STORM WATER DRAINAGE DESIGN MANUAL AND FLOODPLAIN COMPLIANCE GUIDELINE

Prepared for:

CITY OF ARKADELPHIA
CLARK COUNTY
ARKANSAS

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1.0 INTRODUCTION

1.1 Purpose

The purpose of this manual is to establish standard principles and practices for the design and construction of storm water drainage facilities within Arkadelphia, Arkansas. In addition floodplain compliance guidelines will be presented to insure base flood elevations will not rise for the construction or modification of structures or land alteration within the floodplain. The design factors, formula, graphs, and procedures are intended for use as engineering guides in the solution of drainage problems involving determination of the quantity, rate of flow, and conveyance of storm water. The procedures defined herein shall be applied by experienced professional engineers licensed to practice in the state of Arkansas. Also, ultimate responsibility for the design of a storm drainage structures lies with the engineer of record. As such, prudent engineering judgment should be used in the design of any facility within Arkadelphia.

Methods of design other than those indicated herein may be considered in difficult cases where experience clearly indicates they are preferable. However, there should be no extensive variations from the practices established herein without express approval from the City of Arkadelphia.

1.2 Scope

This manual presents various applications of accepted principles of surface drainage engineering and is a working supplement to the information obtained from standard drainage handbooks and other publications on drainage. It is presented in a format that gives logical development of solutions to problems of storm water drainage design and floodplain management.

This manual is intended to be used by the City of Arkadelphia, consulting engineers contracted with the City, and for private development within the planning jurisdiction of the City. This manual applies to storm drainage conditions, which are generally relative to the City of Arkadelphia and the immediate geographical area. Accepted engineering principles, applied to the City of Arkadelphia’s storm drainage requirements, are detailed within this manual.

1.3 Drainage Policy

The basic objective of the City of Arkadelphia is to construct and maintain facilities intended to minimize the threat of flooding to all areas of the City and comply with the requirements of the National Flood Insurance Program. The drainage facilities are defined as all public channels within dedicated right-of-way approved and accepted by the City and all other public flood control structures and facilities dedicated to, approved and accepted by the City. Additionally, it is the City’s intent to insure that adequate facilities are constructed to accommodate new development such that existing property will not be subjected to additional flooding and so as not to increase the limits of the floodplains as shown on the flood insurance rate maps (FIRM's) for the City of Arkadelphia and other entities (County, Levee Improvement Districts, and Municipal Utility Districts).

It is not economically feasible to construct storm sewer facilities, which are large enough to keep the street systems from becoming inundated during severe storm events. The net effect of the City's policies will be of minimum depth and duration, and most importantly, that new structural building slab elevations are, at a minimum, set at least 12 inches above the maximum, anticipated, fully urbanized 100 year base flood elevations. The intent of this policy is that there should be any street ponding for minor storm events, minor street ponding for larger events, and major ponding for the 100-year event storms but without water inundating building structures. Every attempt will be made to design major thoroughfares so that they are passable during severe storm events.

The City has included in this manual criteria covering the design of storm water systems to serve both existing and new developments. The City of Arkadelphia has quantified the needed improvements for existing development in most of the watersheds in the City and is responsible for their approval. Upon the completion of all new design of drainage facilities, the City will accept, maintain, and operate said facilities for flood control purposes as an extension of the City. However, those drainage facilities, including detention facilities, which are planned and accepted for maintenance by some other perpetual special purpose district (such as a Levee Improvement District) will not be accepted by the City.

The criteria in this manual is considered a minimum for the City of Arkadelphia. Approval from other applicable agencies may be required. Ultimate approval for any variance of the criteria contained in this manual must be given by the City of Arkadelphia.

2.0 CONSTRUCTION PLAN PREPARATION

2.1 GENERAL

This section covers the preparation of drainage construction plans for the City of Arkadelphia.

2.2 Design Phase

Plans shall be submitted in accordance with the City of Arkadelphia's Checklist for Storm Drainage Plans. The first engineering plan set submission shall be complete, and in sufficient detail to allow review by the City of Arkadelphia. All topographic surveys should be furnished to allow establishment of alignment, grades, and right-of-way requirements. These may be accomplished by on-the-ground field surveys or aerial photogrametric methods.

The hydraulic design of the proposed facilities shall be accomplished based on the procedures and criteria outlined in this manual. Calculations shall be made on the appropriate forms and submitted as part of the plan set. These plans shall show the alignment, drainage areas, size of facilities, and grades.

Storm drainage plans shall include at a minimum, a drainage area map, plan-profile sheets, and channel cross-sections, if applicable. The proposed improvements shall be produced on 24" x 36" sheets.

Survey control performed for the project shall reference two reference marks established by the City of Arkadelphia. A copy of the reference marks can be obtained from the City Engineer.

Survey control for the project shall conform to the following requirements:

Vertical control will be NAVD 88, Second Order Vertical
Horizontal control will be NAD 83, First Order Horizontal, Arkansas State Plane South Zone

As builts of the project shall be provided to the City of Arkadelphia in a digital exchange format (DXF). Acceptance by the City of Arkadelphia will not be given until as-builts are provided to the City.

2.2.1 Drainage Area Map

The drainage area map shall have a minimum scale of 1" = 200', and show the street right-of-way. For large drainage areas, a map having a minimum scale of 1" = 2000' is usually sufficient.

The following items/information shall be included:

(1) Acres, coefficient, and intensity for each drainage sub-area;

(2) Inlets, their size and location, the flow bypass for each, the direction of flow as indicated by flow arrows, the station for the centerline;

(3) A chart including data shown shall be submitted with the first review, and included on the map with the final review;

(4) Existing and proposed storm sewers;

(5) Sub-areas for alleys, streets, and off-site areas;

(6) Points of concentration;

(7) Runoff to all inlets, dead-end streets, and alleys or to adjacent additions and/or lots;

(8) A table for runoff computations;

(9) Flow arrows to indicate all crests, sags and street and alley intersections;

(10) North arrow;

(11) Any off-site drainage shall be included;

(12) Street names shall be indicated;

When calculating runoff, the drainage area map shall show the boundary of the drainage area contributing runoff into the proposed system. This boundary should be determined from a map having a contour interval not exceeding two feet. The area shall be further divided into sub-areas, sequentially numbered, to determine flow concentration points or inlet locations. The centerline of all streets will normally be a boundary of a drainage area, to insure that inlets are sized and positioned to fill the need without depending on storm water crossing over the street crown for proper drainage.

In residential areas, the centerline of the street will only be used as a drainage area boundary if the flow in either gutter has not exceeded the street crown elevation.

Direction of flow within streets, alleys, natural and man-made drainage ways, and at all system intersections, shall be clearly shown on the drainage area map and/or paving plans. Existing and proposed drainage inlets, storm sewer pipe systems, and drainage channels shall also be clearly shown and identified on the drainage area map. Storm sewers shall show and mark station tic-marks at 100-foot intervals. Plan-profile storm sewer or drainage improvement sheet limits and match lines shall be shown with pipes and channels identified.

The drainage area map should show enough topology to easily determine its location within the City.

2.2.2 PLAN-PROFILE SHEETS

2.2.2.1 Inlets

Inlets shall be given the same number designation as the area or sub-area contributing runoff to the inlet. The inlet number designation shall be shown opposite the inlet. Inlets shall be located at or immediately downstream of drainage concentration points. At intersections, where possible, the end of the inlet shall be ten feet from the curb return P.T., and the inlet location shall also provide minimum interference with the use of adjacent property. Inlets in residential areas should be located in streets and alleys so that driveway access is not prohibited to the lots. Inlets located directly above storm sewer lines, as well as laterals passing through an inlet, shall be avoided. Drainage from abutting properties shall not be impaired, and shall be designed into the storm drainage system.

Data opposite each inlet shall include paving or storm sewer stationing at centerline of inlet, size and type of inlet, number or designation, top of curb elevation and flow line of inlet as shown on the construction plans.

2.2.2.2 Laterals

Inlet laterals leading to storm sewers, where possible, shall enter the inlet and the storm drain main at a 60-degree (60º) angle from the street side. Laterals shall be four feet from top of curb to flow line of inlet, unless utilities or storm sewer depth requires otherwise. Pipe soffits elevations should match at all junctions. Laterals shall not enter the corners or bottoms of inlets. Lateral profiles shall be drawn showing appropriate information including the hydraulic gradient and utility crossings.

2.2.2.3 Storm Sewer

In the plan view, the storm sewer designation, size of pipe, and length of each size pipe shall be shown adjacent to the storm sewer. The sewer plan shall be stationed at one hundred- (100) foot intervals, and each sheet shall begin and end with even or fifty- (50) foot stationing. All storm sewer components shall be stationed.

The profile portion of the storm sewer plan-profile sheet shall show the existing and proposed ground profile along the centerline of the proposed sewer, the hydraulic gradient of the sewer, the proposed storm sewer, and utilities which intersect the alignment of the proposed storm sewer. Also shown shall be the diameter of the proposed pipe in inches, and the physical grade in percent. Hydraulic data for each length of storm sewer between interception points shall be shown on the profile. This data shall consist of pipe diameter in inches, the design storm discharge in cubic feet per second, slope of hydraulic gradient in percent, Manning capacity of the pipe flowing full in cubic feet per second, velocity in feet per second, and $V^2/2g$. Also, the head loss at each interception point shall be shown.

Elevations of the flow line of the proposed storm sewer shall be shown at one hundred-(100) foot intervals on the profile. Stationing and flow line elevations shall also be shown at all pipe grade changes, pipe size changes, lateral connections, manholes, and wye connections. All soffit elevations shall match.

2.2.3 Miscellaneous

All plan sheets shall be drawn in ink on 24” x 36” material, to a standard engineering scale, and shall be clearly legible when sheets are reduced to half scale. All drawings shall be signed and sealed by a Professional Engineer registered in the State of Arkansas. After each review, all review comments shall be addressed, additional data incorporated, and drafting of plans completed. Each plan-profile sheet shall have a benchmark shown.

2.3 Check list for Storm Drainage Plans

2.3.1 Drainage Area Map

(1) Normally, use 1” = 200’ scale for on-site, and 1” – 400’ for off-site. Show match lines between any two (2) or more maps.

(2) Show existing sub-areas for alley, street, and off-site areas.

(3) Indicate sub-areas for alley, street, and off-site areas.

(4) Indicate contours on map for on- and off-site, not to exceed two (2) foot contour interval.

(5) Use design criteria as shown in design manual or as specified by the City Engineer.

(6) Indicate city zoning on drainage area.

(7) Show points of concentration and their designations.

(8) Indicate runoff at all inlets, dead-end streets and alleys, or to and from adjacent additions or acreage.

(9) Provide runoff calculations for all areas showing acreage, runoff coefficient, and inlet time.

(10) For cumulative runoff, show calculations.

(11) Indicate all crests, sags, and street and alley intersections with flow arrows.

(12) Identify direction of north to top page or to the left.

2.3.2 Storm Sewers

(1) Diversion of flow from one natural drainage area to another will not be allowed.

(2) Show plan and profile of all storm sewers.

(3) Pipe Material shall conform to the following minimum requirements:

Under Pavement or Roadway Crossings – RCP Class III

ASTM C-76
ASTM C-506

Mains and Laterals – Aluminized Steel Type 2 Corrugated Steel Pipe

ASTM A-929
ASTM A-760

Or Mains and Laterals – Smooth Interior Corrugated Polyethylene Pipe

AASHTO Type "S" (4"-42")
AASHTO Type "D" (48"-60")
ASTM D3350

(4) Use heavier than Class III RCP pipes were crossing railroads, areas of deep fill and areas subjected to heavy loads.

(5) Specify concrete strength for all structures. The minimum allowable is 3,000 psi.

(6) Provide inlets where street capacity is exceeded. Provide inlets where alley runoff exceeds intersection street capacity.

(7) Do not allow storm water flow from streets into alleys.

