STORMWATER MANAGEMENT AND DRAINAGE MANUAL

November 2024

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1. Submittal Procedures

1.1 General

Recognizing that properly designed stormwater management systems are essential to the general public's health and welfare within a metropolitan area as the City of Little Rock, the City hereby adopts the following criteria for standard procedures in stormwater management. This manual is intended to serve as a guide for the design of all stormwater infrastructure, including the design of new facilities and the upgrading of existing facilities.

Any company, agency, or person proposing to alter current land use within the City's Planning Jurisdiction shall submit drainage plans to the Department of Planning and Development for approval of a stormwater management and drainage plan before grading permits are issued, or subdivisions are approved. No land shall be developed except upon approval by the Department of Planning and Development, in coordination with appropriate departments.

Exceptions where no stormwater management and drainage plan are required according to Section 29-61 of the City of Little Rock Municipal Code:

- One (1) new or existing single-family structure.
- One (1) new or existing duplex family structure.
- One (1) new commercial or industrial structure located on less than a one-acre individual lot.
- One (1) existing commercial or industrial structure where additional structural improvements are less than five hundred (500) square feet.

Development not requiring a stormwater management and drainage plan still must comply with any applicable floodplain or building codes regarding drainage, stormwater, flooding, and finished floor elevation criteria.

The application for a Grading and Land Alteration Permit shall be prepared by the Engineer of Record, who is a licensed Professional Engineer of the State of Arkansas and shall be submitted in accordance with the submittal procedures described in this chapter. The Grading and Land Alteration Permit application shall consist of the Final Drainage Report and the Grading and Drainage Design Plans and Specifications (Plans and Specifications).

1.1.1 Additional Regulatory Requirements

Arkansas Department of Environmental Quality

A NPDES Construction Stormwater General Permit (Permit No. ARR150000) is required for discharges from large and small construction activities that result in a total land disturbance of equal to or greater than one acre, where those discharges enter waters of the State or a municipal separate storm sewer system (MS4).

- Small construction sites (disturbing one acre or more and less than five acres) have automatic coverage under the Construction Stormwater General Permit. Under automatic coverage for small sites, it is not necessary to submit any documents to ADEQ and there is no fee. However, the automatic Notice of Coverage (NOC) must be posted at the site prior to commencing construction and a Stormwater Pollution Prevention Plan (SWPPP) must be prepared and made available at the site prior to commencing construction.
- Large Construction Sites (disturbing five acres or more) must submit a Notice of Intent (NOI), a Stormwater Pollution Prevention Plan (SWPPP) and pay a fee to the Arkansas Department of Environmental Quality (ADEQ) in order to obtain coverage for discharges of stormwater associated with construction activity at any site or common plan of development or sale that will

result in the disturbance of five (5) or more acres of total land area. Additional information may be found at: https://www.adeq.state.ar.us/water

U.S. Army Corps of Engineers
Section 404 of the Clean Water Act requires a permit from the U.S. Army Corps of Engineers (USACE) prior to discharging dredged or fill material into waters of the United States, including wetlands. Activities in waters of the United States regulated under this program include fill for development, water resources projects (such as dams and levees), infrastructure development (such as highways and airports), streambank restoration, and mining projects.

Floodplain Development Permits
Any development within or bordering a Special Flood Hazard Area, as portrayed on FEMA Flood Insurance Rate Maps (FIRMs) is required to obtain a Floodplain Development Permit. Permit requirements and application procedures can be found in Section 13-56 of the City of Little Rock Municipal Code.

1.2 Submittal Procedures

1.2.1 Conceptual Drainage Review

A conceptual drainage plan review with staff is suggested before preliminary platting for the purpose of overall general drainage concept review.

1.2.2 Preliminary Review

A preliminary stormwater and drainage plan, and accompanying information, shall be submitted at the time of preliminary plat submittal. If needed, a review meeting will be scheduled by the Department of Planning and Development with representatives of the developer, including the Engineer of Record, to review the overall concepts included in the preliminary stormwater and drainage plan. The purpose of this review shall be to jointly agree upon an overall stormwater management concept for all phases of the proposed development and to review criteria and design parameters which shall apply to final design of the project. Preliminary drainage plan/study must be approved prior to Preliminary Platting.

1.2.3 Final Drainage Review

Following the preliminary stormwater management and drainage plan review, the final stormwater management and drainage plan shall be prepared for each phase of the proposed project as each phase is developed. The final plan shall constitute a refinement of the concepts approved in the preliminary stormwater and drainage plan with preparation and submittal of detailed information as required in the drainage manual. This plan shall be submitted at the time construction drawings are submitted for approval. Final drainage plan must be approved prior to approval of construction plans. No final plat is to be approved until the drainage structures approved on the construction plans are constructed, inspected, and approved by the Department of Planning and Development.

1.2.4 Online Submittal Procedures

The grading and land alteration permit application process is through the Planning and Development Department's online portal (https://permitpayment.littlerock.gov/ips_PD/Views/Login.aspx) New users must register for an online account. The applicant is required to fill out the online application for a grading & land alteration permit and submit the following documents through the portal: grading & drainage plans, sediment and erosion control plans, land survey, drainage report, and soil loss estimate calculations. When filling out the online application, no fields should be left blank. If a field is not applicable to the project, enter "Not Applicable" in those fields on the application. For required numerical value fields, use only whole numbers (no commas or decimal points).

1.3 Plans and Specifications Requirements

Plans and Specifications for plans including stormwater drainage are to be signed by a Professional Engineer registered in the State of Arkansas in accordance with applicable state statutes and State PELS board licensure requirements. Because all Plans, Specifications, and Calculations will be retained by the City for use as permanent records, neatness, clarity, and completeness are very important, and lack of these qualities will be considered sufficient basis for submittal rejection.

Plan sheet size must be 11 inches x 17 inches with all sheets in each set of plans the same size. Plan and profile drawings shall be prepared according to the following:

Plan Drawings

- Maximum horizontal scale 1 inch = 100 feet

Profile Drawings for drainage ditches and storm systems

- Suggested horizontal scale 1 inch = 20 feet
- Maximum horizontal scale 1 inch = 50 feet
- Minimum vertical scale 1 inch = 5 feet

Each sheet in a set of Plans shall contain a sheet number, the total number of sheets in the plans, proper project identification, and the date. Revised sheets submitted must contain a revision block with identifying notations and dates for revisions. A complete legend shall be included in the set of Plans.

To ensure reviews are completed in a timely manner, Plans and Specifications for all proposed improvements must be submitted in the following format during the project application process, where pertinent, and shall include at a minimum: (1) Title Sheet, (2) General Layout Sheet, (3) Grading and Drainage Plan (4) Paving, and/or Building Plans, (5) Three Phase Erosion and Sedimentation Control Plan, (6) Plan and Profile Sheets, (7) Cross Sections, (8) Standard and Special Detail Sheets, (9) Mapping, and (10) Calculations. A detailed checklist of requirements is included in Appendix A.

1.3.1 Title Sheet

The Title Sheet shall include:

1. The designation of the project which includes the nature of the project, the name of the development, city, and state.
2. Project number.
3. Index of Sheets.
4. Vicinity maps showing project location in relation to streets, railroads, and physical features. The location map shall have a north arrow and appropriate scale.
5. A project control benchmark identified as to the location and elevation with notation referencing City monument(s) used to establish the Project Benchmark.
6. Reference to the horizontal and vertical datum for the project.
    a. Prior to the release of the new Low Distortion Projection Zones (LDPZs) and 2022 Datums, planned to be released in 2025.
        i. The horizontal datum shall be NAD83, Arkansas State Plane, North Zone.
        ii. The vertical datum shall be North American Vertical Datum of 1988 (NAVD88).
    b. After the release of the new Low Distortion Projection Zones (LDPZs) and 2022 Datums, planned to be released in 2025.

i. Any projects started in a legacy zone and datum shall stay in that legacy zone and datum. Any projects being started shall be started utilizing the new zones and datums.
ii. The unit of measurement for NAD83 Horizontal Datum shall be the US Survey Foot.
iii. The unit of measurement for NAVD88 Vertical Datum shall be the US Survey Foot.
iv. The unit of measurement for NATRF2022 Horizontal Datum shall be International Foot.
v. The unit of measurement for NAPGD2022 Vertical Datum shall be the International Foot.

7. Survey metadata shall be noted and include the following:
   a. Horizontal Datum and/or Horizontal published control relied upon.
   b. Vertical Datum and/or Vertical published benchmark relied upon.
   c. Horizontal epoch adjustment:
      i. Any new adjustments that may supersede the 2011 adjustment.
      ii. 2011 Adjustment Epoch 2010.00.
      iii. 2007 NSRS Adjustment (Superseded).
      iv. 1990 HARN Adjustment (Superseded).
      v. 1986 Adjustment (Superseded).
   d. Geoid Model.
   e. State Plane Coordinate Zone or Low Distortion Projection Zone.
   f. The time and date the data was collected.
   g. Noting if legacy data was transformed via “on ground” survey observations.
   h. Unit of measure of Metadata (US Survey Foot or International Foot)
8. The name and address of the owner of the project and the name and address of the engineer preparing the plans.
9. Floodplain statement identifying the FIRM panel, date, and flood zone; and,
10. Engineer’s seal (every sheet).

1.3.2 General Layout Sheet

The general layout sheet shall include:

1. North arrow and scale.
2. Legend of symbols that will apply to all sheets.
3. Name of subdivision, if applicable, and all street names. Unplatted tracts should have an accurate tie to at least one quarter section corner.
4. Boundary line or project area.
5. Benchmark location and benchmark calls.
6. Location and description of existing major drainage facilities within or adjacent to the project area.
7. Location of major proposed drainage facilities.
8. Floodplain Boundaries with mapped BFE, if available
9. Name of each utility within or adjacent to the project area.

10. Standard notes.
11. If more than one general layout sheet is required, a match line should be used to show continuation of coverage from one sheet to the next.

1.3.3 Other Requirements for Plans and Specifications

Other requirements for Plans and Specifications include:

1. The registration seal of the Engineer of Record shall be placed in a convenient place in the lower right-hand corner of each sheet of plans.
2. Elevations on profiles of sections or as indicated on plans shall have survey data or best available topographic data. At least one permanent benchmark in the vicinity of each project shall be noted on the first drawing of each project, and their location and elevation shall be clearly defined.
3. Convention for stationing shall be West to East or South to North from the left to the right side of the sheet respectively.
4. Each project shall show at least 20’ of topography on each side. At least 50’ of topography shall be shown in areas of channel flow at the property boundary. All existing topography and any proposed changes, including utilities, telephone installations, etc., shall be shown on the plans, profiles, and cross-sections.
5. Revisions to drawing shall be indicated above the title block in a revision box and shall show the nature of the revision and the date made.
6. Utilizing the standard symbols for engineering plans, all existing utilities, telephone installations, sanitary and storm sewers, pavements, curbs, inlets, and culverts, etc., shall be shown with a dithered/grayed out linework. Proposed facilities shall be shown with a solid line and land, lot, easement, and property lines shall be shown with bold and black linework.
7. Lot lines and dimensions shall be shown where applicable.
8. Show and label FEMA floodplains and/or the 100-year floodplain.
9. Minimum floor elevation shall be shown on each lot when located in a designated floodplain and in areas where flooding is known to occur. For all occupied structures within a designated floodplain, the top surface of the lowest floor must have an elevation at least one (1) foot or more above the published base flood elevation (BFE). All occupied buildings, whether in or out of a designated floodplain shall have the finished floor elevation (FFE) a minimum of 12” above the land immediately surrounding the building. Reference Section 13-60 of the City of Little Rock Municipal Code for additional information.
10. It shall be understood that the requirements outlined in these sections are only minimum requirements and shall only be applied when conditions, design criteria, and materials conform to the City specifications and are normal and acceptable to the Design Review Engineer. When unusual subsoil or drainage conditions are suspected, an investigation should be made, and a special design prepared in line with good engineering practice.

1.4 Grading and Drainage Report Requirements

1.4.1 Preliminary Grading and Drainage Report

A Preliminary Grading and Drainage Report will be required at the time of the Technical Plat Review for site development projects. The Preliminary Grading and Drainage Report shall follow the Final Grading and Drainage Report Template provided in Appendix A of this Drainage Manual. In addition to the Preliminary Grading and Drainage Report, also submit preliminary grading and drainage drawings.

1.4.2 Final Grading and Drainage Report

A Final Grading and Drainage Report shall be included in the Final Grading and Drainage Permit Application. Computer model input summary tables and output result tables shall also be provided as part of the Final Grading and Drainage Report. An example of input and output data to report may be found in Appendix B.

A Grading and Drainage Permit will not be issued until the Final Report has been submitted, reviewed, and approved. The Design Review Engineer may request a more detailed drainage study prior to the approval of the Final Grading and Drainage Permit application and issuance of the permit.

If hydrologic and hydraulic studies reveal that the proposed development would cause increased frequency of flooding, increased depth of inundation of structures, or inundation of unprotected structures not previously subject to inundation, then the permit application shall be denied unless one or more of the following mitigation measures are used to remove increases: (1) onsite storage, (2) offsite storage, or (3) offsite drainage systems improvements.

If it is determined by the Design Review Engineer that offsite drainage improvements are a feasible option to mitigate development impacts, then cost sharing will be in accordance with City ordinances. If the City is unable to contribute its share of the offsite costs, the developer shall have the option of: a) building the offsite improvements at his own expense, b) providing detention so as to match pre-development downstream capacities, or c) delaying the project until the City is able to share in the offsite costs.

1.5 Electronic As-Builts

Per Section 31-117(b)(18) and Section 29-5 of the City of Little Rock's Municipal Code, the City of Little Rock Public Work's Department requires as-built plans and information submitted from the Engineer of Record with final plats; request for certificates of occupancies on building permits; and following street and drainage infrastructure construction projects. Plans and information should be provided on public and private stormwater drainage systems installed and/or modified.

Final approval of final plat shall not be given until the Department of Planning and Development receives an electronic copy of the Stormwater Drainage Features As-Built, in either a compatible ArcGIS file format (Esri shapefile or Esri geodatabase), or AutoCAD .dwg file format. The As-Built Plan drawings shall be in State Plane Arkansas North Zone coordinates, with the North American Datum 1983 with units as survey feet. The As-Built drawings shall have the stormwater features drawn in a separate layer in AutoCAD so the features can be easily separated from other layers in the drawing. The associated attribute data table will conform to the approved specifications contained in the "SW Attribute Data Entry Template.xlsx" as provided by the City's Public Works Department. On the As-Built Plans, all Control, Linear and Junction map features will be annotated by a unique identifier that will correspond to the same unique identifier in the "SW Attribute Data Entry Template.xlsx" or GIS attribute table. All required attribute information for each Linear and Junction feature will be completed in the "SW Attribute Data Entry Template.xlsx" or GIS attribute table as follows, or as indicted by bold column headings in the "SW Attribute Data Entry Template.xlsx", using the domain values specified in Appendix A of this Drainage Manual.

2. Stormwater Criteria, Planning, and Regulation

2.1 Stormwater Sizing Criteria

Development projects applying for a Grading and Drainage Permit shall meet the following criteria related to stormwater runoff and protection of existing water bodies and properties. For the purposes of this Drainage Manual, pre-development is defined as the existing conditions of the site at the time of development. The primary goal of this manual is to promote the health, safety, and welfare of the public. To achieve this goal, the objective of this manual is to:

- Reduce stormwater runoff pollutants and protect water quality.
- Reduce downstream overbank flooding and channel erosion.
- Safely convey the design storm and extreme storm events.

For the objectives outlined above, the following stormwater sizing criteria have been developed which are used to size and design structural stormwater controls. Table 2.1 briefly summarizes the criteria.

Table 2.1 Summary of Stormwater Sizing Criteria for Stormwater Control and Mitigation.

Sizing Criteria	Description
Water Quality	Provide water quality treatment for the runoff resulting from a rainfall depth of 1.05 inches.
Downstream Flood Protection	Provide peak discharge control of the 2-year, 25-year, and 100-year storm event such that the post-development peak rate does not exceed the pre-development rate. Extend analysis through the zone of influence based on the 10% rule for projects greater than 40 acres.
Level of Service	Provide a level of service for the storm drainage system based on the 25-year storm. Analyze and provide a safe 100-year overflow path for all developments. Based on the Master Street Plan, provide a level of service for cross drainage of: 100-year storm for Principal Arterials, as well as for Major and Minor Arterials, 25-year storm for all other streets.

Each stormwater sizing criterion is intended to be used in common conjunction with the others to address the overall stormwater impacts from a development site. Used as a set, the criteria control a range of hydrologic events from the smallest runoff-producing rainfalls to the 100-year storm.

2.1.1 Water Quality (WQ_v)

In accordance with the City's Municipal Separate Storm Sewer System (MS4) general permit under NPDES Permit No. ARS000002, the stormwater management system should be designed to remove at least 80% of the total suspended solids (TSS) from stormwater flows which exceed pre-development levels and be able to meet any other additional watershed- or site-specific water quality requirements.

The stormwater management system shall be capable of removing at least 80% of TSS from an equivalent onsite impervious area. Any onsite impervious area (draining to a common outlet point) may be treated to meet this requirement – parking areas are preferred due to highest pollutant removal opportunity.

Chapter 8 of this Drainage Manual outlines the methodology to select appropriate structural stormwater controls and design a system or “treatment train” that removes 80% of the TSS from 1.05 inches of rainfall, the Water Quality Treatment Volume (WQ_v). Chapter 8 also covers Low Impact Development (LID) strategies and Best Management Practices (BMPs) that may be utilized to meet the water quality requirement. LID strategies and BMPs shall be selected based on the targeted pollutant removal based off the type of development. Refer to Chapter 8 and Appendix C for information on BMP selection.

The Water Quality sizing criterion specifies a treatment volume, denoted WQ_v, required to size structural stormwater controls to meet the 80% TSS removal. For the City of Little Rock, this value is computed as 1.05 inches of rainfall over the catchment area multiplied by the runoff coefficient (R_v).

The Water Quality Volume is calculated using the formula below:

$$\boldsymbol{WQ_V = \frac{1.05R_VA}{12}} \qquad \textbf{Eq. 2.1}$$

Where: WQ_V = water quality volume (acre-feet)

R_V = 0.05 + 0.009(I) where I is the percentage (%) impervious cover within the project area (post-development)

A = Project Area (acres)

Refer to Chapter 8 for detailed design guidance regarding water quality treatment.

2.1.2 Downstream Flood Protection

Downstream overbank flood protection shall be provided by controlling the post-development peak discharge rate to not exceed the pre-development rate for the 2-, 25-, and 100-year, 24-hour return frequency storm event. The NOAA Atlas 14 rainfall depths that correspond to these events are 4.19, 5.12, 5.94, 7.14, and 9.17 inches, respectively. Please note that these depths are subject to change once NOAA releases their Atlas 15 rainfall depths.

Existing floodplain areas should be preserved to the extent possible. At the discretion of the Design Review Engineer, analysis of floodplain impacts and additional detention or reduction in post-development peak discharge rates may be required for developments. Downstream analysis, when required, shall extend through the zone of influence based on the 10% rule.

Determining the Overbank Flood Protection Volume

- *Peak-Discharge and Hydrograph Generation:* The hydrograph methods provided in Chapter 3 of this Drainage Manual shall be used to compute the peak discharge rate and runoff for the 2-, 25-, and 100-year, 24-hour storm. The runoff hydrographs shall be routed through the proposed detention/retention structures using appropriate software or methodology as outlined in Chapter 7.

2.1.3 Level of Service

The stormwater system (inlets, storm pipes) shall be designed to meet the 25-year Level of Service and cross drainage (cross culverts, bridges) shall be designed to meet the Level of Service based on the Master Street Plan. The Design Storm is based on the fully developed watershed conditions. The Level of Service for the roadway classifications are as follows:

Table 2.2 Design Storm Street Classification.

Street Classification	Design Storm	
	Cross Drainage	Storm System
Principal Arterials	100-year	25-year
Major and Minor Arterials	100-year	25-year
All other streets	25-year	25-year

The hydraulic grade line (HGL) shall be calculated throughout the storm system to ensure the maximum HGL for the design storm is 1-foot below the throat of each roadway inlet. The starting HGL shall be based on the known or calculated tailwater elevation of the receiving channel or waterbody for the design event. The fully developed 100-year storm must be contained within the designated right of way.

Determining the Adequacy of the Stormwater Management System

- *On-site Storm System Sizing:* The Rational method or SCS TR-55 hydrograph method provided in Chapter 3 shall be used to compute peak discharge rate and runoff for the design storm.
- *Downstream Analysis:* Peak discharges at downstream locations shall be checked and evaluated for any increase in peak flow above pre-development conditions. The downstream check shall extend to the point where the developed site area comprises at least 10% of the total drainage area checked. If the post-developed discharges at the downstream checkpoints exceed pre-development conditions, additional mitigation measures shall be required.
- *System Check:* As a final check, the 100-year, 24-hour storm event shall be routed through the drainage system and stormwater management facilities to determine the effects on the facilities, adjacent property, and downstream, and to confirm adequacy of finished floor elevations for structures. Emergency spillways for structural stormwater controls should be designed to safely pass the fully developed 100-year flow. A positive overflow pathway shall be provided that conveys the flow without damaging structures, utilities, or infrastructure.

2.1.4 Pertinent Ordinances and Regulations

Refer to the following chapters of the City of Little Rock Municipal Code and additional documents for information related stormwater management within the City of Little Rock.

Chapter 13 – Floods

Chapter 29 – Stormwater Management and Drainage

Chapter 30 – Streets and Sidewalks

Chapter 31 – Subdivisions

NPDES Permit Number ARS000002

Kaylin Hills et al v. City of Little Rock Planning Commission, 60CV-18-8041, 2019

3. Determination of Storm Runoff

3.1 General

An accurate estimation of storm runoff is critical for planning and development. Atlas 14 rainfall provided by the National Oceanic and Atmospheric Administration (NOAA) will be used in drainage calculations and can be found in section 3.2. Please note that the rainfall depths found in Table 3.2 are subject to change once NOAA releases Atlas 15 rainfall data. Section 3.3 discusses how to estimate flows using the Rational Method. Section 3.4 discusses the Unit Hydrograph method using the Soil Conservation Service (SCS) Technical Release 55 (TR-55) method for hydrograph generation. Section 3.5 details the use of the Runoff Routing Method using Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software for runoff estimation. Table 3.1 summarizes the accepted approaches of runoff determination; however, the Director of Planning and Development may approve other engineering methods when they are shown to be comparable to the following methods.

Table 3.1 Hydrology Methodology.

Methodology (Section)	Watershed Size
Rational Method (3.3)	Less than 40 acres.
Unit Hydrograph (SCS) Method (3.4)	Up to 200 acres.
Runoff Routing (HEC-HMS) Method (3.5)	Greater than 200 acres.

Section 3.7 provides a summary of acceptable runoff analysis software. Other software that utilizes the recommended methodology per Table 3.1 may be used at the City Design Review Engineer's discretion if results are shown to be comparable. The runoff analysis must show the results for the fully developed 2-, 25-, and 100-year, 24-hour storm events.

3.2 Precipitation Data

Rainfall data from the current version of the NOAA Precipitation Frequency Atlas will be used in drainage calculations. The current version, NOAA Atlas 14, is expected to be replaced by Atlas 15 which will be fully released in 2027. The NOAA Atlas 14 rainfall depths for the Clinton National Airport Gauge in Little Rock, Arkansas are provided in Table 3.2. Intensity-Duration-Frequency curve equation coefficients can be found in Table 3.3. The following sections 3.3-3.6 provide more detail on how to use these values according to the appropriate runoff methodology.

Once the drainage basin is defined, the next step in the hydrologic analysis is an estimation of the rainfall that will fall on the basin for a given time period. The duration, depth, and intensity of the rainfall are defined below:

- ***Duration (hours)*** – Length of time over which rainfall (storm event) occurs.
- ***Depth (inches)*** – Total amount of rainfall occurring during the storm duration.
- ***Intensity (inches per hour)*** – Depth divided by the duration.

The frequency of a rainfall event is the recurrence interval of storms having the same duration and volume (depth). This can be expressed in terms of annual chance or return period.

- ***Annual Chance*** – *Percent chance that a storm event having the specified duration and volume will be exceeded in one year/years (e.g., a "25-year" storm has a 4-percent-annual-chance of occurring in any given year).*

- ***Return Period*** – *Average length of time between events that have the same duration and volume* (e.g., 10-year event).

Thus, if a storm event with a specified duration and volume has a 1 percent chance of occurring in any given year, it may be termed a 1-percent-annual chance event. The use of the phrase “return period” is discouraged because it gives a false impression that storm events cannot occur more frequently than the corresponding return periods.

