Drainage Criteria Manual

July 1, 2014

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CHAPTER 1. MINIMUM STORMWATER STANDARDS AND SUBMITTAL REQUIREMENTS

SECTION 1.1. GENERAL

1.1.1 Stormwater Management, Drainage, and Erosion Control Ordinance

This document adopted by ordinance No. 3895 of the City of Fayetteville, provides technical procedures and design standards to support Chapter 170: Stormwater Management, Drainage, and Erosion Control of the Title XV Unified Development Code (UDC).

All development projects meeting the applicability criteria, as stated in Chapter 170.03 of the UDC, shall obtain a Grading and Drainage Permit.

A Stormwater Management, Drainage and Erosion Control Permit is required from the City of Fayetteville for all activities which develop, change to a more intensive land use, construct or reconstruct a structure, or change the size of a structure, or conduct grading, clearing, or filling activities within the corporate limits of the City of Fayetteville.

Exceptions where no drainage permit is required are as follows:

- One single-family residence or duplex (see Section 170.10 of the UDC for one- and two-family residential requirements).
- One commercial or industrial project built on an individual lot that is part of a larger subdivision that has been issued an approved drainage permit when the proposed project is demonstrated to be in compliance with the overall subdivision drainage permit.
- Existing commercial or industrial structure where additional structural improvements are less than 2,000 square feet.
- Maintenance or clearing activity that does not change or affect the quality, rate, volume, or location of stormwater flows on the site or runoff from the site.
- Bona fide agricultural pursuits for which a soil conservation plan has been approved by Washington County Soil and Water Conservation District.
- Action taken under emergency conditions, either to prevent imminent harm or danger to persons, or to protect property from imminent danger of fire, flooding, or other hazards.

The application for a Grading and Drainage 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 Drainage Permit application shall consist of a Transmittal Letter, the Final Drainage Report, and the Grading and Drainage Design Plans and Specifications (Plans and Specifications). The Final Drainage Report Checklist in Section 1.4.3 shall be filled out and signed and sealed by the Engineer of Record.

SECTION 1.2. ADDITIONAL REGULATORY REQUIREMENTS

1.2.1 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: http://www.adeq.state.ar.us/water

1.2.2 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.

1.2.3 Floodplain Development Permits

Any development within or bordering a Special Flood Hazard Area, as portrayed on FEMA Flood Insurance Rate Maps (FIRMs), or bordering a Protected Stream, as portrayed on the Protected Streams Map, is required to obtain a Floodplain Development Permit. Permit requirements and application procedures can be found in Section 168.07 of the City of Fayetteville Unified Development Code.

SECTION 1.3. SUBMITTAL PROCEDURES

1.3.1 Conceptual Review

A preliminary meeting (prior to technical plat review or development of construction documents) with the engineering staff is suggested before developing site improvement plans.

1.3.2 Technical Plat Review

A Grading and Drainage Permit application consisting of a Transmittal Letter, Preliminary Grading and Drainage Report (see Subsection 1.4.2) and Preliminary Grading, Drainage, and Erosion Control Plans shall be submitted to the Planning Department for Technical Plat Review for all site development submittals. The Technical Review meeting will be scheduled by the City of Fayetteville with representatives of the developer, including the Engineer, to review the overall concepts included in the preliminary Grading and Drainage Permit application. The purpose of this review is to assess the overall stormwater management concept for the proposed development and to review criteria and design parameters that shall apply to final design of the project. The Urban Forester's review of the Tree Preservation Ordinance, for example, can impact the grading and stormwater design. Additional submittals may be required for subsequent meetings with the Subdivision Committee and Planning Commission.

1.3.3 Construction Plan Review

Following project approval by the Planning Department, the final civil site package consisting of a Transmittal Letter, Final Grading and Drainage Report (see Subsection 1.4.3) including checklist, grading, drainage, and erosion control plans and specifications shall be submitted to the Engineering Coordinator in .pdf format for review. Developments within the floodplain shall also gain approval from the Floodplain Administrator prior to issuance of a Grading and Drainage Permit. Additional submittals may be necessary. The application submittal shall be in accordance with Section 1.4, Grading and Drainage Permit Application Requirements.

Once the final package has been reviewed for compliance and compliance has been confirmed, a conditional approval letter will be issued to the design engineer. The conditional approval letter allows the contractor to install perimeter erosion control measures. Issuance of the Grading and Drainage Permit is also dependent upon the review and approval of the Urban Forester. Before issuance of the Grading and Drainage Permit the perimeter erosion controls and tree protection measures must be inspected and the preconstruction conference held.

Final approved plans shall be submitted to reviewing divisions for other development permits as needed.

1.3.4 Waivers

Only the City Engineer can grant waivers to the Minimum Standards or any other requirement of this Drainage Criteria Manual, if adequate documentation and supporting calculations are provided that demonstrate such a waiver is warranted. Proof of receipt of such waivers shall be provided to the City as part of subsequent submittals throughout the remainder of the project application process.

SECTION 1.4. GRADING AND DRAINAGE PERMIT APPLICATION REQUIREMENTS

1.4.1 Transmittal Letter

A cover letter shall be included with each submittal. The cover letter should include the Planning Project number assigned by the Planning Department and the project name.

1.4.2 Preliminary Grading and Drainage Report

A Preliminary 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 Section 1.4.3, items 1-15. Also submit preliminary grading and drainage drawings. Items 16 – 25 are not required for the Preliminary Report.

1.4.3 Final Grading and Drainage Report

A Final Grading and Drainage Report, following the Final Grading and Drainage Report Template as provided below, shall be included in the Final Grading and Drainage Permit Application. Computer input and output information shall also be provided as part of the Final Grading and Drainage Report. An example of input and output to report may be found at the end of Chapter 6, and a sample output sheet is provided in Appendix H.

A Grading and Drainage Permit will not be issued until the Final Report has been submitted, reviewed, and approved. The City 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: (1) onsite storage, (2) offsite storage, or (3) offsite drainage system improvements.

If it is determined by the City Engineer that offsite drainage improvements are required, then cost sharing will be in accordance with City ordinances (UDC Chapter 166.04 and UDC Chapter 170.06.F). 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.

Final Drainage Report Template and Checklist
The City of Fayetteville, Arkansas

Project name ____

Engineer of Record ____

Planning Project Number ______________

Revision no. ______________

Date ______________

1. ____ PROJECT TITLE & DATE
2. ____ PROJECT LOCATION - Include street address and Vicinity Map.
3. ____ PROJECT DESCRIPTION - Brief description of the proposed project.

4. ____ NAME, ADDRESS, TELEPHONE NUMBER, AND EMAIL of the owner and developer of the property to be permitted.
5. ____ NARRATIVE SUMMARY - The summary shall include a description of the methods used to meet the requirements of the Minimum Standards. This includes at a minimum a description of the treatment train for Minimum Standard #1 and a description of the detention strategy used to meet the requirements of Minimum Standards #2 through #4. Also include a description of the off-site areas, onsite areas, condition of the downstream receiving areas, existing problems, changes to flows and flow volume, proposed improvements, detention, areas with potential for high pollutant loading, and final conclusions.
6. ____ EXISTING DRAINAGE AREA MAP – Existing drainage area map on a 1-inch = 200-feet minimum scale plan drawing, with 2 foot contours (1 foot contours on “flat” sites), that includes: study points at property lines, time of concentration path, bar scale, and the following information:
   a. ____ Aerial photograph of the project vicinity, covering the project area and the total lands that contribute runoff;
   b. ____ Existing drainage areas and flow patterns to downstream property line, establishing the study points;
   c. ____ Upstream and downstream drainage flow paths for all areas that contribute runoff to the existing site or receive runoff from the site. The downstream area(s) shall be shown as necessary to document the receiving conveyance system; and
   d. ____ Existing land use conditions for the drainage areas that contribute runoff.
7. ____ SOIL MAP - Provide the most recent U.S. Soil Conservation Service soils and vegetation information for both the project area and the drainage area that contributes runoff on a separate map from the Existing Drainage Area Map.
8. ____ PROPOSED DRAINAGE AREA MAP – Proposed drainage area site map on a 1-inch = 200-feet minimum scale plan drawing, with 2’ contours (1 foot contours on “flat” sites), that include: study points, time of concentration path, bar scale, and the following information:
   a. ____ Proposed drainage areas and flow patterns and, if applicable, natural feature protection areas, green stormwater practice and infiltration areas;
   b. ____ Upstream and downstream drainage flow paths for all areas that contribute runoff to the proposed development site or receive runoff from the site. The downstream area(s) shall be shown as necessary to document the receiving conveyance system;
   c. ____ Proposed land use conditions for the development site and drainage areas that contribute runoff; and
   d. ____ Proposed locations of grading and placement of fill material within the project area and drainage areas that contribute runoff.
9. ____ MINIMUM STANDARD #1, Water Quality – Calculations and documentation indicating the percentage of the total suspended solids that the proposed stormwater management system is capable of removing from the stormwater flows EXCEEDING predevelopment levels.
   a. ____ If the TSS Reduction Method described in Chapter 4 is used to meet this standard,

i. provide calculations for each structural control indicating the corresponding level of treatment; and

ii. provide a map showing the impervious area and structural controls

b. ____ If Green Stormwater Practices (GSPs) described in Chapter 5, Low-Impact Development, are used to meet this standard,

i. provide calculations determining the total Volumetric Runoff Coefficient (Rv) for the site. The printed output from the Low Impact Development calculation spreadsheets provided by the City of Fayetteville shall be submitted to confirm the level of treatment provided by the design; and

ii. provide a map showing the impervious areas and GSPs.

c. ____ If the goal of 80% removal is not practicable, provide a description explaining why 80% removal is not achievable.

d. ____ FLOW TABLE - Provide a summary of peak discharges table that lists the pre-development, the post-development without mitigation, and the post-development with mitigation flow rates for the 2-, 10-, 25-, and 100-year storm events for each study point.

10. ____ MINIMUM STANDARD #2, Channel Protection – Provide calculations and documentation indicating compliance with Minimum Standard #2 such that the 1-year post development site flow is captured. The calculations shall include the following information:

a. ____ Calculations of the predevelopment 1-yr, 24-hour peak discharge and whether it is >2cfs. If below 2 cfs, the CPv requirement shall not apply.

b. ____ Calculations and documentation showing that the 1-year, 24-hour storm volume is captured and released over a period of 40 hours.

c. ____ The calculated discharge velocity for the 10-year, 24-hour storm event. If the discharge velocity exceeds or is near to the erosion velocity of the downstream channel system, then energy dissipation, erosion prevention measures, and/or velocity control measures shall be designed to control the velocity of or mitigate erosion potential from the 10-year, 24-hour storm event.

d. ____ Calculations for the energy dissipation measures, if required, to reduce the discharge velocity calculated above.

e. ____ The design shall also comply with all requirements of the City of Fayetteville Streamside Protection Ordinance (UDC Chapter 168.12).

11. ____ MINIMUM STANDARD #3, Overbank Flood Protection – Provide calculations and documentation indicating compliance with Minimum Standard #3 such that the post-development peak discharge rate does not exceed the predevelopment rate for the 2-year, 5-year, 10-year, and 25-year, 24-hour storm events. The calculations shall include the following information:

a. ____ A summary table of runoff discharge flows for the 2-year, 5-year, 10-year and 25-year, 24-hour storm events for the pre-development and post-development conditions for each study point. The summary shall include the existing and proposed flows along with supporting calculations for all of the discharge points to the receiving system. This includes the flow entering each drainage area and the flow generated within each drainage area on the site (do not separate onsite and offsite flows).

12. ____ MINIMUM STANDARD #4, Extreme Flooding – Provide calculations and documentation indicating compliance with Minimum Standard #4, such that the post-development peak discharge rate does not exceed the predevelopment rate for the 100-year, 24-hour storm event. The calculations shall include the following information:

    a. ____ Calculations and results of the extreme flood analysis showing that the 100-year, 24-hour storm event has been controlled as required by Minimum Standard #4 such that the post-development peak discharge rate does not exceed the pre-development rate for this event.

    b. ____ The effects of the 100-year, 24-hour storm event (Q_f) on the stormwater management system, adjacent property, and downstream facilities and property shall be evaluated. The Q_f shall be controlled through the use of structural stormwater controls to protect existing downstream property with no increase in the existing base flood elevation, or calculations shall be provided to indicate that the on-site conveyance system will safely pass Q_f and allow it to discharge into receiving waters where the floodplain is of capacity sufficient to accommodate significant additional discharges without causing damage.

    c. ____ A summary table of discharges for the 100-year 24-hour storm event for the pre-development and post-development conditions for each study point.

13. ____ CHECK FOR EXISTING DOWNSTREAM FLOODING – Describe the existing downstream capacity of each receiving area (study point). Provide documentation of an assessment of downstream conditions a minimum of 1/4 mile downstream of the proposed development in accordance with Section 7.5. Documentation shall include photographs of the existing structures downstream of the development as well as a map showing the locations and distances of downstream structures from the development.

14. ____ FLOW TABLE – List in a summary table the pre-development, post-development without mitigation, and post-development with mitigation flow rates for the 2-, 10-, 25-, and 100-year, 24-hour storm events for each study point.

15. ____ STORMWATER DETENTION DESIGN – If detention is required to comply with the Minimum Standards, include all computations and backup/support data including:

    a. ____ Detention basin size requirement computations (using an approved method).

    b. ____ Release structure design computations including design Water Surface Elevations for the 1- (where required), 10-, 25-, and 100-year storms. If extended detention of 1-year is not provided, design computations for the 2- and 5-year storm are also required.

    c. ____ Stage-Storage and Stage-Discharge curves for the detention facility.

    d. ____ A summary hydrograph of the effect of the detention facility for relevant storms, incorporated with bypass.

    e. ____ Overflow structure(s) size and location(s);

    f. ____ Outfall structure(s), location(s), and orifice size(s).

    g. ____ Emergency overflow path.

h. ____ Results of downstream analysis.

16. ____ PAVEMENT DRAINAGE DESIGN - Include a table listing street classification, width, allowable spread and actual spread for 10 and 100-year, 24-hour storm.

17. ____ STORM SEWER INLET DESIGN - Include all computations for the 10 and 100-year, 24-hour storm. Reference table in Chapter 6 for allowable spread and depth.

18. ____ INLET DRAINAGE AREA MAP – Provide a separate map showing the inlet layout and design including the drainage areas. The map should include the proposed design, drainage areas, time of concentrations paths, runoff coefficients, and bar scale.

19. ____ STORM SEWER DESIGN - Include all computations and hydraulic profiles for the 10 and 100-year, 24-hour storms.

20. ____ CULVERT DESIGN - Include all computations, hydraulic profile, and energy transition to channel.

21. ____ OPEN CHANNEL FLOW DESIGN - Include computations for normal depth and velocity.

22. ____ FEDERAL AND STATE REQUIREMENTS (Answer <u>Yes</u> or <u>No</u> if required).

a. ____ Wetlands determination (if wetlands are present on the site).

b. ____ 404 permit required (include letter from USACE as an exhibit).

c. ____ NPDES Construction Stormwater "Notice of Intent" (ADEQ)(include as an exhibit if required).

d. ____ ANRC permit/review for "dams"(required if a stormwater impoundment qualifies as a dam per ANRC regulations).

e. ____ Other____

23. ____ EXHIBITS – Attach the following exhibits to the final drainage report.

a. ____ Grading and drainage construction drawings.

b. ____ Landscaping Plan.

c. ____ Operations and maintenance plan (see Section 7.4.12).

d. ____ Letter from USACE if answered Yes to 23.b. above.

e. ____ Notice of Coverage (NOC) and completed SWPPP (sites 1 acre or larger).

f. Master Drainage Plan.

24. The following paragraph with relevant information included:

"I, ______________________ , Registered Professional Engineer No. _________ in the State of Arkansas, hereby certify that the drainage studies, reports, calculations, designs, and specifications contained in this report have been prepared in accordance with sound engineering practice and principles, and the requirements of the City of Fayetteville. Further, I hereby acknowledge that the review of the drainage studies, reports, calculations, designs, and specifications by the City of Fayetteville or its representatives cannot and does not relieve me from any professional responsibility or liability."

____
Signed & Sealed by Professional Engineer

25. ____ ARKANSAS REGISTERED ENGINEER SEAL

1.4.4 Plans and Specifications

Grading and Drainage Plans and Specifications are to be signed and sealed by a professional engineer registered in the State of Arkansas in accordance with applicable state statutes and State PELS board licensure requirements. Because plans, specifications, and calculations may 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. A complete legend shall be included in the set of Plans.

Plan sheet size will be 22 inches x 34 inches with all sheets in a given set of plans the same size. Plan drawings will be prepared with a maximum horizontal scale of 1 inch = 100 feet unless otherwise approved by the City Engineer. Profile drawings for storm sewers should be drawn at a suggested horizontal scale of 1 inch = 20 feet, with a minimum scale of 1 inch = 50 feet and a maximum vertical scale of 1 inch = 5 feet. Drainage ditch profiles should be drawn at a suggested horizontal scale of 1 inch = 20 feet, with a minimum scale of 1 inch = 50 feet and a maximum vertical scale of 1 inch = 5 feet. Special cases may warrant use of larger or smaller scale drawings for increased clarity or conciseness of the plans and may be used with prior permission from the City Engineer.

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.

Plans and Specifications for the proposed improvements will 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, Drainage, Paving, and/or Building Plans, (4) Erosion and Sedimentation Control Plan, (5) Plan and Profile Sheet(s), and (6) Standard and Special Detail Sheets.

1.4.4.1 Title Sheet

Title sheet shall include:

1. The designation of the project, which includes the nature of the project, legal description, the name or title, city, and state;
2. Planning Department project number;
3. Index of sheets;
4. Vicinity map showing project location in relation to streets, railroads, and physical features. The map shall have a north arrow and appropriate scale;
5. A project control benchmark identified including the location and elevation with notation referencing City monument(s) used to establish the Project Benchmark;
6. Reference to horizontal and vertical datum for the project;

7. The name, address, telephone number, and email address of the owner of the project and the name, address, telephone number, and email address of the engineer preparing the plans;
8. Floodplain statement identifying the FIRM panel, date, and flood zone; and,
9. Engineer's seal (every sheet).

1.4.4.2 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. Location and description of existing major drainage facilities within or adjacent to the project area.
6. Location of major proposed drainage facilities.
7. Location and dimensions of proposed drainage and utility easements.
8. Name of each utility within or adjacent to the project area.
9. Standard notes.
10. 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 sheet.

1.4.4.3 Other Requirements for Plans and Specifications

1. Topographic sheet size will be 22 inches x 34 inches with all sheets in a given set of topographic surveys the same size. Topographic drawings will be drawn to a recommended horizontal scale of 1 inch = 20 feet having a contour interval of 1 foot, with a maximum horizontal scale of 1 inch = 100 feet and a maximum contour interval of 2 feet. Special cases may warrant use of larger or smaller scale drawings for increased clarity or conciseness of the site topography and may be used with prior permission of the City Engineer.
2. All topographic surveys shall meet those specifications found in Section E, NSPS MODEL STANDARDS FOR TOPOGRAPHIC SURVEYS as approved March 12, 2002.
3. The horizontal datum shall be NAD83, Arkansas State Plane, North Zone. The units shall be U.S. Survey Foot.
4. The vertical datum shall be North American Vertical Datum of 1988 (NAVD88).
5. At least two horizontal control monuments shall be shown on each sheet. At least one benchmark shall be shown on each sheet. A horizontal and vertical tie to at least one City of Fayetteville GPS monument shall be made and the results provided to the City Surveyor.

6. Proposed Grading, Drainage, Paving, and Building Plan showing details of proposed grading, drainage, paving, and buildings. The required details for all plans, including street plans, water and sewer plans, grading and drainage plans shall include all information deemed necessary by the City Engineer for review.
7. The drainage plan (and/or street and drainage plan) shall include plan and profiles of streets and storm drainage, street cross-sections, and details of all drainage systems and appurtenances.
8. Include the Master Residential Lot Grading Plans as an Exhibit to the Plans.
9. Green Stormwater Practices (GSPs) shall be shown on the Plans and Specifications and shall be designed and constructed in accordance with the standards and criteria in Chapter 5, Low Impact Development, and relevant Appendices of the Drainage Criteria Manual.
10. Erosion and Sediment Control Plan identifying the type, location, and schedule for implementing erosion and sediment control measures, including appropriate provisions for maintenance and disposition of temporary measures.
11. Operation and Maintenance Plan, included as a separate exhibit, prepared by a registered professional engineer, describing the activities and schedule required to operate and maintain the permitted facilities. The plan should include specific details such as when and what to mow and when to remove sediment.
12. Elevations on profiles, sections, and plans shall have the vertical datum designated. At least one permanent bench mark in the vicinity of each project shall be noted on the first drawing of each project, and the location and elevation of each benchmark shall be clearly defined.
13. The top of each page shall be either north or east. The stationing of street plans and profiles shall be ascending from left to right and downstream to upstream in the case of channel improvement/construction projects, unless otherwise approved by the City Engineer.
14. Each project shall show topographic data extending a minimum of 20 feet beyond each side of the project area or property boundary. Topographic data extending a minimum of 50 feet beyond the property boundary shall be shown in areas of channel or overbank flow. Provide finished floor elevations (FFE) of existing structures within the extent of topographic data. Any proposed changes, including utilities, telephone installations, etc., shall be shown on the plans and profiles.
15. Revisions to drawings shall be indicated above the title block in a revision block. The revisions shall be annotated and the revision block shall describe the nature and date of the revision made.
16. Standard symbols for engineering plans shall be used and a legend of symbols provided. Existing utilities, telephone installations, sanitary and storm sewers, pavements, curbs, inlets, culverts, etc., shall be shown with a broken line; proposed facilities with a solid line; and land, lot, and property lines with a slightly lighter dashed line.
17. Easements, lot lines, and dimensions shall be shown where applicable. Drainage easements shall be provided in accordance with the following requirements:
    a. Drainage easements shall be a minimum of 20 feet.
    b. For pipe or culverts less than 36-inch in diameter or width, the drainage easement shall be measured from the center of the pipe or culvert. For pipes or culverts greater than 36-inch

diameter or width, the easement shall be a minimum of 10 feet from the outside edges of the pipe or culvert.

c. For channels, a minimum drainage easement of 20 feet shall be provided. However, in addition to the minimum drainage easement width, 10 feet shall be provided from one side top of bank for access to the channel. The 10-feet access may be provided within the minimum 20 feet, if space permits. No fences shall be permitted in the drainage easement.

18. The proposed water surface elevation (WSEL) and corresponding boundaries of inundation resulting from the 100-year storm for all overland flow, including flow in easements, streets, parking lots, swales and between lots shall be calculated and shown on the construction drawings and must also be included in the final plat.

19. Minimum floor elevation shall be shown a minimum of 2 feet above the computed 100-year base flood elevation (BFE) on each lot for which any of the following apply: where located in a designated floodplain, where in a localized low area due to surrounding topographic relief (existing or proposed), and where flooding is known to occur or where City Flood Damage Prevention Code (Ordinance 168) applies. Minimum floor elevations for other areas shall be a minimum of 1 foot above the calculated 100-year WSEL of open channels or swales or overland flow. Within designated regulatory floodplain areas or where City Floodplain Ordinance 168 applies, comply with all relevant FEMA regulatory requirements.

20. It shall be understood that the requirements outlined in these standards are only minimum requirements and shall only be applied when conditions, design criteria, and materials conform to the City's specifications and are normal and acceptable to the City Engineer. When unusual subsoil or drainage conditions are suspected, an investigation should be made and a special design prepared consistent with good engineering practice.

1.4.5 Drainage Criteria for Subdivisions

1. Preliminary Plats shall include a master drainage plan for each lot related to the proposed infrastructure and adjacent lots.
2. Preliminary Plats for residential subdivisions shall provide detailed drainage information including flow arrows and design spot elevations including the proposed finish floor elevation meeting the Arkansas Fire Prevention Code for building safety regulations for positive drainage of each lot.
3. Rear lot drainage easements for nonstructural grassed swales shall not overlap utility easements with above ground structures, i.e., electric transformers, gas meters, communication junctions, etc.
4. The Final Plat shall include the approved master drainage plan to be filed as a supplemental document. The scale shall be legible and approved by the City Engineer.

1.4.6 Project Closeout and Final Acceptance

1. At the completion of a project and prior to final acceptance by the City, the Engineer of Record shall submit the following information for approval by the City Engineer.

   a. As-built survey drawings. The as-built drawings shall include all storm drainage structures including elevations of the top of structures, inverts, pipe sizes, and any other critical design elevations. The as-built shall also include cross-sections of any detention or retention basins. The cross-sections shall provide enough information to calculate the volume of detention provided by the basin.

b. Revised Drainage Report per as-built changes stamped and signed by the Engineer of Record in .pdf format. The Revised Drainage Report shall confirm the material and sizes of drainage pipes and structures, the general grading pattern, and verify drainage boundaries.

c. Certification of Detention supported by field data (cross-sections and as-built measurements).

d. Copies of reviewed and approved construction material or product submittals in .pdf format. Submittals must be reviewed and approved prior to installation.

e. Inspection reports and test reports submitted weekly.

f. Construction costs of public infrastructure for maintenance bond.

g. Final inspection by Design Engineer identifying all deficiencies.

h. Operation and maintenance plan, executed by financially responsible party.

SECTION 1.5. PERTINENT FAYETTEVILLE ORDINANCES

Refer to the following chapters of the UDC and additional documents for information as it relates to development and stormwater management within the City of Fayetteville.

Chapter 161 – Zoning Regulations

Chapter 166 – Development

Chapter 167 – Tree Preservation & Protection

Chapter 168 – Flood Damage Prevention Code

Chapter 169 – Physical Alteration of Land

Chapter 170 – Stormwater Management, Drainage, & Erosion

Chapter 171 – Streets and Sidewalks

Chapter 172 – Parking & Loading

Chapter 173 – Building Regulations

Chapter 177 – Landscape Regulations

Chapter 179 – Low Impact Development

City Plan 2030

City of Fayetteville Drainage Criteria Manual Appendix H Exhibits

City of Fayetteville Landscape Manual

City of Fayetteville Streamside Protection BMP Manual

City of Fayetteville Minimum Street Standards

City of Fayetteville Water and Sewer Specifications

CHAPTER 2. STORMWATER SIZING CRITERIA, PLANNING, AND REGULATIONS

SECTION 2.1. STORMWATER SIZING CRITERIA

2.1.1 Introduction

Development projects applying for a Grading and Drainage Permit shall meet the following four Minimum Standards related to stormwater runoff and protection of existing water bodies and properties. For the purposes of this Drainage Manual, predevelopment is defined as the existing conditions of the site at the time of development. The City of Fayetteville Stormwater Sizing Criteria Flow Chart should be used in conjunction with this manual to design stormwater management systems, to:

- Remove stormwater runoff pollutants and improve water quality (Minimum Standard #1);
- Prevent downstream streambank and channel erosion (Minimum Standard #2);
- Reduce downstream overbank flooding (Minimum Standard #3); and
- Control the peak flow rate of runoff from extreme storm events (Minimum Standard #4).

For these objectives, 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 the stormwater sizing criteria for stormwater control and mitigation.

Sizing Criteria	Description
Water Quality	1. TSS Reduction Method - Provide water quality treatment for the runoff resulting from a rainfall depth of 1.2 inches (where practicable) (Chapter 4), or 2. Runoff Reduction Method - Capture 1.0 inch of rainfall using Low Impact Development strategies. (Chapter 5). Methods are intended to reduce the average annual post-development total suspended solids loadings by 80% from increased impervious areas.
Channel Protection	Provide extended detention of the increased volume of the 1-year storm event released over a period of 40 hours to reduce flows and protect downstream channels from erosive velocities and unstable conditions. Post-development flows shall not exceed the predevelopment flows.
Overbank Flood Protection	Provide peak discharge control of the 2-year, 5-year, 10-year, and 25-year storm event such that the post-development peak rate does not exceed the predevelopment rate.
Extreme Flood Protection	Provide peak discharge control of the 100-year storm event such that the post-development peak rate does not exceed the predevelopment rate.

Each stormwater sizing criterion is intended to be used in 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.

Figure 2.1 illustrates the relative volume requirements of each stormwater sizing criterion and demonstrates that successive criteria are "stacked" upon the previous requirement - i.e., the extreme flood protection volume requirement also contains the overbank flood protection volume, the channel protection volume, and the water quality treatment volume. Figure 2.2 shows how these volumes would be stacked in a typical stormwater pond designed to handle all four criteria.

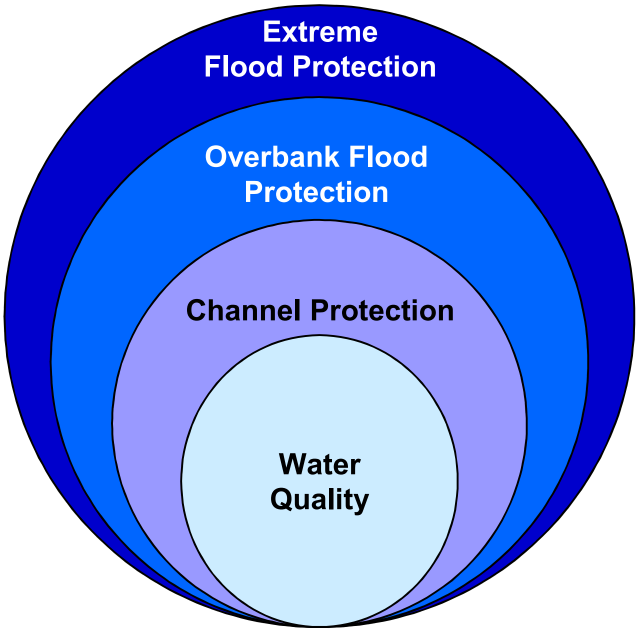

Figure 2.1. Representation of the stormwater sizing criteria.

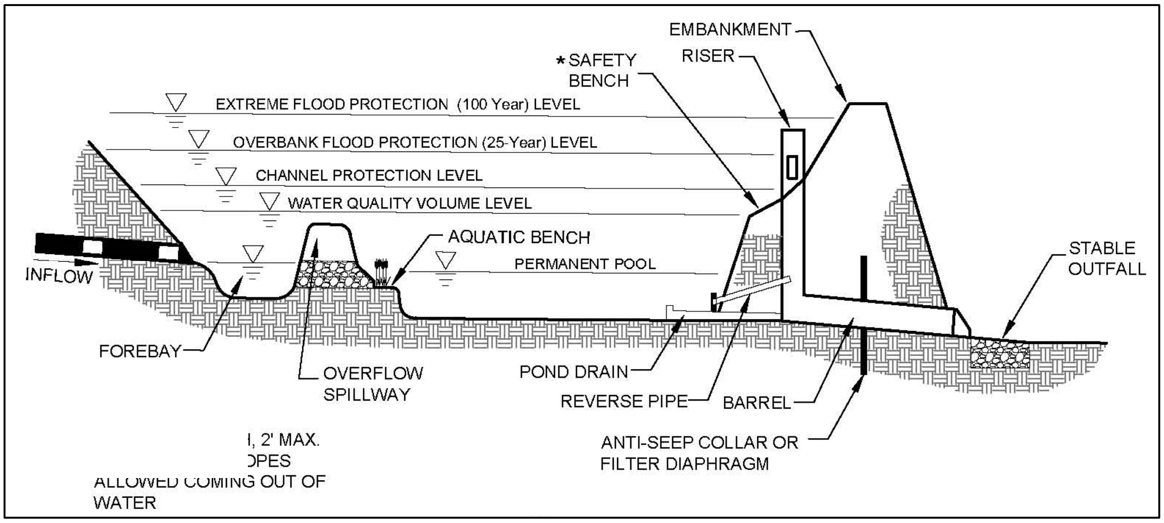

Figure 2.2. Sizing criteria water surface elevations in a stormwater pond.