(8) Do not use high velocities in storm sewer design. A maximum discharge velocity of eight (8) fps at the outfall is required. Velocity dissipation may be necessary to reduce erosion.

(9) Flumes may not be allowed unless specifically designated by the City Engineer.

(10) Provide headwalls and aprons for all storm sewer outfalls. Provide riprap around headwalls where slopes exceed 3:1 or where the velocity out of the storm system exceeds erosive limits set by the City of Arkadelphia.

(11) Discharge flow lines of storm sewers to be two (2) feet above the natural flow line of the channel, unless channel lining is present. Energy dissipation shall be provided when specified by the City Engineer.

(12) Where fill is proposed for trench cut in creeks or outfall ditches, compaction shall be 95% of the maximum density as determined by ASTM D 698.

(13) Investigation shall be made by the engineer to validate the adequacy of the storm sewer outfall to a major stream.

(14) Engineer shall verify the existing outfall capacity to ensure safe and reliable discharge. Provide erosion control facilities with hydraulic data.

(15) Any off-site drainage work or discharge to downstream property will require an easement. Easement shall be sized such that the developed flows can be conveyed within the easement. Submit field notes for off-site easement that may be required (Private development only).

2.3.3 Plan and Profile

(1) Indicate property lines and lot lines along storm sewers, and show easements with dimensions.

(2) If necessary, provide separate plan and profile of storm sewers. The storm drainpipes should also be shown on paving plans with a dashed line, and on sanitary sewer profiles showing the full pipe section.

(3) Tie storm sewer system stationing with paving stations.

(4) Show pipe sizes in plan and profile.

(5) Show hydraulics on each segment of pipe profile to include: Q_{10} and Q_{25}, C , (Manning full flow capacity); S (Slope), V (Velocity), $V^2/2g$ (Velocity Head).

(6) Show curve data for all storm sewer system.

(7) Show all existing utilities in plan and profile. On storm sewer profiles, as a minimum, the sanitary sewer profile will be shown.

(8) Indicate existing and proposed ground line and improvements on all street, alley, and storm sewer profiles.

(9) Show future streets and grades where applicable.

(10) Where connections are made to existing storm sewer, show computations of existing system when available. HGL will be calculated from the outfall to the connection point including the designed flows of the added system. Six inches of freeboard will be provided between the HGL and the inlet curb opening.

(11) Indicate flow line elevations of storm sewers on profile, show pipe slope (percent grade). Match pipe soffits at all junction boxes or inlets.

(12) Intersect laterals at sixty (60) degrees with trunk line.

(13) Show details of all junction boxes, headwalls, storm sewers, flumes, and manholes, when more than one pipe intersects the drainage facility or any other item not a standard detail.

(14) Pipe deflections for directional changes shall be placed at the manufacturers recommendations. Deflections exceeding the manufactures' recommendation will not be tolerated.

(15) Bends in pipes may be used in unusual circumstances with approval by the City. No bend at one location may exceed thirty (30) degrees.

(16) Do not use 90-degree turns on storm sewer outfalls. Provide good alignment with junction structures, manholes, and downstream channels.

(17) Show all hydraulics, velocity head changes, gradients, and computations.

(18) Show water surface elevation of the outfall for Q_{25}.

(19) On all dead-end streets and alleys, show grade out to “daylight” for drainage on the profiles and provide erosion control. Show typical section and slope of “daylight” drainage.

(20) At sags in pavement, provide a positive overflow (paved sidewalk in a swale) to act as a safety path for failure of the storm drain system. Minimum finished floor elevations will be shown on the plat to protect building against flooding should the positive overflow be used.

(21) Where quantities of runoff are shown on plans or profiles, indicate storm frequency design.

(22) Provide sections for road, railroad, and other ditches with profiles and hydraulic computations. Show design water surface on profile.

2.3.4 Laterals

Laterals are defined as minor storm sewer lines that serve the purpose of connecting a single inlet to a larger storm sewer main line. The following is a list of requirements that apply to laterals.

(1) Show laterals on trunk profile with stations.

(2) Provide lateral profiles for laterals exceeding thirty (30) feet in length. Profile short laterals that pass over a sanitary sewer or other profiled utility.

(3) Where laterals tie into trunk lines, place at sixty-degree (60) angles with centerlines. Connect them so that the longitudinal centers intersect.

(4) Minimum pipe diameter shall be eighteen inches (18”).

2.3.5 Inlets and Intakes

(1) Provide inlets where street capacity is exceeded. Provide inlets where runoff from alley causes the capacity of the intersecting street to be exceeded.

(2) Indicate runoff concentrating at all inlets and direction of flow. Show runoff for all stub cuts, pipes, and intakes.

(3) On plan view, indicate size of inlet, lateral size, flow line, top-of-curb elevations, paving station, and inlet designation number.

(4) Use standard curb inlets in streets. Use recessed inlets in divided streets. Use combination inlets in alleys when on a straight run. Do not use grate or combination inlet unless other solution is not available (special situations). These inlets can be seen in Figure 13 of Appendix 2.

(5) Use type "Y" or special "Y" inlets in ditches or swales. Intake structures for a concrete apron shall be constructed around "Y" inlets.

2.3.6 Paving

(1) Provide six (6) inch curb on alleys parallel to creek or channel on creek side of alley.

(2) For a proposed driveway turnout, curb return P.T. must be 10 feet upstream from any existing or proposed inlet or 5 feet downstream of a standard inlet.

(3) Check the need for curbing at all alley turns and "T" intersections. Flatten grades ahead of turns and intersections.

(4) Where inlets are placed in an alley, provide curbing for 10 feet on each side of combination inlet.

2.3.7 Detention

(1) Provide drainage area map and show all computations for runoff affecting the detention basin.

(2) Provide a plot plan with existing and proposed contours for the detention basin and plan for structural measures.

(3) Where earth embankment is proposed for impoundment, furnish a typical embankment section and specifications for fill include profile for the structural outflow structure and Geotechnical report.

(4) Provide structural details and calculations for any item that's not a standard detail.

(5) Provide detention basin volume calculations and elevation versus storage curve.

(6) Provide hydraulic calculations for outflow structure and elevation versus discharge curve.

(7) Provide routings or modified rational determination of storage requirements, (permitted for areas of 300 acres or less)

(8) Fencing may be required around detention area for public safety as determined by the City of Arkadelphia.

2.3.8 Bridges

(1) Clear the lowest member of the bridge by 2 feet above the design water surface, unless otherwise directed by the City Engineering Department.

(2) Show Geotechnical soil boring information on plans.

(3) Provide channel cross sections of the water surface elevations for the design storm immediately upstream and downstream of the structure.

(4) Provide hydraulic calculations on all sections.

(5) Provide structural details and calculations with dead load deflection diagram.

(6) Provide vertical and horizontal alignment.

(7) Show soil erosion protection measures.

(8) Hydraulic calculations for bridges constructed in a designated floodplain shall be performed in HEC RAS. The pre-construction and post construction 100 year floodplain shall be shown.

2.3.9 Open Channels

1) Plan view of channel showing existing and proposed alignment including creek centerline stationing, north arrow, and scale.

2) Profile of existing and proposed creek centerline.

3) Profile of the fully urbanized 100-year and 25-year water surface elevation.

4) Typical cross sections showing dimensions, and the station limits for which they apply.

5) Velocities and discharges for the 100-year and 25-year storms.

6) Limits of temporary erosion protection associated with the construction of the channel needs to be indicated in plan view.

7) Indicate property lines and lot lines along with existing utilities and show easements with dimensions.

8) Include on the construction plans or as a separate report the computations performed in developing the water surface profile.

3.0 HYDROLOGY

3.1 General

The planning, design and construction of drainage facilities are based on the determination of one or more aspects of storm runoff.

Continuous long-term records of rainfall and resulting storm runoff in an area provide the best data source from which to base the design of storm drainage and flood control systems in that area. However, it is not possible to obtain such records in sufficient quantities for all locations requiring storm runoff computations. Therefore, the accepted practice is to relate storm runoff to rainfall, thereby providing a means of estimating the rates, timing and volume of runoff expected within local watersheds at various recurrence intervals.

It is generally accepted that urban development has a pronounced effect on the rate and volume of runoff from a given rainfall. Urbanization generally alters the hydrology of a watershed by improving its hydraulic efficiency, reducing its surface infiltration, and reducing its storage capacity.

For certain small drainage areas (generally less than 300 acres in size), the widely used Rational Method provides a useful means of determining peak discharges. If the engineer wishes to use an alternative design technique, it is recommended that the City of Arkadelphia Engineer be consulted prior to design.

3.2 Rational Method

For smaller basins, less than 300 acres, the Rational Method is recommended for computing the runoff. The Rational formula is:

$$Q = C*I*A \tag{3.1}$$

- Q = Discharge (cfs)
- I = Rainfall intensity (in/hr)
- A = Drainage area (acres)
- C = Runoff coefficient (dimensionless)

3.2.1 The Runoff Coefficient

The runoff coefficient, C-factor, varies according to soil type and land use. It is the ratio of rainfall to runoff. The more impervious and density developed an area is, the higher the value for the C-factor. In storm drain design, the C-factor should always be based on fully developed conditions. This helps to prevent future flooding due to increased development. Zoning ordinances can aid in choosing a C-factor. Table 1 of Appendix 1 shows typical values for the C-factor accepted by the City of Arkadelphia.

3.2.2 Rainfall Intensity

Rainfall intensity (I) is the average rainfall rate in inches per hour, which is considered for a particular basin or sub-basin and is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the time of concentration for all portions of the drainage area under consideration that contribute flow to the point of interest. The frequency of occurrence is a statistical variable, which is established by design standards or chosen by the engineer as a design parameter. The rainfall intensities for various frequencies have been compiled in graphical form on Figure 1 in Appendix 2. The graph plots the rainfall intensity versus time of concentration.

The time of concentration is defined as the longest time, without interruption of flow by detention devices that will be required for water to flow from the upper limit of a drainage area to the point of concentration. This time is a combination of the inlet time, which is the time for water to flow over the surface of the ground from the upper limit of the drainage area to the first storm sewer inlet, and the flow time in the conduit or channel to the point of concentration. The flow time in the conduit or channel is computed by dividing the length of the conduit by the average velocity in the conduit. Typical inlet times can be found in Table 1 of Appendix 1.

Although the basic principles of the Rational Method are applicable to all sizes of drainage areas, natural retention of flow and other interruptions cause an attenuation of the runoff hydrograph resulting in over-estimation of rates of flow for larger areas. For this reason, in development of runoff rates in larger drainage areas, use of the Unit Hydrograph Method is recommended.

3.3 Unit Hydrograph Method

For contributing drainage areas greater than 300 acres the Snyder's Unit Hydrograph Method must be used to calculate the runoff. To do this several parameters must be defined.

3.3.1 Definitions

The computation of runoff quantities utilizing the Unit Hydrograph Method is based on the following equations:

- "T_p" is the lag time, in hours, from the midpoint of the unit rainfall duration to the peak of the unit hydrograph;

- "C_t" and "C_p640" are coefficients related to drainage basin characteristics.

- "L" is the measured stream distance in miles from the point of design to the upper limit of the drainage area;

- "L_{ca}" is the measured stream distance in miles from the point of design to the centroid of the drainage area. This value may be obtained in the following manner:

  Trace the outline of the drainage basin on a piece of cardboard and trim to shape. Suspend the cardboard before a plumb bob by means of a pin near the edge of the cardboard and draw a vertical line. In a similar manner, draw a second line at approximately a 90 degree angle to the first line. The intersection of the two lines is the centroid of gravity of the area.

- "q_p" is the peak rate of discharge of the unit hydrograph for unit rainfall duration in cubic feet per second per square mile;
- "A" is the area in square miles that is draining to the point of design;
- "I" is the rainfall intensity at two hours, in inches per hour, for the appropriate design storm frequency;
- "S_D" is the design storm rainfall in inches for a two-hour period;
- "L_{is}" is the initial and subsequent losses;
- "R_T" is the total runoff in inches;
- "Q_u" is the design storm runoff in cubic feet per second.

