure will require subsurface easements for tie back anchors.

9.3.6 Utility Considerations

Where drainage facilities and utility infrastructure are in close proximity or cross one another, easement configuration shall consider those utilities including the space needed for long term operation and maintenance and the possibility of future replacement, including upgrading and upsizing. Potential impact of utility crossings upon channel conveyance must be considered for all aerial utility stream crossings.

9.4

COORDINATION WITH OTHER AGENCIES

Projects must meet requirements of other regulatory agencies to receive City approval. Submittals to other regulatory agencies may require additional time and cost for review and processing. Project budgets and timelines should be planned accordingly.

The designer is responsible to ensure designs comply with all relevant drainage design criteria and related policy, such as Federal Highway Administration (FHWA) Policy & Memos, Code of Federal Regulations Title 23 Part 650 Subparts A and B, Federal Emergency Management Agency (FEMA), and regulatory agencies (for example US Army Corps of Engineers (USACE), Environmental Protection Agency (EPA), etc), appropriate local agency criteria (Cities, Counties, General Improvement Districts, Irrigation Districts), and to ensure the drainage design is compatible with other criteria and design from other TxDOT Divisions (Legal, Right-of-way, Materials, Bridge, Traffic/Safety, etc.).

9.4.1 County Requirements

The designer shall comply with all applicable county permits for the County wherein the project is located.

9.4.2 NCTCOG

The NCTCOG CDC process aims to stabilize flood risk along the Trinity River. The CDC process does not prohibit floodplain development, but ensures that any development that does occur in the floodplain will not raise flood water levels or reduce flood storage capacity. A CDC permit is required to develop land within a specific area of the Trinity floodplain called the Regulatory Zone, which is similar to the 1% annual chance (100-year) floodplain. Refer to the NCTCOG Common Vision Program website for CDC Permit requirements.

9.4.3 TCEQ

Refer to TCEQ Texas Commission on Environmental Quality Storm Water Permit Program for water quality and TPDES construction permits, water rights permits, and dam safety permits.

9.4.4 TxDOT

Any project that includes construction within TxDOT right-of-way must include coordination with the TxDOT Dallas District and must obtain all applicable TxDOT permits necessary for project design and construction activities.

9.4.5 USACE

Coordination with the USACE is necessary if proposed development is within the Trinity River floodway, or impacts the flow of "waters of the United States" as defined by 40 CFR 230.3(s) as:

1. All waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide;
2. All interstate waters including interstate wetlands;
3. All other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce including any such waters:
    - Which are or could be used by interstate or foreign travelers for recreational or other purposes; or
    - (From which fish or shellfish are or could be taken and sold in interstate or foreign commerce; or
    - Which are used or could be used for industrial purposes by industries in interstate commerce;
4. All impoundments of waters otherwise defined as waters of the United States under this definition;
5. Tributaries of waters identified in paragraphs (1) through (4) of 40 CFR 230.3(s);
6. The territorial sea;
7. Wetlands adjacent to waters (other than waters that are themselves wetlands) identified in paragraphs (1) through (6) of 40 CFR 230.3(s); waste treatment systems, including treatment ponds or lagoons designed to meet the requirements of CWA (other than cooling ponds as defined in 40 CFR 423.11(m) which also meet the criteria of this definition) are not waters of the United States.

When applicable, a jurisdictional wetland delineation shall be performed consistent with the guidelines set forth by the USACE. The delineation shall identify the jurisdictional extents of the Waters of the U.S., as defined in the most recent Waters of the U.S. Rule.

Refer to USACE's EC 1165-2-220 and SWFP 1150-2-1 for guidance when designing and constructing within federal project limits.

Refer to Sections 9.2.3 and 9.2.4 for guidance on 404 and 408 permitting. Refer to USACE's SWFP 1150-2-1 for criteria on design and construction within the limits of existing federal projects and EC 1165-2-220 for policies and procedures for alterations to USACE projects.

9.4.6 Texas Parks & Wildlife

If construction or maintenance activity is taking place in or near public waters, an aquatic resource relocation plan may be needed. Projects that are eligible include pipelines, trenching, road/bridge construction, dam work, dewatering, stream bank restoration, etc. Refer to Texas Parks & Wildlife Code 66.015 for regulation on aquatic species within public waters.

Additionally, certain projects may require compliance with Texas Parks & Wildlife Code Title S, Subtitle I, Chapter 90.

9.4.7 FEMA

See Section 8.6 for coordination with FEMA for activities within a FEMA-designated floodplain.

9.4.8 DART and Other Rail Systems

Any project that includes construction within the right-of-way of DART (Dallas Area Rapid Transit) or any other rail system must include coordination with the appropriate angency and must obtain all applicable permits necessary for project design and construction activities.

9.4.9 Neighboring Cities

If the project is located in multiple cities, coordination is required between all involved entities. The more stringent criteria shall be used for the project.

SECTION 10

Submittal Requirements

10.1

GENERAL

This section outlines the steps involved in preparing submittals and construction plans for the City. Some variation for private development plans is expected; specific guidance should be obtained from the City of Dallas Sustainable Development and Construction Department. Refer to the Street Design Manual for additional requirements for storm drainage plans submitted with paving plans and for drafting standards.

10.2

PLATTING / DEDICATION OF WATER COURSES AND BASINS

Property developments containing Floodway Management Areas, Floodway Easements, Detention Easements, or Drainage Easements shall have on the plat standard language addressing the easements and management areas, and on-ground monumentation.

Fill or development is prohibited in designated or undesignated 1% annual chance floodplain areas except as allowed under the floodplain fill or floodplain alteration permit process (Article V of the Dallas Development Code, Division 51A-5.100-5.105).

10.3

DOCUMENTATION REQUIREMENTS

All supporting documentation must be submitted. Hardcopy and electronic sets of supporting documentation must be submitted. Refer to the Street Design Manual for additional requirements for storm drainage plans submittal. Hydraulic models and relevant GIS, CAD, or Microstation files must be included on a CD, flash drive, or other electronic media as approved by the City.

10.3.1 Drainage Reports

A hard copy of the drainage report should be submitted to the City and a PDF of the report must be included on a CD, flash drive, or other electronic media as approved by the City.

The purpose of the Drainage Report is to identify drainage impacts resulting from land development activities and determine the improvements necessary to control the increase in storm water runoff and to treat the pollutants that can adversely impact water quality. Refer to Article V Section 51A-5.100-5.105 of the Dallas Development Code for submittal requirements for floodplain permits.

The following guidelines shall be followed as applicable to the project. Site-specific documents and additional information may be required. The drainage report narrative shall include the following elements as appropriate:

- Project description (size of the project, location, background information relevant to drainage design).
- Topographic conditions, precipitation data, land use and soil type exhibits.
- Pre-development basin information (On-site & Off-site drainage facilities, natural or constructed, conveyance systems, flows, water surface elevations, and any other special features near project).
- Post-development basin information (Size of roofs & driveways, curve numbers/runoff coefficients, flows, water surface elevations, impervious and pervious areas of each sub-basin).
- Identify and discuss the probable impacts down-gradient of the project site.
- The hydraulic methods and storm events used in sizing the drainage facilities, including the BMPs proposed for the project.
- Discussion of downstream capacity.
- Discussion of future development.
- The results of the calculations and a description of the proposed storm water facilities shall be included.
- A table comparing the pre-developed and post-developed conditions at project outfalls including rates and volumes shall also be included.
- Sufficient information shall be provided about the operation of the storm water system.
- The anticipated location of any off-site easements shall be identified either on the basin map or in a separate schematic. Off-site easements will be required for proposed storm water conveyance or disposal facilities outside the project boundaries. These easements shall be obtained and recorded prior to the acceptance of the final Drainage Submittal.
- A discussion of any expected future impacts on or connections to existing or proposed regional facilities shall be included.
- Existing and proposed drainage area maps.
- Applicable FEMA floodplain maps
- The Drainage Report shall incorporate all calculations used to determine the size of the facilities. Typical calculations include, but are not limited to:
    - Hydrologic/hydraulic calculations including pre- and post-developed peak rate and volume calculations, routing calculations, design information for outflow structures, orifice information, a pond volume rating table or pond volume calculations
    - Time of concentration calculations
    - Curve number (CN) or runoff coefficient (C)
    - Water quality treatment calculations
    - Inlet capacity, bypass, and secondary overflow calculations
    - Calculations for ditches and natural channels
    - Culvert and pipe calculations
    - Energy dissipation calculations
    - Flow spread calculations

10.3.2 Survey

All projects shall include a property boundary survey, unless otherwise approved by the Director. All boundary surveys must comply with the latest version of the Texas Professional Land Surveying Practices Act and General Rules of Procedures and Practices and the City of Dallas Field Note Guidelines.

Horizontal and vertical survey control must be established on-site for all projects. All vertical control (bench marks) must be based on the City of Dallas Bench Mark Network. Bench marks may only be established using Differential Leveling (including Digital Electronic Levels.) The use of Trigonometric Leveling or Global Positioning System observations to establish bench marks will not be accepted. A copy of the Bench Mark Loop run to establish the site bench marks must be submitted to the Survey Division of the Department of Public Works. A Survey Baseline must be established and monumented for all alignment surveys in compliance with City of Dallas standards.

All plan sets must include a Horizontal and Vertical Control drawing, produced in accordance with City of Dallas Standards, signed and sealed by a Texas Registered Professional Land Surveyor, and submitted to the Survey Division for review. The names of all owners, if raw land, and all subdivisions abutting the project must be shown. The datum for all coordinate shown must be given. The basis of bearings must be stated on the drawing.

Rivers, stream, creeks, and outfall ditches will all be surveyed in varying distances as per project scope and engineering requirements. Projects require pre-project and as-built survey based on at least two City of Dallas survey control network monuments. All elevations shall be referenced to the latest vertical datum.

Drainage features are commonly surveyed a reasonable distance upstream and downstream from the end of the drainage structure as necessary to adequately define project tailwater and headwater conditions. The following are some general guidelines for locating and collecting cross section survey data.

10.3.2.1 Outfall Ditches Less Than 3ft Wide

1. Provide two (2) Top of Bank (TS), Creek (CR), or Ditch (DL) along the top outside edge of feature
2. Provide two (2) break lines along the feature as needed to define the shape of the ditch
3. Provide a Drain line (ODL) feature along the deepest section of the feature

Figure 10.1 Outfall Ditch Survey Less Than 3ft

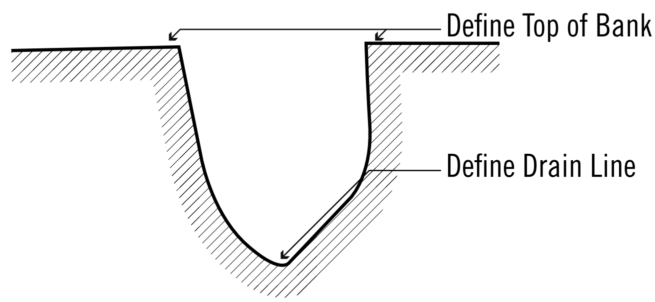
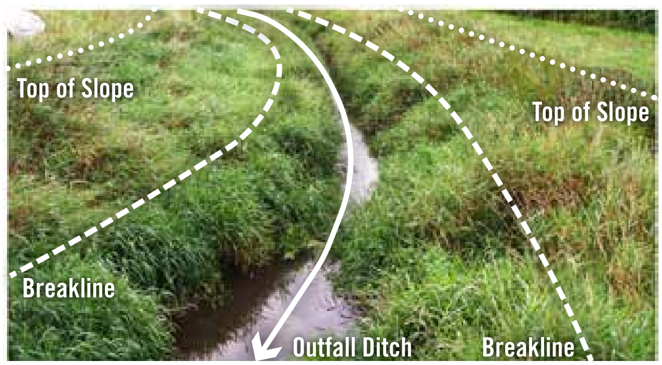


Figure 10.2 Outfall Ditch Survey Example Less Than 3ft



10.3.2.2 Outfall Ditches More Than 3ft Wide

1. Provide two (2) Top of Bank (TS), Creek (CR), or Ditch (DL) along the top outside edge of feature
2. Provide a minimum of two (2) break lines along the feature
3. Provide two (2) Drain Lines (ODL) along the toe or bottom of the feature
4. Provide additional points as necessary to adequately define the shape / conveyance capacity of the ditch

Figure 10.3 Outfall Ditch Survey More Than 3ft

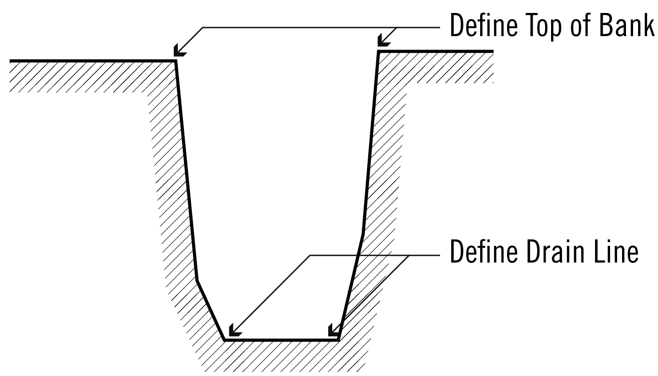


Figure 10.4 Outfall Ditch Survey Example More Than 3ft

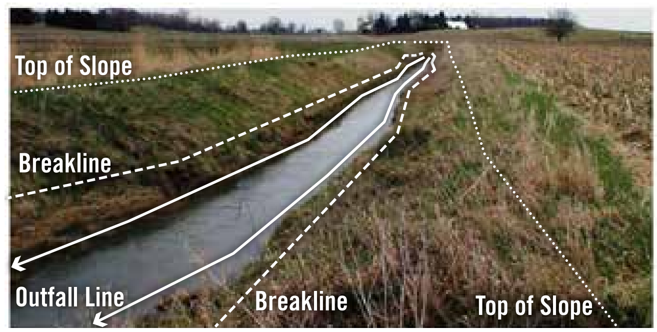
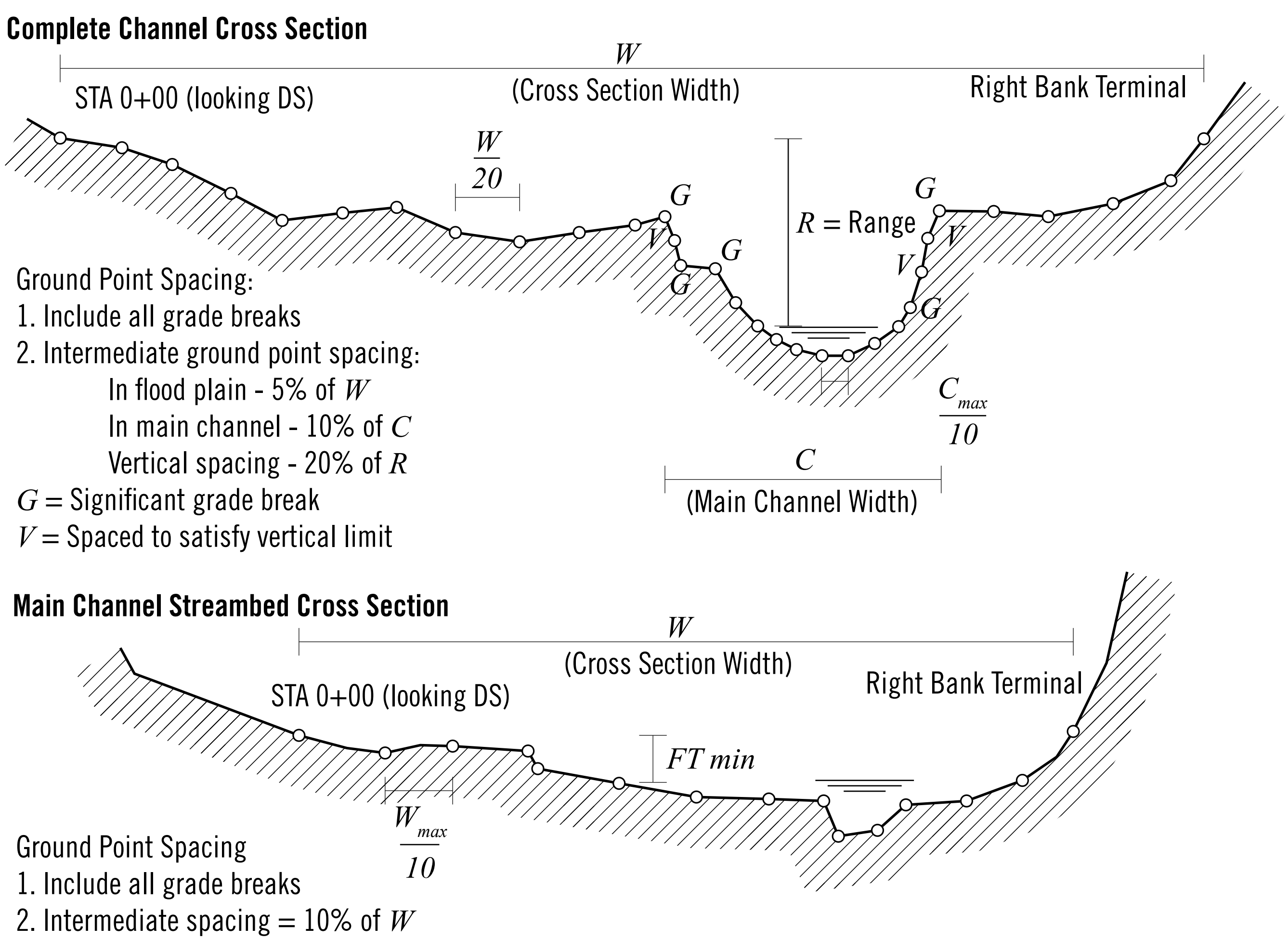


Figure 10.5 Floodplain Location Methods



10.3.2.3 Culverts

Survey culverts by locating the inside face of the inlet and outlet sections. If the culvert has Wingwalls or Headwalls, survey them by locating the top front of beginning and end. If the culvert has a concrete apron, survey it by locating the outer edges. Digital Terrain Model (DTM) shots are needed at the culvert locations, apron locations, and behind the wing walls. The following information will need to be included:

- Culvert locations need to include the culvert inside dimensions
- Multi-Barrel culverts are located by barrel
- Wingwall & Headwall locations need to include wall width

Figure 10.6 Culvert Survey Detail

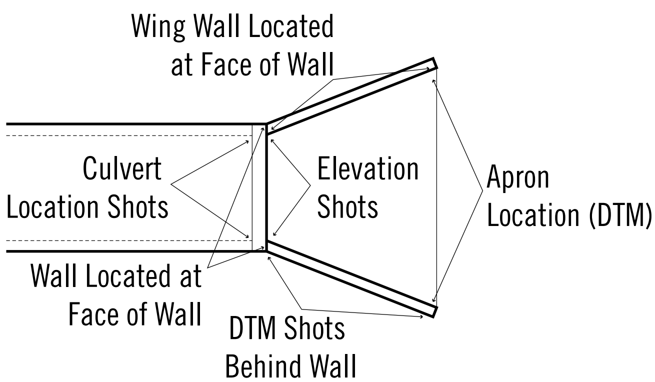


Figure 10.7 Culvert Survey Example

10.3.2.4 Drainage Pipes

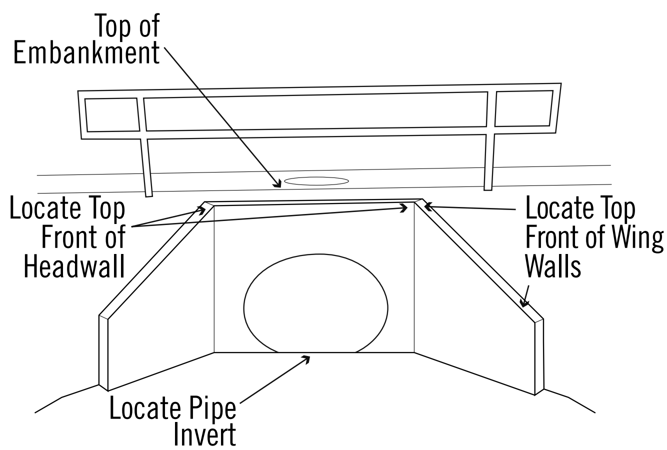
Locate storm sewer pipes with the following information:

- Pipe size
- Pipe Material
- Invert Elevation
- End Treatments (Flared end, Beveled, etc.)
- Special field conditions (crushed end, fully silted, etc)

10.3.2.5 Minor Headwalls and Wingwalls

Locate Minor Structure wingwalls and headwalls in the same fashion as major structures. Walls are generally not included in the DTM file, so be sure to collect sufficient elevation data around the structures.

Figure 10.8 Minor Drainage Wingwall

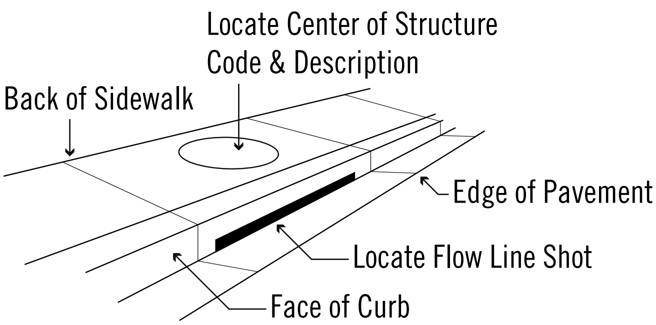


10.3.2.6 Inlet Structures

Curb Inlets are generally located with a minimal amount of shots depicting the following information:

- Center Top of Structure with structure code and description of type or size
- Curb Flow Line Elevation Shot
- Face of curb location and top elevation
- Edge of pavement location/elevation

Figure 10.9 Catch Basin Locations



10.3.2.7 Floodplain Surveys

Some major aspects of these standards are as follows:

- Cross-section data must include full width of the floodplain or include the full width of the channel supplemented by aerial topography
- Cross-sections should indicate the general slope & topography of the plain
- Aerial surveys are often the best way to provide a comprehensive depiction of the floodplain

Figure 10.5 depicts the methodology used by FEMA for their Flood Hazard Mapping Program. Similar level of detail is required for all stream and floodplain cross-sections.

Refer to the FHWA Project Development and Design Manual for the required standards for surveying and mapping flood plains.

10.4

CONSTRUCTION PLAN PREPARATION

10.4.1 General

This section covers the preparation of drainage construction plans for the City of Dallas. The road and drainage plans shall provide enough detail for a third party to construct the proposed facilities per the engineer's design. At a minimum, the plans shall meet the criteria of the City of Dallas standards and specifications.

10.4.2 Conceptual Design Phase

In this phase, major project elements are considered. There may be several viable roadway alignment corridors, interchange types, etc., each with conceptual drainage alternatives. The main purpose of the conceptual design phase is to determine the most viable alternatives that warrant additional consideration.

The conceptual design phase is used to develop the most viable design alternatives and to identify any flaws that result in less desirable alternatives being dropped from consideration. The level of effort should also identify preferred alternatives based on drainage impacts, i.e., impacts such as Federal Emergency Management Agency (FEMA) flood zones, environmentally sensitive areas, etc. Drainage analysis and design is only developed to a level to determine if options are feasible, i.e., this is an order of magnitude evaluation to identify flaws or obvious undesirable drainage impacts. Feasible design alternatives should be discussed with the project team and affected agencies. An order of magnitude cost estimate may be necessary to determine if any options can be eliminated from further consideration.

10.4.2.1 Checklist for Conceptual Design Phase

Conceptual design phase includes, but is not limited to, the following essential elements:

1. Area map showing the section or part of the section in which the site is situated
2. Location and description of all activities that may impact, or be impacted by, the proposed development or redevelopment both on and off the site
3. Acreage of the total site and acreage of the area being affected by the development
4. A conceptual layout of the proposed drainage system for the development or redevelopment

10.4.3 Preliminary Design Phase

The preliminary design phase consists of the development of the project in sufficient detail to allow review for compliance with design criteria. Preliminary submittal for City projects shall conform to requirements given in the engineering services contract. Topographic surveys developed by approved methods should be furnished to allow establishment of alignment, grades and right-of-way requirements for all City projects. Any methods other than field survey shall be field verified. Field notes shall be furnished for all City of Dallas projects.

For Private Development where offsite easements are proposed, field notes shall be signed and sealed by a Registered Professional Land Surveyor and shall begin at an established point such as a street intersection or a corner of an established subdivision. The notes shall be accompanied by a deed of ownership and a location map. All easements pertaining to Private Development should be submitted to the Sustainable Development and Construction Department.