The following text describes the four sizing criteria and presents information on how to properly compute and apply the required stormwater storage volumes. Additional detailed guidance regarding the computations is provided in Appendix E.

2.1.2 Minimum Standard #1 - Water Quality (WQ_v)

In accordance with the City's Municipal Separate Storm Sewer System (MS4) general permit under NPDES Permit Tracking No. ARR040010, the stormwater management system should be designed to remove at least 80% of the total suspended solids (TSS) from stormwater flows which exceed predevelopment levels (where practicable) and be able to meet any other additional watershed- or site-specific water quality requirements.

If a development project increases impervious area by more than 2000 SF, the stormwater management system shall be capable of removing at least 80% of TSS from an equivalent onsite impervious area. For example, if a development increases imperviousness onsite by 3,500 SF, then a minimum of 3,500 SF of impervious area must be treated to remove 80% of TSS. Note that any onsite impervious area (draining to a common study point) may be treated to meet this requirement – parking areas are preferred due to highest pollutant removal opportunity.

There are two methods that can be used to remove 80% of TSS from stormwater runoff:

- The TSS Reduction Method may be used to select appropriate structural stormwater controls and design a system or "treatment train" that removes 80% of the TSS from 1.2 inches of rainfall, the Water Quality Treatment Volume (WQ_v). Refer to Chapter 4 for information on the TSS Reduction Method and structural stormwater controls.
- Low Impact Development strategies and the Runoff Reduction Method (RRM) may be used to design a system that captures and treats the runoff volume resulting from the first 1-inch of rainfall onto the contributing area. Refer to Chapter 5 for information on Low Impact Development and the RRM. Appendices A, B, C, and D provide supporting information regarding Chapter 5 and Low Impact design.

The Water Quality sizing criterion specifies a treatment volume, denoted WQ_v, required to size structural stormwater controls to meet Minimum Standard #1 using the TSS Reduction Method. For the City of Fayetteville, this value is computed as 1.2 inches of rainfall over the catchment area multiplied by the runoff coefficient (R_v).

The Water Quality Volume is calculated using the formula below:

$$WQ_V = \frac{1.2 R_V A}{12}$$

Where:
- WQ_v = water quality volume (acre-feet)
- R_v = 0.05 + 0.009(I) where I is percent impervious cover within the project area (post-development)
- A = project area (acres)

Refer to Chapter 4 for detailed design guidance regarding water quality treatment. The equation above describes the sizing criteria for the TSS Reduction Method. The calculations and requirements for using the Runoff Reduction Method are described in Chapter 5.

2.1.3 Minimum Standard #2 - Channel Protection (CP_v)

Channel protection shall be provided to both downstream and on-site channels by meeting the following three requirements:

1. Capture the increased runoff volume from the 1-year, 24-hour return frequency storm event (3.36 inches in 24 hours) so the post-development volume does not exceed the pre-development volume, and release that volume of runoff over a period of 40 hours. The volume required to detain the 1-year storm over this extended period of time is called the Channel Protection Volume (CP_v). The CP_v is one measure of the stormwater sizing criteria that are used to size and design stormwater management facilities. If the TSS Reduction Method is being used to meet the Water Quality (WQ_v) minimum standard then the WQv may be subtracted from the total CP_v volume calculated;
   a. If the post-development 1-year 24-hour peak discharge flow rate is less than 2.0 cubic feet per second (cfs) then the CP_v detention is not required, but the post-development 1-year, 24-hour peak discharge rate shall not exceed the predevelopment rate.
2. Provide energy dissipation at outfalls to limit the velocity for the 10-year, 24-hour return frequency storm event to a non-erosive velocity from the outfall to the receiving channel; and
3. Preserve the applicable stream buffer in compliance with the City of Fayetteville streamside protection provisions (UDC Chapter 168.12).

Minimum Standard #2 may be waived for sites that discharge directly into the White River, the West Fork of the White River, Lake Sequoyah, and Lake Fayetteville or as approved by the City Engineer. To support the waiver, a No Downstream Impact Certification Statement must be signed and sealed by a Professional Engineer and shall be submitted to the City Engineer as part of the Grading and Drainage Permit Application. The City Engineer will review the request and either approve or deny the waiver request.

Refer to *Appendix E – Detention Structural Controls* and *Appendix G – Outlet Structures* for technical design guidance for extended detention facilities. The use of nonstructural site design practices that reduce the total amount of runoff will also reduce the required channel protection volume by a proportional amount.

<u>Determining the Channel Protection Volume (CP_v)</u>

- *CP_v Calculation Methods:* The SCS TR-55 method is used to calculate the CP_v storage volume required for a site (see Chapter 3).
- *Rainfall Depths:* The rainfall depth of the 1-year, 24-hour storm in the City of Fayetteville is 3.50 inches in 24 hours. For more details, refer to Table 3.1.a in Chapter 3.

- *Hydrograph Generation:* The SCS TR-55 hydrograph methods provided in Chapter 3 can be used to compute the runoff hydrograph for the 1-year, 24-hour storm.
- *Multiple Drainage Areas*: When a development project contains or is divided into multiple drainage areas, CP_v shall be distributed proportionally to each drainage area.
- *Off-site Drainage Areas:* Off-site drainage areas should be modeled as "existing condition" for the 1-year storm event. A structural stormwater control, if located in-line, will need to safely bypass any off-site flows.
- *Routing/Storage Requirements:* The required storage volume for the CP_v may be provided above the WQ_v storage in stormwater ponds and wetlands with appropriate hydraulic control structures for each storage requirement.
- *Control Orifices*: Orifice diameters for CP_v control of less than 3 inches should be in accordance with City details for small outlets.

2.1.4 Minimum Standard #3 - Overbank Flood Protection (Q_{p25})

Downstream overbank flood protection shall be provided by controlling the post-development peak discharge rate to not exceed the predevelopment rate for the 2, 5, 10, and 25-year, 24-hour return frequency storm event (Qp_{25}).

The use of nonstructural site design practices that reduce the total amount of runoff will also reduce Q_{p25} by a proportional amount.

If a waiver has been approved for the 1-year, 24-hour storm under Minimum Standard #2, then for overbank flood protection, the peak flow of the 1-year (Qp_1), 2-year (Qp_2), 5-year (Qp_5), 10-year (Qp_{10}), and 25-year (Qp_{25}) return frequency storm events <u>must</u> be controlled to not exceed the corresponding predevelopment rate. The 24-hour rainfall depths that correspond to these events are 3.50, 3.92, 4.65, 5.31, and 6.27 inches, respectively.

<u>Determining the Overbank Flood Protection Volume (Q_{p25})</u>

- *Peak-Discharge and Hydrograph Generation:* The SCS TR-55 hydrograph method provided in Chapter 3 shall be used to compute the peak discharge rate and runoff for the 2, 5, 10, and 25-year, 24-hour storm.

2.1.5 Minimum Standard #4 - Extreme Flood Protection (Q_f)

Extreme flood protection shall be provided by controlling and/or safely conveying the 100-year, 24-hour return frequency storm event (Q_f). This is accomplished by:

1. Controlling the post-development peak discharge rate to not exceed the predevelopment rate for the 100-year, 24-hour return frequency storm event (Q_f), of 7.91 inches in 24 hours, through the use of on-site or regional structural stormwater controls, or
2. With permission of the City Engineer, performing a downstream hydrologic assessment as described in Section 7.5 of the Drainage Manual to determine if detention of the 100-year, 24-hour return frequency storm event will cause an increase in peak flow rates when

combined with the flow in the downstream system. If, upon analysis, the downstream hydrologic assessment indicates that detaining the 100-year event will cause the peak flows downstream to increase, and the City Engineer accepts the determination, then the following requirement shall be met:

a. Size the on-site conveyance system to safely convey the Q_f and only discharge into receiving waters where the floodplain is demonstrated to have sufficient capacity to accommodate additional discharges without causing damage, even under fully built-out conditions.

Existing floodplain areas should be preserved to the extent possible. At the discretion of the City Engineer, analysis of floodplain impacts and additional detention or reduction in post-development peak discharge rates may be required for developments.

Determining the Extreme Flood Protection Criteria (Q_f)

- *Peak-Discharge and Hydrograph Generation:* The SCS TR-55 hydrograph method provided in Chapter 3 shall be used to compute the peak discharge rate and runoff for the 100-year, 24-hour 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 no more than 10% of the total drainage area checked (see Chapter 7, Section 7.5). If the post-developed discharges at the downstream checkpoints exceed pre-development conditions, mitigation measures may be required by the City Engineer.
- *System Check*: As a final check, Q_f shall be used in the routing of the 100-year, 24-hour runoff 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 (See Chapter 1 final submittal checklist as applicable). Emergency spillways for structural stormwater controls should be designed to safely pass the resulting flows. (Additional drainage easements may be required.)

SECTION 2.2. REFERENCES

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

Atlanta Regional Commission, 2001. Georgia Stormwater Management Manual, Volume 2: Technical Handbook. Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf.

CHAPTER 3. METHODS FOR ESTIMATING STORM WATER RUNOFF

SECTION 3.1. GENERAL

The accepted approaches for drainage analysis within the City of Fayetteville are listed below. However, the City Engineer may approve other engineering methods for calculation of stormwater runoff when they are shown to be comparable to the required methods. The area limits and/or allowed ranges and applicability for the analysis methods are:

Rational Method	0 to 40 acres. May be used only for inlet, culvert, gutter, storm sewer design. May not be used for detention.
SCS TR-55 / TR-20 (preferred method)	up to 2,000 acres (maximum drainage area per sub-basin) with no cumulative upper limit. May be used for pre-development / post-development comparisons and detention, channel, culvert, gutter, inlet, and storm sewer designs.
HEC-HMS (or other Corps of Engineers or FEMA-authorized methods)	up to 2,000 acres (maximum drainage area per sub-basin) with no cumulative upper limit. Recommended within designated regulatory floodplain, and for design of open channels / waterways.

The design of detention areas shall be based on proposed site design and existing conditions upstream of proposed detention. Inlet design shall be based on fully built-out conditions in accordance with the zoning designation, as shall the conveyance systems downstream of detention features. The required design storm frequencies and durations to be computed shall be based on the applicable Minimum Standards as provided in Chapter 2.

SECTION 3.2. PRECIPITATION DATA

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 "10-year" storm has a 10-percent-annual-chance of occurring in a 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.

3.2.1 Precipitation Data and Rainfall Intensity

The precipitation data chart provided for the City of Fayetteville, Arkansas is based on rainfall atlas data from NOAA publications TP-40 and Hydro-35 (US Department of Commerce, 1961 and 1977). Rainfall intensity is the design rainfall rate in inches per hour for a particular drainage basin or subbasin. The available data for a given event varies slightly based on location, however the variation within the City of Fayetteville is so small that the value for a single event may be assumed to be constant within the corporate limits. Rainfall intensity is selected on the basis of 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 chart provided as Table 3.1. The frequency of occurrence is a statistical variable. The frequencies of occurrence to be used for drainage system design in the City of Fayetteville are established by the minimum standards provided in Chapter 2. 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.

If desired for ease of reference, the data in Table 3.1 may be graphed to create a family of Intensity-Duration-Frequency (I-D-F) curves for the City of Fayetteville locale. Such a graph provides the rainfall intensity on the vertical axis, as a function of the time of concentration for the drainage area under consideration, for each storm frequency. Table 3.1.a includes the coefficients used in the IDF curve equation and the IDF equation below Table 3.1.a can be used to determine the intensity for any time of concentration up to 60 minutes. The majority of drainage sub-basins within the City of Fayetteville have a relatively short time of concentration. In general, basins with computed times of concentration in excess of 90 minutes (maximum) should be subdivided to create smaller sub-basins for more accurate computation of peak discharge.

Table 3.1. – Rainfall intensities for Fayetteville, Arkansas.				
	T_c (min)			
Return Period	**5**	**15**	**30**	**60**
2-Yr	5.44	3.47	2.42	1.63
5-Yr	6.68	4.31	3.03	2.06
10-Yr	7.56	4.90	3.46	2.36
25-Yr	8.85	5.75	4.07	2.79
50-Yr	9.84	6.40	4.54	3.12
100-Yr	10.83	7.07	5.02	3.45

Table 3.1.a. – Intensity-Duration-Frequency Curve Coefficients for Fayetteville, Arkansas.			
	Variable		
Return Period	**B**	**D**	**E**
1-Yr	0.000	0.000	0.000
2-Yr	23.629	4.900	0.641
5-Yr	27.686	4.800	0.623
10-Yr	30.844	4.800	0.616
25-Yr	35.311	4.700	0.609
50-Yr	38.983	4.700	0.606
100-Yr	42.641	4.700	0.603

$$Intensity\ (in/hr) = \frac{B}{(T_c + D)^E}$$

Table 3.1.b provides total rainfall for 24-hour storms for the design frequencies.

Table 3.1.b. Design rainfall for Fayetteville, Arkansas.							
Duration	**1 year**	**2 year**	**5 year**	**10 year**	**25 year**	**50 year**	**100 year**
(hours)	(in)	(in)	(in)	(in)	(in)	(in)	(in)
24	3.50	3.92	4.65	5.31	6.27	7.07	7.91

SECTION 3.3. SCS CURVE NUMBER METHOD

The Soil Conservation Service hydrologic method is based on a synthetic unit hydrograph. The SCS TR-55 approach for runoff determination was developed specifically for use in urbanized and urbanizing areas. Multiple software programs are available that accommodate the SCS hydrologic method and several are listed in Appendix H, Stormwater Software. 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 study point.
- Use the Type III 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. Note that design should be based on the highest peak discharge and not necessarily the longest time of concentration.

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 1, 2, 10, 25, 100 years. The appropriate rainfall distribution for the City of Fayetteville is Type III.

3.3.1 Equations and Concepts

Rainfall-Runoff Equation - The following SCS runoff equation is used to estimate direct runoff depth from a 24-hour storm duration. The equation is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{Eq. 3.1}$$

Where:

Q	=	accumulated direct runoff depth (inches)
P	=	accumulated rainfall (potential maximum runoff) (inches)
I_a	=	initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (inches)
S	=	potential maximum soil retention (inches)

An empirical relationship used in the SCS method to estimate I_a is:

$$I_a = 0.2S \qquad \text{Eq. 3.2}$$

Substituting 0.2S for I_a in Equation 3.1, the equation becomes:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{Eq. 3.3}$$

Where $$S = (1000/CN) - 10 \qquad \text{Eq. 3.3a}$$

Equation 3.3 can be rearranged so that the curve number can be estimated if rainfall and runoff volume are known. The equation then becomes (Pitt, 1994):

$$CN = 1000/[10 + 5P + 10Q - 10(Q^2 + 1.25QP)^{1/2}] \qquad \text{Eq. 3.4}$$

3.3.2 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. The higher the CN, the higher the runoff potential. 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.

There are no identified HSG A soils with the City of Fayetteville, based on the latest available NRCS soil survey data (USDA, SSURGO). Spatial analysis of the hydrologic soil group distribution indicated HSG B soils comprise approximately 25% of the City's soils, while HSG C comprises approximately 50%, and HSG D or combined HSG B and D soils constitute the remaining 25%. In areas indicated as combined HSG B and D soils, the more conservative number shall be assumed for purposes of design, unless the use of different numbers can be justified to the City Engineer. 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 3.2. In all areas disturbed by heavy equipment use 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, except as noted in Table 3.2.

Composite curve numbers shall be calculated 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 3.2 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 5. 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 through the use of Equation 3.4 above where the runoff depth Q has been reduced to reflect the effects of infiltration. The procedure for this approach is provided together with an example in Chapter 5, Section 5.3. In cases where such practices do not reduce overall runoff but delay the timing, reductions in runoff rate must be accounted for by routing. For example, EPA SWMM version 5.0 explicitly accommodates routing methods for LID controls that may be used to compute timing-based reductions in discharge.

If the actual impervious area percentage for the proposed design exceeds the proportion assumed for land uses in Table 3.2, a composite CN shall be computed based on actual percentage rather than using the table values. For purposes of Table 3.2, all compacted earthen fill areas shall be classified as Hydrologic Soil Group D.

Table 3.2. Runoff curve numbers[1].				
Cover Description		**Curve numbers for hydrologic soil groups**		
Cover type and hydrologic condition[2]	**Average percent impervious area[3]**	**B**	**C**	**D**
Cultivated land:				
Without conservation treatment		81	88	91
With conservation treatment		71	78	81
Pasture or range land:				
Poor condition		79	86	89
Good condition		61	74	80
Meadow				
Good condition		58	71	78
Wood or forest land:				
Thin stand, poor cover		66	77	83
Good cover		55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[4]				
Poor condition (grass cover <50%)		79	86	89
Fair condition (grass cover 50% to 75%)		69	79	84
Good condition (grass cover > 75%)		61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98
Streets and roads				
Paved; curbs and storm drains (excluding right-of-way)		98	98	98
Paved; open ditches (including right-of-way)		89	92	93
Gravel (including right-of-way)		85	89	91
Dirt (including right-of-way)		82	87	89
Urban districts:				
Commercial and business	85%	92	94	95
Industrial	72%	88	91	93
Residential districts by average lot size:				
1/8 acre or less (town houses)	65%	85	90	92
1/4 acre	38%	75	83	87
1/3 acre	30%	72	81	86
1/2 acre	25%	70	80	85
1 acre	20%	68	79	84
2 acres	12%	65	77	82
Developing urban areas and newly graded areas (pervious areas only, no vegetation).		86	91	94

1. Antecedent Moisture Condition II, and Ia = 0.2S.
2. Areas of compacted earthen fill shall be classified as Hydrologic Soil Group D.
3. 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.
4. CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

3.3.3 Travel Time Estimation

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection. Travel time (T_t) is the time it takes water to travel from one location to another within a watershed subarea, through the various components of the drainage system. Time of concentration (t_c) is computed as the total time for a particle of water to travel through consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the downstream point of interest within the watershed subarea, typically for the 2-year 24-hour or 50-percent-annual-chance frequency event. Total t_c can also be described as the time required for a particle of water to travel from the most hydraulically distant point of the watershed to the downstream point of interest subarea under sheet (or overland) flow (t_o), shallow concentrated flow (t_{conc}), and open channel flow (t_{oc}) conditions to the downstream point of interest. Therefore,

$$t_c = t_o + t_{conc} + t_{oc} \qquad \textbf{Eq. 3.5}$$

The design engineer shall update the most hydraulically distant point and travel path information as required between pre- and post-development computations.

Minimum allowed t_c = 5 minutes

Travel time is also the ratio of flow length for a particular type of flow to flow velocity:

$$T_t = \frac{L}{3600V} \qquad \textbf{Eq. 3.6}$$

Where:
- T_t = travel time (hr)
- L = flow length (ft)
- V = flow velocity (ft/s)
- 3600 = conversion factor from seconds to hours

3.3.3.1 Overland Flow

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

$$t_o = \frac{0.007\,(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} \qquad \textbf{Eq. 3.7}$$

Where:
- t_o = overland flow travel time (hr)
- n = Manning’s roughness coefficient (see Table 3.3)
- L = flow length (ft), limited to 150 feet maximum (NRCS, 2010)
- P_2 = 2-year, 24-hour rainfall depth (in)
- S = land slope (ft/ft)

Table 3.3. Roughness coefficients (Manning's n) for sheet flow[1].		
Surface Description		**n**
Smooth surfaces		
	Concrete, asphalt, gravel, or bare soil	0.011
Fallow (no residue)		0.05
Cultivated soils		
	Residue cover < 20%	0.06
	Residue cover > 20%	0.17
Grass		
	Short grass prairie	0.15
	Dense grasses[2]	0.24
	Bermuda grass	0.41
Range	(Natural)	0.13
Woods[3]		
	Light underbrush	0.40
	Dense underbrush (Use only where slopes <2% and deep forest litter present)	0.80

The n values are a composite of information by Engman (1986) and are not to be used for other flow conditions.
Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
When selecting n, consider cover to a height of about 0.1 foot. This is the only part of the plant cover that will obstruct sheet flow.
Source: SCS, TR-55, Second Edition, June 1986.

3.3.3.2 Shallow Concentrated Flow

After a maximum of 150 feet (NRCS, 2010), sheet flow usually becomes shallow concentrated flow. The average velocity for this type of flow may be determined using the equations below:

Unpaved $$V = 16.13(S)^{0.5} \quad \text{Eq. 3.8}$$

Paved $$V = 20.33(S)^{0.5} \quad \text{Eq. 3.9}$$

Where: V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

Equation 3.6 may be used to compute travel time for the shallow concentrated flow segment. Allowable maximum shallow concentrated flow velocity for post-developed conditions (for any design storm event) is 6 feet/sec (unpaved) and 10 feet/sec (paved).

3.3.3.3 Open Channels

A chart of Manning's roughness coefficients for open channel flow is provided as Table 3.4.

Travel time for open channel flow may be computed using Equation 3.6. In watersheds with storm sewers, carefully identify the hydraulically most distant location and the appropriate flow path to estimate t_c. Allowable velocities for open channels with engineered or vegetated linings are provided in Section 6.4.

Table 3.4. Manning's roughness coefficient – "n"			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
A. Lined or Built-up Channels			
A1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
A2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	--
8. On irregular excavated rock	0.022	0.027	--
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	--

Table 3.4. Manning's roughness coefficient – "n"			
Type of Channel and Description	Minimum	Normal	Maximum
2. Rough	0.016	0.016	--
j. Vegetal lining	0.030	--	0.050
B. 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 or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stone bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline – excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush or 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
C. Natural Streams			
C1. Minor streams (top width at flood stage <100 feet)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
C2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035

Table 3.4. Manning's roughness coefficient – "n"			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
C3. Major streams (top width at flood stage >100 feet). The "n" value is less than that for minor streams of similar description because banks offer less effective resistance			
a. Regular section with no boulders or brush	0.025	--	0.060
b. Irregular and rough section	0.035	--	0.100

Source: Chow (1959), Open-Channel Hydraulics.

3.3.3.4 In-line Detention Check

A culvert or bridge can often act as an in-line detention structure if there is significant upstream storage due to the constriction of discharges. Water surface profiles through the culvert or bridge structures should be checked and if more than 2 feet of hydraulic head change occurs from the downstream to upstream side of a culvert or bridge for the 10-percent-annual-chance (10-year, 24-hour storm) event, or at the discretion of the City Engineer, detailed storage routing procedures shall be used to determine the outflow through the culvert or bridge. The potential loss of this storage should also be accounted for, and the corresponding change to downstream discharges computed with respect to potential downstream impacts, in the case of structure replacements where a larger opening is proposed.

SECTION 3.4. RATIONAL METHOD

The Rational Method formula is:

$$Q = (C * I * A) \qquad \textbf{Eq. 3.10}$$

where Q is defined as the peak rate of runoff in cubic feet per second (cfs). Actually, Q is in units of acre-inches per hour, but the unit conversion difference is less than 1 percent and is therefore neglected.

C is the dimensionless coefficient of runoff representing the ratio of runoff to rainfall

I is the average intensity of rainfall (inch/hr) for a duration equal to the time of concentration, or t_c

A is the drainage area (acres) that contributes to runoff at the point of design or the point under consideration.

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 rate during the time of concentration to that point.
2. The time of concentration 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. The ratio of runoff to rainfall, C, is uniform during the entire duration of the storm event.
4. The rainfall intensity, I, is uniform for the entire duration of the storm event and is uniformly distributed over the entire watershed area.

Precautions to be considered when using this method:

- The value of C assigned for the drainage area should account for the proposed land use within the project area.

Typical values of C are provided in Table 3.5 for various land use conditions, slopes and hydrologic soil types. In addition to land use, slope and soil type, the value of C applied should be based on consideration of other variables including surface infiltration, proportion of impervious surface, localized topography, rainfall intensity, proximity to groundwater and vegetation characteristics. These values and Equation 3.10 as shown above are appropriate for storm events of 10-percent-annual-chance or greater frequency (design storms up to and including the 10-year).

Table 3.5. Runoff coefficients for various land uses[1, 2, 3 ,4].				
Land Use Description	Slope, %	Hydrologic Soil Group		
		B	C	D
Lawns				
	0-2	0.15	0.25	0.35
	2-7	0.25	0.35	0.4
	> 7	0.3	0.35	0.45
Unimproved areas				
Forest		0.15-.2	0.2-.25	0.2-0.3
Meadow		0.2-0.4	0.25-0.45	0.3-0.55
Row crops		0.25-0.6	0.35-0.75	0.4-0.8
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		0.4	0.47	0.57
2 lots / acre		0.35	0.43	0.54
Suburban (1 lot / acre)		0.3	0.38	0.5
Multi-units, detached		0.7	0.75	0.8
Multi-units, attached		0.75	0.8	0.85
Apartments		0.65	0.7	0.75
Industrial				
Light areas		0.6	0.75	0.85
Heavy areas		0.8	0.85	0.9
Parks, cemeteries		0.25	0.35	0.45
Schools, Churches		0.7	0.75	0.8
Railroad yard areas		0.2	0.35	0.5
Asphalt & Concrete Pavements, Roofs.			0.95	
Brick Pavement or Gravel (compacted subgrade)			0.85	
Graded or no plant cover				
	0-2	0.25	0.3	0.35
	2-7	0.35	0.45	0.55
	> 7	0.5	0.65	0.8

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

For storm events of less than 10-percent-annual-chance, or storm frequencies exceeding the 10-year event, a multiplier referred to as C_f shall be used, unless site-specific supporting calculations are provided to address the other variables described above. Table 3.6 provides the multiplier C_f to be used for storm events of various frequency. In these cases, Equation 3.10 is modified to become

$$Q = C_f * (C * I * A) \qquad \text{Eq. 3.11}$$

Table 3.6. Frequency factors for the Rational Formula.	
Return period years (% annual chance)	**Multiplier, C_f**
2 – 10 (50% - 10%)	1.0
25 (4%)	1.1
50 (2%)	1.2
100 (1%)	1.25

C_f * C shall not exceed 1.0.

Selection of design intensity in the Rational Method shall be based on the required design frequency and depends on the time of concentration being consistent with the duration of the rainfall event. The rate of runoff is equal to the rainfall excess when the rainfall event lasts long enough for the entire watershed to contribute to drainage. Where drainage areas have relatively high proportions of impervious area, the drainage area should be subdivided to reflect this so that peak discharge computations appropriately reflect the highest peak discharge and lowest time of concentration for the subdivided portions.

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.

SECTION 3.5. HEC-HMS METHODS

HEC-HMS (USACE, 2000) 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 open channel design. For runoff computations, the model provides several options for the following components:

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

These models are similar to what was provided in HEC-1, the predecessor to HEC-HMS. Other additional features are also available in HEC-HMS.

If HEC-HMS is used to compute runoff, the preferred method for estimation of runoff volume is the SCS Curve Number method. However, other methods may be accepted at the discretion of the City Engineer. Also, the selection of the routing model 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.

SECTION 3.6. STORMWATER 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 Fayetteville and this Drainage Criteria Manual. A list of software that may be used is provided in Appendix H, Stormwater Management Software. Within Special Flood Hazard Areas, FEMA-approved hydrologic models should be used. Regardless of software type, output data provided shall be clearly and concisely labeled based on percent-annual-chance event or design storm, and organized in a consistent fashion. A spatial file or schematic shall be provided for reference that identifies sub-basins for which data are computed. Minimum output data required shall correspond to the Drainage Report requirements detailed in Chapter 6. Example tables depicting required input/output data to be reported are provided within Chapter 6.

SECTION 3.7. REFERENCES

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CHAPTER 4 WATER QUALITY

SECTION 4.1. THE NATURE OF POLLUTANTS IN STORMWATER RUNOFF

Nonpoint source pollution, which is a primary cause of polluted stormwater runoff and water quality impairment, can come from many sources—many of which are the result of human activities within a watershed. 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 Fayetteville's water resources.

In urbanizing watersheds, the potential for water quality degradation occurs as a result 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 a number of 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.

4.1.1 Stormwater Pollution Sources

For effective stormwater management, it is important to understand the nature and sources of urban stormwater pollution. Table 4.1 summarizes the major stormwater pollutants and their effects.

Table 4.1. 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

4.1.2 Areas with High Pollutant 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

SECTION 4.2. WATER QUALITY MANAGEMENT CRITERIA

4.2.1 Overview of Water Quality Criteria

This section presents an integrated approach for meeting the stormwater runoff management requirements of Minimum Standard #1 (see Section 1.2).

There are two complementary approaches to meet the water quality objectives of Minimum Standard #1: (1) Runoff Reduction Methodology (RRM) using Green Infrastructure and other Low Impact Development (LID) practices to capture and hold on-site runoff from a 1-inch rainfall; and, (2) the Total Suspended Solids (TSS) Reduction Method (TRM), a more traditional approach focusing on removal of 80% TSS from a selected rainfall runoff – 1.2 inches in the case of Fayetteville. The RRM is explained in Chapter 5. TRM is covered in this chapter.

TSS Reduction Method (TRM)

The TSS Reduction Method follows the philosophy of removing pollutants and at least 80% of the TSS "where practicable" through the use of 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 structural control facilities that work to remove pollutants from the runoff. The WQ_v is roughly equal to the runoff from the first 1.2 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 1.2 inches or less, as well as the first 1.2 inches of runoff for larger storm events.

The volumetric runoff coefficient (R_v) was derived from a regression analysis performed on rainfall runoff volume data from a number of 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 (WQ_v) is equal to a rainfall depth of 1.2 inches multiplied by the volumetric runoff coefficient (R_v) and the site area, and is calculated using Equation 4.1 below:

$$WQ_v = \frac{PR_vA}{12} \qquad \textbf{Equation 4.1}$$

Where:

WQ_v = water quality volume (ac-ft)

R_v = 0.05 + 0.009(I) where I is percent impervious cover (i.e., 50% impervious is 50 not 0.5)

A = site area (acres)

P = 1.2 inches

Determining the Water Quality Volume (WQ_v)

- *Measuring Impervious Area*: The area of impervious cover shall be based on the proposed project plans and independent of pre-construction conditions.
- *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.
- *Credits for Site Design Practices*: The use of certain site design practices may allow the WQ_v volume to be reduced through the subtraction of a site design "credit." These site design credits are described in Section 4.3.2.
- *Determining the Peak Discharge for the Water Quality Storm:* When designing off-line structural control facilities, 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 1.2 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 4.1.

SECTION 4.3. MEETING THE WATER QUALITY SIZING CRITERIA REQUIREMENTS WITH TOTAL SUSPENDED SOLIDS REDUCTION METHOD (TRM)

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; and
- The use of structural stormwater controls to provide treatment and control of stormwater runoff

4.3.1 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.

4.3.2 Site Design Stormwater Credits

A set of stormwater "credits" has been developed to provide developers and site designers an incentive to use site design practices that can reduce the volume of stormwater runoff and minimize the pollutant loads from a site. The credit system directly translates into cost savings to the developer by reducing the size of structural stormwater control and conveyance facilities.

The credit system recognizes the water quality benefits of certain site design practices by allowing for a reduction in the water quality volume (WQ_v). If a developer incorporates one or more of the credited practices in the design of the site, the volume of stormwater required to be captured and treated to meet Minimum Standard #1 will be reduced. The following credits are only for use with the TRM method. Runoff Reduction credits are discussed in Low Impact Development-Chapter 5.

The site design practices that provide stormwater credits are listed in Table 4.2. Site-specific conditions will determine the applicability of each credit. For example, stream buffer credits cannot be taken on upland sites that do not contain perennial or intermittent streams.

It should be noted that site design practices and techniques that reduce the overall impervious area on a site implicitly reduce the total amount of stormwater runoff generated by a site (and thus reduce WQ_v) and are not further credited in the TRM method.