Rainfall depths for the 24-hour duration storm for the City of Little Rock are provided in Table 3.2 below. If rainfall is required for other than 24-hour duration, it can be taken from the NOAA Atlas 14 Precipitation Frequency Data Server at https://hdsc.nws.noaa.gov/pfds/.

Table 3.2 Atlas 14 Rainfall Depths for 24-hour Duration Storm.

Storm Event	2-year	5-year	10-year	25-year	50-year	100-year
Depth (in)	4.19	5.12	5.94	7.14	8.13	9.17

Rainfall intensity is selected based on the design rainfall duration and frequency of occurrence. The design duration is equal to the time of concentration for a drainage area under consideration. Once the time of concentration is known, the design intensity of rainfall may be determined from the rainfall intensity equation below.

The rainfall intensity is calculated using the formula below:

$$\boldsymbol{I} = \frac{\boldsymbol{b}}{(\boldsymbol{t_c} + \boldsymbol{d})^{\boldsymbol{e}}} \qquad \textbf{Eq. 3.1}$$

Where: I = precipitation intensity (inches per hour)

t_c = time of concentration (minutes)

e,b,d = variable coefficients (see table below)

Table 3.3 includes the coefficients used in the IDF curve equation. The variable coefficients *e, b*, and *d* in the IDF curve equation are derived by fitting the equation to data on the frequency and intensity of storm events from Atlas 14 for the City of Little Rock.

The majority of drainage sub-basins within the City of Little Rock have a relatively short time of concentration. In general, basins with computed times of concentration in excess of 60 minutes (maximum) should be subdivided to create smaller sub-basins for more accurate computation of peak discharge. Where a design time of concentration for a watershed sub-basin exceeds 30 minutes, the applicability of the Rational Method shall be justified with documentation if it is used. In sub- basins with significant channel or overland storage, errors may be introduced by the use of the Rational Method.

Table 3.3 Atlas 14 e,b,d Variable Coefficients.

Return Period	Variable		
	e	***b***	***d***
2yr	0.625	22.461	3.488
5yr	0.628	27.071	3.592
10yr	0.626	30.302	3.550
25yr	0.620	34.032	3.395
50yr	0.619	37.392	3.507
100yr	0.612	39.450	3.253
500yr	0.598	44.067	2.969

3.3 Rational Method

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm pipes, culverts, and roadside ditches. It is most applicable to small, highly impervious areas. The Rational Method was not developed for storage design or any application where a more detailed routing procedure is required. However, for the design of small detention facilities, the Modified Rational Method may be used for sites up to 40 acres. Detention design methodology for the Modified Rational Method is included in Chapter 7 of this manual.

Basic assumptions associated with use of the Rational Method are as follows:

1. The computed peak rate of runoff to the design point is a function of the average rainfall intensity during the time of concentration to that point.
2. The time of concentration (see Section 3.4.1) is the critical value in determining the design rainfall intensity and is equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.
3. Infiltration, represented by the runoff coefficient (C), is uniform during the entire duration of the storm event.
4. The rainfall intensity, I, is assumed to be uniform for the entire duration of the storm event and is uniformly distributed over the entire watershed area.

The Rational Method Equation, shown below, estimates the peak flow based on the runoff coefficient, drainage area, and rainfall intensity associated with the watershed time of concentration.

$$\mathbf{Q = CiA} \qquad \textbf{Eq. 3.2}$$

Where: *Q* – Peak flow (cfs).

C – Dimensionless runoff coefficient.

i – Rainfall intensity (in/hr).

A – Drainage area (acres).

The value of C assigned for the drainage area should be an area-weighted average accounting for all the proposed land uses within the project area. Typical values of C are provided in Table 3.4 for various land use conditions, slopes, and hydrologic soil types.

Table 3.4 Runoff coefficients (C-values) for various land uses.

Land Use Description	Slope, %	Hydrologic Soil Group		
		A/B	C	D
Lawns				
	0-2	0.15	0.25	0.35
	2-7	0.25	0.35	0.40
	> 7	0.30	0.35	0.45
Unimproved areas				
Forest		0.15-0.2	0.20-0.25	0.20-0.30
Meadow		0.20-0.4	0.25-0.45	0.30-0.55
Row crops		0.25-0.6	0.35-0.75	0.40-0.80
Business				
Downtown areas		0.7	0.8	0.9
Neighborhood areas		0.5	0.6	0.7
Residential				
8 lots / acre		0.67	0.71	0.76
4 lots / acre		0.46	0.52	0.61
3 lots / acre		0.40	0.47	0.57
2 lots / acre		0.35	0.43	0.54
Suburban (1 lot / acre)		0.30	0.38	0.50
Multi-units, detached		0.70	0.75	0.80
Multi-units, attached		0.75	0.80	0.85
Apartments		0.65	0.70	0.75
Industrial				
Light areas		0.60	0.75	0.85
Heavy areas		0.80	0.85	0.90
Parks, cemeteries		0.25	0.35	0.45
Schools, Churches		0.70	0.75	0.80
Railroad yard areas		0.20	0.35	0.50
Asphalt & Concrete Pavements, Roofs.			0.95	
Brick Pavement or Gravel (compacted subgrade)			0.85	
Graded or no plant cover				
	0-2	0.25	0.30	0.35
	2-7	0.40	0.50	0.60
	> 7	0.50	0.65	0.80

1. State of Georgia (2001).
2. Oregon Dept. of Transportation (2005).
3. Arkansas Highway and Transportation Department (1982).
4. Virginia Department of Transportation (2002).

- Selection of design rainfall intensity in the Rational Method shall be based on the required design frequency and calculated using the IDF equation provided in Section 3.2.
- Drainage area computations for runoff estimation should be based on the best available data. Where more recent and more detailed site-specific topographic data is not available, the most recent publicly available topographic contour data should be used (www.pagis.org).

The coefficients given in Table 3.4 are applicable for storm events up to the 10-year frequency. Less frequent, higher intensity storms require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff. The adjustment of the Rational Method for use with major storms can be made by multiplying by a frequency factor, C_f. The Rational Formula now becomes:

$$\mathbf{Q} = C_f \mathbf{CiA} \qquad \textbf{Eq. 3.3}$$

The C_f values are provided in Table 3.5. The product of C_f times C shall not exceed 1.0.

Table 3.5 Frequency Factors (C_f) for Rational Equation.

Recurrence Interval	C_f
10-year or less	1.0
25-year	1.1
50-year	1.2
100-year	1.25

Note: Cf * C shall not exceed 1.0

Source: Georgia Stormwater Manual

3.4 Unit Hydrograph (SCS) Method

The Soil Conservation Service (SCS) hydrologic method is based on a synthetic unit hydrograph. The SCS Technical Release 55 (TR-55) approach for runoff determination was developed specifically for use in urbanized and urbanizing areas. Multiple software programs are available that utilize the SCS hydrologic method. A detailed examination of the capabilities and limitations of various software is required to ensure that the appropriate software is used.

In general, the SCS approach considers time distribution of rainfall, initial rainfall losses (infiltration and depression storage), and allows for varying infiltration throughout the storm interval. Further details are provided in the National Engineering Handbook (NRCS, 2004). The SCS method directly relates runoff to rainfall amounts through use of curve numbers (CNs) based on Hydrologic Soil Group (HSG) soil type and on land use.

A typical application of the SCS method includes the following basic steps:

- Determine curve numbers for different land uses and soil types within the drainage area.
- Calculate time of concentration to the drainage area outlet point.
- Use the Type II rainfall distribution to determine excess rainfall.
- Develop the direct runoff hydrograph for the drainage basin.

This method can be used both to estimate stormwater runoff peak discharges and to generate hydrographs for routing stormwater flows. This method may be used for design applications including open channels, small drainage ditches, energy dissipation, storm drain systems, storm sewer networks, inlet and outlet structures, and storage facilities.

Design rainfall may be input into various programs that use the SCS method. For the purpose of pre- and post-development runoff comparisons, the following design storm data shall be used:

> Rainfall amounts for 24-hour storm durations with recurrence intervals of 2-, 25-, and 100- years. The appropriate rainfall distribution for the City of Little Rock is Type II.

3.4.1 Time of Concentration and Travel Time

Time of Concentration must be calculated using SCS TR-55 Method only, other methods shall not be allowed. Time of Concentration (T_C) is the sum of Travel Time values for the various consecutive flow segments: sheet flow, shallow concentrated flow, and channel flow.

$$T_c = T_s + T_{sc} + T_{ch} \qquad \textbf{Eq. 3.4}$$

Where: Tc = Time of Concentration

Ts = Sheet Flow Time

Tsc = Shallow Concentrated Flow Time

Tch = Channel Flow Time

The time of concentration longest flow path shall be provided for each basin and shall be updated as required between pre- and post-development computations. The flow path should be the flow path that best represents the basin which may not be the longest (in length) flow path. The minimum time of concentration is 5 minutes.

a. **Sheet Flow**
The maximum length of sheet flow is 100 feet. For sheet flow of less than 100 feet, use Manning's Kinematic solution (Overtop and Meadows 1976) to compute sheet flow travel time:

$$T_s = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}s^{0.4}} \qquad \textbf{Eq. 3.5}$$

Where: n = Manning's Roughness Coefficient (see TR-55 for roughness coefficients)

L = flow length (ft)

P2 = 2-year, 24-hour rainfall (in)

s = slope of hydraulic grade line (land slope, ft/ft)

b. **Shallow Concentrated Flow**
After a maximum of 100 feet, sheet flow usually becomes shallow concentrated flow. Shallow Concentrated flow transitions to channel flow when the flow reaches the curb or a defined swale or channel. Shallow Concentrated flow shall be calculated based on the average velocity along the flow path using the following equation:

$$Tsc = \frac{L_f}{3600\,V} \qquad \textbf{Eq. 3.6}$$

Where: L_f = Flow length (ft)

V = Velocity (ft/sec)

The average velocity for shallow concentrated flow may be determined using the equations below:

Paved	$V = 20.33(S)^{0.5}$	**Eq. 3.7**
Unpaved	$V = 16.13\ (S)^{0.5}$	**Eq. 3.8**

Where: V = Velocity (ft/sec)

S = watercourse slope (ft/ft)

c. **Channel Flow Time**

The channel flow time can be calculated using the following formula:

$$Tch = \frac{L_f}{3600\,V} \qquad \textbf{Eq. 3.9}$$

Where: L_f = Flow length (ft)

V = Velocity (ft/sec)

The velocity in the channel can be calculated using the Manning's equation.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \textbf{Eq. 3.10}$$

Where: Q = total discharge in cubic feet per second

n = coefficient roughness

A = cross-sectional area of conduit in feet

R = hydraulic radius of channel in feet

S = slope of energy line in feet per foot

V = velocity in feet per second

3.4.2 Runoff Factor

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, the SCS has divided soils into four hydrologic soil groups (HSGs).

Group A Soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D Soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high-water tables, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious parent material. Embankments designated or identified as "hillside" in the City shall be classified as Hydrologic Soil Group D.

For use in hydrologic computations, the most recent soil distribution data can be viewed online and downloaded from the NRCS Web Soil Survey (USDA NRCS).

The effects of urbanization on the natural hydrologic soil group should be accounted for in design. Runoff curve numbers for different land uses are provided in Table 3.6. In all areas disturbed by heavy equipment used during construction or where grading will mix the surface and subsurface soils, the curve numbers shall be shifted to the next higher HSG for design.

Area-weighted composite curve numbers shall be calculated for each drainage area and used in the analysis based on variations in soil type and land use. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses. The drainage area is assigned a composite uniform land use represented by the composite curve number. However, if the spatial distribution of land use is important to the hydrologic analysis, then sub-basins corresponding to the distribution (to the extent possible) should be developed and separate sub-basin hydrographs developed and routed to the study point.

The curve numbers in Table 3.6 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system or occurs as concentrated shallow flow that runs over pervious areas then into a drainage system.

It is possible that curve number values from urban areas could be reduced by disconnecting impervious areas and allowing such runoff to sheet flow over additional significant pervious areas prior to entering the drainage system. Additional information on this approach is described in Chapter 8. The CNs provided for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

- Pervious areas that are not disturbed by construction equipment are equivalent to pasture in good hydrologic condition, and
- Impervious areas have a CN of 98 and are directly connected to the drainage system.

If Low Impact Development, or Green Infrastructure, stormwater controls or practices are implemented in design, the impact of such features in reducing overall stormwater runoff may be accounted for. Practices resulting in increased infiltration will decrease overall runoff and this can be addressed by modifying the Curve Number.

If the actual impervious area percentage for the proposed design exceeds the proportion assumed for land uses in Table 3.6, a composite CN shall be computed based on actual percentage rather than using the table values.

3.5 Runoff Routing (HEC-HMS) Method

For larger watersheds, runoff routing methodology is required to sub-divide the watershed into small sub-catchments to model the runoff generation and flow routing. These models account for the areal distribution of rainfall, land use, catchment, and stream characteristics. The Hydrologic Modeling system (HEC-HMS) is a free hydrologic modeling software available from the USACE. It accommodates significant complexity, and a wide variety of options are available; therefore it is included as an available method. This method may be applied for developing peak discharge and hydrograph information to use in drainage infrastructure and detention design. For runoff computations, the model provides several options for the following components:

i. Various precipitation models – observed conditions, frequency-based, upper limit event.
ii. Runoff volume estimation models.
iii. Direct runoff models that account for overland flow, storage, and energy losses.
iv. Hydrologic routing models.
v. Modeling of natural confluences and bifurcations.
vi. Water-control measures including diversions and storage facilities.

In HEC-HMS a runoff hydrograph for a subbasin is computed from meteorologic data by subtracting losses and transforming excess precipitation. The preferred method for both the Loss Method and the Transform Method is the SCS Curve Number method. However, other methods or software may be accepted at the discretion of the Director of Planning and Development. Complex systems with multiple basins may also require Reaches to connect Subbasins. Reach elements represent segments of a stream or river through which flow is routed. The selection of the reach routing method should consider channel slope, the influence of backwater and whether there is a need for the model method to account for in-line channel storage. The HEC-MHS user's manual provides detailed descriptions and parameters for the various loss, transform, and routing methods.

3.6 Regression Equations

Regression equations are used to determine annual exceedance probability discharges for un-gaged streams in Arkansas based on upon the physical, climatic, and land use of characteristics of a drainage basin. The US Geological Survey (USGS) periodically updates the regional regression equations based on annual peak-discharge data through the latest available water year. There are four flood regions in Arkansas. Pulaski County is split between Region A and Region D, with the majority of the City of Little Rock being within Region A. The current regional regression equations can be found on their website: https://pubs.usgs.gov/publication/sir20165081.

USGS produces a web application, StreamStats, that can be used to delineate drainage areas and estimate design flows based on the regression equations for the flood region. However, regression equations and StreamStats flows should only be used for preliminary estimates and should not be used for final design.

3.7 Offsite Watershed Basins

For offsite basins that pass through the proposed development, the Rational Method may be used for calculation of design flows for basins up to 200 acres. The Rational Method flows can be used to design conveyance of offsite flows through the site and connecting into existing downstream systems. Onsite and detention flows shall be calculated according to Table 3.1. If the Rational method shows the downstream system is undersized, the Director of Planning and Development may allow onsite over-detention, offsite improvements, or reduced level-of-service for offsite conveyance. For over-detention and offsite improvements, cost sharing will be in accordance with City ordinances.

Table 3.6 TR-55 Runoff Curve Numbers[1] (CN)

Cover type and hydrologic condition[2]	Average percent impervious area[3]	Curve numbers for hydrologic soil groups[1]			
		A	B	C	D
Cultivated land					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow					
Good condition		30	58	71	78
Wood or forest land					
Thin stand, poor cover		45	66	77	83
Good cover		30	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[4]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Urban districts					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (town houses)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (pervious areas only, no vegetation).		77	86	91	94

1. Antecedent Moisture Condition II, and Ia = 0.2S.

2. The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.
3. CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

   Source: USDA Technical Release 55 (TR-55)

3.8 Runoff Analysis Software

Computer software shall be used for stormwater runoff analyses in conformance with design criteria to meet the design standards of the City of Little Rock and this Drainage Criteria Manual. Software such as HEC-HMS, HEC-RAS, EPA-SWMM, XPSWMM, Infoworks ICM, Bentley StormCAD, HydroCAD, Bentley Pond-Pack, Autodesk Storm and Sanitary Analysis or comparable may be used for runoff analyses. Use of two-dimensional (2-D) models, with approval by the Director of Planning and Development, may be used when necessitated by site conditions with complicated overland flow paths or other special circumstances. Within Special Flood Hazard Areas, FEMA-approved hydrologic models should be used. A list of FEMA- approved software can be found on their website: https://www.fema.gov/flood-maps/products-tools/numerical-models

For all submittals, the model input and output data provided shall be clearly, concisely, and consistently organized and labeled based on percent-annual-chance events or design storms. A spatial file or schematic that identifies and references subbasins shall be provided that identifies drainage areas for which data are computed. Minimum output data required shall correspond to the Drainage Report requirements detailed in Appendix A. Example tables depicting required input/output data to be reported are provided within Appendix A.

3.9 Rain-on-Mesh

Traditionally, hydrologic and hydraulic (H&H) models have been developed separately. The hydrologic model estimates inflow boundary conditions (rainfall runoff, inflow hydrographs, etc.) and the hydraulic model routes overland and stream flow to estimate water surface elevations, flow velocities, and flood extents. Recent software developments allow both the hydrology and hydraulics to be conducted within one model framework. Rather than delineating basins and calculating runoff at discreet points, rain-on-mesh or rain-on-grid methodology is where precipitation is applied directly to the 2-D grid or mesh of the model. Generally, there are two approaches to this methodology: excess precipitation and 2-D infiltration. Use of a 2-D rain-on-mesh model must be approved by the Director of Planning and Development.

3.8.1 Excess Precipitation

For the excess precipitation method, infiltration losses based on soil and land use data are calculated for the entire basin in HEC-HMS. Then the excess precipitation hyetograph from the HEC-HMS model is applied to the 2-D mesh in the hydraulic model. This method is not recommended as it does not account for the spatial variations in infiltration because the losses are averaged over the entire basin. It also does not account for the timing variation in runoff from directly connected impervious area.

3.8.2 Two-Dimensional Infiltration

For the two-dimensional infiltration method, infiltration losses are calculated within the hydraulic model. A direct rainfall hyetograph is applied to the 2-D mesh and infiltration is subtracted based on the spatial infiltration layer within the model. The infiltration method used within the 2-D model should be the SCS Curve Number method. Other methods such as Deficit & Constant or Green-Ampt may be used with approval by the Director of Planning and Development. The infiltration layer is developed by combining the land use layer and the soils layer. To properly account for directly connected impervious area, the land use layer should not use a composite curve number. Instead, the land use layer should include the impervious percentage and the runoff Curve Number for the pervious area only. This is especially important in highly developed, urban areas where runoff will occur at the very beginning of storms due to impervious areas that are directly connected to the storm runoff system.

4. Storm Drainage System Design

4.1 General

The purpose of this section is to focus on the proper hydraulic design of storm drains, the collection system, and appurtenances. The storm drainage system consists of inlets, grates, parking lots, street gutters, roadside ditches, small channels and swales, and underground pipe systems which collect stormwater runoff and transport it to structural control facilities and/or the major drainage system (i.e., natural waterways, large man-made conduits, large water impoundments).

This section provides criteria and guidance for the design of minor drainage system components including:

- Street and roadway gutters
- Stormwater inlets and grates
- Storm drainpipe systems

Roadside ditch, channel and swale design criteria and guidance are covered in Chapter 5, *Open Channel Design*.

Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate, curb and combination inlets are based on information contained in HEC-22 (USDOT, FHWA, 2009). Storm drainage system design may be based on either the use of the Rational Formula for gutters and inlets, subject to the area limitations provided in Chapter 3, or the SCS or TR-55 methodologies.

4.2 Storm System Design Requirements

In preparation of storm sewer design, the following list of minimum requirements must be prepared for a storm sewer design:

1. Engineering report displaying relevant calculations and design. See Example Report in Appendix A.
2. A drainage area map at a scale of 1” = 200’ with 2-foot minimum contour intervals using USGS datum for areas less than 100 acres or a plan of the drainage area at a scale of 1” = 500’ with 5-foot minimum contour intervals for larger areas.
3. This plan shall include all existing and proposed street, drainage, and grading improvements with flow quantities and direction at all critical points.
4. All areas and subareas for drainage calculations shall be clearly distinguished.
5. Complete hydraulic data showing all calculations, including a copy of all graphs used for your calculations shall be submitted.
6. A plan and profile of all proposed improvements at a scale of 1” = 50’ horizontal and 1” = 5’ vertical shall be submitted. This plan shall include the following:
    a. Locations, sizes, flowline elevations and grades of pipes, channels.
    b. Boxes, manholes, and other structures drawn on standard plan-profile sheets.
    c. Existing and proposed ground line profiles.
    d. List of the kind and quantities of materials.
    e. Typical sections of all boxes and channels.

   f. Location of property lines, street paving, sanitary sewers, and other utilities.

7. A field study of the downstream capacity is required of all drainage facilities if flows are increased. The effect of additional flow from the area to be improved shall be submitted. If effects endanger property or life, the problem must be resolved before the plan will be given approval. Downstream effects shall be evaluated to the point where the drainage area of the site comprises 10% of the total drainage area.
8. Stormwater flow quantities in the street shall be shown at all street intersections and all inlet openings and locations where flow is removed from the streets.

Stormwater system design shall include the hydraulic calculations for all inlet openings and street capacities. The gutter flow shall be limited according to Section 4.3, Street and Roadway Gutters. Any additional information deemed necessary by the City Design Review Engineer for an adequate consideration of the storm drainage effect on the City of Little Rock and surrounding areas must be submitted.

4.2.1 Design Storm

The design storm for the storm drainage system is the 25-year event. The storm drainage system refers to the infrastructure that collects local runoff which includes inlets, gutters, roadside ditches, swales, and underground pipe. The design storm for cross drainage is based on the road classification in the Master Street Plan. However, if a street is the only means of ingress/egress then the cross drainage shall be sized for the 100-year storm regardless of street classification Cross drainage refers to a pipe, culvert, or bridge that conveys water from one side of a roadway to another.. The current version of the Master Street Plan can be found on the City of Little Rock's website: https://www.littlerock.gov/business/planning-and-development/2023-master-street-plan-update-transportation-development/

Design storm requirements are provided in Table 4.1. The fully developed conditions shall be used to calculate flows for the appropriate design storm frequencies. Reasonable assumptions must be made for off-site flows. The 100-year design storm event shall be used as the check storm to estimate runoff for routing to evaluate effects on the facilities, adjacent property, floodplain encroachment and downstream areas. For the 100-year event, ensure that storm pipe systems will safely convey flows that are in excess of pipe design flows without damaging structures or flooding major roadways. The 100-year storm shall not be conveyed through driveway cuts or across private property but shall remain within the ROW and/or a drainage easement. No fences, portable storage buildings, large landscaping features (i.e., boulders, decorative rock), or other obstructions may be placed within drainage easements.

Table 4.1 Design Storm for Street Classification

Street Classification	Design Storm	
	Cross Drainage	Storm System
Principal Arterials	100-year	25-year
Major and Minor Arterials	100-year	25-year
All other streets	25-year	25-year

Note: Fully developed 100-yr flow must be contained within the right of way (ROW)

4.3 Street and Roadway Gutters

The location of inlets and permissible flow of water in the streets should be related to the extent and frequency of interference to traffic and the likelihood of flood damage to surrounding property. Effective drainage of street and roadway pavements is essential to pavement longevity and traffic safety. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, inlet capacity, and adequate subsurface drainage. The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting gutter equation is:

$$Q = (K_u/n) S_x^{1.67} S_L^{0.5} T^{2.67} \qquad \textbf{Eq. 4.1}$$

Where: Q = flow rate

K_u = 0.56

n = Manning's coefficient (Table 4.2)

S_x = Cross slope (ft/ft)

S_L = Longitudinal slope (ft/ft)

T = Width of flow (spread) (ft)

Table 4.2 Manning's n for Street and Pavement Gutters.

Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.019
Asphalt Pavement:	
Smooth texture	0.013
Rough texture	0.016
Concrete gutter-asphalt pavement:	
Smooth	0.013
Rough	0.015
Concrete pavement:	
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above values of "n" by	0.002

Source: HEC-22

4.3.1 Permissible Spread of Water

Inlets shall be installed at low points and at such intervals to provide the appropriate clear traffic lane per street classification in each direction based upon peak discharges from the 25-year design storm. Minimum lane clearance requirements are provided in Table 4.3. All computations for the 25-year design storm and 100-year, 24-hour storm shall be provided.