The hydrologic and hydraulic design is to be based on the criteria outlined in Section 2 and 3 of this manual. All calculations shall be submitted with the preliminary plans. These plans shall show the alignment, drainage areas, size of facilities and grades.

The designer shall be responsible for determining the elevation and location of a utility which may be close to a proposed storm drain line and for showing the utility accurately on the plans with station, elevation and source of elevation given in the profile. Utilities suspected to be within 5 feet of proposed facilities shall be field located by probing or exposure. Each utility company shall be contacted with regard to its policies and procedures for uncovering its respective utility.

The proposed alignment plan/profile sheets, drainage area map and horizontal control sheet, required, shall be submitted in sufficient sets as directed by the engineer or engineering services contract. These drawings shall be labeled “Preliminary Plans.” Channel cross sections shall be included.

A design checklist is located on the City of Dallas website.

10.4.4 Pre-Final Design Phase

During this phase, all design decisions are completed. The major goal of this phase is to develop the design to a level adequate to determine all right-of-way needs. This submittal is intended to be 100% complete and will be used for the checking and specifications review. All drainage QA/QC review is to be completed prior to this submittal. The Special provisions and plans will be distributed to provide a complete package for the entire review team.

A design checklist is located on the City of Dallas website.

10.4.5 Final Design Phase

The final design phase consists of preparing construction plans in final form. All sheets shall be drawn on a 22 inch x 34 inch mylar sheet. Refer to City of Dallas Survey Vault for mylar requirements. The drawings shall be executed in such a manner that they shall be legible when reduced to half size. For all computerized drawings, a level list shall be submitted.

Review comments shall be addressed, additional data incorporated, and final design and drafting completed. Grades, elevations, pipe sizes, utility locations and elevations, items and quantities should be checked. Each plan profile sheet should reference two permanent bench marks shown on the plan in their correct location and annotated on the lower right corner of the plan view. Permanent bench marks shall be located outside the limits of construction. Structural detail sheets, quantity sheets, and horizontal control sheets (if required by the engineer) shall be completed and submitted.

Structural analysis computations should be provided in a legible form for any proposed structure not included in the City of Dallas 251D-1 Standard Construction Details. Items on the plans requiring special provisions and special construction techniques should be clearly delineated on the plans and should be specifically called to the City’s attention by letter prior to final plan submission.

A design checklist is located on the City of Dallas website. The design checklist shall be filled out and provided with the submittal.

10.4.6 Drafting Requirements

Refer to the Street Design Manual for drafting standards.

10.4.7 Plan Requirements

10.4.7.1 Drainage Area Map

The scale of the drainage area map should be determined by the method to be used in calculating the runoff as discussed in Section 2. The map shall be drawn at a scale necessary to provide the required information and to be legible. Generally, a map having a scale of 1" = 200' (showing the street right-of-way) is suitable unless dealing with a large drainage area. When calculating runoff, the drainage area map shall show the boundary of the drainage area contributing runoff into the proposed system. This boundary can usually be determined from a map having a contour interval of 1 to 5 feet. The area shall be further divided into subareas to determine flow concentration points or inlet locations as necessary to provide design information and to adequately define hydrologic response within the watershed.

Direction of flow within streets, alleys, natural & man-made drainage ways, and all system intersections shall be clearly shown on the drainage area map. Existing and proposed drainage inlets, storm sewer pipe systems and drainage channels shall be clearly shown and differentiated on the drainage area map. Plan-profile storm sewer or drainage improvement sheet limits shall also be shown.

All off-site drainage within the natural drainage basin shall be shown and delineated. Runoff calculations including inlet calculations shall be a part of the drainage area map.

There shall be no diversion of drainage between watersheds unless approved by the Director.

The following items/information shall be included in the drainage area map:

1. Acres, runoff coefficient and rainfall intensity for each drainage sub-area
2. Inlets, their size and location, the flow-by for each, the direction of flow indicated by flow arrows, the centerline station
3. Chart including data shown shall be submitted with the first review and included on the map with the final review
4. Existing and proposed storm sewers
5. Subareas for alleys, streets, and off-site areas
6. Zoning boundaries and zoning for each area
7. Runoff to all inlets, dead-end streets, and alleys or to adjacent additions and/or lots
8. A table for runoff computations or unit hydrograph input data and peak discharges
9. Flow arrows to indicate all crests, sags, and street and alley intersections
10. Any off-site drainage shall be included
11. Streets and street names shall be indicated
12. All pertinent files; 421Q (Storm Sewer Plats & Profiles),428Q (Storm Sewer Floodway System), and 515D (Bridges) numbers shall be shown on the map (Forms are provided in Appendix A.5), and a single line indication of the location of the pipes and other facilities shall be included
13. Pre & Post Project 1% annual chance Floodplain shall be indicated on the Drainage Area Map
14. Drainage divides shall be field verified
15. Contour intervals shall be as necessary to adequately define drainage basin boundaries and shall be shown on 5' or smaller increment contours

10.4.7.2 Plan / Profile Sheets

The plan portion of the storm drain plan/profile sheet shall show the following:

1. Plan / profile sheets shall be prepared on a horizontal scale of 1 inch equals 20 feet and a vertical scale of 1 inch equals 6 feet. Unusually large conduits may require different scales to adequately show the system. Any variation in scale must be approved in advance by the City Engineer.
2. In the plan view, the storm drain designation, size of pipe, and item number shall be shown adjacent to the storm drain.
3. The drain plan shall be stationed at 100-foot intervals, and each sheet shall begin and end with even stationing.
4. The plan of the storm drain shall be drawn with a centerline and two sides of the conduit with changes in size clearly indicated as they occur.
5. The conduit shall be shaded for emphasis.
6. If the storm drain alignment requires a horizontal curve, the following curve data shall be shown on the plan:
    - PI Station
    - Deflection Angle
    - Radius
    - Tangent Distance
    - Length of Curve

7. At the beginning and ending of the curve, the PC station and PT station shall be shown.
8. The lateral size and item number shall be shown on the plan.
9. Manholes shall be provided and shown on the plan / profile sheet.
10. Existing topography, storm drains, sprinkler heads, double check assemblies, inlets, curbs, driveways, pavement, manholes, meters, valve boxes, trees, shrubs, and fences, etc., within the right-of-way, shall be shown on the plan with existing pavement type and thickness noted.
11. Item numbers shall be shown for all items of work to be accomplished.
12. A summary of quantities sheet is to be provided.
13. Two permanent bench marks shall be referenced in the lower right corner of each plan view sheet. All bench marks shall be checked out by a level circuit submitted to the City.
14. The storm drain profile is to be positioned on the sheet so that the stationing in the plan is approximately adjacent to the stationing in the profile. Even 100-foot stations shall be shown at the bottom of the profile, and elevations at 5-foot intervals shall be shown at the left and right sides of the profile sheet.
15. Stationing for drainage and paving profiles must be oriented in the same direction.
16. Laterals shall be shown in the profile when they cross an existing utility, when they drain a sag or when they exceed 12 feet in length.

The profile portion of the storm drain plan / profile sheet shall show the following:

1. Elevations of rock line (at boring locations)
2. Soffit
3. Invert
4. Hydraulic grade line
5. Top of pipe
6. Existing ground and proposed finished grade at centerline of pipe
7. Elevation of intersecting utilities
8. Diameter of the proposed pipe
9. Pipe grade in percent

Hydraulic data for each length of storm drain between interception points shall be shown on the profile. This data shall consist of the following:

1. Pipe diameter (in)
2. Design discharge (cfs)
3. Slope of hydraulic gradient (ft/ft)
4. Capacity of pipe (cfs) (Assuming the hydraulic gradient equals the pipe grade)
5. Velocity (ft/s)
6. Velocity head ($V^2/2g$) (ft)
7. Limits and velocity of partial flow where applicable

Inlets shall be given the same number designation as the area or sub-area contribution runoff to the inlet. The inlet number designation shall be shown opposite the inlet

Data opposite each inlet shall include paving or storm sewer stationing at centerline of inlet, size of inlet, type of inlet, number or designation, top of curb elevation and flow line of inlet as shown on the typical plans. Lateral profiles shall be drawn showing appropriate information including the hydraulic gradient.

The hydraulic grade adjustment at each interception point should be shown. Partial flow should be shown by labeling starting and ending stations clearly.

Elevations of the flow line of the proposed storm drain are to be shown at 50-foot intervals on the profile. Stationing and flow line elevations are shown at all pipe grade changes, pipe size changes, lateral connections, manholes and wye connections. Pipe wyes connecting to the storm drain shall be made centerline to centerline, shown in the profile with the size of lateral, flow line of wye and stationing of storm sewer indicated.

Boring locations with elevations of top of rock should be included on the drainage plans, as well as all existing and proposed drainage easements, rights-of-way, letters of permission and required temporary easements.

In preparing the final plans, the Engineer shall ensure that inlet elevations and stations are correctly shown on the storm drainage, structural and paving plans as applicable. Inlet locations on storm drain plans shall conform to inlet locations as shown on the drainage area map. Proposed pavement location shall be cross-referenced and agree horizontally and vertically with paving plans, storm drain plans, structural plans and cross-sections, and existing topographic features.

10.4.7.3 Special Details

Details not shown in the Standard Construction Details, File 251D-1 are to be included in the plans as Special Details. Structural details for bridges, special retaining walls, headwalls, junction boxes, culverts, channel lining, and special inlets should be provided as well as bridge and hand railings, special barricades (permanent and temporary), and warning signs. Detour and traffic control plans shall be provided when required by the Director.

SECTION 11

Operation and Maintenance

11.1

OPERATION AND MAINTENANCE CONSIDERATIONS

Drainage features should be maintained to ensure that capacity and water quality objectives are being met. Regular operation and maintenance activities extend the optimal function of stormwater infrastructure and landscape design and treatments. Significant repair and restoration costs can be avoided with regularly scheduled maintenance.

Closed-circuit television (CCTV) inspection is required for newly installed or rehabbed conduits.

Maintenance access should be provided for all drainage elements in accordance with the size of equipment necessary to maintain. Access routes, including roadways and sidewalks, shall be inspected annually and maintained as needed.

Corrective measures should be taken if flowpaths appear that short-circuit the drainage system.

See Appendix A.6 for an Operation & Maintenance Plan Template. Maintenance responsibility should be noted on the plat.

11.2

OPERATION AND MAINTENANCE PLANS

An operation and maintenance plan (O&M Plan) shall be provided for all pump stations and detention facilities that will not be City owned and operated. At a minimum, the O&M Plan should include:

- The name and contact information of the party (or parties) responsible for maintenance and operation of the system, such as a Home Owners association, management company or the legal property owner.
- Property legal description, address, and project name, if applicable.
- Agreement to maintain facilities in accordance with City of Dallas minimum maintenance standards, provided in the Appendix A.6.
- The O&M Plan and a log of maintenance activities that indicates what actions have been taken, when and by whom shall be made available for inspection by City of Dallas upon request.
- Maintenance instructions for any components not covered by the minimum maintenance standards in Appendix A.6.
- Maintenance manual of any proprietary device specified
- The engineer's narrative description of the storm drainage system and how it is intended to function.
- Site diagram of the constructed (as-built) storm drainage system, identifying its components, with profiles as needed.
- As-built details of components.

A copy of the O&M Plan shall be retained onsite or within reasonable access to the site, and shall be transferred with the property to the new owner. The O&M Plan shall be adjusted or revised at the end of the first year of use, if needed, as a result of inspection findings and recommendations by the City. O&M plans shall be filed with the Courthouse with the plat or deed records. If an easement already exists, a separate instrument document must be filed with the City for maintenance and operation. Future land owners are required to adhere to the approved O&M Plan. Annual inspections shall be performed by a qualified engineer to ensure that facilities are operating as intended.

Vegetation management plans are required for all applicable projects. Refer to Development Code Article V, Section 51A-5.208 for vegetation plan requirements within floodplain areas. If infiltration testing is required, testing procedures shall be included in the O&M Plan.

Appendices

A.1 GENERAL

A.1.1 Applicable Regulations

City of Dallas Procedures for Filling in a Flood Plain Under the Flood Plain Management Guidelines

City of Dallas Plat Regulations

Code of Federal Regulations Title 23 Part 650 Subparts A & B

Dallas Development Code Article V Ch. 51

Dallas Development Code Article V Ch. 51A

Dallas Development Code Article X Ch. 51A

Dallas Municipal Separate Storm Sewer System (MS4) Permit

Escarpment Ordinance

North Texas Council of Governments Corridor Development Certificate

Section 408 U.S. Code Title 33 Chapter 9 Subchapter I

Standard Details (251-D)

Texas Administrative Code Title 30, Part 1, Chapter 210 Subchapter F

Texas Administrative Code Title 30, Part 1, Chapter 299

Texas Parks & Wildlife Code 66.015

Texas Parks & Wildlife Code 90.008

Texas Pollutant Discharge Elimination System General Permit Relating to Stormwater Discharges Associated with Construction Activities

Texas Pollutant Discharge Elimination System General Permit TX040000 (MS4 Permit)

Texas Water Code Title 2 Subtitle B Chapter 11 Subchapter A

Texas Water Code §11.142

USACE EC-1162-2-220

USACE EM 1110-2-1913

USACE ETL 1110-2-583

USACE SWFP 1150-2-1

40 CFR Part 122

44 CFR Part 65

A.1.2 List of Acronyms and Abbreviations

ADA – Americans with Disabilities Act

AEP – Annual Exceedance Probability

ASTM – American Society for Testing and Materials

BMP – Best Management Practice

CCTV – Closed-Circuit Television

CDC – Corridor Development Certificate

CLOMR – Conditional Letter of Map Revision

CN – Curve number

CRS – Community Rating System

DART – Dallas Area Rapid Transit

DWU – Dallas Water Utilities

EPA – Environmental Protection Agency

FEMA – Federal Emergency Management Agency

FHWA – Federal Highway Administration

FIRM – Flood Insurance Rate Map

GIS – Geographic Information Systems

HEC-HMS – Hydrologic Engineering Center's Hydrologic Modeling System

HEC-RAS – Hydrologic Engineering Center's River Analysis System

LEED – Leadership in Energy and Environmental Design

LOMA – Letter of Map Amendment

LOMC – Letter of Map Change

LOMR – Letter of Map Revision

LOMR-F – Letter of Map Revision based on Fill

MS4 – Municipal Separate Storm Sewer System

NCTCOG – North Central Texas Council of Governments

NEH – National Engineering Handbook

NOAA – National Oceanic and Atmospheric Administration

NRCS – Natural Resources Conservation Service

NWS – National Weather Service

PICP – Permeable Interlocking Concrete Pavement

PFDS – Precipitation Frequency Data Server

SCS – Soil Conservation Service

SFHA – Special Flood Hazard Area

SWPPP – Stormwater Pollution Prevention Plan

TCEQ – Texas Commission on Environmental Quality

TPDES – Texas Pollutant Discharge Elimination System

TXDOT – Texas Department of Transportation

USACE – United States Army Corps of Engineers

USGS – United States Geological Survey

A.1.3 Glossary of Terms and Definitions

Annual Exceedance Probability / Annual Chance Storm Event – defined as a percentage based on the probability that a certain amount of rainfall will be exceeded any given year.

Backwater – the increase in the upstream water surface level resulting from an obstruction to flow, such as a roadway fill with a bridge or culvert opening placed on floodplain; can also describe the increase in water surface elevations near the confluence of one stream with another, caused by flood conditions on the larger stream.

Best Management Practices – a practice, or combination of practices, that is determined to be an effective and practicable (including technological, economic, and institutional considerations) means of preventing or reducing the amount of pollution generated by non-point sources to a level compatible with water quality goals.

C Value – the runoff coefficient; a dimensionless coefficient relating the amount of runoff to the amount of precipitation received. It is a larger value for areas with low infiltration and high runoff (pavement, steep gradient), and lower for permeable, well vegetated areas (forest, flat land).

Coincident Peak – occurs when both streams reach their peak flow, for a given return period, at the confluence of the streams at or near the same time, affecting the water surface elevation of each stream.

Coincidental Occurrences – refers to the varying amount of time it takes for different size drainage basins to reach peak flow; allows for storm drains to be designed to different design storm frequencies based on area ratios of receiving stream area.

Curve Number – an expression of the imperviousness of the land under fully developed conditions and the runoff potential of the underlying soil.

Easement – the grant of a nonpossessory property interest that grants the easement holder permission to use another person's land.

Escarpment Face – the portion of the escarpment zone between the crest and the toe.

Escarpment Line – the line formed by the intersection of the plane of the stratigraphic contact between the Austin chalk and the Eagle Ford shale formations and the surface of the land.

Escarpment Zone – the corridor of real property south of Interstate Highway 30 between the following described vertical planes:

A) On the crest side of the escarpment line and measuring horizontally from that line, the vertical plane that is 125 feet from that line, or 35 feet beyond the crest, whichever is farther from that line.

B) On the toe side of the escarpment line and measuring horizontally from that line, the vertical plane that is 85 feet from that line, or 10 feet beyond the toe, whichever is farther from that line.

Floodplain – any land area susceptible to inundation by the design flood.

Floodway – the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the design flood without cumulatively increasing the water surface elevation or to discharge more than a designated height or rate.

Forebay – a human-made pool of water in front of a larger body of water; can serve a variety of functions including a buffer zone during storms, trapping sediment and debris, and acting as a natural habitat for aquatic life

Freeboard – a factor of safety expressed in feet above a design water surface

Froude Number – a dimensionless value that describes different flow regimes of open channel flow

Headwater – the depth of water at the upstream end of a culvert or drainage structure, as measured from the upstream invert of the culvert or drainage structure to the water surface elevation

Hydraulic Grade Line – the locus of elevations to which the water would rise if open to atmospheric pressure (e.g. piezometer tubes) along a pipe run.

Hydraulics – the mechanical properties of water and other liquids and the application of these properties in engineering

Hydrograph – a plot of the variation of discharge with respect to time (it can also be the variation of stage or other water property with respect to time).

Hydrology – the study of water, generally focusing on the distribution of water and interaction with the land surface and underlying soils and rocks.

Infiltration – the process by which precipitation or water soaks into subsurface soils.

In-situ – Latin phrase meaning "on site" or "in position".

Lag Time – the time from the center of mass of excess rainfall to the hydrograph peak; when using the SCS Unit Hydrograph method, lag time can be defined as 60% of the time of concentration for the basin.

Lateral – the pipe that connects individual homes, businesses, etc. to the main storm drain line.

Off-line System – systems that only treat runoff up to the design storm event that require a separate bypass structure for storm events greater than the design storm that can be routed around the system.

On-line System – systems that allow storm events that are greater than the design storm to be bypassed through the treatment unit, eliminating the need for a separate bypass structure.

Permeability – the ability of soil or material to transmit water or air; when used in sustainable drainage measures, permeable describes a system that allows movement of water and air around the paving material. Water enters the joints between the solid impervious pavers and flows through the paver system.

Porosity – the percentage of pore volume or void space, or that volume within rock that can contain fluids.

Porous – admitting the passage of gas or liquid through pores; when used in sustainable drainage measures, porous describes pavers that are a cellular grid system filled with dirt, sand, or gravel through which water can infiltrate.

Probable Maximum Precipitation – theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographic location during a certain time of year.

Probable Maximum Flood – the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the drainage basin under study.

Riprap – rock or other material used to armor shorelines, streambeds, bridge abutments, pilings, and other shoreline structures against scour and water or ice erosion.

Roughness coefficient – designated by "n" in Manning's flow equation, the roughness coefficient is an expression of the resistance to flow of a surface such as the bed or bank of a stream.

Runoff – surface runoff is the flow of water that occurs when excess stormwater, meltwater, or other sources flows over the Earth's surface.

Sedimentation – the natural process in which material (such as stones and sand) is carried to the bottom of a body of water and forms a solid layer.

Shear Stress – a measure of the force of friction from a fluid acting on a body in the path of that fluid. In the case of open channel flow, it is the force of moving water against the bed of the channel.

Short-circuiting – when a system is designed with flow path geometry that allows bypass of the treatment system. Examples of short-circuiting in a bioretention area include inlets or curb cuts that are very close to outlet structures or incoming flow that is diverted immediately to the underdrain through stone layers.

Standard Project Flood – the flood expected from the most severe meteorological and hydrologic conditions that are reasonably characteristic to the project area. A standard project flood usually has between 0.3 and 0.08 percent probability of being equaled or exceeded any given year.

Subcritical Flow – deep slow flow with a low energy state and has a Froude number less than one; the flow at which the depth of the channel is greater than critical depth, the velocity of flow is less than critical velocity, and the slope of the channel is also less than the critical slope.

Sumps – drainage features of levee systems that temporarily store storm water runoff before it is conveyed to a river system by pumping over or draining through a levee.

Supercritical Flow – shallow fast flow with a high energy state and has a Froude number greater than one; the flow at which the depth of the channel is less than critical depth, the velocity of flow is greater than critical velocity and the slope of the channel is also greater than the critical slope.

Sustainable Drainage Measures – stormwater infrastructure that uses vegetation, soils, and other elements and practices to restore some of the natural processes required to manage water and create healthier urban environments; sustainable drainage measures include a range of soil-water-plant systems that intercept stormwater, infiltrate a portion of it into the ground, evaporate a portion of it into the air, and release a portion of it slowly back into the sewer system.

Stormwater – water that originates during precipitation events and snow/ice melt. Stormwater can soak into the soil, be held on the surface and evaporate, or runoff and end up in nearby streams, lakes, and rivers.

State water – as defined in Texas Water Code Title 2 Subtitle B Chapter 11 Subchapter A:

A) The water of the ordinary flow, underflow, and tides of every flowing river, natural stream, and lake, and of every bay or arm of the Gulf of Mexico, and the storm water, floodwater, and rainwater of every river, natural stream, canyon, ravine, depression, and watershed in the state is the property of the state.

B) Water imported from any source outside the boundaries of the state for use in the state and which is transported through the beds and banks of any navigable stream within the state or by utilizing any facilities owned or operated by the state is the property of the state.

Tailwater – the depth of water at the downstream end of a culvert or drainage structure, as measured from the downstream invert of the culvert or drainage structure to the water surface elevation.

Time of Concentration – the time required for runoff to travel from the furthest point, hydraulically, within the delineated area (or watershed) to the outlet of the drainage area (or watershed).

Unit Hydrograph – a hydrograph having a volume of 1 inch of runoff which is associated with a precipitation event of specified duration and areal pattern (uniform over the basin); a theoretical hydrograph which is intended to describe how a river at a particular point will react to 1 inch of runoff, and can in turn be used to derive how the river will react to any amount of runoff.

Valley Storage – the water volume between the water surface and ground surface that occupies a given reach of the river.

A.1.4 Sources

Table 2.4 United States Department of Agriculture Natural Resources Conservation Service. "Urban Hydrology for Small Watersheds - Technical Release 55." June 1986.

Figure 2.1 United States Department of Agriculture Natural Resources Conservation Service. "Urban Hydrology for Small Watersheds - Technical Release 55." June 1986.

Figure 2.2 Asquith, W.H., 1999, Areal-reduction factors for the precipitation of the 1-day design storm in Texas: U.S. Geological Survey Water-Resources Investigations Report 99–4267, 81 p.

Table 2.5 North Central Texas Council of Governments. "ISWM Technical Manual - Hydrology."Integrated Stormwater Management.

Table 2.6 North Central Texas Council of Governments. "ISWM Technical Manual - Hydrology."Integrated Stormwater Management.

Figure 3.11 Denison, Texas. "Storm Drainage System Design Manual." Storm Drainage System Design Manual, Apr. 2017.

Figure 3.26 Texas Department of Transportation. "Hydraulic Design Manual." Hydraulic Design Manual, Oct. 2011.

Figure 3.32 Low, Nathan John. "Theoretical Determination of Subcritical Sequent Depths for Complete and Incomplete Hydraulic Jumps in Closed Conduits of Any Shape." BYU ScholarsArchive, 1 Dec. 2008.

Figure 3.33 United States Department of Agriculture Natural Resources Conservation Service. "Urban Hydrology for Small Watersheds - Technical Release 55." June 1986.

Figure 3.34 United States Department of Agriculture Natural Resources Conservation Service. "Urban Hydrology for Small Watersheds - Technical Release 55." June 1986.