3.3.2 Coefficients

Relationships for the lag time, Cp, and C_t were obtained from the Corps of Engineers, Vicksburg District Office. These relationships were published in a report prepared by Blaylock Threet and Associates, Inc. who performed an analysis of gaged data to develop relationships for computing the Snyder's Coefficients. The following relationships were obtained from the Corps of Engineers.

$$T_p = C_t (L\ L_{ca})0.3 \quad (3.2)$$

$$q_p = \frac{C_p 640}{t_p} \quad (3.3)$$

$$Q_p = q_p\ A \quad (3.4)$$

$$S_D = I \times 2 \quad (3.5)$$

$$R_T = S_D - L_{is} \quad (3.6)$$

$$Q_u = R_T\ Q_p \quad (3.7)$$

$$C_t = 10^{(-0.6017(\log(Sst))-1.4037)} \quad (3.8)$$

$$q_p = 10^{(-1.0485(\log(Tp))+2.6047)} \quad (3.9)$$

$$T_p = C_t (LL_{CA})^{0.3} \quad (3.10)$$

$$Qp = 10^{(-1.0485(\log(tp))-2.6047)} \quad (3.11)$$

$$C_p = \frac{Q_p t_p}{640} \quad \text{or} \quad Cp640 = Q_{p*} t_p \quad (3.12)$$

Where:

S_{st} = weighted stream slope (ft/ft)

3.3.3 Initial and Subsequent Losses

Initial loss is defined as the maximum amount of precipitation that can occur under specific conditions without producing runoff. Initial loss values for basins in humid areas of the United States may range from a minimum value of a few tenths of an inch during relatively wet seasons to approximately 2 inches during dry summer and fall months. The initial loss for conditions usually preceding major floods in humid regions normally ranges from about 0.2 to 0.5 inch.

In view of the approximations involved in infiltration estimates, the average loss rate computed from rainfall-runoff data for natural drainage basins will be referred to herein as "infiltration index". Infiltration index is defined as an average rate of loss such that the volume of rainfall in excess of that rate will equal the volume of direct runoff. The infiltration index should range from 0.05 – 0.15 in/hr.

3.3.4 Rainfall

Rainfall data for the various storms can be determined from a combination of the TP-40 and the Hydro-35. Table 3 of Appendix 1 shows values for the rainfall data which has been obtained from the TP 40 and the Hydro-35.

3.3.5 Requirements

The amount of runoff can either computed by hand or by using the HEC-HMS computer program. HEC-HMS can be obtained from the Corps of Engineers. It can be downloaded from their website at www.hec.usace.army.mil\.

4.0 STORM DRAIN AND DRAINAGE APPURTENANCES

4.1 General

This section contains storm drainage design criteria and demonstrates the design procedures to be employed on drainage projects within the City of Arkadelphia. All drainage design shall be submitted both as required on the construction plans and on the forms in Appendix 3.

4.2 Design Frequencies

Table 7 of Appendix 2 shows the appropriate design frequencies to be used for fully developed watershed conditions in storm drain designs in the City of Arkadelphia.

4.3 Runoff Calculations

To begin design of a storm drainage system, it is necessary to compute the amount of runoff that accumulates upstream of the intake structures. To begin to quantify this accumulation the Rational Method can be employed to convert rainfall to runoff. The formula for the rational method is shown in Formula (4-1).

$$Q = C*I*A \tag{4.1}$$

Q = Discharge (cfs)
I = Rainfall intensity (in/hr)
A = Area (acres)

The rainfall intensity is based on rainfall depths obtained from the *Technical Paper Number 40* (TP-40) and the Hydro-35 published by the *National Weather Service.* Figure 1 of Appendix 2 contains a graph of rainfall intensity versus time of concentration.

The time of concentration is the longest time water takes to travel from the upper boundary of the watershed to the design point. This includes both the time to the inlet and the travel time in the storm pipe. Table 1 of Appendix 1 contains the minimum inlet times to be used in the design of the storm drainage system. The time of concentration should be used to obtain an intensity from Figure 1 of Appendix 2. Table 1 also gives values for the runoff coefficient. The discharge for the appropriate frequency storm can then be calculated.

4.4 Street Flow

The next step in the design of the storm drain system is to calculate the flow within the streets.

4.4.1 Definitions

The following street classifications will provide clarity in discussing the requirements and methodology to calculate the flow in streets:

Principal Arterials: Serve the major centers of activity
Minor Arterials: Intended to provide land access
Collectors: Connect local streets in residential neighborhoods
Locals: Provide access to various public and private properties

The street classifications given above are adopted from the Arkadelphia Master Street 1995-2015, as amended.

Straight Crown – A constant slope from one gutter flow line across the street to the other gutter flow line.

Parabolic Crown – A pavement surface shaped in a parabolic from one gutter flow line to the other.

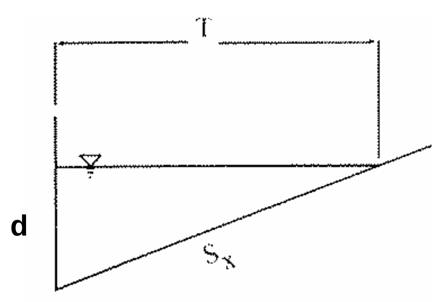
Vertical Displacement Between Gutter Flow Line: Due to topography, it will be necessary at times that the curbs on a street be placed at different elevations.

4.4.2 Calculation of Flow in Streets

The calculation of flow in streets is dependent on street width and shape. Generally, there are two shapes for streets; straight crown and parabolic crown. The straight crowned street can be further subdivided into two types of gutters; uniform and composite. The following discussion covers the methodology used to compute the flow in the street.

Table 7 in Appendix 1 shows the requirements for the design of the roadway drainage. These requirements are based on the *Federal Highway Administration* (FHWA) Circular Number 22.

4.4.3 Uniform Gutter Sections



Uniform Road Section

S_X = Pavement (road) cross-slope (ft/ft) = d/T
T = Total width of flow or spread
Q = Total discharge (cfs)
S_L = Longitudinal slope of road

The runoff in the gutter is generally treated as open channel flow. Therefore, Manning's Equation can be used to calculate the flow or spread in the road section. The following formula is a modified version of the Manning's Equation. It incorporates the geometry of the uniform roadway section.

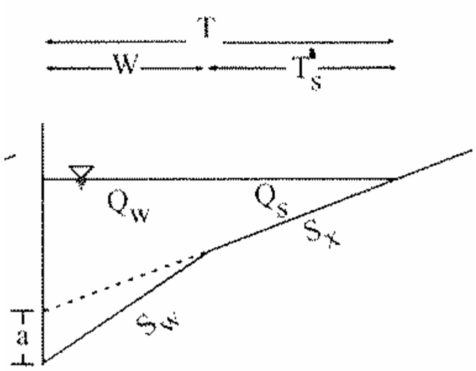
$$Q = \frac{K_C}{n} S_X^{1.67} S_L^{0.5} T^{2.67} \tag{4.2}$$

Where K_C = 0.56
n = Manning's roughness coefficient (0.013 for concrete)
S_L = Longitudinal slope of road (ft/ft)

This equation assumes that the depth of flow, d, is small when compared to the overall topwidth and therefore the topwidth is assumed to be equal to the wetted perimeter. Also, the friction along the curb height is assumed to be negligible when compared to the friction along the spread.

The roadway should be designed such that the spread will be maximized just upstream of the inlet.

4.4.4 Composite Gutter Sections



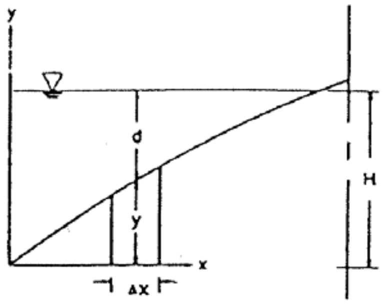
Composite Road Section

Q_W = Flow in depressed section (ft^3/s)
Q_S = Side flow (cfs)
S_W = Gutter Cross Slope (ft/ft)
S_X = Pavement (road) cross-slope (ft/ft)
W = Width of depressed gutter (ft)
T_S = Width of side flow (ft)
T = Total width of flow (ft)
a = Continuous gutter depression (in)

In order to calculate the flow in a composite section the ratio of frontal flow to total gutter flow, E_o, can be calculated using Formula (4.3) in conjunction with Formula (4.2).

$$E_O = 1 \Big/ \left\{ 1 + \frac{S_W}{S_X}\left[\left(1 + \frac{S_W / S_X}{(T/W) - 1}\right)^{2.67} - 1\right]^{-1} \right\} = \frac{Q_W}{QTOTAL} \tag{4.3}$$

4.4.5 Parabolic Street Sections



For residential streets, parabolic sections are often used because they provide a flatter driving surface than uniform sections. However, the flow capacity is less for the parabolic section than the uniform section. The following formulas can be used to calculate the flows and associated spread in a parabolic section.

c. Parabolic

$$y = ((Q/S^{0.5})^{C2})\ /C1 \qquad (4.4)$$

$$Q = (y{*}C1)^{(1/C2)}{*}S^{0.5} \qquad (4.5)$$

$$T = B - (C3\text{-}C4y)^{0.5} \qquad (4.6)$$

$$A = (T \times y)\ /\ 3 \qquad (4.7)$$

$$V = Q/A \qquad (4.8)$$

Where:

y = Flow depth in gutter for one side of street (ft)

Q = Gutter discharge for one side of the street (cfs)

T = Spread for one side of the street (ft)

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

V = Velocity of flow ($ft/_S$)

B = 1/2 of the street width

Table 4,4: Parabolic Roadway Coefficients

	CROWN	C1	C2	C3	C4
	6”	9.2180	0.3405	169	338
26’	* 5”	9.9714	0.3404	169	405.6
	4”	10.9821	0.3404	169	507
	7”	9.1145	0.3418	225	385.7143
31’	6”	9.7396	0.3418	225	450
	* 5”	10.5346	0.3418	225	540
	8”	9.1888	0.3421	324	486
36’	7”	9.7317	0.3421	324	555.4286
	* 6”	10.4020	0.3422	324	648
44’	* 8”	9.9146	0.3433	484	726
	7”	10.4975	0.3433	484	829.7143
	6”	11.2173	0.3433	484	968
* These crown heights shall be used for new developments					
Note: These constants were derived for a Manning’s n of 0.016.					

Alternatively, the nomagraphs included in Appendix 4 can be used as aids in designing parabolic roadway drainage.

4.5 Drainage Inlet Design

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public.

In general inlets should be placed to meet the requirements summarized in Table 7 in Appendix 1. In addition, inlets should be spaced at a maximum distance of 600 feet apart.

4.5.1 Inlet Types

Storm drain inlets are used to collect runoff and discharge it to an underground storm drainage system. Inlets are typically located in gutter sections, paved medians, and roadside and median ditches. Inlets used for the drainage of highway surfaces can be divided into the following three classes:

1. Grate inlets
2. Curb-opening inlets
3. Combination inlets

Grate inlets consist of an opening in the gutter or ditch covered by a grate. Curb-opening inlets are vertical openings in the curb covered by a top slab. Combination inlets consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

4.5.2 Interception Capacity of Inlets on Grade

Inlet interception capacity, Q_j, is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet, E, is the percent of total flow that the inlet will intercept for those conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and, to a lesser extent, pavement roughness. In mathematical form, efficiency, E, is defined by the following equation:

$$E = \frac{Q_j}{Q} \qquad (4.9)$$

Where:

- E = Inlet Efficiency (ft^3/s)
- Q = Total Gutter Flow
- Q_j = Intercepted F=low, (ft^3/s)

Flow that is not intercepted by an inlet is termed carryover or bypass and is defined as follows:

$$Q_b = Q - Q_j \qquad (4.15)$$

Where:

- Q_b = bypass flow, m^3/s (ft^3/s)

In Appendix 4, design charts for inlets on grade and procedures for using the charts are presented for the various inlet configurations. Remember that for locally depressed inlets, the quantity of flow reaching the inlet would be dependent on the upstream gutter section geometry and not the depressed section geometry.

