DRAINAGE CRITERIA MANUAL

CITY OF CONWAY, ARKANSAS
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TABLE OF CONTENTS

SECTION 1. SUBMITTAL REQUIREMENTS
SECTION 2. GENERAL DESIGN CRITERIA
SECTION 3. HYDROLOGY
SECTION 4. STORM DRAINAGE SYSTEMS
SECTION 5. CULVERTS
SECTION 6. OPEN CHANNELS
SECTION 7. DETENTION

1.0 SUBMITTAL REQUIREMENTS

1.1 OVERVIEW

The purpose of this chapter is to provide a means to standardize the plans and drainage reports for proposed developments submitted to the City for review. In order to expedite City review of drainage reports and calculations, the documentation outlined below must be included in the submitted drainage report and/or plans. Calculations shall be stamped by a professional engineer licensed in the State of Arkansas.

1.2 PLAN REQUIREMENTS

All plans must include the requirements of the City of Conway Subdivision Regulations (Ordinance O-00-03) and the City of Conway Zoning Code (Ordinance O-17-91).

1.3 DRAINAGE REPORT REQUIREMENTS

The following items shall be included in the Drainage Report that accompanies each proposed development plan set submitted to the City.

- Project title and date
- Project location – include street address and vicinity map
- Project description – a brief description of the proposed project
- Project owner – name, address and telephone number
- Site area – to the nearest 0.1 acre
- Site drainage description – a brief description of the site drainage for the proposed project
- Area drainage problems – provide a description of any known on-site, downstream or upstream drainage/flooding problems
- Drainage area maps – pre- and post-development drainage area maps as well as inlet area maps with time of concentration flow paths and proposed/existing topography shown as appropriate
- Summary of runoff – provide a table with the 2, 10, 25, 50 and 100-year frequency storm events (existing and proposed conditions) for each discharge point
- Calculations – provide copies of calculations performed, including (as applicable):
    - Runoff flow calculations for the 2, 10, 25, 50 and 100-year frequency storm events (for both pre- and post-development conditions)
    - Time of concentration calculations
    - Runoff coefficients or curve numbers
    - Gutter flow calculation
    - Inlet calculations
    - Pipe or culvert calculations
    - Open-channel calculations including any flumes
    - Erosion control and scour calculations for pipe outlets and downstream channels
    - Detention calculations including
        - Pre- and post-development hydrographs

        - Basin sizing calculations
        - Outlet structure design with release rates for the 2, 10, 25, 50 and 100-year frequency storm events
        - Stage-storage and stage-discharge curves or tables
        - Routing calculations
- Hydraulic grade line calculations
  Recommendations / Summary – provide a description of any assumptions made in the calculations, proposed drainage improvements to be made to the site and the expected effects of the project
- Storm hydrographs, including inflow and outflow hydrographs for detention basins
- References for all formulas, coefficients, graphs, etc.
- Certification – all drainage reports shall be signed, sealed and dated by an engineer licensed in the state of Arkansas
- Operations and maintenance plan for any proposed stormwater detention facilities

1.4 APPROVED SOFTWARE

The following are approved computer applications for use in the City of Conway. Other software may be used with approval by the City Engineer. In all cases, a summary of the input data, computation worksheets, and final computed results must be provided with the submittal.

Hydrology

- U.S. Army Corps of Engineers Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS)
- Natural Resources Conservation Service - WinTR-55
- U.S. Environmental Protection Agency – Storm Water Management Model (SWMM)
- AutoDesk, Inc. - Hydraflow Hydrographs
- Bentley Syestems, Inc. - PondPack

Storm Drainage Systems

- Bentley Systems, Inc. - *StormCAD*
- Autodesk, Inc. - *Hydraflow Storm Sewers*
- Hydrology Studio - *Stormwater Studio*
- U.S. Environmental Protection Agency – *Storm Water Management Model (SWMM)*

Culverts

- Bentley Systems, Inc. - *CulvertMaster*
- Autodesk, Inc. - *Hydraflow Express*
- Hydrology Studio - *Culvert Studio*
- U.S. Federal Highway Administration - *HY-8 Culvert Hydraulic Analysis Program*
- U.S. Army Corps of Engineers Hydrologic Engineering Center - *River Analysis System (HEC-RAS)*

Open Channels

- Bentley Systems, Inc. - *FlowMaster*
- Autodesk, Inc. - *Hydraflow Express*
- Hydrology Studio - *Channel Studio*
- U.S. Army Corps of Engineers Hydrologic Engineering Center - *River Analysis System (HEC-RAS)*

Detention

- U.S. Army Corps of Engineers Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS)
- Bentley Systems, Inc. - *PondPack*
- Autodesk, Inc. - *Hydraflow*
- Hydrology Studio
- Hydrocad

1.5 AS-BUILT CERTIFICATIONS

Final as-built plans and a certification letter shall be submitted to the City's Planning Office upon completion of all work for the drainage improvements. The certification letter shall be signed by the engineer of record affirming that all improvements have been constructed in accordance with the as-built plans, which shall conform with the approved construction plans except for modifications approved through the city. All improvements must be in place and as-builts, certifications, one-year maintenance bond for 100% of the cost of drainage improvements and easements provided to the City Planner prior to Final Plat for a subdivision or issuance of the Certificate of Occupancy for a Large Scale Development. As-built plans shall be based on surveyed data of the constructed improvements. As-builts will be submitted in the following formats:

- An AutoCAD file, and
- One PDF copy of as-built plans and drainage report (signed, sealed and dated by the engineer of record)

2.0 GENERAL DESIGN CRITERIA

2.1 OVERVIEW

Hydraulic and structural designs must be such that minimal risks to traffic, property damage, and failure from floods are based on the results of good engineering practice and economics. The engineer of record may choose to design to a lower frequency occurrence if in his opinion a higher level of risks exists with respect to the potential for losses related to flooding. For projects within the City of Conway or otherwise subject to approval by the City, the following design storm events are the minimum acceptable criteria. For projects subject to special approval by the City (e.g, Planned Unit Developments, etc.), the City reserves the right to impose more stringent design criteria on the project.

2.1.1 General Requirements

All storm drainage systems shall be designed and constructed per the City of Conway Subdivision Regulations, the City of Conway Zoning Code, the City of Conway Design Standards Pattern Book, and the City of Conway Standard Details.

2.1.2 Construction

Construction shall meet the General Construction Requirements as listed on Sheet G-1, General Information, of the Standard Details and the City of Conway Subdivision Regulations. The Street Department shall be notified 24-hours prior to placement of storm drain pipe or drainage structures and shall approve subgrade prior to placement of materials. Storm drainage pipes, channels, ditches and drainage structures must be free of sediments, trash, debris and ponding water prior to final approval.

2.1.3 Design Storm Events

- Open channels conveying runoff from a drainage basin 5 acres or larger shall be designed to convey the 4% annual chance (25-yr) event with 1 ft of freeboard at any point in the channel. 1% annual chance (100-yr) event must be conveyed within a drainage easement or dedicated right-of-way (channel depth calculations, whether by hand calculation or computer modeling, shall account for backwater effects and not be based merely on normal depth calculations).
- Storm sewers (i.e., within developments where the drainage system will be dedicated to the City) shall be designed for a minimum of the 10% annual chance (10-yr) event with the 1% annual chance event conveyed within a dedicated drainage easement or dedicated right-of-way when conveying runoff from a drainage basin 5 acres or larger. Privately owned and maintained systems shall be designed to the same criteria. Where privately owned systems discharge into an existing storm drainage system, the designer shall demonstrate that the proposed improvements will not exceed the capacity of the existing system for the specified design event and shall provide such flow attenuation as may be required to prevent discharges from increasing downstream flooding.

- Roadway drainage systems (i.e., longitudinal drainage and crossings that are part of a subsurface system) shall be designed based on the street classification using the storm frequency criteria in Table 4.1. Consideration must be given to downstream system capacity (e.g., if a roadway drainage system is to be designed for a 4% or 2% annual chance event, a downstream storm sewer system being designed as part of the same project must to be designed for the same design event, even if it otherwise would be designed for a higher frequency event).
- Cross Drains/Culvert Design (i.e., road crossings that are direct crossings connecting open channels or streams) shall be designed for a minimum of a 4% annual chance event unless located in a Special Flood Hazard Area (SFHA) in which case the design storm shall be the 1% annual chance event.
- Detention pond outlets shall be designed to meet the requirements of Section 7.

3.0 HYDROLOGY

3.1 INTRODUCTION

This section provides an overview of the methods approved by the City for calculation of flows in the drainage design process. While the details of each method are not included in this section, key parameters have been identified and reasonable values for these parameters are listed. The information provided in this section is intended to provide the design professional with commonly applied and accepted standards for drainage design; however, additional methodologies may be accepted by the City with approval by the City Engineer. If proposing an alternate methodology or using a parameter that differs from the values listed in this section, the design professional should discuss the proposed method or parameter and provide justification for its use to the City Engineer for review and approval prior to preparing design computations.

3.2 METHODOLOGY

Multiple methods may be used to determine stormwater runoff quantity and timing; however, the method used should be selected based on the size of the area being studied and the intended use of the results. The three methods approved for use by the City of Conway are the Rational Method, the Modified Rational Method, and the SCS Method. Alternative methods may be submitted for consideration with approval being at the sole discretion of the City Engineer.

The rational method may only be used for drainage areas less than 100 acres for which only peak flow values are required.

For single basin drainage areas less than 25 acres for which detention is required, the Modified Rational Method may be used.

Hydrologic calculations for drainage areas over 25 acres, or that require hydrograph timing and downstream routing of flows through multiple basins, the SCS Method must be used. For large watersheds, drainage basins shall be subdivided so that an individual basin does not exceed 320 acres (1/2 square mile).

The hydraulic and structural designs must be such that minimal risks to traffic, property damage, and failure from floods are based on the results of good engineering practice and economics.