Figure 3.35 U.S. Department of Transportation Federal Highway Administration. "Hydraulic Design of Energy Dissipators for Culverts and Channels." Hydraulic Engineering Circular, vol. 13, no. 3, July 2006.

Figure 3.36 U.S. Department of Transportation Federal Highway Administration. "Hydraulic Design of Energy Dissipators for Culverts and Channels." Hydraulic Engineering Circular, vol. 13, no. 3, July 2006.

Figure 3.37 U.S. Department of Transportation Federal Highway Administration. "Hydraulic Design of Energy Dissipators for Culverts and Channels." Hydraulic Engineering Circular, vol. 13, no. 3, July 2006.

Figure 5.2 U.S. Department of Transportation Federal Highway Administration. "Hydraulic Design of Energy Dissipators for Culverts and Channels." Hydraulic Engineering Circular, vol. 13, no. 3, July 2006.

Figure 5.3 U.S. Department of Transportation Federal Highway Administration. "Hydraulic Design of Energy Dissipators for Culverts and Channels." Hydraulic Engineering Circular, vol. 13, no. 3, July 2006.

A.2

HYDROLOGY

A.2.1 C Adjustment Factor Example

Example Site

There is a 3-acre site proposed to hold office buildings, and the developer wishes to capture the first inch of rainfall to be treated by rain gardens. The site is 90% impervious. From Table 2.3, the C value is 0.90.

Determine the design water quality volume using the methods described in Section 2.3.3:

Determine the runoff coefficient.

From Table 2.3, $C = 0.9$

Calculate the water quality capture depth.

$$D_{WQ} = P_{WQ} * C$$

$$D_{WQ} = 1.0\ in*(0.90)$$

$$D_{WQ} = 0.90\ in$$

Calculate the design water quality volume.

$$V_{WQ} = D_{WQ} * A$$

$$V_{WQ} = 0.90\ in*(1\ ft)/(12\ in)*3\ acres*\ (43560\ ft^2)/acre$$

$$V_{WQ} = 9801\ ft^3$$

Size rain gardens to hold the design water quality volume.

Determine the adjusted C value:

$$C_{adj} = C*(1\text{-}1/2\ *P_{WQ})$$

$$C_{adj} = 0.90*(1\text{-}1/2*(1\ in))$$

$$C_{adj} = 0.45$$

Compare pre- and post-project runoff for the 50% annual chance storm event using the adjusted C factor for proposed conditions. If the proposed runoff is higher than existing, additional detention measures are needed to contain the 50% annual chance storm event. Continue with detention analysis methods described in the manual for the 10%, 2%, and 1% annual chance storm events.

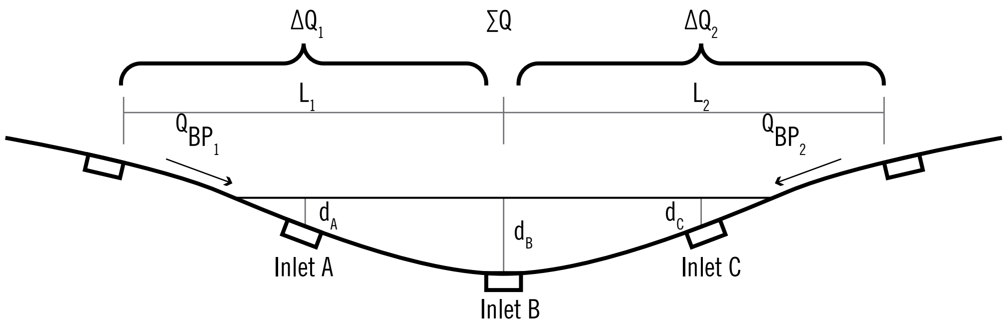
A.3

HYDRAULICS

A.3.1 Sag Inlet Design Worksheet

SAG INLET DESIGN WORKSHEET

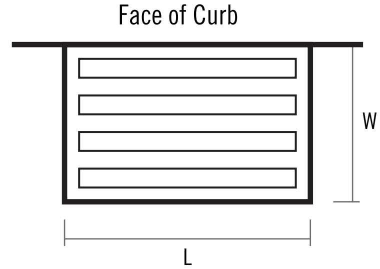
COMBINATION INLET AT LOW POINT



660+10	661+1	660+53	662+05	663+50
Station	Station	Station	Station	Station

Transverse Slope	S_T	0.02	ft/ft	Shoulder Width	6	ft
Allowable	Z_d	12	ft	Lane Width	12	ft
Allowable	d_a	0.24	ft	($d_A = d_C = 0.12$ ft allowable)		
Time of Concentration	T_c	5.0	min			
50 yr. Rainfall	m	8.28				
Coefficients	n	0.587				
Rainfall Intensity	I	3.32	in/hr	(for 5 min duration)		
Distance Between Last Inlet and Low Point	L_1	153	ft	L_2	6	ft
Width of Catchment Area	W_1	24.5	ft	W_2	12	ft
Bypass from Last Inlet	Q_{BP1}[4]	0.28	cfs	Q_{BP2}[4]	0.24	cfs
Discharge of Catchment Area	Q_1	0.26	cfs	Q_2	0.33	cfs

Face of Curb



$Q_{total} = Q_{BP1} + Q_1 + Q_{BP2} + Q_2$

$Q_{total} = 0.28+0.26+0.24+0.33 =$ 1.1 cfs

Combination or Grate Inlet for Sag	P_B(C/G)	C					
Effective Perimeter of Grate Inlets	P_A	0	ft	Width	0	Length	0
(reduced By 50% for Plugging)	P_B	3.87	ft	Width	1.31	Length	1.25
	P_C	0	ft	Width	0	Length	0

See Figure 3.9 in Manual for Grate Dimensions

$\Sigma Q = Q_A + Q_B + Q_C$

[2]$\Sigma Q + C_{WA} P_A (0.5 d_a)^{1.5} + C_{WB} P_B d_b^{1.5} + C_{WC} P_C (0.5 d_c)^{1.5}$ 1.37 cfs

$$d_B = \left(\frac{\Sigma Q}{C_{WA} P_A 0.3536 + C_{WB} P_B + C_{WB} + C_{WC} P_C 0.3536} \right)^{2/3}$$ 0.21 ft

Capacity is Adequate, Design is Complete

Check Calculated d_B against allowable d_C

If $d_B <$ allowable d_B the design is complete

If $d_B >$ allowable d_B, additional inlets must be added[3] and the process repeated

Notes:

[1] If using a combination inlet for the sag, the flank grate inlets are nor required except in a depressed area.

[2] Formulas based on weir flow. See Manual Section 3.3.3.1.

[3] To add more than one outlet in the sag or flanks just increase the width and length values to the sum of all values. Inlets can be different sizes. See Figure 3.9 in Manual for grate dimensions.

[4] Q_{BP1} and Q_{BP2} come from the inlet spreadsheet.

A.3.2 Manning's n for Channels

Type of Channel and Description	Minimum	Normal	Maximum
Natural streams – minor streams (top width at floodstage < 100 ft)			
Main Channels			
A. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
B. Same as above, but more stones and weeds	0.030	0.035	0.040
C. Clean, winding, some pools and shoals	0.033	0.040	0.045
D. Same as above, but some weeds and stones	0.035	0.045	0.050
E. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
F. Same as "d" with more stones	0.045	0.050	0.060
G. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
H. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
bottom: cobbles with large boulders	0.040	0.050	0.070
Floodplains			
A. Pasture, no brush	-	-	-
1. short grass	0.025	0.030	0.035
2. high grass	0.030	0.035	0.050
B. Cultivated areas	-	-	-
1. no crop	0.020	0.030	0.040
2. mature row crops	0.025	0.035	0.045
3. mature field crops	0.030	0.040	0.050
C. Brush	-	-	-
1. scattered brush, heavy weeds	0.035	0.050	0.070
2. light brush and trees, in winter	0.035	0.050	0.060
3. light brush and trees, in summer	0.040	0.060	0.080
4. medium to dense brush, in winter	0.045	0.070	0.110
5. medium to dense brush, in summer	0.070	0.100	0.160
D. Trees	-	-	-
1. dense willows, summer, straight	0.110	0.150	0.200
2. cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120

Type of Channel and Description	Minimum	Normal	Maximum
5. same as D.4 with flood stage reaching branches	0.100	0.120	0.160
Excavated or Dredged Channels			
A. Earth, straight, and uniform	-	-	-
1. clean, recently completed	0.016	0.018	0.020
2. clean, after weathering	0.018	0.022	0.025
3. gravel, uniform section, clean	0.022	0.025	0.030
4. with short grass, few weeds	0.022	0.027	0.033
B. Earth winding and sluggish	-	-	-
1. no vegetation	0.023	0.025	0.030
2. grass, some weeds	0.025	0.030	0.033
3. dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. earth bottom and rubble sides	0.028	0.030	0.035
5. stony bottom and weedy banks	0.025	0.035	0.040
6. cobble bottom and clean sides	0.030	0.040	0.050
C. Dragline-excavated or dredged	-	-	-
1. no vegetation	0.025	0.028	0.033
2. light brush on banks	0.035	0.050	0.060
D. Rock cuts	-	-	-
1. smooth and uniform	0.025	0.035	0.040
2. jagged and irregular	0.035	0.040	0.050
E. Channels not maintained, weeds and brush uncut	-	-	-
1. dense weeds, high as flow depth	0.050	0.080	0.120
2. clean bottom, brush on sides	0.040	0.050	0.0800
3. same as above, highest stage of flow	0.045	0.070	0.110
4. dense brush, high stage	0.080	0.100	0.140
Lined or Constructed Channels	-	-	-
A. Cement	-	-	-
1. neat surface	0.010	0.011	0.013
2. mortar	0.011	0.013	0.015
B. Wood	-	-	-
1. planed, untreated	0.010	0.012	0.014
2. planed, creosoted	0.011	0.012	0.015
3. unplaned	0.011	0.013	0.015
4. plank with battens	0.012	0.015	0.018
5. lined with roofing paper	0.010	0.014	0.017

Type of Channel and Description	Minimum	Normal	Maximum
C. Concrete	-	-	-
1. trowel finish	0.011	0.013	0.015
2. float finish	0.013	0.015	0.016
3. finished, with gravel on bottom	0.015	0.017	0.020
4. unfinished	0.014	0.07	0.020
5. gunite, good section	0.016	0.019	0.023
6. gunite, wavy section	0.018	0.022	0.025
7. on good excavated rock	0.017	0.020	-
8. on irregular excavated rock	0.022	0.027	-
D. Concrete bottom float finish with sides of:	-	-	-
1. dressed stone in mortar	0.015	0.017	0.020
2. random stone in mortar	0.017	0.020	0.024
3. cement rubble masonry, plastered	0.016	0.020	0.024
4. cement rubble masonry	0.020	0.025	0.030
5. dry rubble or riprap	0.020	0.030	0.035
E. Gravel bottom with sides of:	-	-	-
1. formed concrete	0.017	0.020	0.025
2. random stone mortar	0.020	0.023	0.026
3. dry rubble or riprap	0.023	0.033	0.036
F. Brick	-	-	-
1. glazed	0.011	0.013	0.015
2. in cement mortar	0.012	0.015	0.018
G. Masonry	-	-	-
1. cemented rubble	0.017	0.025	0.030
2. dry rubble	0.023	0.032	0.035
H. Dressed ashlar/stone paving	-	-	-
I. Asphalt	-	-	-
1. smooth	0.013	0.013	-
2. rough	0.016	0.016	-
J. Vegetal lining	0.030	-	0.500

A.3.3 Manning's n for Closed Conduits Flowing Partly Full

Type of Conduit and Description	Minimum	Normal	Maximum
A. Brass, smooth	0.009	0.010	0.013
B. Steel	-	-	-
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
C. Cast Iron	-	-	-
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
E. Wrought Iron	-	-	-
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
F. Corrugated Metal	-	-	-
1. Subdrain	0.017	0.019	0.021
2. Stormdrain	0.021	0.024	0.030
G. Cement	-	-	-
1. Neat Surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
H. Concrete	-	-	-
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
I. Wood	-	-	-
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
J. Clay	-	-	-
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified Subdrain with open joint	0.014	0.016	0.018

Type of Conduit and Description	Minimum	Normal	Maximum
K. Brickwork	-	-	-
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
3. Sanitary sewers coated with sewage slime with bends and connections	0.012	0.013	0.016
4. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
5. Rubble masonry, cemented	0.018	0.025	0.030

A.3.4 Gutter Flow / Inlet Computations

GUTTER FLOW / INLET COMPUTATIONS

INLET		D.A.	Q	CO	Qa	z	n	s	y	w	a	qL	Lr	La	La/Lr	a/y	QI/QA	QI	QI-Qa	Carry	Remarks
ID	LOCATION	No.	cfs	cfs	cfs			ft/ft	ft	ft	ft	cfs/ft	ft	ft				cfs	cfs	over to:	

Q — Discharge calculated for DA
CO — Carryover from upstream inlet
Qa — Actual discharge= Q+CO
z — Reciprocal of cross slope (can use average slope equivalent for parabolic crown)
n — Manning's roughness (0.0175 normal)
s — Gutter slope
y — Gutter depth of flow= $\left(\frac{Qa}{0.56\,(z/n)\,s^{1/2}}\right)^{3/8}$

ql — Capture per foot of inlet with 100% interception= $\left(\frac{0.7}{y}\right)\left[(a+y)^{5/2} - (a)^{5/2}\right]$
Lr — Required inlet for 100% interception= $\frac{Qa}{q_L}$
La — Actual inlet length
QI/QA — From figure (10)
QI — Actual Inlet Interception

A.3.5 Full Flow Coefficient Values

A.3.5.1 Elliptical Concrete Pipe

Pipe Size R =S (HE) S = R (VE) (Inches)	Approximate Equivalent Circular Diameters (Inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = 1.486/n \times A \times R^{2/3}$			
				n = 0.010	n = 0.011	n = 0.012	n = 0.013
14 x 23	18	1.8	0.367	138	125	116	108
19 x 30	24	3.3	0.490	301	274	252	232
22 x 34	27	4.1	0.546	405	368	339	313
24 x 38	30	5.1	0.613	547	497	456	421
27 x 42	33	6.3	0.686	728	662	607	560
29 x 45	36	7.4	0.736	891	810	746	686
32 x 49	39	8.8	0.812	1,140	1,036	948	875
34 x 53	42	10.2	0.875	1,386	1,260	1,156	1,067
38 x 60	48	12.9	0.969	1,878	1,707	1,565	1,445
43 x 68	54	16.6	1.106	2,635	2,395	2,196	2,027
48 x 76	60	20.5	1.229	3,491	3,174	2,910	2,686
53 x 83	66	24.8	1.352	4,503	4,094	3,753	3,464
58 x 91	72	29.5	1.475	5,680	5,164	4,734	4,370
63 x 98	78	34.6	1.598	7,027	6,388	5,856	5,406
68 x 106	84	40.1	1.721	8,560	7,790	7,140	6,590
72 x 113	90	46.1	1.845	10,300	9,365	8,584	7,925
77 x 121	96	52.4	1.967	12,220	11,110	10,190	9,403
82 x 128	102	59.2	2.091	14,380	13,070	11,980	11,060
87 x 136	108	66.4	2.215	16,770	15,240	13,970	12,900
92 x 143	114	74.0	2.340	19,380	17,620	16,150	14,910
97 x 151	120	82.0	2.461	22,190	20,180	18,490	17,070
106 x 166	132	99.2	2.707	28,630	26,020	23,860	22,020
116 x 180	144	118.6	2.968	36,400	33,100	30,340	28,000

A.3.5.2 Concrete Arch Pipe

Pipe Size R x S (Inches)	**Approximate Equivalent Circular Diameters (Inches)**	**A Area (Square Feet)**	**R Hydraulic Radius (Feet)**	**Value of $C_1 = 1.486/n \times A \times R^{2/3}$**			
				n = 0.010	**n = 0.011**	**n = 0.012**	**n = 0.013**
11 x 18	15	1.1	0.25	65	59	54	50
$13^{1/2}$ x 22	18	1.6	0.30	110	100	91	84
151 x 26	21	2.2	0.36	165	150	137	127
18 x $28^{1/2}$	24	2.8	0.45	243	221	203	187
$22^{1/2}$ x $36^{1/4}$	30	4.4	0.56	441	401	368	339
$26^{5/8}$ x $43^{3/4}$	36	6.4	0.68	736	669	613	566
$31^{5/18}$ x $51^{1/8}$	42	8.8	0.80	1,125	1,023	938	866
36 x $58^{1/2}$	48	11.4	0.90	1,579	1,435	1,315	1,214
40 x 65	54	14.3	1.01	2,140	1,945	1,783	1,646
45 x 73	60	17.7	1.13	2,851	2,592	2,376	2,193
54 x 88	72	25.6	1.35	4,641	4,219	3,867	3,569
63 x102	84	34.6	1.57	6,941	6,310	5,784	5,339
72 x 115	90	44.5	1.77	9,668	8,789	8,056	7,436
$77^{1/4}$ x 122	96	51.7	1.92	11,850	10,770	9,872	91,122
$87^{1/8}$ x 138	108	66.0	2.17	16,430	14,940	13,690	12,640
$96^{7/8}$ x 154	120	81.8	2.42	21,975	19,977	18,312	16,904
$106^{1/2}$ x $168^{3/4}$	132	99.1	2.65	28,292	25,720	23,577	21,763

A.3.5.3 Precast Concrete Box Section

Box Size Span x Rise (Feet)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = 1.486/n \times A \times R^{2/3}$		Box Size Span x Rise (Feet)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = 1.486/n \times A \times R^{2/3}$	
			n = 0.012	n = 0.013				n = 0.012	n = 0.013
3 x 2	5.78	0.63	524	484	9 x 5	43.88	1.67	7,060	7,070
3 x 3	8.78	0.78	923	852	9 x 6	52.88	1.87	9,950	9,180
4 x 2	7.65	0.69	743	686	9 x 7	61.88	2.05	12,400	11,400
4 x 3	11.65	0.90	1,340	1,240	9 x 8	70.88	2.20	14,800	13,700
4 x 4	15.65	1.04	1,990	1,840	9 x 9	79.88	2.33	17,400	16,100
5 x 3	14.50	0.98	1,770	1,630	10 x 5	48.61	1.73	8,690	8,020
5 x 4	19.50	1.16	2,660	2,460	10 x 6	58.61	1.95	11,300	10,462
5 x 5	24.50	1.30	3,620	3,340	10 x 7	68.61	2.14	14,100	13,000
6 x 3	17.32	1.04	2,200	2,030	10 x 8	78.61	2.31	17,000	15,700
6 x 4	23.32	1.25	3,350	3,100	10 x 9	88.61	2.46	20,000	18,500
6 x 5	29.32	1.42	4,590	4,240	10 x 10	98.61	2.59	23,000	21,300
6 x 6	35.32	1.56	5,880	5,430	11 x 4	42.32	1.52	6,930	6,390
7 x 4	27.11	1.33	4,050	3,740	11 x 6	64.32	2.02	12,730	11,700
7 x 5	34.11	1.52	5,590	5,160	11 x 8	86.32	2.41	19,200	17,700
7 x 6	41.11	1.68	7,200	6,650	11 x 10	108.32	2.72	26,100	24,100
7 x 7	48.11	1.82	8,880	8,200	11 x 11	119.32	2.85	29,700	27,400
8 x 4	31.11	1.39	4,790	4,420	12 x 4	46.00	1.55	7,630	7,050
8 x 5	39.11	1.60	6,630	6,120	12 x 6	70.00	2.08	14,100	13,000
8 x 6	47.11	1.78	8,760	7,920	12 x 8	94.00	2.50	21,400	19,800
8 x 7	55.11	1.94	10,600	9,790	12 x 10	118.00	2.83	29,300	27,000
8 x 8	63.11	2.07	12,700	11,700	12 x 12	142.00	3.11	37,500	34,600

A.3.5.4 Circular Concrete Pipe

D Pipe Diameter (Inches)	A Area (Square Feet)	R Hydraulic Radius (Feet)	Value of $C_1 = 1.486/n \times A \times R^{2/3}$			
			n = 0.010	n = 0.011	n = 0.012	n = 0.013
8	0.349	0.167	15.8	14.3	13.1	12.1
10	0.545	0.208	28.4	25.8	23.6	21.8
12	0.785	0.205	46.4	42.1	38.6	35.7
15	1.227	0.312	84.1	76.5	70.1	64.7
18	1.767	0.375	137	124	114	105
21	2.405	0.437	206	187	172	158
24	3.142	0.500	294	267	245	226
27	3.976	0.562	402	366	335	310
30	4.909	0.625	533	485	444	410
33	5.940	0.688	686	624	574	530
36	7.069	0.750	867	788	722	666
42	9.621	0.875	1,308	1,189	1,090	1,006
48	12.566	1.000	1,867	1,698	1,556	1,436
54	15.904	1.125	2,557	2,325	2,131	1,967
60	19.635	1.250	3,385	3,077	2,821	2,604
66	23.758	1.375	4,364	3,967	3,636	3,357
72	28.274	1.500	5,504	5,004	4,587	4,234
78	33.183	1.625	6,815	6,195	5,679	5,242
84	38.485	1.750	8,304	7,549	6,920	6,388
90	44.170	1.875	9,985	9,078	8,321	7,684
96	50.266	2.000	11,850	10,780	9,878	9,119
102	56.745	2.125	19,340	12,670	11,620	10,720
108	63.617	2.250	16,230	14,760	13,530	12,490
114	70.882	2.375	18,750	17,040	15,620	14,420
120	78.540	2.500	21,500	19,540	17,920	16,540
126	86.590	2.625	24,480	22,260	20,400	18,830
132	95.033	2.750	27,720	25,200	23,100	21,330
138	103.870	2.875	31,210	28,370	26,010	24,010
144	113.100	3.000	34,960	31,780	29,130	26,890

A.3.6 Energy Equations in Open Channels

A.3.6.1 Energy

Conservation of energy is a basic principal in open channel flow. As shown in Figure A.3.1, the total energy at a given location in an open channel is expressed as the sum of the potential energy head (elevation), pressure head, and kinetic energy head (velocity head). The total energy at given channel cross section can be represented as:

$$E_t = Z + y + (V^2/2g) \quad \textit{(Equation A.3.1)}$$

E_t = total energy (ft)

Z = elevation above given datum (ft)

y = flow depth (ft)

V = mean velocity (ft)

g = gravitational acceleration = 32.2 ft/s^2

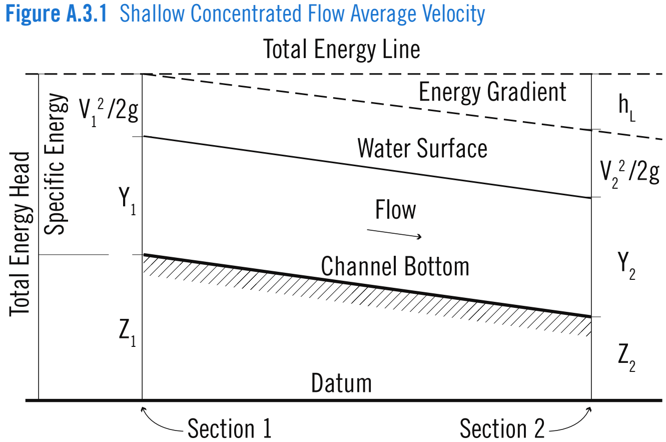
Written between an upstream cross section designated 1 and a downstream cross section designated 2, the energy equation becomes the following:

$$Z_1 + y_1 + (V_1^2)/2g = Z_2 + y_2 + (V_2^2)/2g + h_L \quad \textit{(Equation A.3.2)}$$

h_L = head or energy loss between Section 1 and 2 (ft)

The terms in the energy equation are illustrated in Figure A.3.1. The energy equation states that the total energy head at an upstream cross section is equal to the total energy head at a downstream section plus the energy head loss between the two sections.