Charts for grate inlet interception have been made and are applicable to all grate inlets tested for the Federal Highway Administration. The chart for frontal flow interception is based on test results which show that grates intercept all of the frontal flow until a velocity is reached at which water begins to splash over the grate. At velocities greater than “Splash-over” velocity, grate efficiency in intercepting frontal flow is diminished. Grates also intercept a portion of the flow along the length of the grate, or the side flow. A chart in Appendix 4 is provided to determine side-flow interception. Chart 5 in Appendix 4 determines the “splash” over velocity”.

A procedure for determining the interception capacity of combination inlets is also presented.

4.5.2.1 Grate Inlets

Grate inlets as a class, perform satisfactory over a wide range of gutter grades. Grate inlets generally lose capacity with increase in grade, but to a lesser degree than curb opening inlets. The principal advantage of grate inlets is that they are installed along the roadway where the water is flowing. Their principal disadvantage is that they may be clogged by floating trash or debris. For this reason, it will be assumed that, on designing grate inlets, their capacity is reduced by fifty percent (50%). For safety reasons, preference should be given to grate inlets where out-of-control vehicles might be involved. Additionally, where bicycle traffic occurs, grates should be bicycle safe.

Grates are effective highway pavement drainage inlets where clogging with debris is not a problem. Where clogging may be a problem, see Table 4-5 where grates are ranked for susceptibility to clogging based on laboratory tests using simulated “leaves.” This table should be used for relative comparisons only.

Table 4-5. Average Debris Handling Efficiencies of Grates Tested

Rank	Grate	Longitudinal Slope	
		0.005	0.04
1	Curbed Vane	46	61
2	30 - 85 Tilt Bar	44	55
3	45 - 85 Tilt Bar	43	48
4	P – 50	32	32
5	P – 50x100	18	28
6	45 - 60 Tilt Bar	16	23
7	Reticuline	12	16
8	P-30	9	20

When the velocity approaching the grate is less than the “splash-over” velocity, the grate will intercept essentially all of the frontal flow. Conversely, when the gutter flow velocity exceeds the “splash-over” velocity for the grate, only part of the flow will be intercepted. A part of the flow along the side of the grate will be intercepted, dependent on the cross slope of the pavement, the length of the grate, and flow velocity.

The ratio of frontal flow to total gutter flow, E_o for a uniform cross slope is expressed by equation 4-10:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - \frac{W}{T}\right)^{2.67} \qquad (4.10)$$

Where:

- Q = total gutter flow, (ft^3/s)
- Q_w = flow in width W, (ft^3/s)
- W = width of depressed gutter or grate, (ft)
- T = total spread of water, (ft)

Chart 2 in Appendix 4 provides solutions of E_o for either uniform cross slopes or composite gutter sections.

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

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \tag{4.11}$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by equation 4.12:

$$R_f = 1 - K_u (V - V_o) \tag{4.12}$$

Where:

- K_u = 0.09 in English Units
- V = velocity of flow in the gutter, ft/s
- V_o = gutter velocity where splash-over first occurs, ft/s (computed from Chart 5 in Appendix 4
- (Note: R_f cannot exceed 1.0)

This ratio is equivalent to frontal flow interception efficiency. Chart 5 in Appendix 4 provides a solution for equation 4.18 which takes into account grate length, bar configuration, and gutter velocity at which splash-over occurs. The average gutter velocity (total gutter flow divided by the area of flow) is needed to use Chart 5 in Appendix 4. This velocity can also be obtained from Chart 4 in Appendix 4.

The ratio of side flow intercepted to total side flow, R_s. is side flow interception efficiency, is expressed by equation 4.13. Chart 6 in Appendix 4 provides a solution to equation 4.13.

$$R_s = 1 / \left(1 + \frac{K_u V^{1.8}}{S_x L^{2.3}}\right) \tag{4.13}$$

Where:

- K_u = 0.15 in English Units
- L = length of grate along gutter, (ft)
- S_x = roadway cross slope
- V = velocity, (ft/s)

The efficiency, E, of a grate is expressed as provided in equation 4.14:

$$E = R_f E_o + R_s (1 - E_o) \tag{4.14}$$

The first term on the right side of equation 4.14 is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow as represented in equation 4.15.

$$Q_j = E\,Q = Q\,[R_f\,E_o + R_s\,(1 - E_o)] \qquad (4.15)$$

4.5.2.2. Opening Inlets

Curb-opening inlets are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. The interception capacity of curb-opening inlets decreases as the gutter grade increases. Consequently, the use of curb-opening inlets is recommended in sags and on grades less than 3%. Of course, they are bicycle safe as well.

Curb-opening inlets are effective in the drainage of pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are less susceptible to clogging and offer little interference to traffic operation. They are a viable alternative to grates on flatter grades where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

Curb opening heights vary in dimension; however, a typical maximum height is approximately 4 to 6 in. The length of the curb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed by equation 4.16:

$$L_T = K_u\,Q^{0.42}\,S_L^{0.3}\left(\frac{1}{nS_x}\right)^{0.6} \qquad (4.16)$$

Where:

- K_u = 0.6 in English Units
- L_T = curb opening length required to intercept 100 percent of the gutter flow, (ft)
- S_L = longitudinal slope
- Q = gutter flow, (ft^3/s)

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by equation 4.17:

$$E = 1 - \left(1 - \frac{L}{L_T}\right)^{1.8} \qquad (4.17)$$

Where:

- L = curb-opening length, (ft)
- L_t = curb opening length at 100% efficiency, (ft)

Chart 7 in Appendix 4 is a nomograph for the solution of equation 4.16 and Chart 8 provides a solution of equation 4.17.

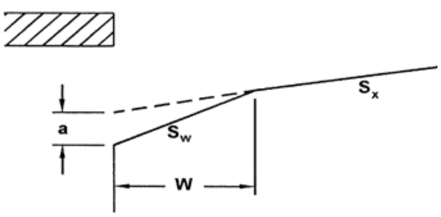
The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in equation 4.16 in place of S_x. S_e can be computed using equation 4.18.

$$S_e = S_x + S'_w E_o \qquad (4.18)$$

Where:

- S_x = roadway cross slope, (ft/ft)
- S'_w = cross slope of the gutter measured from the cross slope of the pavement, S_x, (ft/ft)
- S'_w = (a/[12 W], for W in ft) or = S_w - S_x
- a = gutter depression, (in)
- E_o = ratio of flow in the depressed section to total gutter flow determined by the gutter configuration upstream of the inlet

The following diagram shows the depressed curb inlet for equation 4.24, E_o is the same ratio as used to compute the frontal flow interception of a grate inlet.



As seen from Chart 7 of Appendix 4, the length of curb opening required for total interception can be significantly reduced by increasing the cross slope or the equivalent cross slope. The equivalent cross slope can be increased by use of a continuously depressed gutter section or a locally depressed gutter section.

Using the equivalent cross slope, S_e, equation 4.16 becomes:

$$L_T = K_T Q^{0.42} S_L^{0.3} \left(\frac{1}{nS_e}\right)^{0.6} \qquad (4.19)$$

Where:

- K_T = 0.6 in English Units

Equation 4.17 is applicable with either straight cross slopes or composite cross slopes. Charts 7 and 8 in Appendix 4 are applicable to depressed curb-opening inlets using S_e.

4.5.2.2 Combination Inlets

Combination inlets provide the advantages of both curb opening and grate inlets. This combination results in a high capacity inlet which offers the advantages of both grate and curb-opening inlets. When the curb opening precedes the grate in a “Sweeper” configuration, the curb-opening inlet acts as a trash interceptor during the initial phases of a storm. Used in a sag configuration, the sweeper inlet can have a curb opening on both sides of the grate.

The interception capacity of a combination inlet consisting of a curb opening and grate placed side-by-side, is no greater than that of the grate alone. Capacity is computed by neglecting the curb opening. A combination inlet is sometimes used with a part of the curb opening placed upstream of the grate. The curb opening in such an installation intercepts debris which might otherwise clog the grate and is called a “sweeper” inlet. A sweeper combination inlet has an interception capacity equal to the sum of the curb opening upstream of the grate plus the grate capacity, except that the frontal flow and thus the interception capacity of the grate is reduced by interception by the curb opening.

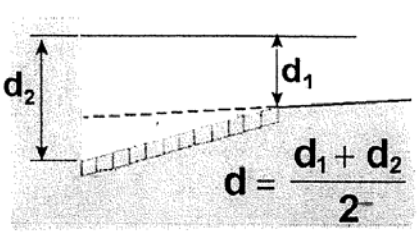
4.5.3 Interception Capacity of Inlets in Sag Locations

Inlets in sag locations operate as weirs under low head conditions and as orifices at greater depths. Orifice flow begins at depths dependent on the grate size, the curb opening height, or the slot width of the inlet. At depths between those at which weir flow definitely prevails and those at which orifice flow prevails, flow is in a transition stage. At these depths, control is ill-defined and flow may fluctuate between weir and orifice control. Design procedures presented here are based on a conservation approach to estimating the capacity of inlets in sump locations.

The efficiency of inlets in passing debris is critical in sag locations because all runoff which enters the sag must be passed through the inlet. Total or partial clogging of inlets in these locations can result in hazardous ponded conditions. Grate inlets alone are not recommended for use in sag locations because of the tendencies of grates to become clogged. Combination inlets or curb-opening inlets are recommended for use in these locations.

4.5.3.1 Grate Inlets in Sags

A grate inlet in a sag location operates as a weir to depths dependent on the size of the grate and as an orifice at greater depths. Grates of larger dimension will operate as weirs to greater depths than smaller grates.



The capacity of grate inlets operating as weirs is:

$$Q_j = C_w P d^{1.5} \quad (4.20)$$

Where:

- P = perimeter of the grate in (ft) disregarding the side against the curb
- C_w= 3.0 in English Units
- d = average depth across the grate; 0.5 (d_1 + d_2), (ft)

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

$$Q_j = C_o A_g (2 g d)^{0.5} \qquad (4.21)$$

Where:

- C_o = orifice coefficient = 0.67
- A_g =clear opening area of the grate, (ft^2)
- g = 32.2 ft/s^2

Use of equation 4.21 required the clear area of opening of the grate. Opening ratios for the grates are given on Chart 9 in Appendix 4.

Chart 9 in Appendix 4 is a plot of equation 4.20 and 4.21 for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

4.5.3.2 Curb-Opening Inlets

The capacity of a curb-opening inlet in a sag depends on water depth at the curb, the curb opening length, and the height of the curb opening. The inlet operates as a weir to depths equal to the curb opening height 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.

Spread on the pavement is the usual criterion for judging the adequacy of a pavement drainage inlet design. It is also convenient and practical in the laboratory to measure depth at the curb upstream of the inlet at the point of maximum spread on the pavement. Therefore, depth at the curb measurements from experiments coincide with the depth at curb of interest to designers. The weir coefficient for a curb-opening inlet is less than the usual weir coefficient for several reasons, the most obvious of which is that depth measurements from experimental tests were not taken at the weir, and drawdown occurs between the point where measurement were made and the weir.

The weir location for a depressed curb-opening inlet is at the edge of the gutter, and the effective weir length is dependent on the width of the depressed gutter and the length of the curb opening. The weir location for a curb-opening inlet that is not depressed is at the lip of the curb opening, and its length is equal to that of the inlet, as shown in Chart 10 in Appendix 4.