Table 4.2. Summary of site design practices that receive site design stormwater credits.	
Practice	**Description**
Natural Area Conservation	Undisturbed natural areas are conserved on a site, thereby retaining their pre-development hydrologic and water quality characteristics.
Stream buffers	Stormwater runoff is treated by directing sheet flow runoff through a naturally vegetated or forested buffer as overland flow.
Use of vegetated channels	Vegetated channels are used to provide stormwater treatment.
Overland flow filtration/infiltration zones	Overland flow filtration/infiltration zones are incorporated into the site design to receive runoff from rooftops and other small impervious areas.
Environmentally sensitive large lot subdivisions	A group of site design techniques are applied to low and very low density residential development.

For each potential credit, there is a minimum set of criteria and requirements which identify the conditions or circumstances under which the credit may be applied.

Site designers are encouraged to utilize as many credits as they can on a site. Greater reductions in stormwater storage volumes can be achieved when many credits are combined. However, credits cannot be combined by applying any portion of multiple practices to the same area.

For practices that do not apply to the same area - for example, Natural Area Conservation and Overland flow filtration/infiltration zones - the site design stormwater credits may be applied cumulatively, by adding the reduction in required water quality volume computed for each practice individually and subtracting the sum of the reductions from the original total that would be required if no practices were applied.

4.3.2.1 Site Design Credit #1: Natural Area Conservation

A stormwater credit can be taken when undisturbed natural areas are conserved on a site. Under this credit, a designer would be able to subtract conservation areas from total site area when computing water quality volume requirements. Additionally, the post-development peak discharges will be smaller, and hence water quantity control volumes (CP_v, Q_{p25}, and Q_f) will be reduced due to lower post-development curve numbers or rational formula "C" values.

Rule: Water quality volume requirements may be reduced by the proportion of the Natural Conservation Area to the total site area.

Criteria:

- Conservation area cannot be disturbed during project construction;
- Area shall be protected by limits of disturbance clearly shown on all construction drawings;
- Area shall be located within an acceptable conservation easement instrument that ensures perpetual protection of the proposed area. The easement must clearly specify how the natural area vegetation shall be managed and boundaries will be marked [Note: managed turf (e.g., playgrounds, regularly maintained open areas) is not an acceptable form of vegetation management];

- Area shall have a minimum contiguous area requirement of 10,000 square feet; and
- R_v is kept constant when calculating WQ_v.

Example:

Residential Subdivision; Total Area = 38 acres
Natural Conservation Area = 7 acres
Impervious Area = 13.8 acres

R_v = 0.05 + 0.009 (I) = 0.05 + 0.009 (36.3) = 0.377

Credit:
7.0 acres in natural conservation area
New drainage area = 38 – 7 = 31 acres

Before credit:
WQ_v = (1.2)(0.377)(38)/12 = 1.43 ac-ft
With credit:
WQ_v = 1.43 * 31/38 = 1.17 ac-ft

(18% reduction in required water quality volume)

4.3.2.2 Site Design Credit #2: Stream Buffers

This credit can be taken when stormwater runoff is effectively treated by directing overland flow through a naturally vegetated or forested stream buffer. The areas draining via overland flow to the buffer may be subtracted from the total site area when computing water quality volume requirements. In addition, the volume of runoff draining to the buffer can be subtracted from the channel protection volume. The design of the stream buffer treatment system must use appropriate methods for conveying flows above the annual recurrence (1-yr storm) event.

Rule: Water quality volume requirements may be reduced by the proportion of the area draining via overland flow to the buffer to the total site area.

<u>Criteria:</u>

- The minimum undisturbed buffer width shall be 50 feet.
- The maximum contributing length shall be 150 feet for pervious surfaces and 75 feet for impervious surfaces. (Areas with lengths exceeding this criteria shall not be counted.)
- The average contributing slope shall be 3% maximum unless a flow spreader is used.
- Runoff shall enter the buffer as overland sheet flow. A flow spreader can be supplied to ensure this, or if average contributing slope criteria cannot be met.
- Not applicable if overland flow filtration/groundwater recharge credit is already being taken.
- Buffers shall remain unmanaged other than routine debris removal.
- R_v is kept constant when calculating WQ_v.

Example:

Residential Subdivision; Total Area = 38 acres
Impervious Area = 13.8 acres
Area Draining to Buffer = 5 acres

R_v = 0.05 + 0.009 (I) = 0.05 + 0.009 (36.3) = 0.377

Credit:

5.0 acres draining to buffer
New drainage area = 38 – 5 = 33 acres

Before credit:
WQ_v = (1.2)(0.377)(38)/12 = 1.43 ac-ft
With credit:
WQ_v = 1.43 ac-ft * 33/38 = 1.24 ac-ft
(13% reduction in water quality volume)

4.3.2.3 Site Design Credit #3: Vegetated Channels

This credit may be taken when vegetated (grass) channels constructed in accordance with Appendix B of this manual are used for water quality treatment. A designer is able to subtract the areas draining to a grass channel from total site area when computing water quality volume requirements. A vegetated channel can fully meet the water quality volume requirements for certain kinds of low-density residential development. An added benefit will be that the post-development peak discharges will likely be lower due to a longer time of concentration for the area draining to the grass channel.

This credit cannot be taken if grass channels are used as a structural stormwater control towards meeting the 80% TSS removal goal for WQ_v treatment. That is, the vegetated channels credit may reduce the area to be treated or be used toward the 80% TSS removal goal for stormwater treatment, but not both.

Rule: Water quality volume requirements may be reduced by the proportion of the area draining to a grass channel to the total site area.

Criteria:

- The credit shall only be applied to moderate or low density residential land uses (three dwelling units per acre maximum). Maximum 5 acres may drain to a single channel.
- The maximum flow velocity for water quality design storm shall be less than or equal to 1.0 feet/s.
- The minimum channel residence time for the water quality storm shall be 5 minutes, with a minimum 300 feet channel length.
- The bottom width shall be a maximum of 6 feet. If a larger channel is needed use of a compound cross section (with low-flow channel and bench on at least one side for higher flows) is required.
- The side slopes shall be 3:1 (horizontal:vertical) or flatter.
- The channel slope shall be 3% or less.
- R_v is kept constant when calculating $WQ_{v.}$

Example:

Residential Subdivision; Total Area = 38 acres
Impervious Area = 13.8 acres

$R_v = 0.05 + 0.009\ (I) = 0.05 + 0.009\ (36.3) = 0.377$

Credit:

12.5 acres meet grass channel criteria
New drainage area = 38 – 12.5 = 25.5 acres

Before credit:
$\mathbf{WQ_v = (1.2)(0.377)(38)/12 = 1.43}$ **ac-ft**
With credit:
$\mathbf{WQ_v = 1.43}$ **ac-ft** $\mathbf{* 25.5/38 = 0.96}$ **ac-ft**
(33% reduction in water quality volume)

4.3.2.4 Site Design Credit #4: Overland Flow Filtration/Groundwater Recharge Zones

This credit can be taken when "overland flow filtration/infiltration zones" are incorporated into site design to receive runoff from rooftops or other small impervious areas (e.g., driveways, small parking lots, etc.). This can be achieved by grading the site to promote overland vegetative filtering or by providing infiltration or "rain garden" areas. If impervious areas are adequately disconnected, they can be deducted from total site area when computing the water quality volume requirements. An added benefit will be a longer time of concentration for the area draining into the feature.

Rule: Water quality volume requirements may be reduced by the proportion of the adequately disconnected impervious area draining to an infiltration area, to the total site area.

Criteria:

- Relatively permeable soils (generally, hydrologic soil group B) should be present.
- Runoff shall not come from an area with high pollutant discharge potential.
- The maximum contributing impervious flow path length shall be 75 feet.
- Downspouts shall be at least 10 feet away from the nearest impervious surface with drainage diverted away to discourage "re-connections".
- The disconnection shall drain continuously through a vegetated channel, swale, or filter strip to the property line or structural stormwater control.
- The length of the "disconnection" shall be equal to or greater than the contributing length.
- The entire vegetative "disconnection" shall be on a slope less than or equal to 3%.
- The surface impervious area directed to any one discharge location shall not exceed 5,000 square feet.
- For those areas draining directly to a buffer, either the overland flow filtration credit *-or-* the stream buffer credit can be used.

- R_v is kept constant when calculating WQ_v.

Example:

Site Area = 3.0 acres
Impervious Area = 1.9 acres (or 63.3% impervious cover)
"Disconnected" Impervious Area = 0.5 acres

$R_v = 0.05 + 0.009\ (I) = 0.05 + 0.009\ (63.3) = 0.62$

Credit:

0.5 acres of surface imperviousness hydrologically disconnected
New drainage area = 3 – 0.5 = 2.5 acres

Before credit:
$WQ_v = (1.2)(0.62)(3)/12 = 0.19$ ac-ft
With credit:
$WQ_v = 0.19$ ac-ft $*2.5/3 = 0.16$ ac-ft
(17% reduction in water quality volume)

4.3.3 Structural Stormwater Control Practices

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 a number of 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 two categories: *general application and limited application* controls. These controls are targeted at 80% TSS pollution reduction and only incidentally reduce the runoff volume. Green Stormwater Practices (GSPs) are discussed in detail at the end of Chapter 5 and are not presented in this section. GSPs are targeted at runoff volume reduction and assume that all pollutants contained within the reduced volume are 100% removed. Detention structural controls are discussed in Chapter 7.

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 (WQ_v) 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 (CP_v), overbank flood protection (Q_{p25}) and/or extreme flood protection (Q_f). 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 F, Water Quality Structural Controls. Detailed descriptions of the low-impact structural controls along with design criteria and procedures are provided in Appendix B, GSP Specifications.

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, and
- 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, and
- 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, and
- Perimeter Sand Filter.

Low Impact Structural Controls

Bioretention Areas

Bioretention areas are shallow stormwater basins or landscaped areas that utilize engineered 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, and
- Wet Swale/Wetland Channel.

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, and/or (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 F, Water Quality Structural Controls.

Filtering Practices

- Organic Filter, and
- Underground Sand Filter.

Wetland Systems

- Submerged Gravel Wetland.

Hydrodynamic Devices

- Gravity (Oil-Grit) Separator.

Proprietary Systems

- Commercial Stormwater Controls.

4.3.3.1 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 for different pollutant types.

Pollutant removal capabilities for a given structural stormwater control practice are based on a number of 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 4.3 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, the refer to the National Pollutant Removal Performance Database (2nd Edition) available at www.cwp.org and the National Stormwater Best Management Practices (BMP) Database at www.bmpdatabase.org

Table 4.3. Design pollutant removal efficiencies for structural stormwater controls.

Structural Control	Total Suspended Solids	Total Phosphorus	Total Nitrogen	Fecal Coliform	Metals
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.

SECTION 4.4. USING STRUCTURAL STORMWATER CONTROLS IN SERIES

4.4.1 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 4.1.

Figure 4.1. 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 and site design credits to reduce the water quality volume (WQ_v),
- Limited application structural controls that provide pretreatment, and
- 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, and
- Detention structural controls.

4.4.2 Use of Multiple Structural Controls in Series

Many combinations of structural controls in series may exist for a site. Figure 4.2 provides a number of hypothetical examples of how the stormwater sizing criteria may be addressed by using structural stormwater controls.

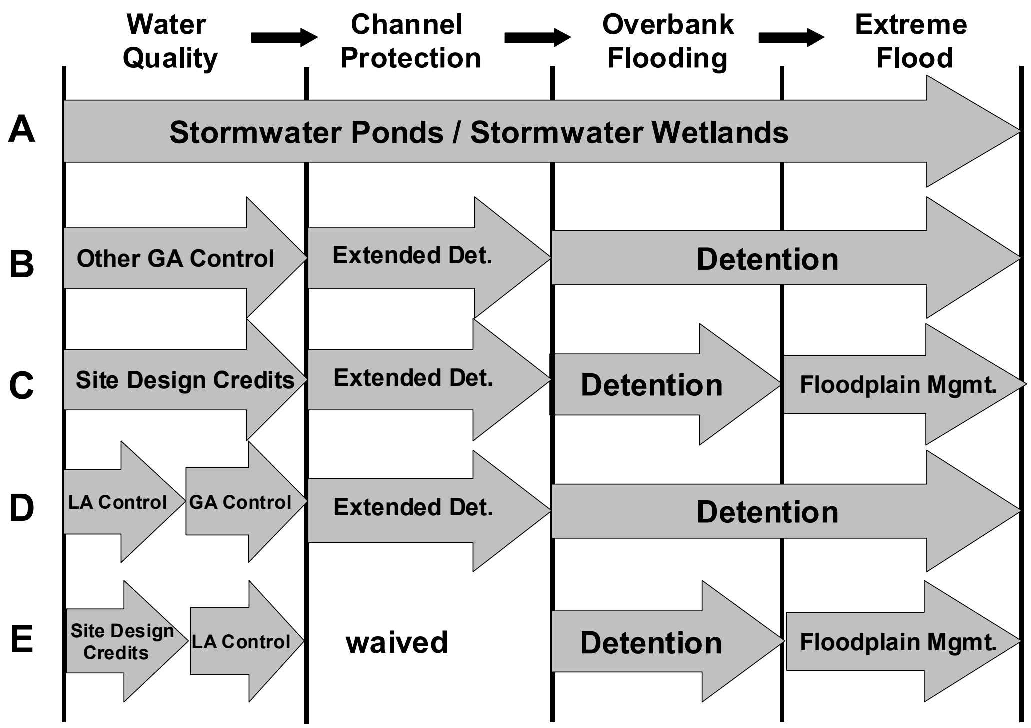

Figure 4.2. Examples of structural controls used in series.

Referring to Figure 4.2 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 so as 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.

E. Site design credits have been employed to partially reduce the water quality volume requirement. In this case, for a smaller site, a well designed and tested Limited Application structural control provides adequate TSS removal while a dry detention pond handles the overbank flooding criteria. For this location, direct discharge to a large stream and local downstream floodplain management practices have eliminated the need for channel protection volume and extreme flood protection structural controls on site.

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 4.3 illustrates the application of the treatment train concept for a large shopping mall site.

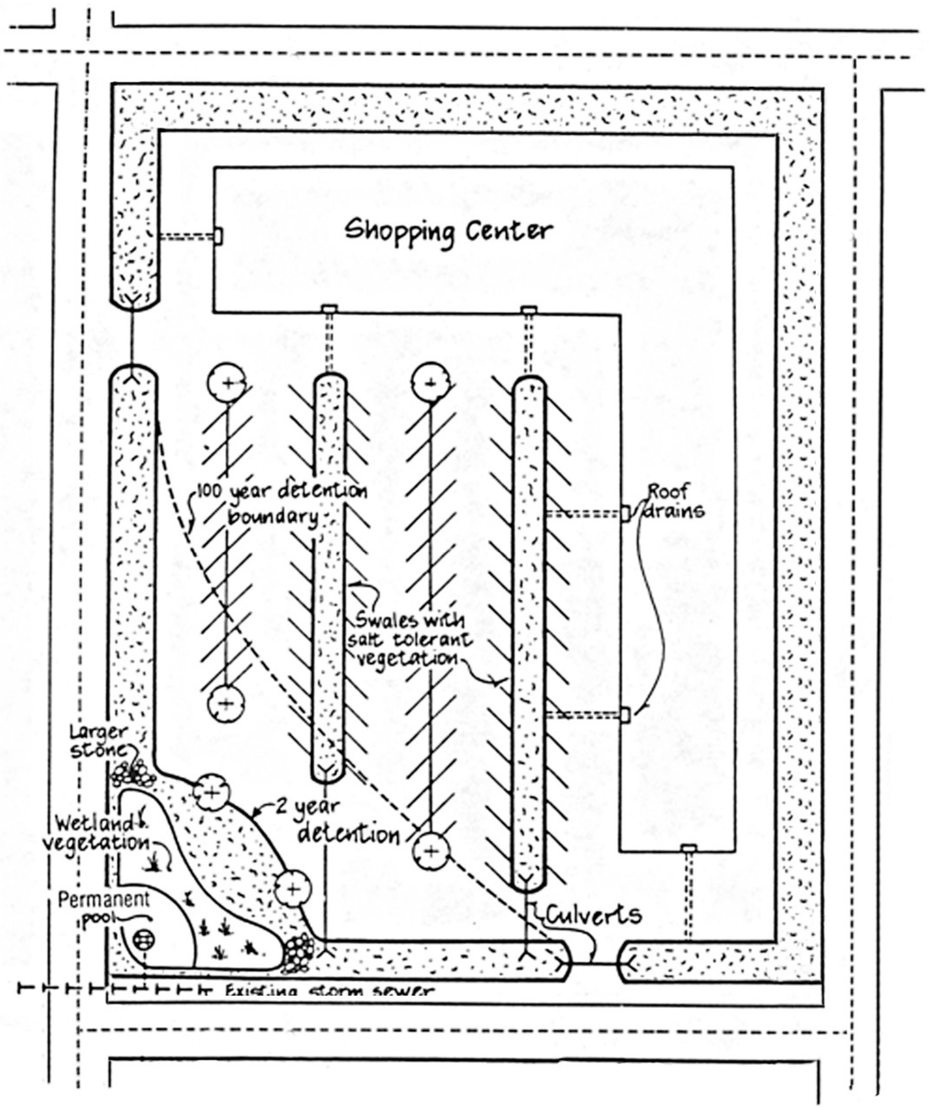

Figure 4.3. Example treatment train – commercial development (Source: NIPC, 2000).

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. Overbank and extreme flood protection are provided through parking lot detention.

4.4.3 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.

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 4.3 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

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**

4.4.4 Routing with WQ_v Removed

When off-line structural controls such as bioretention areas, sand filters and infiltration trenches capture and remove some portion of the water quality volume (WQ_v), downstream structural controls do not have to account for the removed volume during design. That is, the volume removal may be subtracted from the total volume that would otherwise need to be routed through the downstream structural controls.

From a calculation standpoint this would amount to removing the initial WQ_v (or removal portion) from the beginning of the runoff hydrograph – thus creating a "notch" in the runoff hydrograph. Since most commercially available hydrologic modeling package do not accommodate this, the following method has been created to facilitate removal from the runoff hydrograph of approximately the WQ_v:

- Enter the horizontal axis on Figure 4.4 with the impervious percentage of the watershed and read upward to the predominant Hydrologic Soil Group (HSG).
- Read left to the factor.
- Multiply the curve number for the sub-watershed that includes the water quality basin by this factor – this provides a smaller curve number.

The difference in curve number will generate a runoff hydrograph that has a volume less than the original volume by an amount approximately equal to the WQ_v. This method should be used only for bioretention areas, filter facilities and infiltration trenches where the drawdown time is ≥ 24 hours.

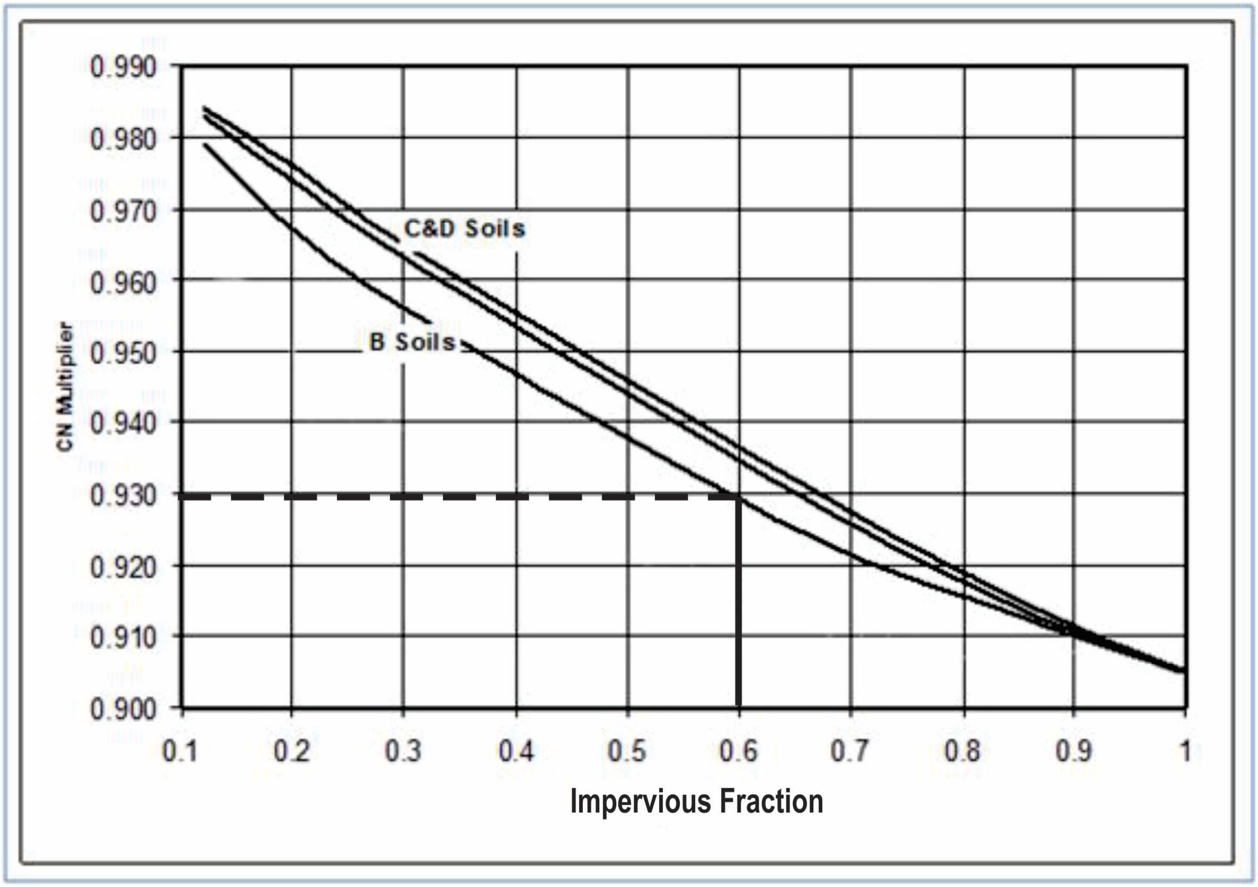

Figure 4.4. Curve number adjustment factor.

<u>Example</u>

A site design employs an infiltration trench for the WQ_v and has a curve number of 72, is B-type soil, and has an impervious percentage of 60%, the factor from Figure 4.4 is 0.93. The curve number to be used in calculation of a runoff hydrograph for the quantity controls would be: (72*0.93) = 66.

SECTION 4.5. STORMWATER QUALITY BMP LONG TERM 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; and

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.

SECTION 4.6. REFERENCES

Atlanta Regional Commission, 2001. Georgia Stormwater Management Manual, Volume 2: Technical Handbook. Atlanta, GA. http://www.georgiastormwater.com/GSMMVol2.pdf

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CHAPTER 5. LOW IMPACT DEVELOPMENT

SECTION 5.1. INTRODUCTION

5.1.1 Background and Purpose

This chapter provides guidance on (1) implementing the principles of LID, (2) implementing the design, installation, and maintenance of LID-based Green Stormwater Practices (GSPs) for new development and re-development projects, and (3) applying the principles of the Runoff Reduction Method (RRM) to calculate runoff volume reduction and make runoff computation adjustments. This chapter should be used as a resource for incorporating LID elements in any project, whether or not the volume reduction goals can be met.

The use of LID for new development and redevelopment projects within the City of Fayetteville is encouraged. The City also encourages the use of LID stormwater management strategies in street design and construction. The Master Street Plan provides information for incorporating LID into the City's typical street sections. Any LID elements incorporated into the street typical section and drainage design should be designed in accordance with the information provided in this chapter and the Master Street Plan.

Runoff Reduction Method (RRM)

The RRM is based on the approach that runoff volume reduction equals pollutant reduction with respect to stormwater runoff. This method assigns a rating in terms of percent rainfall capture to every post-development land surface. In Fayetteville the percent rainfall capture goal is 80% volume capture. To meet this capture goal the designer must apply site layout techniques and use a combination of infiltration, evapotranspiration, harvest and/or rainfall reuse practices, to capture and treat 80% of the rainfall volume based on a 1-inch storm of moderate intensity. The steps outlined below highlight the process of Runoff Reduction design.

- **Step 1**: Reduce runoff through land use, site layout and ground cover decisions with Intrinsic Green Stormwater Practices (GSPs).
- **Step 2**: Direct runoff from impervious areas to pervious areas.
- **Step 3**: Apply on-site structural GSPs to capture the remaining volume.

It should be noted that the RRM is not required within the City of Fayetteville, however the use of this method and the design techniques and best management practices described in this chapter are encouraged. The RRM can be used to replace the TRM approach described in Chapter 4, or the two approaches may be used in combination.

The runoff reduction goal for LID presented in this chapter is accomplished through volume removal. For developments implementing a LID site design, the project design team should use the guidance contained within this chapter to provide as much volume reduction as is feasible, with the goal of reaching 80% runoff reduction. If the 80% goal is not feasible for the project, the project will still be considered provided that all the other design requirements of the Drainage Criteria Manual, all applicable ordinances, and the Master Street Plan are met. Any LID features that are incorporated into the site design should be designed in accordance with this chapter, and any volume reduction achieved by using LID should be accounted for in the hydrologic and hydraulic calculations required by the Drainage Criteria Manual. The project will benefit from any achieved reduction in runoff volume by reducing the post-development runoff; the corresponding credit is the reduced detention requirements for the site.

5.1.2 Overview: LID and Stormwater Management

LID works to control stormwater runoff volume by attempting to mimic a site's natural hydrology through the use of design techniques that promote infiltration, filtration, storage, and evapotranspiration. LID uses GSPs that slow runoff, spread the flow of stormwater, and allow it to soak into the ground, thereby reducing the volume of runoff from the post-developed site. GSPs can be used in conjunction with traditional techniques to develop a comprehensive stormwater management plan.

To implement LID, the designer should select a combination of GSPs that take into consideration existing hydrology on the site, complement traditional design techniques and provide volume reduction to help meet stormwater management goals.

Traditional stormwater management does not typically account for the increase in the total volume of runoff that occurs from increases in impervious area. It also doesn't typically address pollutants that can occur from post-development land use and the associated increased impervious area. Applying the LID techniques or GSPs presented in this chapter to a site design can reduce the volume of stormwater discharged from a site as well as the quantity of pollutants discharged from impervious surfaces.

The calculation method used for LID projects in Fayetteville is called the Runoff Reduction Method (RRM). The RRM quantifies the volume of runoff associated with particular land surfaces and GSPs by assigning a rating for rainfall capture for each land surface and a corresponding Runoff Reduction Credit (RR Credit) for each GSP. By quantifying the runoff volume for each land cover and design technique, the designer can compare the amount of runoff captured using various combinations of surface cover and design techniques and select the site design approach that achieves the desired volume reduction.

Table 5.1. presents a comparison of the stormwater management goals for traditional stormwater design and LID design. This table emphasizes that LID works to complement, not replace, traditional flood management techniques, as both are equally important to a comprehensive stormwater management plan.

Table 5.1. Stormwater management goals: Traditional design vs. LID.

Traditional Design	Low Impact Development GSPs
Focus on large infrequent storms	Focus on small frequent storms
Prevent flooding by mitigating peak flow rates	Reduce total stormwater runoff volume
	Promote infiltration and groundwater recharge
	Improve and protect water quality

5.1.3 Chapter Components

Section 5.2 provides guidance for the site planning and design of LID projects. It examines the specific considerations and processes used to select intrinsic, structural, and non-structural GSPs as part of the site design process. The planning and site design section provides guidance for using natural properties of a site to manage stormwater using and preserving the pre-development characteristics of the site. General considerations for selection of the appropriate structural and non-structural GSPs are also detailed.

Section 5.3 presents the Runoff Reduction Methodology. The Runoff Reduction Method (RRM) serves as the basis for Fayetteville's approach to LID using volume removal. Through this method every land surface can now have an assigned rating in terms of percent rainfall capture. Even impervious surfaces capture a small amount of water and therefore do not generate 100% runoff. Thus, understanding and calculating key aspects of a site's land condition in relation to volume removal is important to this process demonstrated with step-by-step equations and methodology.

Section 5.4 presents intrinsic and structural/non-structural GSPs. Each GSP has a design specification included in either Appendix A or Appendix B. The design specifications include detailed design guidance for each GSP allowing the designer to plan and appropriately select the GSP or combination of GSPs that will help achieve the volume reduction goals presented in Section 3. Additional specifications are presented regarding the performance of infiltration tests that are required for certain media-based GSPs, and soil mix designs for use in various GSPs.

5.1.4 How Does this Chapter Relate to the LID Ordinance, the Fayetteville Drainage Criteria Manual, and other Ordinances?

In 2009, the City passed a Low Impact Development Ordinance (Chapter 179: Low Impact Development of Title XV Unified Development Code). The design concepts presented in Chapter 5 are in support of the LID Ordinance. Implementing LID and using GSPs in accordance with this chapter, as specified in the LID Ordinance, is voluntary. A site development project will be required to meet all stormwater management requirements outlined and described throughout the Drainage Criteria Manual whether or not LID is implemented. Implementing LID in accordance with guidelines in this chapter may, however, assist in meeting the requirements of the DCM by reducing the post-development runoff volume and flow. The following ordinances and documents should also be considered through the process of implementing LID in Fayetteville.

- Chapter 161 – Zoning Regulations
- Chapter 166 – Development
- Chapter 167 – Tree Preservation & Protection
- Chapter 168 – Flood Damage Prevention Code
- Chapter 169 – Physical Alteration of Land
- Chapter 170 – Stormwater Management, Drainage, & Erosion
- Chapter 171 – Streets and Sidewalks
- Chapter 172 – Parking & Loading
- Chapter 173 – Building Regulations
- Chapter 177 – Landscape Regulations
- Chapter 179 – Low Impact Development

- City Plan 2030
- City of Fayetteville Landscape Manual
- Streamside Protection BMP Manual

The LID design techniques presented in this chapter or other similar techniques may be required in order to meet future MS4 NPDES stormwater permit requirements for post construction stormwater quality, increasing the importance that designers and owners understand LID design concepts and procedures presented herein.

5.1.5 How to Use this Chapter?

Engineers, Planners, Developers, and City staff should use this chapter for direction on the process for designing, implementing, constructing, and maintaining LID projects. To obtain credit in the form of reduced detention, LID features shall be designed and constructed in accordance with this guidance.

The process for implementing LID on a development project is shown in Figure 5.1.

Step 1 - Site Selection, Site Layout/Planning

- Conduct a comprehensive site analysis and inventory. Include consideration of any additional federal or state requirements, such as wetlands permitting requirements.
- Refer to **Section 5.2: Planning and Site Design.**

Step 2 - Apply Intrinsic Green Stormwater Practices

- Reduce runoff by making the best use of existing natural features with proper site planning and design processes.
- Refer to **Section 5.2.2 : Applying Intrinsic Green Stormwater Practices.**

Step 3 - Select Structural & Nonstructural Green Stormwater Practices

- Select preferred GSPs for use on development or redevelopment site.
- Refer to **Section 5.3 : The Runoff Reduction Method** and **Section 5.4: Green Stormwater Practices.**

Step 4 - Apply Runoff Reduction Method Principles

- Evaluate the site to determine the runoff reduction level achieved through GSP selection.
- Refer to **Section 5.3: The Runoff Reduction Method.**

Step 5 – Achieve 80% Runoff Reduction

- If necessary, apply additional or different GSPs to the development or redevelopment site to reach the 80% runoff reduction goal.
- Refer to **Section 5.3: The Runoff Reduction Method** and **Section 5.4: Green Stormwater Practices.**

Figure 5.1. LID implementation process.