Table 4.3 Flow Spread Limits for Inlets

Street Classification	Minimum clear space
Principal and Arterial Streets	Two 12-feet traffic lanes, one in each direction, independent of curb and gutter
Collector Streets	One 12-feet traffic lane within 6 feet of roadway centerline
All other streets	One 10-feet traffic lane within 4 feet of roadway centerline

4.3.2 Flow Bypass

Bypass flow occurs when storm sewer inlets do not capture 100% of the flow upstream of their location. A variety of factors, including organic debris, gutter flow rate, longitudinal slope, and inlet type/geometry, play a role in the capture efficiency of an individual inlet. Flow bypassing each inlet must be included in the total gutter flow to the next inlet downstream. A bypass of 10 to 20% per inlet will result in a more economical drainage system.

4.3.3 Minimum and Maximum Velocities

To ensure cleaning velocities at very low flows, the gutter shall have a minimum longitudinal slope of 0.005 feet per foot (0.5%).

The maximum velocity of gutter flow shall be 10 feet per second. Along sharp horizontal curves, peak flows tend to jump behind the curb line at driveways and other curb breaks. Water running behind the curb line can result in considerable damage due to erosion and flooding. Inlets should be placed upstream of horizontal curves to capture gutter flow and limit flow along the curve.

4.4 Storm Drain Inlets

The primary purpose of storm drain inlets is to intercept excess surface runoff and deposit it in a drainage system, thereby reducing the possibility of surface flooding.

The most common location for inlets is in streets which collect and channelize surface flow making it convenient to intercept. Because the primary purpose of streets is to carry vehicular traffic, inlets must be designed so as not to conflict with that purpose.

The following guidelines shall be used in the design of inlets to be located in streets:

1. Grate inlets shall not be used in a roadway.
2. Inlets shall not be placed on the radius of a curve.
3. Placing inlets downstream of a radius should be avoided.
4. Design and location of inlets shall take into consideration pedestrian and bicycle traffic.
5. Inlet design and location must be compatible with the spread limitations presented in Table 4.3.

4.4.1 Classification

Inlets are classified into three major groups, mainly: inlets in sumps (Type A), inlets on grade without gutter depression (Type B), and inlets on grade with gutter depression (Type C). Each of the three major classes include several varieties, shown in Table 4.4. Recessed inlets are identified by the suffix (R, i.e.: A-1 (R)). The term "continuous grade" or "on grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when street grade is less than 1% or the inlet is located at a low point allowing water to enter from both directions.

The Department of Planning and Development review of a proposed drainage plan shall include examination of the supporting inlet computations. Inlet calculations must be submitted on separate tabulations sheets convenient for review and use of a permanent record in order to speed review.

Table 4.4 Stormwater Inlet Types

Inlets in Sumps	
Type A-1	Curb Opening
Type A-2	Grate
Type A-3	Combination (Grate and Curb Opening)
Type A-4	Drop
Type A-5	Drop (Grate Covering)
Inlets on Grade Without Gutter Depression	
Type B-1	Curb Opening
Type B-2	Grate
Type B-3	Combination (Grate and Curb Opening)
Inlets on Grade with Gutter Depression	
Type C-1	Curb Opening
Type C-2	Grate
Type C-3	Combination (Grate and Curb Opening)

4.4.2 Inlets in Sumps

Inlets in sumps are inlets placed in low points of surface drainage to relieve ponding. Inlets with a 5-inch depression located in streets of less than one percent (1.0%) grade, shall be considered inlets in sumps. The capacity of inlets in sumps must be known in order to determine the depth and width of ponding for a given discharge. Capacity calculations should be based on HEC-22 methodology or manufactures capacity curves.

Inlets in sumps function like weirs for shallow depths. The hydraulic capacity of a curb opening inlet or a vaned grate inlet operating as a weir is expressed as:

$$\boldsymbol{Q_i = C_w L_w d^{1.5}} \qquad \textbf{Eq. 4.2}$$

Where: Q_i = inlet capacity (CFS)

C_w = weir discharge coefficient

Lw = weir length (ft), length of inlet opening acting as a weir

d = flow depth (ft)

Curb opening inlets and drop inlets in sumps have a tendency to collect debris at their entrances. For this reason, the calculated inlet capacity shall be reduced by 20 percent to allow for clogging. Grate inlets have a tendency to clog when flows carry debris such as leaves and papers. For this reason, the calculated inlet capacity of a grate inlet shall be reduced by 50 percent to allow for clogging.

Table 4.5 Sump Inlet Discharge Variables and Coefficients for weir inlets.

Weir Inlet Types	C_w	L_w	Weir equation valid for
Curb opening inlet	3.00	L	d < h
Recessed curb opening inlet	2.30	L+1.8W*	d < h + a
Vane Grate Inlet	3.00	L+2W	$d < 1.79 (A_o/L_w)$

Definition of terms:

L = length of curb opening

h = height of curb opening

d = depth of water at curb opening

W* = lateral width of recessed section

a = depth of curb depression

Ao = clear opening area

W = width of grate

As the depth of stormwater increases, inlet sumps begin to function like an orifice. HEC-22 provides guidance on the transition region based on significant testing. At depths above 1.4 times the opening height, the inlet operates as an orifice and between these depths, transition between weir and orifice flow occurs. The hydraulic capacity of a curb opening inlet or a vane grate inlet operating as an orifice is expressed as:

$$Q_i = C_o A_o \sqrt{2gd} \qquad \textbf{Eq. 4.3}$$

Where: Q_i = inlet capacity (cfs)

C_o = orifice coefficient

A_o = orifice area (ft^2)

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

d = characteristic depth (ft) defined in Table 4.6

Table 4.6 Sump Inlet Discharge Variables and Coefficients for orifice inlets.

Orifice Inlet Types	C_o	A_o	Orifice equation valid for
Curb opening inlet or recessed curb opening inlet	0.67	hL	di* > 1.4h
Vane Grate Inlet	0.67	Clear opening area	d** > 1.79(Ao/Lw)

* di = depth of water at curb opening

** d = depth of water over grate

Note: The orifice area (A_o) should be reduced where clogging is expected.

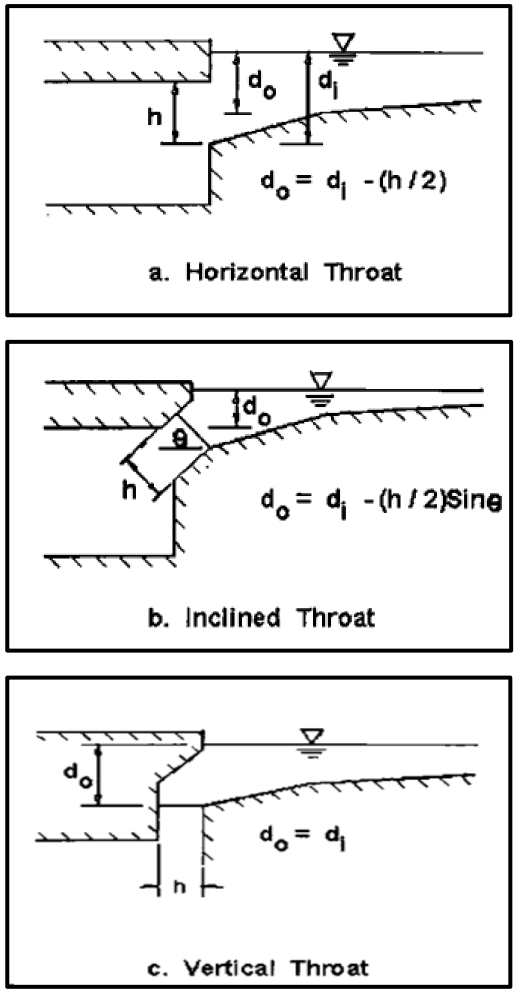


Figure 4.1. Curb-Opening Inlets

4.4.3 Inlets on Grade

Curb opening inlets are effective in the drainage of roadway pavements and in parking lots where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. Street inlets shall be depressed 4 inches with a 12-feet transition upstream and 4-foot transition downstream. Where stormwater flow approaches an arterial street or tee intersection, an inlet is required.

Inlet dimensional requirements: clear throat opening shall be 6 inches in height and 4-feet minimum length. For all throat extensions, clear dimensions shall be 6 inches in height and 3 feet, 6 inches in length. City of Little Rock standard drawings and details shall be used.

Inlets with extensions shall have a maximum clear opening dimension that complies with the City of Little Rock standard drawings. For inlets with extensions, the spacing of 4-inch stools shall not exceed 4 feet in length. If additional length is needed to accommodate City spread and ponding depth requirements, additional inlets shall be added upstream. No clogging factor is required to be applied for curb inlets on grade. For calculation of the interception capacity of inlets on grade, refer to HEC-22.

Flow bypassing each inlet must be included in the total gutter flow to the inlet downstream. A bypass of 10 to 20 percent per inlet will result in a more economical drainage system.

4.4.4 Combination Inlets

The capacity of a combined inlet Type A-3 consisting of a grate and curb opening inlet in a sump shall be considered to be the sum of the capacities. When the capacity of the gutter is not exceeded, the grate inlet accepts the major portion of the flow. Under severe flooding conditions the curb inlet will accept most of the flow.

Combination inlets and sumps have a tendency to clog and collect debris at their entrance. For this reason, the calculated grate capacity of the inlet shall be reduced by 50 percent to allow for this clogging.

4.4.5 Plan and Calculation Submittals

It is important to carefully track runoff flow rates, bypassing flow rates, flow spreads, and other parameters related to gutter flow and inlet capacity for all design storms. Details of grate inlets, net opening, and ratings curves are required to be turned in to the Department of Planning and Development.

4.5 Flow in Storm Drains

Storm drainpipe systems, also known as storm sewers, are pipe conveyances used in the stormwater drainage system for transporting runoff from roadway and other inlets to outfalls at structural stormwater controls and receiving waters. Pipe drain systems are suitable mainly for medium to high-density residential and commercial/industrial development where the use of natural drainageways and/or vegetated open channels is not feasible.

There are several general rules to be observed when designing storm sewer systems. When followed, they will tend to alleviate or eliminate the common issues made in storm sewer design. These rules are as follows:

1. Select pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decrease at inlets, bends or other changes in geometry or configuration. A 15” pipe diameter is the minimum acceptable pipe diameter for maintenance purposes.
2. Do not discharge the contents of a larger pipe into a smaller one, even though the capacity of the smaller pipe may be greater due to steeper slope.
3. At changes in pipe sizes, match the soffits of the two pipes at the same level rather than matching the flow lines.
4. Conduits are to be checked at the time of their design with reference to critical slope. If the slope of the line is greater than critical slope, the unit will likely be operating under entrance control instead of the originally assumed normal flow. Conduit slopes should be kept below critical slope if at all possible. This also removes the possibility of a hydraulic jump within the line.

4.5.1 Hydraulic Grade Line

The water surface elevation, or hydraulic grade line, shall be at least 1 foot below the inlet throat elevation for the design flow. Where required, adjustments shall be made in the system to reduce the elevation of the hydraulic grade line to meet this requirement. All head losses in a storm sewer system including minor losses are considered in computing the hydraulic grade line to determine the water surface elevations, under design conditions, in the various inlets, catch basins, manholes, junction boxes, etc. The starting elevation of the hydraulic grade line shall be set to the tailwater elevation of the receiving stream or waterbody matching the design storm. For example, if the storm pipe system is designed for the 25-year storm, then the tailwater elevation shall be based on the 25-year storm elevation in the receiving channel.

4.5.2 Roughness Coefficients

Any storm drainpipe located in a right-of-way or drainage easement shall be reinforced concrete pipe (RCP) except for side drains which are allowed to be high density polyethylene (HDPE). Table 4.7 below should be used for Manning's n-values for conduits.

4.5.3 Manhole Location

Manholes shall be located at intervals not to exceed 500 feet. Manholes shall preferably be located at street intersections, conduit junctions, changes of grade, changes of horizontal alignment and all changes of pipe sizes. For manholes and junction boxes deeper than 3 feet, steps shall be added.

4.5.4 Minor Head Losses at Structures

The following total energy head losses at structures shall be determined for inlets, manholes, wye branches or bends and the design of closed conduits. Minimum head loss used at any structure shall be 0.10 foot, unless otherwise approved. The basic equation for most cases, where there is both upstream and downstream velocity, takes the form as seen below with the various conditions of the coefficient of K_j shown in Tables 4.8 and 4.9.

$$h_j = K_j \frac{\left|{V_2}^2 - {V_1}^2\right|}{2g} \qquad \textbf{Eq. 4.4}$$

Where: h_j = Junction or structure head loss and feet.

V_1 = Velocity in upstream pipe and feet per second.

V2 = Velocity in downstream pipe and feet per second.

K_j = Junction or structure coefficient of loss.

In the case where the initial velocity is negligible, the equation for head loss becomes:

$$h_j = K_j \left(\frac{V^2}{2g}\right) \qquad \textbf{Eq. 4.5}$$

Short radius bends may be used on 24 inch or larger pipes where flow must undergo a direction change at a junction or bend. Reductions in head loss at manholes may be realized in this way. A manhole shall always be located at the end of such short radius bends.

Table 4.7 Manning's n-values for conduits.

Conduit Material	Manning's n-value
Steel:	
Lockbar and welded	0.012
Riveted and spiral	0.016
Cast Iron:	
Coated	0.013
Uncoated	0.014
Wrought Iron:	
Black	0.014
Galvanized	0.016
HDPE:	
Smooth	0.012
Corrugated	0.024
Corrugated Metal:	
Storm Drain	0.024
Cement:	
Neat, surface	0.011
Mortar	0.013
Concrete:	
Culvert, straight and free of debris	0.011
Culvert with bends, connections, and some debris	0.013
Sewer with manholes, inlet, etc., straight	0.015
Unfinished, steel form	0.013
Unfinished, smooth wood form	0.014
Unfinished, rough wood form	0.017
Wood:	
Stave	0.012
Laminated, treated	0.017
Brickwork:	
Lined with cement mortar	0.015
Sanitary sewers coated with sewage slime with bends and connections	0.013
Paved invert, sewer, smooth bottom	0.019
Rubble masonry, cemented	0.025

Source: Chow, 1959

The values of the coefficient K_j for determining the loss of head due to obstructions in pipe are shown in Table 4.8 and the coefficients are used in the previous equation to calculate the head loss at the obstruction. The values of the coefficient K_j for determining the loss of head due to sudden enlargements and sudden contractions in pipes are shown in Table 4.9 and the coefficients are used in the previous equation to calculate the head loss at the change.

Table 4.8 Minor Loss Coefficients.

Type of Obstruction	Coefficient (K_j)
22.5-Degree Bend	0.20
45-Degree Bend	0.35
60-Degree Bend	0.43
90-Degree Bend	0.50
Straight through Manhole	0.05
Inlet on main line	0.50
Inlet on main line with a lateral branch	0.25

Table 4.9 Minor Loss Coefficients for Junctions.

Type of Junction	Coefficient (K_j)
Junction or manhole on main line with a 22.5-degree lateral branch	0.75
Junction or manhole on main Line with a 45-degree lateral branch	0.50
Junction or manhole on main line with 60-degree lateral branch	0.35
Junction or manhole on main line with 90-degree lateral branch	0.25
Inlet entrance	1.25
Conduit Projecting from Fill, Socket End (Groove End)	0.20
Projecting from Fill, Square Cut End	0.50
Socket End of Pipe (Groove-End)	0.20
Square-Edge	0.50
Rounded	0.20
Mitered to Conform to Fill Slope	0.70

Source: Brazoria County, Texas Drainage Manual

4.5.5 Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent excessive deposition of solid material; otherwise, objectionable clogging may result. The controlling velocity is near the bottom of conduits and considerably less than the mean velocity. Storm drains shall be designed to have a minimum velocity flowing full of 2.5 feet per second (fps). Table 4.10 indicates the grades for both concrete pipe (n = 0.013) and for HDPE pipe (n = 0.024) to produce a velocity of 2.5 fps, which is considered to be the lower limit of scouring velocity. Grades for closed storm sewers and open paved channels shall be designed so that the velocity shall not be less than 2.5 fps nor exceed 12 fps. All other structures such as junction boxes or inlets shall be in accordance with City standard drawings. The minimum slope for standard construction procedures shall be 0.40 percent when possible. Any variance must be approved by the Director of Planning and Development. Closed storm sewers extending to furthest downstream point of development shall consider velocities and discharge energy dissipators to prevent erosion and scouring along downstream properties.

Table 4.10 Minimum slope required to produce scouring velocity.

Pipe Size (Inches)	Concrete Pipe Slope ft/ft	Corrugated HDPE Pipe ft/ft
15	0.0023	0.0076
18	0.0018	0.0060
21	0.0015	0.0049
24	0.0013	0.0041
27	0.0011	0.0035
30	0.0009	0.0031
36	0.0007	0.0024
42	0.0006	0.0020
48	0.0005	0.0016
54	0.0004	0.0014
60	0.0004	0.0012
66	0.0004	0.0011
72	0.0003	0.0010
78	0.0003	0.0009
84	0.0003	0.0008

4.5.6 Utilities

In the design of a storm drainage system, the engineer is frequently confronted with the problem of grade conflict between the proposed storm drain and existing utilities, such as communications, water, gas, and sanitary sewer lines. When conflicts arise between a proposed drainage system and utility system, the owner of the utility system shall be contacted and made aware of the conflict. Any adjustments necessary to the drainage system or the utility can then be determined.

Due to the difficulty and expense to the public with regard to hand cleaning, clearing, and other ditch maintenance, the following ditch requirements are specified to expedite small equipment cleaning and access to drainage easements and ditches:

- Manholes are not allowed in drainage ditches.
- Access Easements shall be required every 500 feet.
- Utility Crossings (above the channel flowline) shall be limited to one per block.
- Utilities shall not be located beneath a concrete bottom except at crossings.

4.5.7 Easements

Drainage easements shall be provided in accordance with the following requirements:

- Drainage easements shall be a minimum of 15 feet.
- For pipe or culverts less than 36-inch in diameter or width, the pipe shall be centered within the easement. For pipes or culverts greater than 36-inch diameter or width, the easement shall provide a minimum of 10 feet from the outside edges of the pipe or culvert on each side.

Minimum widths given above are for installations with depths of cover of 10-feet or less (measured at the top of pipe). For each additional 5-feet of cover over 10-feet (rounded up), the minimum easement width shall be increased by 10-feet.

4.6 References

Flood-Control-and-Water-Quality-Protection-Manual-April-2022 (springfieldmo.gov)

5. Open Channel Design

5.1 General

Open channel systems are an integral part of stormwater drainage design. In addition to natural channels, dry and wet swales, drainage ditches, riprap channels, concrete lined channels, and grass channels, encompass open channels. This section provides an overview of open channel design criteria and methods.

5.1.1 Considerations for Use of Open Channels

Open channels for use in the major drainage system have significant impact to stormwater drainage design in the aspects of cost, capacity, and aesthetic purposes. Disadvantages of open channels include right-of-way needs and maintenance costs.

Open channels may be used in lieu of storm sewers to convey storm runoff where:

- Sufficient right-of-way is available.
- Sufficient cover for the storm sewers is not available.
- It is important to maintain compatibility with existing or proposed developments.
- Economy of construction can be shown without long-term public maintenance expenditures.

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

The ideal channel is a natural one carved by nature over a long period of time and protected through comprehensive stormwater management techniques. The benefits of a natural channel include:

- Lower velocities and increased stability in channel bottom and banks.
- Channel storage tends to decrease peak flows.
- Decrease in maintenance associated with stability.
- Increase in ecological habitat.
- Channel provides desirable green belt.

In general, the man-made channel that most nearly conforms to the character of the natural channel is the most efficient and the most desirable.

In many areas undergoing development, the runoff has been so minimal that natural channels do not exist. However, a small trickle path nearly always exists which provides an excellent basis for location and construction of channels. Utilization of these trickle paths reduces development costs and minimize drainage problems by following grade and working with gravity.

Channel stability is a well-recognized problem in urban settings because of the significant increase in low flows and peak storm discharges.

Sufficient right-of-way or permanent easements should be provided adjacent to open channels to allow entry of city maintenance vehicles.

5.1.2 Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include natural, grass-lined, grass with mulch, sod and lapped sod, and wetland channels. Riprap and gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

<u>Vegetative Linings</u> – Vegetation, where practical, is the preferred lining for man-made channels. It stabilizes the channel body and bed, reduces erosion on the channel surface, and provides habitat and water quality benefits.

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

- High velocities.
- Lack of maintenance required to prevent growth of taller or woody vegetation and invasive species.
- Lack of nutrients and inadequate topsoil.
- Severe lack of access for maintenance.

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. Post construction care of vegetation is critical to successful establishment.

<u>Flexible Linings</u> – Rock riprap, including rubble, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy. These linings are usually less expensive than rigid linings. However, they may require the use of a filter fabric depending on the erosive characteristics of the underlying soils, and the growth of grass and weeds may present maintenance problems. Silty sand or silty loam soils typically require the use of a filter fabric. The US Army Corps of Engineers provides detailed design approach for riprap in Engineer Manual No. 1110-2-1601, Hydraulic Design of Flood Control Channels.

<u>Rigid Linings</u> – Rigid linings are generally constructed of articulated block or concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and may lead to channel head cutting.

5.2 Design Criteria

Open channels shall be designed to the following criteria:

- In all cases for open channels, the design engineer shall calculate the 100-year flow and show the 100-year flow boundary and water surface elevation in the Plans and Specifications.
- Channel or adjacent public drainage/floodplain easement, etc., shall be capable of containing the fully developed 100-year storm with a minimum one-foot freeboard. Public drainage easements should encompass the width of the flow channel, floodplain, etc., with an additional 15 feet on each side of the specified design. For example, if the channel, or floodplain width is 50 feet wide, the drainage easement width at the same point will be 80 feet.
- Trapezoidal or parabolic cross sections are preferred.
- Channel side slopes shall be designed to have a maximum slope of 3:1 unless otherwise justified and designed with proper slope stabilization practices. Roadside ditches should have a maximum side slope of 3:1.
- Channel design shall consider effects of channel lining.
- If a stream channel must be relocated, the cross-sectional shape, meander, pattern, roughness, sediment transport capacity, and slope should conform to the existing conditions to the extent practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
- Streambank stabilization should be provided as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.

5.2.1 Velocity Limitations

The final design of engineered open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for earthen materials categories are presented in Table 5.1. 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 5.1. Erosion Control Matting may be used if designed and constructed in accordance with manufacturer's specifications subject to the limitations provided in this manual.

Table 5.1 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 Design Review Engineer.

Table 5.2 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	0-10	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.
5. Source: Manual for Erosion and Sediment Control in Georgia, 1996.

5.2.2 Channel Cross Sections

The channel shape may be almost any type suitable to the location and to the environmental conditions. The shape may be able to be designed to promote open space, recreational needs and to create additional benefits.

1. Bend Radius: 25 feet or 10 times the bottom width, whichever is larger, is the minimum bend radius required for open channels.
2. Freeboard: Freeboard to top of bank shall be based on velocities associated with the design storm and shall be a minimum of 1 foot for channel velocities up to 8 ft/s and 2 feet for velocities exceeding 8 ft/s at the design storm. For deep flows with high velocities, greater freeboard shall be required, calculated in accordance with the following formula:

$$\boldsymbol{Freeboard\ (ft) = 1.0 + 0.025\, vD^{1/3}} \qquad \textbf{Eq. 5.1}$$

Where: v = velocity of flow (ft/s)

D = depth of flow (ft)

For freeboard of a channel on a sharp curve less than the minimum bend radius, additional freeboard, to account for superelevation of the water surface, shall be computed as:

$$\boldsymbol{H = v^2\,((T + b)/2gR_c)} \qquad \textbf{Eq. 5.2}$$

Where: H = additional height on the outside edge of channel (ft)

v = velocity of flow (ft/s)

T = top width of water surface (ft)

b = bottom width of channel (ft)

g = acceleration of gravity (32.2 ft/s2)

Rc = mean radius of bend (ft)

3. Connections: Connections at the junction of two or more open channels shall be designed to minimize transition loss for both vertical and horizontal transitions. Pipe and box culverts or sewers entering an open channel shall not project into the normal channel section. Nor will they be permitted to discharge into an open channel at an angle that directs flow upstream.

5.2.3 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. Downcutting and lateral cutting just downstream from the drops is a common problem which must be protected against. FHWA's HEC-14 manual and programs like HY-8 can be used to calculate scour potential and design energy dissipators.