Figure A.3.1 Shallow Concentrated Flow Average Velocity



A.3.6.2 Specific Energy

The specific energy of flow in a channel section is defined as the energy per pound of water measured with respect to the channel bottom. Specific energy, E (expressed as head in feet), is given by the following:

$$E = y + V^2/2g = y + \left(Q^2/(2gA^2)\right) \quad \textit{(Equation A.3.3)}$$

y = depth (ft)

V = mean velocity (ft)

g = gravitational acceleration = 32.2 ft/s^2

Q = discharge (cfs)

A = area of channel cross section (ft^2)

A.3.6.3 Froude Number

The influence of gravity on fluid motion in open channel flow can be expressed in a dimensionless quantity called a Froude Number (Fr). The Froude Number is expressed in the following equation.

$$Fr = \frac{V}{\sqrt{gd}} \quad \textit{(Equation A.3.4)}$$

V = mean velocity (ft/s)

g = acceleration of gravity = 32.2 ft/s^2

d = hydraulic depth (ft)

The hydraulic depth is defined as the cross sectional area of the channel perpendicular to the flow divided by the free water surface topwidth.

A.4

PROCEDURES

All complaints must be entered into the Customer Service Request (CSR) / 311 system and they will be addressed in accordance with the department service level agreements.

A.5

SUBMITTAL REQUIREMENTS AND FORMS

A.5.1 Checklist for Storm Drainage Plans

Drainage Area Map

1. Use 1”=200’ scale for onsite and 1”=400’ for creeks off site and show match lines between any two or more maps.
2. Show existing and proposed storm drains and inlets.
3. Indicate subareas for each alley, street, and offsite area.
4. Indicate contours on map for on and offsite.
5. Use design criteria as shown in design manual.
6. Indicate zoning on drainage area.
7. Show points of concentration.
8. Indicate runoff at all inlets, dead-end streets and alleys, or to adjacent additions or acreage.
9. Provide runoff calculations for all areas showing acreage, runoff coefficient, inlet time.
10. For cumulative runoff, show calculations.
11. Indicate all crests, sags, and street and alley intersections with flow arrows.
12. Identify direction of north.
13. Show limits of 1% annual chance floodplain on drainage area map.

Plan, Profile, and Detail Sheets

1. Show plan and profile of all storm drains.
2. Specify Class III pipe unless otherwise noted in profile.
3. Provide inlets where street capacity is exceeded. Provide inlets where alley runoff exceeds intersecting street capacity.
4. Indicate property lines along storm drainage and show easements with dimensions.
5. Show all existing utilities in plan and profile of storm drains with elevations.
6. Indicate existing and proposed ground line and improvements on all street, alley, and storm drain profiles.
7. Show all hydraulics, velocity head changes, gradients, computations, and profile outfall with typical section and computations.
8. Show laterals on trunk profile with stations.
9. Indicate size of inlet on plan view, lateral size and flow line, paving station and top-of-curb elevation.
10. Indicate runoff concentrating at all inlets and direction of flow. Show runoff for all stub outs, pipes, and intakes.
11. Show future streets and grades where applicable.
12. Do not use 90-degree turns on storm drains or outfall. Provide good alignment with junction structures or manholes (for small systems).
13. Discharge storm drains at the flow line of creeks and channels unless competent rock is present.
14. Show water surface at outfall of storm drain.
15. Where fill is proposed for trench cut in creeks or outfall ditches, specify compacted fill.
16. Use Y-inlets in ditches.
17. Where connections are made to existing storm drain, show computations of existing system when available.
18. Show pipe sizes in plan and profile.
19. Provide separate plan and profile for both storm drain and paving plans. The storm drain pipes should also be shown on paving plans with a dashed line.
20. Use heavier than Class III pipes where crossing railroads, deep fill, and heavy loads.
21. Show details of all junction boxes, headwalls on storm drain, flumes, and manholes when more than one pipe intersects the manhole or any other item not a standard detail.
22. All Y-inlets and inlets 10 ft or greater have a min 21" lateral and all smaller inlets have a min 18" lateral.
23. Provide headwalls for all storm drains at outfall.
24. Check the need for curbing at all alley turns and "T" intersections. Flatten grades ahead of turns and intersections.
25. Calculate hydraulic grade line for laterals and inlets to ensure collection of storm water. For inlets, provide HGL on profile for all profiled laterals with hydraulic data. Laterals longer than 80 feet require special analysis.
26. Where inlets are placed in alley, provide curbing for 10 feet on each side of inlet and on other side of alley, where the top of inlet elevation is even with the high edge of alley pavement. The width between curbs shall be equal to or greater than 10 feet.
27. Use standard curb inlets in streets and alleys. Use recessed inlets in divided streets. Do not use grate or curb and grate inlet unless other solution is not available.
28. Provide 7 ½-inch curb on alleys parallel to creek or channel on creek side of alley.
29. Indicate flow line elevations of storm drains on profile, show percent grade. Match top inside of pipe where adjacent to other size pipe.
30. Where laterals tie into trunk line, channel or creek, place at 60 degree angle with centerlines. Connect them so that the longitudinal centers intersect.
31. Show curve data for all storm drains.
32. Tie storm drain stationing with paving stations.
33. Do not flow storm water from streets into alleys.
34. On all dead-end streets and alleys, show grade out for drainage on the profiles and provide erosion control.
35. Specify concrete strength for all structures. The minimum allowable is 3600 psi
36. Where quantities of runoff are shown on plan or profile, indicate storm frequency design.
37. Provide sections for road, railroad, and other ditches with profiles and hydraulic computations. Show design water surface on profile.

38. Investigation shall be made by the engineer to validate the adequacy of the storm drain outfall.

39. Do not use high velocities in storm drain design. Maximum discharge velocity should not exceed 6 ft/s at outfall.

40. Flumes may not be allowed unless specifically designated.

41. If time of concentration is different, provide the calculation for inlet time and pipe travel time.

42. Provide lateral profiles for laterals exceeding 12 ft in length.

43. Proposed driveway turnouts must be 10 feet from any existing or proposed inlet.

44. Do not use bends for pipe sizes less than 30-inch diameter unless specifically authorized by the Director.

Statements

1. Any offsite drainage work or discharge to downstream property will require a letter of permission or easements. Submit field notes for offsite easement that may be required.

2. Provide written statement signed by a Professional Engineer acknowledging that he/she has analyzed the proposed storm drainage outfall effects on the adjoining property, and that the discharge will not adversely affect or jeopardize this property. Provide letter of acceptance from adjoining property owner, if post-development discharges exceed pre-development rates (Development Only).

3. Check for escarpment area restrictions. If in Geologically Similar Areas, design in accordance with escarpment ordinance.

Bridges

1. Clear the lowest member of the bridges by 2 feet above the design water surface unless otherwise directed by the City.

2. Show geotechnical soil boring information on plans.

3. Show bridge sections upstream and downstream.

4. Provide hydraulic calculations on all sections.

5. Provide structural details and calculations with dead load deflection diagram.

6. Provide vertical and horizontal alignment.

7. Provide drainage area map and show all computations for runoff affecting a detention basin.

8. Provide a plot plan with existing and proposed contours for a detention basin and plan for structural measures.

9. Where earth embankment is proposed for impoundment, furnish a typical embankment section and specifications for fill; include profile for the structural outfall structure and geotechnical report.

10. Provide structural details and calculations for any item and geotechnical report not a standard detail.

11. Provide detention basin volume calculations and elevations versus storage curve.

12. Provide hydraulic calculations for outflow structure and elevation versus discharge curve for a detention basin.

13. Provide unit hydrograph routings, or modified rational, where permitted for 100 acres or less for a detention basin.

A.5.2 Non-Standard Material Agreement Template

AGREEMENT

CITY OF DALLAS NON-STANDARD MATERIALS FOR CONSTRUCTION
WITHIN PUBLIC RIGHT-OF-WAY

PROJECT:____

LOCATION:____

DESCRIPTION:____

MATERIAL:____

The work described above is proposed for construction in the public right-of-way. The plans have been reviewed by the Building Inspection Department and the Proposed Improvements are approved subject to the following conditions:

1 All construction must conform to the requirements of the Public Works Department standard construction details and the standard specifications for Public Works construction.

2 The owner accepts full responsibility for all maintenance and the replacement of the sidewalk surface, setting bed, and concrete base including those instances when the city or utility company, in the discharge of its responsibilities, removes any portion of these improvements. All improvements located in the public right-of-way are there at the pleasure of the city and must be maintained to the satisfaction of the city.

3 All sidewalks must be constructed barrier-free to the handicapped.

4 Traffic signs and signals must not be altered unless approved by the Department of Transportation.

5 Obtain driveway/sidewalk permits from Building Inspection at 320 E. Jefferson Blvd., Room 118, 948-4480, 948-4173, 948-4174.

The owner's attention is directed to Sections 43-33 and 43-44 of the city code which specifies the property owner's responsibility and liability for all claims or damages resulting from injury of loss due to the abutting sidewalk or curbs. Approval of plans by the Building Inspection Department does not change or assume any of the property owner's responsibility.

The agreement is binding upon the owner, heirs, successors or his assigns. The owner will execute this agreement in duplicate and return both copies to the Building Inspection Department for approval. One signed copy will be forwarded to the owner.

Execution of this agreement by the owner signifies acceptance of all provisions and willingness to comply.

OWNER'S NAME (PRINT)____

MAILING ADDRESS____

CITY, STATE ____

____

Street address (site)   Lot   Block

SIGNED: ______________________________ DATE:______________

(Property Owner)   (Title)

PRINTED NAME:______________________________

SUBSCRIBED AND SWORN TO, BEFORE ME, THIS ___ DAY OF _______ 20____.

On this day personally appeared______________________,

known to me to be the person whose name is subscribed to the foregoing instrument and acknowledged to me that he executed the same for the purposes and consideration herein expressed.

SEAL   ______________________________

NOTARY PUBLIC

APPROVED BY THE CITY OF DALLAS:

______________________________DATE:______________

BUILDING OFFICIAL

PRINTED NAME: ________________________________

SUBSCRIBED AND SWORN TO, BEFORE ME, THIS ___ DAY OF _______ 20____.

On this day personally appeared _____________________,

known to me to be the person whose name is subscribed to the foregoing instrument and acknowledged to me that he executed the same for the purposes and consideration herein expressed.

SEAL ____

NOTARY PUBLIC

A.6
OTHER

A.6.1 Materials Matrix

The following are permissible, not exclusive.

Application	Soil Preparation or Media	Recommended Materials*	Maintenance	Installation	Additional Notes
Open Channels	Topsoil with minimum 5% organic content: bladed and ripped for interface into subbase soils	Seed Mix #1 - Bottom & Sides and Seed Mix #2 - Bottom Nurse Crop Seed for winter - temporary (if seed Mixes #1 & 2 installation windows are missed)	Mow two times per year: July 1st and in dead of Winter (January 1st) to no shorter than 8 inches in height. Bag and remove from site all clippings and trimmings. See also: Maintenance List #1	Seed applied uniformly with: Cyclone, Drill, Cultipacker Seeder, Hydroseed (slurry with seed, fertilizer, & binder), or hydraulic mulch with seed. Seed Mixes: Spring (March 1st - May 15th until it gets hot and rain stops) or Fall (September 1st - November 15th)	No unvegetated area shall exceed 10 square feet in order to limit erosion.
Open Channels with Regular Drainage or Seepage	Topsoil with minimum 5% organic content: bladed and ripped for interface into subbase soils	Seed Mix #1 - Bottom & Sides and winter overseed of Seed Mix #4 - Bottom	Mow two times per year: July 1st and in dead of Winter (January 1st) to no shorter than 8 inches in height. Bag and remove from site all clippings and trimmings. See also: Maintenance List #1	Seed applied uniformly with: Cyclone, Drill, Cultipacker Seeder, Hydroseed (slurry with seed, fertilizer, & binder), or hydraulic mulch with seed. Seed Mixes: Spring (March 1st - May 15th until it gets hot and rain stops); overseed: Winter (November 1st - February)	No unvegetated area shall exceed 10 square feet in order to limit erosion.
Detention Pond	Topsoil with minimum 5% organic content: bladed and ripped for interface into subbase soils	Seed Mix #1 - Bottom & Sides, Seed Mix #2 - Bottom, Seed Mix #3 - Upper Slopes and Dry Side of Bank, Nurse Crop Seed for winter - temporary (if seed Mixes #1, 2 & 3 installation windows are missed)	Mow two times per year: July 1st and in dead of Winter (January 1st) to no shorter than 8 inches in height. Bag and remove from site all clippings and trimmings. See also: Maintenance List #1	Seed applied uniformly with: Cyclone, Drill, Cultipacker Seeder, Hydroseed (slurry with seed, fertilizer, & binder), or hydraulic mulch with seed. Seed Mixes: Spring (March 1st - May 15th until it gets hot and rain stops) or Fall (September 1st - November 15th)	No unvegetated area shall exceed 10 square feet in order to limit erosion.

* Seed mixes and plant lists can be found on following pages

Application	Soil Preparation or Media	Recommended Materials*	Maintenance	Installation	Additional Notes
Service Access/ Maintenance Ramps or Pedestrian Access Slopes	Topsoil with minimum 5% organic content: ripped or bladed for interface into subbase soils	Seed Mix #5 Nurse Crop Seed for winter - temporary (if seed Mix #5 installation window is missed)	Mow two times per year: July 1st and in dead of Winter (January 1st) to no shorter than 3 inches in height. See also: Maintenance List #1	Seed applied uniformly with: Cyclone, Drill, Cultipacker Seeder, Hydroseed (slurry with seed, fertilizer, & binder), or hydraulic mulch with seed. Seed Mixes: Spring (March 1st - May 15th) or Fall (Sept. 1st - Nov. 15) if regular rain and up until 8 weeks before average frost date	No unvegetated area shall exceed 10 square feet in order to limit erosion.
Rain Gardens	Rain Garden Planting Soil/ Amended Soils or Bioretention Soil Mix #1	Plant List #2	Maintenance List #1		No unvegetated area shall exceed 9 square feet in order to limit erosion. Use of fertilizers, fungicides, herbicides and pesticides are prohibited without approval from the Director. Organic alternatives may be used. Hand pull weeds early in growth stage, before they set seed - critical in first 3 years until plantings establish. Maintain 3 inch mulch topdressing with rough/coarse cut mulch.
Bioretention	Bioretention Soil Mix #1	Plant List #2	Maintenance List #1		No unvegetated area shall exceed 9 square feet in order to limit erosion. Use of fertilizers, fungicides, herbicides and pesticides are prohibited without approval from the Director. Organic alternatives may be used. Hand pull weeds early in growth stage, before they set seed - critical in first 3 years until plantings establish. Maintain 3 inch mulch topdressing with rough/coarse cut mulch.
Tree-box filters	Bioretention Soil Mix #1	Plant List #2	Maintenance List #1		-

* Seed mixes and plant lists can be found on following pages

Application	Soil Preparation or Media	Recommended Materials*	Maintenance	Installation	Additional Notes
Bioswales	Topsoil with minimum 5% organic content: bladed and ripped for interface into subbase soils	Seed Mix #1 - Bottom & Sides and Seed Mix #2 - Bottom Nurse Crop Seed for winter - temporary (if seed Mixes #1 & 2 installation windows are missed)	Maintenance List #1	Seed applied uniformly with: Cyclone, Drill, Cultipacker Seeder, Hydroseed (slurry with seed, fertilizer, & binder), or hydraulic mulch with seed. Seed Mixes: Spring (March 1st - May 15th until it gets hot and rain stops) or Fall (September 1st - November 15th)	No unvegetated area shall exceed 10 square feet in order to limit erosion.
Bioretention Swales	Bioretention Soil Mix #1	Plant List #2	Maintenance List #1		No unvegetated area shall exceed 9 square feet in order to limit erosion.
Permeable Unit Pavements	Reservoir Course #1		Maintenance List #1		
Porous Pavements	Base Course #1	Sod or Seed Mix #5	Maintenance List #1		If Porous Grass Turf Paving: No unvegetated area shall exceed 9 square feet in order to limit erosion.
Sand and organic media filters	Base Media #1	Seed Mix #5 if grass is added	Maintenance List #1		-
Stormwater Ponds	Pond Liner Base #1, Aquatic Bench Media #1	Plant List #4 - Aquatic Bench, Low Marsh, and High Marsh Zone, Plant List #5 - Wetland Plants, Plant List #7 - Wooded Vegetation	Maintenance List #1		-

* Seed mixes and plant lists can be found on following pages

Application	Soil Preparation or Media	Recommended Materials*	Maintenance	Installation	Additional Notes
Constructed Wetlands		Plant List #3 - Deep Water Zone, Plant List #4 - Aquatic Bench, Low Marsh, and High Marsh Zone, Plant List #6 - Semi-Wet Zone Plant List #7 - Wooded Vegetation	Maintenance List #1		-

* Seed mixes and plant lists can be found on following pages

A.6.2 Media

Bioretention Soil Mix #1	
Material	**Percentage (%)**
Screened Cushion Sand	≥ 25
Expanded Shale	25
Fully Mature "Finished" Compost	≤ 50

A.6.3 Seed Mixes

Nurse Crop Seed					
All Applications for Cool Weather Seeding					
Seed/Seed Mixes	**Grain**				
Scientific Name	**Common Name**	**Percentage (%) by Weight**	**Germination Percentage (%)**	**Purpose**	**Installation Window**
Secale Cereale	Cereal Rye	100.00	100	Cool-season nurse crop to hold soils	October to March 1st
Application Rate = 100 lbs./acre (1 lb. / 436 sf.)					

Seed Mix #1					
Dry/Wet Conditions - Bottom and Sides					
Wildflowers (Forbs)					
Scientific Name	**Common Name**	**Percentage (%) by Weight**	**Germination Percentage (%)**	**Purpose**	**Installation Window**
Bouteloua gracilis	Blue Grama	31.05	75	Soil Stabilization - Warm-season crop	Spring (March 1st - May 15th) or Fall (Sept. 1st - Nov. 15th) if with Seed Mix #3
Bouteloua dactyloides	Buffalograss	11.00	94	-	-
Schizachyrium scoparium	Little Bluestem	11.00	42	-	-
Bouteloua curtipendula	Sideoats Grama	10.18	86	-	-
Andropogon gerardii	Big Bluestem	4.95	-	-	-
Leptochloa dubia	Green Sprangletop	4.95	-	-	-
Tripsacum dactyloides	Eastern Gamagrass	4.90	-	-	-
Sporobolus cryptandrus	Sand Dropseed	4.90	-	-	-
Elymus canadensis	Prairie Wildrye	4.75	-	-	-
Panicum virgatum	Switchgrass	3.50	-	-	-
Eragrostis trichodes	Sand Lovegrass	3.02	-	-	-
Sorghastrum nutans	Yellow Indiangrass	2.90	-	-	-
Pascopyrum smithii	Western Wheatgrass	1.00	-	-	-
Bothriochloa barbinodis	Cane Bluestem	0.07	-	-	-
Eriochloa sericea	Texas Cupgrass	0.55	-	-	-
Hilaria belangeri	Curly Mesquite	0.25	-	-	-
Tridens albenscens	White Tridens	0.20	-	-	-
Andropogon glomeratus or Panicum hallii	Bushy Bluestem or Halls Panicum	0.20	-	-	-
Application Rate = 29 lbs./acre (1 lb. / 1,500 sf.)					

Seed Mix #2					
Dry/Wet Conditions - Bottom					
Wildflowers (Forbs)					
Scientific Name	**Common Name**	**Percentage (%) by Weight**	**Germination Percentage (%)**	**Purpose**	**Installation Window**
Centaurea americana	American Basketflower	49.66	8	Soil Stabilization - Cool-season crop	Fall (Sept. 1st - Nov. 15th) or Spring (March 1st - May 15th) if with Seed Mix #1
Desmanthus illinoensis	Illinois Bundleflower	23.91	12	-	-
Dracopis amplexicaulis	Clasping Coneflower	8.07	99	-	-
Coreopsis tinctoria	Plains Coreopsis	3.23	-	-	-
Rudbeckia hirta	Black-Eyed Susan	3.23	-	-	-
Salvia splendens	Scarlet Sage	3.07	-	-	-
Engelmannia peristenia	Cutleaf Daisy	2.67	-	-	-
Oenothera speciosa	Pink Evening Primrose	1.94	-	-	-
Helianthus maximilliani	Maximillian Sunflower	0.96	-	-	-
Solidago gigantea	Giant Goldenrod	0.17	-	-	-
Application Rate = 22 lbs./acre (1 lb. / 2,000 sf.)					

Seed Mix #3					
Dry Conditions - Upper and Back Slopes					
Wildflowers (Forbs)					
Scientific Name	**Common Name**	**Percentage (%) by Weight**	**Germination Percentage (%)**	**Purpose**	**Installation Window**
Centaurea americana	American Basketflower	19.93	8	Soil Stabilization - Cool-season crop	Fall (Sept. 1st - Nov. 15th) or Spring (March 1st - May 15th) if with Seed Mix #1
Lupinus texensis	Texas Bluebonnet	18.23	21	-	-
Coreopsis tinctoria	Plains Coreopsis	15.52	15	-	-
Gaillardia pulchella	Indian Blanket	15.32	88	-	-
Monarda citriodora	Lemon Mint	11.82	77	-	-
Dalea purpurea	Purple Prairie Clover	10.31	26	-	-
Dalea candida	White Prairie Clover	6.66	91	-	-
Chamaecrista fasciculata	Partridge Pea	1.90	-	-	-
Liatris punctata var. mucronata	Gayfeather	0.20	-	-	-
Solidago altissima	Tall Goldenrod	0.10	-	-	-
Lindheimera texana	Texas Yellow Star	0.01	-	-	-
Application Rate = 26 lbs./acre (1 lb. / 1,700 sf.)					

Seed Mix #4 - Overseed					
Wet and Moist Conditions - Regular Drainage or Seepage Along Bottom					
Grasses					
Scientific Name	**Common Name**	**Percentage (%) by Weight**	**Germination Percentage (%)**	**Purpose**	**Installation Window**
Secale cereale	Cereal Rye Grain	75.39	98	Nurse crop to hold soils	Nov. 1st - Feb.
Andropogon gerardii	Big Bluestem	8.20	98	Soil Stabilization - Warm-season crop	-
Tripsacum dactyloides	Eastern Gamagrass	-	-	-	-
Elymus canadensis	Prairie Wildrye	-	-	-	-
Leptochloa dubia	Green Sprangletop	-	-	-	-
Panicum virgatum	Switchgrass	-	-	-	-
Tridens albenscens	White Tridens	-	-	-	-
Andropogon virginicus	Broomsedge Bluestem	-	-	-	-
Andropogon glomeratus	Bushy Bluestem	-	-	-	-
Application Rate = 218 lbs./acre (1 lb. / 200 sf.)					

Seed Mix #5					
Access Roads or Pedestrian Access Ramps					
Grasses					
Scientific Name	**Common Name**	**Percentage (%) by Weight**	**Germination Percentage (%)**	**Purpose**	**Installation Window**
Bouteloua dactyloides	Buffalograss	82.00	98	Soil Stabilization - Warm-season crop	Spring (March 1st - May 15th); Fall (Sept. 1st - Nov. 15) if regular rain and up until 8 weeks before avg. frost date
Bouteloua gracilis	Blue Grama	17.00	98		-
Hilaria belangeri	Curly Mesquite	1.00	-	-	-
Application Rate = 44 lbs./acre (3 lbs. / 1,000 sf. or 1 lb. / 333 sf.)					