The equation for the interception capacity of a depressed curb-opening inlet operating as a weir as:

$$Q_j = (L + 1.8\ W)d^{0.5} \tag{4.22}$$

Where:

- C_w= 2.3
- L = length of curb opening, (ft)
- W = lateral width of depression, (ft)
- d = depth at curb measured from the normal cross slop, (ft), i.e., $d = T\ S_x$

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression. Thus, the limitation on the use of equation 4.22 for a depressed curb-opening inlet is:

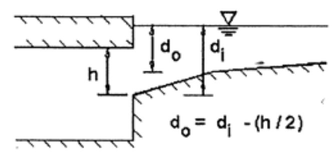
$$d \le h + a/12, \text{ in English Units} \tag{4.23}$$

Where:

- h = height of curb-opening inlet, (ft)
- a = depth of depression, (in)

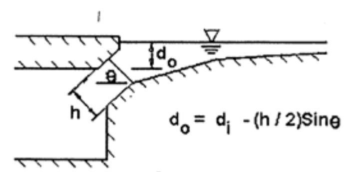
The weir equation for curb-opening inlets without depression becomes:

$$Q_j = C_w\ L\ d^{1.5} \tag{4.24}$$



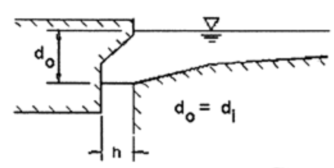
a. Horizontal Throat

Without depression of the gutter section, the weir coefficient, C_w, becomes 3.0, English system. The depth limitation for operation as a weir becomes $d \le h$.



b. Inclined Throat

At curb-opening lengths greater than 12 ft, equation 4.24 for non-depressed inlet produces intercepted flows which exceed the values for depressed inlets computed using equation 4.24. Since depressed inlets will perform at least as well as non-depressed inlets of the same length, equation 4.24 should be used for all curb-opening inlets having lengths greater than 12 ft.



Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity can be computed by equation 4.25a and equation 4.25b. These equations are applicable to depressed and undepressed curb-opening inlets. The depth at the inlet includes any gutter depression.

$$Q_j = C_o\ h\ L\ (2\ g\ d_o)^{0.5} \tag{4.25a}$$

Or

$$Q_j = C_o A_g \left[2g\left(d_i - \frac{h}{2}\right)\right]^{0.5} \qquad (4.25b)$$

Where:

- C_o = orifice coefficient (0.67)
- d_o = effective head on the center of the orifice throat, (ft)
- L = length of orifice opening, (ft)
- A_g = clear area of opening, (ft^2)
- d_i = depth at lip of curb-opening, (ft)
- h = height of curb-opening orifice, (ft)
- g = gravitational constant (32.2 ft/s^2)

The height of the orifice in equations 4.25a and 4.25b assumes a vertical orifice opening. As illustrated in the adjacent figure, other orifice throat locations can change the effective depth on the orifice and the dimension (d_j – h/2). A limited throat width could reduce the capacity of the curb-opening inlet by causing the inlet to go into orifice flow at depths less than the height of the opening.

For curb-opening inlets with other than vertical faces, equation 4.25a can be used with:

- h = orifice throat width, (ft)
- d_o = effective head on the center of the orifice throat, (ft)

Chart 10 in Appendix 4 provides solutions for equations 4.22 and 4.25 for depressed curb-opening inlets, and Chart 11 in Appendix 4 provides solutions for equations 4.24 and 4.25 for curb-opening inlets without depression. Chart 12 in Appendix 4 is provided for use for curb openings with other than vertical orifice openings.

4.5.3.3 Combination Inlets

Combination inlets consisting of a grate and a curb opening are considered advisable for use in sags where hazardous ponding can occur. Equal length inlets refer to a grate inlet placed along side a curb-opening inlet, both of which have the same length. A sweeper inlet refers to a grate inlet placed at the downstream end of a curb-opening inlet. The curb-opening inlet is longer than the grate inlet and intercepts the flow before the flow reaches the grate. The sweeper inlet is more efficient than the equal length combination inlet and the curb-opening has the ability to intercept any debris which may clog the grate inlet. The interception capacity of the equal length combination inlet is essentially equal to that of a grate alone in weir flow. In orifice flow, the capacity of the equal combination inlet is equal to the capacity of the grate plus the capacity of the curb-opening.

Equation 4.20 and Chart 9 in Appendix 4 can be used for grates in weir flow or combination inlets in sag locations. Assuming complete clogging of the grate, equations 4.22, 4.23, and 4.24 and Charts 10, 11, and 12 in Appendix 4 for curb-opening inlets are applicable.

Where depth at the curb is such that orifice flow occurs, the interception capacity of the inlet is computed by adding equations 4.21 and 4.24a as follows:

$$Q_j = 0.67\ A_g\ (2\ g\ d)^{0.5} + 0.67\ h\ L\ (2\ g\ d_o)^{0.5} \qquad (4.26)$$

Where:

- A_g = clear area of the grate, (ft^2)
- g = gravitational constant (ft/s^2)
- d = average depth over the grate, (ft)
- h = height of curb-opening orifice (ft)
- L = length of curb-opening, (ft)
- d_o = effective depth at the center of the curb opening orifice, (ft)

Trial and error solutions are necessary for determining the depth at the curb for a given flow rate using Charts 9, 10, and 11 in Appendix 4 for orifice flow. Different assumptions for clogging of the grate can also be examined using these charts.

4.5.4 Inlet Locations

4.5.4.1 Geometric Controls

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

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

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

4.5.4.2 Inlet Spacing on Continuous Grades

Design spread is the criterion used for locating storm drain inlets between those required by geometric or other controls. The interception capacity of the upstream inlet will define the initial spread. As flow is contributed to the gutter section in the downstream direction, spread increases. The next downstream inlet is located at the point where the spread in the gutter reaches the design spread. Therefore, the spacing of inlets on a continuous grade is a function of the amount of upstream bypass flow, the tributary drainage area, and the gutter geometry. However, the inlets shall not be spaced any more than 600 feet apart.

For a continuous slope, the designer may establish the uniform design spacing between inlets of a given design if the drainage area consists of pavement only or has reasonably uniform runoff characteristics and is rectangular in shape. In this case, the time of concentration is assumed to be the same for all inlets.

4.6 HYDRAULIC DESIGN OF CLOSED CONDUITS

All closed conduits shall be hydraulically designed through the application of Manning's Equation, (non critical flows) expressed as follows:

$$Q = A\ V \tag{4.27}$$

$$Q = \frac{1.486}{n}\ AR^{2/3}\ S_f^{1/2} \tag{4.28}$$

$$R = \frac{A}{P} \tag{4.29}$$

- "Q" is the flow (ft^3/s);
- "A" is the cross sectional area, (ft^2);
- "V" is the velocity of flow in the conduit, (ft/s);
- "n" is the roughness coefficient of the conduit (see Table 2, Appendix 1);
- "R" is the hydraulic radius which is the area of flow divided by the wetted perimeter, (ft);
- "S_f" is the channel slope of the conduit in (ft/ft);
- "P" is the wetted perimeter, (ft).

4.6.1 Velocity in Closed Conduits

Storm sewers should operate within certain velocity limits to prevent excessive deposition of solids due to low velocities, and to prevent invert erosion and undesirable and hazardous outlet conditions due to excessively high velocity. A minimum velocity of 2.5 feet per second and a maximum velocity of 15 feet per second shall be observed. In extreme conditions where the maximum velocity must be exceeded, prior approval must be obtained from the City Engineer.

4.6.2 Roughness Coefficients for Closed Conduits

Roughness coefficients are directly related to construction procedures. When alignment is poor and joints have not been properly assembled, extreme head losses will occur.

Coefficients used in this matter are related to construction procedures, and assume that the pipe will be manufactured with a consistently smooth surface. The minimum roughness coefficient allowed for closed conduit shall be 0.013 regardless of material composition

Minor Head Losses in Closed Conduits

Head losses at structures shall be determined for manholes, junction boxes, wye branches, bends, curves and changes in pipe sizes in the design of closed conduits. Minimum head loss used at any structure shall be 0.10 foot. Properly designed curves may have zero losses.

A. Head losses and gains for wyes and pipe size changes will be calculated by the following formulas:

Where $V_1 < V_2$: Where $V_1 > V_2$:

$$\frac{V_2^{\ 2}}{2g} - \frac{V_1^{\ 2}}{2g} = HL \qquad\qquad \frac{V_2^{\ 2}}{4g} - \frac{V_1^{\ 2}}{4g} = HL \qquad (4.30)$$

and V_1 is upstream velocity and V_2 is downstream velocity. It should be noted that new storm sewer design shall be designed where the receiving pipe velocity increases going downstream. Otherwise a hydraulic jump may occur. Deviations to this requirement shall be handled on a case by case basis by the City Engineer.

B. Head losses and gains for manholes, bends, curves and junction boxes will be calculated as shown in Table 5A and Table 5B in Appendix 1.

1) The basic equation for most cases where there is both upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient "Kj" shown in Table 5A in Appendix 1..

$$hj = \frac{V_2^{\ 2}}{2g} - Kj\,\frac{V_1^{\ 2}}{2g} \qquad (4.31)$$

- "hj" is the junction or structure head loss, (ft);
- "V_1" is the velocity in upstream pipe, (ft/s);
- "V_2" is the velocity in downstream pipe, (ft/s);
- "Kj" is the junction or structure coefficient of loss (Table 5A, Appendix 1)

2) In the case where the inlet is at the very beginning of a line, or the line is laid with bends or obstructions, the equation is revised as follows, without any approach velocity.

$$hj = Kj \frac{V_2^{\ 2}}{2g} \qquad (4.32)$$

5.0 OPEN CHANNELS

5.1 General

Channel design involves the determination of a channel cross-section required to convey a given design flow. The method outlined in the section 9 titled “Floodplain Guidelines” of this manual explains the guidelines for the analysis of an existing channel. This section describes the criteria for the design of proposed channels. The minimum slope for all proposed channels shall be 0.5% unless otherwise indicated by the City of Arkadelphia.

The hydraulic characteristics of improved channels are to be determined through the application of Manning’s Equation. In lieu of Manning’s Equation, HEC-RAS can be used to determine the water surface profile. The City, at its option, can require the use of the HEC-RAS Computer Program. The HEC-RAS Computer Program is available from the U.S. Army Corps of Engineers, Hydrologic Engineering Center, 609 Second Street, Davis, California 95616; or it can be downloaded from their website at www.hec.usace.army.mil/.

The proposed channel shall be designed to carry the 25-year fully urbanized flows plus one foot of freeboard.

A dedicated drainage easement shall be provided to the city for open channels. The easement width shall be the width of the channel from bank edge to bank edge plus an additional fifteen foot access easement on one side of the channel for maintenance access, minimum 20 feet.

5.2 Cross Sections

Figure 2 in Appendix 2 contains typical sections that are to be used in the design of open channels. Due to recent changes in the environmental permitting process, few if any concrete lined channels are being constructed. Although concrete lined channels are hydraulically quite efficient, the Design Engineer is reminded that it may be extremely difficult to obtain the proper permits from the State and Federal authorities. Also, developers are responsible for acquisition of all regulatory agency permits. The developer shall also be responsible for the initial (one time) channel modifications, to include the replacement of trees at the direction of the City of Arkadelphia, and as required by State and Federal agencies. Initial (one time) clearing of debris, small trees, brush, vines, etc. from floodways and floodplains of channels shall be the responsibility of the developer as allowed by the current permitting requirements. In addition, the developer may dedicate the floodway and/or floodplain as a deed-restricted greenbelt area.

All improved channels shall be designed to carry the 25-year flow plus one foot of freeboard, based on the fully urbanized development for each watershed. Adjacent building structures finish floor elevations shall be at least one foot above the 100-year water surface elevation.

Unlined improved channels that contain bends shall be designed such that erosion at the bends is minimized. Erosion protection at bends shall be determined based on the velocity along the outside of the channel bend. Unlined improved channels shall have side slopes no steeper than 4:1 and lined channels shall have side slopes no steeper than 2:1, unless authorized by the City Engineer. A soil analysis shall be performed to determine maximum slope that the soil at the channel improvement site will sustain without failure.

Roadside ditches shall be designed to carry the 25-year frequency runoff below the roadway elevation. The 100-year flood runoff shall remain within the right-of-way.