3.3 PRECIPITATION

Precipitation values are expressed in units of either rainfall intensity (inches per hour) or total rainfall depth (inches) for a given storm. The Rational Method utilizes rainfall intensity as presented in Table 3.1 – Rainfall Intensity-Duration-Frequency. The SCS Method utilizes rainfall depth as presented in Table 3.2 – Rainfall Depth-Duration-Frequency.

This precipitation data may be used to develop synthetic rainfall events for use in many common hydrologic modeling software packages. The synthetic event, or design storm, that shall be used for drainage calculations is the SCS Type II 24-hour rainfall distribution.

The data in these tables represents the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 Point Precipitation Frequency Estimates downloaded from the NOAA National Weather Service Precipitation Frequency Data Server (PFDS) for the Conway, AR (03-1596) station. Data is provided for the design storm events required for various drainage calculations required by this Manual, including 50%-, 10%-, 4%-, 1%-, and 0.2% annual chance events (2-year, 10-year, 25-year, 100-year, and 500-year events, respectively). Precipitation data for additional events can be obtained from NOAA Atlas 14.

Table 3.1. Rainfall Intensity-Duration-Frequency

Duration (min)	Percent Annual Chance Return Period (inches/hour)				
	50% (2-year)	10% (10-year)	4% (25-year)	1% (100-year)	0.2% (500-year)
5	5.86	7.88	9.12	11.00	13.10
10	4.29	5.77	6.68	8.05	9.61
15	3.49	4.69	5.43	6.54	7.81
30	2.54	3.43	3.96	4.76	5.66
60	1.67	2.24	2.59	3.15	3.82
120	1.03	1.38	1.60	1.96	2.40
180	0.77	1.03	1.20	1.49	1.86
360	0.47	0.64	0.75	0.95	1.21
720	0.29	0.40	0.47	0.60	0.78
1440	0.17	0.24	0.29	0.38	0.48

Source: NOAA Atlas 14

Table 3.2. Rainfall Depth-Duration-Frequency					
Duration (min)	Percent Annual Chance Return Period (inches)				
	50% (2-year)	10% (10-year)	4% (25-year)	1% (100-year)	0.2% (500-year)
5	0.49	0.66	0.76	0.92	1.09
10	0.72	0.96	1.11	1.34	1.60
15	0.87	1.17	1.36	1.64	1.95
30	1.27	1.72	1.98	2.38	2.83
60	1.67	2.24	2.59	3.15	3.82
120	2.06	2.76	3.21	3.92	4.81
180	2.31	3.09	3.62	4.48	5.59
360	2.82	3.81	4.51	5.69	7.27
720	3.44	4.76	5.69	7.27	9.39
1440	4.13	5.83	7.01	9.00	11.60

Source: NOAA Atlas 14

3.4 TIME OF CONCENTRATION

The time of concentration is the time required for water to flow from the most hydraulically-distant point of a drainage area to the discharge point under consideration (the longest flow path based on time of flow). This represents the time for the drainage system to accumulate all flows and achieve the peak flow rate for the drainage area. The time of concentration is computed in three distinct flow patterns: overland sheet flow, shallow concentrated flow, and channelized flow. All segments must be computed based on the methods and equations shown in NRCS TR-55: *Urban Hydrology for Small Watersheds*. Additional requirements for each flow pattern are described below. The minimum time of concentration for any individual watershed shall not be less than 6 minutes.

3.4.1 Overland Sheet Flow

The overland sheet flow equation is based on using flow length, slope, Manning's "n" roughness coefficient for sheet flow, and the 2-year, 24-hour rainfall depth. The flow length for this segment may not exceed 100 feet. Overland sheet flow lengths greater than 100 feet are unlikely in an urban setting and will result in excessively long flow travel times. Additionally, overland sheet flow should transition to shallow concentrated or channel flow where the longest flow path enters a swale, ditch, or gutter that is identifiable in the site topography (survey or LiDAR) or aerial imagery. Manning's "n" values for sheet flow are provided in Table 3.3 below. These differ from Manning's "n" values used for open channel calculations and are only valid for very shallow depths. The 2-year, 24-hour rainfall depth used for this method is 3.7 inches.
The overland flow time, t_o, may be calculated using the following equation:

$$t_o = \frac{0.42(n*L)^{0.8}}{(P_2)^{0.5}*S^{0.4}}$$

In which:

t_o = overland flow time (minutes)
n = Manning's roughness coefficient (refer to Section 6.4.2)
L = length of overland flow in feet
P_2 = 2-year, 24-hour rainfall (inches)
S = average basin slope (ft/ft)

Table 3.3. Manning's "n" roughness coefficients for sheet flow	
Surface Description	Manning's "n"
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils: Residue cover ≤20%	0.06
Cultivated soils: Residue cover >20%	0.17
Grass: Short grass prairie	0.15
Grass: Dense grasses	0.24
Grass: Bermudagrass	0.41
Range (natural)	0.13
Woods: Light underbrush	0.40
Woods: Dense underbrush	0.80

Source: NRCS TR-55

3.4.2 Shallow Concentrated Flow

The shallow concentrated flow equation is computed using flow length, slope, and the equation used varies based on surface type as presented in NRCS documentation.
If the shallow concentrated flow path traverses both paved and unpaved reaches, either the paved-surface equation must be used or the segment must be split into multiple reaches and computed independently.

The shallow concentrated flow time, t_s, may be calculated using the following equation:

$$t_s = \frac{L}{60*V}$$

In which:

t_s = shallow concentrated travel time (minutes)
L = flow length (feet)
V = average velocity (feet per second)

The average velocity for shallow concentrated flow may be calculated as follows:

$V =$ $20.3282*S^{0.5}$ (paved areas)
$V =$ $16.1345*S^{0.5}$ (unpaved areas)

Where:

S = slope (ft/ft)

3.4.3 Channelized Flow

The channelized flow equation is computed using flow length, slope, Manning's "n" roughness coefficient for open channel flow, and the geometry of the channel, expressed as the hydraulic radius. The hydraulic radius is the cross-sectional flow area of the channel or culvert divided by the wetted perimeter of the channel or culvert. These measurements are to be determined for bank-full conditions for natural channels, or an approximation of the two-year flow in designed channels and culverts.

The channelized flow time, t_t, may be calculated using the following equation:

$$t_t = \frac{L}{60*V}$$

In which:

t_t = channelized flow travel time (minutes)
L = flow length (feet)
V = average velocity (feet per second)

And where:

$$V = \frac{1.49}{n} * R^{2/3} * S^{1/2} \qquad \text{(Manning's Equation)}$$

In which:

V= average velocity (feet per second)
n = Manning's roughness coefficient (Table 3.3 and Section 6.4.2)
R = hydraulic radius (feet) and is equal to A/P_w
A = cross-sectional flow area (square-feet)
P_w = wetted perimeter (feet)
S = average channel slope (ft/ft)

3.5 SOIL TYPES

Soil type is an important variable in determining the potential runoff from a given surface. It is a factor in both hydrologic methods described below. One of four Hydrologic Soil Group (HSG) designations (A, B, C, or D) is defined for each soil type determines the infiltration potential for the area; with HSG A having the highest potential for infiltration and HSG D having the lowest potential for infiltration. Additionally, three dual groups (A/D, B/D, C/D) are defined, in which a soil's runoff potential is dependent on the presence of a water table within 24 inches of the surface. If dual groups are encountered on the proposed project site, the more conservative HSG D should be applied or justification for use of the alternate HSG should be provided to the City Engineer for review and approval prior to use for drainage computations. It should be noted that NRCS periodically updates various soil-type parameters, including Hydrologic Soil Group. If using a previously downloaded dataset, the site-specific soil types should be reviewed and verified on the NRSC Web Soil Survey prior to use for drainage computations (https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx).

3.6 RATIONAL METHOD

The Rational Method uses the drainage area (in acres), rainfall intensity (i), and a runoff coefficient "C" to determine the peak flow rate for a basin. This method assumes that the rainfall intensity is uniform over a time period equal to the time of concentration for the discharge point under consideration. For this reason, the Rational Method may not be used for detention design. After calculating the time of concentration for a drainage area, refer to the Intensity-Duration-Frequency Curves at the end of this section and select the intensity for the required design storm. A detailed description of the Rational Method can be found in numerous standard engineering texts.

3.6.1 Runoff Coefficient

The runoff coefficient relates ground cover, slope and soil type to determine runoff as a percentage of total precipitation. Values used shall conform to the values provided in Table 3.4. For basins with multiple surface types, a composite "C" shall be determined using an area-weighted average.

Table 3.4. Rational Method runoff coefficient “C”	
Type of Drainage Area	Runoff Coefficient, “C”
Business:	
Downtown areas	0.85
Neighborhood areas	0.60
Residential:	
Single-family (< 6,000 sq. ft.)	0.50
Single-family (> 6,000 sq. ft.)	0.40
Multi-units, detached	0.55
Multi-units, attached	0.70
Suburban	0.35
Apartment dwelling areas	0.65
Industrial:	
Light areas	0.65
Heavy areas	0.75
Parks, cemeteries	0.20
Playgrounds	0.30
Railroad yard areas	0.30
Unimproved areas (w/ vegetation)	0.30
Lawns:	
Sandy soil, flat, 2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Heavy soil, flat, 2%	0.15
Heavy soil, average, 2 - 7%	0.20
Heavy soil, steep, > 7%	0.30
Streets:	
Asphaltic	0.90
Concrete	0.90
Brick	0.80
Drives and walks	0.90
Roofs	0.90
Gravel	0.60
Forest	0.15

3.7 MODIFIED RATIONAL METHOD

The Modified Rational Method involves analyzing varying storm durations to determine which storm length will produce the maximum required detention volume. The equation used is the same as the standard Rational Method; however, the storm duration is increased for each calculation until the required detention volume decreases. If the Modified Rational Method is used, the incremental storm duration increase for each calculation shall be no greater than 5 minutes. The Modified Rational Method may be used in detention design for sites with a total drainage basin area less than 25 acres.