SECTION 5.2. PLANNING AND SITE DESIGN

5.2.1 Introduction and Design Principles

The correct pairing of land uses and Green Stormwater Practices (GSPs) is an important first step in site planning. Land use and project type should be the basis for selection of GSPs detailed on the criteria specification sheets in Appendix A and Appendix B. The Runoff Reduction Method detailed in Section 5.3 is a three step process used to select intrinsic, non-structural, and structural GSPs, and to account for stormwater volume capture on a site.

Site Design Objectives

- **Achieve Multiple Objectives**
- **Conserve Natural Features and Resources**
- **Minimize Soil Compaction**
- **Manage Stormwater Close to the Source**
- **Reduce and Disconnect Impervious Surfaces**

This section describes the specific considerations and processes used to select intrinsic, non-structural, and structural GSPs as a part of site design. This section also provides guidance for using the natural properties and existing conditions of a site to optimize the management of stormwater using LID, and provides general information to aid selection of the most appropriate structural and non-structural GSPs.

There are several important design goals and principles involved in incorporating GSPs. Some of these basic concepts are listed below.

Achieve Multiple Objectives

Stormwater management should be comprehensive and designed to achieve multiple stormwater objectives such as: managing peak flow and total volume; improving water quality control; maintaining or improving the pre-development hydrologic regime; and maintaining water temperature. In some cases, this requires multiple structural techniques; however, one objective of GSPs is to allow for less complex management systems to achieve multiple objectives through implementation of site planning as a precursor to the design process.

Conserve Natural Features And Resources

The conservation of natural features such as floodplains, higher permeability soils, and vegetation helps to retain predevelopment 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 adverse to predevelopment 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 currently required by the City of Fayetteville Streamside Ordinance;
- Preserve sensitive environmental areas; historically undisturbed vegetation; and native trees; also currently required in the City of Fayetteville;
- Conform to watershed; conservation; and open space plans;
- Design development to fit the site terrain and build roadways along site contours wherever possible;
- Use cluster development to preserve higher permeability soils, natural streams, and natural slopes; and
- 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 pre-existing 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, and
- Avoid working or driving on wet soil.

Manage Stormwater Close to the Source

Redirecting runoff back into the ground, close to the point of origin, provides both environmental and economic benefits. Techniques to manage stormwater runoff close to the source include:

- The use of GSPs to infiltrate stormwater into the ground instead of concentrating and collecting of flow and routing it offsite, and
- Disconnection of impervious surfaces wherever feasible.

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 amount 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 roofs;
- Direct roof downspouts to vegetated areas, bioretention, cisterns, or planter boxes, and route runoff into vegetated swales instead of 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; and
- Minimize impervious footprints.

5.2.2 Applying Intrinsic Green Stormwater Practices

Intrinsic Green Stormwater Practices

- **Minimize Soil Compaction**
- **Minimize Total Disturbed Area**
- **Protect Natural Flow Pathways**
- **Protect Riparian Buffer Areas**
- **Protect Sensitive Areas**
- **Reduce Impervious Surfaces**

The first tool in addressing stormwater management in new development and redevelopment involves making the best use of the existing natural features with proper site planning and design processes by applying Intrinsic GSPs. The Runoff Reduction Method, as described in Section 5.3, provides a quantitative and qualitative approach to allow capture credit for the majority of natural land cover types and areas preserved using intrinsic GSPs. Intrinsic site design practices or intrinsic GSPs include evaluation not only of a site, but also where the stormwater is falling on a site and how to manage that rainfall before routing to a structural GSP. The goal is to minimize impervious cover and mass site grading and to maximize the retention of forest and vegetative cover, natural areas and undisturbed soils, especially those most conducive to landscape-scale infiltration. Through these methods, the amount of runoff and pollutants generated from both large-scale development projects and individual lot development can be reduced. Intrinsic GSPs are very site specific and should be carefully examined for applicability on different types of sites and for different proposed development. The goals of intrinsic GSPs are as follows:

- Manage stormwater (quantity and quality) as close to the point of origin as possible and minimize collection and conveyance;
- Prevent stormwater impacts rather than mitigating them downstream;
- Use simple, non-structural methods for stormwater management that are less costly and require less maintenance than structural controls;
- Create a multifunctional landscape;
- Use hydrology as a framework for site design; and
- Protect in-situ soils.

The goal of this first step in the design process is to reduce the anticipated environmental impact "footprint" of the development and to maintain or improve the natural ability of the site to capture runoff while retaining and enhancing the owner/developer's purpose and vision for the site. Intrinsic site design concepts can reduce the cost of infrastructure while maintaining or even increasing the value of the property.

Additional information for Intrinsic GSPs can be found in the specification sheets in Appendix A. The following benefits apply to almost all the Intrinsic GSPs (Southeastern Michigan Council of Government (SEMCOG), 2008).

- Reduced land clearing costs,
- Reduced costs for total infrastructure,
- Reduced total stormwater management costs,
- Enhanced community and individual lot aesthetics, and
- Improved overall marketability and property values.

Intrinsic GSPs are both water quantity and water quality management tools and can reduce the size and cost of required structural GSPs. 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, has a lower maintenance burden, and is more sustainable.

5.2.2.1 Initial Intrinsic GSP Site Considerations

The process of selecting intrinsic GSPs begins after initial evaluation of the site and consideration of additional federal or state requirements and City ordinances. The goal of this process is to generate an initial concept design for the intrinsic GSPs that can then be evaluated for water quality and quantity volumes and further used to plan any additional structural GSPs that can be added to meet water quality goals. Using initial information collected during the site evaluation, the intrinsic practices can be conceptually designed to address the following considerations:

Evaluating the Existing Site
By taking a thorough inventory of the site's initial layout and resources, some intrinsic GSPs may be immediately eliminated from consideration. The designer should answer the following questions:

- What features currently exist onsite that have the potential to be preserved as conservation/sensitive areas, natural flow paths, and areas of no soil compaction?
- Are there riparian buffer areas that can be protected?
- Are there any areas that could be preserved from disturbance?

Proposed Site Use
The proposed use of the site should be evaluated to correctly select the intrinsic GSPs that will most likely best fit the development. The following questions should be answered:

- What type of development is to be placed on the site and what intrinsic GSPs are typically associated with this type of development?
- Is cluster development an option?
- Are there chances to reduce impervious cover and utilize stormwater disconnection?

5.2.2.2 Intrinsic GSP Selection Process

There are several strategies to aid in selection of intrinsic GSPs to achieve runoff reduction. For each category the Intrinsic GSPs are listed adjacent to the category description. Some of the intrinsic GSPs bridge multiple categories. For additional information for each of the Intrinsic GSPs, see Appendix A.

Conservation of Natural Features and Resources
Identify and preserve the natural features and resources that can be used to protect water resources, reduce stormwater runoff, provide runoff storage, reduce flooding, prevent soil erosion, promote infiltration, and remove stormwater pollutants. These features include:

Conservation Intrinsic GSPs:
- **Protect Natural Flow Pathways**
- **Protect Riparian Buffers**
- **Minimize Soil Compaction**

- Areas of undisturbed vegetation
- Floodplains and riparian areas
- Ridgetops and steep slopes
- Natural drainage pathways
- Intermittent and perennial streams
- Aquifers and recharge areas
- Wetlands
- Soils
- Other natural features or critical areas

Perform a delineation of natural features and a comprehensive site analysis and inventory before site layout design is performed. Approaches should:

- Preserve undisturbed natural areas and riparian buffers.
- Avoid floodplains and steep slopes.
- Minimize siting on highly permeable or highly erodible soils.

Low Impact Site Design Strategies

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.

Site Design Intrinsic GSPs:
- **Protect Sensitive Areas**
- **Minimize Total Disturbed Area**

Reduction of Impervious Cover

Methods include:

- Reduce roadway lengths.
- Reduce roadway widths.
- Reduce building footprint(s).
- Reduce parking footprint(s).
- Reduce setbacks and frontages.
- Install fewer or alternative cul-de-sacs.

Intrinsic GSPs:
- **Reduce Impervious Area**

Use Natural Features for Stormwater Management

Careful site design can reduce the need and size of structural conveyance systems and controls through use of natural site features and drainage systems. Methods of incorporating natural features into an overall stormwater management site plan may include:

- Use buffers and undisturbed areas.
- Use natural drainageways instead of storm sewer systems.
- Use vegetated swales instead of curb and gutter.
- Drain runoff to pervious areas.

Intrinsic GSPs:

- **Protecting Natural Flow Pathways**

5.2.2.3 Implementing a Stormwater Sensitive Site Design

Taken from the methodology above, Table 5.2 provides a review checklist for site design. The questions are organized by GSP categories.

Table 5.2. Questionnaire for designer on implementing stormwater better site design practices.

Conservation of Natural Features	**Natural Area Conservation** • Is natural vegetation preserved on-site?
	Tree Conservation • Are original trees preserved on-site?
	Stream Buffers • Are stream buffer requirements properly enforced with at least the minimum area left undisturbed?
	Floodplains • Are buildings to be located out of the 100-year floodplain?
	Steep Slopes and Limiting Soils • Have buildings on steep slopes and slopes with highly erodible soils been minimized or restricted?
Low Impact Site Designs	**Fitting Site Designs to the Terrain** • Has developer worked to best fit design concepts to the site topography and to protect key site resources?
	Clearing and Grading • If multi-phased project, has developer limited the amount of cleared land to what is needed on a phase-by-phase basis?
	Is the Development Located in a Less Sensitive Area?
	Open Space Development • Was a cluster development considered, if an option? • Is a significant percentage of the open space managed in an undisturbed, natural condition? • Is an association planned that can effectively manage open space?
	Nontraditional Lot Designs • Are nontraditional lot designs and shapes included in the development?
	Creative Development Design • Are Planned Unit Developments (PUD's) included in the development?

Table 5.2. Questionnaire for designer on implementing stormwater better site design practices.	
Reduction of Impervious Cover	**Roadway Length** • Are the most efficient site and street layouts used consistent with Fayetteville Master Street Plan to reduce overall street length?
	Roadway Width • Are the minimum pavement widths used based on Master Street Plan LID sections?
	Building Footprint • Are taller buildings and structures utilized, if permitted, to reduce the development's overall impervious footprint?
	Parking Footprint • Is the minimum applicable parking ratio used for the planned development? • If mass transit is provided nearby, are parking ratios reduced? • Are the minimum stall width and length used for a standard parking space? • Do at least 30% of the spaces at larger commercial parking lots have smaller dimensions for compact cars? • Are shared parking arrangements used? • Is parking within structured decks or ramps rather than surface parking lots utilized? • Are porous surfaces used for overflow parking areas? • Are bioretention islands and other structural control practices used within landscaped areas or setbacks?
Site Layout Requirement	**Setbacks and Frontages** • Are minimum front, rear, and side setbacks used for residential lots in accordance with cluster development? • Is the minimum frontage distance used for residential lots?
	Alternative Cul-de-sacs • Are cul-de-sacs designed for the minimum allowed radius? • If allowed, are landscaped islands utilized within cul-de-sacs? • Are alternative turnarounds such as "hammerheads" used on short streets in low density residential neighborhoods?
Utilization of Natural Features for Stormwater Management	**Using Buffers and Undisturbed Areas** • Are level spreaders used to promote sheet flow of runoff across buffers and natural areas?
	Using Natural Drainageways • Where possible, are natural systems used in place storm sewer systems?
	Using Vegetated Swales • Are vegetated swales used instead of curb and gutter, where possible?
	Rooftop Runoff • Is rooftop runoff designed to permanently discharge to pervious yard areas? • Where possible, is temporary ponding of runoff on lawns or rooftops implemented?

Additional guidance on implementing better site design is provided in "*Low Impact Development: A Design Manual for Urban Areas"* published in 2010 by the Community Design Center at the University of Arkansas (http://uacdc.uark.edu/books.php).

5.2.3 Green Stormwater Practices Selection Criteria

Factors for GSP Selection
- **Close to Source**
- **Maximize Dual Use**
- **Site Features**
- **Contributing Drainage Area**
- **Applicability of Land Use**
- **Runoff Quality and Quantity**
- **Costs**
- **Construction**
- **Maintenance**
- **Aesthetics**

Steps 2 and 3 of the Runoff Reduction Method (Section 5.3) involve the use of structural and non-structural GSPs for a site. Following are general guidelines and limiting features to aid GSP selection. In some cases, limiting factors to GSP use may be overcome through innovative design. In all cases, selected GSPs, along with supporting criteria and computations, and any compromises or design features, shall be presented to the City to ensure proper evaluation and review.

Structural or non-structural GSP selection should be based on the functional goal of the practice. The decision making process used to select a GSP must balance the goals of the proposed facility against site constraints and the limiting characteristics of the GSPs. A successful design process requires balancing the technical and nontechnical factors and is summarized in Figure 5.2.

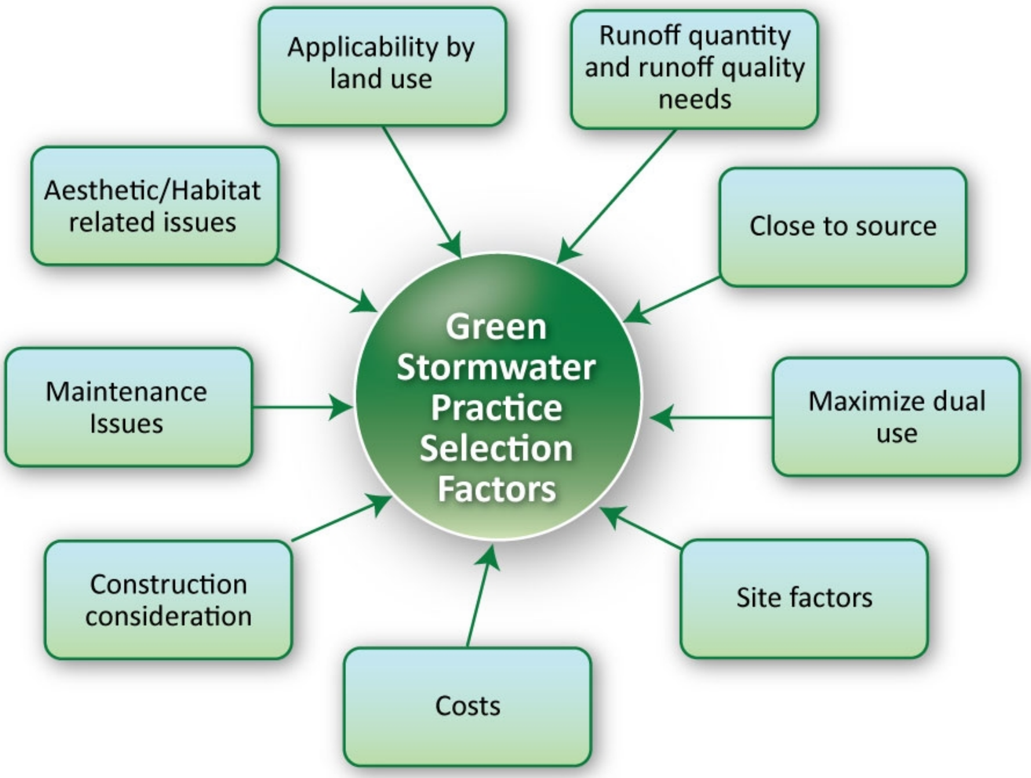

Figure 5.2. Green stormwater practice selection factors.

Site feasibility factors to consider include:

- Proximity to runoff source,

- Maximization of dual use,
- Topographic and/or geologic constraints,
- Contributing drainage area size, and
- Applicability by land use.

The GSP factors to consider include:

- Runoff quality and quantity,
- Costs,
- Construction considerations,
- Maintenance, and
- Aesthetics.

When selecting the most appropriate GSP for a site, a treatment train or a set of GSPs in series may be necessary to achieve the reduction goals on a site for which one GSP is not sufficient. Treatment trains can have many combinations and used for all types of sites, as dictated by the site layout and proposed GSPs. An example is provided in Figure 5.3. The approach for including GSPs in series is located in Section 5.3.

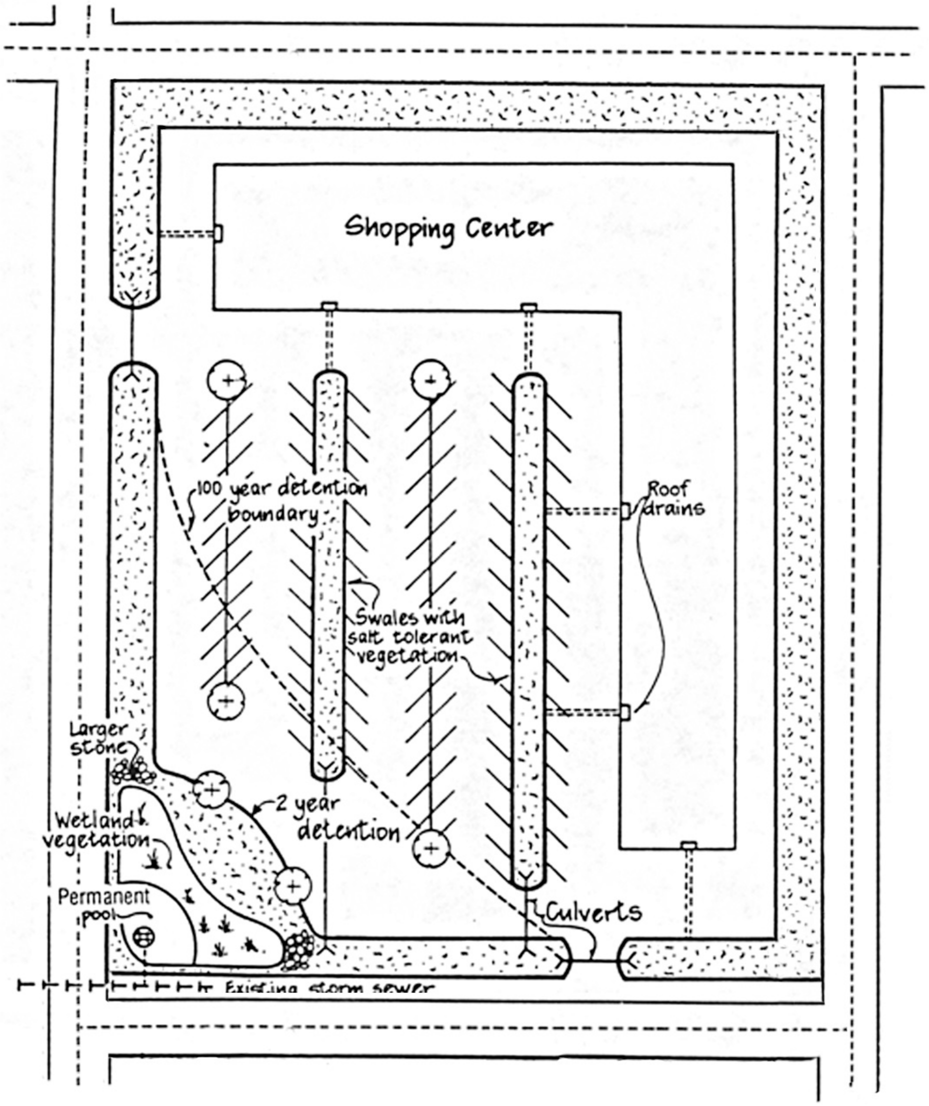

Figure 5.3. Treatment train example - commercial development (Source: NIPC, 2000).

5.2.3.1 Site Feasibility Factors

Source Considerations

Manage stormwater runoff as close to the source, or origin, as possible. Implementing this factor will vary by site and by the proposed development. GSPs should be selected and placed so that runoff from treatment areas (impervious surfaces) flows directly into GSPs via sheet flow or is piped a short distance and then discharged into a GSP. Managing stormwater close to the source means treating the stormwater on site and minimizing piping to a centralized end-of-pipe treatment system.

Maximize Dual Use
Integrate stormwater management into already disturbed areas where possible (e.g., stormwater infiltration systems beneath parking areas, play fields on infiltration basins).

Site Factors
Each site should be inventoried for certain characteristics (e.g., soil type, depth to water table, slopes) that are a part of GSP selection and design. The GSP specification sheets highlight these site factors which are discussed in more detail in Appendix B. The following list describes some of the site factors that may need to be considered in Fayetteville.

1. *Karst topography:* These areas present very difficult challenges since any GSP which impounds water may cause underlying caverns or sink holes to expand and open at the surface. The use of liners may help the GSP hold the runoff as intended; however, the conveyance to the BMP, as well as the conveyance from the GSP to the receiving channel, must also be considered since the overall volume of runoff may be increasing or directed to areas previously not impacted by runoff. The presence of karst topography may allow a direct path for the stormwater runoff to enter the water table with little or no filtering of pollutants. Design in areas suspected to include karst topography should be supported by a karst survey and, if warranted, further geotechnical investigation. A Karst Area Sensitivity Map of Washington County (The Nature Conservancy, 2007) is available for review at http://nwarpc.org/pdf/GIS-Imagery/KASM_WASHINGTON_CO.pdf.
2. *High water table:* A high water table can impact the proper function of a GSP. Infiltration GSPs are restricted since a high water table will prevent the percolation of stormwater into the subsoils. A high water table may also cause dry detention GSPs to evolve into wet facilities.
3. *Bedrock:* The presence of bedrock near the surface can significantly impact a development project. The excavation costs can increase considerably.
4. *Proximity to structures and steep slopes.* One of the goals of stormwater facilities is to provide groundwater recharge. This tends to saturate the adjacent ground during and for a period of time after a storm event. Building foundations, basements, and other structures may be impacted by the wet/dry cycle of the surrounding soils. Saturating the soils on or adjacent to steep slopes (6 to 10 percent or greater) can cause a failure of the slope and adjacent structures.

   Areas of high pollutant potential are defined as land uses that generate higher concentrations of a particular pollutant or pollutants, such as sediment, hydro-carbons, trace metals, or toxicants, than are found in typical stormwater runoff. The use of GSPs is limited on sites considered to be have high pollution potential. Due to the potential for groundwater contamination, infiltration facilities are not recommended in such areas. The use of impoundment type structures in such areas should be qualified by an adequate vertical separation from the seasonal groundwater table (4 feet separation is desirable, and a 2 foot separation minimum); alternately, an impermeable liner may be used to prevent infiltration. Table 5.3 includes a list of typical areas with high pollution potential.

Table 5.3. Areas of high pollution potential.	
The following land uses and activities are typical areas of concern:	
Vehicle salvage yards and recycling facilities	Marinas (service and maintenance)
Vehicle fueling stations	Outdoor liquid container storage
Vehicle service and maintenance facilities	Outdoor loading/unloading facilities
Vehicle and equipment cleaning facilities	Public works storage areas
Fleet storage areas (bus, truck, etc.)	Facilities that generate or store hazardous materials
Industrial sites (for SIC codes reference Arkansas Department of Environmental Quality (ADEQ))	Commercial container nursery

Applicability by Land Use
Some land uses lend themselves to certain GSPs. Low density residential development lacks large congregate parking areas conducive to pervious pavement with infiltration, though pervious pavement might be appropriate for street side parking areas or roads. Conversely, rain barrels are especially good for residential use, but vegetated roofs are unlikely to be used on single-family homes.

5.2.3.2 Practice Feasibility Factors

Not all structural GSPs are appropriate for varying types of sites and development. The selection process for the large array of structural GSPs can be complex, due to the number of factors. Tables 5.4 and 5.5 below provide a summary of the stormwater quality and quantity functions, cost, relative difficulty to construct, relative maintenance intensity, and performance level for each GSP, to aid in the selection process. The following factors should be considered when selecting GSPs:

Water Quality and Quantity Goals
The City of Fayetteville currently has a goal of at least 80% removal of TSS from flows that exceed predevelopment levels where practicable. The City also has water quantity standards for design. The post-development peak rate of surface discharge must not exceed the existing discharge for the 10-year, 25-year and the 100-year, 24-hour storms. Additional channel protection requirements also apply. Refer to Chapter 1 for the Minimum Standard requirements for stormwater management.

Cost
GSP costs include both construction and long-term maintenance activities. Costs are often related to the size and nature of the development. "BMP and LID Whole Life Cost Models: Version 2.0", WERF (2009), and "Rapid Assessment of the Cost-Effectiveness of Low Impact Development for CSO Control", Montalto et al. (2007) are two resources that may aid in initial GSP selection decisions based on cost.

Construction
The GSP specifications in Appendices A and B include general construction guidelines that provide instruction on proper installation practices and materials. These guidelines shall be followed for projects designed to meet water quality goals within the City of Fayetteville.

Maintenance
A list of maintenance requirements must be included with GSP design, and considered when selecting a GSP. Some GSPs require greater maintenance to function properly. Vegetated GSPs require various types of landscape care. Structural GSPs such as pervious pavement require periodic maintenance such as

vacuuming, while infiltration basins, trenches, and grass channels are likely to require less maintenance. Some BMPs, especially those with plantings, may naturally improve in performance over time as vegetation grows and matures. In any case, general maintenance requirements are discussed for each GSP specification in Appendices A and B.

Aesthetics/Habitat

Landscape enhancement is an important goal in the City of Fayetteville. GSPs can fulfill the dual purpose of stormwater management and landscape feature. Bioretention and urban bioretention, water quality swales and filter strips, vegetated roofs, and many other GSPs should be integrated into landscape design when used, and can create value in addition to solving stormwater problems.

Table 5.4. GSP selection factors.

Structural Control	Selection Criteria	
	Volume	Water Quality
Bioretention	●	●
Tree Planters/Urban Bioretention	⦿	●
Permeable Pavement	●	⦿
Infiltration	●	●
Water Quality Swales (Dry)	⦿	●
Dry Ponds	○	○
Downspout Disconnection	⦿	⦿
Grass Channels	○	○
Sheet Flow	●	⦿
Urban Reforestation	●	●
Rain Tanks/Cisterns	⦿	○
Green Roofs	⦿	⦿

● Effective.
⦿ Moderately Effective.
○ Less Effective.

Table 5.5. GSP cost selection factors.			
Structural Control	Selection Criteria		
	Cost	Construction	Maintenance
Bioretention	⦿	●	⦿
Tree Planters/Urban Bioretention	⦿	⦿	⦿
Permeable Pavement	⦿	●	●
Infiltration	⦿	⦿	○
Water Quality Swales (Dry)	○	⦿	○
Dry Ponds	●	●	⦿
Downspout Disconnection	○	○	⦿
Grass Channels	○	○	○
Sheet Flow	○	○	○
Urban Reforestation	⦿	○	⦿
Rain Tanks/Cisterns	⦿	⦿	⦿
Green Roofs	●	●	⦿

● High/Intensive.
⦿ Moderate/Moderately Intensive.
○ Low/Less Intensive.

SECTION 5.3. THE RUNOFF REDUCTION METHOD

5.3.1 Introduction

5.3.1.1 Background

The Runoff Reduction Method (RRM) serves as the basis for the City of Fayetteville's approach to LID. The RRM derivation can be found in original references (Chesapeake Stormwater Network, not dated, Center for Watershed Protection, 2008). Volume removal is the focus of this approach, as volume reduction equals pollution reduction with respect to stormwater runoff. Through this method every post-development land surface is assigned a rating in terms of percent rainfall capture. Where practicable, runoff that is not captured through careful land use shall be managed using Green Stormwater Practices (GSPs).

The RRM also provides a way to achieve removal of Total Suspended Solids (TSS) and some other pollutants. This is accomplished by removing the volume of water that contains TSS and other pollutants. For the purposes of the RRM, it is assumed that 100% of the TSS is removed from volumes that are infiltrated, evapotranspired, or reused for purposes such as landscape irrigation or grey water reuse. Any pollutant removal is accomplished via settling, filtering, adsorption, and/or biological uptake.

The proportion of rainfall to be captured on the proposed post-development site shall be computed by calculating the weighted volumetric runoff coefficient (Rv) for the site using the assigned Rv values for each land use and Hydrologic Soil Group (HSG). For the purposes of this calculation, there are only three post-development land uses to chose from: forest or open space, disturbed soils, or impervious area.

Rv is the proportion of the total precipitation that runs off a specific land use area. Rv is equal to the post-development runoff depth divided by the target rainfall depth at a given site (one inch in this case). For example, if the weighted Rv for the developed site is 0.20, then 80% of the rainfall has been captured.

Once the Rv is determined, GSPs shall be applied, if needed, to reduce the runoff volume. The weighted Rv shall be reduced by treating areas of the site with GSPs described in Appendices A and B that have been assigned specific values corresponding to the proportion of runoff volume reduction. Where practicable, GSPs shall be implemented together with careful land use to achieve a weighted Rv for the post-developed site of 0.20, which will accomplish the capture of 80% of the runoff volume.

5.3.1.2 Objectives

The basis for the RRM is a percent rainfall volume capture goal. In Fayetteville the goal is **80%** volume capture as explained in the Chapter 5 Executive Summary.

To meet the 80% capture goal, where practicable, the designer shall lay out the site such that 80% of the rainfall is captured and treated on site through a combination of infiltration, evapotranspiration, harvest and/or use. This objective is accomplished through site layout and GSP design.

Whether or not the 80% goal is achieved, if the LID approach is desired to be used to attain credit for runoff reduction, the designer shall calculate the Rv and an adjusted Curve Number (CN_{adj}) to use in the runoff calculations for the site. The credit for using the RRM to calculate a CN_{adj} is that the post-development flows may be reduced accordingly, thereby reducing stormwater detention requirements.

5.3.1.3 Conceptual Design Steps in the Runoff Reduction Method

Conceptually, the RRM follows the steps shown in the flowchart in Figure 5.4 and briefly described below:

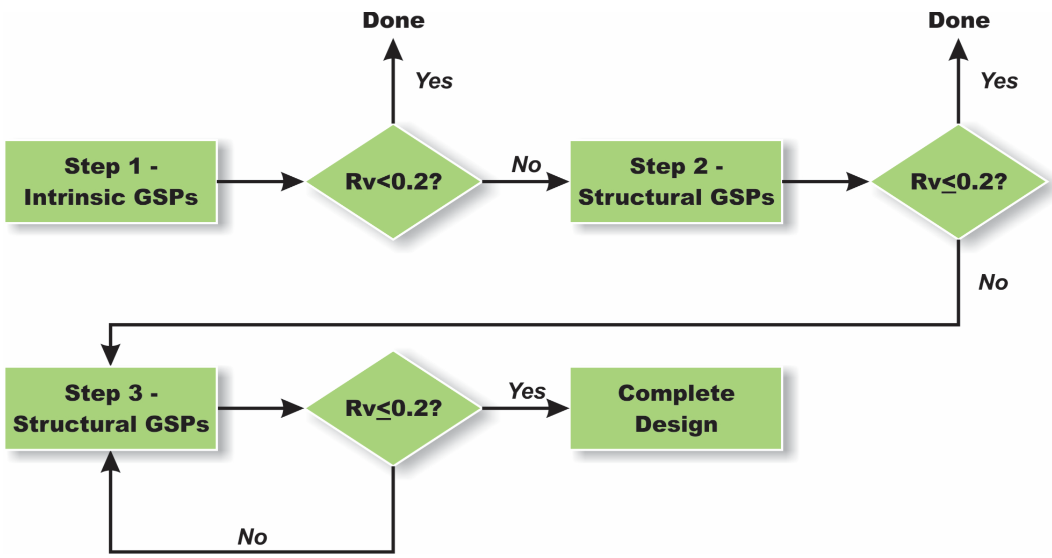

Figure 5.4. Runoff reduction design process.