Plant List #1			
Recommended Plants for Rain Gardens and Bioretention			
Trees		**Shrubs**	
Scientific Name	**Common Name**	**Scientific Name**	**Common Name**
Cercis canadensis	Eastern Redbud	Baptisia australis	False Indigo
Lagerstroemia indica	Crapemyrtle	Callicarpa americana	American Beautyberry
Styphnolobium affine	Eve's Necklace	Salvia greggii	Autumn Sage
Chilopsis linearis	Desert Willow	Hesperaloe parviflora	Red Yucca
Ilex vomitoria	Yaupon Holly	Rosmarinus officinalis	Rosemary
Quercus sp.	Oak	Symphoricarpos orbiculatus	Coralberry
Acer truncatum	Shantung Maple		

Herbaceous Species	
Scientific Name	**Common Name**
Eutrochium maculatum	Joe Pye Weed
Lobelia cardinalis	Cardinal Flower
Rudbeckia hirta	Black-Eyed Susan
Malvaviscus arboreaus var. drummondii	Turk's Cap
Symphyotrichum oblongifolium	Aromatic Aster
Salvia farinacea	Mealy Blue Sage
Iris ser. Hexagonae	Louisiana Iris
Muhlenbergia capillaris	Gulf Muhly
Nassella tenuissima	Mexican Feathergrass
Muhlenbergia lindheimeri	Lindheimer Muhly

Plant List #2					
Recommended Plants for Rain Gardens and Bioretention					
Trees					
Scientific Name	**Botanical Name**	**Size (Height x Spread)**	**Evergreen or Deciduous**	**Sun or Shade**	**Location (Wet or Dry)**
Cercis canadensis	Eastern Redbud	20’-30’ x 20’-30’	Deciduous	Sun/Part Shade/ Shade	Dry
Cercis reniformis ‘Oklahoma’	Redbud var. - ‘Oklahoma’, Texas or Mexican variety	15’-20’ x 15’-20’	Deciduous	Sun/Part Shade	Dry
Lagerstroemia indica	Crapemyrtle	28’-30’ x 30’-35’	Deciduous	Sun	Dry
Lagerstroemia indica var.	Crapemyrtle (“Indian” name var.)	28’-30’ x 30’-35’	Deciduous	Sun	Dry
Chilopsis linearis	Desert Willow	25’ x 25’	Deciduous	Sun	Dry
Acer truncatum	Shantung Maple	25’ x 20’	Deciduous	Sun/Part Shade	Semi-Wet/Dry
Styphnolobium affine	Eve’s Necklace	30’ x 20’	Deciduous	Sun/Shade	Semi-Wet/Dry
Ilex decidua	Possumhaw (away from road edges)	15’-30’ x 15’-20’	Deciduous (with berries)	Sun/Part Shade	Wet/Dry
Ilex vomitoria	Yaupon Holly (Male by roadways)	20’ x 20’	Evergreen	Sun/Shade	Wet/Dry
Ilex vomitoria	Yaupon Holly (Female - other areas)	20’ x 20’	Evergreen (with berries)	Sun/Shade	Wet/Dry
Quercus sp.	Oak varieties	50’-60’ x 60’-70’	Evergreen & Deciduous	Sun	Wet/Dry
Ulmus crassifolia	Cedar Elm	60’ x 30’	Deciduous	Sun	Wet/Dry

Shrubs					
Scientific Name	**Common Name**	**Size (Height x Spread)**	**Evergreen or Deciduous**	**Sun or Shade**	**Location (Wet or Dry)**
Symphorcarpos orbiculatus	Indiancurrant Coralberry	1'-6' x 1'-2'	Deciduous	Part Shade/ Shade	Dry
Hesperaloe parviflora 'Perpa' 'Brakelights'	'Brakelight' Red Yucca	2' x 2'	Evergreen	Sun/Part Shade	Dry
Hesperaloe parviflora	Red Yucca (Hesperaloe)	3' x 3'-5'	Evergreen	Sun/Part Shade	Dry
Hesperaloe parviflora 'Yellow'	Yellow Yucca (Hesperaloe)	3' x 3'-5'	Evergreen	Sun/Part Shade	Dry
	Rosemary	4'-5' x 4'-5'	Evergreen/ Semi-Evergreen	Sun	Dry
Rosmarinus officinalis Arp	'Arp' Rosemary	2'-3' x 4'-5'	Evergreen/ Semi-Evergreen	Sun	Dry
Salvia greggii	Autumn Sage	2'-3' x 4'-5'	Evergreen/ Semi-Evergreen	Sun	Dry
Amorpha fruticosa	False Indigo Bush	8'-10' x 10'-12'	Deciduous	Part Shade/ Shade	Semi-Wet/ Semi-Dry/Dry
Yucca pallida	Pale-Leaf Yucca	1'-2' x 1'-3'	Evergreen	Sun	Wet/Dry
Callicarpa americana	American Beautyberry	4'-6' x 5'-8'	Deciduous	Part Shade/ Shade	Wet/Dry
Sabal minor	Dwarf Palmetto (Texas)	4' x 5'	Evergreen	Shade	Wet/Dry
Ornamental Grasses					
Scientific Name	**Common Name**	**Size (Height x Spread)**	**Winter Foliage Benefit**	**Sun or Shade**	**Location (Wet or Dry)**
Nassella tenuissima	Mexican Feathergrass	1'-2' x 1'-2'	Yes	Sun	Dry
Muhlenbergia capillaris 'Regal Mist'	'Regal Mist' Gulf Muhly	2' x 3'	Yes	Sun	Wet/Dry
Muhlenbergia lindheimeri	Lindheimer Muhly	4'-5' x 3'-4'	Yes & Semi-Evergreen	Sun/Part Shade	Wet/Dry
Carex cherokeensis	Cherokee Sedge	1'-2' x 1'-2'	Evergreen	Part Shade/ Shade	Wet/Dry
Carex emoryi	Emory's Sedge	2' x 2'	Semi-Evergreen	Sun/Part Shade	Wet/Semi-Dry
Chasmanthium latifolium	Inland Sea Oats	3'-4' x 2'	Yes	Part Shade/ Shade	Wet/Semi-Dry

Perennials					
Scientific Name	**Common Name**	**Size (Height x Spread)**	**Dieback or Evergreen**	**Sun or Shade**	**Location (Wet or Dry)**
Wedelia acapulcensis var. hispida	Zexmenia	1’-2’ x 2’-3’	Evergreen/ Semi-Evergreen	Sun/Part Shade	Semi-Wet/Dry
Symphyotricum oblongifolium	Aromatic Aster	1’-3’ x 1’-2’	Winter Dieback	Sun/Part Shade	Semi-Wet/Dry
Conoclinium greggii	Gregg’s Mistflower	18”-2’ x 2’-3’	Winter Dieback	Sun/Part Shade	Semi-Wet/ Semi-Dry/Dry
Salvia farinacea	Mealy Blue Sage	2’-3’ x 2’-3’	Winter Dieback	Sun	Semi-Wet/ Semi-Dry/Dry
Rudbeckia hirta	Black-Eyed Susan	2’ x 2’	Winter Dieback	Sun	Wet/Dry
Malvaviscus arboreaus var. drummondii	Turk’s Cap	2’-3’ x 3’-4’	Winter Dieback	Part Shade/ Shade	Wet/Dry
Symphyotricum praealtum var. praealtum	Tall Aster	1’-3’ x 1’-2’	Winter Dieback	Sun/Part Shade/ Shade	Wet/Semi-Wet
Eutrochium maculatum	Joe Pye Weed	4’ x 2’	Winter Dieback	Part Shade/ Shade	Wet/Semi-Wet
Lobelia cardinalis	Cardinal Flower	2’ x 2’-4’	Winter Dieback	Part Shade/ Shade	Wet/Semi-Wet
Iris ser. Hexagonae	Louisiana Iris	3’ x 6”	Winter Dieback	Part Shade/ Shade	Wet/Semi-Wet

A.6.4 Recommended Plants

Constructed Wetlands

Plant List #3 - Deep Water Zone	
(6' to 18")	
Scientific Name	**Common Name**
Nuphar luteum	Spatterdock, Cow Lily
Mympheae mexicana	Yellow Water Lily
Mynphaea odorata	Fragrant (White) Water Lily

Plant List #4 - Aquatic Bench, Low Marsh, and High Marsh (18" to NWL)		
Scientific Name	**Common Name**	**Depth of Water - Planting**
Acrous calumus	Sweetflag	
Andropogon glomeratus	Bushy Bluestem	0”
Carex spp.	Caric Sedges	
Eleocharis quadrangulata	Squarestem Spikerush	0”-6”
Helianthus angustifolius	Swamp Sunflower	
Hibiscus laevis	Halberdleaf Hibiscus	
Hibiscus moscheutos	Swamp Rosemallow	0”
Hymenocallis liriosme	Spring Spiderlily	0”-3”
Iris virginica	Southern Blue Flag Iris	0”-3”
Juncus effuses	Soft Rush, Common Rush	0”-3”
Leersia oryzoides	Rice Cut Grass	
Panicum virgatum	Switchgrass	0”
Peltandra virginica	Arrow Arum, Smart Arum, Green Arum	0”-3”
Polygonum hydropiperoides	Smartweed	
Pontederia cordata	Pickerelweed, Pickerel Rush	0”-6”
Pontederia lanceolata	Pickerelweed, Pickerel Rush	0”-6”
Rudbeckia maxima	Swamp Coneflower	0”
Sagittaria lancifolia	Lance-leaf Arrowhead, Bulltongue Arrowhead	0”-3”
Sagittaria latifolia	Duck Potato	
Sagittaria platyphyla	Delta Arrowhead	0”-6”
Saururus cernuus	Lizard’s Tail	0”-3”
Schoenoplectus americanus	Three-square	
Schoenoplectus californicus	Giant Bulrush, California Bulrush, Southern Bulrush	
Schoenoplectus tabernaemontani	Softstem Bulrush	
Tradescantia ohioensis	Spiderwort	0”
Woodwardia virginica	Virginia Chain Fern	

Stormwater Ponds

Plant List #5 - Wetland Plants	
(NWL to +6")	
Scientific Name	**Common Name**
Andropogon glomeratus	Bushy Bluestem
Chasmanthium latifolium	Upland Sea Oats
Coreopsis tinctoria	Dwarf Tickseed
Euptorium serotinum	Late Boneset
Helianthus maximiliani	Maximilian Sunflower
Hibiscus laevis	Halberdleaf Hibiscus
Liatris spicata	Spiked Gayfeather
Lobelia cardinalis	Cardinal Flower
Osmunda cinnamomea	Cinnamon Fern
Osmunda regalis	Royal Fern
Panicum capillare	Witchgrass
Pontederia cordata	Pickerelweed, Pickerel Rush
Schoenoplectus tabernaemontani	Softstem Bulrush
Tripsacum dactyloides	Eastern Gamagrass

Constructed Wetlands

Plant List #6 - Semi-Wet Zone	
(NWL to +4')	
Scientific Name	**Common Name**
Andropogon glomeratus	Bushy Bluestem
Andropogon virginicus	Broomsedge, Broom Grass
Chasmanthium latifolium	Upland Sea Oats
Coreopsis tinctoria	Dwarf Tickseed
Elymus cadensis	Canadian Wildrye
Elymus virginicus	Virginia Wildrye
Eupatorium fistolosum	Joe Pye Weed
Eupatorium serotinum	Late Boneset
Eustoma grandiflora	Texas Bluebells
Helianthus maximiliani	Maximilian Sunflower
Hibiscus laevis	Halberdleaf Hibiscus
Liatris spicata	Spiked Gayfeather
Lobelia cardinalis	Cardinal Flower
Malvaviscus drummondii	Turk's Cap
Osmunda cinnamomea	Cinnamon Fern
Osmunda regalis	Royal Fern
Panicum capillare	Witchgrass
Pontederia cordata	Pickerelweed, Pickerel Rush
Rudbeckia hirta	Black-eyed Susan
Schoenoplectus tabernaemontani	Softstem Bulrush
Sorgham nutans	Yellow Indian Grass
Tripsacum dactyloides	Eastern Gamagrass

Plant List #7 - Wooded Vegetation	
(+6" and up)	
6" to 4' - Regularly Inundated	
Herbaceous Species	
Scientific Name	**Common Name**
Andropogon virginicus	Broomsedge, Broom Grass
Elymus canadensis	Canadian Wildrye
Elymus virginicus	Virginia Wildrye
Eupatorium serotinum	Late Boneset
Eustoma grandiflora	Texas Bluebells
Helianthus maximiliani	Maximilian Sunflower
Malvaviscus drummondii	Turk's Cap
Panicum capillare	Witchgrass
Rudbeckia hirta	Black-eyed Susan
Sorghastrum nutans	Yellow Indian Grass
Tripsacum dactyloides	Eastern Gamagrass
Shrubs and Ornamental Trees	
Scientific Name	**Common Name**
Cephalanthus occidentalis	Common Buttonbush
Ilex decidua	Possumhaw (Deciduous Holly)
Myrica cerifera	Southern Waxmyrtle
Ptelea trifoliata	Wafer Ash, Common Hoptree
Trees	
Scientific Name	**Common Name**
Taxodium distichum	Bald Cypress
Taxodium distichum var. nutans	Pond Cypress
Fraxinus pennsylvanica	Green Ash
Salix nigra	Black Willow

4' and up - Periodically Inundated	
Herbaceous Species	
Scientific Name	**Common Name**
Elymus canadensis	Canadian Wildrye
Elymus virginicus	Virginia Wildrye
Helianthus maximiliani	Maximilian Sunflower
Liatris pycnostachya	Prairie Gayfeather
Malvaviscus drummondii	Turk's Cap
Panicum capillare	Witchgrass
Tripsacum dactyloides	Eastern Gamagrass
Shrubs and Ornamental Trees	
Scientific Name	**Common Name**
Cornus drummondii	Rough-Leaf Dogwood
Ilex decidua	Possumhaw (Deciduous Holly)
Lindera benzoin	Spicebush
Myrica cerifera	Southern Waxmyrtle
Ptelea trifoliata	Wafer Ash, Common Hoptree
Trees	
Scientific Name	**Common Name**
Taxodium distichum	Bald Cypress
Taxodium distichum var. nutans	Pond Cypress
Fraxinus pennsylvanica	Green Ash
Diospyros virginiana	Common Persimmon
Ulmus crassifolia	Cedar Elm
Quercus lyrata	Overcup Oak
Quercus nigra	Water Oak
Quercus phellos	Willow Oak
Salix nigra	Black Willow

1 INTRODUCTION
2 HYDROLOGY
3 ROADWAY DRAINAGE DESIGN
4 BRIDGE HYDRAULIC DESIGN
5 OPEN CHANNEL DESIGN
6 DRAINAGE STORAGE DESIGN
7 EROSION & SEDIMENT CONTROL MEASURES
8 FLOODPLAIN & SUMP DESIGN REQUIREMENTS
9 OTHER REGULATORY REQUIREMENTS
10 SUBMITTAL REQUIREMENTS
11 OPERATION & MAINTENANCE
APPENDICES

Infrequently Inundated	
Herbaceous Species	
Scientific Name	**Common Name**
Andropogon gerardii	Big Bluestem
Asclepias tuberosa	Butterfly-weed
Bouteloua curtipendula	Sideoats Grama
Bouteloua (Buchloe) dactyloides	Buffalograss
Cynodon dactylon	Bermuda Grass
Echinacea purpurea	Purple Coneflower
Helianthus maximiliani	Maximilian Sunflower
Leptochola dubia	Green Sprangletop
Liatris puncata	Dotted Gayfeather
Liatris puncata var. mucronatum	Texas Gayfeather
Liatris pycnostachya	Prairie Gayfeather
Malvaviscus drummondii	Turk's Cap
Panicum capillare	Witchgrass
Pennisetum alopecuroides	Fountaingrass
Poa arachnifera	Texas Bluegrass
Salvia farinacea	Mealy Blue Sage
Salvia greggii	Autumn Sage
Schizachyrium scoparium	Little Bluestem
Tripsacum dactyloides	Eastern Gamagrass
Valpia octoflora	Common Sixweeksgrass
Shrubs and Ornamental Trees	
Scientific Name	**Common Name**
Cornus drummondii	Rough-Leaf Dogwood
Ilex decidua	Possumhaw (Deciduous Holly)
Lindera benzoin	Spicebush
Myrica cerifera	Southern Waxmyrtle
Crataegus reverchonii	Reverchon Hawthorn

Trees	
Scientific Name	**Common Name**
Taxodium distichum	Bald Cypress
Fraxinus pennsylvanica	Green Ash
Ulmus crassifolia	Cedar Elm
Quercus lyrata	Overcup Oak
Quercus nigra	Water Oak
Quercus phellos	Willow Oak
Quercus shumardii	Shumard Oak
Magnolia virginiana	Sweetbay Magnolia

A.6.5 Operation and Maintenance Plan

Structural Components	**Corrective Action**
– Clogged inlets or outlets – Damaged liners and walls – Cracked or exposed drain pipes – Check dams – Clogged surface – Cracked pipe, vault, or tank	– Remove sediment and debris from basins, drains, inlets, and pipes to maintain at least 50% conveyance capacity. – Replace broken downspouts, curb cuts, standpipes, and screens as needed. Repair/seal cracks. – Extend and secure liners above the high water mark to maintain water tight conditions. – Vacuum or dry sweep permeable/porous pavers once a year. – Maintain rock check dams as per standard details. – Repair cracks with grout or City-approved materials or replace when cracks are 1 inch wide or more.
Vegetation	
– Dead or strained vegetation – Tall or overgrown plants – Weeds – Large shrubs and trees	– Replant per original planting plan, or substitute from Recommended Plant List. Maintain 90% cover in vegetated areas. – Irrigate as needed. Mulch as needed. Do not apply fertilizers, herbicides, and pesticides. – Prune to allow foot traffic and to insure inlets and outlets can freely convey stormwater. – Manually remove weeds. Remove all plant debris. – Prevent large root systems from damaging subsurface structural components. – Mow natural grassed areas two times per year: July 1st and in dead of Winter (January 1st) to no shorter than 8 inches in height. Bag and remove from site all clippings and trimmings.
Growing / Filter Medium	
– Erosion and/or exposed soils – Scouring at inlets – Ponding – Slope slippage – Aggregate loss in pavers	– Fill and lightly compact areas of concern with soil mix from Materials Matrix. Stabilize soils with plantings from Recommended Plant List. – Replace splash pads at inlets with gravel/rock. – Stabilize 3:1 slopes with plantings from Recommended Plant List. – Remove the top 2-4 inches of sediment at inlets. Add soil mix from Materials Matrix to match elevation of inlet. Rake, till, or amend with soil mix to restore infiltration rate. – Replace paver pore space with aggregate per original design.

A.6.6 Annual Maintenance Schedule

Summer: Make any structural repairs. Improve filter medium as needed. Clear drains and inlets. Irrigate as needed.

Fall: Replant exposed soil and replace dead plants. Remove sediment and plant debris. Vacuum sweep pavers.

Winter: Monitor infiltration/flow-through rates. Clear inlets and outlets/overflows to maintain conveyance.

Spring: Remove sediment and plant debris. Replant exposed soil and replace dead plants. Mulch as needed but do not block the inlets, outlets, or flow paths with mulch.

All seasons: Weed as necessary.

A.6.7 Maintenance Records

All facility operators are required to keep an annual inspection and maintenance log. Record the date description, and contractor (if applicable) for all structural repairs, landscape maintenance, and facility cleanout activities. Keep work orders and invoices on file and make available upon request of the City inspector.

Access: Maintain ingress/egress to design standards.

Infiltration/Flow Control: All facilities shall drain within 24 hours. Record the time/date, weather, and site conditions when ponding occurs.

Pollution Prevention: All sites shall implement BMPs to prevent hazardous or solid wastes or excessive oil and sediment from contaminating stormwater. Record the time/date, weather, and site conditions if site activities contaminate stormwater. Record the time/date and description of the corrective action taken.

Vectors (Mosquitoes and Rodents): Stormwater facilities shall not harbor mosquito larvae or rats that pose a threat to public health or that undermine the facility structure. Monitor standing water for small wiggling sticks perpendicular to the water's surface. Note holes/burrows in and around facilities. Record the time/date, weather, and site conditions when vector activity is observed

A.7

FIGURES

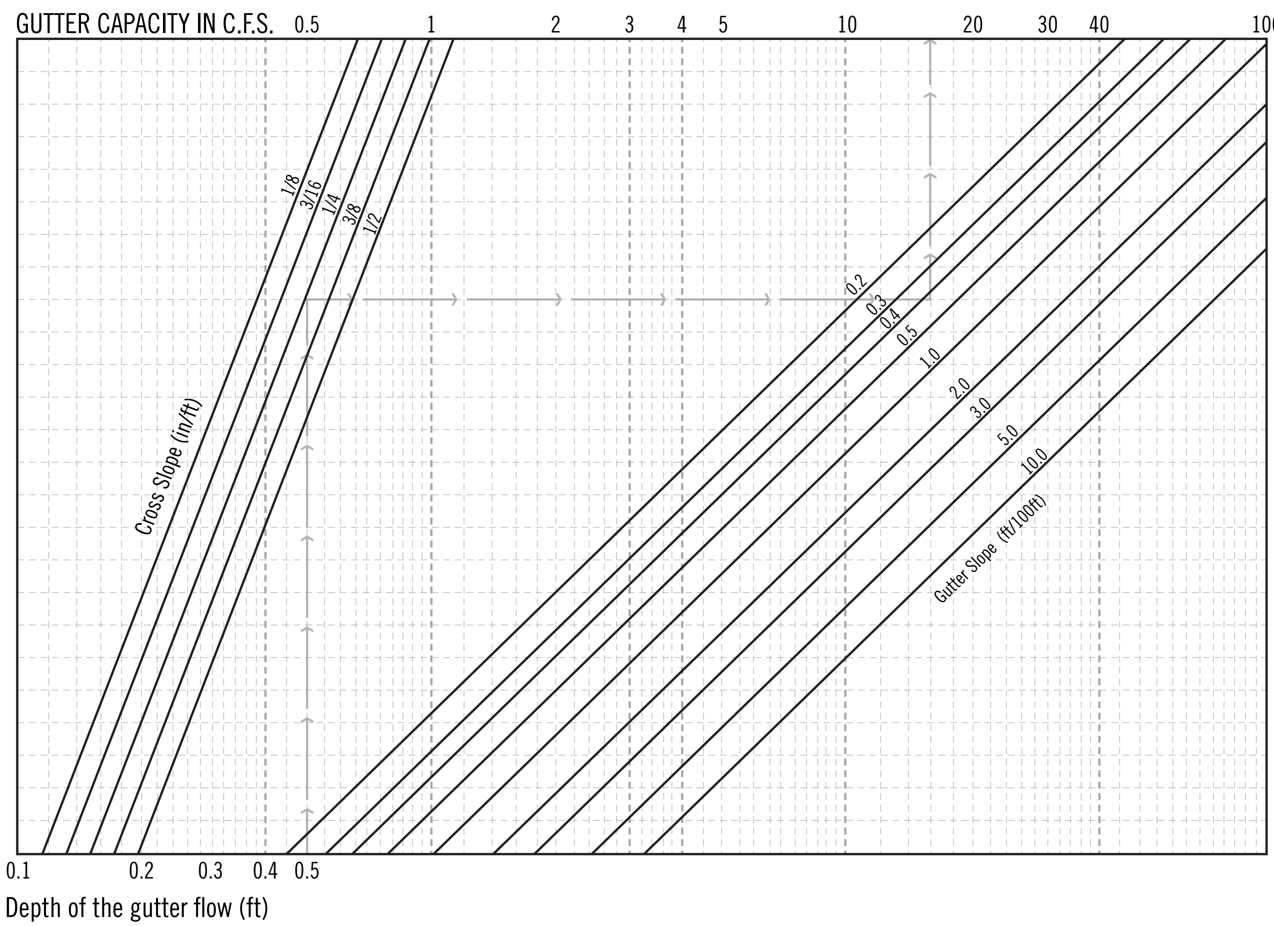
A.7.1 Capacity of Triangular Gutters

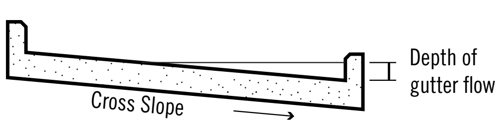
EXAMPLE

KNOWN
Major Street, Type MB
Pavement Width = 33”
Gutter Slope = 1.0%
Pavement Cross Slope = 1/2” / 1’
Depth of Gutter Flow = 0.5’
FIND
Gutter Capacity

SOLUTIONS
Enter Graph at .5’
Intersect Cross Slope = 1/2” / 1’
Intersect Gutter Slope = 1.0%
Read Gutter Capacity = 12 c.f.s.