3.8 SCS TR-55 METHOD

The TR-55 method uses the drainage area, rainfall depth, time of concentration, and a runoff Curve Number (CN) to determine a peak flow rate for a basin. The tabular hydrograph method may also be applied to develop a runoff hydrograph for each drainage area; however, most hydrologic modeling software packages create the hydrograph automatically.

3.8.1 Curve Number

The CN relates ground cover, slope and soil type to determine runoff as a percentage of total precipitation, and may vary based on the Antecedent Moisture Condition (AMC) of the basin. AMC II, representing the average moisture condition, should be used for all design computations. The CN should generally conform to the range of values provided in Table 3.5, which presents common surfaces that will be encountered in urban areas. Values for agricultural areas and rangelands may be found in the NRCS TR-55 publication. For basins with multiple surface types, a composite CN should be determined using an area-weighted average.

3.8.2 Routing

Hydrologic routing is a method to attenuate flood hydrographs as they progress through downstream drainage basins. The peak flow is reduced based on the length of the river reach through the downstream basin, the channel and floodplain geometry and roughness, and potential storage either in the overbank or upstream of structures which significantly restrict flow (dams, roadway embankments with undersized drainage structures, etc.). The three methods approved for use are the Modified Puls Method, the Muskingum-Cunge method, and the Kinematic Wave Method.

The Modified Puls Method is applicable for most scenarios and includes the effects of backwater in the attenuation of the peak flow. This method requires development of a hydraulic model to calculate a storage-discharge relationship for the reach, and model cross sections should be positioned to represent significant changes in channel and floodplain geometry.

The Muskingum-Cunge Method is similar to the Modified Puls Method; however, it uses a simplified, representative channel cross section to approximate the entire routing reach. This method is applicable for man-made or channelized streams and includes the effects of backwater in the attenuation of the peak flow. It may not be applicable for natural streams with substantial geometric variation. This method is not applicable when significant backwater influence may influence the discharge, such as storage behind dams or roadway embankments.

The Kinematic Wave Method is only applicable when steady uniform flow can be assumed, such as in storm sewer systems. This method does not allow for attenuation due to storage, it is only a translation of the time to peak flow based on the travel time through the drainage basin.

Table 3.5. Runoff Curve Numbers for Urban Areas				
Cover type and hydrologic condition	CN for Hydrologic Soil Group			
	A	B	C	D
Open space:				
Poor condition (grass cover <50%)	68	79	86	89
Fair condition (grass cover 50% to 75%)	49	69	79	84
Good condition (grass cover >75%)	39	61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)	98	98	98	98
Streets and roads:				
Paved; curbs and storm sewers (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	89	92	94	95
Industrial	81	88	91	93
Residential districts by average lot size:				
1/8 acre or less (town houses)	77	85	90	92
1/4 acre	61	75	83	87
1/3 acre	57	72	81	86
1/2 acre	54	70	80	85
1 acre	51	68	79	84
2 acres	46	65	77	82
Newly graded areas (pervious areas only, no vegetation)	77	86	91	94
Pasture, grassland, or range:				
Poor condition (ground cover <50%, heavily grazed)				
Fair condition (ground cover 50% to 75%, not heavily	68	79	86	89
grazed)	49	69	79	84
Good condition (grass cover >75%, lightly grazed)	39	61	74	80
Meadow	30	58	71	78
Woods:				
Poor condition (forest litter, small trees, and brush are destroyed by heavy grazing or regular burning)	45	66	77	83
Fair condition (grazed but not burned, and some forest litter covers soil)	36	60	73	79
Good condition (protected from grazing, and litter and brush adequately cover soil)	30	55	70	77

Source: NRCS TR-55

4.0 STORM DRAINAGE SYSTEMS AND APPURTANENCES

4.1 OVERVIEW

Storm sewers shall be designed for the total intercepted flow based on the design event per Section 4.3.1. Where a portion of the storm sewer also serves as a culvert (cross drain) for a major intersection, major conveyance system or state route — that portion must be designed to accommodate the design flow from that drainage area. The placement and hydraulic capacities of storm drainage structures and conveyances should be designed to take into consideration damage to adjacent property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements, and available funds.

This chapter provides design criteria and guidance on storm drainage system components and appurtenances including street and roadway gutters, inlets and storm drain pipe systems. Other components such as culverts used as cross-drains, open channels, and energy dissipation devices for outlet protection are described elsewhere in this manual.

4.1.1 Definitions

Common definitions used in the analysis and design of storm drainage systems include:

- *Bypass* is flow which bypasses an inlet and is carried in the street or channel to the next inlet downgrade. This may be by design.
- A *Combination Inlet* is a drainage inlet composed of a curb-opening and a grate inlet.
- A *Curb-Opening Inlet* is a drainage inlet consisting of an opening in the roadway curb.
- A *Drop Inlet* or *Grate Inlet* is a drainage inlet with a horizontal or nearly horizontal opening, typically located in a low point, swale or channel and may utilize a grate.
- The *Energy Grade Line (EGL)* represents the total energy at any point along the storm drainage system.
- *Flanking inlets* are inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve to intercept debris and to act in relief of the central inlet.
- The *Gutter* is the portion of the roadway section adjacent to the curb which is utilized to convey stormwater. The gutter may include a portion or all of a lane or shoulder.
- The *Hydraulic Grade Line* (HGL) represents the depth to which water would rise in vertical tubes connected to the sides of a pipe barrel and in pipe junctions.
- *Inlet Efficiency* is the ratio of flow intercepted by an inlet to total flow in the gutter or channel.
- *Invert* refers to the inside bottom of the storm drainage pipe or structure.
- *Crown* refers to the outside top of the storm drainage pipe.
- *Soffit* refers to the inside top of the storm drainage pipe.
- *Spread* refers to the width of flow at a curb or drop inlet as measured laterally along the roadway curb.

4.1.2 References

The following documents are recommended for reference to provide background information, technical information and appropriate methodology for the analysis and design of storm drainage systems:

- *HEC-22, Urban Drainage Design Manual*, Federal Highway Administration, 3rd Edition, Revised 2013 or most recent edition.
- *AASHTO Highway Drainage Guidelines, Chapter 9,* AASHTO, 2007 or most recent edition.
- *Municipal Storm Water Management*, Debo, Thomas N. and Reese, Andrew J, CCR Press LLC, 1995.
- *Hydraulic Design Manual, Chapter 8,* Texas Department of Transportation, 2016 or most recent edition.
- *City of Conway Municipal Code*
- *Standard Details for Roadway and Drainage Construction,* City of Conway, Arkansas, Street and Engineering Department, 2018 or most recent edition.

4.2 GENERAL REQUIREMENTS

4.2.1 Surface Drainage Requirements

Surface, or pavement, drainage components including gutters and inlets shall be designed to limit the depth and spread of water in roadways and onto adjacent properties.

4.2.1.1 Gutters

Curb and gutters shall be constructed as described in the City of Conway Standard Details ST-1, Type "A" Curb & Gutter, ST-1A, 4" Curb & Gutter, and ST-2, Modified Curb & Gutter, and ST-4, Underdrain (where required). Gutter cross slopes shall match design pavement cross slopes. Lateral slopes shall match roadway profile. The minimum lateral gutter slope shall be 0.30%.

4.2.1.2 Inlets

The primary aim of drainage design is to limit the amount of water flowing along the gutters or ponding at the sags to quantities which will not interfere with the passage of traffic for the design frequency.

Grate inlets and depressions of curb inlets should be located outside the through-traffic lanes. Inlet grates should safely accommodate bicycle and pedestrian traffic where appropriate.

Inlets should be spaced based upon inlet capacity and width of spread. Special consideration shall be taken where size and location may affect access ramps, pedestrian crossings, speed bumps or other special features.

Curb inlets shall be constructed as described in the City of Conway Standard Details D-1, Typical Curb Inlet, and D-5.

4.2.2 Subsurface Drainage System Requirements

4.2.2.1 Pipe Materials

Dependent on location, acceptable materials may include RCP, polymer-coated corrugated metal pipe (PCMP) and corrugated HDPE pipe. Box culverts may be either pre-cast concrete or cast-in-place concrete. All storm drainage pipe located under city streets or within city right-of-way under driving surfaces shall be reinforced concrete pipe (RCP) Class III unless approved otherwise by the City Engineer. Corrugated HDPE pipe shall be allowed under private concrete driveways and behind curb and gutter. Unless otherwise approved by the City Engineer, gasketed HDPE pipe shall be required in all public drainage easements in areas outside of the roadway right-of-way. Material selected must structurally and hydraulically support the culvert application. Material selection should additionally consider replacement cost, difficulty of construction and economic factors including life expectancy.

4.2.2.2 Pipe Shapes

Allowable shapes include circular, arch, elliptical and box.

4.2.2.3 Minimum Sizes

Storm drainage pipes shall not be less than 15 inches in diameter or height at the minimum internal dimension. Potential for blockage by debris from upstream drainage areas shall be considered in selection of the pipe size and access spacing.

4.2.2.4 Cover and Bedding Requirements

Storm drainage pipes shall be constructed with sufficient bedding and cover to ensure structural integrity and to prevent failure during construction and service lifetime. A minimum of 1 foot of cover is required. Furthermore, the crown of the pipe (including the bell) may not protrude into the base material of the roadway pavement section.

Storm drainage systems shall provide 1-foot minimum clearance at utility crossings and 2-foot minimum clearance when parallel to utilities unless otherwise approved by the City Engineer. Electrical utilities and natural gas lines should never come in direct contact with storm drainage systems.

Storm drainage pipe shall be installed and bedded as described in the City of Conway Standard Details D-3 and D-4.

4.2.2.5 Junction Boxes

Junction boxes shall be sized and constructed per the City of Conway Standard Details D-2 and D-2A.