Step 1: Reduce Runoff Through Land Use and Ground Cover Decisions (Intrinsic GSPs).
This step focuses on the existing and proposed land cover and how much of the rainfall it receives and removes from runoff. Design activities in Step 1include impervious area minimization, reduced soil disturbance, forest preservation, etc. The goal is to minimize impervious cover and mass site grading and

to maximize the retention of forest and vegetative cover, natural areas and undisturbed soils, especially those most conducive to landscape-scale infiltration. Information describing the methods and design criteria for Intrinsic GSPs can be found in **A**ppendix A, Intrinsic Green Stormwater Practice Specifications.

RRM calculations for Step 1 shall be based on the proposed post-developed site and cover and include the computation of volumetric runoff coefficients (Rv) for land use and existing Hydrologic Soil Group (HSG) combinations, including impervious cover.

Step 2: Apply Environmental Site Design Practices (Non-Structural GSPs).
If the target rainfall volume capture (Rv ≤ 0.2) has not been attained in Step 1, then non-structural GSPs provided in Appendix B can be implemented during the early phases of site layout to reduce the Rv. In this step, the designer enhances the ability of the existing land cover to reduce runoff volume through the planned and engineered use of such GSPs as disconnection of impervious areas (e.g., rooftops to sheet flow), pervious pavers, planned reforestation, etc. Each of these practices is assigned an ability to reduce one-inch of rainfall in a storm of moderate intensity; and this assignment is captured in the Runoff Removal Credit (RR Credit) in Table 5.7.

Step 3: Apply Structural GSPs.
If the target volume capture (Rv ≤ 0.2) has not been attained in Step 2, structural GSPs provided in Appendix B can be implemented to reduce the Rv. In this step, the designer evaluates combinations of engineered practices such as infiltration, bioretention, green roofs, stormwater planters, rainwater harvesting, etc. The designer applies the RR Credit for the area to be treated by each GSP to incrementally reduce the weighted Rv.

- At the end of Step 3, the designer will have computed the weighted Rv and can determine the percentage of rainfall captured. The designer will have met the 80% volume capture goal if the weighted Rv is 0.20 or less. The following sections describe how to calculate Rv and the associated variables.

5.3.2 Technical Design Procedure

5.3.2.1 STEP 1: Land Use Rv Values

As stated above, the volumetric runoff coefficient (Rv) is the ratio of the runoff depth divided by the target rainfall depth (one-inch in this case). If 45% of rainfall runs off the post-developed site, the Rv value equals 0.45. Unlike a Rational Method C Factor, for example, Rv is not a constant individual storm-based value, but is rainfall intensity and duration dependant. Rv values could be developed for individual storm intensities, seasons, or even annually. It should be noted that Rv is not equivalent to Curve Number.

Site layout shall be designed to include Intrinsic GSPs where practicable to reduce impervious area and disturbed soils. Once the general design concept is developed, the site layout shall be characterized into areas of either undisturbed soil, disturbed soils, or impervious area with respect to each. For each Hydrologic Soil Group (HSG), land use area, the volumetric runoff coefficients listed in Table 5.6 can be applied to the site. A weighted Rv value shall be calculated to determine the proportion of runoff leaving the site.

Table 5.6 shows the Rv values assigned for the City of Fayetteville's post-development conditions. The values were derived by comparison with other sources, review of Fayetteville's land use and rainfall conditions, and simulation modeling of various land use categories and soil types.

Table 5.6. Site cover runoff coefficients.				
Soil Condition	**Volumetric Runoff Coefficient (Rv)**			
Impervious Cover	0.95			
Hydrologic Soil Group	**A**	**B**	**C**	**D**
Forest Cover/Open Space[1]	0.02	0.03	0.04	0.05
Disturbed Soils[2]	0.15	0.18	0.20	0.23

1. Forest – undisturbed and non-compacted soils, protected forest, or reforested land.
2. Disturbed Soils – amended soils consisting of managed turf, graded for yards, or other landscaped areas
3. To be mowed/managed. Compacted fill qualifies as HSG D type soils only, whereas natural soils disturbed by light grading only shall be considered to be in the next higher HSG.

Note: Areas where earthwork other than light grading has been performed (i.e., proofrolled, received structural/non-structural fill, compacted) shall be considered HSG group D, and the Rv value of 0.23 shall be used for that area, regardless of whether topsoil and sod are or have been applied.

The area-weighted estimate of the total site Rv value is calculated using Equation 5.1.

$$\text{Weighted } R_V = R_{V1}(R_{V1}\ \%\text{ of site}) + R_{V2}(R_{V2}\ \%\text{ of site}) \qquad \textbf{Eq. 5.1}$$

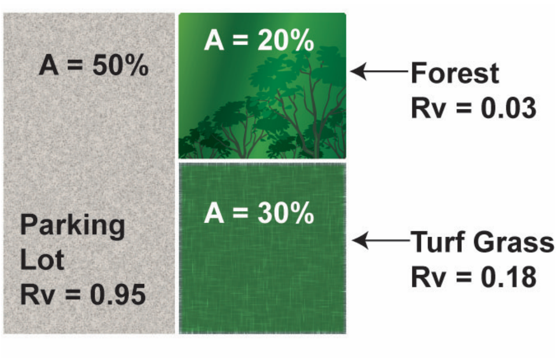

Figure 5.5. Site example with land uses.

EXAMPLE 5.1. Intrinsic GSPs

As shown in Figure 5.5, a site is designed using Intrinsic GSPs and 50% of the site is impervious, 20% is undisturbed forest, and 30% is landscaped turf grass with underlying HSG B soils, the Rv value would be:

Site Weighted Rv = 0.50*0.95 + 0.20* 0.03 + 0.30*0.18 = 0.54

That is, 54% of the rainfall on the site runs off. In order to achieve a higher percentage of runoff removal, i.e., to meet the 80% goal, additional GSPs must be planned and implemented.

5.3.2.2 STEPS 2 AND 3: Green Stormwater Practice Rv Values

Steps 2 and 3 of the RRM describe how planning and design of additional Green Stormwater Practices (GSPs) shall be performed in cases where the goal is to reduce the total site Rv to 0.20 or less. The twelve GSPs that are acceptable for use in Fayetteville to attain additional volume reduction credit are listed in Table 5.7.

Each GSP can be designed to either a Level 1 or Level 2 runoff reduction capability. The Level 1 designs have slightly less stringent design guidelines and therefore do not provide the same level of runoff reduction of a Level 2 design. Level 2 designs provide significant runoff reduction capability; however, very specific design requirements must be met to achieve the Level 2 treatment. Refer to *Appendix B – GSP Specifications* for design guidance for Green Stormwater Practices.

Note that the first six GSPs themselves occupy site land area. Because of their ability to absorb the rain that falls on them they are assigned the corresponding Forest Cover Rv values from Table 5.6. Other GSPs, where applicable, are assigned the Disturbed Soils land cover Rv values from Table 5.6.

<table>
<tr><th colspan="9">Table 5.7. Green stormwater practices runoff reduction credit percentages.</th></tr>
<tr><th rowspan="2"></th><th colspan="8">Rainfall Volume Removed (Captured)</th></tr>
<tr><th colspan="4">Level 1</th><th colspan="4">Level 2</th></tr>
<tr><td>Structural</td><td colspan="8"></td></tr>
<tr><td>Bioretention (GSP-01)</td><td colspan="4">60</td><td colspan="4">90</td></tr>
<tr><td>Urban Bioretention (GSP-02)</td><td colspan="4">60</td><td colspan="4">N/A</td></tr>
<tr><td>Permeable Pavement (GSP-03)</td><td colspan="4">45</td><td colspan="4">75</td></tr>
<tr><td>Infiltration Trench (GSP-04)</td><td colspan="4">50</td><td colspan="4">90</td></tr>
<tr><td>Water Quality Swale (GSP-05)</td><td colspan="4">40</td><td colspan="4">60</td></tr>
<tr><td>Extended Detention (GSP-06)</td><td colspan="4">0</td><td colspan="4">20</td></tr>
<tr><td>Grass Channel (GSP-08)</td><td colspan="4">10/20</td><td colspan="4">20/40</td></tr>
<tr><td>Rain Tanks/Cisterns (GSP-11)</td><td colspan="8">Design dependent</td></tr>
<tr><td>Green Roof (GSP-12)</td><td colspan="4">80</td><td colspan="4">90</td></tr>
<tr><td>Non-Structural</td><td colspan="8"></td></tr>
<tr><td>Disconnection – downspout (GSP-07)</td><td colspan="4">25</td><td colspan="4">50</td></tr>
<tr><td>Disconnection – sheet flow (GSP-09)</td><td colspan="4">50</td><td colspan="4">75</td></tr>
<tr><td>Reforestation (A, B, C, D soils) (GSP-10)</td><td>96</td><td>94</td><td>92</td><td>90</td><td>98</td><td>97</td><td>96</td><td>95</td></tr>
</table>

To calculate the R_v value for a contributing drainage area flowing through a GSP use Equation 5.2.

$$\mathbf{Rv_{reduced}} = \mathbf{CDA\ R_V(1 - RR\ Credit)} \qquad \textbf{Eq. 5.2}$$

where CDA Rv equals the Contributing Drainage Area volumetric runoff coefficient for the drainage area being treated. CDA Rv should be weighted using Equation 5.1 if the drainage area has multiple land uses. If the drainage area contains only one land use, CDA Rv is the Rv for that land use.

EXAMPLE 5.2. Calculating Rv Reduced.

Part of the proposed site is impervious and has a grass landscaped area draining to it. The CDA Rv = 0.85. This entire area is to be treated by a Level 2 bioretention structure (90% RR Credit). The reduced weighted volumetric runoff coefficient $Rv_{reduced}$ would be calculated as follows:

$$Rv_{reduced} = 0.85 *(1-0.90) = 0.09$$

Thus the bioretention facility meeting the Level 2 design criteria would cause the drainage area to exceed the goal of 80% volume reduction or better (an Rv of 0.20 or less) as the Rv is 0.09.

5.3.2.3 Rv Values for GSPs in Series

The volume removal rate for controls in series may be computed by extending Equation 5.2, if appropriate GSPs are used as described herein. The upstream control has the benefit of initially addressing runoff from all storms, while the second control in the series must handle the overflow from the first, that will consist of a subset of fewer and larger storms. Therefore the ability to capture instantaneous volumes and store them for later removal is key for downstream controls.

In addition to cisterns, only the first six controls in Table 5.7 can be used as the second GSP in a series volume removal calculation: bioretention, urban bioretention, permeable pavement, infiltration trench, water quality swale, and extended detention.

The following equation shall be used for calculation of the total Rv factor for GSPs in series:

$$\mathbf{Rv\ Series} = \mathbf{CDA\ Rv(1 - RR_1\ Credit)\ (1 - RR_2\ Credit)} \qquad \textbf{Eq. 5.3}$$

where CDA Rv is the first GSP in the series, for example below (Rv = 0.95 for the impervious area), RR_1 Credit and RR_2 Credit are respective the percent volume reduction credits for the first and second GSPs in the series from Table 5.7. **Credit will be granted for, at most, two controls used in a series.** Any more than that will not be counted towards the runoff reduction.

EXAMPLE 5.3. GSPs in Series

Runoff from an impervious area (Rv = 0.95) is treated with sheet flow disconnection then enters a bioretention facility. The following calculation gives the final Rv for that impervious area. The sheet flow GSP is Level 2 (RR Credit 75%) while bioretention design is Level 1 (RR Credit 60%).

Total Rv credit = 0.95*(1-0.75)*(1-0.6) = 0.095

That is, 95% of the rainfall runs off the impervious area and enters the sheet flow area. Of that runoff, 75% is captured in the sheet flow area. The remainder (the larger storms) enters the bioretention facility and 60% of that is captured by that GSP, allowing 9.5% of the rainfall to overflow the facility. Hence, this CDA has a 90.5% rainfall removal rate – well above the 80% (Rv of 0.20 or less) goal.

5.3.2.4 Sizing of Media-Based GSPs

For the sizing of media-based GSPs such as bioretention, tree planters, permeable pavement, infiltration trenches, water quality swales, and others it is assumed that the runoff from a one-inch storm is instantaneously contained within the control, and that the control is completely drained prior to the storm event, with no moisture susceptible to gravity drainage. These assumptions result in a design that conservatively approximates an 80% removal of runoff volume (Rv = 0.20) for native soil infiltration rates. To accommodate removal, underdrains are required for parent material infiltration rates less than 0.5 in/hr or for GSP designs where no infiltration test is performed to support calculations. As such the following guidance is provided for sizing these types of facilities. Details for each type are provided in the respective specification section in Appendices A and B. Details for sizing cisterns are also located in the GSP-11 specification in Appendix B. An in-situ infiltration test procedure is provided in Appendix C.

Table 5.8 provides volume-based specifications for the engineered soil-based and gravel media prepared in accordance with the GSP designs in this manual.

Average total porosity of non-compacted soil and gravel can range from 0.25 to 0.50 (Freeze and Cherry, 1979). Field capacity of the soil is the amount of moisture typically held in the soil/gravel after any excess water from rain events has drained and varies greatly between soil-based media and gravel. Effective porosity is defined as the difference between total porosity and field capacity. Values to be used for design are reported in Table 5.8.

Table 5.8. Media volume-based specifications.

Parameter	Value
	Total Porosity, n
Soil-Based Media[1]	0.40
Gravel[2]	0.35
Ponding	1.0 (void ratio)

1. Soil-Based Media GSPs - bioretention, water quality swales and tree planter boxes.
2. Gravel GSPs - design alternatives for soil-based GSPs, storage layers for permeable pavement, and infiltration trenches.

All media-based GSPs shall be sized to provide storage volume for the complete runoff from one inch of rain over the contributing drainage area (CDA) in order to achieve the listed runoff reduction credit for the GSP. A GSP may be sized using less than one inch of rainfall over the contributing drainage area; however, Equation 5.12 in Section 5.3.2.6 shall be used to calculate the percentage of rainfall volume the GSP captures. All of the other design requirements detailed in Appendix B for the GSPs shall be followed. All media storage GSPs shall be sized using the following equations:

$$\mathbf{T_V = P(CDA)(R_V)\left(\frac{43560\ ft^2}{1\ ac}\right)\left(\frac{1\ ft}{12\ in}\right) = n(D)(SA)} \qquad \textbf{Eq. 5.4}$$

where:

Tv	GSP treatment volume in cubic feet.
CDA	The drainage area in acres.
P	1 inch.

Rv	Runoff coefficient for the CDA.
SA	Surface area in square feet of the GSP.
D	Media depth of GSP in feet.
n	Total Porosity.
D	Depth, Dw if more than one media type is required. See Equation 5.5.

The equivalent storage depth for media-based GSPs with multiple layers of media must be calculated using the following equation:

$$\textbf{Weighted Storage Depth} = \mathbf{D_W} = \mathbf{n_1}(\mathbf{D_1}) + \mathbf{n_2}\,(\mathbf{D_2}) + \qquad \textbf{Eq. 5.5}$$

Where n_1 and D_1 are for the first layer, etc.

Note that the Rv value is for the total area draining to the control. So if a filter strip is included in the area then a weighted Rv should be calculated but not a credit reduced Rv.

EXAMPLE 5.4. Media-Based GSP

A 1.5 acre parking lot is to drain to a Level 1 Bioretention GSP with the following media composition: 2 ft soil media and 0.5 ft of ponding. Then by application of **Equations 5.4** and **5.5,** solving for SA:

Tv = 1”*1.5*0.95*(43560/12) = 5173 cu ft = SA*Dw = SA (0.5 ft (1.0) + ((0.40) x 2 ft)) = SA (1.3 ft)
SA of GSP = 3,979 sq ft

5.3.2.5 Calculation of Curve Numbers with Volume Removed

The removal of volume by GSPs changes the runoff depth entering downstream flood control facilities. The resulting decrease is accounted for by calculating runoff based on an “adjusted SCS curve number” using (CN_{adj}) which is less than the actual curve number (CN). The reduced runoff allows reduced detention storage.

Standard SCS rainfall-runoff Equations 5.6 and 5.7 provide a way to calculate a total runoff if the rainfall and curve number are known, as:

$$\mathbf{Q} = \frac{(\mathbf{P}-(\mathbf{0.2}*\mathbf{S}))^2}{(\mathbf{P}+(\mathbf{0.8}*\mathbf{S}))} \textbf{ and } \mathbf{S} = \frac{\mathbf{1000}}{\mathbf{CN}} - \mathbf{10} \qquad \textbf{Eqs. 5.6 and 5.7}$$

Where P is the rainfall depth for the 24-hour design storm (Table 5.9), Q is the total runoff depth for that storm, S is potential maximum soil moisture retention, and CN is runoff curve number.

.

Table 5.9. Fayetteville 24-hour rainfall depths.	
Return Period (Years)	Rainfall Depth (Inches)
2	4.10
5	5.26
10	6.10
25	7.16
50	7.96
100	8.8

The adjusted total runoff depth entering the flood control facility downstream of a GSP is calculated as the difference between total runoff in depth and the depth captured by the GSP, as shown below.

$$\mathbf{Q_{adj} = Q - Q_{removed}} \qquad \textbf{Eq. 5.8}$$

$Q_{removed}$ is defined in Equation 5.10.

The depth of captured rainfall ($Q_{removed}$)over the CDA is determined by first calculating the available volume that the GSP can capture (V_{cap}) as shown in Equation 5.9:

$$\mathbf{V_{cap} = (D_p)(SA) + (n_1)(D_1)(SA) + (n_2)(D_2)(SA)} \qquad \textbf{Eq. 5.9}$$

Where depths are in feet:

V_{cap} Volume captured by the GSP, ft^3
D_p Depth of ponding (if applicable), ft
SA Surface area of the GSP, ft^2
n_1 Total porosity of first soil media or gravel layer
D_1 Depth of first soil media or gravel layer, ft

The depth of total runoff, in inches, removed from site by the GSP, is calculated by Equation 5.10:

$$\mathbf{Q_{removed} = \frac{(V_{cap})(12)}{43560(A)}} \qquad \textbf{Eq. 5.10}$$

Where A is site area in acres.

Equation 5.11 provides a method to calculate the modified curve number once the Q_{adj} is found.

$$\mathbf{CN_{adj} = \frac{1000}{10+5P+10Q_{adj}-10\left(Q_{adj}{}^{2}+1.25Q_{adj}P\right)^{1/2}}} \qquad \textbf{Eq. 5.11}$$

The steps in calculating an adjusted Curve Number (CN_{adj}) are:

Step 1. **Calculate Total Runoff for Storm (Q)** Choose the design return period(s), and using appropriate return period rainfall as P, calculate an initial Q using Equations 5.6 and 5.7, with the calculated pre-GSP site curve number.

Step 2. **Calculate GSP Capture Volume (Vcap)** Compute the captured volume in the GSP control using the dimensions of the media or proven cistern volume assuming a 72-hour dry period since the last cistern filling event.

Step 3. **Calculate Depth of Total Runoff Removed from Site ($Q_{removed}$)** Convert the captured volume into inches over the site or relevant CDA (Equation 5.10).

Step 4. **Calculate Adjusted Total Runoff (Q_{adj})** As shown in Equation 5.8, subtract $Q_{removed}$ from Q computed in Step 1.

Step 5. **Calculate Adjusted Curve Number (CN_{adj})** Using Q_{adj} and the appropriate design storm rainfall depth P, calculate CN_{adj} from Equation 5.11.

Step 6. Use CN_{adj} in calculations for the appropriate return period(s) in question.

The following example illustrates this procedure.

EXAMPLE 5.5. Compute Adjusted Curve Number

A 1.5-acre parking lot (CN=98) is to drain into a site detention pond for the 2-year through 100-year storm. To account for a bioretention basin through which the parking lot drainage is directed prior to overflows entering the detention pond, an adjusted curve number (CN) should be calculated for the parking lot. The following example shows the calculation of a CN_{adj} for the 100-year storm event for the situation described. The CN_{adj} for the other storms should be calculated to complete the routing calculations for the detention pond.

Step 1. Using Equations 5.6 and 5.7 for the 100-year storm (P = 8.8), the calculated runoff depth (Q) = 8.56 in.

$$Q = \frac{(8.8 - (0.2 * 0.20))^2}{(8.8 + (0.8 * 0.20))} = 8.56 \text{ where } S = \frac{1000}{98} - 10 = 0.20$$

Step 2. Using the bioretention facility sized from the previous example the available captured volume (V_{cap}) = 5,173 cu ft.

$$V_{cap} = (0.5)(3{,}979) + (0.4)(2)(3{,}979) = 5{,}173 \text{ cu ft}$$

Step 3. Over 1.5 acres the depth of runoff removed ($Q_{removed}$) = 0.95 in.

$$Q_{removed} = \frac{(5{,}173)(12)}{43{,}560(1.5)} = 0.95 \text{ in.}$$

Step 4. Qadj = 7.61 in.

$$Q_{adj} = 8.56 - 0.95 = 7.61 \text{ in.}$$

Step 5. Using Qadj and the 100-year P in Equation 5.11 we obtain the adjusted curve number of 87 (rounded to the next whole number). We can check our work by substituting this CN back into Equation 5.7 and applying Equation 5.6 to obtain the Q of step 4.

$$CN_{adj} = \frac{1000}{10 + 5(8.8) + 10(7.61) - 10[(7.61)^2 + 1.25(7.61)(8.8)]^{1/2}} = 90$$

The use of underdrains with media-based GSPs is required for soils that have infiltration rates of less than 0.5 inches/hour or if no in-situ infiltration test is provided to support calculations. When underdrains are used, the curve numbers shall not be adjusted but the portion of discharge captured by the GSP may be hydrologically routed through the GSP and underdrain system for the purpose of computing required detention storage.

5.3.2.6 Calculation of Rainfall Removal Based on Capture Depth

If a GSP is sized to capture less than 80% rainfall removal (1 inch of rainfall), then the calculation of actual percentage of removal must be provided for review to the City Engineer together with the calculated RR credit. Equation 5.12 shall be used to calculate the percentage of rainfall capture. Figure 5.2 shows the relationship between capture depth and percent rainfall removal for the City of Fayetteville. For example, if a GSP captures 0.5 inches of runoff the capture is 56%. The rainfall capture is calculated as follows:

$$\text{Rainfall Capture} = -0.0223D^4 + 0.2262D^3 - 0.8496D^2 + 1.4316D + 0.0161 \qquad \text{Eq. 5.12}$$

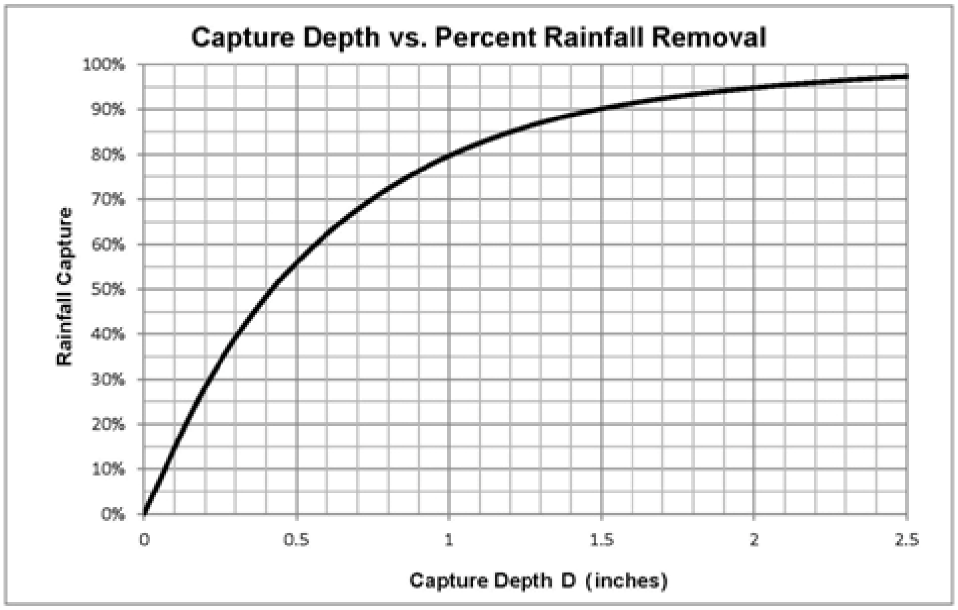

Figure 5.6. Capture depth vs. rainfall capture.

SECTION 5.4. GREEN STORMWATER PRACTICES

5.4.1 Overview

Green Stormwater Practices (GSPs) are intended to mimic the natural hydrologic condition and promote infiltration, filtration, storage/reuse, and evapotranspiration of stormwater runoff. The GSPs detailed in this manual include: bioretention, urban bioretention, permeable pavements, infiltration devices, water quality swales, extended detention ponds, downspout disconnection, grass channels, sheet flow, reforestation, rain tanks/cisterns, and green roofs.

As detailed, GSPs are designed to meet multiple stormwater management objectives, including reductions in runoff volume, peak flow rate reductions, and water quality protection. Multiple small, localized controls may be combined into a "treatment train" to provide comprehensive stormwater management. The methodology for this is provided in Section 5.3. The GSPs in this section have been designed to be integrated into many common urban land uses on both public and private property, and may be constructed individually, or as part of larger construction projects. Decentralized management strategies are encouraged to be tailored to individual sites, which can eliminate the need for large-scale, capital-intensive centralized controls, and may improve the water quality in Fayetteville's streams.

GSPs and the Transect: The Transect is a system of classifying urban environments from rural to urban according to intensity of development and land use character. The goal of Transect-based planning and design is to serve a variety of preferences for urban environments delivered in neighborhoods that are laid out and designed in an internally consistent manner. These zones may range in size from a few acres to square miles. Lower density Transect zones (rural, suburban, and general urban) can more easily absorb stormwater runoff using natural green solutions – urban forests, disconnected impervious areas, swales, and larger bio-retention areas. As the density increases to the more urban Transects (urban center, urban core, special urban) the options to use Green Infrastructure is reduced to those elements that have a small footprint or none at all. In these cases volume-based stormwater designs make use of infiltration trenches, porous pavement, cisterns, urban bioretention, green roofs, etc.

5.4.2 Implementing GSPs

The GSPs referenced throughout this chapter are located in the appendices specified below. Additional guidance on GSP selection is provided herein.

Appendix A – Intrinsic Green Stormwater Practice Specifications includes intrinsic GSPs. These GSPs should be considered in the initial stages of site design to allow planning that incorporates the best suited practices for intrinsic volume control and water quality benefits to the site. Specification sheets for each intrinsic practice include the description, stormwater functions and calculations, maintenance, cost, and construction guidance, if applicable.

Appendix B – Structural and Non-Structural Green Stormwater Practice Specifications includes twelve of the most common GSPs. A summary of these are shown in Tables 5.10 and 5.11 below. These tables are included to facilitate selection of the most appropriate practices for a given situation. Specification sheets for each practice provide a brief introduction to the practice, details on performance, suitability, limitations, and maintenance requirements. In addition, each practice is assigned a percentage of volume removal based on the design level selected and the particular GSP's ability to manage the first inch of runoff volume from a storm. The total runoff reduction percentage goal for a site is 80%, as explained in Section 5.3.

Table 5.10. Effectiveness of GSPs in meeting stormwater management objectives.

GSP	Volume	Peak Discharge	Water Quality
Bioretention	●	●	●
Tree Planters/Urban Bioretention	◉	◉	●
Permeable Pavement	●	●	◉
Infiltration	●	●	●
Water Quality Swale	◉	◉	●
Extended Detention Basins	○	●	○
Downspout Disconnection	◉	◉	◉
Grass Channels	○	○	○
Sheet Flow	●	●	◉
Reforestation	●	●	●
Rain Tanks/Cisterns[1]	◉	○	○
Green Roofs	◉	●	●

1. A single cistern typically provides greater volume reduction than a single rain tank.

● High effectiveness.
◉ Medium effectiveness.
○ Low effectiveness.

Rankings are qualitative. "High effectiveness" means that one of the primary functions of the GSP is to meet the objective. "Medium effectiveness" means that a GSP can partially meet the objective but should be used in conjunction with other BMPs. "Low effectiveness" means that the contribution of the GSP to the objective is a byproduct of its other functions, and another decentralized control should be used if that objective is important.

Table 5.11. Green stormwater practice lands use and land area selection matrix.									
GSP	**Criteria**								
	Land Use								**Land Area Required**
	Schools	**Comm'l**	**Indust.**	**Single Family Resid.**	**Multi-Family Resid.**	**Parks/Open Space**	**City ROW/ Roadside**	**Utility Easements**	
Bioretention	●	●	N	●	●	●	●	N	⦿
Tree Planters/Urban Bioretention	⦿	●	N	N	●	●	●	N	○
Permeable Pavement	●	●	⦿	●	●	●	●	N	○
Infiltration	●	●	N	●	●	●	⦿	N	○
Water Quality Swale	●	●	N	N	●	●	●	⦿	⦿
Extended Detention Basins	●	●	●	N	●	⦿	●	●	○
Downspout Disconnection	●	⦿	N	●	●	●	N	N	○
Grass Channels	●	●	N	●	●	⦿	●	●	⦿
Sheet Flow	●	●	N	●	●	⦿	●	●	⦿
Reforestation	⦿	N	⦿	⦿	⦿	●	●	N	○/●
Rain Tanks/Cisterns	●	⦿	⦿	●	●	N	N	⦿	○
Green Roofs	●	●	●	N	●	N	N	N	○

● Well suited for land use applications or relatively high dedicated land area required.
⦿ Average suitability for land use applications or relatively moderate dedicated land area required.
○ Relatively low dedicated land area required.
N Not typically applicable for land use.

SECTION 5.5. REFERENCES

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CHAPTER 6. STORM DRAINAGE SYSTEM DESIGN

SECTION 6.1. STORMWATER DRAINAGE DESIGN OVERVIEW

6.1.1 Stormwater Drainage System Design

6.1.1.1 Drainage System Components

In the City of Fayetteville, the drainage system may be classified as the minor system and the major system. Three considerations largely shape the design of both these systems: flooding, public safety and water quality.

The minor drainage system is designed to remove stormwater from areas such as streets and sidewalks for public safety reasons. The minor drainage system consists of inlets, street and roadway gutters, roadside ditches, small channels and swales, and small underground pipe systems which collect stormwater runoff and transport it to structural control facilities, pervious areas and/or the major drainage system (i.e., natural waterways, large man-made conduits, and large water impoundments).

Paths taken by runoff from very large storms are called major systems. The major system (designed for the less frequent storm up to the 100-year event) consists of natural waterways, large man-made conduits, and large water impoundments. In addition, the major system includes some less obvious drainageways such as overland relief swales and infrequent temporary ponding areas. The major system includes not only the trunk line system that receives the water from the minor system, but also the natural backup system which functions in case of overflow from or failure of the minor system. Overland drainage must be designed to protect houses, buildings or other property from flooding.

The major/minor concept may be described as a 'system within a system' for it comprises two distinct but conjunctive drainage networks. The major and minor systems are closely interrelated, and their design needs to be done in tandem and in conjunction with the design of structural stormwater controls and the overall stormwater management concept and plan (Section 6.2).

This chapter provides design criteria and guidance on drainage system components, including street and roadway gutters, inlets and storm drain pipe systems (Section 6.2); culverts and bridges (Section 6.3); vegetated and lined open channels (Section 6.4); and energy dissipation devices for outlet protection (Section 6.5). The rest of this section covers important considerations to keep in mind in the planning and design of stormwater drainage facilities.

6.1.1.2 Checklist for Drainage Planning and Design

The following is a general procedure for drainage system design on a development site.