5.3 Roughness Coefficients

The roughness coefficients that are to be used are shown in Table 4A and Table 4B in Appendix 1. Variations from that which is shown must be approved by the City Engineer.

5.4 Velocity Requirements

Likewise the velocity limits for open-channel flow are given in Table 4A and Table 4B in Appendix 1. The creeks for which the velocity exceeds these limits will have to be protected by some sort of erosion protection or energy dissipater. Otherwise, variations will have to be approved by the City Engineer.

5.5 Channel Drop Structures

The function of a drop structure is to reduce channel velocities by allowing flatter upstream and downstream channel slopes. Sloping channel drops and vertical channel drops are two commonly used drop structures.

The flow velocities in the channel upstream and downstream of the drop structure need to satisfy the permissible velocities allowed for channels (Table 4). The velocities shall be checked for flows produced by the 10-, 50- and 100-year frequency events.

An apron shall be constructed immediately upstream of the chute or stilling basin to protect against the increasing velocities and turbulence which result as the water approaches the drop structure. The apron shall extend at least five (5’) feet upstream of the point where flow becomes supercritical. In no case shall the length of the upstream apron be less than ten (10’) feet.

An apron shall be constructed immediately downstream of the chute or stilling basin to protect against erosion due to the occurrence of the hydraulic jump. The apron shall extend a minimum of ten (10) feet beyond the anticipated location of the hydraulic jump.

The design of drop structures is based on the height of the drop, the normal depths upstream and downstream of the drop structure and discharge.

When used, channel drop structures shall be located near bridges or culverts, as directed by the City Engineer.

5.5.1 Vertical Drop Structures

The approximate height of the drop required to stabilize the hydraulic jump should be determined.

The drop length and the hydraulic jump length of the drop structure should be calculated to determine the length of the downstream apron required to prevent erosion.

5.5.2 Sloping Drop Structures

The location of the hydraulic jump should be determined based on the upstream and downstream flow depths and channel slopes.

The length of the hydraulic jump should be calculated to determine the length of the downstream apron required to prevent erosion.

5.6 Maintenance Access

Access roads shall be provided for the city to all channels to allow for maintenance. The developer shall provide a point of access from the improvement to the channel. Access roads shall have a width of at least ten (10) feet, cross slope no greater 0.02 ft/ft, and longitudinal slope not exceeding 0.12 ft/ft. The access road shall be constructed of six inches of Class 7 Base Course in accordance with Section 303 of the Arkansas State Highway and Transportation Department Standard Specifications for Highway Construction,

The improvement shall contain an access ramps into the channel. The access ramp shall be at least ten (10) feet wide and shall have a vertical grade no steeper than 6:1. The access ramps shall be placed at a minimum of every ¼ mile per development and should be located within a dedicated drainage easement.

For all channels within a floodplain, designated by FEMA, an access road shall be provided along the entire channel length. If the channel depth exceeds four (4) feet an access road will be required on both sides of the channel.

Specific guidelines on the computation of the water surface profile for the existing or proposed channels can be found in the "Floodplain Guidelines" section of this report.

6.0 CULVERTS

6.1 General

The design theory outlined herein is a modification of the method used in the hydraulic design of concrete box and pipe culverts, as discussed in the Federal Highway Administration's Hydraulic Design Series Number 5 titled "Hydraulic Design of Highway Culverts".

The hydraulic capacity of culverts is computed using various factors and formulas. Laboratory tests and field observations indicate that culvert flow may be controlled either at the inlet or outlet. Inlet control involves the culvert cross-sectional area, the ponding of headwater at the entrance, and the inlet geometry. Outlet control involves the tailwater elevation in the outlet channel, the slope of the culvert, the roughness of the surface and length of the culvert barrel.

Channel modifications must comply with Section 5 and Section 9 of this manual and meet FEMA requirements.

Provide one foot of freeboard between the headwater and tailwater surface elevation and the roadway.

6.2 Culverts Flowing with Inlet Control

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of the headwater and entrance geometry, including the barrel shape and cross- sectional area, and the type of inlet edge.

Nomographs for determining culvert capacity for inlet control are shown in Appendix 5. These nomographs were developed by the Division of Hydraulic Research, Bureau of Public Roads, from analysis of laboratory research reported in the National Bureau of Standards Report No. 444, entitled "*Hydraulic Characteristics of Commonly Used Pipe Entrances",* by John L. French, and "*Hydraulics of Conventional Highway Culverts*" by H. G. Bossy. Experimental data for box culverts with headwalls and wingwalls were obtained from an unpublished report of the U.S. Geological Survey.

6.3 Culverts Flowing with Outlet Control

The culvert is designed so that the depth of headwater, which is the vertical distance from the upstream culvert flow line to the elevation of the ponded water surface, does not encroach on the allowable freeboard during the design storm.

Headwater depth, HW, can be expressed by a common equation for all outlet control conditions:

$$HW = H + h_O - L\,(S_O) \qquad (6.1)$$

- "HW" headwater depth in feet from the flow line of the culvet, (ft);
- "H" head or energy required to pass a given discharge through a culvert, (ft);

- "h_O" vertical distance from the downstream culvert flow line to the elevation from which H is measured, (ft)
- "L" length of culvert, (ft);
- "S_O" culvert barrel slope, (ft).

The head, H, is made up of three parts, including the velocity head, exist loss (H_V) and entrance loss (H_e), and a friction loss (H_f). This energy is obtained from the ponding of water at the entrance and is expressed as:

$$H = H_V + H_e + H_f \qquad (6.2)$$

- "H" is head or energy in feet of water;
- H_V" is $\frac{V^2}{2g}$ where V is average velocity in culvert or $\frac{Q}{A}$;
- "H_e" is K_e $\frac{V^2}{2g}$ where K entrance loss coefficient (Table 8, Appendix 1);
- "H_f" is the energy required to overcome the friction of the culvert barrel and expressed as:

$$H_f = \left[\frac{29.2n^2L}{R^{1.33}}\right]\frac{V^2}{2g} \qquad (6.3)$$

Where:

- n coefficient of roughness (see Table 4);
- L length of culvert barrel, (ft);
- V average velocity in the culvert, (ft/s);
- g gravitational acceleration (32.2 ft/s^2);
- R hydraulic radius (Area / Wetted Perimter, ft).

Substituting into the previous equation:

$$H = \frac{V^2}{2g} + K_e\frac{V^2}{2g} + \left[\frac{29.2n^2L}{R^{1.33}}\right]\frac{V^2}{2g} \qquad (6.4)$$

and simplifying:

$$H - \left[1 + K_e + \frac{29.2n^2L}{R^{1.33}}\right]\left[\frac{V^2}{2g}\right] \text{ for full flow.} \qquad (6.5)$$

For various conditions of outlet control flow, h_O is calculated differently. When the elevation of the water surface in the outlet channel is equal to or above the elevation of the top of the culvert opening at the outlet, h_O is equal to the tailwater depth or:

$$h_O = TW$$

If the tailwater elevation is below the top of the culvert opening at the outlet, h_O is the greater of two values: (1) Tailwater, TW, as defined above, or (2) $(d_C + D)/2$, where d_C = critical depth. The critical depth, d_C, for box culverts may be obtained from Appendix 4 or may be calculated from the formula:

$$d_C = 0.315\left[\frac{Q}{B}\right]^{2/3} \quad (6.6)$$

- dc - critical depth for box culvert, (ft);
- Q - discharge, (ft^3/s);
- "B" - bottom width of box culvert, (ft).

- The critical depth for circular pipes may be obtained from Appendix 5, or may be calculated by trial and error. Charts developed by the Bureau of Public Roads may be used for determining the critical depth. Utilize values of D, A and d_C, which will satisfy the equation:

$$\frac{Q^2}{g} = \frac{A^3}{D} \quad (6.7)$$

- "d_C" is the critical depth for culvert, (ft);
- Q is the discharge, (ft^3/s);
- g is the gravitational constant (32.2 ft/s^2);
- A is the cross-sectional area, (ft^2).

The equation is also applicable for trapezoidal or irregular channels, in which instances "D" becomes the channel top width in feet.

7.0 BRIDGES

Once a design discharge and depth of flow have been established, the size of the bridge opening may be determined. The bridge opening shall be designed so that it is in compliance with Section 5 and Section 9.0 of this manual and meets all FEMA requirements.

The basic hydraulic calculations involved in the hydraulic design involve solution of the following:

$$V = \frac{Q}{A} \tag{7.1}$$

- "V" is the average velocity through the bridge, (ft/s);
- "Q" is the flow (ft^3/s)
- "A" is the actual flow area, (ft^2).

$$h_f = K_b \frac{V^2}{2g} \tag{7.2}$$

- "h_f" is the head loss through the bridge in feet;
- "K_b" is a head loss coefficient (Normally .2 to .5);
- "V" is the average velocity through the bridge in feet per second;
- "g" is the gravitational acceleration (32.2 feet per second per second).

As can be seen from the above, the loss of head through the bridge is a function of the velocity head. This methodology is based on a bridge opening with no piers.

Even though the principle fundamental theory is shown above for a simple crossing, the bridge opening shall designed utilizing the latest version of HEC-RAS computer software. The HEC-RAS Computer Program is available from the U.S. Army Corps of Engineers, Hydrologic Engineering Center, 609 Second Street, Davis, California 95616; or it can be downloaded from their website at www.hec.usace.army.mil/.

Input and output data from the software shall be included within the storm drainage calculations as required by Section 2 of this manual.

8.0 DETENTION POND DESIGN

8.1 General

Storm water runoff and the velocity of discharge are considerably increased through development and growth of the City. Prior to the development of land, surface conditions provide a high percentage of permeability and longer time of concentration. With the construction of buildings, parking lots, and subdivisions, permeability and the time of concentration are significantly decreased. These modifications may create harmful effects on properties downstream.

The intent of detention pond design in the City of Arkadelphia storm water management policy is to control flood discharges for ultimate fully urbanized watershed development conditions without increasing peak discharges above the peak discharges for natural watershed or pre-development conditions.

8.2 Applicable Design Criteria

Preservation of existing floodplains along the major creeks shall be strongly encouraged; therefore, regional detention/retention ponds may be used to maintain the natural floodplain environment. On-site detention/retention ponds may be allowed for developments, and in such cases, the detention/retention pond shall be a minimum size of one acre, where feasible, to allow for proper maintenance, side slopes and outlet work operation. Detailed engineering studies of the entire watershed basin shall be required to evaluate the timing of hydrographs from regional and on-site facilities.

All detention pond designs shall be performed by an engineer registered in the State of Arkansas. The following criteria shall serve as minimum requirements for detention pond design:

Storage will be required for any commercial development of one acre or more in size. In addition, storage will be required for any parking lot containing 5,000 square feet of paved surface. The requirement can be satisfied by providing an off-site storage facility.

8.3 Methodology

Criteria for differential runoff and detention guidelines are set out below to attempt to decrease the possible effects of development on downstream properties due to increased runoff.

Differential runoff evaluation consists of determination of rates of runoff before and after development, determination of required volume of detention and verification of adequacy of discharge and control structures.

8.3.1 Differential Runoff Rates

Differential runoff rates shall be evaluated by the rational formula. Differential runoff rates shall be evaluated by equation:

$$R = (Rd - Ru) \tag{8.1}$$

Where R = differential runoff rate
Rd = C .I. factor for developed conditions
Ru = C.I. factor for undeveloped conditions
Determine " C" value (Table 1, Appendix 1).

Use Figure 1, Appendix 2 to find time of concentration and determine intensity (I).

8.3.2 Volume of Detention

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

A. Volume of detention for projects of less than 50 acres shall be evaluated by the "simplified volume formula."

B. Volume of detention for projects 50 acres or greater but less than 300 acres may be evaluated either by the "unite hydrograph method".