All underground piping systems shall conform to the requirements of Section 5.

4.2.3 Construction

Refer to Section 2.1.2.

4.3 DESIGN CRITERIA

The capacity of all storm drainage systems shall be determined by using an approach giving due consideration to rainfall intensity, soil characteristics, proper runoff coefficients, slope, and the hydraulic properties of the pipes and conduits used.

4.3.1 Design Storm Frequency

The design storm frequency for storm drain systems associated with streets and roads shall be selected based upon the following:

- Arterial – 2% annual chance occurrence (50-yr)
- Collector – 4% annual chance occurrence (25-yr)
- Local – 10% annual chance occurrence (10-yr)

The City Engineer may modify these requirements for a project based upon upstream and downstream structures or drainage systems, potential for flooding, safety, potential for future development, or other issues.

The system must be checked for the 100-year (1% chance occurrence) storm event for drainage basins over 5 acres in size.

4.3.2 Inlet Spread

Inlets shall be spaced at low points and at such intervals to provide the appropriate clear traffic lane per street classification in each direction during the design storm as defined below:

- Major and minor arterial streets shall be designed so that a minimum of one traffic lane, independent of the curb and gutter, is provided in each direction during the peak flows of the design storm.
- Collector streets shall be designed so that a minimum of one 9’ wide traffic lane, independent of the curb and gutter, is provided in each direction during the peak flows of the design storm.
- Local streets shall be designed such that a minimum 8-ft lane in the center of the street will remain clear during peak runoff from the design storm. The centerline of the clear lane shall be within 4-ft of the roadway centerline.

Table 4.1. Roadway Design Storm Criteria		
Description	Minimum Overtopping Design Storm Frequency	Minimum Open Roadway Width During a 10-year Frequency Storm Event
Trails	2-year	N/A
Local	10-year	Min. 8 feet
Collector	25-year	Min. 9 feet each direction
Arterial	50-year	Min. 11 feet each direction

4.3.3 Velocity Limitations

Minimum and maximum pipe flow velocities must be analyzed during design of the storm drainage system. Minimum velocity shall not be less than 2.5 feet per second when the pipe is partially full to minimize deposition of sediment and debris in the drainage system pipe during partial flow. Maximum velocity shall not exceed 10 feet per second. Outlet protection may be required where storm drainage systems discharge to open channels or impoundments.

4.3.4 Slope and Hydraulic Gradient

Slope shall be selected such that the drainage system pipe meets the velocity limitations listed above when pipes are flowing full. The minimum slope of any pipe shall be 0.3%.

4.3.5 Flow Conditions

Storm drainage systems should generally be designed as gravity systems for the design storm event.

4.3.6 Friction Losses

Friction, entrance and exit losses should be considered in the analysis and design of storm drainage systems. Expansion, contraction, bend and junction losses should be considered in complex systems or where there is the potential to flood offsite areas or create hazardous conditions. When hydraulic calculations do not consider minor energy losses such as expansion, contraction, bend, junction, and manhole losses, the elevation of the hydraulic gradient for design flood conditions should be at least 1.0 foot below ground elevation/gutter line at any point in the profile. As a general rule, minor losses should be considered when the velocity exceeds 6 feet per second (lower if flooding could cause critical problems). If all minor energy losses are accounted for, it is usually acceptable for the hydraulic gradient to reach the gutter elevation.

DESIGN PROCEDURES

The hydraulic design procedure for storm drainage systems should generally be the use of computer. However, hand calculations are acceptable but must include all of the same calculations generated by typical software packages. Refer to Section 1.4 for a list of approved computer applications.

5.0 CULVERTS

5.1 OVERVIEW

For the purposes of this manual, a culvert is defined as a structure used to convey stormwater through embankments or beneath roadways and may be designed to have a submerged inlet to increase hydraulic capacity where conditions permit. In addition to meeting hydraulic requirements, a culvert is typically covered and surrounded by structural material, designed to support the embankment or roadway. For the purposes of this section, "roadway" may refer to a road, driveway or pedestrian pathway.

The hydraulic and structural designs must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics.

5.1.1 Definitions

- The *Energy Grade Line (EGL)* represents the total energy at any point along a culvert barrel.
- *Headwater (HW)* refers to the depth of water at the culvert inlet.
- *The Hydraulic Grade Line (HGL)* represents the depth to which water would rise in vertical tubes connected to the sides of a culvert barrel.
- *Inlet Control* occurs when water can flow through and out of a culvert faster than it can enter it.
- *Invert* refers to the inside bottom of the culvert.
- *Outlet Control* occurs when water can flow into a culvert faster than it can flow through and out of it.
- *Crown* refers to the outside top of the culvert.
- *Soffit* refers to the inside top of the culvert.
- *Tailwater* refers to the depth of water at the culvert outlet.

5.1.2 References

The following documents are recommended for reference to provide background information on the hydraulics of culverts and technical information and appropriate methodology for the analysis and design of culverts:

- *HDS-7, Hydraulic Design of Safe Bridges*, Federal Highway Administration, 2012 or most recent edition.
- *HDS-5, Hydraulic Design of Highway Culverts*, Federal Highway Administration, 3rd Edition, 2012 or most recent edition.
- *AASHTO Highway Drainage Guidelines, Chapter 4 and Chapter 7*
- *Municipal Storm Water Management*, Debo, Thomas N. and Reese, Andrew J, CCR Press LLC, 1995.
- *Hydraulic Design Manual, Chapter 8,* Texas Department of Transportation, 2016 or most recent edition.
- *Standard Details for Roadway and Drainage Construction,* City of Conway, Arkansas, Street and Engineering Department, 2018 or most recent edition.

5.2 GENERAL REQUIREMENTS

5.2.1 Applications

Culverts may be utilized as stormwater cross drains through embankments and beneath roadways, driveways and pedestrian pathways, and, when appropriate, as outlets for stormwater detention and retention structures.

5.2.2 Culvert Selection

5.2.2.1 Culvert Materials

Refer to Section 4.2.2.1.

5.2.2.2 Culvert Shapes

Refer to Section 4.2.2.2.

5.2.2.3 Multiple Barrels

Multiple barrels shall be composed of common length, shape, and size and shall be constructed in parallel. Variances may be allowed with sufficient justification. Where practicable, multiple barrel culverts should fit within the natural existing channel without significant widening to maintain required velocities in each barrel to prevent sediment buildup. Adequate separation between the pipe runs must be provided to allow proper compaction of backfill / bedding material and meet manufacturer's minimum recommendations. In no case shall the separation between the exterior surfaces of the pipes be less than 1'-0".

Where fish passage is required, multiple barrel culverts should be avoided, if possible, or designed such that adequate low flows are provided for passage.

5.2.2.4 Minimum Sizes

Culverts shall not be less than 15 inches for roadways and 12 inches for private driveways in diameter or height at the minimum internal dimension. Potential for blockage by debris from upstream drainage areas shall be considered in selection of the culvert size, especially where upstream drainage area is steep, culverts are under high fill, or where clean out access is limited.

5.2.3 Culvert Skews

Culverts may be skewed up to an angle of 45 degrees as measured from a line perpendicular to the embankment or roadway centerline to match the channel alignment. Entrances and exits shall be beveled or, in the case of multiple-barrel culverts, offset to be parallel to the centerline or edge of the embankment or roadway.

5.2.4 Cover and Bedding Requirements

Refer to Section 4.2.2.4.

5.2.5 Excavation

Excavations for installation of culverts shall be performed with safety as the primary objective, and shall comply with all OSHA requirements. Excavations shall additionally be designed to minimize cost and disruption to existing traffic. Backfill shall be selected and compacted, at a minimum, to meet the requirements of the City's *Standard Details for Roadway and Drainage Construction*.

5.2.6 Erosion

Culverts shall be analyzed for potential to create streambed and streambank scour or erosion downstream of the culvert outlet. Structures including concrete aprons, energy dissipators, scour holes and other structures may be used to prevent downstream channel scour and erosion and to prevent undercutting and failure of the culvert.

Potential for erosion of the embankment slopes at the culvert entrance and outlet due to high approach or exit velocities, runoff from adjacent areas and rising and falling water levels shall also be considered in the inlet and outlet design. Utilize headwalls, wingwalls or slope protection to prevent erosion. Mitering culverts to match the embankment slope will not be considered sufficient to prevent erosion without additional protective measures.

5.2.7 Construction

Refer to Section 2.1.2.

5.3 DESIGN CRITERIA

5.3.1 Design Storm Frequency

Refer to Section 4.3.1 for design storm frequency requirements.

5.3.2 Headwater Depth

For a given design discharge there will be a corresponding headwater depth. The headwater depth is critical in determining the performance of the culvert, usually required to force flow through the culvert. Allowable design headwater elevation is restricted by the potential for damage to adjacent property, damage to the culvert and the roadway, traffic interruption, hazard to human life, and damage to stream and floodplain environment. Potential damage to adjacent property or inconvenience to owners should be of primary concern in the design of all culverts. Other possible critical elevations on the roadway itself include edge of pavement, sub-grade crown, and top of headwall. Upstream flow shall be contained within the drainage easement. Additionally, the headwater depth must meet the following requirements.

5.3.3 Overtopping

Overtopping of the roadway or flooding of any portion of the roadway shall not be allowed for the design storm frequency. Headwater elevation and potential for overtopping shall be determined for the 100-year (1% chance-occurrence) event to determine potential extents of flooding in flood prone areas at the request of the City Engineer.

5.3.4 Upstream Flooding

Upstream flooding for the design storm event shall meet freeboard requirements for the design storm and shall be limited to the extents of the drainage easement. It shall also not cause damage to adjacent property or affect upstream drainage structures.