5.2.4 Baffle 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. Refer to FHWA's HEC-14 manual for baffle design.

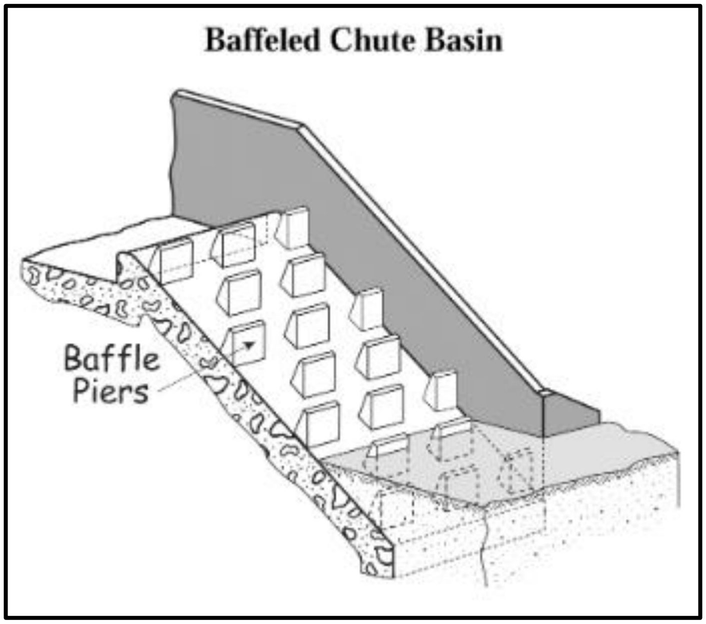


Figure 5.1. Example of Baffle Chute Basin

5.3 Computation and Software

Computer programs that utilize the Manning's equation shall be used for open channel design. Computer programs such as Hydraflow Express or the FHWA Hydraulic Toolbox may be used for Uniform Flow conditions; however, for more complex reaches or streams with higher flows, a backwater model such as HEC-RAS should be used. The general information to be provided in an open channel design is:

- Plan View - Existing and proposed topography.
- Profile - Left and right top of bank, 100-yr HGL, slope, invert flowlines.
- Cross-section - Dimensions, 100-yr hydraulic parameters.

Design flow and applicable design standards, design geometry required based on operational characteristics – freeboard, velocity, minimum standard capacity and site requirements, and flow regime – subcritical or supercritical – shall be reported and taken into consideration as part of design. Table 5.3 is a sample output file using Hydraflow Express computer software; FHWA Hydraulic Toolbox software provides a similar output report.

Table 5.3 Channel report output file.

Channel Section			
Channel Section Data:		**Highlighted:**	
Bottom Width (ft)	2.00	Depth (ft)	0.80
Side Slopes (z:1)	3.00, 3.00	Q (cfs)	13.00
Total Depth (ft)	2.00	Area (sq ft)	3.52
Invert Elevation (ft)	100.00	Velocity (ft/s)	3.69
Slope (%)	1.00	Wetted Perimeter (ft)	7.06
N-Value	0.025	Critical Depth, Yc (ft)	0.77
		Top Width (ft)	6.80
Calculations:		EGL (ft)	1.01
Compute by:	Known Q		
Known Q (cfs)	13.00		

5.3.1 Manning's n Values

Recommended Manning's n values for artificial channel linings are given in Table 5.4. The Values in Table 5.4 are based off the flow depth of the channel. For natural channels, earthen channels, and various types of vegetation, Manning's n values should be estimated using experienced judgment and based on the information in Table 5.5. 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 5.4 Manning's roughness coefficients (n) for artificial lined 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	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	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	0.040

Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary with the flow depth.

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

Table 5.5 Uniform flow values of roughness coefficient (n) for natural channels.

Type of Channel and Description	Minimum	Normal	Maximum
Natural Streams - Minor streams (top width at flood stage < 100 ft)			
1. Main Channels			
a. Clean, straight, full stage	0.025	0.030	0.033
b. Same as above, but some stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Clean, winding, but some weeds and some stones	0.035	0.045	0.050
e. Same as 4, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as 4, but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
2. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
a. Bottom: gravels, cobbles, few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070
3. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated 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

Table 5.5 Uniform flow values of roughness coefficient n for natural channels.

Type of Channel and Description	Minimum	Normal	Maximum
5. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds/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

5.3.2 Channel Discharge - Manning's Equation

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

$$V = (1.49/n)\, R^{2/3}\, S^{1/2} \qquad \textbf{Eq. 5.3}$$

$$Q = (1.49/n)\, AR^{2/3}\, S^{1/2} \qquad \textbf{Eq. 5.4}$$

$$S = \left[Qn\, /\, (1.49\, AR^{2/3})\right]^2 \qquad \textbf{Eq. 5.5}$$

Where: V = average channel velocity (ft/s)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

A = cross-sectional area (ft2)

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 complicated by bridge openings, curbs, and structures. Typically backwater computations will be required for channel design work; however, a check should also be performed for velocity based on headwater-controlled conditions.

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 calculation procedures, 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 Director of Planning and Development or the Public Works Director, channel velocities in man-made channels shall not exceed those specified in Tables 5.1 and 5.2. 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 expeditiously after grading has been completed.

5.4 Channel Lining Design

5.4.1 Vegetative Design

For channels with vegetative and temporary lining, design stability shall be determined using Manning's n based upon poor vegetation conditions and for design capacity better conditions should be used. Channel velocities shall not exceed the maximum permissible velocities given in Tables 5.1 and 5.2. For more details on vegetative design refer to HEC-15.

5.4.2 Riprap Design

Where the use of riprap is allowed by the City Design Review Engineer, riprap sizing shall be determined based on maximum anticipated channel velocities. Adequate erosion protection shall be provided for the design configurations. For example, if riprap will extend into a stream with higher water surface elevations and/or velocities, i.e., at a pipe outfall going into a creek, then the riprap must be sized to resist the forces of the higher flow in the creek. 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.

Isbash Equation

The Isbash formula (Isbash 1936) was developed for the construction of dams by depositing rocks into moving water. The Isbash curve should only be used for quick estimates or for comparisons. A coefficient is provided to target high- and low-turbulence flow conditions, so this method can be a high- or low-energy application. The equation is:

$$\boldsymbol{V_c = C\left[2g\left(\frac{\gamma_S - \gamma_W}{\gamma_W}\right)\right]^{0.50}(D_{50})^{0.50}} \qquad \textbf{Eq. 5.6}$$

Where: V_C = critical velocity (ft/s)

C = 0.86 for high turbulence

C = 1.20 for low turbulence

g = 32.2 ft/s^2

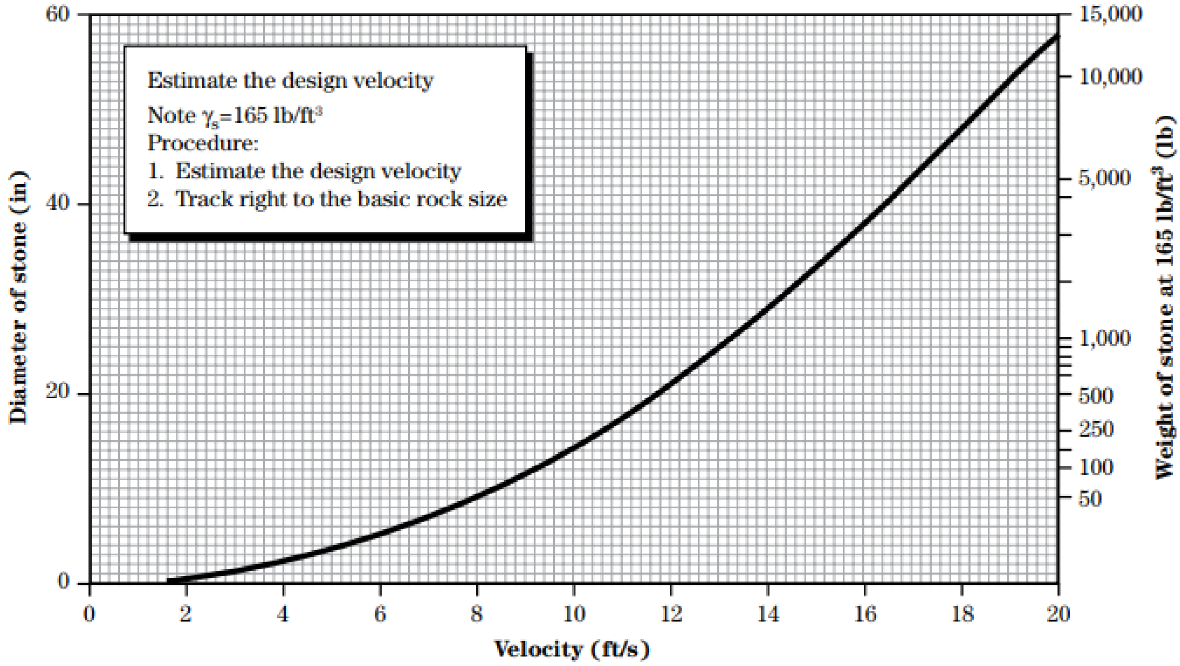
γ_S = stone density (lb/ft^3)

γ_W = water density (lb/ft^3)

D_{50} = median stone diameter (ft)

Figure 5.0 provides general riprap sizing criteria. For more detailed design, reference the US Army Corps of Engineers *Hydraulic Design of Flood Control Channels* manual. The design velocity should be based on the highest velocity of the design storm events, include velocities in receiving stream, if applicable. Extend a vertical line from the x-axis of the figure at the appropriate velocity until the curve is intersected, then extend a horizontal line to intersect the y-axis at the corresponding D_{50}, or median stone diameter for which no more than 50% of the stone by weight is smaller.

Figure TS14C–5 Rock size based on Isbash curve



(210–VI–NEH, August 2007)

Figure 5.3. Riprap sizing curve.

5.5 References

Chow, V.T., 1959, Open-channel hydraulics: New York, McGraw-Hill Book Co., 680 p.

Outlet Erosion Control Structures (Stilling Basins) | Association of State Dam Safety

Natural Resource Conservation Service, 2007. National Engineering Handbook, Part 654.

Drainage-Criteria-Manual-2014-PDF (fayetteville-ar.gov)

6. Culvert Hydraulics

6.1 General

A culvert is defined as a short conduit used to convey stormwater runoff under an embankment such as a roadway or driveway whose primary purpose is to convey surface water. Alongside the hydraulic capabilities of culverts, a culvert must support the embankment and/or roadway while also protecting traffic and adjacent property owners from flood hazards to the extent practicable.

6.1.1 Criteria for Use of Culverts

Culvert design shall be based upon peak discharges for the appropriate design storm based on roadway type. Requirements are provided in Table 6.1. All computations, hydraulic profiles, and energy transition to channel shall be provided for the design event and the 100-year storm check.

Table 6.1 Culvert and Bridge Sizing Requirements

Roadway Classification	Design Storm Event	Minimum Freeboard (Culvert)	Minimum Freeboard (Bridges)
Principal Arterial Streets	100-year	1 foot	1 foot
Major and Minor Arterial Streets	100-year	1 foot	1 foot
All other streets	25-year	1 foot	1 foot

Note: Freeboard for culverts shall be from top of low point in road. Freeboard for Bridges shall be measured from the low chord.

Route the fully developed conditions 100-year frequency storm through all culverts to be sure building structures (i.e., houses, commercial buildings) are not flooded or increased damage does not occur to the roadway or adjacent property for the 100-year storm event. The runoff generated from the 100-year event shall be safely conveyed through drainage easements and/or the Right of Way. Appropriate tailwater conditions shall be used from receiving waters. See section 6.2.5, tailwater considerations.

6.2 Design Criteria

6.2.1 Velocity Requirements

The final design of culverts should consider the minimum and maximum velocities. A minimum velocity of 3 ft/s is required when a culvert is flowing partially full to ensure no siltation occurs. There is no maximum velocity constraint, however; if velocities exceed 10 ft/s, chances of abrasion due to bedload movement and erosion downstream increase significantly. When velocities exceed the permissible velocity for the receiving channel type, energy dissipators are necessary and should be included in the culvert design. Energy dissipators are discussed in Section 6.2.2.

6.2.2 Energy Dissipators

To prevent scour at stormwater outlets, protect the outlet structure, and minimize the potential of downstream erosion, energy dissipators are required to reduce the flow to a non-erosive velocity. Some common types of energy dissipators include:

- **Rock-Protected Outlets**

  Rock is often placed around the outlet of culverts to protect against the erosive action of the water. Typical placement of rock protection is shown in Figure 6.1. The material size used is dependent on the velocity and should be determined using a full flow analysis as noted in Table 6.2. Riprap is required to have a minimum depth of 12 inches.

- **Other Energy-Dissipating Structures**

  Other structures include baffled outlets, plunge pools, internal dissipators, impact basins, and stilling basins designed according to the FHWA's HEC-14, "Hydraulic Design of Energy Dissipators for Culverts and Channels."

Energy dissipators should be analyzed and designed using HY-8 Culvert Hydraulic Analysis Program or an approved equivalent.

Energy dissipators are known to collect debris so the possibility of debris collection should be considered when choosing a dissipator design. Dissipators should be kept open and easily accessible to maintenance crews and provisions should be made to allow water to overtop without causing excessive damage.

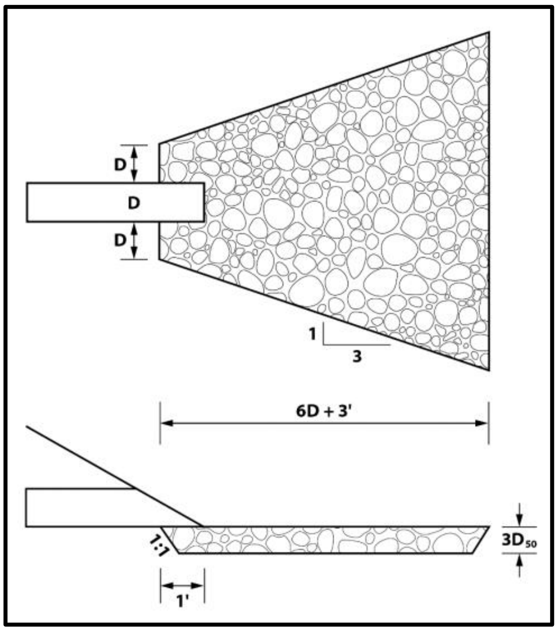


Figure 6.1. Typical Rock Protection Placement

Table 6.2 Outlet Protection Material Size

Outlet Velocity (ft/s)	Material
Up to 10	Dumped Riprap
>10	Foundation Protection Riprap

Note: All rock protection shall be designed in accordance with ARDOT Standard Specifications for Highway Construction Section 816.

6.2.3 Length and Slope

The maximum culvert slope using a reinforced concrete pipe shall be 10%. For culverts with a slope greater than 10%, the culvert must be approved by the Director of Planning and Development or Design Review Engineer to ensure proper pipe restraints. while the minimum slope for standard construction procedures shall be 0.5% when possible. Maximum drop in a drainage structure or junction box is 10 feet unless approved by the Director of Planning and Development or Design Review Engineer.

6.2.4 Headwater Limitations

Headwater is the water above the culvert invert at the entrance of the culvert. Headwater will be non-damaging to adjacent property and/or roadways. The maximum permissible headwater is determined based on hydraulic evaluation and the proposed or existing roadway elevation and is the primary basis for sizing a culvert.

The following headwater criteria apply to culvert design:

- The allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood.
- Headwater shall have no adverse impact on upstream property.
- Maximum headwater depth for the design storm shall be 1 foot lower than the lowest top of road or curb elevation.
- Ponding depth shall be no greater than the elevation where flow diverts around the culvert.
- For drainage facilities with a cross-sectional area equal to or less than 30 sq.ft., headwater to depth ratio (HW/D) should be equal to or less than 1.5.
- For drainage facilities with a cross-sectional area greater than 30 sq.ft., HW/D should be equal to or less than 1.2.
- The headwater should be checked against the 100-year flood (base flood) elevation to ensure compliance with floodplain management criteria.
- The culvert should be sized to maintain flood-free conditions on principal and minor arterials with 1-foot freeboard from the low point of the road.
- Identify the maximum acceptable outlet velocity, based on receiving channel conditions. Reference Section 5.2.1 to determine acceptable velocities based on channel type.
- The constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.
- Bridges require 1 foot of freeboard from the low chord.

6.2.5 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine tailwater depth for a range of discharge for the appropriate design storm and the 100-yr storm. At times, there may be a need for calculating backwater curves to establish the tailwater conditions. When evaluating the tailwater, the following must be considered:

- If the culvert outlet is operating with a freefall outfall, the critical depth and hydraulic grade line shall be determined.
- For culverts that discharge into an open channel, the water surface elevation in the open channel for the relevant design storm events. Tailwater conditions for all required flood events should be evaluated as a part of the culvert capacity and velocity computations. See Chapter 5, Open Channel Design.

- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.
- If the culvert discharges to a lake, pond, or other major water body, the expected high-water elevation for the design storm of the water body may establish the culvert tailwater.

6.2.6 Culvert End Treatments

The culvert inlet often has a significant impact on the culvert's hydraulic capacity, efficiency, and cost. The inlet coefficient, $K_{e,}$ is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Table 6.3 provides recommended inlet coefficients.

Culvert end treatments are required for all culverts installed in public right of ways or drainage easements. Some common end treatments include:

Headwalls
Headwalls shall be constructed with reinforced concrete. Straight, flared, and warped headwalls are all permissible depending on site conditions. Headwalls are required to be included in the culvert design when culverts cross the embankments at angle of 15-degrees or greater.

Wingwalls
Wingwalls are required when the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

Aprons
If the approach velocity in the channel will cause scour, channel aprons at the toe are required to be included in the culvert design. Aprons shall extend a minimum of one pipe diameter upstream from the culvert entrance. The top of apron elevation shall not protrude above the normal streambed elevation.

6.2.7 Size and Material Selection

Reinforced concrete pipe (RCP) is to be used in roadway areas including under curbs. Sections where other material is used must be approved by the City Design Review Engineer. Polyvinyl chloride pipe (PVC) and high-density polyethylene pipe (HDPE) may be used in non-roadway areas. Galvanized CMP is not permissible and shall not be used in any culvert design. Coated CMP and HDPE flared end sections are prohibited within right of ways and drainage easements. All pipes shall be installed according to the standard details provided by the City of Little Rock including bedding, backfill, and compaction. Details for bedding, backfill, and compaction must be included in the Plans and Specifications.

The minimum allowable circular pipe diameter shall be 18 inches for culverts.

6.3 Design Procedure

6.3.1 Flow Type

Inlet and outlet control are the two basic types of flow control defined by the FHWA. The characterization of pressure, subcritical, and supercritical flow regimes play an important role in determining the control type. The control type also plays a significant role in determining the hydraulic capacity of a culvert. Proper culvert design requires checking for both inlet and outlet control to determine which will govern culvert designs.

Inlet Control

Inlet control occurs when the culvert barrel can convey more flow than the inlet will accept. In this control, critical depth occurs just inside the entrance of the culvert and the flow regime immediately downstream is supercritical. The upstream water surface elevation and the inlet geometry represent the major flow controls. Figure 6.2 depicts a typical inlet control flow section.

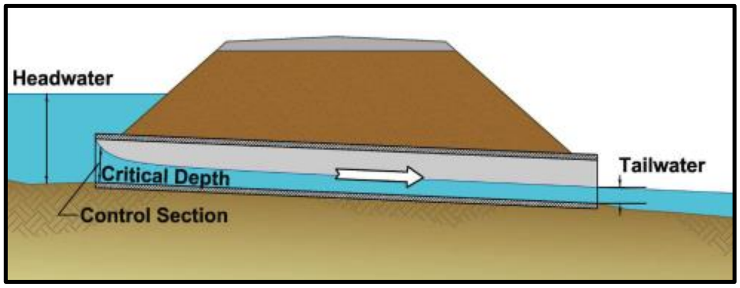


Figure 6.2. Typical Inlet Control Flow Section

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 is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. All geometric and hydraulic control characteristics, including all factors governing inlet control, water surface elevation at the outlet, and barrel characteristics, play a role in determining the culvert's capacity. Figure 6.3 depicts two typical outlet control flow sections.

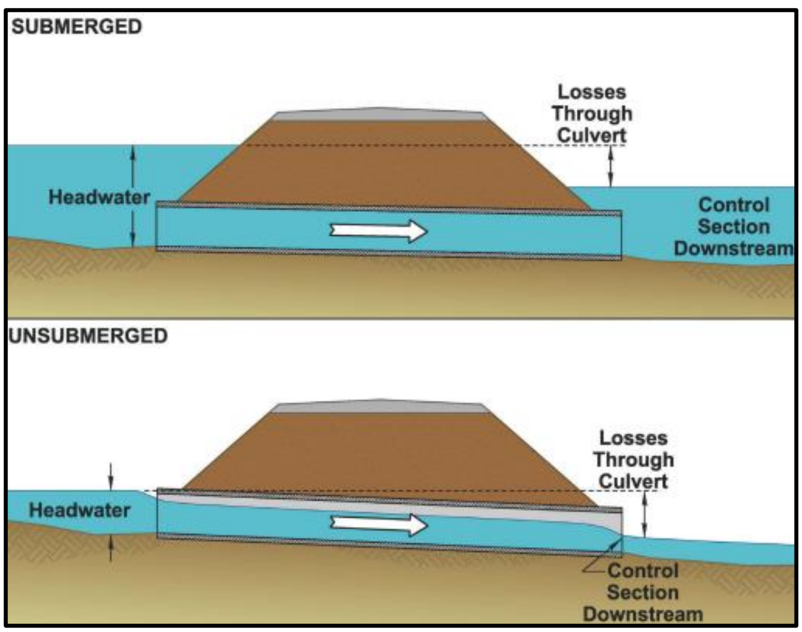


Figure 6.3. Typical Outlet Control Flow Sections

6.4 Design Software

It is recommended to use HY-8 Culvert Hydraulic Analysis Program developed by the Federal Highway Administration for all culvert design and analysis. Additional software may be accepted for use by the City Design Review Engineer provided it is shown to be equivalent to HY-8. For culvert crossings along creeks, include the culvert crossing in the HEC-RAS analysis.

6.4.1 Design Procedure

The following design procedure should be conducted using HY-8 or an approved equivalent.

Step 1. – List Design Input 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
Material Type	HW = allowable headwater depth (ft)

Step 2. – Determine Trial Size.

Assume a trial velocity of 3-5 ft/s and compute the culvert area using A = Q/V. Determine the culvert shape, open size (diameter or span and rise), and number of barrels.

Step 3. – Calculate HW for Inlet and Outlet Control

For inlet control, enter inlet control data into the software with D and Q and find HW/D for the entrance type. If HW is too large, adjust the opening size and recompute until the HW is acceptable.

For outlet control, enter the outlet control data into the software with the culvert length, entrance loss coefficient, and trial culvert diameter. Use Equation 6.1 to compute the HW elevation.

$$HW = H + h_0 - LS \qquad \textbf{Eq. 6.1}$$

Where: h_0 = ½ (critical depth + D) or tailwater depth, whichever is greater.

Step 4. – Determine if the culvert is under Inlet or outlet control.

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 repeat the steps. Unless material or entrance conditions change, the inlet control conditions for the larger pipe does not need to be rechecked.

Step 5. – Check Potential Scour.

Calculate the exit velocity and if erosion problems are expected. Modify the culvert size to eliminate the erosion problems. If erosion problems cannot be eliminated, refer to section 6.2.2 for appropriate energy dissipation design.

6.4.2 Multi-barrel Installations

For culverts installations with multiple barrels exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of a multi-barrel culvert is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert. Multi-barrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

For multi-barrel culvert installations, one culvert in the middle of the channel should be set at the elevation of the existing flowline and the remaining culverts should be set at the elevation of the bank full channel. Figure 6.4 provides an example of a multi-barrel culvert installation.