1. Analyze topography
    a. Check drainage pattern.
    b. Check on-site topography for surface runoff and storage, and infiltration
        i. Determine runoff pattern; high points, ridges, valleys, streams, and swales.

   ii. Overlay the grading plan, delineate watershed areas; calculate square footage (acreage), points of concentration, low points, etc.

   c. Check potential drainage outlets and methods

      i. On-site (structural control, receiving water)

      ii. Off-site capacity (highway, storm drain, receiving water, regional control)

      iii. Natural drainage system capacity (swales)

      iv. Existing drainage system capacity (drain pipe)

2. Analyze other site conditions.

   a. Land use and physical obstructions such as walks, drives, parking, patios, landscape edging, fencing, grassed area, landscaped area, tree roots, etc.

   b. Soil type determines the amount of water that can be absorbed by the soil. Areas of fill will be considered Hydrologic Soil Group D.

   c. Vegetative cover will control the erosion potential of drainage paths.

      i. Analyze areas for probable location of drainage structures and facilities.

      ii. Identify the type and size of drainage system components that are required. Design the drainage system and integrate with the overall stormwater management system and plan.

If it is determined that offsite drainage improvements are required, 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 or her own expense, (b) providing retention to match pre-development downstream discharges, or (c) delaying the project until the City is able to share in the offsite costs.

6.1.2 Design Considerations

6.1.2.1 General Drainage Design Considerations and Requirements

- Stormwater systems should be planned and designed to generally conform to natural drainage patterns and discharge to natural drainage paths within a drainage basin. These natural drainage paths should be modified as necessary to contain and safely convey the peak flows generated by the development.
- Runoff must be discharged in a manner that will minimize adverse impacts on downstream properties or stormwater systems. In general, runoff from development sites within a drainage basin should be discharged at the existing natural drainage outlet or outlets. Existing drainage problems or flooding at or adjacent to the project site should be identified by the downstream assessment to be performed as part of the drainage report. Offsite improvements may be required at the discretion of the City Engineer.
- It is important to ensure that the combined minor and major system can handle flows in excess of the design capacity to minimize the likelihood of nuisance flooding or damage to private properties. If failure of minor systems and/or major structures occurs during these periods, the risk to life and

property could be significantly increased. If there are already existing drainage problems or historical flooding issues, downstream assessments should be performed as provided in Chapter 2, or as required by the City Engineer.

- Design storm requirements for various components of the minor and major drainage systems are provided below, in the applicable sections. The full build-out 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.

6.1.2.2 Inlets and Drains

Inlets should be located where they will not compromise safety or aesthetics or allow standing water in areas of vehicular or pedestrian traffic, but they should take advantage of natural depression storage where possible.

6.1.2.3 Storm Drain Pipe Systems (Storm Sewers)

Includes storm drainage systems and pipe network that convey runoff in streets, parking lots (public or private), public right of ways and drainage easements or where permitted by the city engineer.

- 10-year design storm (for pipe design)
- 10-year design storm (for on-grade inlet)
- 10-year design storm (for sumped inlet)

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.

6.1.2.4 Open Channels

Open channels include all channels, swales, etc.

- Ten-year design storm, evaluate and provide that 100-year design storm is contained within public right of ways and/or drainage easements. Use backwater from receiving channel for the same design event for checks (i.e., 10-year and 100-year respectively).
- Meander bends, if used, must be designed to accommodate increased shear stress.
- Check that design flow velocity does not exceed the maximum velocity for channel lining proposed.

Channels may be designed with multiple stages (e.g., a low flow channel section containing the 2-year to 5-year flows, and a high flow section that contains the design discharge) to improve stability and better mimic natural channel dimensions. Where flow easements can be obtained and structures kept clear, overbank areas shall be designed as part of a conveyance system wherein floodplain areas are designed for storage and/or conveyance of larger storms. The 100-year design storm shall be calculated and demonstrated to be contained within public right of ways and/or drainage easements.

6.1.2.5 Energy Dissipaters

Design energy dissipation as needed to return flows to non-erosive velocities at and immediately downstream of the termination of project improvements. Includes all outlet protection facilities.

- 10-year design storm, evaluate for 100-year design storm

SECTION 6.2. MINOR DRAINAGE SYSTEM DESIGN

6.2.1 Introduction

Minor stormwater drainage systems quickly remove runoff from areas such as streets and sidewalks for public safety purposes. The minor 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, pervious areas and/or the major drainage system (i.e., natural waterways, large man-made conduits, and large water impoundments).

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

- Street and roadway gutters, parking lots
- Stormwater inlets
- Storm drain pipe systems

Ditch, channel and swale design criteria and guidance are covered in Section 6.4, *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-12 (USDOT, FHWA, 1984). Storm drain 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.

6.2.1.1 General Criteria

<u>Criteria for Public Streets and Parking Lots</u>
Inlets shall be installed at low points and at such intervals to provide the appropriate clear traffic lane per street classification in each direction and for maximum ponding depths based upon peak discharges from the 10-year design storm and the 100-year check storm. Minimum lane clearance and maximum ponding depth requirements are provided in Table 6.1. All computations for the 10-year and 100-year, 24-hour storm shall be provided.

Table 6.1. Flow spread limits & ponding depths for inlets – streets and parking (10-year design storm).			100-year check
Roadway Classification	**Minimum clear space**	**Maximum ponding depth**	**Maximum ponding depth**
Principal and Arterial Streets	Two 12-feet traffic lanes, one in each direction, independent of curb and gutter	0.5 feet	1.0 feet
Collector Streets	One 12-feet traffic lane within 6 feet of roadway centerline	0.5 feet	1.0 feet
Local and Residential Street	One 8-feet traffic lane within 4 feet of roadway centerline	0.5 feet	1.5 feet
Parking Lots	One 8-feet traffic lane to points of egress	0.5 feet	1.0 feet

Gutter design constraints include:

- Per City standards, street and gutter cross slope shall match.
- Minimum road and gutter cross slope (S_X)= 0.005 ft/ft.
- Minimum longitudinal slope (S)= 0.005 ft/ft.
- Standard gutter width = 1.0 ft.

6.2.2 Bypass Flow

Bypass flow occurs when storm sewer inlets do not capture 100% of the flow upstream of their location. A variety of factors, including 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 may result in a more economical drainage system. Refer to Sections 6.2.5 – 6.2.8 for inlet design.

6.2.3 Symbols and Definitions

Use the symbols listed in Table 6.2 to provide consistency within this section as well as throughout this Manual. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 6.2. Symbols and definitions.		
Symbol	**Definition**	**Units**
A	Gutter depression	in
A	Area of cross section	ft^2
d or D	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow Qw/Q	-
g	Acceleration due to gravity (32.2 ft/s^2)	ft/s^2
h	Height of curb opening inlet	ft
H	Head loss	ft
K	Loss coefficient	-
L or LT	Length of curb opening inlet	ft
L	Pipe length	ft
n	Roughness coefficient in the modified Manning's formula for triangular gutter flow	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
Q	Rate of discharge in gutter	cfs
Q_b	Rate of bypass flow	cfs
Q_i	Intercepted flow	cfs
Q_s	Gutter capacity above the depressed section	cfs
S or S_x	Cross Slope - Traverse slope	ft/ft
S or SL	Longitudinal slope of pavement	ft/ft
S_f	Friction slope	ft/ft
S'w	Depression section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
T_s	Spread above depressed section	ft
V	Velocity of flow	ft/s
W	Width of depression for curb opening inlets	ft
Z	T/d, reciprocal of the cross slope	-

6.2.4 Street and Roadway Gutters

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. Reference the Manning's "n" values in Table 6.9, in Section 6.3 for appropriate values.

6.2.4.1 Design Procedure

(Step 1): Determine maximum allowable flow before an inlet is required, based upon the street classification and physical parameters of the proposed design. Flatter grades upstream of the proposed inlet must also be considered to ensure higher discharges do not occur elsewhere in the system.

(Step 2): Identify required sump locations based on maximum allowable discharges and site conditions.

(Step 3): Determine actual discharges (for the design and check storms) for each inlet by delineating the drainage area, determining the rational coefficient, and calculating the time of concentration.

(Step 4): Determine inlet capacity and gutter capacity in each direction, repeat process as needed based upon capacity and maximum allowable spread.

Condition 1: Compute spread, given gutter flow.
Establish longitudinal slope (S), cross slope (S_x), gutter flow (Q), and Manning's n. Input parameters to calculate gutter spread.

Condition 2: Compute gutter flow, given spread.
Establish longitudinal slope (S), cross slope (S_x), spread (T), and Manning's n. Input parameters to calculate gutter flow.

Below is a sample output file based on Hydraflow Express computer software, applied for Condition 2.

Table 6.3. Sample gutter report output file.

Gutter Channel Section			
Gutter Section Data		Highlighted Parameters	
Cross Sl, Sx (ft/ft)*	0.030	Depth (ft)	0.18
Cross Sl, Sw (ft/ft)*	0.030	Q (cfs)	2.769
Gutter Width (ft)	1.00	Velocity (ft/s)	5.13
Invert Elevation (ft)	100.00	Spread Width (ft)	6.00
Slope (%)	5.20		
N-Value	0.013		
Calculations			
Compute by:	Known Depth		
Known Depth (ft)	0.18		

*Gutter slope and road cross slope are 0.5% (minimum) per City standards.

6.2.5 Stormwater Inlets

Inlets are drainage structures used to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts.

Inlets used for the drainage of roadway surfaces can be divided into three major classes:

- Curb Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Grate Inlets – These drop inlets consist of an opening in the gutter covered by one or more grates.
- Combination Inlets – These inlets usually consist of both a curb opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening must be located in part upstream of the grate.

Inlets may be classified as being on a continuous grade or in a sump. 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.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets, or extensions, on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

Curb inlet design is discussed in Section 6.2.5, grate inlet design in Section 6.2.6, and combination inlets in Section 6.2.7.

6.2.6 Curb Inlet Design

6.2.6.1 Curb 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 Fayetteville standard drawings and details shall be used.

Inlets with extensions shall have a maximum clear opening dimension of 18-feet. This length represents a 4-foot inlet opening with two 8-feet extensions (four openings with clear opening dimensions of 3 feet, 6 inch 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.

6.2.6.2 Curb Inlets in Sump

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height (6-inch standard) and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage. A 20% clogging factor shall be applied for curb inlets in sump.

6.2.6.3 Design Steps

(Step 1) Determine the following inputs and constraints:

Cross slope = S_X (ft/ft)   Longitudinal slope = S (ft/ft)

Gutter flow rate = Q (cfs)   Manning's n = n

Maximum Spread of water on pavement = T (ft)

(Step 2) Assume an initial inlet geometry and apply clogging factor (where applicable):
Apply inlet dimensional requirements for the City of Fayetteville. The gutter shall be depressed 4 inches. No clogging factor for inlets on grade, 20% clogging factor for inlets in a sump.

(Step 3) Compute inlet ponding depth and spread of water on pavement (T, ft) for design and check storms. Check gutter spread of approach each direction. Repeat process as needed by adding or removing throat extensions or additional inlets upstream based upon capacity and maximum allowable spread.

Table 6.4 below depicts input and output for a sample inlet, based on Hydraflow Express computer software.

Table 6.4. Inlet report output file.

Sample Inlet Report

													Output					
Inlet ID	Q=CIA (cfs)	Q carry (cfs)	Q capt (cfs)	Q Byp (cfs)	Junc Type	Curb Inlet		Gutter							Inlet			Byp Line No
						Ht (in)	L (ft)	So (ft/ft)	W (ft)	Sw (ft/ft)	Sx (ft/ft)	N	Depth (ft)	Spread (ft)	Dpt (ft)	Sprd (ft)	Dep (in)	
Inlet A	4.02	0.00	4.02	0.00	Curb	2.0	8.80*	Sag	1.00	0.030	0.030	0.013	0.30	9.96	0.63	9.96	4.0	Off
Inlet B	4.76	0.49	5.25	0.00	Curb	2.0	8.80*	Sag	1.00	0.030	0.030	0.013	0.36	11.9	0.69	11.9	4.0	Off
Inlet C	1.77	0.00	1.75	0.17	Curb	2.0	11.00	0.052	1.00	0.030	0.030	0.013	0.14	4.73	0.36	0.99	4.0	2
Inlet D	1.25	0.00	1.22	0.32	Curb	2.0	7.50	0.032	1.00	0.030	0.030	0.013	0.14	4.57	0.35	0.96	4.0	2

*L adjusted based on City requirement of 20% clogging factor for inlets in a sump.

6.2.7 Grate Inlet Design

6.2.7.1 Grate Inlets on Grade

Grate inlets (when approved by the City Engineer) must be bicycle safe and adequately support traffic with appropriate frames provided. Grates shall only be considered where drop inlets cannot function. A 20% clogging factor shall be used for grate inlets on grade.

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs.

Bicycle-safe grates shall be used without exception. The curved vane grate or the tilt bar grate may be considered, for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction at installation. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was made to qualitatively simulate field conditions.

6.2.7.2 Grate Inlets in Sag

A 50% clogging factor shall be used, and grate width shall not extend beyond gutter (1 foot). A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.4 feet above the top of grate and when depth of water exceeds about 1.4 foot, the grate begins to operate as an orifice. Between depths of about 0.4 foot and about 1.4 foot, a transition from weir to orifice flow occurs. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or the orifice equation. Typical design software will account for this transition zone during computations.

The capacity of grate inlets (on-grade or in sag) operating as a weir is:

$$\mathbf{Q_i = CPd^{1.5}} \qquad \textbf{Eq. 6.1}$$

Where: P = perimeter of grate excluding bar widths and the side against the curb, feet

C = 3.0 (weir coefficent)

d = depth of water above grate, feet

and as an orifice is:

$$\mathbf{Q_i = CA(2gd)^{0.5}} \qquad \textbf{Eq. 6.2}$$

Where: C = 0.67 orifice coefficient

A = clear opening area (do not include area of bars) of the grate, $feet^2$

g = 32.2 ft/s^2

The tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in a sag where ponding can occur, and flanking inlets on the low gradient approaches. Grates shall not be used in sump conditions unless flanked by deep inlets as a combination inlet.

6.2.8 Combination Inlets

6.2.8.1 Combination Inlets On Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb opening inlet for the design procedures. A combination inlet on grade is required on gutter slopes of 7% or greater. For steep slopes, the grate should be located at the uphill side of the extension, not at the box.

6.2.8.2 Combination Inlets In Sump

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity.

6.2.9 Storm Drain Pipe Systems

6.2.9.1 Introduction

Storm drain pipe systems, also known as *storm sewers*, are pipe conveyances used in the minor 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. An example of storm sewer output data to include in a final drainage report is provided as Exhibit 1 to Appendix H of this manual.

6.2.9.2 Requirements and General Design Procedure

All computations and hydraulic profiles for the 10-year and 100-year, 24 hour storms shall be provided.

The design of storm drain systems generally follows these steps:

(Step 1) Determine inlet location and spacing.

(Step 2) Determine drainage areas and compute runoff.

(Step 3) Prepare a tentative plan layout of the storm sewer drainage system including:

- Location of storm drains
- Direction of flow
- Location of manholes
- Location of existing facilities such as water, gas, or underground cables

(Step 4) After the tentative locations of inlets, drain pipes, and outfalls (including tailwater elevations) have been determined and the inlets sized, compute of the rate of discharge to be carried by each storm drain pipe and determine the size and gradient of pipe required to convey this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the pipe serving that discharge is sized, and the process is repeated for the next run downstream.

(Step 5) Examine assumptions to determine if adjustments are needed to the final design.

It should be recognized that the rate of discharge to be carried by any particular section of storm drain pipe is not typically the sum of the inlet design discharge rates of all inlets above that section of pipe, but is typically somewhat less than this total due to attenuation of peaks caused by variations in the timing of the peak discharges. As the time of concentration grows larger, the appropriate rainfall intensity to be used in the design grows smaller.

6.2.9.3 Design Criteria

Storm drain pipe systems should conform to the following criteria:

- Storm drain pipes shall be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. Pipes shall be designed to have crowns matched and shall not discharge into a smaller pipe. The Manning Formula is recommended for capacity calculations. The hydraulic grade line for the system should be contained in the pipe and below the crown for the 10-year storm.
- The minimum circular diameter for any public storm drain pipe is 18 inches.
- The maximum hydraulic gradient shall not produce a velocity that exceeds 20 feet/s for the design storm.
- The minimum desirable physical slope shall be 0.5% or the slope that will produce a minimum velocity of 2.5 ft/s for the design storm when the storm sewer is flowing full, whichever is greater.
- Any storm drain pipe located in a right of way or drainage easement shall be reinforced concrete pipe (RCP) unless approved by the City Engineer.
- The water surface elevation shall be at least 1 foot below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest. Where required, adjustments shall be made in the system to reduce the elevation of the hydraulic grade line to meet this requirement.
- The 100-year storm shall be used as the check storm. Combined capacity of the street and minor systems must be equal to or greater than the peak rate of flow for the 100-year storm. Apply maximum ponding depths from Table 6.1.

6.2.9.4 Capacity Calculations

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning's Formula, expressed by the following equation:

$$\mathbf{V = [1.486\, R^{2/3} S^{1/2}]/n} \qquad \textbf{Eq. 6.3}$$

Where: V = mean velocity of flow, ft/s

R = the hydraulic radius, ft - defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)

S = the slope of hydraulic grade line, ft/ft

n = Manning's roughness coefficient (see table 6.9 for values)

In terms of discharge, the above formula becomes:

$$\mathbf{Q = [1.486\, AR^{2/3} S^{1/2}]/n} \qquad \textbf{Eq. 6.4}$$

Where: Q = rate of flow, cfs

A = cross sectional area of flow, ft^2

For pipes flowing full, the above equations become:

$$V = [0.590\, D^{2/3} S^{1/2}]/n \qquad \textbf{Eq. 6.5}$$

$$Q = [0.463\, D^{8/3} S^{1/2}]/n \qquad \textbf{Eq. 6.6}$$

Where: D = diameter of pipe, ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$H_f = [2.87\, n^2 V^2 L]/[S^{4/3}] \qquad \textbf{Eq. 6.7}$$

$$H_f = [29\, n^2 V^2 L]/[(R^{4/3})\,(2g)] \qquad \textbf{Eq. 6.8}$$

Where: H_f = total head loss due to friction, ft

n = Manning's roughness coefficient

D = diameter of pipe, ft

L = length of pipe, ft

V = mean velocity, ft/s

R = hydraulic radius, ft

g = acceleration of gravity = 32.2 ft/s^2

6.2.9.5 Hydraulic Grade Lines

All head losses in a storm sewer system 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.

Hydraulic control is a set water surface elevations from which the hydraulic calculations are begun. All hydraulic controls along the alignment are established. If the control is at a main line upstream inlet (inlet control), the hydraulic grade line is the water surface elevation minus the entrance loss minus the difference in velocity head. If the control is at the outlet, the water surface is the outlet pipe hydraulic grade line.

6.2.9.6 Junctions and Manholes

Manholes, junction boxes or maintenance access ports will be required for public storm drain systems whenever there is a change in size, direction, elevation, grade, or where there is a junction of two or more sewers. The maximum spacing between manholes and manhole diameter for various pipe sizes shall be in accordance with Table 6.5. Wye connections can be used up to and including 24 inches x 24 inches. Wye connections larger than 24 inches x 24 inches may be used with the approval of the City Engineer.

Table 6.5. Manhole sizes and spacing.	
Manhole Sizes	
Storm Sewer Diameter	**Manhole Diameter**
15 inches to 18 inches	4 feet
24 inches to 42 inches	5 feet
48 inches to 54 inches	6 feet
60 inches and larger	To be approved by City
Manhole Spacing	
Storm Sewer Diameter	**Maximum Allowable Spacing**
15 inches to 36 inches	400 feet
42 inches and larger	500 feet

6.2.9.7 Minimum Grade

All storm drains should be designed such that velocities of flow will not be less than 2.5 feet/s at design flow or lower, with a minimum slope of 0.5%. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. Upper reaches of a storm drain system should have flatter slopes than slopes of lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deter settling of particles.

The minimum slopes are calculated by the modified Manning's formula:

$$S = [(nV)^2]/[2.208R^{4/3}] \qquad \textbf{Eq. 6.9}$$

Where:
S = the slope of the hydraulic grade line, ft/ft
n = Manning's roughness coefficient
V = mean velocity of flow, ft/s
R = hydraulic radius, ft (area divided by wetted perimeter)

SECTION 6.3. CULVERT and BRIDGE DESIGN

Criteria for Culverts

Culvert design shall be based upon peak discharges for the appropriate design storm based on roadway type. Requirements are provided in Table 6.6. All computations, hydraulic profile, and energy transition to channel shall be provided for the design event and the 100-year check storm.

Table 6.6. Culvert and bridge sizing requirements based on roadway type.[1]			
Roadway Classification	**Design Storm Event**	**Minimum Freeboard[2] (Culvert)**	**Minimum Freeboard[2] (Bridge)**
Principal and Minor Arterial Streets	50-year (2 %-annual-chance)	2 feet	1 foot
Collector Streets	25-year (4 %-annual-chance)	2 feet	1 foot
Local and Residential Streets (all others)	10-year (10 %-annual-chance)	2 feet	1 foot

1. Additional limitations apply to culverts or bridges along City of Fayetteville Protected Streams and within FEMA Regulatory Flood Hazard Areas
2. Freeboard for culverts shall be from top of low point in road. Freeboard for bridges shall be measured from low chord.

Route the 100-year frequency storm through all culverts to be sure building structures (e.g., houses, commercial buildings) are not flooded or increased damage does not occur to the roadway or adjacent property for this design event. The flow shall be safely conveyed through drainage easements and/or the ROW. Use appropriate tailwater conditions, assuming the 100-year event in receiving waters.

6.3.1 Overview

A *culvert* is a short conduit that conveys stormwater runoff under an embankment, usually a roadway or driveway. The primary purpose of a culvert is to convey surface water. In addition to the hydraulic function, a culvert must also support the embankment and/or roadway, and protect traffic and adjacent property owners from flood hazards to the extent practicable.

6.3.2 Protected Streams

New stream crossings including driveways, roadways, trails, or railroads, are allowed on City of Fayetteville Protected Streams when the City Engineer determines there is no practical and feasible alternative. The Protected Streams map, located on the City of Fayetteville website, may be accessed at:
http://www.accessfayetteville.org/government/strategic_planning/documents/general_documents/Streamside_Protection_Map_2_01_11.pdf

The following criteria apply:

- Minimize or reduce stream crossings through proper planning,
- Minimize the amount of excavation and filling,
- Maintain the dimension, pattern, and profile of the stream.
- Minimize scour, erosion and flooding.

Methods to minimize stream crossing impacts include:

- Construct stream crossings during periods of low flow.
- Locate crossings where streambed and banks are composed of firm, cohesive soils to minimize erosion.

- Design crossings to reduce the possibility of obstructions such as debris and silt blockages through the minimization of channel obstructions.
- Bridges and bottomless arches, wide enough to span the stream and allow for some dry ground or an artificial ledge beneath the bridge on one or both sides are preferred and should be used whenever possible.
- Bridge soffits should be a minimum of 1 foot above the height of adjacent banks--high enough to allow wildlife passage.
- Exceptionally wide stream crossings may be allowed to utilize piers in the channel under the discretion of the City Engineer.
- Maintain a natural substrate underneath the bridge. If concrete is necessary to prevent scour, then it is recommended to cover the concrete with a natural substrate.
- All disturbed areas shall be revegetated immediately upon completion of the work.

The use of culverts on protected streams should be avoided. If culverts must be used, the following installation guidelines should be followed:

- Provide water depths and velocities (at low flows) matching natural areas upstream and downstream of the crossing.
- Create no drop-offs or plunge pools and no constriction of the channel.

The practices listed may be subject to additional regulation per UDC Chapter 168 Flood Damage Prevention Code, Chapter 169 Physical Alteration of Land, and Chapter 170 Stormwater Management, Drainage and Erosion Control.

6.3.3 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual the symbols listed in Table 6.7 will be used. These symbols were selected because of their wide use.

Table 6.7 Symbols and definitions.		
Symbol	Definition	Units
A	Area of cross section of flow	ft^2
B	Barrel width	ft
C_d	Overtopping discharge coefficient	-
D	Culvert diameter or barrel depth	inches or ft
d	Depth of flow	ft
d_c	Critical depth of flow	ft
d_u	Uniform depth of flow	ft
g	Acceleration of gravity	ft/s
H_f	Depth of pool or head, above the face section of invert	ft
h_o	Height of hydraulic grade line above outlet invert	ft
HW	Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)	ft
k_e	Inlet loss coefficient	-
L	Length of culvert	ft
N	Number of barrels	-
Q	Rate of discharge	cfs
S	Slope of culvert	ft/f
TW	Tailwater depth above invert of culvert	ft
V	Mean velocity of flow	ft/s
V_c	Critical velocity	ft/s

6.3.4 Design Criteria

6.3.4.1 Velocity Limitations

Consider minimum and maximum velocities when designing a culvert. The maximum allowable velocity for reinforced concrete pipe is 20 ft/s. To ensure self-cleaning during partial depth flow, a minimum velocity of 2.5 ft/s for the 2-year flow, when the culvert is flowing partially full, is required.

6.3.4.2 Length and Slope

The maximum culvert slope using concrete pipe shall be 10%, while the minimum slope for standard construction procedures shall be 0.4% when possible. Maximum drop in a drainage structure or junction box is 10 feet.

6.3.4.3 Headwater Limitations

Headwater is the water above the culvert invert at the entrance end of the culvert. The maximum allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/or the roadway and is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. It is this allowable headwater depth that is the primary basis for sizing a culvert.

The following criteria relates to headwater:

- 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 minimal impact on upstream property,
- Maximum headwater depth for design storm shall be 2 feet lower than top of road or curb,
- Ponding depth shall be no greater than the elevation where flow diverts around the culvert,
- For drainage facilities with cross-sectional area equal to or less than 30 feet^2, HW/D should be equal to or less than 1.5,
- For drainage facilities with cross-sectional area greater than 30 ft^2, 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 major thoroughfares with 18-inch freeboard from the low point of the road,
- Identify the maximum acceptable outlet velocity, based on receiving channel conditions and Tables 6.12 and 6.13,
- Either set the headwater to produce acceptable velocities, or use stabilization or energy dissipation where acceptable velocities are exceeded,
- The constraint that gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.
- Bridges require 1-feet freeboard from the low chord.

6.3.4.4 Tailwater Considerations

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

- If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic gradeline shall be determined.
- For culverts that discharge to an open channel, the stage-discharge curve for the channel must be determined. The water surface elevation in the open channel for the relevant design storm event for the culvert should be evaluated as part of culvert capacity computations. See Section 6.4, *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 particular water body may establish the culvert tailwater.

6.3.4.5 Culvert Inlets

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient K_e, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 6.8.

6.3.4.6 Inlets with Headwalls

Concrete headwalls or equivalent end treatments are required for all culverts installed in public right of ways or drainage easements. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, provide a channel apron at the toe of the headwall. Extend the apron at least one pipe diameter upstream from the entrance. The top of apron elevation shall not protrude above the normal streambed elevation.

6.3.4.7 Wingwalls and Aprons

Wingwalls are required where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow. Aprons shall be applied where required to prevent scour.

6.3.4.8 Material Selection

If material other than reinforced concrete pipe (RCP) is to be used in roadway areas including under curbs, it shall be approved by the City Engineer. Galvanized CMP is not accepted. Coated corrugated metal pipe (CMP) and high density polyethylene pipe (HDPE) may be used in non-roadway areas. Coated CMP and HDPE flared end sections are prohibited within the right of way and drainage easements. All pipe shall be installed to manufacturer's recommendations including bedding, backfill and compaction. Bedding, backfill and compaction details must be included in the construction plans.

6.3.4.9 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval by the City Engineer.

6.3.4.10 Culvert Sizes

The minimum allowable circular pipe diameter shall be 15 inches for culverts.

6.3.4.11 Outlet Protection

See Section 6.5 for information on the design of outlet protection. Outlet protection shall be provided for the 10-year storm event.

Table 6.8. Inlet coefficients.			
Type of Structure and Design of Entrance			**Coefficient K_e[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, 33.7° or 45 ° 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 ° or 45 ° 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° to 75° 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° or 25° to barrel		
		Square-edged at crown	0.5
	Wingwalls parallel (extension of sides) Square-edged at crown		0.7
	Side- or slope-tapered inlet		0.2

1. The K_e values for corrugated metal pipes are also recommended for HDPE pipes.

* Note: End Section conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Source: HDS No. 5, 1985.

Table 6.9. Manning's n values.		
Type of structure, material and joint description[1,2]		**Manning's n**
Gutter or Pavement		
	Concrete gutter, smooth	0.013
	Concrete gutter, rough	0.016
	Concrete gutter & asphalt pavement	0.014
	Concrete gutter & pavement, float finish	0.014
	Concrete gutter & pavement, broom finish	0.016
Pipe, Concrete		0.013
Concrete Box		0.013
Corrugated Metal Pipes and Annular Corrugations[3]		
	2 2/3- by ½-inch corrugations (unpaved / paved)	0.024/0.012
	6- by 1-inch corrugation (unpaved / paved)	0.025/0.012
	3- by 1-inch corrugations (unpaved / paved)	0.027/0.012
	6-by 2-inch structural plate	0.033
	9-by 2-1/2 inch structural plate	0.035
Pipes, Helical	24-inch plate width	0.012
Spiral Rib Metal Pipe	3/4 by 3/4 inch recesses at 12 inch spacing, good joint	0.013
High Density Polyethylene (HDPE)	Corrugated Smooth Liner	0.012

1. Source: HDS No. 5 (1985)
2. Estimates are by or based on the Federal Highway Administration, Source: USDOT, FHWA, HDS No. 3 (1961).
3. Source: Modern Sewer Design (1999)

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163 (HDS-5).

6.3.5 Design Procedures

6.3.5.1 Types of Flow Control

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

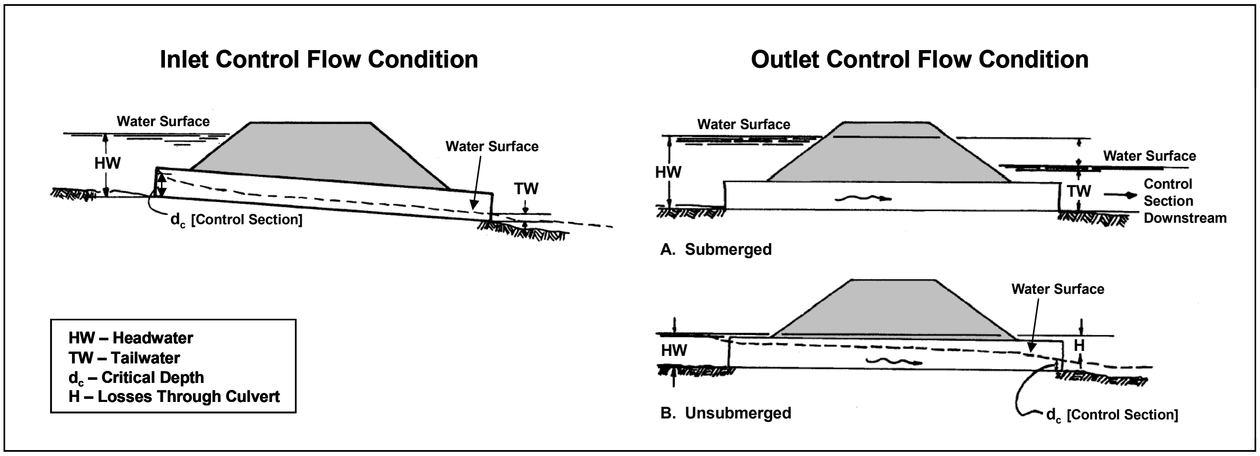


Figure 6.1. Culvert flow conditions (Adapted from: HDS-5, 1985).

Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet, such as when the slope is steep. In this condition the control section of a culvert is just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical.

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

Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control, see HDS-5.

6.3.5.2 Procedures

A list of acceptable software is provided in Appendix H, Stormwater Software. Additional software may be accepted for use by the City Engineer provided it is shown to be equivalent to approved softwares.