5.3.5 Tailwater Depth

Tailwater depth is critical in determining the performance of the culvert for the design storm event. Tailwater depth shall not be assumed to be zero and shall be determined by analyzing the downstream channel and downstream drainage structures, where appropriate, to determine flow depth at the design discharge. In Special Flood Hazard Areas, where water surface profiles are available for the design storm, the tailwater shall be based on the water surface elevation (WSEL) published in the Flood Insurance Study (FIS). When not available, designer shall provide tailwater depth analysis.

Similar to headwater depth, tailwater depth shall not be allowed to damage downstream property.

5.3.6 Velocity Limitations

Refer to Section 4.3.3 for velocity limitations in culverts.

5.3.7 Length and Slope

Existing structures shall not be extended without first determining the effects on the performance of the structure. Where possible, the length and slope of the culvert shall match the existing topography and inverts should match the channel bottom and skew angle of the stream.

5.3.8 Culvert End Treatments

Headwalls, wingwalls, or flared end sections shall be used to increase the efficiency of the culvert, provide stability to the embankment, protect against erosion and to shorten the required length of the culvert. Headwalls are required for all culverts where buoyancy protection is necessary.

If special inlet conditions are assumed in the culvert analysis (e.g., skew angles, beveled or chamfered inlets, etc.) that affect entrance loss coefficients, the designer shall include such details on design documents/drawings.

5.4 DESIGN PROCEDURES

5.4.1 Types of Flow Control

There are two types of flow conditions for culverts that are based upon the location of the control section and the critical flow depth:

Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. This typically happens when a culvert is operating on a steep slope. The control section of a culvert is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions.

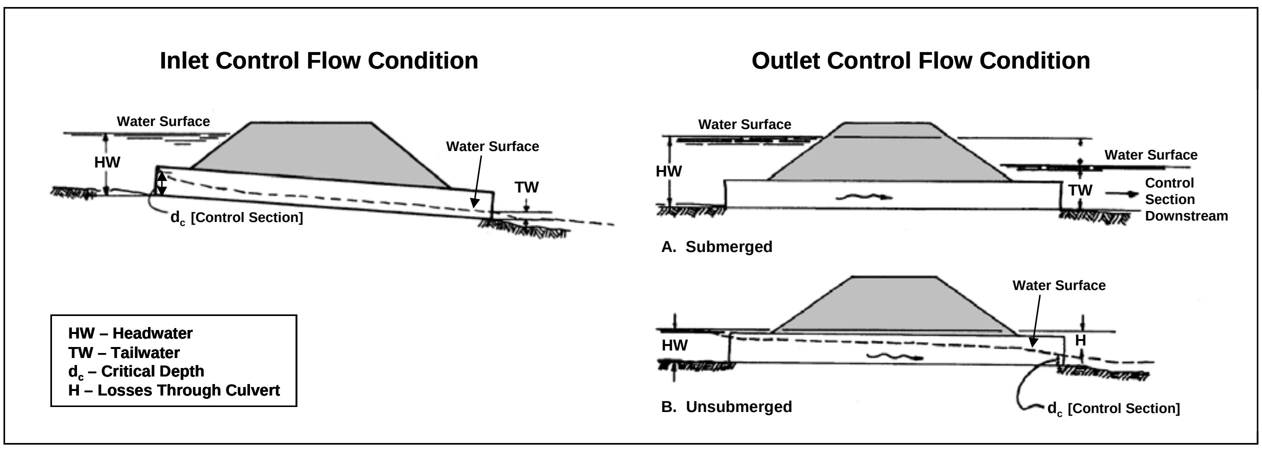

Figure 5.1. Culvert flow conditions
(Adapted from: HDS-5, 1985).

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see the FHWA Hydraulic Design of Highway Culverts, HDS-5, 1985.

5.4.2 Computer Applications

Refer to Section 1.4 for a list of approved computer applications.

5.4.3 Design Procedure

The following describes the general design procedure.

(Step 1) Determine required design data:

- Q = discharge (cfs)
- L = culvert length (ft)

- S = culvert slope (ft/ft)
- TW= tailwater depth (ft)
- V = velocity for trial diameter (ft/s)
- K_e= inlet loss coefficient
- HW = allowable headwater depth for the design storm (ft)

(Step 2) Determine trial culvert size by assuming a trial velocity.

(Step 3) Find the actual *HW* for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control data into software with *D* and *Q* and find *HW/D* for the proper entrance type.
- Compute *HW* and, if too large or too small, try another culvert size before computing *HW* for outlet control.
- For outlet control enter the outlet control data into software with the culvert length, entrance loss coefficient, and trial culvert diameter.
- Compute the headwater elevation, *HW*.

(Step 4) Compare the computed headwaters and use the higher *HW* to determine if the culvert is under inlet or outlet control.

- If inlet control governs, then the design is complete and no further analysis is required.
- If outlet control governs and the *HW* is unacceptable, select a larger trial size and find another *HW* with the outlet control. Since the smaller size of culvert had been selected for allowable HW by the inlet control, the inlet control for the larger pipe need not be checked.

(Step 5) Calculate exit velocity and if erosion problems might be expected, design appropriate energy dissipation.

5.4.4 Performance Curves - Roadway Overtopping

To complete the culvert design, roadway overtopping must be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. The performance curve for a culvert specifies the relationship between the headwater and culvert barrel discharge. In the development of performance curves, data from unsubmerged and submerged inlet control conditions are often evaluated separately. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

(Step 1) Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

(Step 2) Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate.

(Step 3) Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

5.5 ENERGY DISSIPATION DESIGN

The outlets of pipes and lined channels are points of critical erosion potential. Stormwater that is transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at stormwater outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity.

5.5.1 General Criteria

- Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.
- Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream area channel system (refer to section 6.3.4).
- Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

5.5.2 Recommended Energy Dissipators

For many designs, the use of a riprap apron, riprap basin or-baffled outlet provides sufficient protection at a reasonable cost. This section focuses on the design of these measures. Refer to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of Energy Dissipators for Culverts and Channels, for the design procedures of other energy dissipators.

5.5.3 Design Guidelines

Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur. Downstream evaluation of channel should be conducted on a case by case basis.

5.5.4 Baffled Outlets

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 6.4. This type of outlet protection has been used with outlet velocities up to 50 feet per second. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

5.5.4.1 Design Procedure

The following general design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 5.2 should be calculated as follows:

(Step 1) Determine input parameters, including:
h = Energy head to be dissipated (ft), can be approximated as the difference between channel invert elevations at the inlet and outlet
Q = Design discharge (cfs)
v = Theoretical velocity (ft/s = 2gh)
A = Q/v = Flow area (ft^2)
d = $A^{0.5}$= Representative flow depth entering the basin (ft), assumes square jet
Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

(Step 2) Calculate the minimum basin width, W, W, in ft, using the following equation.

$W/d = 2.88Fr^{0.566}$ or $W = 2.88dFr^{0.566}$

Where: W = minimum basin width (ft)
d = depth of incoming flow (ft)
Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

The limits of the W/d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

(Step 3) Calculate the other basin dimensions as shown in Figure 6.4, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

(Step 4) Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum), and depth f (W/6). The side slopes should be 1.5H:1V, and median rock diameter should be at least W/20.

(Step 5) Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

(Step 6) Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft/s flowing full.

(Step 7) If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.

(Step 8) If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent approximately 1/6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

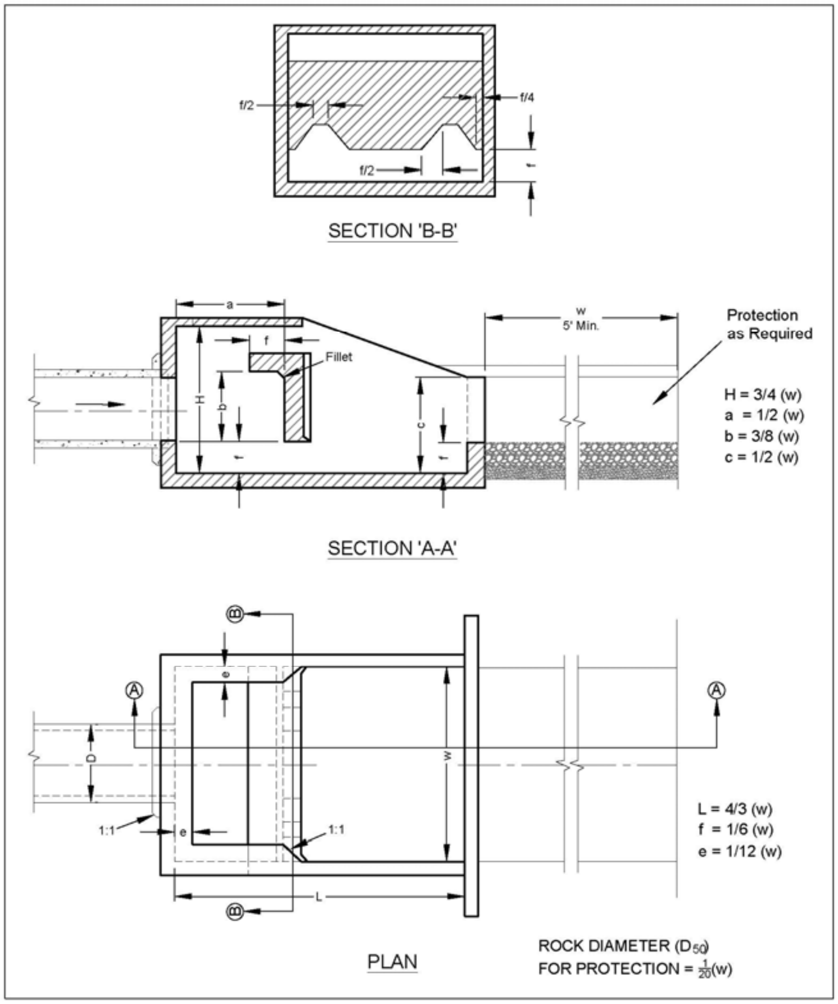

Figure 5.2. Schematic of Baffled Outlet
(Source: U.S. Dept. of the Interior, 1978).