C. For projects larger than 300 acres, the developer shall submit his proposed method of evaluation for the sizing of the retention basin or detention basin to the City of Arkadelphia. The method will be evaluated for a professional acceptance, applicability and reliability. No detail review for projects larger than 300 acres will be rendered before the method of evaluation of the retention or detention basin is approved.

D. Other analytical methods of evaluation of volume of detention require approval by the City of Arkadelphia.

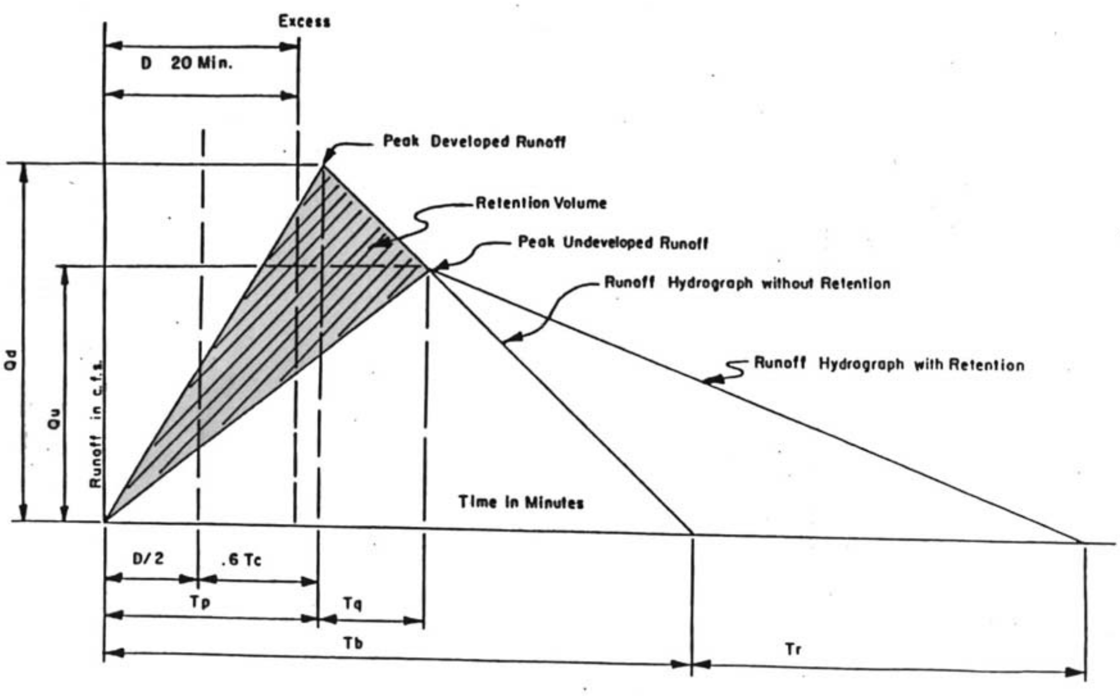
8.3.3 Simplified Volume Formula

Total volume of detention shall be computed by the equation:

$$V = R \text{ x } A \text{ x } T_c \text{ (minutes) x 60 (sec./min.)} \tag{8.2}$$

- V = Total volume of retention
- R = Differential runoff rate
- A = Area of the project and the acres
- T_c = Time of concentration from Figure 1, Appendix 2.

8.3.3.1 Graphic Representation



For purpose of further analysis, the simplified volume formula may be represented by a triangular synthetic hydrograph as shown in Figure 8.3 with the following elements:

- T_d = base time of hydrograph for developed project without retention
- T_d = 60 minutes
- T_p = Time of peak runoff for developed project
- T_p = 20 minutes
- Q_d = total peak runoff for developed project, (ft^3/s)
- Q_d = A x RD (see 8.1)
- Q_u = total peak runoff of unimproved project in (ft^3/s)
- Q_u = A x Ru (see 8.1)
- A = Total Area of Project, (Acres)
- T_Q = Assumed time of peak differential for unimproved project
- T_r = Assumed precedent recedence time differential for discharge at rates no greater than unimproved condition
- $T_r = (60\ Q_d/Q_u) - 60$
- V = Volume of detention
- V = $(Q_d - Q_u)$ x 39 (min.) x 60 (sec./min.)

8.3.4 Modified Rational Hydrograph Method

This is a modification of the Unit Hydrograph Method of hydrologic evaluation simplified to reflect features of present practice and some elements of topographic characteristics, concentration patterns, and routing. Figure 5-1 illustrated the elements of the modified hydrograph. Steps to develop the hydrograph are as follows:

1. Determine the time of concentration for the project by the use of Figure 1, Appendix 2.

   For analysis of large improved channels, time of travel for overland flow and channel are to be analyzed to determine reasonable (T_c) time of concentration.

2. Determine time of peaking by equation:

   $$T_p = D/2 + .6\,T_c$$

   Where T_c = time of peak discharge of developed project in minutes

   D = 20 min. – storm duration in minutes

3. Determine the base time of the hydrograph without detention, by equation.

4. Determine the base time of the hydrograph with detention by equation.

   $$T_r = T_b\,(\,(Q_d/Q_u) - 1)$$

   Where T_r = Additional time required for discharged at a rate no greater than that of the undeveloped condition.

   Q_d = total peak runoff of the improved project, (ft^3/s)

   Q_d = A x RD (see 8.1)

   Q_u = total runoff of unimproved project, (ft^3/s)

   Q_u = A x R_u (see 8.1)

5. Determine the required volume of detention by equation:

   $$V = ½\,(Q_d - Q_u)\;T_b \times 60$$

The simplified approach shown above is for use with a single watershed within a development. Complex, multiple, watersheds within a development, where the contributing drainage areas are larger than 300 acres, a software program to determine the unit hydrograph and flows should be used to route the inflow hydrograph through the watersheds to the storage area. The Corp's of Engineers HEC-HMS software is the preferred method for this routing, however, other programs can be used with the approval from the City of Arkadelphia.

8.4 Design Requirements

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. Conditions for general location of detention facilities are identified in the following sections.

Analysis of all elements of design is always performed by the engineer of record. The following outline is provided to ascertain that certain critical elements of design are in workable compliance to the aims of design. For all projects submit routing calculations of tabulated proof of adequacy for tributary runoff or detention, it is recommended that verification be made of: (a) volume of detention for the total project, (b) tributary (Q) peak runoff to the basin, (c) balance maximum outflow rate from the low-flow structure, (d) ratios of in flow to out flow rates, (e) detention dikes or dams, (g) safety features, (h) maintenance features.

In addition, routing calculations, (projects greater than 50 but less than 300 acres) shall be submitted in legible tabulated form. Proof of adequacy of volume of detention and sizing computations for low-flow structure shall also be submitted. Features of stability and safety may also need to be documented if the scope of the project requires special attention in this area of design.

Projects over 300 acres in area shall provide documented verification of adequacy according to scope and complexity of design.

8.4.1 Dry Reservoirs

Wet weather ponds or dry reservoirs shall be designed with proper safety, stability, and ease of maintenance facilities, and shall not exceed four (4) feet in depth. Maximum side slopes for grass reservoirs shall not exceed maximum side slopes for grass reservoirs shall not exceed one (1) foot vertical for three (3) feet horizontal (3:1) unless adequate measures are included to provide for the above noted features. In no case shall the limits of maximum water surface elevation be closer than thirty (30) feet horizontally from any building and less than one (1) foot vertically below the lowest sill or floor elevation.

The entire reservoir area shall be seeded, fertilized, mulched, sodded or paved as required prior to final plat approval or issuance of certificate of occupancy. Overflow areas shall be protected against erosive velocities.

8.4.2 Open Channels

Normally permitted open channels may be used as detention areas provided that the limits of the maximum design water surface elevation are not closer than thirty (30) feet horizontally from any buildings, and less than one (1) foot below the lowest sill or floor elevation of any building. No detention will be permitted within public road rights-of-way unless approval is given by the City of Arkadelphia. Maximum depth of retention and open channels shall be four (4) feet. Minimum flow line grade shall be 0.5 percent for grass or untreated bottoms or 0.3 percent for paved channels.

For trapezoidal sections, the maximum side slopes of the detention of the channel shall not exceed one (1) foot vertical for three (3) feet horizontal (3:1). For design of stability, and ease of maintenance shall be observed by the design engineer.

The entire reservoir area of the open channel shall be seeded, fertilized, mulched, sodded or paved as required in elevations resulting from channel detention shall not adversely effect adjoining properties.

8.4.3 Permanent Lakes

Permanent lakes with fluctuating volume controls may be used as retention areas provided that the limits of the maximum water surface elevation are no closer than thirty (30) feet horizontal from any building and less than one (1) foot below the lowest sill or floor elevation of any building.

Maximum side slopes for the fluctuating area of permanent lakes shall be one (1) foot vertical to three (3) feet horizontal (3:1) unless provisions are included for safety, stability, and ease of maintenance.

Suggested maximum fluctuation from permanent pool elevations to maximum water surface elevation shall be three (3) feet. Each design has its own particular parameters in relation to the adjoining topography.

Special consideration is suggested to safety and accessibility to small children in design of permanent lakes in residential areas. It is suggested that the minimum of twenty-five percent (25%) of the permanent pool area be no less than 10 feet. Allowances for silting under denuded soil conditions (during construction) for a period no less than one year is also recommended.

The entire fluctuating area of the permanent reservoir shall be seeded, fertilized and mulched, sodded or paved. Any area susceptible to or designed as overflow by higher design intensity rainfall (100-year frequency) shall be sodded or paved, depending on the design velocities.

An analysis shall be furnished of any proposed earthen dam construction soil. A boring of the foundation for the earthen dam may be requested by the City of Arkadelphia. Earthen dam structures shall be designed by a licensed Professional Engineer in the state of Arkansas.

8.4.4 Parking Lots

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

The minimum freeboard and the maximum design water surface elevation to the lowest sill or floor elevation shall be one (1) foot.

8.4.5 Control Structures

The 100-year frequency storm is to be used to determine the volume of detention storage required. In addition, the outlet structure shall be designed such that peak discharges for the fully urbanized development are not increased for the pre-developed storm frequencies of the 5-year, 10 year, 25 year, 50 year, and 100-year storm event.

Detention facilities shall be provided with obvious and effective control structures. Plan view and sections of the structure with adequate details shall be included in plans.

The maximum discharge shall be designed to take place under total anticipated design-head conditions.

Sizing of the low-flow pipe shall be by inlet control or hydrologic control or hydrologic gradient requirements. Low-flow pipes shall not be smaller than twelve (12) inches in diameter to minimize maintenance and operating problems except in parking lot and roof retention where minimum size of openings shall be designed specifically for each condition. A bar-screen on a minimum 3:1 slope to reduce blockage by debris is suggested on the flow-pipe.

An emergency spillway or overflow area shall be provided at the maximum 100-year pool level. Where the outflow structure conveys flow through the embankment in a conduit, the conduit shall be reinforced concrete designed to support the external loads. The conduit is to withstand the internal hydraulic pressure without leakage under full external load or settlement, and must convey water at the design velocity without damage to the interior surface of the conduit.

The outflow structure shall discharge flows into the natural stream or unlined channels at a non-erosive rate in accordance with the requirements of this design manual.

Earth embankments used to impound required detention volume shall be constructed according to specifications for fill and shall be based on a Geotechnical investigation for the site. The Geotechnical investigation shall be performed by a registered Professional Engineer in the state of Arkansas, with an emphasis in geotechnical analysis and shall include, as a minimum, the type of material on-site, water content, liquid limit, plasticity index and desired compaction.

8.5 Maintenance

Detention facilities, when mandatory, 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. Silt and debris connected with early construction shall be removed periodically from the detention area and control structure in order to maintain a close to full storage capacity.

Maintenance of detention facilities is divided into two components. The first is long-term maintenance which involves removal of sediment from the basin and outlet control structure. Maintenance to an outlet structure is minimal due toe initial design of permanent concrete or pipe structures. Studies indicate that in developing areas, to be needed once every 5 to 10 years. The City is responsible for long-term maintenance.