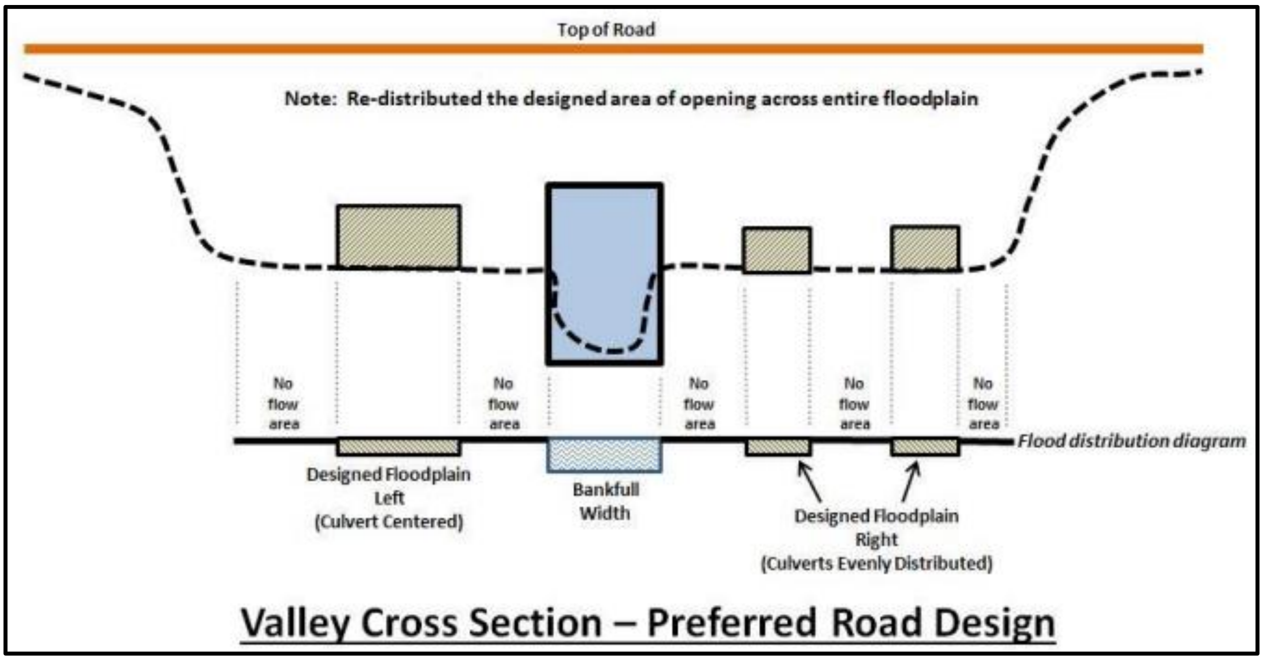


Figure 6.4. Road bisecting the floodplain with floodplain culverts.

Table 6.3 Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient Ke[1]
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)]	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7o or 45 o bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal[1]	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7 o or 45 o bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or beveled edges on 3 sides	0.2
Wingwalls at 30^{o} to 75^{o} to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10^{o} or 25^{o} to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides) Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

1. The K_e values for corrugated metal pipes are also recommended for HDPE pipes.
Source: HDS No. 5, 1985.

6.5 References

https://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf

https://wsdot.wa.gov/publications/manuals/fulltext/m23-03/chapter3.pdf

7. Stormwater Detention

7.1 General

This section provides guidance on stormwater runoff storage for meeting stormwater management control requirements (i.e., water quality treatment, downstream channel protection, overbank flood protection, and extreme flood protection).

Stormwater detention within a stormwater management system is essential to provide the required flow reduction for water quality treatment and downstream channel protection, as well as for peak flow attenuation of larger flows for overbank and extreme flood protection. Runoff storage can be provided within an on-site system through the use of structural stormwater controls and/or nonstructural features and landscaped areas. Additional design guidance is provided in Chapter 8, Water Quality.

7.2 Method of Evaluation

Pre-development and post-development runoff shall be calculated to evaluate the use of stormwater detention. Runoff shall be evaluated for the 2-, 25-, and 100-year 24-hour storm events. For areas less than 40 acres, the Modified Rational Method may be used to evaluate pre-development and post-development runoff conditions. Areas greater than 40 acres shall be evaluated using the SCS TR-55 Method.

If another method is used, the Owner's Engineer shall submit the proposed method of evaluation for the sizing of the retention basin or detention basin to the City Design Review Engineer. The method will be evaluated for professional acceptance, applicability, and reliability by the City Design Review Engineer. No detailed review will be rendered before the method of evaluation of the retention or detention basin is approved.

7.2.1. The Modified Rational Method

The Modified Rational Method uses the peak flow calculating capability of the Rational Method paired with assumptions about the inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations. This approach has many variations. Figure 7.0 illustrates one application. The rising and falling limbs of the inflow hydrograph have a duration equal to the time of concentration (t_c). An allowable peak outflow is set (Q_a) based on pre-development conditions. The storm duration is t_d and is varied until the storage volume (shaded gray area) is maximized.

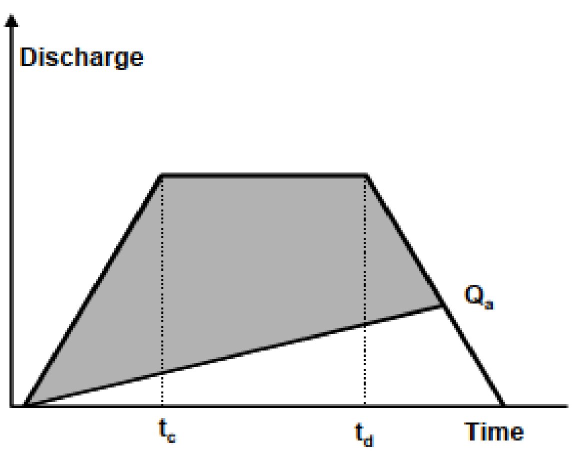


Figure 7.0. Modified Rational Definitions

The peak runoff rate can be determined from:

$$q_{pi} = C_a \, i \, A \qquad \textbf{Eq. 7.0}$$

Where: q_{pi} = peak runoff from site (cfs)

C_a = post-development Rational Method runoff coefficient

i = rainfall intensity for the corresponding time of concentration (in/hr)

A = area (acres)

It is assumed the peak of the outflow hydrograph falls on the recession limb of the inflow hydrograph and the rising limb of the outflow hydrograph can be approximated by a straight line.

The storage volume is determined by the critical (inflow) duration and using a constant outfall release rate. With these assumptions:

$$S_d = q_{pi} t_d - \frac{Q_a(t_d + t_c)}{2} \qquad \textbf{Eq. 7.1}$$

Where: S_d = detention volume required (ft^3)

Qa = allowable peak (pre-development) outflow rate (cfs)

td = design storm duration (sec)

tc = time of concentration for the watershed (sec)

The design storm duration is the duration that maximizes the detention storage volume, S_d, for a given return period. The storm duration can be found by trial and error using rainfall data from NOAA Atlas 14. This is normally an iterative process done by hand or with a spreadsheet. Software such as HydraFlow Hydrographs will compute storm durations and determine the estimated volume for a storm event. Downstream analysis is not possible with this method, as only approximate graphical routing takes place.

7.2.2. Flood Routing

The most commonly used method for calculating detention basin volume is to route an inflow hydrograph through a detention pond utilizing the Storage Indication or modified Puls method. This method compares the difference in the average values of two closely spaced inflows and outflows, yielding the change in storage over a given time period. By continuing this process for the duration of the storm and beyond, the total required storage for the basin can be determined. This is the methodology utilized by HEC-HMS, WinTR-55, and other hydrology software and can also be completed through the use of a spreadsheet. A detailed description of the manual process for routing a storm through a detention basin is presented in Chapter 8 of FHWA's HEC-22.

7.3 Detention Volume

The detention volume calculated using the Modified Rational method should not be used for final design. The final design should be verified by using an appropriate software to route the inflow hydrograph and determine if the proposed volume is adequate. Software such as HEC-HMS, Hydraflow Hydrographs, and others have capabilities to route hydrographs through detention basins.

The volume of the basin is determined by developing a hydrograph and routing the design storm through the basin. If the design storm can be routed through the basin without overtopping or exceeding the freeboard requirements, the basin volume is adequate. If the routing procedure indicates the storage elevation of the basin exceeds the freeboard requirements or overtops the basin, additional volume in the basin is required.

The final design of a detention facility requires three items:

- an inflow hydrograph
- a stage vs. storage curve
- a stage vs. discharge curve

1. To check the capacity of a basin with a known volume, use the methods described in the previous sections.
   a. Develop an inflow hydrograph for the storm in question.
   b. Develop the stage-storage and stage-discharge curves for the basin.
   c. Route the storm through the basin to determine the outflow hydrograph. Check the peak of the outflow hydrograph to ensure that it does not exceed the allowable value. Also, check the peak storage volume to ensure that it does not exceed the capacity of the basin.

2. Analyzing a known basin utilizing the methods developed in the previous sections is relatively straightforward. However, determining the required size of a proposed basin is an iterative process, and can be quite time consuming without a method to develop a preliminary volume estimate. TR-55 provides a method for determining quick estimates of detention basin volumes.

   a. Figure 7.1 relates two ratios: peak outflow to peak inflow (q_o/q_i) and storage volume to runoff volume (V_s/V_r). The value for q_i is determined by the peak of the inflow hydrograph. The value for q_o is normally dictated by the allowable release rate. The volume of runoff can be calculated by the SCS method or tabular hydrograph method. The relationships in Figure 7.1 were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data.

   b. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that:
      1) Each stage requires a design storm and a computation of the storage required for it.
      2) The discharge of the upper stage(s) includes the discharge of the lower stage(s).

   c. The brevity of the procedure allows the designer to examine many combinations of detention basins. When combined with the Tabular Hydrograph Method, the procedure's usefulness is increased. Its principal use is to develop preliminary indications of storage adequacy.

      This estimating technique becomes less accurate as the q_o/q_i ratio approaches the limits shown in Figure 7.1. The curves in Figure 7.1 depend on the relationship among available storage, outflow device, inflow volume, and shape of the inflow hydrograph. When the storage volume (V_s) required is small, the shape of the outflow hydrograph is sensitive to the rate of the inflow hydrograph. Conversely, when V_s is large, the inflow hydrograph shape has little effect on the outflow hydrograph. In such instances, the outflow hydrograph is controlled by the hydraulics of the outflow device and the procedure therefore yields consistent results. When the peak outflow discharge (q_o) approaches the peak inflow (q_i), the parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant.

The procedure should not be used to perform final design if an error in storage of 25% cannot be tolerated. Figure 7.1 is biased to prevent under sizing of outflow devices, but it may significantly overestimate the required storage capacity. More detailed hydrograph development and routing will often pay for itself through reduced construction costs.

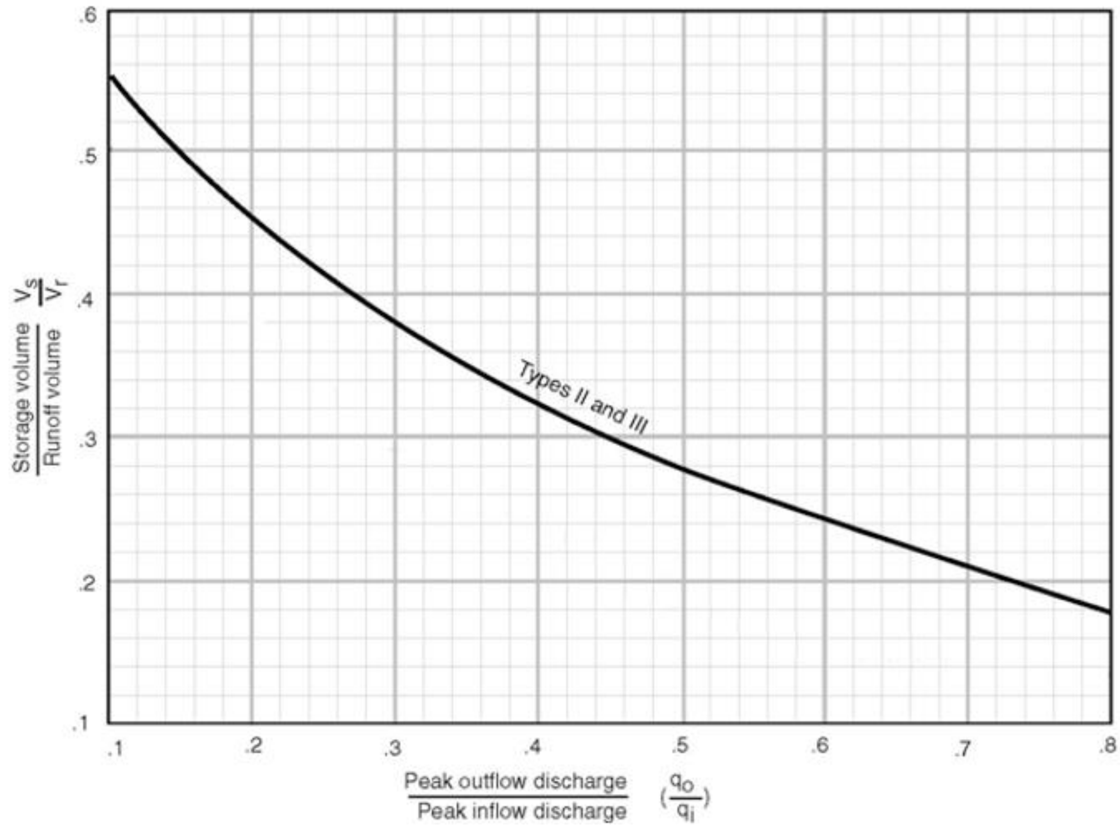


Figure 7.1. Approximate Detention Basin Routing for Type II and III

d. The purpose of Figure 7.1 is to provide a starting point for the size of the basin. The process may have to be repeated several times to achieve a basin that has sufficient volume and meets specific inlet and outlet controls.
e. Little Rock falls within the Type II rainfall category.

7.4 Methods of detention

Detention storage may be categorized as inline or offline. The City of Little Rock only allows inline storage if it can be demonstrated that offline storage is not practicable. Cost is not a justification to deem a design as not practicable; site constraints or other technical considerations are required for consideration of allowing inline storage. Figure 7.2 illustrates inline versus offline storage.

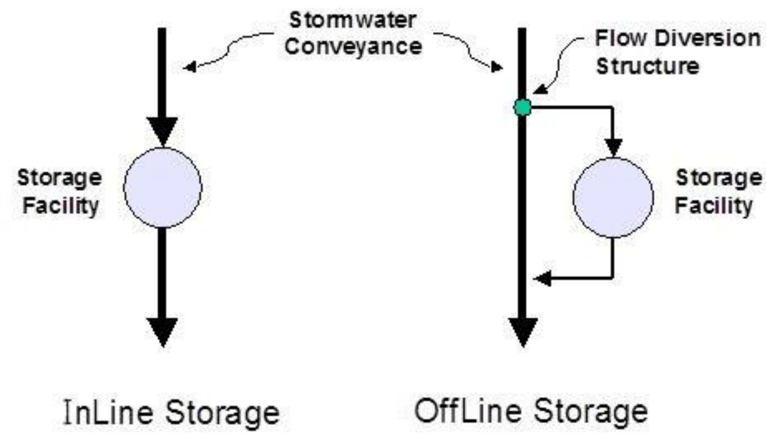


Figure 7.2. Depiction of inline versus offline storage

7.4.1. Structural Controls Appropriate for Detention

The following sections list the structural control practices appropriate for detention that are approved for use in the City of Little Rock. Mosquito control measures shall be taken for all proposed ponds. Avoid creating areas of shallow stagnant water and low dissolved oxygen which create mosquito habitat. To avoid creating a mosquito habitat, for wet detention, pools of water should be at least 5 feet deep and for dry detention hydraulic residence time should be less than 72 hours.

7.4.1.1. Stormwater Ponds

Stormwater ponds (also referred to as *retention ponds*, *wet ponds*, *or wet extended detention ponds*) are constructed stormwater retention basins that have a permanent (dead storage) pool of water throughout the year. They are categorized in this Manual as water quality structural controls and can meet the intent of the water quality criteria, however; they also can provide detention storage to meet the other stormwater criteria (Section 2.1).

In a stormwater pond, a certain design volume of runoff from each rain event is detained and treated in the pool through gravitational settling and biological uptake until it is displaced by runoff from the next storm. The permanent pool also serves to protect deposited sediments from re-suspension. Above the permanent pool level, additional temporary storage (live storage) is provided for runoff quantity control. Stormwater ponds are among the most cost-effective and widely used stormwater practices. A well-designed and landscaped pond can be an aesthetic feature on a development site when planned and located properly.

The most common of stormwater pond designs include the wet pond, the wet extended detention pond, and the micropool extended detention pond. In addition, multiple stormwater ponds can be placed in series or parallel to increase performance or meet site design constraints.

7.4.1.2. Stormwater Wetlands

Stormwater wetlands (also referred to as constructed wetlands) are constructed shallow marsh systems that are designed to both treat urban stormwater and control runoff volumes. As stormwater runoff flows through the wetland facility, pollutant removal is achieved through settling and uptake by marsh vegetation.

Stormwater wetlands are categorized as water quality structural controls to meet the water quality criteria, however; they also can provide detention storage to meet the other stormwater criteria (Section 2.1).

Wetlands are an effective stormwater practices in terms of water quality and offer aesthetic value and wildlife habitat. Stormwater wetlands require a continuous base flow or a high-water table to support aquatic vegetation. There are several design variations of the stormwater wetland, each design differing in the relative amounts of shallow and deep water, and dry storage above the wetland. These include the shallow wetland, the extended detention shallow wetland, pond/wetland system and pocket wetland.

7.4.1.3. Dry Detention / Dry ED Basins

Dry detention and dry extended detention (ED) basins are surface facilities intended to provide for the temporary storage of stormwater runoff to meet the downstream flood protection criteria. These facilities temporarily detain stormwater runoff, releasing the flow over a period of time. They are designed to completely drain following a storm event and are normally dry between rain events.

Both dry detention and dry ED basins provide limited pollutant removal benefits and are not intended for water quality treatment. Detention-only facilities should be used in a treatment train approach with other structural controls to provide water quality treatment.

7.4.1.4. Multi-purpose Detention Areas

Multi-purpose detention areas are site areas primarily used for one or more specific activities that are also designed to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. Examples of multi-purpose detention areas include:

- Sports Fields
- Recessed Plazas

Multi-purpose detention areas are normally dry between rain events, and by their nature must be usable for their primary function the majority of the time. As such, multi-purpose detention areas should be used for meeting the downstream flood protection criteria, but not for water quality criteria.

Multi-purpose detention areas should be used in a treatment train approach with other structural controls to provide water quality treatment.

7.4.1.5. Underground Detention

Underground detention facilities such as vaults, pipes, tanks, and other subsurface structures are designed to temporarily store stormwater runoff for water quantity control. As with above ground detention ponds, underground detention facilities are designed to drain completely between runoff events, thereby providing storage capacity for subsequent events. Underground detention facilities are intended to control peak flows, limit downstream flooding, and provide some channel protection. However, they provide little, if any, pollutant removal and are susceptible to re-suspension of sediment during subsequent storms.

Underground detention systems serve as an alternative to surface dry detention for stormwater quantity control, particularly for space-limited areas where there is not adequate land for a dry detention basin or multi-purpose detention area. Basic storage design and routing methods are the same as for detention basins except that the bypass for high flows is typically included.

Underground detention facilities may only be used where the hydraulic grade line (HGL) of the existing storm sewer network is low enough to allow adequate drainage to meet City design requirements within 72 hours after any design storm event. Underground detention facilities are not generally intended for water quality treatment and, unless it is specifically accommodated in design, should be used in a treatment train approach with other structural controls to provide water quality treatment. Providing treatment prior to discharging to the underground detention facility will help prevent the underground system from becoming clogged with trash or sediment and significantly reduces the maintenance requirements for the system.

7.5 Detention Design Criteria

Stormwater detention systems shall be designed to meet the stormwater sizing criteria described in Section 2.1 and shall provide structural control as needed to meet the pre-development rate for the design storm events.

7.5.1 Design Procedure

A general procedure for the design of storage facilities is presented below.

Step 1 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs as described above in Sections 7.21 and 7.3.

Step 2 Determine the physical dimensions necessary to hold the estimated volume from Step 1. The maximum storage requirement calculated from Sections 7.21 and 7.3 should be used. From the selected shape determine the maximum depth in the pond. Develop the stage-storage curve for the detention basin.

Step 3 Select the desired type of outlet and size the outlet structures based on allowable discharges for the design storm events, beginning with outlet structure sizing for the smaller events to the extreme flood event and taking into consideration the tailwater in the receiving stream. The estimated peak stage for each storm event (2-,25-,and 100- year) will occur for the maximum associated volume from Step 2. The outlet structure(s) should be sized to convey the allowable discharge for the corresponding stage for each flood event. The outfall structure shall be designed with appropriate erosion prevention measures.

Step 4 Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using a storage routing computer model.

Step 5 Evaluate whether the routed post-development peak discharges from the design storms exceed the existing pre-development peak discharges. If so, then revise the dimensions of the pond or outlet device geometry accordingly and repeat Steps 2 through 4 until the post-development peak discharges do not exceed the existing pre-development peak discharges for the watershed.

Step 6 Evaluate the downstream effects of detention outflows for the 100-year 24-hour storm event to ensure that the routed hydrograph does not cause downstream flooding problems. The outflow hydrograph from the storage facility should be routed through the downstream channel system to a confluence point that reflects no appreciable increase in discharges compared to the pre-development discharges at that location, or to a point designated by the City (see Section 7.5.3).

Step 7 Evaluate the control structure outlet velocity for all storms and provide channel and bank stabilization if the outlet velocities from any of the design storms will cause erosion problems downstream. Outlet protection shall include checking velocities and ensuring adequate erosion prevention measures to beyond the confluence with the receiving stream channel. Riprap placement or energy dissipator devices may be required. Guidance for riprap sizing and extents of placement and outlet design is provided in Section 6.2.

Routing of hydrographs through storage facilities is critical to the proper design of these facilities. Although storage design procedures using inflow/outflow analysis without routing have been developed, their use is not accepted by the City of Little Rock.

Water quality requirements and the associated Water Quality Protection Volume (WQv) shall be addressed in the design. Details regarding these requirements and the approach that may be used to address them are provided in Chapter 8, Water Quality.

For this Manual, it is assumed that designers will be using one of the many computer programs available for storage routing and thus other procedures and example applications will not be given here.

7.5.2 Detention Design Standards

The following conditions and limitations shall be observed in selection and use of the method or type of detention.

7.5.2.1 General

Detention facilities shall be located within the parcel limits of the project under consideration. No detention or ponding will be permitted within public road right-of-ways. Location of detention facilities immediately upstream or downstream of the project will be considered by special request if proper documentation is submitted with reference to practicality, feasibility, and proof of ownership or right-of-use of the area proposed. To be in accordance with design requirements and not exceed pre-development discharges, orifices shall be provided to limit outflows.

7.5.2.2 Dry Detention / Dry ED Basins

Dry detention ponds or dry reservoirs shall be designed with proper safety, stability, and ease of maintenance facilities. Maximum side slopes for grass reservoirs shall not exceed 1-foot vertical for 3-feet horizontal (3:1) unless approved by the City Design Review Engineer. For dry detention ponds, pond bottom slopes must be a minimum of 1% (longitudinal and cross-slope) to ensure positive drainage to outlet works. In no case shall the limits of maximum ponding elevation be closer than 20 feet horizontally from any building and less than 1 foot vertically below the lowest adjacent grade. The entire reservoir area shall be stabilized with vegetation established prior to final approval or issuance of certificate of occupancy unless approved by the Director of Planning and Development. Any area susceptible to, or designed as, overflow by higher design intensity rainfall shall be stabilized with sod or other approved vegetative stabilization practice or paved depending upon the outflow velocity. Plan view and cross-sections with adequate details for any dry detention basins and forebays and dry ED basins shall be provided in the construction plans.

7.5.2.3 Stormwater Ponds

Stormwater ponds with fluctuating volume controls may be used as detention areas provided that the limits of maximum ponding elevations are no closer than 50-feet horizontal from any building, are at least 2 feet below the lowest sill or floor elevation of any building, and at least 1 foot below lowest adjacent grade.

Maximum side slopes for the fluctuating area of stormwater ponds shall be l-foot vertical to 3-feet horizontal (3:1) unless provisions are included for safety, stability, and ease of maintenance. Safety railing or other safety measures such as a shallow shelf shall be provided for ponds located in residential areas. All stormwater ponds shall include a sediment forebay at the inflow to the basin to allow heavier sediments to drop out of suspension before runoff enters the permanent pool. Sediment forebays shall be located at each point where piping or other conveyances discharge into the stormwater pond. Forebays shall be located such that they are accessible by maintenance equipment. Forebays shall be designed with adequate depth (preferably 4 to 6 feet to dissipate turbulent inflow - lesser design depths may be justified with supporting velocity computations) and volume to dissipate the energy of incoming stormwater flows and allow coarse-grained sediments and particulates to settle out of the runoff. The sediment forebay should be sized to accommodate 0.25 inches of runoff per contributing on-site impervious acre of drainage area and should allow flow to exit the forebay at non-erosive velocities from the 1-year to 10-year 24-hour storm events. The forebay may be included as part of the required volume for detention with permanent pools.

The entire fluctuating area of the permanent reservoir shall be stabilized with vegetation established prior to final approval or issuance of certificate of occupancy unless approved by the Director of Planning and Development. Also, calculations must be provided to ensure adequate "live storage" is provided for the difference between the post- and pre-developed 100-year, 24-hour storm. Any area susceptible to or designed as overflow by higher design intensity rainfall (100-year frequency) shall be sodded, stabilized with an approved vegetative stabilization practice, or paved, depending on the design velocities. An engineering analysis shall be furnished of any proposed earthen dam or embankment configuration, with appropriate geotechnical testing and computations. Earthen dam structures shall be designed by an Arkansas Licensed Professional Engineer. Plan view and cross-sections with adequate details for any stormwater ponds shall be provided in the Construction Plans. Detention basin embankment shall have a minimum 10-foot crown width.