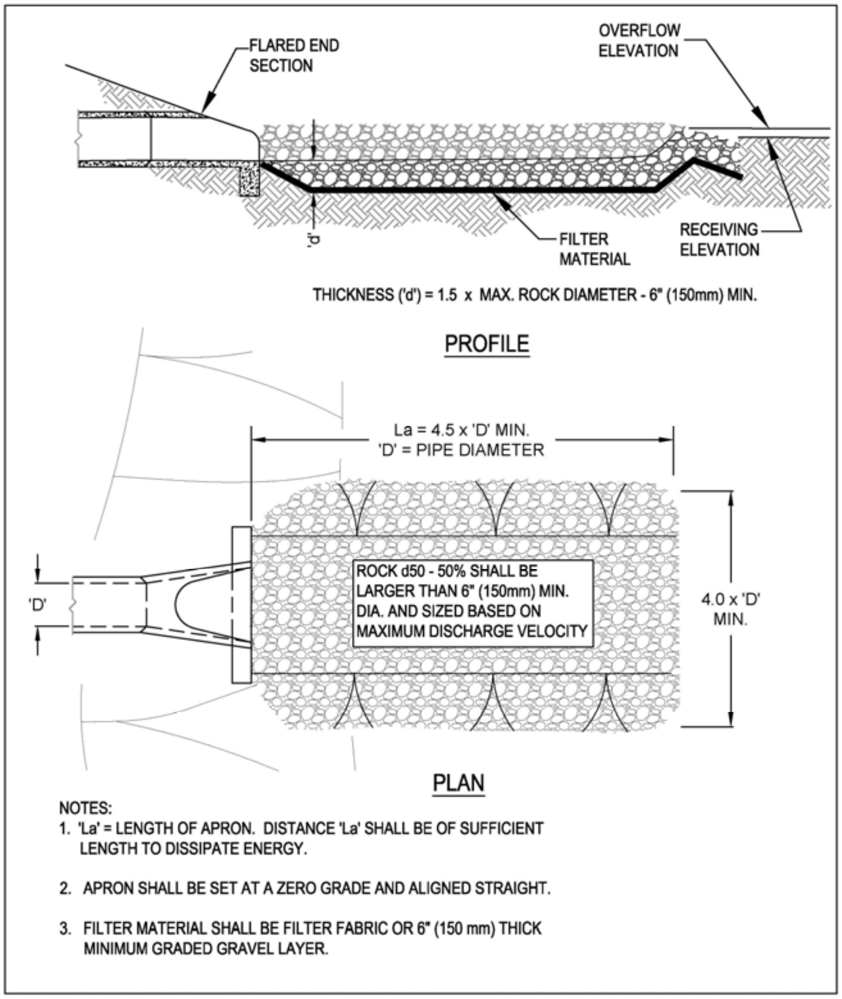

Figure 5.3. Storm drain outlet protection.

6.0 OPEN CHANNELS

6.1 OVERVIEW

Open channels include drainage ditches, grass channels, dry and wet swales, riprap (or other hard armor) lined channels and concrete-lined channels. Open channel hydraulics in urban environments may include design of artificial channels, improvements to existing channels or analysis of natural channels.

Storm water runoff from projects should not discharge onto adjacent property except at appropriate outlet points. Thus, the purpose of open channels is to convey storm water runoff from, through, and/or around facilities without damage to the facilities or adjacent property.

6.1.1 Definitions

- *Critical flow* occurs when the variation of specific energy with depth at a constant discharge is at a minimum. The depth of flow at which this occurs is the *critical depth* at which the Froude number has a value of 1.
- The *Energy Grade Line (EGL)* represents the total energy at any point along a channel alignment.
- A *hydraulic jump* occurs at abrupt transitions from flow below critical flow, or *subcritical flow*, and flow above critical flow, or *supercritical flow*. Hydraulic jumps result in energy dissipation due to significant changes in depth and velocity and may be employed to dissipate energy.
- Channel *invert,* or *flowline,* refers to the lowest point in a channel cross section.
- *Steady flow* occurs when the flow is constant, does not vary with time. It may further be categorized as:
  - *Uniform flow*, where the channel cross-section, roughness and slope are constant and, as a result, the EGL is parallel to the flow line of the channel, or
  - *Non-Uniform flow, or varied flow,* where channel properties vary along the channel alignment and the water surface is not parallel to the channel flow line. Varied flow may be gradually varied or rapidly varied depending upon the rate of change in the channel characteristics.
- *Unsteady Flow* occurs when flow varies with time, as opposed to steady flow.

6.1.2 References

The following documents are recommended for reference to provide background information on the hydraulics of culverts and technical information and appropriate methodology for the analysis and design of open channels:

- *Highway Drainage Guidelines, Chapter 10,* AASHTO, 2007 or most recent edition.
- *HEC-15, Design of Roadside Channels with Flexible Linings*, Federal Highway Administration (FHWA), September 2005 or most recent edition.
- *HDS-4, Introduction to Highway Hydraulics*, FHWA, June 2008 or most recent edition.
- *HDS-7, Hydraulic Design of Safe Bridges*, FHWA, 2012 or most recent edition.
- *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS 84-204, 1984

- *Hydraulic Design Manual, Chapter 8,* Texas Department of Transportation, 2016 or most recent edition.
- *Municipal Storm Water Management*, Debo, Thomas N. and Reese, Andrew J, CCR Press LLC, 1995.
- *Standard Details for Roadway and Drainage Construction,* City of Conway, Arkansas, Street and Engineering Department, 2018 or most recent edition.
- *Stormwater Management Manual, Volume 2, Chapter 3.0,* City of Memphis Division of Public Works and Engineering and the Shelby County Public Works Department, Ensafe Inc., June 2006 or most recent edition.

6.1.3 Considerations for Use of Open Channels

The ideal channel is that carved by natural drainage processes over a long period of time. The benefits of such a channel are lower velocities and more stable channel bottom and banks, channel and overbank storage reducing peak flows, decreased maintenance associated with stability, and retention of desirable green belt area. Generally, the natural channel or the man-made channel that most nearly conforms to the character of the natural channel is the most desirable.

In many areas experiencing development, the runoff has been so minimal that natural channels do not exist. However, a small trickle path nearly always exists that provides an excellent basis for location and construction of channels to reduce development costs and minimize drainage problems.

Channel stability is a recognized problem in urban hydrology because of general increases in low flows and peak storm discharges. A natural channel with increased capacity demands due to development should be augmented with necessary measures to avoid future bottom scour and bank cutting.

Sufficient right-of-way or permanent easement shall be provided adjacent to open channels (beyond the top bank) to allow entry of City maintenance vehicles. These easements shall be 10-foot minimum or channel width (top of bank to top of bank) plus 5 feet.

6.1.4 Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block. Flexible and vegetated linings will be allowed only with express permission from the City Engineer.

6.1.4.1 Vegetated Lining

<u>Vegetation</u>, where practical, is the preferred lining for existing waterways, and the developer is encouraged to preserve natural waterways where practical. If the capacity of the existing waterway in inadequate and the size of the existing

stream channel must be enlarged, a rigid channel lining will be required, or the developer may submit a detailed stream restoration plan prepared by a licensed landscape architect. In any case, hydraulic design of the channel must be prepared by a licensed engineer.

Conditions under which vegetation may not be acceptable include but are not limited to:

- High velocities,
- Standing water,
- Lack of maintenance needed to prevent growth of taller or woody vegetation,
- Lack of nutrients and inadequate topsoil,
- Lack of access for maintenance, and
- Excessive shade

Proper seeding, mulching and soil preparation are required during construction to assure establishment of healthy vegetation. Also, erosion control matting or other geofabrics may be required to be placed along the base and / or side slopes of these channels to allow establishment of vegetation.

6.1.4.2 Flexible Lining

Rock riprap, including rubble, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. The use of a filter fabric or stone filter is required to protect the underlying soils. The use of riprap lining is typically not allowed and must be approved by the City Engineer.

6.1.4.3 Rigid Lining

Rigid linings are the preferred alternative for man-made channels, and are generally constructed of articulated block or concrete and required for new channels or where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel head-cutting. Filter fabric may be required to prevent soil loss through pavement cracks.

6.2 GENERAL REQUIREMENTS

6.2.1 Allowable Applications

Open channels may be used in lieu of storm sewer systems to convey storm runoff where: sufficient right-of-way is available, sufficient cover for storm sewers is not available, it is important to maintain compatibility with existing or proposed developments, and economy of construction can be shown without excessive long-term public maintenance expenditures.

Intermittent alternating reaches of opened and closed systems should be avoided.

6.2.2 Channel Geometry

Trapezoidal cross sections are preferred over v-shaped channels.

Channels with bottom widths greater than 10 feet shall be designed with a minimum bottom cross slope of 2%.

Channel side slopes shall be stable throughout the entire length and shall consider the channel material. A maximum of 3H:1V is allowed for vegetated slopes and 2:1 for rigid and flexible lined slopes, unless otherwise justified by calculations. Refer to standard details D-6 and D-7.

Low-flow sections shall be considered in the design of channels with large cross sections (flows greater than 100 cfs).

6.2.3 Channel Alignment

The minimum bend radius required for open channels is 25 feet or 10 times the bottom width, whichever is larger.

Connections at the junction of two or more open channels shall be smooth. Pipe and box culvert or sewers entering an open channel shall not project into the normal channel section and shall discharge into the receiving stream at an angle that directs flow downstream.

6.2.4 Freeboard

Open channels shall be designed to convey the design event with a minimum of 1 foot of freeboard at any point in the channel. In all cases, the 100-year (1% annual chance) event must be conveyed within the drainage easement for any drainage area larger than 5 acres. Channel depth calculations, whether by hand or computer model, should account for backwater effects and not be based merely on normal depth calculations. Superelevation of the water surface at horizontal curves shall be accounted for by increased freeboard.