6.3.5.3 Design Procedure

The following design procedure requires the use of computer design software.

(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 for design storm (ft)

(Step 2) Determine trial culvert open area by assuming a trial velocity 3 to 5 ft/s and computing the culvert area, A = Q/V. Determine the culvert shape, open size (diameter or span and rise), and number of barrels.

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

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

$$\mathbf{HW = H + h_o - LS} \qquad \textbf{Eq. 6.10}$$

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

L = culvert length

S = culvert slope

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

- If inlet control governs, then the design is complete and no further analysis is required.
- If outlet control governs and the HW is unacceptable, select a larger trial size and repeat steps. Unless material or entrance conditions change, the inlet control conditions for the larger pipe need not be re-checked.

(Step 5) Calculate exit velocity and, if erosion problems are expected, modify culvert size to reduce or eliminate erosion problems. If not achievable, refer to Section 6.5 for appropriate energy dissipation designs.

Below is a sample output file using Hydraflow Express computer software.

Table 6.10. Sample culvert output file.

Sample Culvert			
Culvert Data:		**Calculations:**	
Invert Elevation Down (ft)	100.00	Qmin (cfs)	13.00
Pipe Length (ft)	80.00	Qmax (cfs)	13.00
Slope (%)	0.60	Tailwater Elevation (ft)	(dc+D)/2
Invert Elevation Up (ft)	100.48		
Rise (in)	18.0	**Highlighted:**	
Shape	Circular	Qtotal (cfs)	13.00
Span (in)	18.0	Qpipe (cfs)	13.00
No. Barrels	1	Qovertop (cfs)	0.00
n-Value	0.013	Velocity Down (ft/s)	7.50
Culvert Type	Circular Concrete	Velocity Up (ft/s)	7.36
Culvert Entrance	Square edge w/headwall (C)	HGL Down (ft)	101.43
Coefficients[1] K,M,c,Y,k_e	0.0098,2,0.0398,0.67,0.5	HGL Up (ft)	102.60
		Hw Elevation (ft)	103.63
Embankment Data:		Hw/D (ft)	2.10
Top Elevation (ft)	105.00	Flow Regime	Inlet Control
Top Width (ft)	50.00		
Crest Width (ft)	30.00		

1. HDS-5 (coefficients K, M, c, Y are based on edge configurations).

6.3.5.4 Performance Curves - Roadway Overtopping

A performance curve for any culvert can be obtained by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert.

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

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

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

(Step 3) When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and Equation 6.11 to calculate flow rates across the roadway.

$$Q = C_d L (HW)^{1.5} \qquad \text{Eq. 6.11}$$

Where: Q = overtopping flow rate (ft^3/s)

C_d = overtopping discharge coefficient

L = length of roadway (ft)

HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (feet)

Note: Overtopping discharge coefficients may be obtained from a reference source or computed by the design software based on material and geometric configuration. Confirm appropriateness of discharge coefficients used. For more information, see HDS-5.

(Step 4) Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve.

6.3.5.5 Multibarrel Installations

For multibarrel installations 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, B, 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 multibarrel installations 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.

Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

SECTION 6.4. OPEN CHANNEL DESIGN

Criteria for Open Channels

Open channel design parameters shall be:

- 10-year storm frequency shall be used,
- Maintain minimum freeboard of 1 foot (for v_{max} less than 8 ft/s),
- Maintain minimum freeboard of 2 feet (for v_{max} greater than 8 ft/s) - see Section 6.4.4.3 for flow depth exceeding 5 feet,
- Maximum allowable channel velocities for earthen materials are provided in Table 6.12,
- Maximum allowable channel velocities for vegetative linings are provided in Table 6.13,
- Maximum allowable velocity for rigid-lined channel shall not exceed 20 feet/s,
- Minimum bend radius is 25 feet or bottom width multiplied by 10 (whichever is greater).

The 100-year storm shall be used for the check storm. Discharges from the 100-year event shall be safely conveyed through drainage easement or right-of-way.

6.4.1 Overview

6.4.1.1 Introduction

Open channel systems are an integral part of stormwater drainage design, particularly for development sites utilizing better site design practices and open channel structural controls. Open channels include drainage ditches, grass channels, dry and wet swales, riprap channels and concrete-lined channels. This section provides an overview of open channel design criteria and methods.

6.4.1.2 Considerations for Use of Open Channels

Open channels in major drainage systems have significant advantages in regard to cost and capacity. Disadvantages include increased right-of-way requirements, maintenance costs and habitat for insects.

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

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

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

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

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

Channel stability is a well recognized problem in urban hydrology because of general increases in low flows and peak storm discharges. A natural channel with increased capacity demands due to development should be augmented with necessary measures to avoid future bottom scour and bank cutting in accordance with Minimum Standard No. 2.

Sufficient right-of-way or permanent drainage easements shall be provided adjacent to open channels to allow entry of city maintenance vehicles. Typically these easements are 10-feet minimum or as approved by the City Engineer. Drainage easements may not be in the same location as utility easements. Where easements overlap, adequate dimensions shall be provided for both maintenance vehicle entry and utility structures.

6.4.2 Open Channel Types

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

<u>Vegetative Linings</u> – Vegetation, where practical, is the preferred lining for manmade channels. It stabilizes the channel body and bed, reduces erosion on the channel surface, and provides habitat and water quality benefits (see Chapters 4 and 5 for more details on using enhanced swales and grass channels for water quality purposes).

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

- High velocities
- Standing or continuously flowing water
- Lack of maintenance needed to prevent growth of taller or woody vegetation
- Lack of nutrients and inadequate topsoil
- Lack of access for maintenance, including the back of residential lots where fences are likely
- Excessive shade

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

Flexible Linings – 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.

Rigid Linings – 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 channel headcutting.

6.4.3 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 6.11 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 6.11. Symbols and definitions.		
Symbol	**Definition**	**Units**
α	Energy coefficient	-
A	Cross-sectional area	ft^2
b	Bottom width	ft
C_g	Specific weight correction factor	-
D or d	Depth of flow	ft
d	Stone diameter	ft
delta d	Superelevation of the water surface profile	ft
$d_{\#}$	Diameter of stone for which some percentage, by weight, of the gradation is finer	ft
E	Specific energy	ft
Fr	Froude Number	-
g	Acceleration of gravity	32.2 ft/s^2
h_{loss}	Head loss	ft
K	Channel conveyance	-
k_e	Eddy head loss coefficient	ft
K_T	Trapezoidal open channel conveyance factor	-
L	Length of channel	ft
L_p	Length of downstream protection	ft
n	Manning's roughness coefficient	-
P	Wetted perimeter	ft
Q	Discharge rate	cfs
R	Hydraulic radius of flow	ft
R_c	Mean radius of the bend	ft
S	Slope	ft/ft
SW_s	Specific weight of stone	lbs/ft^3
T	Top width of water surface	ft
V or v	Velocity of flow	ft/s
w	Stone weight	lbs
y_c	Critical depth	ft
y_n	Normal depth	ft
z	Critical flow section factor	-

6.4.4 Design Criteria

6.4.4.1 General 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 on the grading plan.

- Channel or adjacent public drainage easement, floodway, etc., shall be capable of carrying the 100-year storm. Adjacent public drainage easements shall contain the width of flow of channel, floodway, floodplain, etc., plus an additional 15 feet each side of the defined design of flood pool. For example, if the channel, floodway, or floodplain width is 50 feet wide, the drainage easement width at the same point will be 80 feet.
- Channel side slopes shall be designed to have a maximum slope of 3:1 to allow for maintenance, unless otherwise justified. Roadside ditches should have a maximum side slope of 3:1.
- Trapezoidal or parabolic cross sections are preferred.
- For channels with vegetative 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 Table 6.13.
- 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, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site. Disturbance of streambanks may be performed only in accordance with the City Streamside Protection Ordinance (Chapter 168.12 of Unified City Code).

6.4.4.2 Velocity Limitations

The final design of artificial 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 6.12. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 6.13. 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 6.12. Maximum velocities for comparing lining materials.

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

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

Table 6.13. 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.
Source: Manual for Erosion and Sediment Control in Georgia, 1996.

6.4.4.3 Channel Cross Section Requirements

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

1. Bend Radius: The minimum bend radius required for open channels is 25 feet or 10 times the bottom width, whichever is larger.
2. Freeboard: Freeboard shall be based on velocities associated with the design storm and shall be a minimum of l 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:

$$\text{Freeboard (ft)} = 1.0 + 0.025\, vD^{1/3} \qquad \text{Eq. 6.12}$$

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

D = depth of flow (ft)

Additional freeboard shall be computed for a channel with a sharp curve less than the minimum bend radius, as:

$$\mathbf{H = v^2\,((T + b)/2gR_c)} \qquad \textbf{Eq. 6.13}$$

Where: H = additional height on 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/s^2)

R_c = mean radius of bend (ft)

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

6.4.4.4 Channel Drops

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

6.4.4.5 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.

6.4.4.6 Computation and Software

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

- Design data (i.e., location, area, runoff coefficients, typical section, slope, etc.).

Appropriate discharge volume and applicable design standards, design geometry required based on operational characteristics – freeboard, velocity, minimum standard capacity and site requirements. Flow regime – subcritical or supercritical – shall be reported and taken into consideration as part of design. Below is a sample output file using Hydraflow Express computer software.

Table 6.14. 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		

6.4.5 Manning's n Values

Recommended Manning's n values for artificial channel linings are given in Table 6.15. 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 6.16. Additional details are provided in the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS 84-204, 1984.

Table 6.15. 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	0.04	0.03	0.028
	Stone Masonry	0.042	0.032	0.03
	Soil Cement	0.025	0.022	0.02
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.02	0.02
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.03
	2-inch D_{50}	0.066	0.041	0.034
Rock Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	----	0.078	0.04

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 6.16 Uniform flow values of roughness coefficient n.			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
EXCAVATED OR DREDGED			
a. Earth, straight and uniform	0.016	0.018	0.020
1. Clean, recently completed	0.018	0.022	0.025
2. Clean, after weathering	0.022	0.025	0.030
3. Gravel, uniform section, clean	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds/plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050

Table 6.16 Uniform flow values of roughness coefficient n.			
Type of Channel and Description	**Minimum**	**Normal**	**Maximum**
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
Minor streams (top width at flood stage < 100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage	0.025	0.030	0.033
2. Same as above, but some stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Clean, winding, but some weeds and some stones	0.035	0.045	0.050
5. Same as 4, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles, few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
FLOODPLAINS			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land, tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
MAJOR STREAMS (top width at flood stage > 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	---	0.060
b. Irregular and rough section	0.035	---	0.100

Source: HEC-15, 1988.

6.4.6 Uniform Flow Calculations

6.4.6.1 Channel Discharge – Manning's Equation

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

$$v = (1.49/n)\, R^{2/3} S^{1/2} \qquad \text{Eq. 6.14}$$

$$Q = (1.49/n)\, A\, R^{2/3} S^{1/2} \qquad \text{Eq. 6.15}$$

$$S = [Q_n/(1.49\, A\, R^{2/3})]^2 \qquad \text{Eq. 6.16}$$

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

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

A = cross-sectional area (ft^2)

R = hydraulic radius A/P (ft)

P = wetted perimeter (ft)

S = slope of the energy grade line (ft/ft)

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

Open channel flow in urban drainage systems is complicated by bridge openings, curbs, and structures. Typically backwater computations will be required for channel design work; however, a check sould 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 computer routines (Appendix H, Stormwater Software), taking into consideration all losses due to the changes in velocity, drops, bridge openings, and other obstructions.

Where practical, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not cause erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Unless approved otherwise by the City Engineer, channel velocities in man-made channels shall not exceed those specified in Tables 6.12 and 6.13.

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

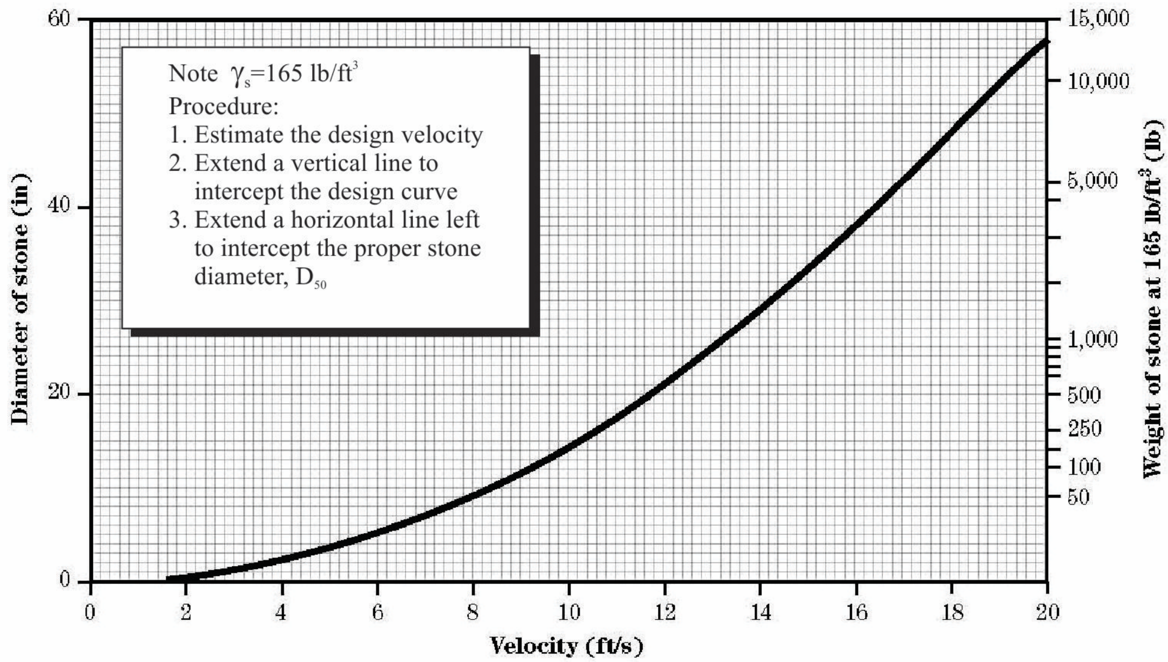
6.4.7 Vegetative Design Requirements

Final design of temporary and vegetative channel linings involves the use of Tables 6.12 and 6.13 for both stability and design capacity.

6.4.8 Riprap Design

Where the use of riprap is allowed by the City 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. See Figure 6.2 for riprap sizing criteria. The design velocity should be based on the higher velocity from the 10-year design event or 100-year check storm event, including 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 6.2. Riprap sizing curve.

6.4.9 Gradually Varied Flow – Backwater Modeling and Data Requirements

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

Many computer programs are available for computation of backwater curves. For further information on acceptable software to use, refer to Appendix H, Stormwater Software.

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

SECTION 6.5. ENERGY DISSIPATION DESIGN

Criteria for Energy Dissipation

1. Energy dissipation is required at outlets and along transitions to existing channels where velocities are high to reduce the potential for erosion and slow the water. Evaluate the downstream channel stability and outlet velocities based on Tables 6.12 and 6.13 as appropriate, including that of the receiving stream, and provide appropriate erosion protection if channel degradation is expected to occur.
2. Additional downstream channel assessment should be conducted on a case by case basis as determined by the City Engineer.

6.5.1 Overview

6.5.1.1 Introduction

The outlets of pipes and lined channels are points of critical erosion potential. Stormwater that is transported through man-made conveyance systems at design capacity generally reaches a velocity that exceeds the capacity of the receiving channel or area to resist erosion. To prevent scour at stormwater outlets, protect the outlet structure and minimize the potential for downstream erosion, a flow transition structure is needed to absorb the initial impact of flow and reduce the speed of the flow to a non-erosive velocity. General guidance for design of outlet protection is provided in Sections 6.5.3 (Baffled Outlets) and 6.5.4 (Outfall Protection). Additional guidance is provided in Appendix G, Outlet Structures.

6.5.1.2 General Criteria

- Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.
- Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosive velocity of the downstream area channel system.

- Energy dissipator designs will vary based on discharge specifics and tailwater conditions.
- Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

6.5.1.3 Recommended Energy Dissipators

For many designs, the use of a baffled outlet provides sufficient protection at a reasonable cost.

This section focuses on the design on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design of Energy Dissipators for Culverts and Channels, for the design procedures of other energy dissipators.

6.5.2 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 6.17 will be used. These symbols were selected because of their wide use. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

Table 6.17. Symbols and definitions

Symbol	Definition	Units
A	Cross-sectional area	ft^2
D	Height of box culvert	ft
d_{50}, d_{10}	Size of riprap	ft
d_w	Culvert width	ft
Fr	Froude Number	-
g	Acceleration of gravity	ft/s^2
h_s	Depth of dissipator pool	ft
L	Length	ft
L_a	Riprap apron length	ft
LB	Overall length of basin	ft
L_s	Length of dissipator pool	ft
PI	Plasticity index	-
Q	Rate of discharge	cfs
S_v	Saturated shear strength	lbs/in^2
t	Time of scour	min.
t_c	Critical tractive shear stress	lbs/in^2
TW	Tailwater depth	ft
VL	Velocity L ft from brink	ft/s
Vo	Normal velocity at brink	ft/s
Vo	Outlet mean velocity	ft/s
V_s	Volume of dissipator pool	ft^2
W_o	Diameter or width of culvert	ft
W_s	Width of dissipator pool	ft
y_e	Hydraulic depth at brink	ft
y_o	Normal flow depth at brink	ft

6.5.3 Baffled Outlets

6.5.3.1 Description

The baffled outlet (also known as the Impact Basin - USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 6.3. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 feet per second. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

6.5.3.2 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U.S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 6.3 should be calculated as follows:

(Step 1) Determine input parameters, including:

h = Energy head to be dissipated (ft), can be approximated as the difference between channel invert elevations at the inlet and outlet

Q = Design discharge (cfs)

v = Theoretical velocity (ft/s = 2gh)

A = Q/v = Flow area (ft^2)

d = $A^{0.5}$= Representative flow depth entering the basin (ft), assumes square jet

Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

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

$$W/d = 2.88Fr^{0.566} \text{ or } W = 2.88dFr^{0.566} \qquad \textbf{Eq. 6.17}$$

W = minimum basin width (ft)

d = depth of incoming flow (ft)

Fr = $v/(gd)^{0.5}$ = Froude number, dimensionless

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

(Step 1) Calculate the other basin dimensions as shown in Figure 6.3, as a function of W. Construction drawings for selected widths are available from the U.S. Department of the Interior (1978).

(Step 2) Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or 5-feet minimum), and depth f (W/6). The side slopes should be 1.5H:1V, and median rock diameter should be at least W/20.

(Step 3) Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

(Step 4) Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 feet/s flowing full.

(Step 5) If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 feet before entering the baffled outlet.

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

6.5.4 Outfall Protection

A design procedure is provided in Figure 6.4. The sizing of d_{50} of the rock shall be determined based on the maximum discharge velocity by use of the curve provided as Figure 6.2. The maximum stone diameter should be 1.5 times the median diameter. If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion based on velocities computed for the design and check storms. Velocities below the outlet shall be computed to confirm they are below erosive velocities for the receiving channel as provided in Tables 6.12 and 6.13. If not, additional energy dissipation shall be provided or the protective riprap apron extended to and across the receiving channel and protection provided to a minimum of 6 inches above the water surface elevation at the design storm for the discharging outlet. The protective apron extents in the receiving channel shall extend to a width three times that of the outlet width where it intersects the receiving channel.

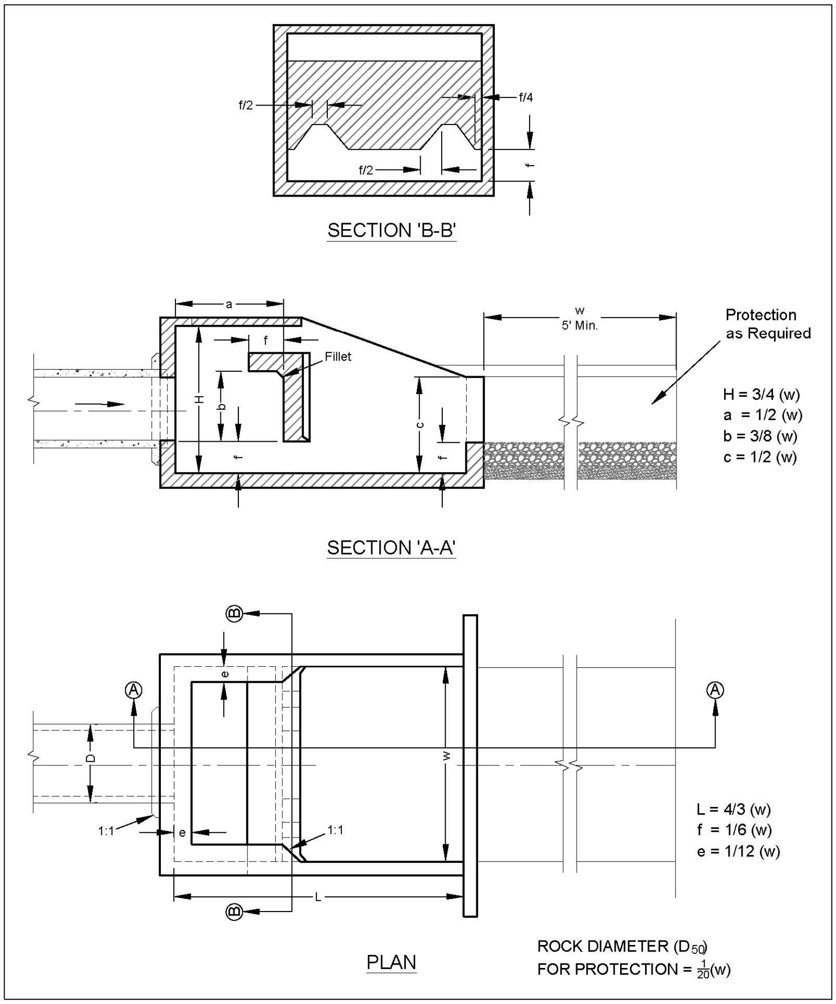
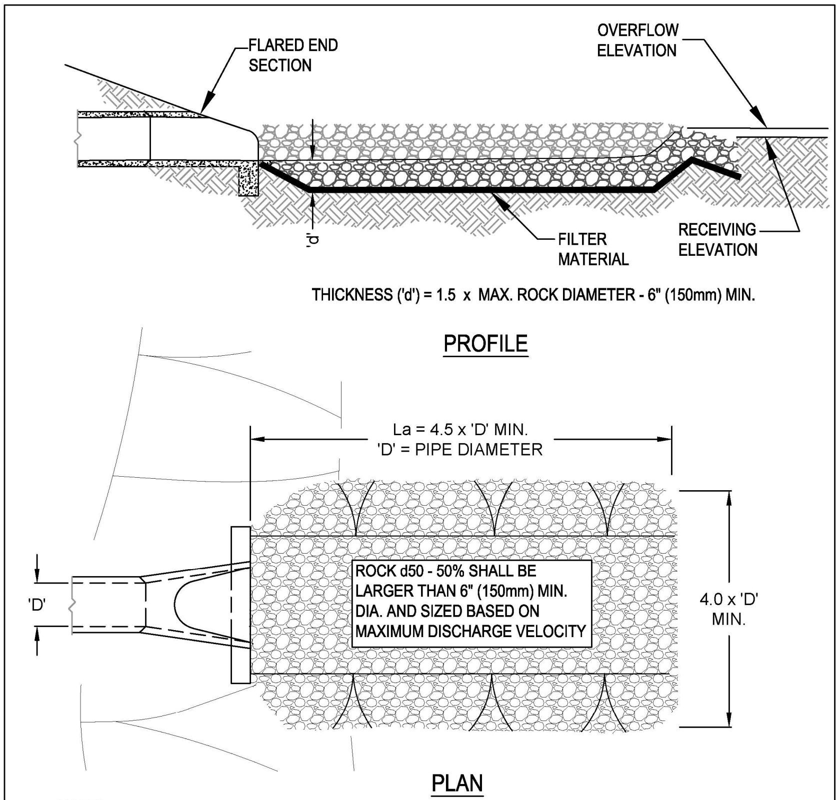


Figure 6.3. Schematic of baffled outlet (Source: U.S. Dept. of the Interior, 1978).



NOTES:
1. 'La' = LENGTH OF APRON. DISTANCE 'La' SHALL BE OF SUFFICIENT LENGTH TO DISSIPATE ENERGY.
2. APRON SHALL BE SET AT A ZERO GRADE AND ALIGNED STRAIGHT.
3. FILTER MATERIAL SHALL BE FILTER FABRIC OR 6" (150 mm) THICK MINIMUM GRADED GRAVEL LAYER.

Figure 6.4. Storm drain outlet protection.

SECTION 6.6. REFERENCES

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CHAPTER 7. STORMWATER DETENTION

SECTION 7.1. GENERAL

7.1.1 Introduction

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).

Storage of stormwater runoff within a stormwater management system is essential to providing the extended detention of flows 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. Design guidance is provided in Appendix F, Water Quality Structural Controls.

7.1.2 Volume of Detention

Volumes of detention shall be evaluated according to the following methods:

1. The Soil Conservation Service (SCS) method shall be used for design of detention basins.
2. 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 Engineer. The method will be evaluated for professional acceptance, applicability, and reliability by the City Engineer. No detailed review will be rendered before the method of evaluation of the retention or detention basin is approved.

7.1.3 Design Criteria

Stormwater detention systems shall be designed to meet the stormwater sizing criteria described in Chapter 2 and shall provide structural control as needed to meet the Minimum Standards.

SECTION 7.2. DETENTION DESIGN PROCEDURES

7.2.1 Introduction

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. Where present, the permanent pool elevation is taken as the “bottom” of storage and is treated as if it were a solid basin bottom for routing purposes.

The location of structural stormwater controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Therefore, a downstream peak flow analysis should be performed as part of the storage facility design process (see Section 7.5).

7.2.2 Estimating Detention Volume

To estimate the storage volume (Vs) required to meet the four minimum standards, the following procedure may be used.

1. Compute required water quality volume, $WQ_{v,}$ as described in Chapter 2, Section 2.1.2 and amplified in Chapter 4. If a permanent pool is used, the water quality volume may be split evenly between the permanent pool and extended detention storage (to be released over 24 hours).
2. Compute existing (predevelopment) and proposed (post development) peak discharges (pre-detention) for the 1-, 2-, 5-, 10-, 25-, and 100-year 24-hour storm events. The 2-, 5-, and 10-year peak discharges may only be needed to perform checks to verify adequate design.
3. Compute elevations in the receiving channel at the outlet for the storm events listed in step 2. The elevation for a storm event shall be applied as the tailwater condition for detention outlet calculations for that same given event.
4. Compute the Channel Protection volume, CPv, or storage volume required for channel protection based on the runoff hydrograph from the 1-year, 24-hour storm event as described in Chapter 2, Section 2.1.3. The C_{Pv} shall be released over a 40-hour period (72-hour maximum drain time). If the post-development discharge is less than 2 cfs, then C_{Pv} storage is not required, but the 1-year, 24-hour storm post development discharge shall not exceed the 1-year, 24-hour storm predevelopment discharge for the project area, so a corresponding storage volume shall be provided.
5. Use the SCS Method described in steps 6 through 9 to compute the Extreme Flood, Qf, volume required such that the post-development peak discharge rate does not exceed the predevelopment rate for the 100-year, 24-hour return frequency storm event (Q_F).
6. As a starting point, assume the peak discharge q_o from the proposed detention basin is equal to the corresponding predevelopment peak discharge determined in step 1 (q_f), and assume the proposed inflow to the detention basin (q_i) is equal to the post-development peak discharge determined in Step 2.
7. Compute qo/qi and, using Figure 7.1 below, determine V_s/V_r .

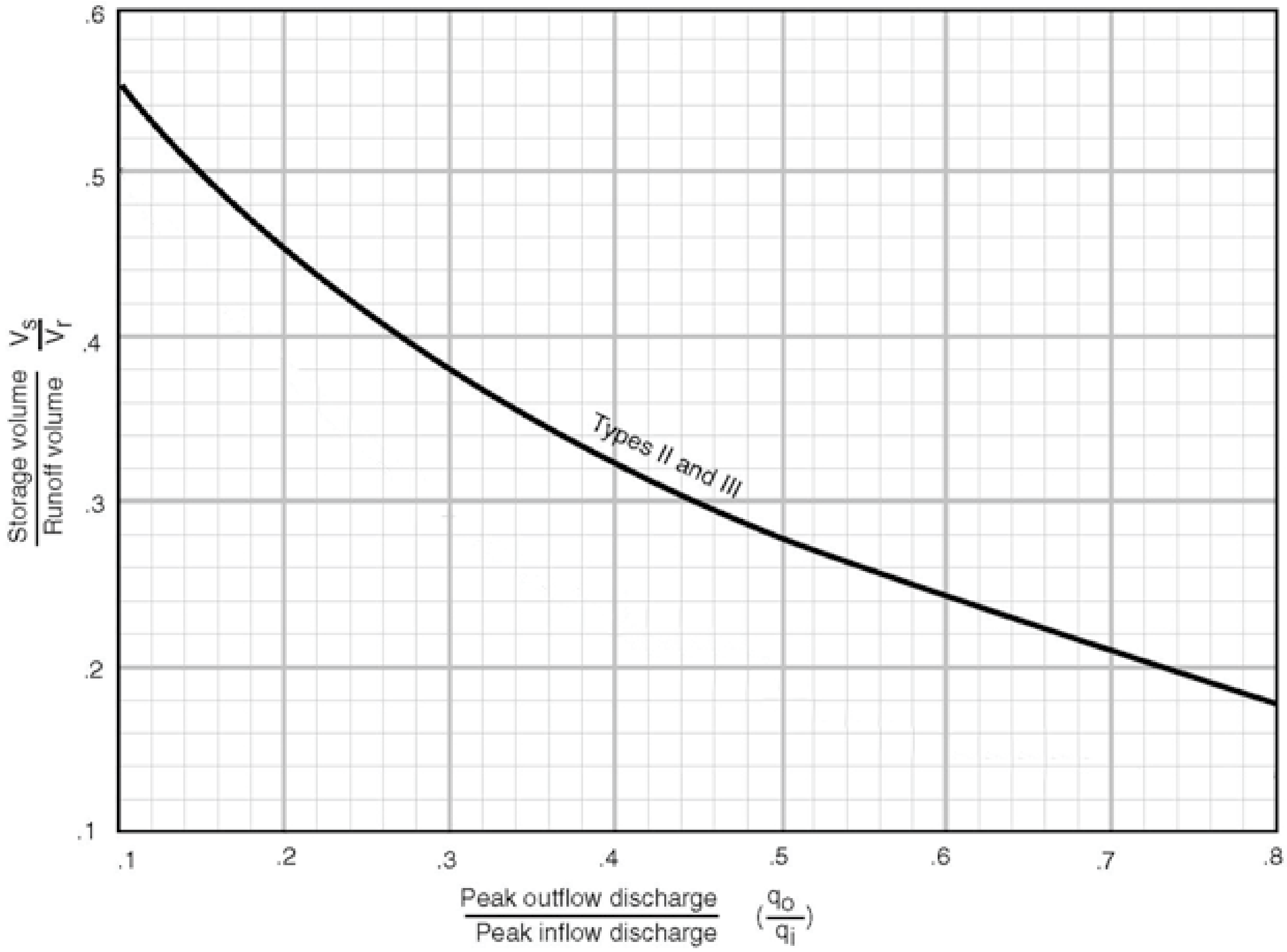

Figure 7.1. Approximate detention basin routing for Type III rainfall distribution.