7.5.2.4 Low Impact Development Practices

Low impact development (LID) practices can help reduce the peak flow of stormwater leaving the site. If LID practices are used on the project, they should be used upstream of any proposed detention facility. This will potentially result in reducing the quantity of stormwater necessary to be detained. Refer to Chapter 8, section 8.2 for detailed design requirements for LID practices and for the approach to adjust peak discharges, where appropriate, based on implementation of LID features.

7.5.2.5 Other Methods

If other methods of detention are proposed, proper documentation of hydrologic and hydraulic calculations, soil data, percolation, geological features, etc., will be needed for review and consideration.

7.5.2.6 Outlet Works

Detention facilities shall be provided with effective outlet works. Flows shall be limited to pre-development rates for the design storm events.

Safety considerations shall be an integral part of the design of all outlet works. Plan view and sections of the structure with adequate construction details shall be included in Plans.

Overflow openings (emergency spillway) are required for all ponds. The overflow opening shall be designed to accept the fully urbanized 100-year flood event assuming blockage of the closed conduit portion of the outlet works with 6 inches of freeboard. Spillway requirements must also meet all appropriate state and federal criteria. Design calculations shall be included for all spillways.

7.5.2.7 Discharge Systems

Existing upstream detention structures may be accounted for in design. Field investigations and hydrologic analysis shall be performed to substantiate benefits. A field survey of the existing physical characteristics of both the outlet structure and ponding volume shall be performed. A comprehensive hydrologic analysis shall be performed that simulates the attenuation of the contributing area ponds. This should not be limited to a linear additive analysis, but rather should consist of a network of hydrographs that considers incremental timing of discharge and potential coincidence of outlet peaks.

7.5.2.8 Ownership of Stormwater Detention Ponds

Ownership of stormwater detention ponds that are not dedicated by the City of Little Rock shall be vested in the property owner.

The City will not process the Final Plat if all the drainage features are not complete. No alteration of the drainage system will be allowed without the approval of the Design Review Engineer.

7.5.2.9 Easements

Easements shall be provided on the plans for detention facilities. A minimum 20-feet wide drainage easement shall be provided along the reservoir area, providing vehicular access to the facility, and connecting the tributary pipes and the discharge system along the most passable route, when the discharge system is part of the public drainage system.

7.5.2.10 Maintenance

Detention facilities, when required, are to be built in conjunction with storm sewer installation and/or grading. Since these facilities are intended to control increased runoff, they must be partially or fully operational soon after the clearing of the vegetation. During project construction, silt and debris shall be removed as needed from the detention area and control structure(s) after each storm event to maintain the storage capacity of the facility.

Post-construction maintenance of detention facilities is divided into two components. The first is long-term maintenance that involves removal of sediment from the basin and outlet control structure. Maintenance to an outlet structure is minimal with proper initial design of permanent concrete or pipe structures. Studies indicate that in developing areas, basin cleaning by front-end loader or grader is estimated to be needed once every 5 to 10 years.

Annual maintenance is the second component and is the responsibility of the developer or association throughout the construction phases and of the pond owner in perpetuity after acceptance of the final plat or filing of the last subdivision phase that substantially adds stormwater to a detention basin.

These items include:

1. Minor dirt and mud removal,
2. Outlet cleaning,
3. Mowing,
4. Herbicide spraying (in strict conformance with the City's policies and procedures),
5. Litter control, and
6. Forebay cleaning (where applicable)

The responsibility for maintenance of the detention facilities and single-lot development projects shall remain with the general contractor until final inspection of the development is performed and approved, and a legal occupancy permit is issued. After legal occupancy of the project, the maintenance of detention facilities shall be vested with the owner of the detention pond.

7.5.3 Downstream Hydrologic Assessment

7.5.3.1 Introduction

The purpose of the overbank flood protection and extreme flood protection criteria is to protect downstream properties from increases in flood hazard due to upstream development. These criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals pre- development peak discharge. In certain cases, this does not always provide effective water quantity control downstream from the site and may exacerbate flooding problems downstream. The reasons for this have to do with the timing of the flow peaks, and the total increase in volume of runoff. This section outlines the procedure for determining the impacts of post-development stormwater peak flows and volumes on downstream flows. For sites less than 40 acres a downstream assessment is not required, however the site still must meet the pre-development discharge rates.

7.5.3.2 Reasons for Downstream Problems

Flow Timing

If water quantity control (detention) structures are indiscriminately placed in a watershed and changes to the flow timing are not considered, the structural control may increase the peak discharge downstream. The reason for this may be seen in Figure 7.3. The peak flow from the site is reduced appropriately, but the timing of the flow is such that the combined detained peak flow (the larger dashed triangle) is higher than if no detention were required. In this case, the shifting of flows to a later time brought about by the detention pond makes the downstream flooding worse than if the post- development flows were not detained.

Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with discharge from downstream tributaries to increase the downstream peak flows.
Figure 7.3 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume needs to be increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

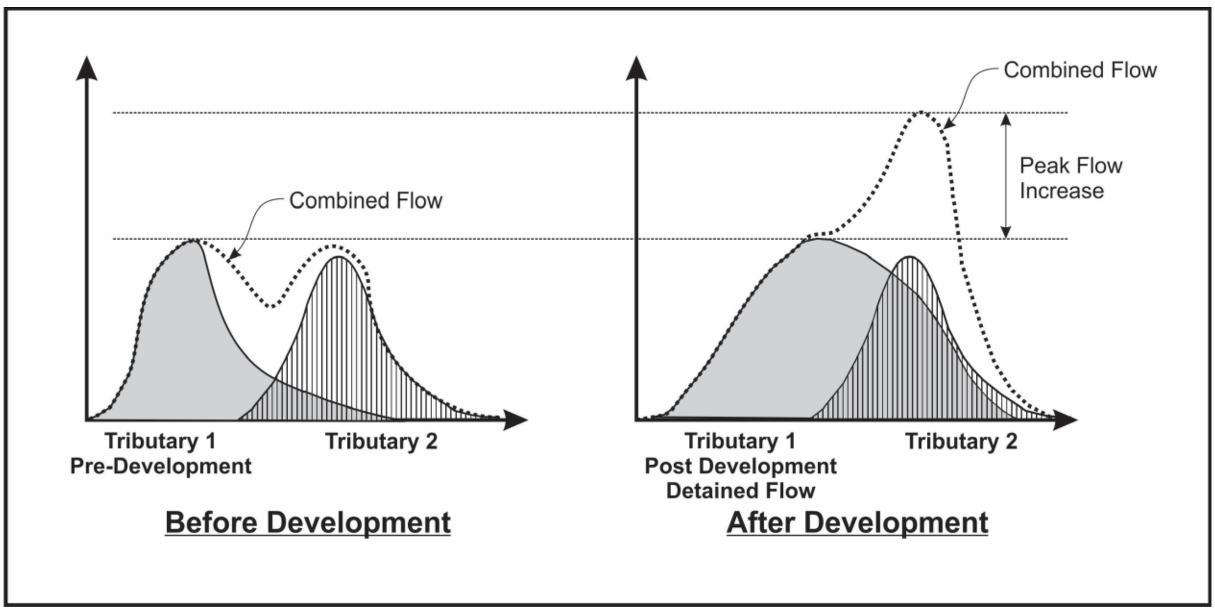


Figure 7.3. Downstream hydrograph comparison of pre and post development of detention structure.

7.5.3.3 The Ten-Percent Rule

In this Manual the “ten percent” criterion has been adopted as the most flexible and effective approach for ensuring that stormwater quantity detention ponds perform the desired function of maintaining pre-development peak flows throughout the system downstream.

The ten-percent rule recognizes the fact that a structural control providing detention has a “zone of influence” downstream where its effectiveness can be felt. Beyond this zone, the influence of the structural control becomes relatively small and insignificant compared to the runoff from the total drainage area at that point. Based on studies and master planning results for a large number of sites, that zone of influence is considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if the structural control drains 40 acres, the zone of influence ends at the point where the total drainage area is 400 acres or greater, see Figure 7.4. If the downstream assessment point extends past the confluence with a FEMA detailed study (Zone AE) stream, then the downstream assessment will end at the confluence. The City Design Review Engineer may assign additional locations for assessment based on locations of known downstream flooding, high erosion potential, downstream development, and channel constrictions.

Typical steps in the application of the ten-percent rule are:

1. Determine the target peak flow for the site for pre-development conditions.
2. Using a topographic map, assess the anticipated lower limit of the zone of influence (10% point).
3. Using a hydrologic model, to the same level of detail as for site project design, determine the pre- development peak flows and timing of those peaks at each tributary junction beginning at the pond outlet and ending at the next tributary junction beyond the 10% point. The designer shall use hydrologic models obtained from the City of Little Rock or the data therefrom, if available, for the assessment of the downstream subareas.
4. Change the land use on the site to post-development and rerun the model.
5. Design the structural control facility such that the post development peak discharges are not increased above pre-development discharges at the outlet and the determined tributary junctions.

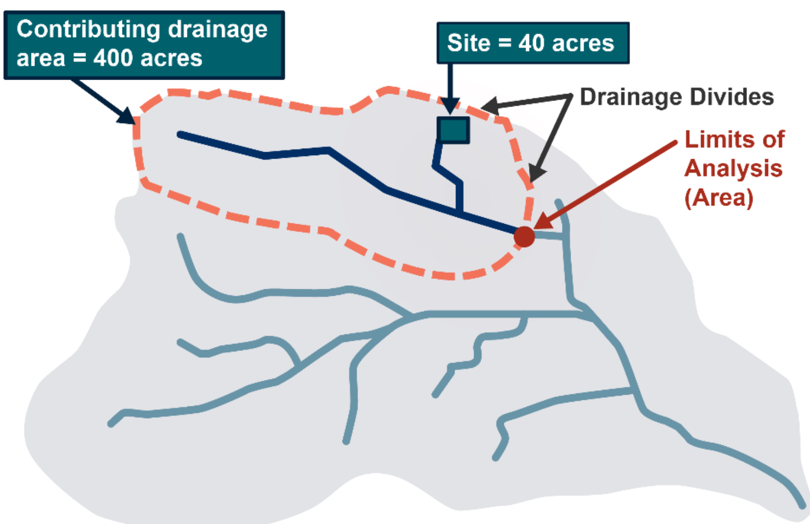


Figure 7.4. Schematic of zone of influence using the 10% rule.

7.6 References

[Drainage Criteria Manual | Fayetteville, AR - Official Website (fayetteville-ar.gov)]

Atlanta Regional Commission, 2001. Georgia Stormwater Management Manual, Volume 1: Chapter 6, Floodplain Management.

[Section 2G-1 - General Information for Detention Practices (iastate.edu)]
[HEC-22, 3rd edition Urban Drainage Design Manual (dot.gov)]

8. Water Quality

8.1 General

Nonpoint source pollution is a primary cause of polluted stormwater runoff and water quality impairment, and nonpoint source pollution can come from many sources. Development can concentrate and increase the amount of these nonpoint source pollutants. As stormwater runoff moves across the land surface, it has the potential to pick up and carry away both natural and human-made pollutants, depositing them into Little Rock's water resources.

In urbanizing watersheds, the potential for water quality degradation occurs because of development and other human activities. Erosion from construction sites and other disturbed areas can contribute large amounts of sediment to streams. As construction and development proceed, impervious surfaces replace the natural land cover and pollutants from human activities begin to accumulate on these surfaces. During storm events, these pollutants could be washed off into the streams. Excess stormwater also has the potential to cause discharges from sewer overflows and leaching from septic tanks. There are several other causes of potential nonpoint source pollution in urban areas that are not specifically related to wet weather events including leaking sewer pipes, sanitary sewage spills, illegal dumping of pollutants into streams or storm drains by individuals, and illicit discharge of commercial/industrial wastewater and wash waters to storm drains.

8.2 Low Impact Development

This section also provides guidance for using the natural properties and existing conditions of a site to optimize the management of stormwater using Low Impact Development (LID).

After conservation areas are delineated, development of site design should include planning to avoid future downstream stormwater impacts from the development. Planning techniques should:

- Fit the design to the terrain.
- Reduce the limits of clearing and grading.
- Locate development in less hydrologically sensitive areas.
- Utilize open space development and/or nontraditional lot designs for residential areas.
- Consider creative development design.

More details on practices that reduce the impact of development are covered below.

Conserve Natural Features and Resources

The conservation of natural features such as floodplains, higher permeability soils, and vegetation helps to retain pre-development hydrologic functions, thus reducing runoff volumes. Impacts to natural features should be minimized by reducing the extent of construction impacts and minimize development practices that are averse to pre-development hydrology functions. Conservation techniques include the following:

- Build upon the least permeable soils and limit construction activities to previously disturbed soils.
- Avoid mass clearing and grading and limit the clearing and grading of land to the minimum needed to construct the development and associated infrastructure.
- Avoid disturbance of vegetation and highly erodible soils on slopes and near surface waters.
- Leave undisturbed stream buffers along both sides of natural streams as covered in Section 8.5.2.
- Preserve sensitive environmental areas; historically undisturbed vegetation; and native trees as currently required in the City of Little Rock ordinances.
- Conform to watershed, conservation, and open space plans.

- Design development to fit the natural site terrain and build roadways along existing site contours wherever possible.
- Use cluster development to preserve higher permeability soils, natural streams, and natural slopes.
- Develop on previously developed sites (redevelopment or infill).

Minimize Soil Compaction
Soil compaction disturbs native soil structure, reduces soil porosity and permeability, affecting infiltration rates, and limits root growth and re-establishment of vegetation. While soil compaction is necessary within a structure footprint to provide structurally sound foundations, areas away from foundations are often excessively compacted by traffic during construction. Minimizing soil compaction can be achieved by the following methods:

- Reduce disturbance through design and construction staging practices.
- Limit areas of access for heavy equipment.
- Avoid extensive and unnecessary clearing and stockpiling of topsoil.
- Maintain existing topsoil and/or use quality topsoil during construction.
- Rapid establishment of vegetative cover in bare but otherwise undisturbed areas to minimize compaction by rainfall.
- Avoid working or driving on wet soil.
- Protect soil under tree canopies by covering with mulch or plywood to allow vehicle traffic for construction.

Reduce and Disconnect Impervious Surfaces
Reducing and disconnecting impervious surfaces increases the rainfall that infiltrates into the ground. Impervious areas may be reduced by maximizing landscaping and using pervious pavements. In addition, the number of impervious areas with direct hydraulic connections to impervious conveyances (e.g., driveways, walkways, culverts, streets, or storm drains) should be minimized. The following measures are applicable:

- Install green or blue roofs.
- Direct roof downspouts to vegetated areas, bioretention, cisterns, or planter boxes, and route runoff into vegetated swales instead of onto driveways and in gutters.
- Use porous pavements, where permitted.
- Install shared driveways that connect two or more homes, where permitted, or install residential driveways with center vegetated strips.
- Allow for shared parking in commercial areas.
- Maximize usable space, not through large building footprints but through taller buildings with more floors.
- Minimize impervious footprints.

8.3 Pollutants in Stormwater Runoff

8.3.1. Areas with High Pollution Discharge Potential
Areas with high pollutant discharge potential are areas of the urban landscape that often produce higher concentrations of certain pollutants, such as hydrocarbons or heavy metals, than are normally found in urban runoff. These areas merit special management and the use of specific pollution prevention activities and/or structural stormwater controls. Examples of areas with high pollutant discharge potential include:

- Gas / fueling stations.
- Vehicle maintenance areas.

- Vehicle washing / steam cleaning.
- Auto recycling facilities.
- Outdoor material storage areas.
- Loading and transfer areas.
- Landfills.
- Construction sites.
- Industrial sites.
- Industrial rooftops.

8.3.2. Stormwater Pollutant Sources

For effective stormwater management, it is important to understand the nature and sources of urban stormwater pollution. Table 8.1 summarizes the major stormwater pollutants and their effects.

Table 8.1 Summary of Urban Stormwater Pollutants

Constituents	Effects
Sediments: Total Suspended Solids (TSS), Dissolved Solids, Turbidity	Stream turbidity Habitat changes Recreation/aesthetic loss Contaminant transport Filling of lakes and reservoirs
Nutrients: Nitrate, Nitrite, Ammonia, Organic Nitrogen, Phosphate, Total Phosphorus	Algae blooms Eutrophication Ammonia and nitrate toxicity Recreation/aesthetic loss
Microbes: Total and Fecal Coliforms, Fecal Streptococci, Viruses, E.Coli, Enterocci	Ear/Intestinal infections Recreation/aesthetic loss
Organic Matter: Vegetation, Sewage, Other Oxygen Demanding Materials	Dissolved oxygen depletion Odors Fish kills
Toxic Pollutants: Heavy Metals (cadmium, copper, lead, zinc), Organics, Hydrocarbons, Pesticides/Herbicides	Human & aquatic toxicity Bioaccumulation in the food chain
Thermal Pollution	Dissolved oxygen depletion Habitat changes
Trash and debris	Recreation/aesthetic loss

8.4 Water Quality Criteria

The Total Suspended Solids (TSS) Reduction Method (TRM) focuses on the removal of 80% TSS from a selected rainfall runoff, 1.05 inches for new development and significant redevelopment in Little Rock. This runoff value represents the 80^{th} of rainfall events that Little Rock experiences per year. The percentiles were calculated using rainfall records from the Little Rock airport (Adams Field) rain gage.

TSS Reduction Method (TRM)

The TSS Reduction Method follows the philosophy of removing pollutants and at least 80% of the TSS "where practicable" using a percentage removal performance goal. The approach provides treatment of the Water Quality Volume (WQ_v) from a site to reduce post-development TSS loadings by 80%, as measured on an average annual basis. This performance goal is based on the ADEQ NPDES small MS4 permit in accordance with U.S. EPA guidance.

The WQ_v is used to size stormwater control measures that work to remove pollutants from the runoff. The WQ_v is roughly equal to the runoff from the first 1.05 inches of rainfall within the catchment area. A stormwater management system designed to treat the WQ_v will treat the runoff from storm events of 1.05 inches or less, as well as the first 1.05 inches of runoff for larger storm events.

The volumetric runoff coefficient (Rv) was derived from a regression analysis performed on rainfall runoff volume data from several cities nationwide and is a shortcut method considered adequate for runoff volume calculation for the type of small storms considered in stormwater quality calculations.

The Water Quality Volume (WQv) is equal to a rainfall depth (P) of 1.05 inches multiplied by the volumetric runoff coefficient (Rv) and the site area (A), and is calculated using Equation 8.1 below:

$$\mathbf{WQ_V} = \frac{\mathbf{PR_VA}}{\mathbf{12}} \qquad \textbf{Eq. 8.1}$$

Where: WQv = water quality volume (ac-ft)

Rv = 0.05 + 0.009(I)
where I is the <u>percent</u> impervious cover (i.e., 50% impervious is 50 not 0.5)

A = site area (acres)

P = 1.05 inches (new development)

<u>Determining the Water Quality Volume (WQ_v)</u>

- ***Measuring Impervious Area*:** The area of impervious cover shall be based on the proposed project plans and shall be independent of pre-construction conditions.
- ***Site Area:*** For redevelopment, the site area is considered only the portion of the site that is being modified.
- ***Multiple Drainage Areas*:** When a development project contains or is divided into multiple drainage areas, WQ_v should be calculated and addressed separately for each drainage area.
- ***Off-site Drainage Areas:*** Off-site existing impervious areas are excluded from the calculation of the WQ_v volume.
- ***Determining the Peak Discharge for the Water Quality Storm:*** When designing off-line stormwater control measures, the peak discharge of the water quality storm (Q_{wq}) can be determined using the SCS method provided in Chapter 3. The water quality storm is equivalent to 1.05 inches of rainfall in 24 hours.
- ***Extended Detention of the Water Quality Volume:*** The water quality treatment requirement can be met by providing a 24-hour drawdown of a portion of WQ_v in a stormwater pond or wetland system. Referred to as water quality ED (extended detention), it is different than providing extended detention of the 1-year 24-hour storm for the channel protection volume (CP_v). Where used, the ED portion of the WQ_v may be included when routing the CP_v.

WQ_v can be expressed in cubic feet by multiplying by 43,560. WQ_v can also be expressed in watershed-inches as simply PR_v by removing the area (A) and the 12 from Equation 8.1.

8.5 Water Quality Volume and Sizing

There are two primary approaches for managing stormwater runoff and addressing the water quality (and quantity-based) criteria requirements on a development site:

- The use of site design practices to reduce the amount of stormwater runoff and pollutants generated and/or provide for natural treatment and control of runoff.
- The use of stormwater control measures to provide treatment and control of stormwater runoff.

8.5.1 Site Design as the First Step in Addressing Requirements

Using the site design process to reduce stormwater runoff and pollutants should always be the first consideration of the site designer and engineer in the planning of the stormwater management system for a development.

Site design concepts can be used as both water quantity and water quality management tools and can reduce the size and cost of required structural stormwater controls. The site design approach can result in a more natural and cost-effective stormwater management system that better mimics the natural hydrologic conditions of the site and has a lower maintenance burden.

8.5.2 Stream Buffer Requirements

Natural stream channels shall be preserved as continuous systems and not segmented on a project-by-project basis because the frequent intermixing of natural and man-made systems tends to degrade the function of both. Stream buffers (also called riparian buffers) are vegetated areas along and adjacent to streams where clearing, grading, filling, building of structures, and other activities are limited or prohibited. Stream buffers act as the "right-of-way" for the stream and protect and enhance water quality and stream health in two primary ways: (1) reducing the number of pollutants entering the stream in stormwater runoff flowing overland through the vegetated buffer; and (2) preserving and enhancing stream channel stability, in-stream habitat, and the stream's natural ability to process pollutants in stream flow.

The stream buffer shall be measured from the streambank of the active channel. Buffer widths shall meet or exceed the distances specified in Table 8.2. No clearing, grading, filling, or structures are allowed in stream buffer zones other than as authorized in Sections 8.5.2.1 and 8.5.2.2.

Table 8.2 Stream Buffer Width Requirements.

Contributing Drainage Area	Stream Category	Buffer Width (ft.)	
		Streamside Zone	Outer Zone
Greater than 4 square miles	A - Large stream	50	50
1 to 4 square miles	B - Small stream	40	40
160 to 640 acres	C - Large tributary	25	25
64 to 160 acres	D - Small tributary	15	15

8.5.2.1 Streamside Zone

The structures, practices, and activities permitted in the Streamside Zone of the buffer are limited to the following:

- Stream crossings for roads, drives, trails or other pedestrian paths, and utilities.

- Utility corridors if no feasible alternative exists.
- Stormwater discharge structures in accordance with City-approved stormwater plans only
- if discharge further upland is not feasible.
- Vegetation management to maintain or improve native vegetation, including:
  a. Removal of diseased, dead, or hazard trees.
  b. Tree pruning in accordance with accepted arborist practices.
  c. Selective spraying or mechanical removal of noxious or invasive vegetation
  d. consistent with accepted best practices.
  e. Vegetation planting and seeding to improve the density, species, and diversity of native vegetation.
- Removal of trash.
- Removal of accumulated debris to maintain stream flow conveyance.
- Water quality monitoring and stream gauging.
- City-approved stream bank stabilization measures.
- Maintenance of City-approved improvements, including utilities.

The above structures, practices, and activities shall be accomplished using methods that minimize soil disturbance, clearing of vegetation, and use of motorized equipment.

8.5.2.2 Outer Zone

The structures, practices and activities permitted in the Outer Zone of the buffer are limited to the following:

- All uses permitted in the streamside zone.
- Trail corridors that are designed to minimize stream buffer impacts.
- Fences.
- Stormwater control measures in accordance with City-approved stormwater plans only if it is not feasible to locate them outside (upland) of the stream buffer.
- Managed lawns are permitted in the Outer Zone of stream categories B-D although property owners are encouraged to preserve or plant native vegetation to increase the benefits of the buffer. Existing, healthy trees must be preserved in managed lawn areas and property owners are encouraged to plant trees in managed lawn areas. Managed lawns in Outer Zones shall not be fertilized unless as recommended by a soil test. Use of pesticides/herbicides on managed lawns in Outer Zones is discouraged and if used, shall be in accordance with integrated pest management practices.

8.5.3 Structural Stormwater Control Measures

Structural stormwater controls (sometimes referred to as *structural best management practices* or *BMPs*) are constructed stormwater management facilities designed to treat stormwater runoff and/or mitigate the effects of increased stormwater runoff peak rate, volume, and velocity due to urbanization.

This Manual recommends several water quality structural stormwater controls that can be implemented to help meet the stormwater management Minimum Standards.