6.2.5 Erosion

Open channels shall be analyzed for potential to create streambed and streambank scour or erosion. Channel linings shall be selected to minimize erosion potential. Design and calculations shall be submitted to the City for review and approval.

Potential for erosion at channel bends, drops, crossings and confluences and rising and falling water levels shall also be considered in the design.

6.2.6 Construction

Refer to Section 2.1.2

6.3 DESIGN CRITERIA

6.3.1 Design Storm Frequency

The design storm frequency for open channels shall be selected based upon the following:

- Special Flood Hazard Areas – 1% annual chance occurrence (100-year)
- All others – 25-year (4% chance occurrence)
- Refer to Section 6.2.4 for additional requirements

The City Engineer may increase these requirements for a project based upon upstream and downstream structures or conditions, potential for upstream flooding, safety, potential for future development, or other issues.

6.3.2 Manning’s “n” Value

The following general factors should be considered when selecting the value of Manning's n:

(1) As a general rule, retardance is increased when conditions tend to induce turbulence and reduced when they tend to minimize turbulence.

(2) The physical roughness of the bottom and sides of the channel should be taken into account. Fine particle soils on smooth, uniform surfaces result in relatively low values of n. Coarse materials, such as gravel or boulders, and pronounced surface irregularity cause higher values of n.

(3) The value of n will be affected by the height, density, and type of vegetation. Consideration should be given to density and distribution of the vegetation along the reach and the wetted perimeter, the degree to which the vegetation occupies or blocks the cross section of flow at different depths, and the degree to which the vegetation may be bent or "shingled" by flows of different depths. The n value will increase in the spring and summer, as vegetation grows and foliage develops, and diminish in the fall, as the dormant season approaches.

(4) Channel shape variations, such as abrupt changes in channel cross sections or alternating small and large cross sections, will require somewhat larger n values than normal. These variations in channel cross section become particularly important if they cause the flow to meander from side to side.

(5) A significant increase in the value of n is possible if severe meandering occurs in the alignment of a channel. Meandering becomes particularly important when frequent changes in the direction of curvature occur with relatively small radii of curvature.

(6) Active channel erosion or sedimentation will tend to increase the value of n, since these processes may cause variations in the shape of a channel. The

potential for future erosion or sedimentation in the channel should also be considered.

(7) Obstructions such as log jams or deposits of debris will increase the value of n. The level of this increase will depend on the number, type, and size of obstructions.

(8) To be conservative, it is better to use a higher resistance for capacity calculations and a lower resistance for stability calculations.

(9) Proper assessment of natural channel n values requires field observations and experience.

Special attention is required in the field to identify floodplain vegetation and evaluate possible variations in roughness with depth of flow. All of the factors listed above should be studied and evaluated with respect to type of channel, degree of maintenance, seasonal requirements, and other considerations as a basis for making a determination of an appropriate design n value. The probable condition of the channel when the design event is anticipated should be considered. Values representative of a freshly constructed channel are rarely appropriate as a basis for design capacity calculations. Refer to Section 6.4.2 for additional guidance.

6.3.3 Existing Stream Modifications

Modifications to existing streams may impact facilities upstream and downstream of the proposed modification area. When modifications to existing streams are required, channel stability must be assessed. The following considerations and actions should be considered/performed:

(1) A study should be made of the stream to be modified and should include historical information, evidence of other instability (i.e., bank caving, channel movement), results of other developments, and aerial photos showing alignment changes, either natural or man-made. The composition and erodibility of bed and bank material should be determined.

(2) Backwater calculations should be performed through the reach for a range of flows, including bankfull flow. Existing worst-case velocities and slopes should be calculated and related to the existing channel configuration to determine maximum velocities and shear stress values for actual conditions.

(3) A channel modification scheme that results in minimal interference with the channel is preferred.

(4) Channel modifications should generally be sized to match existing sizes and shapes. A narrow channel will deteriorate and a wide channel may collect silt. Floodways or high-flow channels should be used to carry extreme events rather than over-sizing a channel. Backwater should be recalculated through the modified reach.

(5) Protection should be provided where needed, from the downstream through the upstream extent of modification effects (i.e., effects of modification are often felt beyond the project limits). For flow with significant overbank components, a central section velocity must be used instead of the mean flow velocity. Protection should be sized for the design event and design-smooth

transitions. If velocities exceed the allowable values (refer to Tables 6.1 and 6.2), grade control structures or check dams should be considered.

(6) Streams that are classified as “Waters of the State” require the Arkansas Department of Environmental Quality’s (ADEQ’s) approval prior to any modifications.

Typically, existing stream channels should not be relocated. If relocation of a stream channel is unavoidable, the relocated channel shall be concrete as required by the City of Conway Subdivision Regulations.

cross-sectional shape, meander pattern and other existing conditions should be duplicated, if practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated to protect downstream channel sections and properties. Appropriate State and Federal permits will be obtained prior to construction and submitted to the City Engineer.

Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.

6.3.4 Velocity and Sediment Transport

Minimum and maximum flow velocities must be analyzed during design of an open channel. Sediment transport requirements must be considered for flows below the design frequency. A low flow channel component within a larger channel can reduce maintenance by improving sediment transport in the channel.

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in Table 6.1. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 6.2. Erosion Control Matting may be used if designed and constructed in accordance with specifications. Channel velocities shall not exceed the maximum permissible velocities given in Table 6.1 and 6.2.

Table 6.1. Maximum velocities for comparing lining materials.

Material	Maximum Velocity (ft/s)
Sand	2.0
Silt	3.5
Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay	5.0
Graded Loam or Silt to Cobbles	5.0
Coarse Gravel	6.0
Shales and Hard Pans	6.0
Erosion control matting	*

Source: AASHTO Model Drainage Manual, 1991.
* Based on manufacturer specifications and subject to approval by City Engineer.

Table 6.2. Maximum velocities for vegetative channel linings.		
Vegetation Type	**Slope Range (%)[1]**	**Maximum Velocity[2] (ft/s)**
Bermuda grass	0->10	5
Bahia		4
Tall fescue grass mixtures[3]	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	05-10	5
	>10	4
Grass mixture	0-5[1]	4
	5-10	3
Annuals[4]	0-5	3
Sod		4
Staked sod		5

1 Do not use on slopes steeper than 10% except for side-slope in combination channel.
2 Use velocities exceeding 5 ft/s only where good stands can be maintained.
3 Mixtures of Tall Fescue, Bahia, and/or Bermuda.
4 Annuals - use on mild slopes or as temporary protection until permanent covers are established.
Source: Manual for Erosion and Sediment Control in Georgia, 1996.

6.3.5 Channel Transitions

The following criteria should be considered at channel transitions in order for the channel system to operate as designed:

(1) Transitions should be smooth and gradual.
(2) A straight line connecting flow lines at the two ends of the transition should not make an angle greater than 12.5 degrees with the axis of the main channel.
(3) Transition sections should be designed to provide a gradual transition to avoid turbulence and eddies.
(4) Energy losses in transitions should be accounted for as part of the water surface profile calculations.
(5) Scour downstream from transitions between rigid and natural and from steep to mild slopes should be accounted for through velocity slowing and energy dissipating devices.

6.3.6 Channel Drops

Sloped drops shall have roughened faces and shall be no steeper than 2:1. They shall be adequately protected from scour and shall not cause an upstream water surface drop that will result in high velocities upstream. The design shall include protection against side cutting just downstream from the drop, which is a common problem.

6.3.6.1 Baffled Chutes

Baffle chutes are used to dissipate the energy in the flow at a larger drop. They require no tailwater to be effective. They are partially useful where the water surface upstream is held at a higher elevation to provide head for filling a side storage pond during peak flows.

Baffle chutes may be used in channels where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. The baffled apron shall be designed for the full discharge design flow and shall be protected from scouring at the lower end. A stilling basin shall be added where appropriate based on velocities.

6.4 DESIGN PROCEDURES

Open channels include drainage ditches, grass channels, dry and wet swales, riprap channels and concrete-lined channels. This section provides an overview of open channel design criteria and methods.

6.4.1 Computation and Software

Refer to Section 1.4 for a list of approved computer applications.

6.4.2 Determination of Manning's n Values

Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and riprap linings are given in Table 6.3. Recommended values for vegetative linings may be determined using Table 6.4. for various types of vegetation. Additional details are provided in the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA TS 84 204, 1984.

Table 6.3. Manning's roughness coefficients (n) for artificial channels.

Category	Lining Type	Depth Ranges		
		0-0.5 ft	0.5-2.0 ft	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.04	0.03	0.028
	Stone Masonry	0.042	0.032	0.03
	Soil Cement	0.025	0.022	0.02
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.02	0.02
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D50	0.044	0.033	0.03
	2-inch D50	0.066	0.041	0.034
Rock Riprap	6-inch D50	0.104	0.069	0.035
	12-inch D50	----	0.078	0.04

*Some "temporary" linings become permanent when buried.
Source: HEC-15, 1988.

Table 6.4 Uniform flow values of roughness coefficient n.			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
EXCAVATED OR DREDGED			
a. Earth, straight and uniform	0.016	0.018	0.020
1. Clean, recently completed	0.018	0.022	0.025
2. Clean, after weathering	0.022	0.025	0.030
3. Gravel, uniform section, clean	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/plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
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	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
Minor streams (top width at flood stage < 100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds, and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and some stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, few boulders	0.030	0.040	0.050
2. 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

Table 6.4 Uniform flow values of roughness coefficient n.			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
b. Cultivated area			
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, 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
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
MAJOR STREAMS (top width at flood stage > 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025		0.060
b. Irregular and rough section	0.035		0.100

6.4.3 Uniform Flow Calculations

6.4.3.1 Channel Discharge – Manning's Equation

Manning's Equation, presented in three forms below, shall be used for evaluating uniform flow conditions in open channels:

$v = (1.49/n)\ R^{2/3}\ S^{1}$ Eq. 6.1

$Q = (1.49/n)\ A\ R^{2/3}\ S^{1/2}$ Eq. 6.2

$S = [Q_n/(1.49\ A\ R^{2/3})]^2$ Eq. 6.3

Where:

v = average channel velocity (ft/s)
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient (see Section 1.4.2)
A = cross-sectional area (ft^2)
R = hydraulic radius A/P (ft)
P = wetted perimeter (ft)
S = slope of the energy grade line (ft/ft)

If the channel is uniform in resistance and gravity forces are in exact balance, the water surface will be parallel to the bottom of the channel. This is the condition of uniform flow.