8. Use the runoff depth (watershed inches) that was determined from the SCS method while computing post development peak discharges q_i in step 2 to compute the runoff volume (V_r).
9. $V_r = \frac{(Q*A_m)}{12}$ **Eq. 7.1**
10. where:
11. V_r = runoff volume (acre-feet)
12. Q_i = runoff depth (watershed inches) for the given post development inflow event
13. A_m = drainage area (acres)

14. Use the results from steps 7 and 8 to compute V_s.

$$V_s = V_r\left(\frac{V_s}{V_r}\right) \qquad \textbf{Eq. 7.2}$$

where: V_s = storage volume required (acre-feet)

15. Due to the requirement for extended detention of the WQ_v and CP_v, it may be appropriate to add the storage volume required to meet these standards to the storage volume required for the extreme flood protection standard to obtain an initial total storage volume for detention design.
16. The stage in the detention basin corresponding to V_s must be equal to the stage used to generate q_o.
17. Once the initial estimates of stage-volume data are developed, detention design procedures described below in Section 7.2.3 shall be followed.

7.2.3 Detention Basin 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 Section 7.2.2.

Step 2 Determine the physical dimensions necessary to hold the estimated volume from Step 1. The maximum storage requirement calculated from Section 7.2.2 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 Water Quality and Channel Protection Volume events to the extreme flood event and taking into consideration the tailwater in the receiving stream. The estimated peak stage for each storm event 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. Refer to Appendix G, Section 3 for detailed steps regarding outfall design. 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 an approved storage routing computer model (See Appendix H for approved analysis and design software).

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 25- and 100-year 24-hour storm events 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).

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 dissipater devices may be required. Guidance for riprap sizing and extents of placement and outlet design is provided in Section 6.5 and Appendix G.

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 Fayetteville. Additional information regarding the design requirements for extended detention and associated outlet design is provided in Appendix F.

Water quality requirements and the associated Water Quality Protection Volume (WQ_v) shall be addressed in the design. Details regarding these requirements and the approach that may be used to address them are provided in Chapter 4, Water Quality, and Appendix F.

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. A list of approved software to perform storage routing calculations is provided in Appendix H, Stormwater Software.

SECTION 7.3. METHODS OF DETENTION

Detention storage may be categorized as inline or offline. The City of Fayetteville only allows inline storage if it can be demonstrated that offline storage is not practicable. Figure 7.2 illustrates inline versus offline storage.

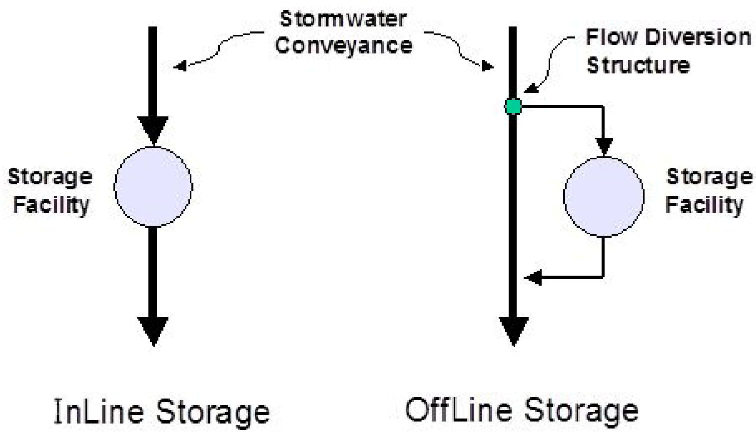

Figure 7.2. Inline versus offline storage.

7.3.1 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 Fayetteville. A brief description is provided in this section, with detailed design requirements and procedures provided in Appendices E and F. Regardless of the detention practice selected, mosquito control measures shall be taken. Avoid creating areas of shallow stagnant water and low dissolved oxygen which create mosquito habitat. To avoid creating habitat, pools of water should be at least 5 feet deep and residence time should be less than 72 hours (excepting permanent pools).

7.3.1.1 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 Minimum Standard #1, however; they also can provide detention storage to meet the other Minimum Standards.

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. Refer to Appendix F for more information on stormwater ponds, including example details and design requirements.

7.3.1.2 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 Minimum Standard #1, however; they also can provide detention storage to meet the other Minimum Standards.

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. Refer to Appendix F for more information on stormwater wetlands, including example details and design requirements.

7.3.1.3 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 Minimum Standards two through four. 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. Refer to Appendix E for more information on dry detention and dry ED basins, including example details and design requirements.

7.3.1.4 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. Example of multi-purpose detention areas include:

- Parking Lots
- 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 Minimum Standards # 3 and 4, but not for water quality treatment or channel protection with extended detention (Minimum Standards # 1 and 2). Multi-purpose detention areas should be used in a treatment train approach with other structural controls to provide water quality treatment. Refer to Appendix E for more information on multi-purpose detention areas, including example details and design requirements.

7.3.1.5 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 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. Refer to Appendix E for more information on underground detention, including example details and design requirements.

SECTION 7.4. DETENTION DESIGN STANDARDS

The following conditions and limitations shall be observed in selection and use of the method or type of detention. Appendices E and F also provide detailed requirements for the various types of detention allowed in Fayetteville.

7.4.1 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. Pond bottom slopes must be a minimum of 1% (longitudinal and cross-slope) to ensure positive drainage to outlet works. Orifices shall be provided that limit outflows to be in accordance with design requirements and to not exceed pre-development discharges.

7.4.2 Dry Detention / Dry ED Basins

Wet weather 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 Engineer. 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, where practicable. The entire reservoir area shall be stabilized with vegetation established prior to final approval or issuance of certificate of occupancy unless approved by the City Engineer. 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 Plans.

7.4.3 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 coarsegrained 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 shall be stabilized with vegetation established prior to final approval or issuance of certificate of occupancy unless approved by the City Engineer. 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 a Professional Engineer. Plan view and cross-sections with adequate details for any stormwater ponds shall be provided in the Plans.

In karst sensitive areas, or areas with high pollutant discharge potential, pond liners shall be used. If permanent pool areas are desired, use of pond liners may be permitted where necessary based on soil infiltration characteristics.

7.4.4 Parking lots

Detention is permitted in parking lots to a maximum depth of 6 inches. In no case should the maximum limits of ponding (including inlet ponding) be designed closer than 10 feet from a building unless waterproofing of the building and pedestrian accessibility and safety are properly documented and approved.

The maximum ponding elevation shall be 1 foot or more below the lowest sill or floor elevation and shall be at least below the lowest adjacent grade of contiguous habitable and commercial structures. In floodplain areas, Floodplain Ordinance requirements must also be met. Plan view and cross-sections of the parking lot with adequate details shall be included in Plans.

7.4.5 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 5 of the Drainage Criteria Manual for detailed design requirements for LID practices and for the approach to adjust peak discharges, where appropriate, based on implementation of LID features.

7.4.6 Underground Detention

Underground detention, if used, shall be designed in accordance with the recommendations provided in Appendix E, DSC-03: Underground Detention.

7.4.7 Wetlands

Wetlands, if designed for detention, shall be in accordance with recommendations provided in Appendix F, WSC-02: Stormwater Wetlands.

7.4.8 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.

7.4.9 Verification of Adequacy

Project closeout submittals shall include documented verification of adequacy in accordance with Section 1.4.5 of this manual.

7.4.10 Outlet Works

Detention facilities shall be provided with effective outlet works. Flows shall be limited to design storm events consistent with applicable Minimum Standards. See Appendix G for example details and design requirements for various outlet structures.

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

Overflow openings are required for all ponds. The overflow opening shall be designed to accept the total peak runoff of the improved tributary area.

7.4.11 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.

7.4.12 Ownership of Stormwater Detention Ponds

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

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

7.4.13 Easements

Easements shall be provided on the plans for detention facilities. A minimum 20-feet wide drainage easement shall be provided within 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.

7.4.14 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.

SECTION 7.5. DOWNSTREAM HYDROLOGIC ASSESSMENT

7.5.1 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 actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) 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.

7.5.2 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 actually increase the peak discharge downstream. The reason for this may be seen in Figure 7.4. 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 actually higher than if no detention were required. In this case, the shifting of flows to a later time brought about by the detention pond actually makes the downstream flooding worse than if the post-development flows were not detained.

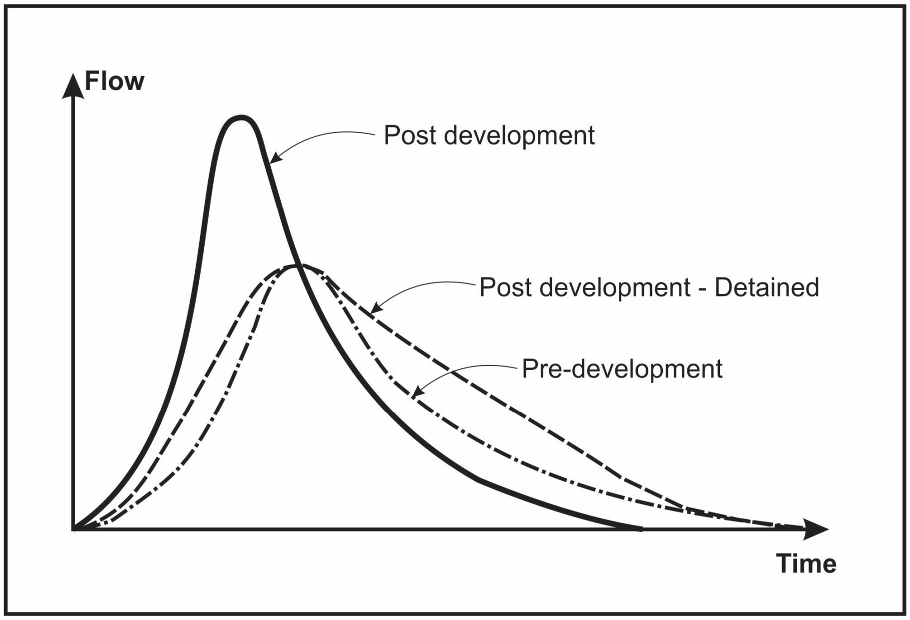

Figure 7.4. Detention timing example.

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 7.5 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 would have to have been increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

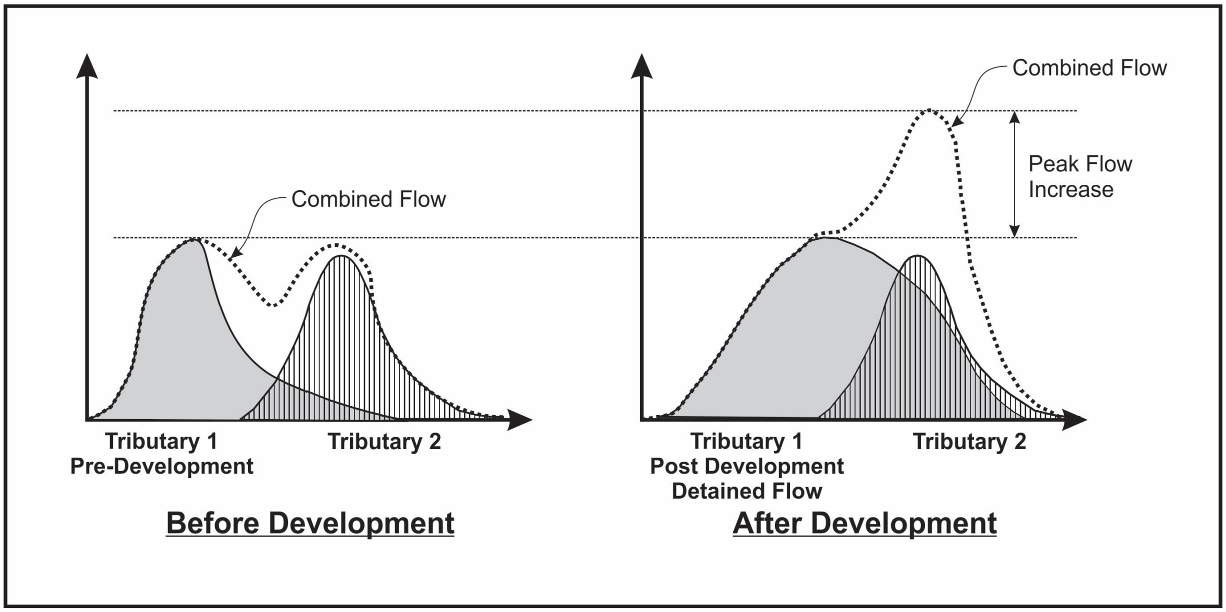

Figure 7.5. Effect of increased post-development runoff volume with detention on a downstream hydrograph.

7.5.3 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 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater. The City 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 predevelopment 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 Fayetteville 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 overbank and extreme flood protection post development peak discharges are not increased above pre-development discharges at the outlet and the determined tributary junctions.

7.5.4 Example Problem

Figure 7.6 illustrates the concept of the ten-percent rule for two sites in a watershed.

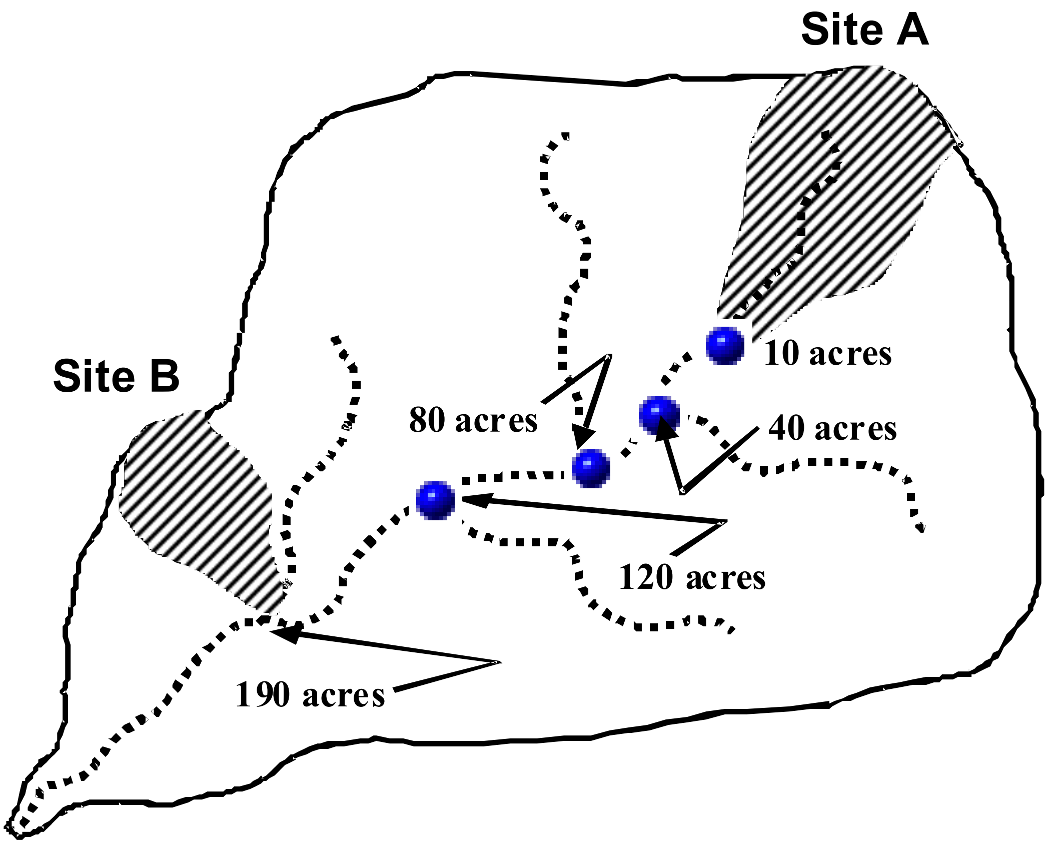

Figure 7.6. Example of the ten-percent rule.

<u>Discussion</u>

Site A is a development of 10 acres, all draining to a wet ED stormwater pond. The overbank flooding and extreme flood portions of the design are going to incorporate the ten-percent rule. Looking downstream at each tributary in turn, it is determined that the analysis should end at the tributary marked "80 acres." The 100-acre (10%) point is in between the 80-acre and 120-acre tributary junction points.

The assumption is that if there is no peak flow increase at the 80-acre point then there will be no increase through the next stream reach downstream through the 10% point (100 acres) to the 120-acre point. The designer constructs a HEC-HMS model of the 80-acre area using single existing condition sub-watersheds for each tributary. For the site area, watershed conditions will be modeled to the level of detail used for design. Detention structures existing in other tributaries must be modeled. Since flooding is an issue downstream, the pond design is iterated with subsequent re-checks until the peak flow does not increase at junction points downstream to the 80-acre point.

Site B is located downstream at the point where the total drainage area is 190 acres. The site itself is only 6 acres. The first tributary junction downstream from the 10% point is the junction of the site outlet with the stream. The total 190 acres is modeled as one basin (with the site at the same level of detail as for design) with care taken to estimate the time of concentration for input into the model of the watershed. The model shows that the detention facility, as designed, will actually increase the peak flow in the stream. The detention facility design is subsequently modified to eliminate the increase in peak flow.

SECTION 7.6. STORMWATER DETENTION ANALYSIS SOFTWARE

The city encourages the use of software to model the stormwater management system. Refer to Appendix H for additional information regarding appropriate software for stormwater management analysis in Fayetteville.

SECTION 7.7. REFERENCES

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

CHAPTER 8. CONSTRUCTION SITE STORMWATER MANAGEMENT

SECTION 8.1. PERMITS AND PLANS

8.1.1 Stormwater Pollution Prevention Plan

The site owner bears responsibility for implementation and preparation of the SWPPP and notification of all contractors and utility agencies on the site.

The Arkansas Department of Environmental Quality (ADEQ) requires that all construction activities disturbing one acre or more shall have a SWPPP to support issuance of a construction stormwater general permit. Sites with disturbance of more than one acre and less than 5 acres have automatic coverage.

- For the small construction sites (1 to 5 acres), no submittal of individual permit documents is required and there is no fee; however the SWPPP and the automatic Notice of Coverage (NOC) shall be posted at the site prior to commencing construction.
- For larger construction sites (disturbance of 5 acres or more), SWPPP documents and a Notice of Intent (NOI) must be submitted to ADEQ with fee for review and approval.

For more information on specific requirements, please visit the ADEQ Construction Stormwater Program website at: http://www.adeq.state.ar.us/water/branch_permits/general_permits/stormwater/construction/construction.htm

8.1.2 Grading and Drainage Permits

A grading permit will not be issued until the perimeter sediment controls and permit box have been installed and approved by the City Engineering Division, and a preconstruction conference has been held.

8.1.3 Phased Construction

The area of disturbance onsite at any one time shall be limited to 20 acres. An additional 20 acres (a maximum of 40 acres of disturbance at any one time) may be stripped with the permission of the City Engineer in order to balance cut and fill onsite. No additional area may be open without the permission of the City Engineer until the previously disturbed areas have been temporarily or permanently stabilized.

8.1.4 Installation and Maintenance.

8.1.4.1 Stormwater Pollution Prevention Plans.

Preparation and implementation of Stormwater Pollution Prevention Plans for construction activity shall comply with the following:

- BMPs shall be installed and maintained by qualified persons in accordance with applicable City of Fayetteville and State requirements. The owner or their representative shall maintain a copy of the SWPPP on site and shall be prepared to respond to unforeseen maintenance requirements of specific BMPs.

8.1.5 Qualified Inspector

A qualified inspector (provided by the owner/developer/builder) shall inspect disturbed areas of the construction site and areas used for storage of materials that are exposed to precipitation that have been finally stabilized, and locations where vehicles enter or exit the site. BMPs must be observed to ensure proper operation. Inspectors must inspect for evidence of, or the potential for, pollutants entering the stormwater conveyance system. Discharge locations must be inspected to determine whether BMPs are effective in preventing significant impacts to waters of the State, where accessible. Where discharge locations are inaccessible, nearby downstream locations must be inspected to the extent that such inspections are practicable. The inspections must be conducted at least once every seven (7) calendar days or at least once every 14 calendar days and within 24 hours of the end of a storm that is 0.5 inches or greater as measured at the site or generally reported in the vicinity of the site. A rain gauge must be maintained on-site.

A report shall be prepared for each inspection summarizing the scope of the inspection; name(s), title(s) and qualifications of personnel making the inspection; the date of the inspection; amount of rainfall and days since last rain event, BMPs on-site; observations relating to whether BMPs are in working order and whether maintenance is required (when scheduled and completed); the locations and dates when major construction activities begin, occur, or cease; and the signature of the inspector. The reports shall be retained as part of the stormwater pollution prevention plan for at least three (3) years from the date the site is finally stabilized and shall be made available upon request to the City.

8.1.6 Modifications

Changes to the SWPPP are often necessary and may be made during the construction phase. Based on inspections performed by the owner or by authorized City personnel, modifications to the SWPPP will be necessary if at any time the specified BMPs do not meet the objectives of the City of Fayetteville UDC 170.10, Stormwater Discharges From Construction Activities. In this case, the owner/developer/builder or authorized representative shall meet with authorized City personnel to determine the appropriate modifications. All modifications shall be completed within seven (7) days of the referenced inspection, except in circumstances necessitating more timely attention, and shall be recorded on the owner's copy of the SWPPP.

8.1.7 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 period 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 Chapter 167 of the UDC for tree protection requirements.

SECTION 8.2. EROSION, RUNOFF, AND SEDIMENT CONTROLS FOR CONSTRUCTION SITES

8.2.1 Erosion Control

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.

EPA NPDES Fact Sheets for the following twelve Erosion Control BMPs are provided under the Erosion Control section of Appendix J:

Compost Blankets	**Dust Control**	**Geotextiles**
Gradient Terraces	**Mulching**	**Riprap**
Seeding	**Sodding**	**Soil Retention**
Soil Roughening	**Temporary Slope Drain**	**Temporary Stream Crossings**

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

- **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 with regard to 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 City of Fayetteville UDC Chapter 169.04, Minimal Erosion Control 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.

- **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. Additional information regarding plantings suitable for use in the area is provided in Appendix D. 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.
- **Dust Control.** Saturate ground or apply dust suppressors. Keeping dust down to tolerable limits on the construction site and haul roads is very important.
- **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 a number of 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 5 and Appendix D.
- **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 dissipater.
- **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.

8.2.2 Runoff Control

Protect streams from chemicals, fuels, lubricants, sewage, or other pollutants. Do not place or dispose of fill in floodplains or drainageways. Use temporary bridges with culverts to ford streams. Avoid developing borrow areas where pollution from this operation cannot be controlled.

EPA NPDES Fact Sheets for the following four Runoff Control BMPs are provided under the Runoff Control section of Appendix J:

Check Dams	**Grass-Lined Channels**
Permanent Slope Diversions	**Temporary Diverson Dikes**

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

- **Erosion Control for Open Channels.** In designing channels for erosion control, the velocity for the 10-year event shall be computed and compared to the erosive potential of the channel material, in accordance with procedures and allowable velocities provided in Chapter 6, Section 6.4. Where the allowable velocity for a turf channel or earthen channel will be exceeded, alternatives include: lining the channel with impervious material, using drop structures or other velocity and erosion control measures; placing gravel or riprap bottoms with riprap side slopes; and gabions (rock enclosed in wire baskets) especially for steeper slope applications.

  The open channel and swale design should be evaluated for the extreme flood runoff with respect to flow velocities and erosion potential. Antecedent flow conditions resulting from previous storms are an important consideration. Open channels and swales may suffer damage during major storms, even if properly designed. Such damage shall be repaired promptly to prevent further erosion.

  It is important that open channels be constructed in accordance with design plans. When side slopes of intermittent channels are sodded to the depth of the expected flow, they can immediately provide erosion control for runoff from minor storms. It is not practical to establish turf in a drainage channel by seeding and mulching unless jute mats, or other similar erosion control matting materials, are placed over the seedbed.
- **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.
- **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.

- **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.
- **Level Spreader.** This is a temporary structure that is constructed at zero grade across the slope where concentrated runoff may be intercepted and diverted onto a stabilized outlet. The concentrated flow or stormwater is converted to sheet flow at the outlet.
- **Diversions.** These are designed, graded channels with a supporting ridge on the lower side constructed across the slope. Their purpose is to intercept surface water. Diversion structures may be temporary or permanent and graded or level. They are useful above cut slopes, borrow areas, gully heads, and similar areas. They can be constructed across cut slopes to reduce slope plains into nonerosive segments and can be used to move runoff water away from critical construction sites. They may be used at the base of cut or fill slopes to carry sediment-laden flow to traps or basins. Diversions should be located so that the water will empty into established runoff areas, natural outlets, or prepared individual outlets. Individual outlets can be designed as grass or paved waterways, chutes, or buried pipes.

8.2.3 Sediment Control

EPA NPDES Fact Sheets for the following eleven Sediment Control BMPs are provided under the Sediment Control Section of Appendix J:

Compost Filter Berms	**Compost Filter Socks**	**Construction Entrance/Exits**
Fiber Rolls	**Filter Berms**	**Sediment Basins and Rock Dams**
Sediment Filters and Chambers		**Sediment Traps**
Silt Fences	**Storm Drain Inlet Protection**	**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 entirely eliminate 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, straw wattles, 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 exits shall comply with City of Fayetteville UDC 170.10, Stormwater Discharges From Construction Activities, and corresponding erosion control at such exits and on public streets shall comply with City of Fayetteville UDC 169.04, Minimal Erosion Control Requirements.

- **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.
- **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 overall 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.
- **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.

8.2.4 Good Housekeeping

EPA NPDES Fact Sheets for the following Good Housekeeping BMPs are provided under the Good Housekeeping section of Appendix J:

Concrete Washout	**Spill Prevention and Control**

Additional items with respect to City good housekeeping practices and requirements are provided herein:

- **Storage of Materials.** Public streets and sidewalks shall not be used for temporary storage of any containers or construction materials, especially loose gravel and topsoil. In addition to on-street storage being a violation, all liability for any accidents and/or damages due to such storage will be the responsibility of the owner of the stored materials.
- **Excavation Material.** Excavation material shall not be deposited in or so near streams and other stormwater drainage systems where it may be washed downstream by high water or runoff. All excavation material shall be stabilized immediately with erosion control measures.
- **Debris, Mud, and Soil.** Debris, mud and soil shall not be allowed on public streets but if any debris, mud, or soil from development sites reaches the public street it shall be immediately removed via sweeping or other methods of physical removal. Debris, mud, or soil in the street may not be washed off the street or washed into the storm drainage system. Storm drainage systems downstream of a development site should be protected from debris, mud, or soil in the event that debris, mud, or soil reaches the drainage system.
- **Dirt and Top Soil Storage.** Top soil shall be stockpiled and protected for later use on areas requiring landscaping. All storage piles of soil, dirt or other building materials (e.g. sand) shall be located more than 25 feet from a roadway, drainage channel or stream (from top of bank), wetland, and stormwater facility. The City Engineer may also require top soil stockpiles to be located up to 50 feet from a drainage channel or stream, as measured from the top of the bank to the stockpile, for established TMDL water bodies; streams listed on the State 303(d) list; an Extraordinary Resource Water, Ecologically Sensitive Waterbody, and/or Natural and Scenic Waterbody, as defined by Arkansas Pollution Control and Ecology Commission Regulation No. 2; and/or any other uses at the discretion of the City Engineer.

  Topsoil piles surfaces must be immediately stabilized with appropriate stabilization measures. Stabilization practices may include: temporary seeding (i.e. annual rye or other suitable grass), mulching, and other appropriate measures. Sediment control measures such as silt fence shall be provided immediately for stockpiles and remain in place until other stabilization is in place. Storm drain inlets must be protected from potential sedimentation from storage piles by silt fence or other appropriate barriers.

- **Concrete Washout.** No washing of concrete trucks or chutes is allowed except in specific concrete wash pits located onsite. Proper runoff and erosion controls must be in place to retain all concrete wash water.
- **Spill Prevention and Control.**

SECTION 8.3. UNDERGROUND UTILITY CONSTRUCTION – PLANNING AND IMPLEMENTATION

The property owner or main contractor onsite will be responsible for restoring all erosion and sediment control systems and public infrastructure damaged or disturbed by underground private or franchise utility construction such as water and sewer service leads, telephone, gas, cable, etc. Erosion and sediment control systems must be immediately restored after each utility construction.

Utility agencies shall develop and implement Best Management Practices (BMPs) to prevent the discharge of pollutants and release of sediments from utility construction sites. Disturbed areas shall be minimized, disturbed soil shall be managed and construction site entrances shall be managed to prevent sediment tracking. Excessive sediment tracked onto public streets shall be removed immediately. Prior to entering a construction site or subdivision development, utility agencies shall have obtained from the owner or developer a copy of any SWPPPs for the project. Any disturbance to BMPs resulting from utility construction shall be repaired in compliance with the SWPPP. The property owner or main contractor is responsible for restoration of any damage by private or franchise utility construction, in accordance with City of Fayetteville Ordinance UDC 170.10 (A)(10).

SECTION 8.4. POST-CONSTRUCTION SITE STABILIZATION STANDARDS

Revegetation of disturbed areas shall be performed as soon after the completion of construction activities as is practicable. The area of disturbance at any one time shall be limited to 20 acres. No additional area may be open without the permission of the City Engineer until the previously disturbed areas have been temporarily or permanently stabilized.

Revegetation shall be required to meet the following performance standards prior to issuance of the Final Plat or Certificate of Occupancy:

1. **Topsoil**: A minimum of 4 inches of topsoil shall be required in areas to be revegetated. Any application of topsoil and seeding under the drip line of a tree should be minimized to 3 inches so as not to damage the root system of the tree.
2. **Zero to 10% grade**: Revegetation shall be a minimum of seeding and mulching. Said seeding shall provide complete and uniform coverage that minimizes erosion and runoff in no more than two growing seasons.
3. **10:1 up to 4:1 grade**: Revegetation shall be a minimum of hydroseeding with mulch and fertilizer, or staked sod, or groundcover. Said planting shall provide complete and uniform coverage that minimizes erosion and runoff in no more than two growing seasons.

4. **4:1 to 3:1 grade**: The slope shall be covered with landscape fabric and hydro-seeded with mulch and fertilizer or staked sod groundcover. Said planting shall provide complete and uniform coverage in no more than two growing seasons.
5. **Steeper than 3:1 grade**: The slope shall be stabilized with one or more of the following:
   a. Retaining walls,
   b. Cribbing with landscaping fabric,
   c. Terracing with groundcover,
   d. Riprap,
   e. Staked Sod (up to 2:1 slope), or
   f. If cribbing, terracing, or rip rap are used, the slope stability and erodibility characteristics must be equivalent to or better than its predevelopment state.
6. **Hillside/Hilltop Overlay District**: Revegetation of lands within the Hillside/ Hilltop Overlay District shall be planted immediately after the physical alteration of the land with complete and uniform ground cover. Sod, erosion fabric, herbaceous groundcover (in wooded areas), and/or a hydroseed with warm season grasses is required. Revegetation requirements shall be met prior to the issuance of the certificate of occupancy. Cut and Fill tie-back slopes shall be re-vegetated with appropriate tree species to achieve a minimum of 25% tree canopy at maturity.

Land shall be revegetated and restored as close as practically possible to its original conditions so far as to minimize runoff and erosion are concerned. Previously forested areas shall follow the City's Landscape Manual for mitigation of forested areas.

1. **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.

Water Control:

1. **Subsurface Drains**: 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.

SECTION 8.5. REFERENCES

Arkansas Department of Environmental Quality (ADEQ), 2012. Stormwater Pollution Prevention Plan.

Arkansas Department of Environmental Quality (ADEQ), 2011. Construction Stormwater General Permit.

City of Fayetteville, AR, 1995. Drainage Criteria Manual.

City of Fayetteville, AR, various dates. City Ordinances UDC 169, 170.