The recommended water quality controls are divided into three categories:

1. General application controls.
2. Limited application controls.
3. Floatables control.

These controls are targeted at 80% TSS pollution reduction. Detention structural controls are discussed in Chapter 7.

8.5.3.1 General Application Controls

General application structural controls are recommended for use with a wide variety of land uses and development types. These structural controls have a demonstrated ability to effectively treat the Water Quality Volume (WQv) and are presumed to be able to remove 80% of the total annual average TSS load in typical post-development urban runoff when designed, constructed, and maintained in accordance with recommended specifications. Several of the general application structural controls can also be designed to provide water quantity control, i.e., downstream channel protection (CPv), overbank flood protection (Qp25) and/or extreme flood protection (Qf). General application controls are the recommended stormwater management facilities for a site wherever feasible and practical.

There are six types of general application controls, which are summarized below. They are broken up into two categories, water quality structural controls and low impact structural controls. Detailed descriptions of the water quality structural controls along with design criteria and procedures are provided in Appendix C, Stormwater Control Measures (SCMs).

Design for all basins (extended detention, bioretention, sand filter, etc.) shall have the following design features:

A. Basins shall be designed as offline facilities, with a splitter structure used to isolate the water quality volume. The splitter box, or other City Design Review Engineer approved flow diverting approach, should be designed to covey the 100-year storm event.
B. Online facilities may be approved only with an exception from the City Design Review Engineer. Exception may only be considered if the design engineer provides justifiable reason why the facility must be online and the facility is designed to contain the 100-year storm and meet all other requirements of a detention pond as outlined in Chapter 7 of this manual for capacity, freeboard, emergency overflow, access, etc.
C. In areas where it may be difficult to treat every drainage area leaving the site, an exception may be granted by the City Design Review Engineer to provide overtreatment of drainage areas. Exception will only be considered if design engineer has made every reasonable effort to treat as much of the site as possible, detailed calculations are provided showing the amount of overtreatment being provided to cover untreated portion of the site, and a maximum of 10% of the site is left untreated and accounted for in overtreatment of other drainage areas.

Water Quality Structural Controls

Stormwater Ponds

Stormwater ponds are constructed stormwater detention basins that have a permanent pool (or micropool) of water. Runoff from each rain event is detained and treated in the pool. Pond design variants include:

- Wet Pond.
- Wet Extended Detention Pond.
- Micropool Extended Detention Pond.
- Multiple Pond Systems.

Stormwater Wetlands

Stormwater wetlands are constructed wetland systems used for stormwater management. Stormwater wetlands consist of a combination of shallow marsh areas, open water, and semi-wet areas above the permanent water surface. Wetland design variants include:

- Shallow Wetland.
- Extended Detention Shallow Wetland.
- Pond/Wetland Systems.
- Pocket Wetland.

Sand Filters

Sand filters are multi-chamber structures designed to treat stormwater runoff through filtration, using a sand bed as the primary filter media. Filtered runoff may be returned to the conveyance system or allowed to fully or partially exfiltrate into the soil. The two sand filter design variants are:

- Surface Sand Filter.
- Perimeter Sand Filter.

Low Impact Structural Controls

Bioretention Areas

Bioretention areas are shallow stormwater basins or landscaped areas that utilize engineered or amended native soils and vegetation to capture and treat stormwater runoff. Runoff may be returned to the conveyance system or allowed to fully or partially exfiltrate into the soil.

Infiltration Trenches

An infiltration trench is an excavated trench filled with stone aggregate used to capture and allow infiltration of stormwater runoff into the surrounding soils from the bottom and sides of the trench.

Enhanced Swales

Enhanced swales are vegetated open channels that are explicitly designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means. The two types of enhanced swales are:

- Dry Swale.
- Wet Swale/Wetland Channel.

8.5.3.2 Limited Application Controls

Limited application structural controls are those that are recommended only for limited use or for special site or design conditions. Generally, these practices:

1. Cannot alone achieve the 80% TSS removal target.
2. Are intended to address specific land use constraints or conditions.
3. May have high or special maintenance requirements that may preclude their use.

Limited application controls are typically used for *water quality treatment only*. Some of these controls can be used as a pretreatment measure or in series with other structural controls to meet pollutant removal goals. Limited application structural controls should be considered primarily for commercial, industrial, or institutional developments, and not residential developments.

The following limited application controls are provided for consideration in this Manual. Each is discussed in detail with appropriate application guidance in Appendix C, Stormwater Control Measures.

Filtering Practices

- Organic Filter.
- Underground Sand Filter.

Wetland Systems

- Submerged Gravel Wetland.

Hydrodynamic Devices

- Gravity (Oil-Grit) Separator.

Proprietary Systems

- Commercial Stormwater Controls.

8.5.3.3 Floatables Control

Floatable materials can be defined as any foreign matter that may float or remain suspended in the water column. The term includes plastic, aluminum cans, wood products, bottles, and paper products.

During storm events, runoff picks up floatable litter and transports it downstream into the storm system and ultimately the receiving waterbody. Floatables control technologies are designed to reduce or eliminate the visible solid waste that is often present in stormwater runoff. See EPA's fact sheet "Combined Sewer Overflow – Floatables Control" (EPA 832-F99-008) for additional information on floatables control. There are many floatables control products, such as hydrodynamic separators, available from multiple manufacturers that utilize methods discussed below.

Example floatables control technologies include:

- Baffles.
- Screens and trash racks.
- Catch basin modifications.
- Netting.

Baffles

Baffles are simple floatables control devices that are typically installed at manholes or catch basins within the stormwater system. They consist of vertical steel plates or concrete beams that extend from the top of the sewer to just below the top of the regulating weir. During an overflow event, floatables are retained by the baffles while water passes under the baffles, over the regulator, and into the receiving water body. When the flow recedes below the bottom of the baffle, floatable material is deposited in the bottom of the catch basin and then can be removed. An example of a typical baffle in a CSO regulator can be found in Appendix C, Figure C-18.

Screens and Trash Racks

Screens and Trash Racks consist of a series of vertical and horizontal bars or wires that trap floatables while allowing water to pass through the openings between the bars or wires. Screens can be installed at select points within a stormwater system to capture floatables and prevent their discharge into natural waterbodies. Screens used for floatable control include mechanically cleaned permanent screens, static screens, traveling screens, or drum screens. Screens can also be divided into three categories according to the size of floatable material they are designed to capture.

These are:

- Bar screens (> 1-inch openings).
- Coarse screens (0.19 – 1-inch openings).
- Fine screens (0.004 - 0.19-inch openings).

The screens most commonly used in stormwater systems are trash racks (a type of bar screen primarily used as an end-of-pipe control) and coarse screens. See EPA's fact sheet "Screens" (EPA 832-F99-027) for additional information on screens for floatable control.

Catch Basin Modifications

Catch basins are surface-level inlets to the sewer system that are often used to allow runoff from streets and lawns to enter the storm system. These basins are often modified to prevent floatables from entering the system. Inlet grates installed at the top of the catch basins reduce the amount of street litter and debris that enters the catch basin. If floatables enter the basin through these grates, they can be collected in colander-like structures called trash buckets installed in the basin beneath the grate. These structures retain floatables while letting water flow through to the downstream system. Other catch basin modifications, such as hoods and submerged outlets, alter outlet pipe conditions and keep floatables from entering the storm system. Hoods are vertical cast iron baffles installed in catch basins. Submerged outlets are located below the elevation of the storm system and are connected by a riser pipe. A typical modified catch basin with hood is presented in Appendix C, Figure C-19.

Netting

Two types of netting systems can be used to collect floatables in a storm system: in-line netting, and floating units.

In-line netting can be installed at strategic locations throughout the storm system. The nets would be installed in underground concrete vaults containing one or more nylon mesh bags and a metal frame and guide system to support the nets. The mesh netting is sized according to the volume and types of floatables targeted for capture. The stormwater flow carries the floatables into the nets for capture. Bags are replaced after every storm event.

Floating units consist of an in-water containment area that funnels stormwater flow through a series of large nylon mesh nets. Mesh size depends on the volume and type of floatables expected at the site. This system is passive and relies on the energy of the overflow to carry the floatables to the nets. However, nets must be located some distance from the outfall (often 15 meters [50 feet] or more) to allow floatables entrained in the turbulent stormwater flow to rise to the flow surface and be captured. The nets are single use, and after an overflow, the nets are typically removed and taken to a disposal area. Additional information on one type of floating unit, the TrashTrap™ system, is provided in a separate fact sheet (EPA 832-F-99-024).

8.5.3.4 Structural Stormwater Control Pollutant Removal Capabilities

General and limited application structural stormwater controls are intended to provide water quality treatment for stormwater runoff. Though each of these structural controls provides pollutant removal capabilities, the relative capabilities vary between structural control practices and different pollutant types.

Pollutant removal capabilities for a given structural stormwater control practice are based on several factors including the physical, chemical and/or biological processes that take place in the structural control and the design and sizing of the facility. In addition, pollutant removal efficiencies for the same structural control type and facility design can vary widely depending on the tributary land use and area, incoming pollutant concentration, rainfall pattern, time of year, maintenance frequency and numerous other factors.

Table 8.3 provides design removal efficiencies for each of the general and limited application control practices. It should be noted that these values are *conservative* average pollutant reduction percentages for design purposes derived from sampling data, modeling, and professional judgment. A structural control design may be capable of exceeding these performances, however the values in the table are generally reasonable values that can be assumed to be achieved when the structural control is sized, designed, constructed, and maintained in accordance with recommended specifications in this Manual.

Where the pollutant removal capabilities of an individual structural stormwater control are not deemed sufficient for a given site application, additional controls may be used in series in a "treatment train" approach. More detail on using structural stormwater controls in series are provided in the next section.

For additional information and data on the range of pollutant removal capabilities for various structural stormwater controls, refer to the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org, the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org, or EPA's fact sheet "Combined Sewer Overflow" (EPA 832-F99-008).

Table 8.3 Design Pollutant Removal Efficiencies for Structural Stormwater Controls

Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metal
General Application Structural Controls					
Stormwater Ponds	80	50	30	70*	50
Stormwater Wetlands	80	40	30	70*	50
Bioretention Areas	80	60	50	---	80
Sand Filters	80	50	25	40	50
Infiltration Trench	80	60	60	90	90
Enhanced Dry Swale	80	50	50	---	40
Enhanced Wet Swale	80	25	40	---	20
Limited Application Structural Controls					
Organic Filter	80	60	40	50	75
Underground Sand Filter	80	50	25	40	50
Submerged Gravel Wetland	80	50	20	70	50
Gravity (Oil-Grit) Separator	40	5	5	---	---
Proprietary Systems	***	***	***	***	***

* If no resident waterfowl population present.
*** The performance of specific proprietary commercial devices and systems must be provided by the manufacturer and should be verified by independent third-party sources and data.
--- Insufficient data to provide design removal efficiency.

8.5.4 Stormwater Treatment Trains

A stormwater "treatment train" is an integrated planning and design approach with components that work together to limit the adverse impacts of urban development on downstream waters and riparian areas. When considered comprehensively a treatment train consists of all the design concepts and nonstructural and structural controls that work together to attain water quality and quantity goals. This is illustrated in Figure 8.1.

Figure 8.1. Generalized stormwater treatment train.

<u>Runoff and Load Generation</u> – The initial part of the “train” is located at the source of runoff and pollutant load generation and consists of better site design and pollution prevention practices that reduce runoff and stormwater pollutants.

<u>Pretreatment</u> – The next step in the treatment train consists of pretreatment measures. These measures typically do not provide sufficient pollutant removal to meet the 80% TSS reduction goal but do provide calculable water quality benefits that may be applied towards meeting the WQ_v treatment requirement. These measures include:

- The use of stormwater better site design practices to reduce the water quality volume (WQ_v).
- Limited application structural controls that provide pretreatment.
- Pretreatment facilities such as sediment forebays on general application structural controls.

<u>Primary Treatment and/or Quantity Control</u> – The last step is primary water quality treatment and/or quantity (channel protection, overbank flood protection, and/or extreme flood protection) control. This is achieved through the use of:

- General application structural controls.
- Limited application structural controls.
- Detention structural controls.

Use of Multiple Structural Controls in Series

Many combinations of structural controls in series may exist for a site. Figure 8.2 provides several hypothetical examples of how the stormwater sizing criteria may be addressed by using structural stormwater controls.

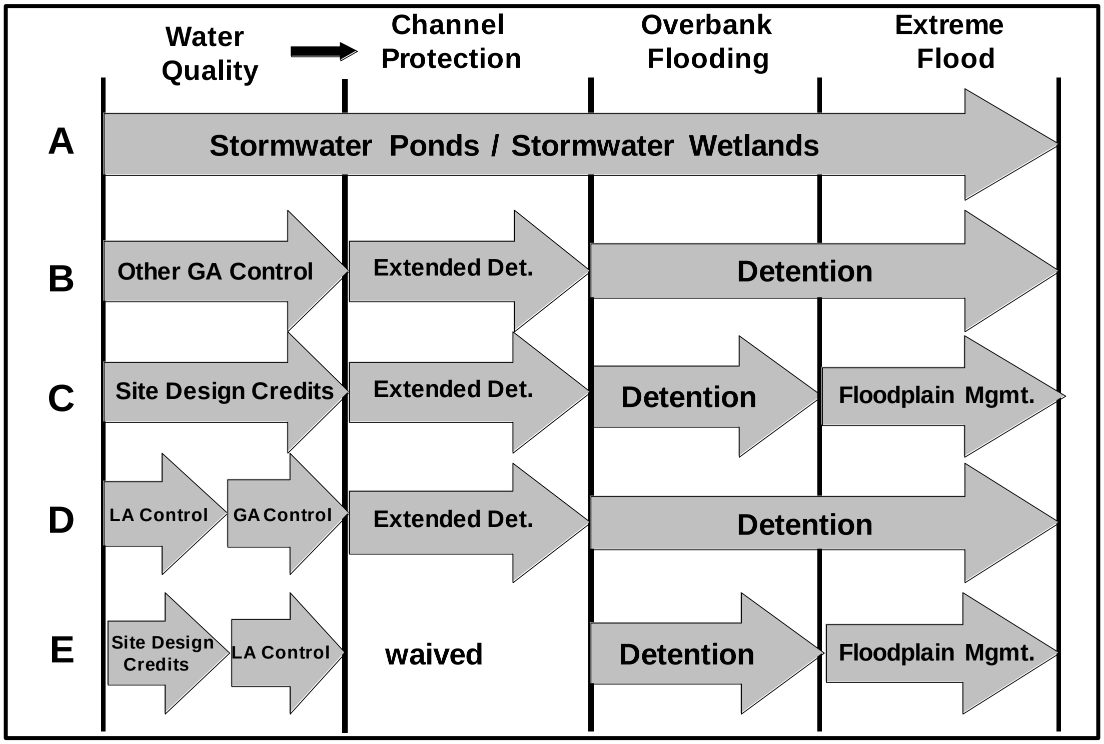


Figure 8.2. Examples of structural controls used in series.

Referring to Figure 8.2 by line letter:

A. Two general application (GA) structural controls, *stormwater ponds* and *stormwater wetlands*, can be used to meet the unified stormwater sizing criteria in a single facility.
B. The other general application structural controls (*bioretention, sand filters, infiltration trench and enhanced swale*) are typically used in combination with detention controls to meet the unified stormwater sizing criteria. The detention facilities are located downstream from the water quality controls either on-site or combined into a regional or neighborhood facility.
C. Line C represents a special case where an environmentally sensitive large lot subdivision has been developed that can be designed to waive the water quality treatment requirement altogether. However, detention controls may still be required for downstream channel protection, overbank flood protection and extreme flood protection.
D. Where a limited application (LA) structural control does not meet the 80% TSS removal criteria, another downstream structural control must be added. For example, areas with high pollutant loading potential may be fit or retrofit with devices adjacent to parking or service areas designed to remove petroleum hydrocarbons. These devices may also serve as pre-treatment devices removing the coarser fraction of sediment. One or more downstream structural controls is then used to meet the full 80% TSS removal goal, as well as water quantity control.

The combinations of structural stormwater controls are limited only by the need to employ measures of proven effectiveness and meet local regulatory and physical site requirements. Figure 8.3 illustrates the application of the treatment train concept for a large shopping mall site.

In this case, runoff from rooftops and parking lots drains to a depressed parking lot, perimeter grass channels, and bioretention areas. Slotted curbs are used at the entrances to these swales to better distribute the flow and to settle out the very coarse particles at the parking lot edge for removal. Runoff is then conveyed to a wet ED pond for additional pollutant removal and channel protection.

Calculation of Pollutant Removal for Structural Controls in Series

For two or more structural stormwater controls used in combination, it is important to have an estimate of the pollutant removal efficiency of the treatment train. Pollutant removal rates for structural controls in series are not additive. For pollutants in particulate form, the actual removal rate (expressed in terms of percentage of pollution removed) varies directly with the pollution concentration and sediment size distribution of runoff entering a facility.

For example, a stormwater pond facility will have a much higher pollutant removal percentage for very turbid runoff than for clearer water. When two stormwater ponds are placed in series, the second pond will treat an incoming particulate pollutant load differently from the first pond. The upstream pond captures the easily removed larger sediment sizes, passing on an outflow with a lower concentration of TSS but with a higher proportion of finer particle sizes. Hence, the removal capability of the second pond for TSS is considerably less than the first pond. Recent findings suggest that the second pond in series can provide as little as half the removal efficiency of the upstream pond.

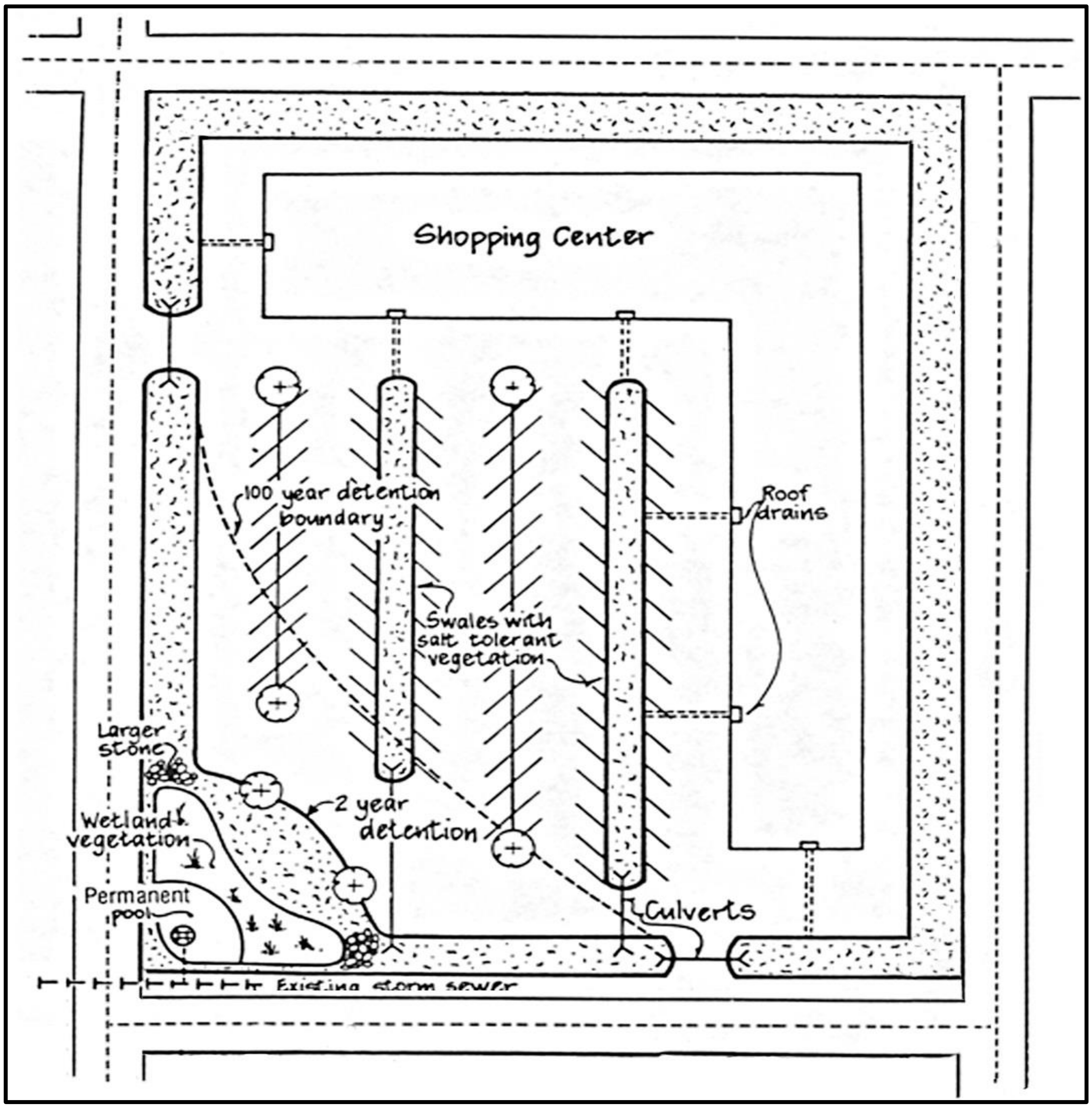


Figure 8.3. Example of treatment train of commercial development.

To estimate the pollutant removal rate of structural controls in series, a method is used in which the removal efficiency of a downstream structural control is reduced to account for the pollutant removal of the upstream control(s). The following steps are used to determine the pollutant removal:

- For each drainage area, list the structural controls in order, upstream to downstream, along with their expected average pollutant removal rates from Table 8.3 for the pollutants of concern.
- For any general application structural control located downstream from another general application control or a limited application structural control that has TSS removal rates equivalent to 80%, the designer should use 50% of the normal pollutant removal rate for the second control in series. For a general application structural control located downstream from a limited application structural control that cannot achieve the 80% TSS reduction goal the designer should use 75% of the normal pollutant removal rate for the second control in series.
- For example, if a general application structural control has an 80% TSS removal rate, then a 40% (0.5 x 80%) TSS removal rate would be assumed for this control if it were placed downstream from another general application control in the treatment train. If it were placed downstream from a limited application structural control that cannot achieve the 80% TSS reduction goal a 60% (0.75 x 80%) TSS removal rate would be assumed. This rule should always be used with caution depending on the actual pollutant of concern and with allowance for differences among structural control pollutant removal rates for different pollutants. Actual data from similar situations should be used where available.
- For cases where a limited application control is located upstream from a general application control in the treatment train, the downstream general application structural control is given full credit for removal of pollutants.
- Apply the following equation for calculation of approximate total accumulated pollution removal for controls in series:
- **Final Pollutant Removal = (Total load * Control1 removal rate) + (Remaining load * Control2 removal rate) + removal for other Controls in series.**

Example 8.1

TSS is the pollutant of concern, and a commercial gravity (oil/grit) separator is inserted that has a 40% sediment removal rate. A stormwater pond is designed at the site outlet. What is the total TSS removal rate? The following information is given:

Control 1 (Commercial Device) = 40% TSS removal

Control 2 (Stormwater Pond 1) = 70% TSS removal (use 1.0 x design removal rate)

Then applying the controls in order and working in terms of "units" of TSS starting at 100 units:

For Control 1: 100 units of TSS * 40% removal rate = 40 units removed

100 units - 40 units removed = 60 units of TSS remaining

For Control 2: 60 units of TSS * 70% removal rate = 42 units removed

60 units - 42 units removed = 18 units of TSS remaining

For the treatment train in total = 100 units TSS – 18 units TSS remaining = **82% removal**

8.6 Water Quality SCM Maintenance

Each water quality BMP installed on a site requires regular maintenance to ensure that it functions properly. A BMP-specific maintenance agreement for each development site is required. The maintenance agreement consists of the following:

1. An Inspection and Maintenance Agreement signed by the developer or BMP owner.
2. A long-term maintenance plan written by the engineer or site designer that includes a description of the stormwater system and its components, inspection priorities and schedule for each component, and BMP schematics for each BMP. The plan should also include requirements for the proper disposal of any materials removed from the BMP during maintenance.
3. A drawing of easements on a plat or a system location map to enable the City to locate BMPs as needed.

The maintenance agreement and its attachments must be submitted for review by the City with the site plans. After the plans and the agreement are approved, the property owner shall record the maintenance agreement and its attachments with the register of deeds. The property owner, under the maintenance agreement, shall be responsible for inspecting and maintaining the BMPs and for turning in inspection reports annually to show that the facilities have been inspected and maintained.