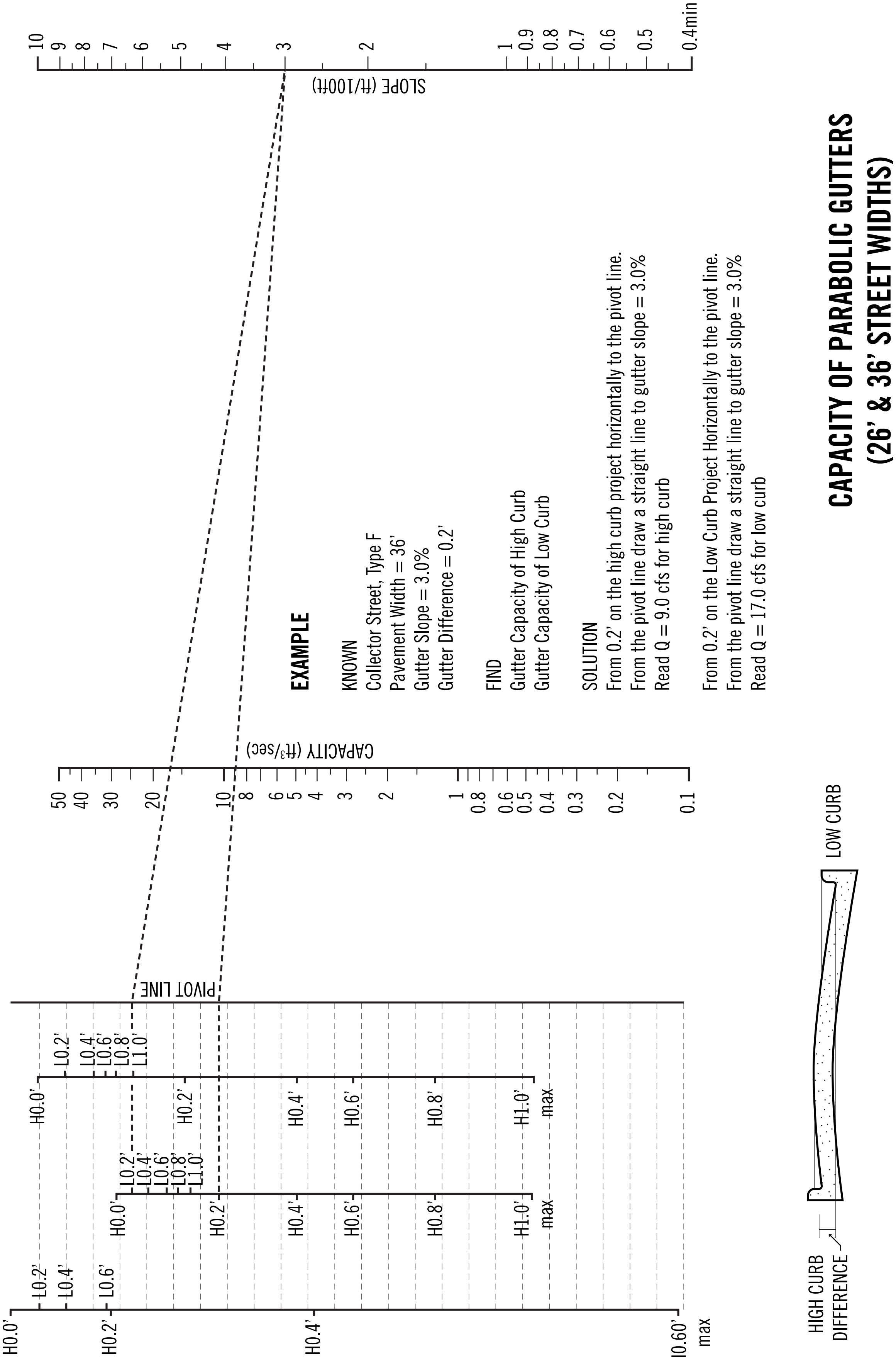
CAPACITY OF TRIANGULAR GUTTERS





(Roughness Coeffecient n = 0.0175)

A.7.2 Parabolic Crown Street Flow Capacity



EXAMPLE

KNOWN
Collector Street, Type F
Pavement Width = 36'
Gutter Slope = 3.0%
Gutter Difference = 0.2'

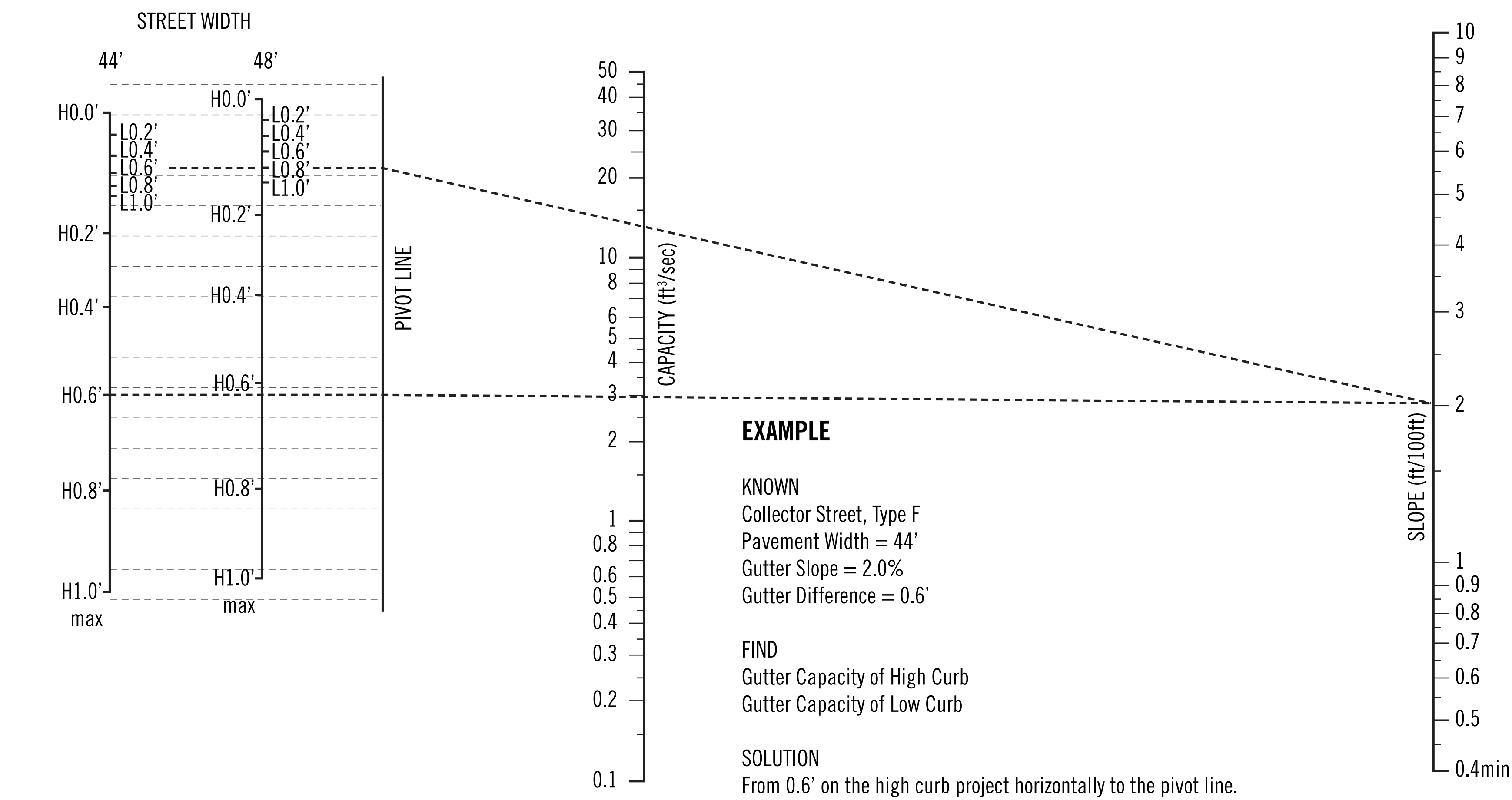
FIND
Gutter Capacity of High Curb
Gutter Capacity of Low Curb

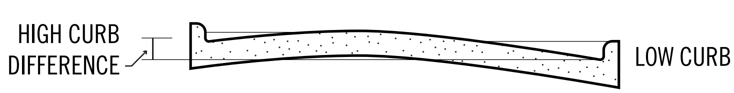
SOLUTION
From 0.2' on the high curb project horizontally to the pivot line.
From the pivot line draw a straight line to gutter slope = 3.0%
Read Q = 9.0 cfs for high curb

From 0.2' on the Low Curb Project Horizontally to the pivot line.
From the pivot line draw a straight line to gutter slope = 3.0%
Read Q = 17.0 cfs for low curb

CAPACITY OF PARABOLIC GUTTERS (26' & 36' STREET WIDTHS)

CAPACITY OF PARABOLIC GUTTERS (44’ & 48’ STREET WIDTHS)





EXAMPLE

KNOWN
Collector Street, Type F
Pavement Width = 44’
Gutter Slope = 2.0%
Gutter Difference = 0.6’

FIND
Gutter Capacity of High Curb
Gutter Capacity of Low Curb

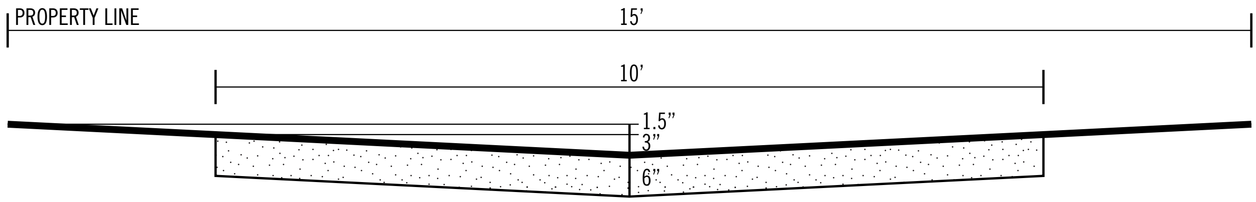
SOLUTION
From 0.6’ on the high curb project horizontally to the pivot line.
From the pivot line draw a straight line to gutter slope = 2.0%
Read Q = 3.0 cfs for high curb

From 0.6’ on the Low Curb Project Horizontally to the pivot line.
From the pivot line draw a straight line to gutter slope = 2.0%
Read Q = 13.0 cfs for low curb

A.7.3 Alley Conveyance Without Curb

EXAMPLES

A) ALLEY WITHOUT CURBS



GIVEN

Concrete n = .0175
Grass n = .035
Alley width = 10’
D = Alley depression = 3” = 0.25’
Alley easment width = 15’
D = Alley depression = 4.5” = 0.375’
Gutter slope = S_y (check your profile; assume normal flow conditions)

REQUIRED

(1) Concrete Alley Conveyance

$$K = \frac{1.486\ AR^{2/3}}{n}$$

(2) Alley Easement Conveyance

(3) Gutter Flow $Q = KS_y^{1/2}$ (assuming S_y = 2.2%)

SOLUTIONS

(1) Concrete Alley Conveyance
D = 0.25’
n = 0.0175

$$A = \frac{(10' * 0.25')}{2} + 1.25\ ft^2$$

$$P = 2\,[(0.25)^2 + (5)^2\,]^{1/2} + 2(0.5) = 10.012ft$$

$$R = \frac{A}{P} = \frac{1.25ft^2}{10.012ft} = 0.125ft$$

$$K = \frac{1.486AR^{2/3}}{n} = 26.51ft^3$$

$$Q = KS_y^{1/2} = (26.51)\ (0.022)^{1/2} = 3.93\ cfs$$

(2) Alley Easment Conveyance
D = 0.375’
n = 0.0233

$$A = \frac{15' * 0.375'}{2} + 2.813\ ft^2$$

$$P = 2\,[(0.375)^2 + (7.5)^2\,]^{1/2} = 15.019ft$$

$$R = \frac{A}{P} = 0.187ft$$

Weighted (perimeter) “n” value:

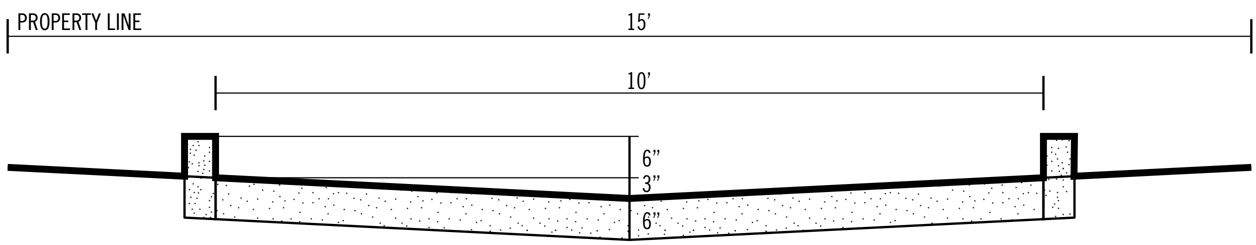
$$n = \frac{(10.012')\,(0.0175) + (5.006')\,(0.035)}{15.018'} = 0.0233$$

$$K = \frac{1.486AR^{2/3}}{n} = 58.71ft^3$$

$$Q = KS_y^{1/2} = (58.71)\ (0.022)^{1/2} = 8.71\ cfs$$

A.7.4 Alley Conveyance With Curb

B) ALLEY WITH CURB



CONCRETE ALLEY CONVEYANCE

$$A = \frac{(10' * 0.25') + (10' * 0.5')}{2} = 6.25\ ft^2$$

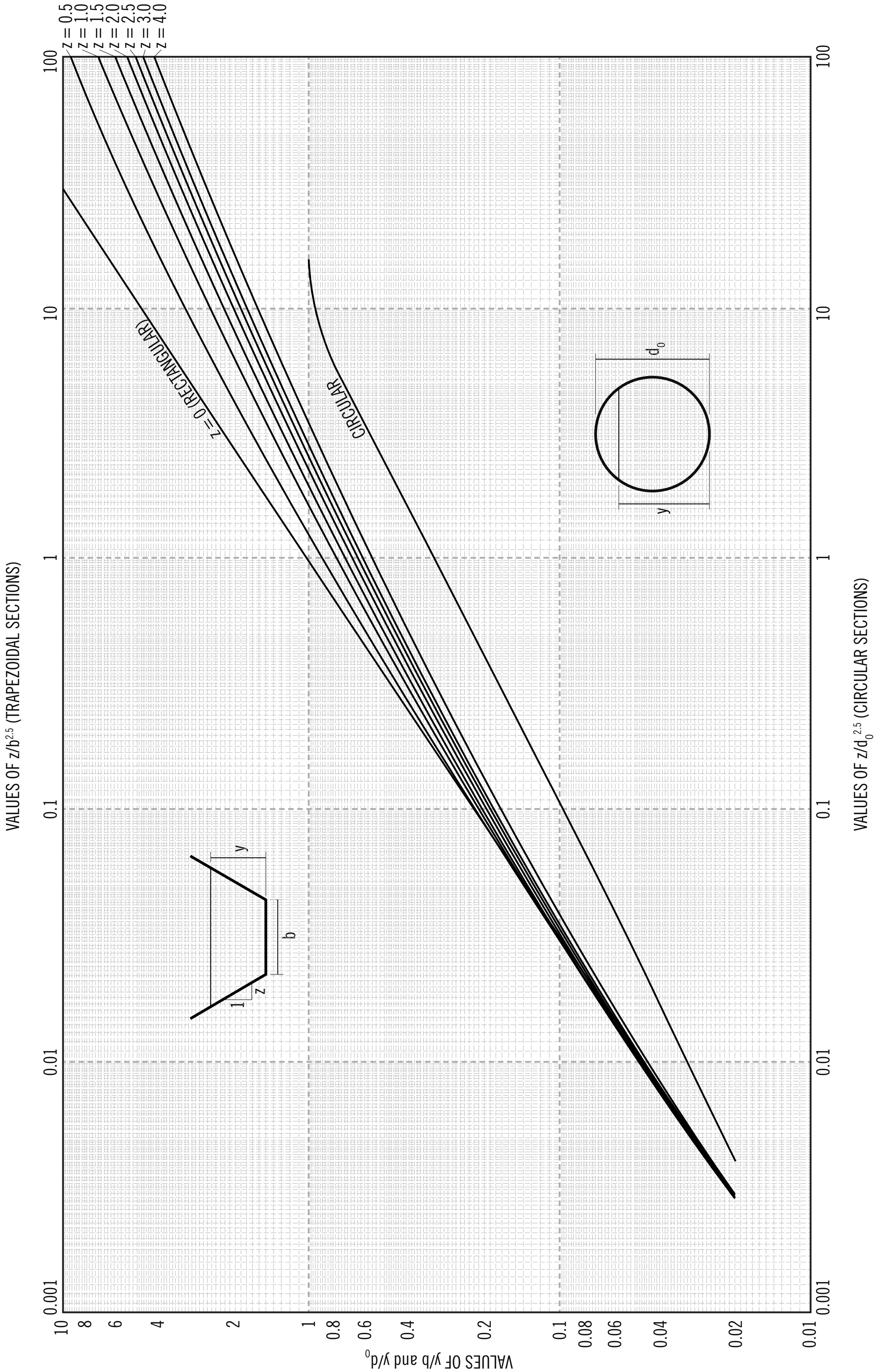
$$P = 2\,[(0.25)^2 + (5)^2\,]^{1/2} + 2(0.5) = 11.012ft$$

$$R = \frac{A}{P} = \frac{6.25ft^2}{11.012ft} = 0.568ft$$

$$K = \frac{1.486AR^{2/3}}{n} = 363.8ft^3$$

$$Q = KS_y^{\ 1/2} = (368.8)\ (0.022)^{1/2} = 53.96\ cfs$$

A.7.5 Curves for Determining the Critical Depth in Open Channels



A.7.6 Section 2 Figures

Figure 2.1 Shallow Concentrated Flow Average Velocity

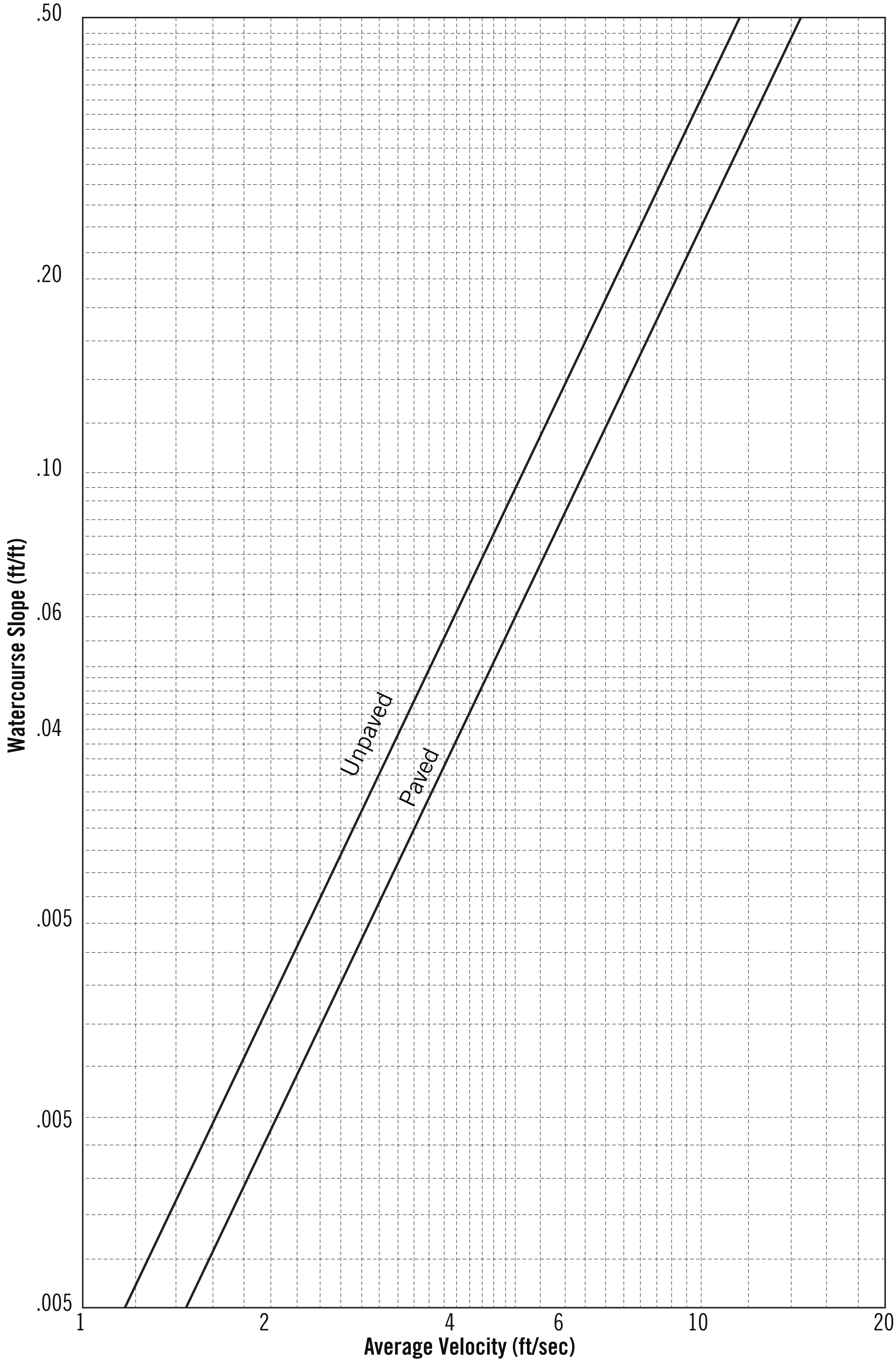
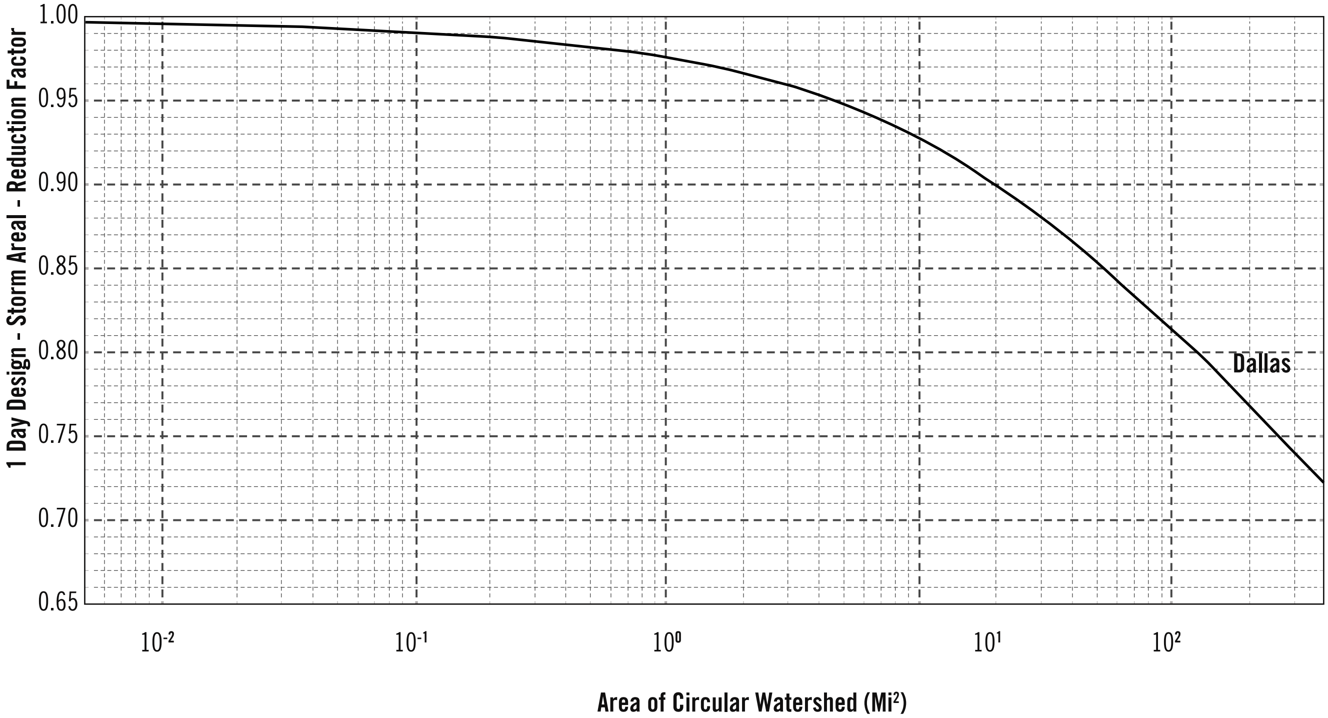


Figure 2.2 Areal Reduction Factors for Small Watersheds for 50% AEP or Greater 1-day Design Storms