Open channel flow in urban drainage systems is usually nonuniform because of bridge openings, curbs, and structures. This necessitates the use of backwater computations for all final channel design work.

A water surface profile shall be computed for all channels and shown on all final drawings. Computation of the water surface profile should utilize standard backwater methods or acceptable computer routines, taking into consideration all losses due to the changes in velocity, drops, bridge openings, and other obstructions.

Where practical, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not cause erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Unless approved otherwise by the City Engineer, channel velocities in man-made channels shall not exceed those specified in Section 6.3.4.

Where velocities exceed specified velocities riprap, pavement, or other approved erosion protection measures shall be required. As minimum protection to reduce erosion, all open channel slopes shall be seeded or sodded as soon after grading as possible.

6.4.3.2 Vegetative Design Requirements

Final design of temporary and vegetative channel linings involves the use of the tables in Section 1.4.2 for both stability and design capacity.

6.4.3.3 Riprap Design

Where the use of riprap is allowed by the City Engineer, riprap sizing shall be determined based on maximum anticipated channel velocities and the provision of adequate erosion protection for the design configurations. When rock riprap is used, the need for an underlying filter material must be evaluated. The filter material may be either a granular blanket or plastic filter cloth.

6.4.3.4 Gradually Varied Flow – Backwater Modeling and Data Requirements

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, inline storage, or channel constrictions. For these conditions, the flow depth will exceed normal depth in the channel and the water surface profile should be computed using backwater techniques.

For the use of step-backwater computations for the purpose of site design, detailed topographic data of an accuracy to support 1-ft or smaller contour intervals is required for existing and as-built models. In other areas (for downstream assessments, for example), the most recent publicly available data will generally be accepted.

7.0 STORMWATER DETENTION

7.1 INTRODUCTION

This section provides an overview of the requirements for design of detention facilities. While the details of each method are not included in this section, detailed design requirements and key parameters have been identified and reasonable values for these parameters are listed. The information provided in this section is intended to provide the design professional with commonly applied and accepted standards for detention design; however, additional approaches may be accepted by the City with approval. If proposing an alternate approach or using a parameter that differs from the values listed in this section, the design professional should discuss the method or parameter proposed to be used and provide justification for its use to the City Engineer for review and approval prior to preparing design computations.

7.2 DESIGN OBJECTIVES

The goal of stormwater detention design is to reduce post-development peak flows to pre-developed levels or less by providing an adequate storage volume to accommodate the difference and a controlled release of the stormwater through culverts, weirs, risers, or a combination of these structures. Both the peak flow and peak timing should be examined to determine the impact of the development on the watershed.

In some instances, detaining stormwater may not be beneficial in protecting downstream reaches. If the site discharges into a large stream, detaining runoff may cause the timing of the peak flow for the project site to align with the peak flow of the upstream basin, resulting in a higher cumulative peak flow for the downstream reach. If the site discharges to a regional detention facility that has been sized for anticipated development conditions in the watershed, on-site detention may not be required; although an in-lieu fee may be required by the City. In either of these cases, the engineer must submit detailed hydrologic and hydraulic analyses to support their recommendation that detention should not be provided.

7.3 TYPES OF DETENTION FACILITIES

Various types of detention facilities may be used depending on the needs of the development and space constraints within the project.

Dry detention basins are typically designed to handle stormwater during rainfall events, but drain completely within a day or two of the end of the rainfall event. This type of basin may also be used for alternate purposes, such as recreation, if designed appropriately.

Wet detention basins are those facilities which have been over-excavated to establish a year-round normal pool elevation. Typically designed for aesthetics, these facilities must be designed with safety in mind as well as to limit algae growth either through aeration or sufficient depth (minimum of six feet) and baseflow to discourage its growth. Pond volumes below the normal pool elevation may not be considered as stormwater storage in sizing calculations for design storm events.

Subsurface detention can be used on sites where space is limited. This type of detention can be constructed beneath parking areas or in green spaces where an open pond may impractical due to safety concerns or aesthetic reasons.

Each type of detention will require periodic maintenance to ensure design volumes and outflow rates are maintained. Dry and wet basins will require mowing, clearing of debris from outflow structures, and potentially repair of eroded areas on the pond sided slopes or berms to limit the potential for slope failures. Underground detention must have regular debris removal from the inlets, storage areas and outlet structures, accessed through multiple designed entrance points, to limit the changes of system clogging and surcharging/bypass of the detention facility during storm events. The pond design must include an operation and maintenance plan detailing the method and frequency of the required periodic maintenance.

7.4 METHODOLOGY

Reservoir routing, pond, and outlet sizing must be performed using the Modified Puls Method based on the pre- and post-developed runoff hydrograph computation methods described in Chapter 3 – Hydrology.

7.5 DETENTION POND DESIGN STANDARDS

7.5.1 Design Storm Event

All stormwater detention structures shall be designed to attenuate the post-development peak flow rate of the 50%, 10%, 4% and 1% annual chance (2-year, 10-year, 25-year, and 100-year), 24-hour duration design storm events. Release rates for each design storm frequency shall be less than or equal to the predevelopment runoff rates for the site. In cases where a grading permit has previously been issued for the site, and grading of the site resulted in a change in the runoff curve number (RCN), the pre-grading RCN value shall be used to calculate the predevelopment runoff rates for the site.

7.5.2 Outlet and Freeboard Criteria

Outlet structures shall be sized to limit post-development flow rates to pre-development levels for all of the design storms listed above. Outlet flow rates and velocities shall be examined, and adequate energy dissipation shall be designed to protect the structure, pond berm, and receiving stream from erosion. The pond outlet structure shall also be designed to convey the 1% annual chance design flow while maintaining a minimum of 1 ft of freeboard above the computed maximum water surface elevation in order to avoid overtopping during the design event. Additionally, structures within one hundred (100) feet horizontal distance to the pond shall have Finished Floor Elevations at least two (2) feet higher than the maximum water surface elevation computed for the 1% annual chance event, and shall be located a minimum of twenty (20) feet from the top bank of the detention basin.

7.5.3 Grading and Erosion Protection

Detention facilities shall be designed with a minimum of 2% slope (0.02 ft/ft) of the pond bottom, and shall include a minimum 36-inch wide concrete trickle channel designed to provide positive flow from each stormwater point source to the outlet structure in order to avoid stagnant water and to limit erosion of the pond bottom. Additional erosion protection may be required at any concentrated inflows to the pond, such as culverts or ditches, depending on design flow rates and computed velocities.

This erosion protection should be designed such that it does not reduce the storage volume of the pond; rip-rap, if used, should be finished flush with the side slopes or bottom of the pond.

Side slopes of the pond shall have a maximum slope of 3H:1V. Where depths exceed six (6) feet, a ten (10) foot wide safety shelf with a slope of no more than 10H:1V shall be designed at a depth of no more than one (1) foot below the computed maximum water surface elevation. Alternately, complete perimeter fencing of the pond may be substituted for the safety shelf.

Dry detention basins shall be completely vegetated as soon as practicable after completion of grading or excavation. Wet detention basins shall be vegetated above the normal pool elevation as soon as practicable after completion of grading or excavation. Cover may be established using native grass seed and straw or, if outside of the typical growing season or in the presence of unsuitable soils, Bermuda grass sod may be placed.

7.5.4 As-Built Documentation

The owner/developer shall provide an as-built survey of the final dimensions of the pond and outlet structure to assure that storage volumes and outlet structure geometry were built in accordance with design.

7.6 OUTLET STRUCTURE DESIGN STANDARDS

Outlet structures shall be sized to limit post-development flow rates to pre-development levels for all of the design storms listed above. Outlet flow rates and velocities shall be examined, and adequate energy dissipation shall be designed to protect the structure, pond berm, and receiving stream from erosion. The pond outlet structure shall also be designed to convey the 1% annual chance design flow while maintaining a minimum of 1 ft of freeboard above the computed maximum water surface elevation in order to avoid overtopping during the design event. Additionally, a low-flow release should be provided at the lowest dry point of the basin to allow complete drainage of dry detention basins or maintain the normal pool elevation of a wet pond during minor rainfall events.

Outlet structures may utilize a single structure, such as a pipe culvert or v- or t-notch weir, or may be a composite structure, such as a riser with weir, orifice, and pipe culvert components. Minimum sizes of outlet components are described below:

- Orifices must be a minimum of 4” in diameter or height and width.
- Pipe culverts must be a minimum of 12” in diameter.

Trash racks protecting pipe culverts or riser structures should be considered to mitigate the potential for clogging the outlet structure and overtopping the basin.

7.6.1 Pipe Culverts

Culvert length should be kept to a minimum and existing structures should not be extended without first determining the effects on the performance of the structure.

Where possible, the length and slope of the culvert should match the existing topography and inverts should match the channel bottom and skew angle of the stream.

Refer to Section 5 for additional requirements.

7.6.2 Headwalls and Wingwalls

Refer to Section 5.3.8 for requirements.

7.7 EASEMENTS

An easement must be dedicated to allow access to the pond and outlet structure by the owner or agency responsible for maintenance as indicated in the operation and maintenance plan for the basin. These easements must be designed to support access by maintenance equipment including slopes less than 10% and a solid driving surface comprised of gravel, concrete, asphalt, or reinforced turf.