8.7 References

- Drainage Criteria Manual | Fayetteville, AR - Official Website (fayetteville-ar.gov)
- Managing Stormwater in Your Community: A Guide for Building an Effective Post-Construction Program (epa.gov)
- Combined Sewer Overflow Technology Fact Sheet: Floatables Control (epa.gov)
- NIPC, 2000
- Assessing and Monitoring Floatable Debris (epa.gov)
- Flood-Control-and-Water-Quality-Protection-Manual-April-2022 (springfieldmo.gov)
- Sweeney and Newbold, 2014

9. Construction Site Stormwater Management

9.1 General

Construction activities produce many kinds of pollutants which can cause water quality problems. In addition to erosion and sedimentation, construction activities often require the use of toxic or hazardous materials such as petroleum products and fuels, pesticides, herbicides, fertilizers, asphalt, concrete and sealants. These types of materials often contain small amounts of toxic substances which may harm human, plant and animal life along receiving streams and within lakes and ponds.

Management practices which control erosion and sedimentation fall into three main types; those which divert runoff from construction areas, those which prevent erosion on the construction site, and those which trap sediment before it can leave the construction site.

This section of the Stormwater Management and Drainage Manual provides information on many management practices and controls which can be used to comply with the conditions of a grading and land alteration permit. While specific practices are identified, careful consideration must be given to selecting the most appropriate management practices based upon site-specific conditions and installing controls in a timely and proper manner. New, novel, innovative, and proven BMP’s for stabilization, erosion control, sediment control, and outlet protection measures not explicitly mentioned below are allowed and encouraged. New and novel measures require a submittal of a proposed alternative for staff’s review and approval. It also must be noted that proper maintenance is required on all controls for them to remain effective.

9.1.1 Grading and Drainage Plans

City code and state regulations provide that any person proposing to engage in grading, clearing, filling, cutting, quarrying, construction or similar activities regulated by this article shall apply to the Director of Planning and Development for approval of plans and issuance of a grading and land alteration permit.

There are several exceptions to grading and land alteration permit requirements, and these exceptions are considered by the City Design Review Engineer during Grading and Drainage Plan review. There are no exceptions to the need to prepare and submit a Grading and Drainage Plan. Proposed development which does not meet the criteria for a Grading and Drainage Plan, as set forth in the following paragraphs, must include certification from the Architect and/or Engineer that the criteria are not applicable to the proposed development. Failure to provide certification will result in the plan being rejected by the City Design Review Engineer.

A Grading and Drainage Plan is required for **any** of the following activities:

A. Cut or fill activity greater than fifteen (15) vertical feet in height.
B. Cut or fill volume equal to or greater than one thousand (1,000) cubic yards.
C. Clearing that exceeds one (1) acre in size.
D. Clearing that is less than one (1) acre in size but is a part of a larger development.
E. Any land alteration on properties that are located within the 100-year floodplain boundary.

The Grading and Drainage Plan is submitted to and reviewed by the City Design Review Engineer to determine if a Grading Permit is required. Where vertical cut or fill activity greater than thirty (30) feet is indicated, Planning Commission approval is required. It should be noted that all construction work must include appropriate drainage and erosion control measures, regardless of whether a grading and land alteration permit is required.

The Universal Soil Loss Equation has been adopted by the City of Little Rock to enable planners and developers to predict the average rate of soil erosion from construction sites. The City has established an allowable rate of soil loss at five tons per acres per year (5 t/a/yr). Grading plan development requires the application of the Universal Soil Loss Equation to determine the potential soil erosion from a site, which establishes the need for erosion controls. Once erosion controls are identified, the Universal Soil Loss Equation can be used to estimate the effectiveness and adequacy of erosion controls.

The application of erosion and sedimentation controls falls within the sequence of design; stabilization practices; erosion controls; sedimentation controls; and other controls, as applicable. It is possible to control site runoff with stabilization practices alone, where stabilization can reduce the potential soil loss from the site to at or below five (5) tons per acre per year. Where stabilization cannot reduce the potential soil loss to at or below the allowable limit, then erosion controls are also required. Where a combination of stabilization practices and erosion controls are not effective in reducing potential soil loss, sedimentation controls are also required.

9.1.2 Sketch Grading and Drainage Plan Requirements

A Sketch Grading and Drainage Plan may be submitted for agricultural land uses or forestry activities on land owned by forest-related industry. A Sketch Plan is required as a part of the Planning Commission Application for any of the activities identified in Section 9.1.1 above, and for planned unit developments, conditional use permits, site plan reviews, subdivision approvals, or multiple building site approvals.

A Sketch Plan must identify all the following:

A. Acreage of the proposed project.
B. Land areas to be disturbed, (hatching or shading).
C. Stages of grading which identify the limits of sections to be disturbed and the approximate order of development.
D. Extent of cut and fill, shown by placing a dashed line at the top and toe of cut or fill slopes, and indicating on the plan the height and slope of the cut or fill.
E. Provisions for collecting and discharging surface water.
F. Erosion and sediment control measures, including structural and vegetative measures.

The Sketch Plan requires the seal and signature of a registered engineer, architect or landscape architect certifying that the Sketch Plan complies with Municipal Code. Plans for areas of less than five (5) acres where vertical cut or fill height does not exceed ten (10) feet, or where only tree clearing activities will take place may be prepared by a contractor or the property owner.

A Grading and Drainage Plan Checklist has been prepared and is included in Appendix A. Refer to Section 9.1.4 for information on other local, state, and federal permitting requirements.

9.1.3 Complete Grading and Drainage Plan Requirements

A Complete Grading and Drainage Plan includes the requirements for a Sketch Plan and the following additional information:

A. A vicinity drawing identifying property lines, existing or platted streets and public ways within or immediately adjacent to the site.
B. Location of all known existing sewers, water mains, culverts, underground utilities, and existing permanent buildings within and adjacent to the tract.
C. Citation of any existing legal rights-of-way or easements affecting the property.
D. Soil loss calculations as estimated by the Universal Soil Loss Equation (Section 9.6).

E. A plan of the site at a minimum scale of 1'' = 100', showing:
   1. Address and telephone number of the owner, developer, and permittee.
   2. Approximate location and width of proposed streets.
   3. Approximate location and dimension of all proposed or existing lots.
   4. Approximate location and dimension of all parcels of land which will be dedicated to open space, public use, or will remain undisturbed.
   5. Existing and proposed topography at a maximum contour interval of five (5) feet.
   6. An approximate timing schedule indicating the starting and completion dates of the development.
   7. A timing schedule for the sequence of grading and the application of erosion and sediment control measures.
   8. Acreage of the proposed project.
   9. Identification of unusual material or soils in land areas to be disturbed and engineering recommendations for correcting any problems.
   10. Identification of suitable fill materials, including the type and source of outside fill materials.
   11. Specification of measures to control runoff, erosion, and sedimentation during construction, noting the areas where controls are required, and the type of controls employed.
   12. Measures to protect neighboring built-up areas and city property during construction.
   13. Provisions to stabilize soils and slopes after construction is complete, including when and where stabilization measures will be employed.

The Complete Plan must include the seal and signature of a registered engineer. If all boundary street and drainage improvements are in place, the seal and signature of a registered architect or landscape architect is acceptable.

The stormwater pollution prevention plan (SWPPP) shall be included with the grading & land alteration permit submittal to verify compliance with state's MS4 permit agreement with the City. A Grading and Drainage Plan Checklist has been prepared and is included in Appendix A.

9.1.4 Other permit Requirements

Application to Develop in a Flood Hazard Area

Proposed development within Special Flood Hazard Areas of the City requires the developer or their agent to obtain and complete an Application and Permit Form to Develop in a Flood Hazard Area. Work within the 100-year floodplain requires the applicant to complete FEMA Form TOD-1: Certification/Application Forms for Letters of Map Amendment/Revision Based on Fill. Work within the regulatory floodway, including changes in base flood elevations, fill, channelization, and bridge/culvert replacement projects require the applicant to prepare and submit applicable portions of FEMA Form RSD-1: Revisions to National Flood Insurance Maps. Additional information is available from the City's Floodplain Administrator.

Section 10 of the Rivers and Harbors Act of 1899

This Act prohibits the obstruction or alteration of navigable waters of the Unites States without a permit. Section 10 Permits are issued by the United States Army Corps of Engineers Permits Branch, who should be contacted at (501) 324-5295 for additional information.

Clean Water Act Section 404 Permits

Section 301 of this Act prohibits the discharge of dredged or fill material into waters of the United States without a permit. Section 404 Permits are issued by the United States Army Corps of Engineers. Contact the United States Army Corps of Engineers Permits Branch at (501) 324-5295 for more information.

Arkansas Solid Waste Management Code of 1984

The Arkansas Department of Pollution Control and Ecology authorizes all legitimate fill operations. An Application for Request for Fill Area is required to be completed and approved for development consisting of fill for the purpose of surface leveling. Contact the Solid Waste Division of the Arkansas Department of Pollution Control and Ecology at (501) 570-2858 for more information.

Arkansas State Water and Air Pollution Control Act

The Arkansas Department of Pollution Control and Ecology authorizes the discharge of stormwater associated with industrial activity from construction sites - those areas or common plans of development or sale that will result in the disturbance of five or more acres total land area. The Department has issued General Permit ARR10A000 for construction activities. A Notice of Intent is required at least 48 hours prior to commencing land disturbance activities. Contact the NPDES Branch of the Arkansas Department of Pollution Control and Ecology at (501) 562-7444 for more information.

Burning Permit

Site clearing often generates timber debris which may be mulched or burned on site. Burning requires approval from the City Fire Marshall on a permit form furnished by the Fire Department. Contact the Fire Department at (501) 371-4796 for more information, and to obtain permit forms.

Burning of demolition and construction debris is regulated by the Arkansas Department of Pollution Control and Ecology Air Division. Contact the Air Division at (501) 570-2161 for more information.

9.2 Sediment Control Criteria and Requirements

Stabilization

A record of the dates when grading activities occur, when construction activities temporarily or permanently cease on a portion of the site except as provided within bulleted text below, and when stabilization measures are initiated shall be included in the erosion and sediment control plan. Stabilization measures shall be initiated as soon as practicable in portions of the site where construction activities have temporarily or permanently ceased, but in no case more than 14 days after the construction activity in that portion of the site has temporarily or permanently ceased.

- Where the initiation of stabilization measures by the 14th day after construction activity temporarily or permanently ceases is precluded by snow cover, stabilization measures shall be initiated as soon as practicable.
- Where construction activity will resume on a portion of the site within 21 days from when activities ceased, (e.g., the total time that construction activity is temporarily ceased is less than 21 days) then stabilization measures do not have to be initiated on that portion of the site by the 14th day after construction activity temporarily ceased.

Stabilization practices may include temporary seeding, permanent seeding, mulching, geotextiles, sod stabilization, vegetative buffer strips, protection of trees, and preservation of mature vegetation and other appropriate measures. See Section 15-52 of the Little Rock City Code for tree protection requirements.

9.3 Stabilization Practices

EPA NPDES Fact Sheets for the following Erosion and Sediment Control BMPs can be found on their website under the National Menu of Best Management Practices (BMPs) for Stormwater-Construction | US EPA.

Examples of construction site stabilization practices:

- **Chemical Stabilization**
- **Filter Strips**
- **Preservation of natural vegetation**
- **Seeding**
- **Sod stabilization**
- **Stream bank stabilization**
- **Subsurface drains**

Chemical Stabilization. Chemical stabilization practices involve spraying soil surfaces with various man-made materials to hold the soil in place temporarily. This control is an alternative where temporary seeding is not practical because of the season or climate. Chemical stabilization can provide immediate and effective erosion control anywhere on a construction site.

Existing and Natural vegetation. Every means shall be taken to conserve and protect existing vegetation. The potential for soil loss shall be minimized by retaining natural vegetation wherever possible. Development in the Hillside/Hilltop Overlay District should comply with the recommendations of the Hillside/Hilltop Best Management Practices Manual about the retention of natural vegetation on Hillside/Hilltops.

Establishing New Vegetation. Vegetation practices may be either temporary or permanent and, at a minimum, should comply with Little Rock City Code Section 29-197, Advanced Grading Plan Requirements. They may be applied singularly or in combination with other practices. Cutting, filling, and grading soils with heavy equipment results in areas of exposed subsoils or mixtures of soil horizons. Conditions such as acidity, low fertility, compaction, and dryness or wetness often prevail and are unfavorable to plant growth and should be accounted for in the selection of plantings is required as specified for each BMP.

Long slopes and steep grades shall not be created. Stormwater drainage structures where such conditions already exist are normally subjected to hydraulic forces requiring both special establishment techniques and grasses that have high resistance to scouring. Vegetation practices and structural techniques are available to provide both temporary and permanent protective cover on these difficult sites, where encountered.

Filter Strips. These are areas of the site left undisturbed by clearing and construction activities. They are like buffer zones, slowing runoff velocity to allow sediment to settle out. However, filter strips serve the additional purpose of allowing runoff to infiltrate into the ground. Filter strips are an effective control applicable to sites where adequate space exists to leave undisturbed areas. Filter strips should be aligned perpendicular to the line of flow and can be used along with diversion ditches or berms to direct flow onto the vegetated surface area.

Temporary Vegetation. Earth moving activities such as heavy cutting, filling, and grading are generally performed in several stages and are often interrupted by lengthy periods, during which the land lies idle and is subject to accelerated erosion especially during rainfall events. In addition, final land grading may be completed during a season not favorable for immediate establishment of permanent vegetation. In such conditions, rapid growing annual grasses shall be used to rapidly establish protective cover. This can later be worked into the soil for use as mulch when the site is prepared for establishment of permanent vegetation.

Permanent Vegetation. Final selection should be based on adaptation of the plants to the soils and climate, suitability for their specific use, ease of establishment, longevity, or ability to reseed, maintenance requirements, aesthetics, and other special qualities. Maintenance must be the most important consideration in selecting plants for permanent stabilization.

Plants that provide long-lived stabilization with the minimum amount of required maintenance should be selected. Where management potential is limited because of specialized circumstances, the best plants to choose are those that are well adapted to the site and to the specific purpose for which they are to be used. For example, grasses used for waterway stabilization must be able to withstand submergence and provide a dense cover to prevent scouring of the channel boundary.

In playgrounds, grasses must lend themselves to close grooming and be able to withstand heavy trampling. In some places, such as southern-exposed cut-and-fill slopes, the plants must be adapted to full sunlight and drought conditions. In other places, plants must be able to tolerate shade or high moisture conditions. Some plants can be used for beautification as well as for soil stabilization.

9.4 Erosion Control Methods

Control of erosion during construction requires an examination of the entire site to identify potential problem areas such as steep slopes, highly erodible soils, soil areas that could be unprotected for long periods or during peak rainy seasons, and natural drainageways. Assure erosion control in these critical areas. After a rain, the effectiveness of erosion control measures must be re-evaluated. Maintenance and cleaning of these facilities is also important.

Examples of Erosion Control Methods:

- **Buffer zone**
- **Compost Blankets**
- **Drainage Swale**
- **Diversion Dike**
- **Geotextiles**
- **Gradient Terraces**
- **Interceptor Dikes and Swales**
- **Mulching**
- **Outlet Protection**
- **Riprap**
- **Seeding**
- **Sodding**
- **Soil Retention**
- **Surface Roughening**
- **Temporary Slope Drain**

Additional items with respect to City erosion control requirements are provided herein:

Buffer zones. These are vegetated strips of land which control erosion by reducing the speed of runoff. Buffer zones can be areas left undisturbed during clearing and construction (filter strips), or they can be newly planted in areas that were previously disturbed by clearing and site activity.

Diversion Dike. This is a compacted earthen ridge constructed immediately above a cut or fill slope. Its purpose is to intercept storm runoff from upstream soil drainage areas and divert the water away from the exposed stabilized **outlet.**

Interceptor Dike. This is a temporary ridge of compacted soil or, preferably, gravel constructed across disturbed rights-of-way. An interceptor dike reduces erosion by intercepting stormwater and diverting it to stabilized outlets.

Perimeter Dike. This is a compacted earthen dike constructed along the perimeter of a disturbed area to divert sediment-laden stormwater to onsite trapping facilities. It is maintained until the disturbed area is permanently stabilized.

Flexible Down Drain. This is a temporary structure used to convey stormwater from one elevation to another without causing erosion. It is made of heavy-duty fabric or other material that can be removed when the permanent water disposal system is installed.

Mulching. When final grading has not been completed, straw, wood chips, jute matting, or similar materials can be applied to provide temporary protection. Areas brought to final grade during midsummer or winter can be mulched immediately and overseeded at the proper season with several permanent grasses or legume species. Application of mulch to disturbed areas allows for more infiltration of water into the soil, reduces runoff, holds seed, fertilizer, and lime in place, retains soil moisture; helps maintain temperatures conducive to germination, and greatly retards erosion. Mulch is essential in establishing good stands of grasses and legumes in disturbed areas. It is important to stabilize or anchor mulch using such practices as an anchoring tool, biodegradable tackifier (hydromulch), netting, peg and twine, or slitting to prevent it from blowing or washing off the site. Use of mulch in combination with Green Stormwater Practices shall comply with the requirements established in Chapter 8 and Appendix C.

Riprap. This is a layer of loose rock placed over the soil surface to prevent erosion by surface flow or wave action. Riprap may be used, as appropriate, at storm drain outlets, channel bank and bottom protection, roadside ditch protection, drop structures, etc.

Storm Drain Outlet Protection. This practice involves putting paving or riprap on channel sections immediately below storm drain outlets. A storm drain outlet is designed to reduce the velocity of flow and prevent downstream channel erosion. It is also known as an energy dissipator.

Temporary Storage, Shop and Staging Areas. Locate storage and shop yards where erosion and sediment hazards are slight. If this is not feasible, apply necessary paving and erosion control practices.

9.5 Siltation and Sediment Control

Examples of Siltation and Sediment Control Methods:

- **Compost Filter Berms**
- **Construction Entrance/Exits**
- **Compost Filter Socks**
- **Dust Control**
- **Fiber Rolls**
- **Filter Berms**
- **Sediment Basins and Rock Dams**
- **Sediment Filters and Chambers**
- **Sediment Traps**
- **Silt Fences**
- **Storm Drain Inlet Protection**
- **Temporary Storm Drain Diversion**
- **Temporary Stream Crossing**
- **Vegetated Buffers**

Additional items with respect to City sediment control requirements are provided herein:

Control and prevention of soil erosion during and after construction is the most important element of siltation and sediment control. However, it is physically and economically impractical to eliminate all soil erosion. Therefore, provisions must be made to trap eroded material at specified points. Some measures to implement are as follows:

- As inlet protection and on long slopes or runs, silt fence or rock check dams shall be used to create temporary ponds that store runoff and allow suspended solids to settle. These temporary ponds may be retained as part of the permanent storage system after construction; however, they must be inspected / surveyed to ensure that the design capacity of the system was not compromised by siltation.
- Inlet protection shall be maintained throughout construction and shall not be removed until vegetation is established. Such measures shall be periodically inspected in accordance with the requirement of the SWPPP and repaired / replaced when no longer functioning in accordance with design. Silted-in areas shall be mucked out after significant rainfall to restore capacity.
- Egress points from construction sites should be controlled so that the sediment is not carried offsite by construction traffic. A temporary construction entrance shall be constructed at points where traffic will be entering from or leaving a construction site to public right-of-way, street, alley, sidewalk, or parking area. Its purpose is to reduce or eliminate the transport of mud from the construction area onto the public right-of-way by motor vehicles or by runoff. An additional track-out area should be established where appropriate if traffic from heavy equipment is limited to areas not typically disturbed by passenger vehicles entering / leaving the construction site.

Construction Entrance/Exits. A stabilized rock exit is required on construction sites. Rock exits must be at least 20 feet wide by 20 feet long (1 & 2 family residential) or 50 feet long (all other construction sites) by 6-inch-thick stabilized rock having a minimum average diameter of 3 inches. If there is an existing curb, loose material such as fill dirt or gravel shall not be used to ramp up to it from the street. Temporary wooden ramps in front of curbs are acceptable.

Dust Control. Saturate ground or apply dust suppressors. Keeping dust down to tolerable limits on the construction site and haul roads is very important.

Outlet Protection. Outlet protection reduces the speed of concentrated stormwater flows. Stone, riprap, concrete aprons, paved sections and settling ponds below outlets prevent scouring and erosion around the outlet. Outlet protection should be applied at locations of all pipes, dike, swale, and channel outlets. Outlet protection should be installed early in the development process and can be added later as necessary to prevent erosion.

Sediment Basins and Rock Check Dams. A rock check dam is an auxiliary structure installed in combination with and as a part of a diversion, interceptor, or perimeter dike, or other structures designed to temporarily detain sediment-laden stormwater. The rock check dam provides a means of draining off and filtering the stormwater while retaining the sediment behind the structure.

Sediment basins can be used to trap runoff waters and sediment from disturbed areas. The water is temporarily detained to allow sediment to drop out and be retained in the basin while the water is automatically released. Sediment basins usually consist of a dam or embankment, a pipe outlet, and an emergency spillway. They are usually situated in natural drainageways or at the low corner of the site. In situations where embankments may not be feasible, a basin excavated below the earth's surface may serve the same purpose. A special provision, however, must be made for draining such an impoundment.

Sediment basins may be temporary or permanent. Temporary basins serve only during the construction stage and are eliminated when vegetation is established, and the area is stabilized. Permanent structures are designed to fit into the plan for the permanent installation. Design shall conform to the requirements within this manual. State and local safety regulations must be observed regarding design, warning signs, and fencing of these structures.

Sediment Traps. A sediment trap is a structure of limited capacity designed to create a temporary pond around storm drain inlets or at points where silt-laden stormwater is discharged. It is used to trap sediment on construction sites, prevent storm drains from being blocked, and prevent sediment pollution of watercourses. Sediment traps should also be located where permanent Stormwater Control Measures (SCM) will be constructed.

Silt Fences. This is a temporary barrier constructed across or at the toe of the slope. Its purpose is to intercept and detain sediment from areas one-half acre or smaller where only sheet erosion may be a problem.

Dewatering. All rainwater pumped out of sumps and depressions on construction sites should be clear and free of sediment, and must discharge to a sedimentation pond, sediment bag, or settling tank in such a manner as to not cause additional erosion problems.

Water Control. Subsurface drains used to remove excess groundwater are sometimes required at the base of fill slopes or around building foundations. When heavy grading is done and natural water channels are filled, the subsurface drains may be used to prevent accumulation of groundwater. Subsurface drains may be needed in vegetated channels to lower a high-water table and to improve drainage conditions so vegetation can be established and maintained.

9.6 Design of Erosion and Sediment Controls

Universal Soil Loss Equation

The Universal Soil Loss Equation adopted by the City of Little Rock is:

$$\boldsymbol{A = R(K)(L)(S)(C)(P)} \qquad \textbf{Eq. 9.1}$$

Where: A= soil loss in tons per acre per year

R = rainfall and runoff factor

K = soil erodibility factor

L = slope length factor

S = slope-steepness factor

c = cover and management factor

P = practice factor

The rainfall and runoff factor (R) is 300 for Pulaski County, Arkansas. Therefore, the Universal Soil Loss Equation takes the following form:

$$\boldsymbol{A = 300(K)(L)(S)(C)(P)} \qquad \textbf{Eq. 9.2}$$

Soil erodibility factors K are presented in Table D-1 (Appendix D) for all soils identified by soil classification within Pulaski County, Arkansas. Soil association maps are found in the Soil survey of

Pulaski County, Arkansas, published by the USDA Soil Conservation Service (See References, Appendix E.)

The slope length factor (L) and the slope-steepness factor (S) can be combined and identified as the topographic factor (LS). Values for LS are found in Table D-2 (Appendix D), which identifies the topographic factor for specific combinations of slope length and steepness.

The cover and management factor (C) represents the ratio of soil loss from land managed through mulching, vegetation, and revegetation to soil loss from disturbed and unprotected lands. Where site preparation removes all vegetation and the root zone of plants, the soil is left completely unprotected and the value of C = 1. Cover and management factors for mulching are presented in Table D-3 (Appendix D). Cover and management factors for vegetation practices are presented in Table D-4 (Appendix D).

The practice factor (P) represents the ratio of soil loss with a specific management practice to the corresponding loss from unprotected slopes. Where no management practice for erosion control is provided, the value of P = 1. Practice factors for gradient terraces, earth dikes, and interceptor dikes and swales are presented in Table D-5 (Appendix D). Practice factors for buffer zones, filter strips and natural vegetation are presented in Table D-6 (Appendix D).

The above information, when incorporated into the Universal Soil Loss Equation, produces the following:

$$\boldsymbol{A = 300(K)(LS)(C)(P)}$$ **Eq. 9.3**

Where: A = soil loss in tons/acre/year

R = 300, a constant

K = soil erodibility factor (Table D-1)

LS = slope length factor (Table D-2)

C = cover & management factor (Tables D-3,4)

P = practice factor (Tables D-5, 6)

9.7 Reference

Drainage Criteria Manual | Fayetteville, AR - Official Website (fayetteville-ar.gov)

LRDM 2016