A.7.7 Section 3 Figures

Figure 3.2 Parabolic Crown Street Flow Capacity

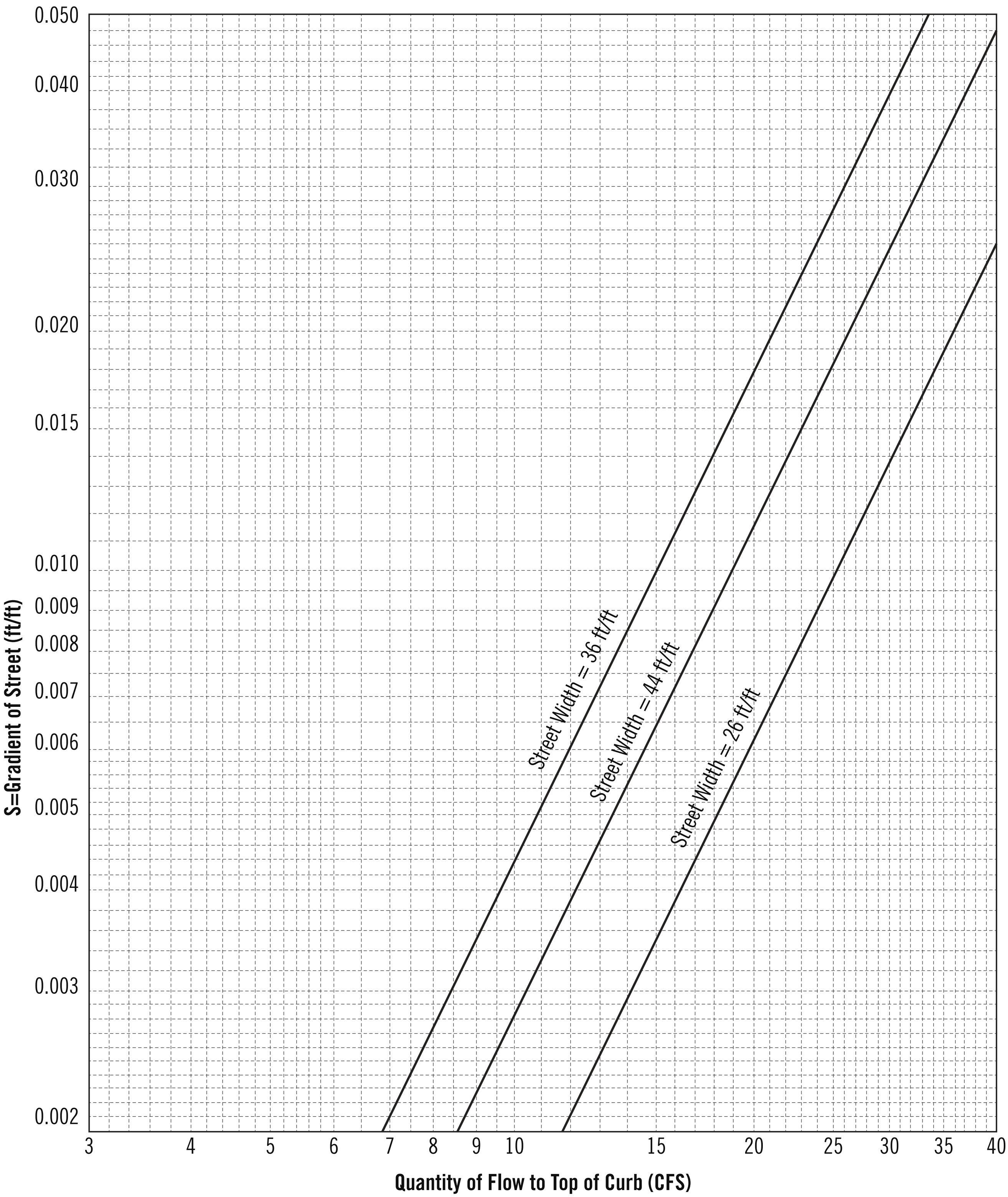


Figure 3.10 Curb Inlets on Grade: Ratio of Intercepted to Total Flow

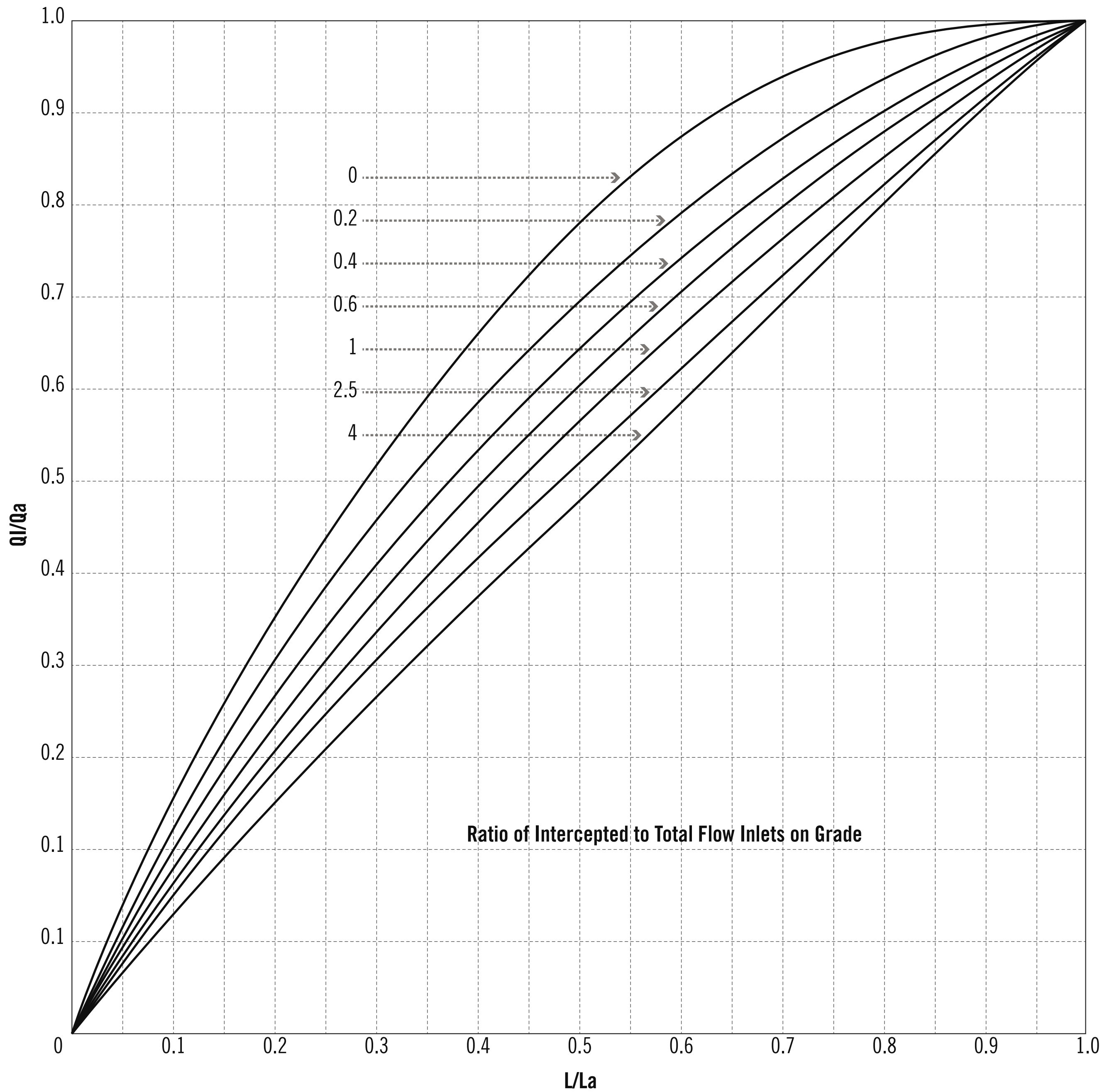


Figure 3.11 Y-Inlet Capacity Curves at Sag Point

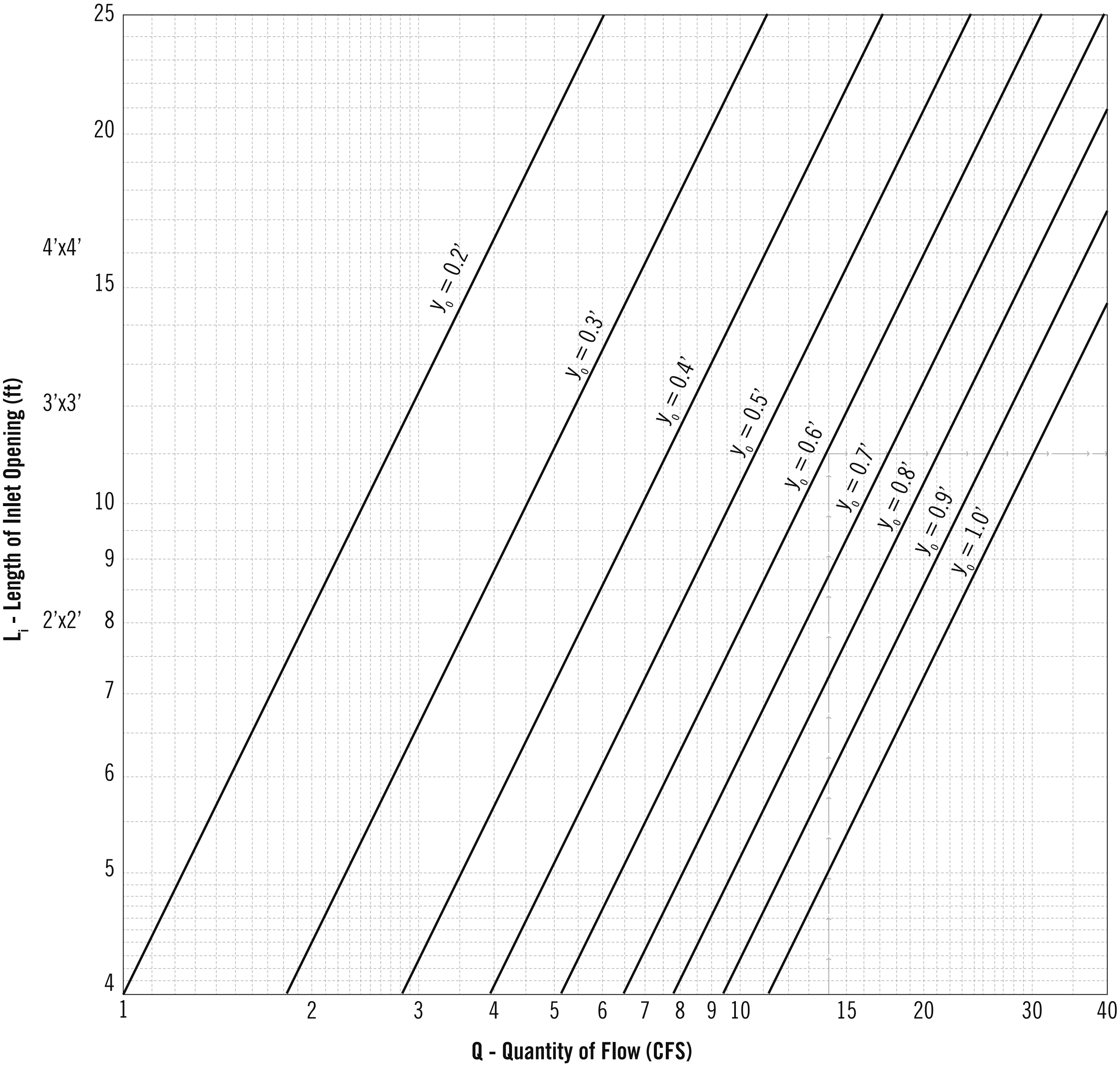


Figure 3.17 Head Losses in Laterals, Wyes, and Enlargements

Head Losses & Gains for Laterals

① $H_L = \frac{V_2^2}{2g} - \frac{V_1^2}{2g}$

② $H_L = \frac{V_2^2}{4g} - \frac{V_1^2}{4g}$

③ $H_L = 1.5\ \frac{V_2^2}{2g}$

④ $H_L = LA * S_fA$

⑤ $H_L = LB * S_fB$

⑥ $H_L = LC * S_fC$

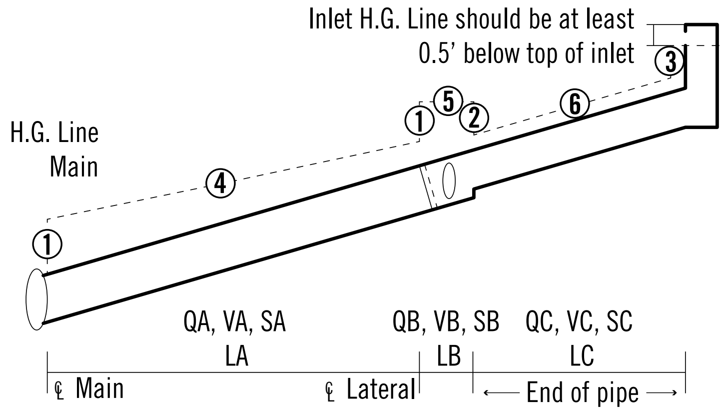
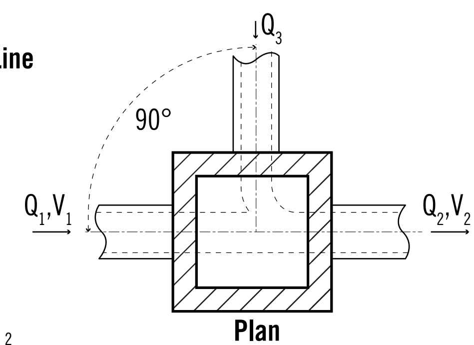


Figure 3.18 Head Losses for Junction Boxes, Manholes, and Bends

90° Lateral
Manhole on Main Line



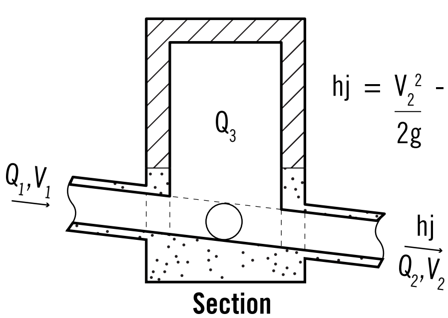
Note

60° Lateral

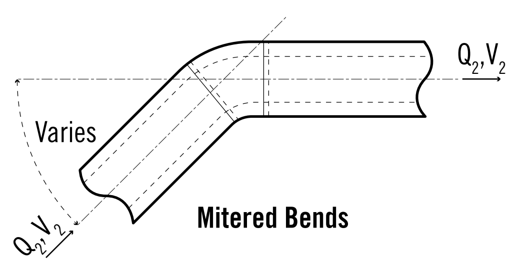
$hj = \frac{V_2^2}{2g} - \frac{0.35\ V_1^2}{2g}$

45° Lateral

$hj = \frac{V_2^2}{2g} - \frac{0.5\ V_1^2}{2g}$



$hj = \frac{V_2^2}{2g} - \frac{0.25\ V_1^2}{2g}$



Note

Head loss applied at beginning of bend. Bends to be used only with the permission of the drainage engineer.

90° Bend
$hj = 0.80\ \frac{V_2^2}{2g}$

60° Bend
$hj = 0.60\ \frac{V_2^2}{2g}$

45° Bend
$hj = 0.50\ \frac{V_2^2}{2g}$

30° Bend
$hj = 0.45\ \frac{V_2^2}{2g}$

Figure 3.25 Over-Embankment Flow Adjustment Factor

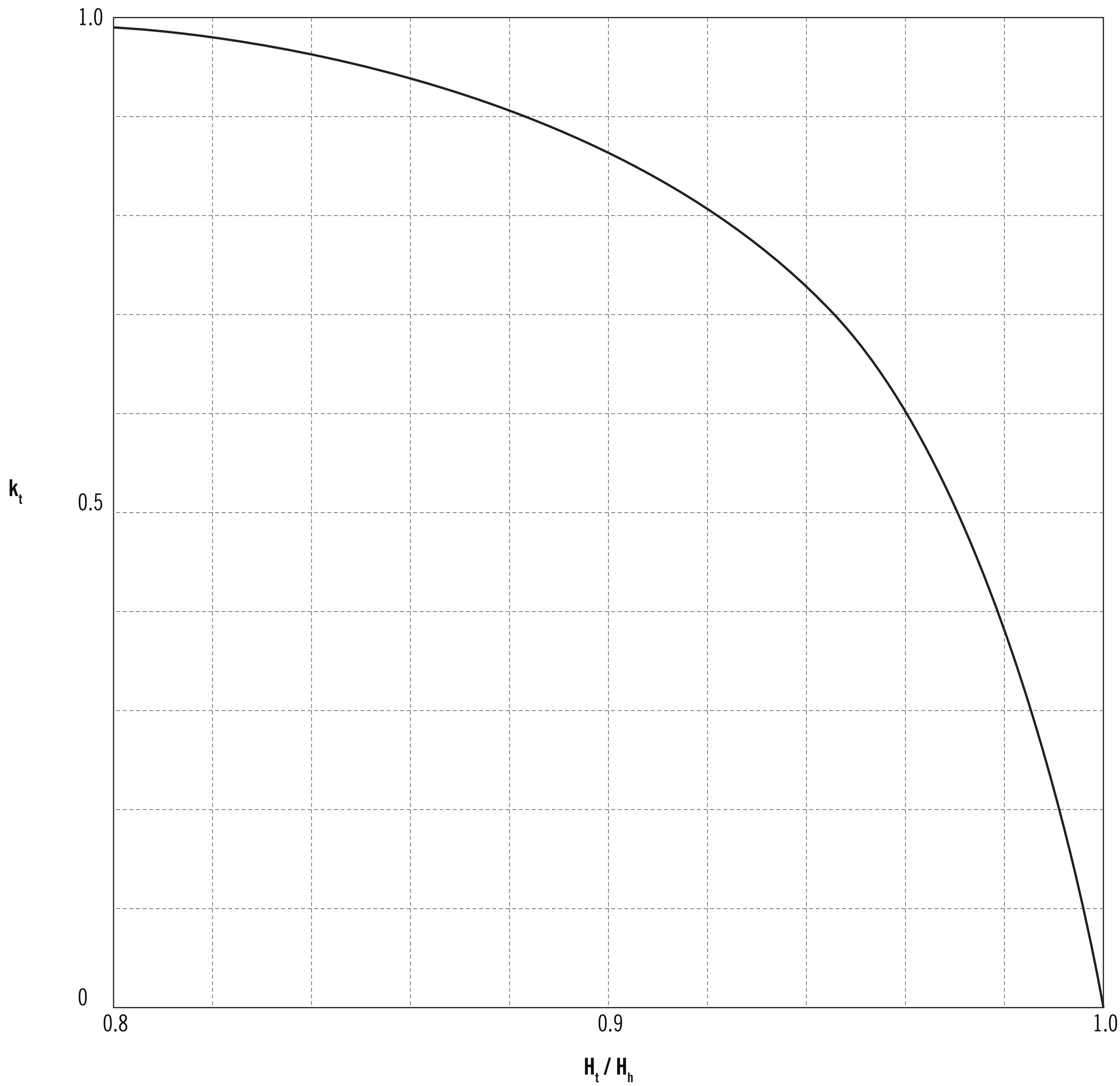


Figure 3.32 Design of Riprap Apron under Minimum Tailwater Conditions

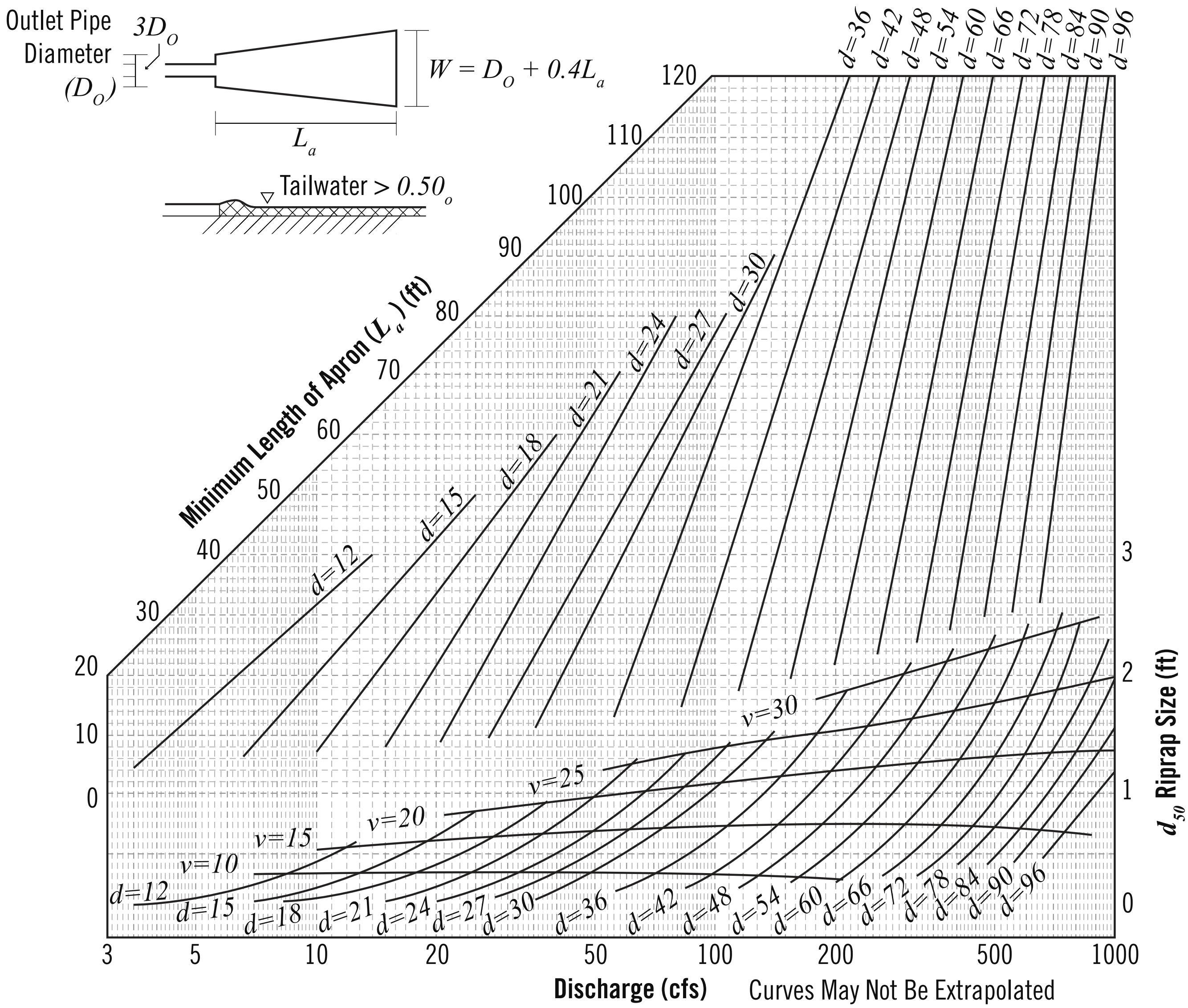


Figure 3.33 Design of Riprap Apron under Maximum Tailwater Conditions

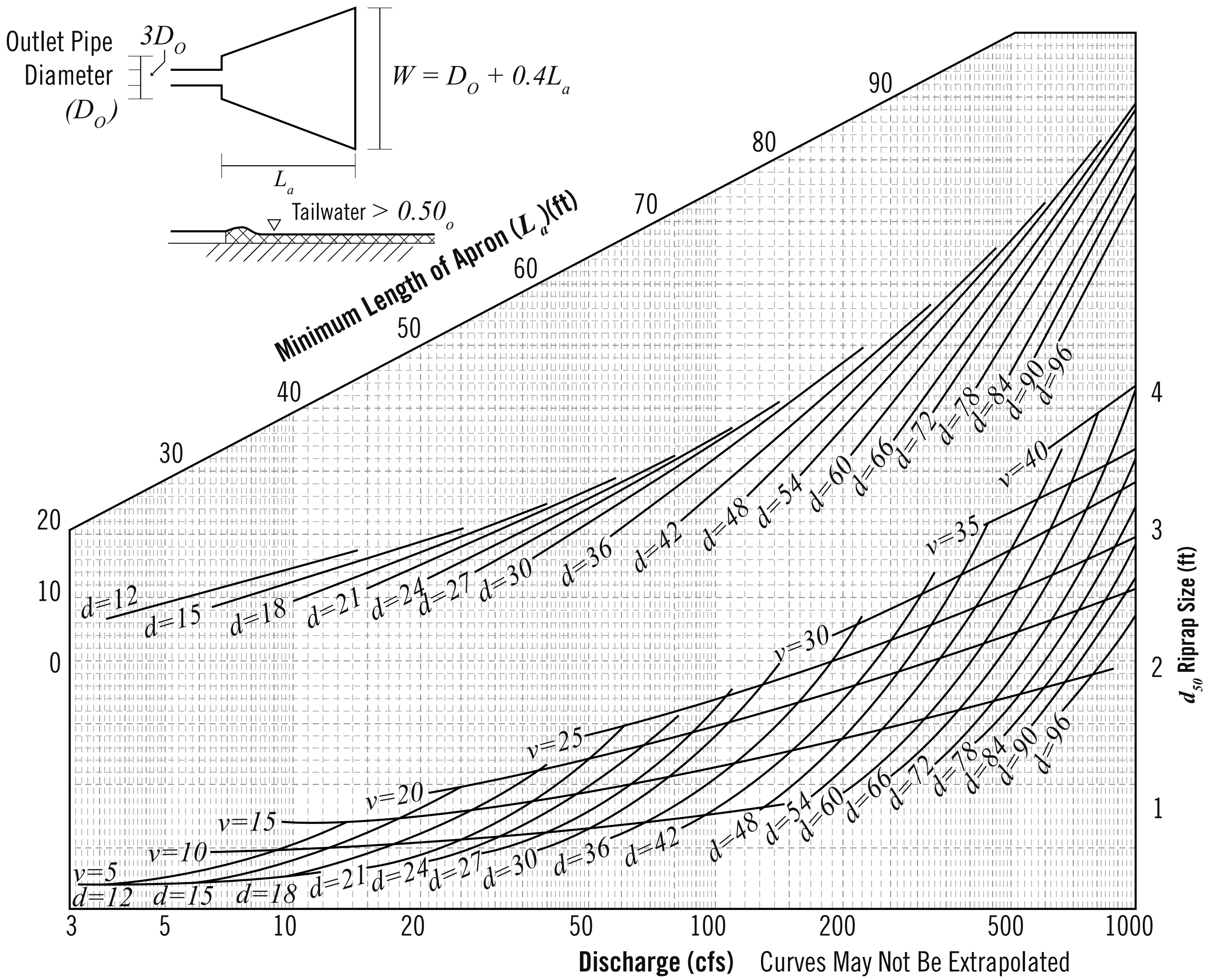


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Figure 3.34 Dimensionless Rating Curves for the Outlets of Rectangular Culverts on Horizontal and Mild Slopes

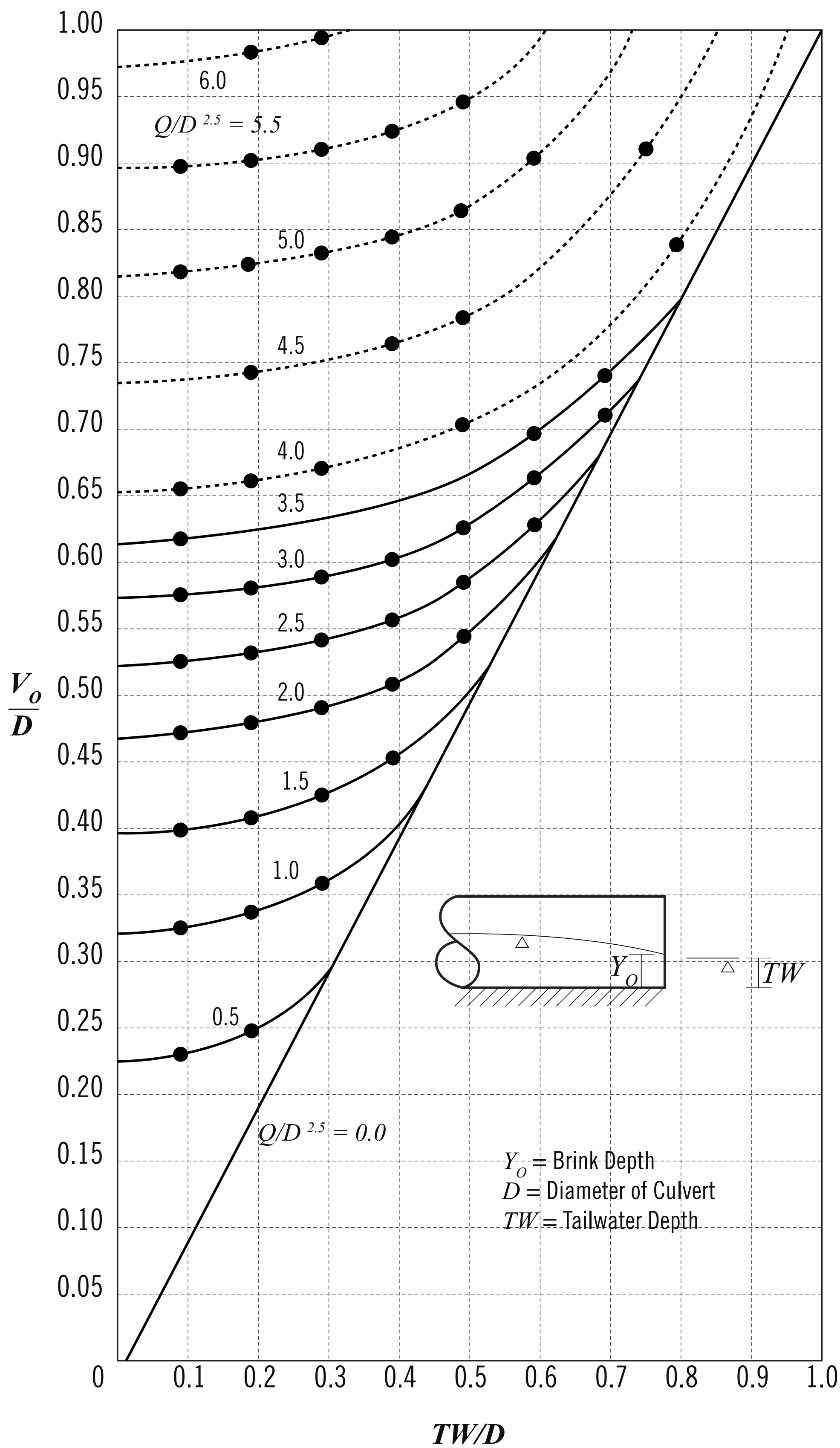


Figure 3.35 Dimensionless Rating Curves for the Outlets of Circular Culverts on Horizontal and Mild Slopes

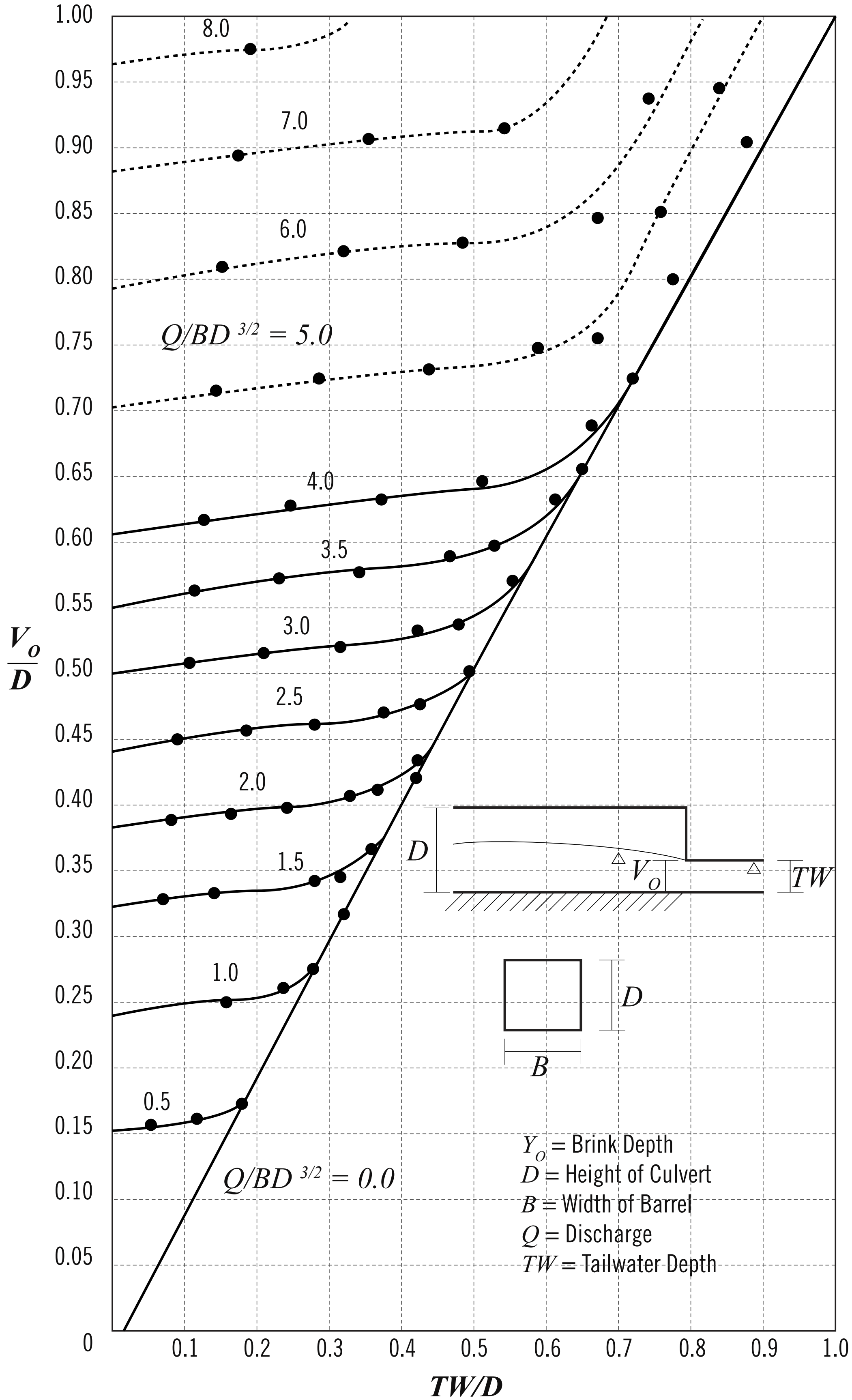
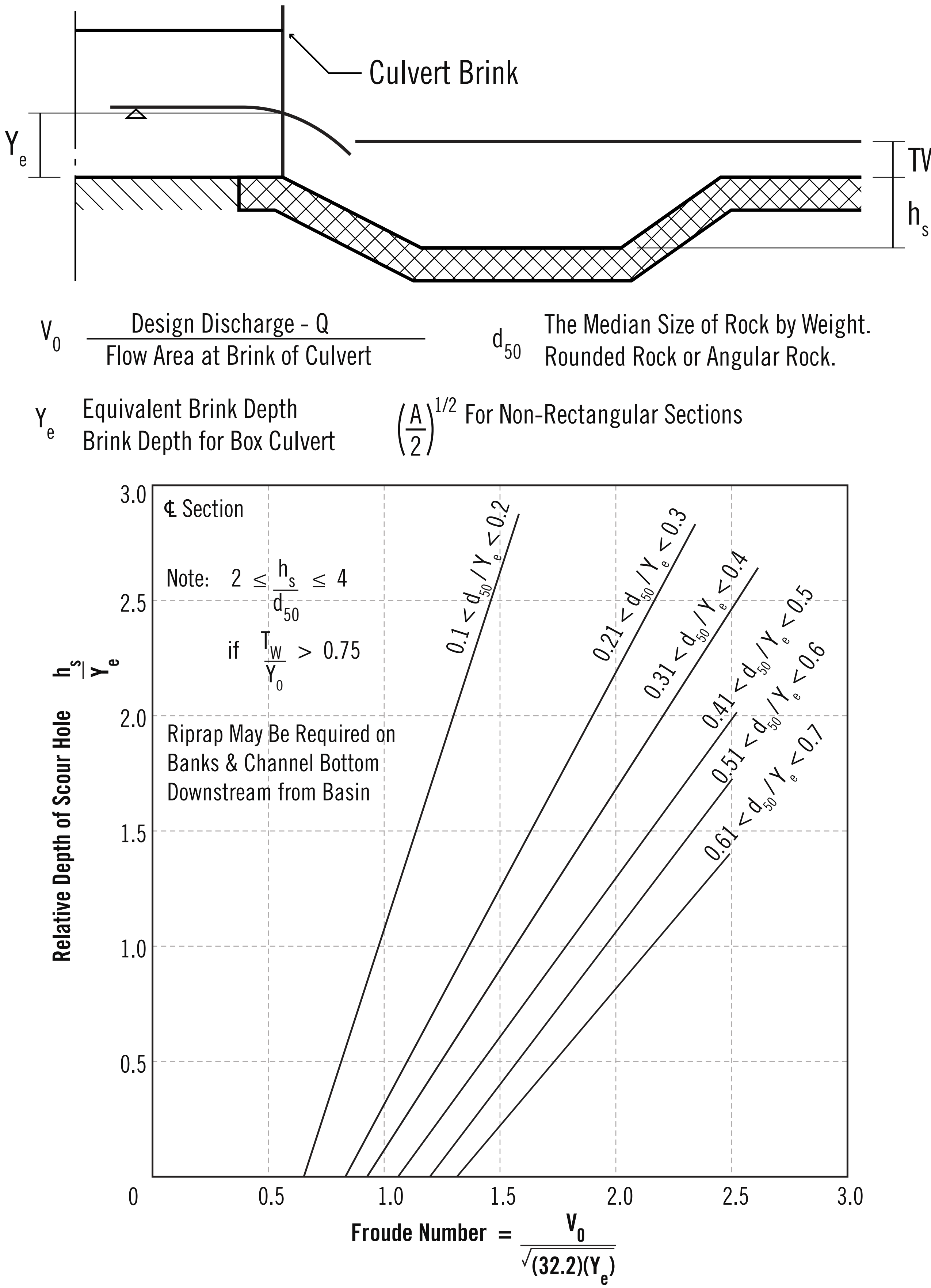


Figure 3.36 Relative Depth of Scour Hole Versus Froude Number at Brink of Culvert with Relative Size of Riprap as a Third Variable



A.7.8 Section 5 Figures

Figure 5.1 Hydraulic Jump

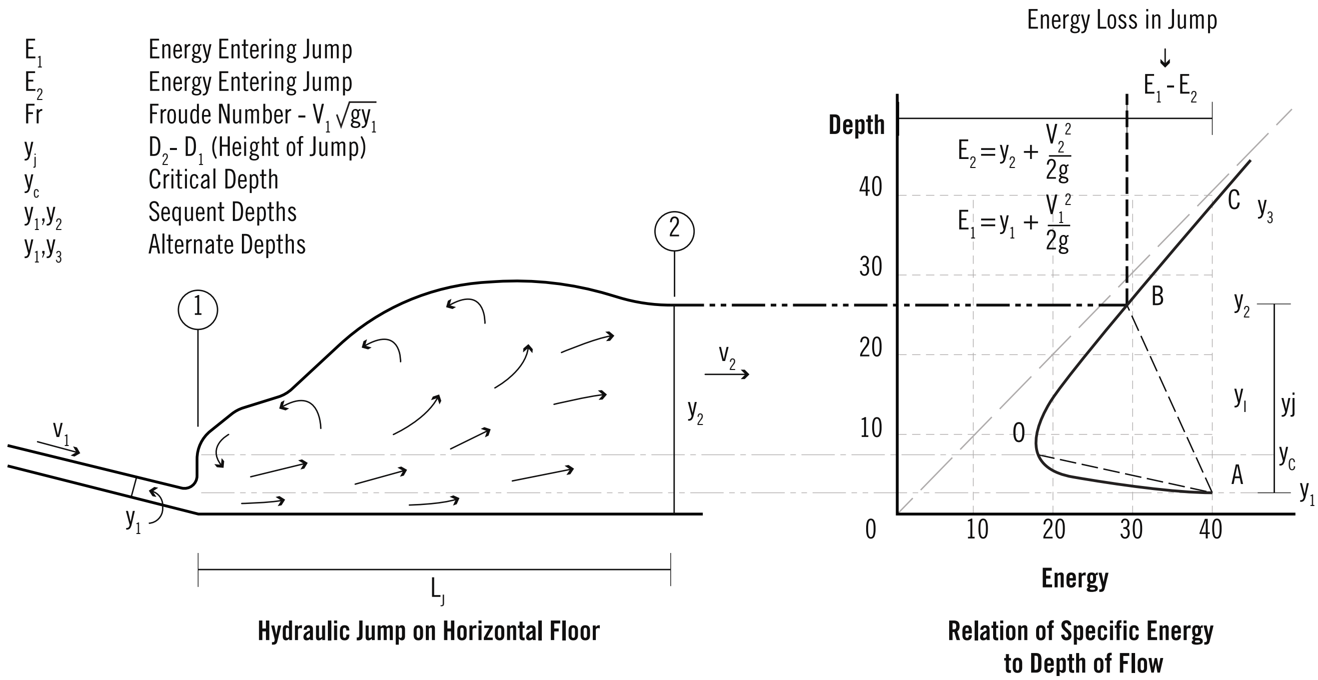


Figure 5.2 Hydraulic Jump Conjugate Depth – Horizontal, Rectangular Channel

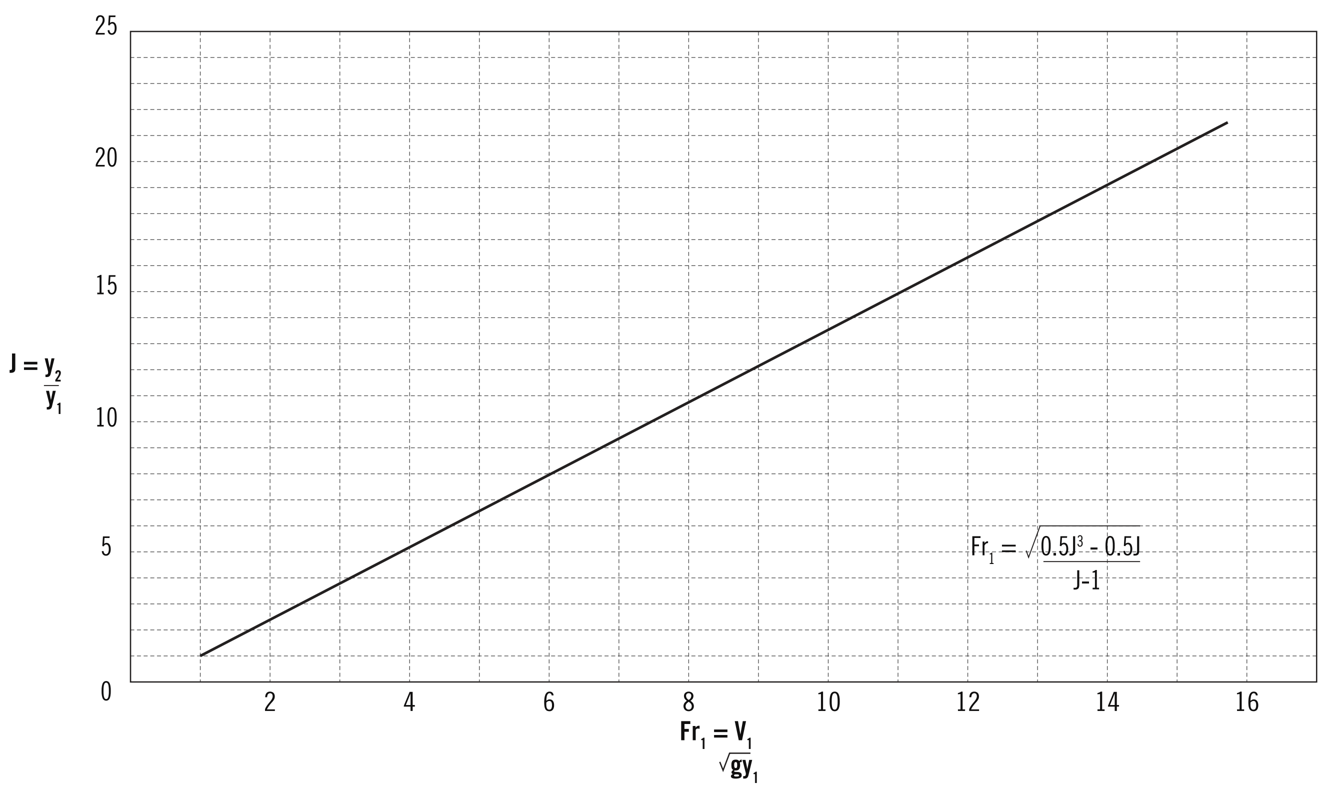


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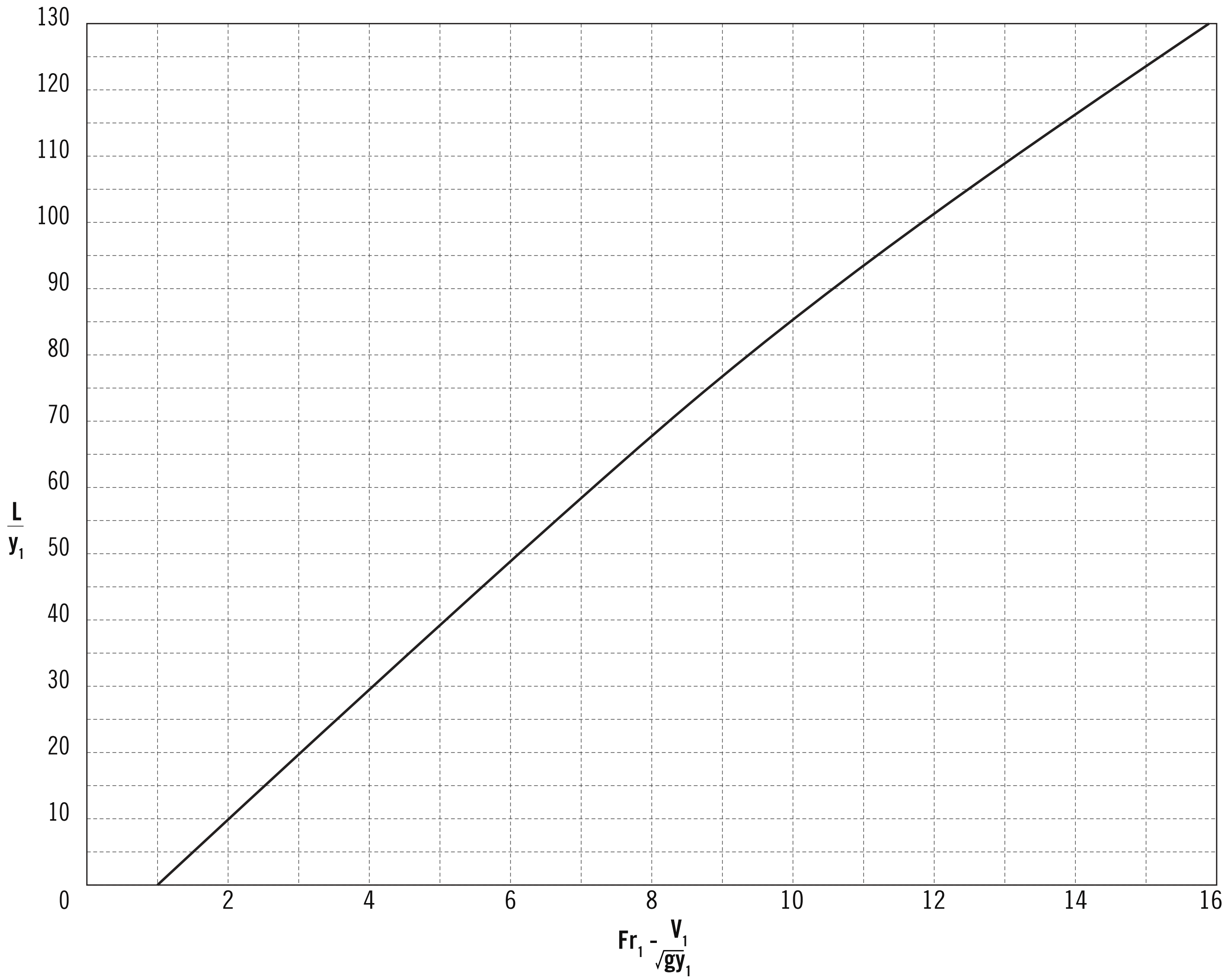
Figure 5.3 Length of Jump for a Rectangular Channel



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