Short-term maintenance or annual maintenance is the second component and is the responsibility of the property owner or association. The items considered short-term maintenance are as follows:

1. Minor dirt and mud removal
2. Outlet cleaning
3. Mowing
4. Herbicide spraying
5. Litter Control

The responsibility of all maintenance of the detention facilities and subdivision projects shall remain with the developer until the project has been approved for final platting. Upon final plat approval, the short-term maintenance responsibility shall be vested in the trustees indenture of trusts shall clearly indicate resident responsibility for maintenance in cases of projects without common ground.

The responsibility of maintenance of the retention 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 of the project, the maintenance of detention facilities shall be bested with the owner of the project.

If the trustees or owner fail to provide reasonable degree of maintenance and the facilities become inoperable or ineffective, as determined by the City of Arkadelphia, remedial work will be performed by the City of Arkadelphia and assess the trustees or owner for the cost of repair and maintenance.

Access must be provided for maintenance in detention basin design for periodic desilting and debris removal. Basins with permanent storage must include dewatering facilities to provide for maintenance. Detention basins with a drainage area of 300 acres or more must include a desilting basin in the upstream pool area.

Security fencing with a minimum height of 6 feet shall encompass the detention storage area if the velocity, depth, or slopes create a potentially dangerous condition as determined by the City Engineer. The fence shall be designed so as to allow access for maintenance and so as not to restrict storm water flow into or out of the detention basin.

A maintenance equipment access ramp shall be provided. The slope of the ramp shall not exceed 6:1 and the minimum width shall be 10 feet.

8.5.1 Easements

Two types of easements shall be provided in plans for detention facilities:

A. Maintenance Easement:

All detention reservoirs with the exception of parking lot and roof detention shall be enclosed by a maintenance easement for public use. The limits of the easement shall extend ten (10) feet beyond the maximum anticipated flooding area.

B. Drainage Easement:

A minimum ten (15) foot wide drainage easement shall be provided within the reservoir area, connecting the tributary pipes and the discharge system for possible future elimination of detention.

8.6 Calculations

Computations performed for detention pond sizing shall be submitted in a format consistent with Form 6 in Appendix 3.

9.0 FLOODPLAIN GUIDELINES

9.1 General

The purpose of this section is to give guidelines on the modifications and management of the floodplains within the City of Arkadelphia. The requirements given here apply to the study of existing water courses and the definition of the associated floodplains. Any variances to these requirements must be reviewed and approved by the City of Arkadelphia.

9.2 General Guidelines

Any developer or individual that wishes to modify a floodplain, be it 100-year or some other frequency, or floodway, must prepare and submit to the City of Arkadelphia a report that explains the nature of this modification. This report must be based on the fully-urbanized 100-year floodplain. This report needs to be reviewed and approved by the City Engineer before the modifications can take place. This report shall include the revised mapped 100-year fully-urbanized floodplain. It shall also include any revised hydrology that has been computed.

In addition, anyone wishing to modify the floodplain or floodway must comply with all of the regulations set forth in the National Flood Insurance Program (NFIP). That is to say, a Letter of Map Revision must be submitted to the Federal Emergency Management Agency, to incorporate changes to the floodplain. In some cases a Conditional Letter of Map Revision may be required prior to construction. All of FEMA's requirements regarding LOMR's and CLOMR's must be met.

In order for the construction plans to be approved by the City there must be one (1) foot of freeboard between the 100-year fully-urbanized water surface elevation and the adjacent finish floors of any structure.

There shall not be any rise in the floodplain or floodway base flood water surface elevation. Encroachment shall be limited to the floodplain fringe. Floodway encroachment will not be permitted. Construction within or modification to the floodplain or floodway will only be permitted if FEMA requirements are met and the base flood water surface elevation is not modified. In addition the modifications within the floodplain or floodway must maintain the pre-developed storage capacity of the floodplain.

9.3 Hydrology

Any hydrologic study performed within the City of Arkadelphia must comply with the guidelines set forth in this manual. This includes submitting to the City a hydrologic work map that includes watershed boundaries, and all other hydrologic parameters.

9.3.1 Rational Method

For areas less than 300 acres, the rational method can be used to compute runoff. This method is described in Section 3 of this manual. Table 1 in Appendix 1, shows typical values for C-factors and shows some typical values for the inlet time. The intensity-duration-frequency curve for Arkadelphia is included in Figure 1, Appendix 2.

9.3.2 Unit Hydrograph Method

For areas greater than 300 acres the Snyder's Unit Hydrograph Method is to be used. Section 3 of this manual details the computational procedure for this method.

Complex, multiple, watersheds within a development, where the contributing drainage areas are larger than 300 acres, a software program to determine the unit hydrograph and flows should be used to route the inflow hydrograph through the watersheds to the storage area. The Corp's of Engineers HEC-HMS software is the preferred method for this routing, however, other programs can be used with the approval from the City of Arkadelphia.

9.4 Hydraulics

Any modification to a floodplain within the City of Arkadelphia requires a hydraulic study, performed by a Professional Engineer licensed in the State of Arkansas, to determine the floodplain elevations. All hydraulic studies along water courses must comply with FEMA's guidelines; however, there are a few additional requirements that must be met before the flood study is approved and modifications of the floodplain is allowed.

First of all, the flood study should map the 100-year fully developed floodplain boundary. Lots and other substantial structures adjacent to the watercourse must be constructed one (1) foot above the 100-year fully urbanized water surface elevation. In addition, a Letter of Map Revision Request is required for all developments that modifies the floodplain. A conditional Letter of Map Revision is required only in those instances that FEMA requires one. All other FEMA guidelines must be met.

The Corps of Engineers HEC-RAS computer program shall be used to compute the water surface elevation. To do so, cross sections along the watercourse must be surveyed no greater than 400 ft. apart for tangent sections of channel and no greater than 200 ft. apart for curvilinear sections of channel unless otherwise approved by the City of Arkadelphia. Roughness values shall be determined based on the table in Table 4, Appendix 1. A printout of the computer model as well as an electronic copy of the HEC-RAS files shall be submitted with the Hydraulic Report for any work proposed in the floodplain

Table 4, Appendix 1, shows the maximum permissible velocities that are to be allowed in the channel. Velocities above that which is shown in Table 4 must be reduced or approved by the City of Arkadelphia.

10. EROSION CONTROL

10.1 General

This section summarizes the requirements set forward by the Environmental Protection Agency's (EPA's) Clean Water Act, administered in the state by the Arkansas Department of Environmental Quality (ADEQ). The Arkansas Department of Environmental Quality can be contacted at 501-372-0688 or www.adeq.state.ar.us

Further details regarding these requirements can be found in the *Federal Register,* Volume 63, Number 128 dated July 6, 1998. Details for some standard erosion control devices are included in Figures 3 through 10 in Appendix 2. A copy of the permit (Permit No. ARR10A000) application, Notice of Intent, and Notice of Termination are included in Figure 11 and 12, respectively, in Appendix 3.

Any development within the City of Arkadelphia must comply with both Federal and State requirements regarding erosion protection.

10.2 National Pollutant Discharge Elimination System (NPDES) Requirements

As part of the Water Quality Act of 1987, storm water discharges associates with industrial activity from a point source to waters of the United States is unlawful, unless authorized by an National Pollutant Discharge Elimination System (NPDES) permit. Construction activities which disturb an area greater than 5 acres by grading, clearing, grubbing or other construction activity is defined as an industrial activity, subject to the requirement of an NPDES permit. In order to effectively manage the permit process, the EPA has produced a general permit for construction activities, which defines specific conditions and requirements to be met as part of the permit. The general permit establishes the procedures required for proper coverage, the requirement for a Storm Water Pollution Prevention Plan (SWPPP) and requirements for termination of permit coverage.

In addition to NPDES permits for construction activities, large and medium size municipalities are required to obtain NPDES permits for their Municipal Separate Storm Sewer System (MS4) to control storm water outflow into waters of the United States. This permit will require local jurisdictions to take an active role in monitoring and controlling pollution due to storm water runoff from a variety of sources including construction activities. Therefore, in addition to meeting the requirements for the general permit, the site operator is obligated to contact the local jurisdiction to determine local requirements that may be required in addition to general permit coverage for discharge of industrial storm water runoff.

As noted above, all construction activities that disturb 5 acres or more land area or are a part of a common development or plan of sale is subject to the NPDES permit requirement. Failure to abide by the terms of the general permit or failure to develop and implement a site-specific NPDES permit is a violation of federal law, which can subject the owner or operator to severe fines or imprisonment.

Compliance with the requirements of the general permit consists of four major components: determination of eligibility, preparation and implementation of a Storm Water Pollution Prevention Plan, submission of the Notice of Intent, and submission of the Notice of Termination. Note that the Storm Water Pollution Prevention Plan (SWPPP) is prepared in conjunction with the construction documents for the site and before the submission of the Notice of Intent (NOI) to the EPA.

10.3 Eligibility Determination

Permittees are only eligible for coverage under the Construction General Permit (CGP) provided that their storm water discharges and storm water discharge-related activities do not adversely impact federally listed endangered or threatened species or critical habitats. Applicants are required to conduct an assessment of the impacts of their storm water discharges and storm water discharge-related activities on endangered and threatened species and critical habitat. Addendum A of the Construction General Permit provides detailed instructions to assist applicants in conducting an assessment and pursuing formal consultation with federal wildlife protection agencies if necessary. The process is briefly described below, however, operators are strongly advised to refer to addendum A of the permit.

1. *Determine if the construction site is found within designated critical habitat for listed species.* Operators may contact the U.S. Fish and Wildlife Service for information on critical habitat. If the project site is located within critical habitat, the operator must consider impacts to critical habitat when following the remaining steps.

2. *Determine if listed species are located in the county(ies) where the construction activity will occur.* Operators can obtain a county-by-county list of federally listed threatened and endangered species from the U.S. Fish and Wildlife Service (FWS) office listed above. After a review of the information from the FWS, if no listed species are located in the proposed site's county(ies) (or if the site's county is not listed), and the construction site is not located within designated critical habitat, the operator is eligible for CGP coverage without further inquiry into the presence of or effect to listed species. If it is determined that listed species are located in the site's county, the operator must proceed to step 3.

3. *Determine if any federally listed endangered or threatened species may be present in the project area*) refer to Addendum A for a description of what constitutes the project area). Addendum A provides several options that the operator may conduct to satisfy this requirement. The operator may be able to obtain further guidance from the local Fish and Wildlife Service office on which option is most appropriate for the operator to conduct given the site location and listed species occurring in the country. If through the operator's assessment no species are determined to exist in the project area, the operator is eligible for CGP coverage (not that due to the large number of requests, the FWS typically will not conduct its own assessment or provide any formal "approval" or written documentation at this stage in the process). If listed species are found in the project area, applicants must indicate the location and nature of this presence in the SWPPP and proceed to the following step.

4. *Determine if listed species or critical habitats are likely to be adversely affected by the construction activity's storm water discharges or storm water discharge-related activities.* The scope of effects to consider will vary with each site. If it is unclear whether the proposed project is likely to adversely a listed species or

critical habitat, the local Fish and Wildlife Service office should be contacted for assistance. If it is determined that adverse effects are likely, the operator must follow step 5.

5. *Determine if measures can be implemented to avoid any adverse effects.* In some cases measures may be able to be implemented to avoid or eliminate the likelihood of adverse effects prior to applying for permit coverage. These measures may involve relatively simple changes to the construction activity such as rerouting a storm water discharge to bypass an area where species are located, relocating BMPs, or by changing the “footprint” of the construction activity. The operator may wish to contact the FWS to find out what measures might be suitable to avoid or eliminate the likelihood of adverse impacts to listed species and/or critical habitat. If measures are adopted (or changes are made to the proposed construction activity) that would avoid or eliminate adverse effects, the operator must abide by those measures during the course of permit coverage. If measures to avoid the likelihood of adverse effects are not available, the applicant must proceed to the step 6.

6. *If a determination has been made that construction activity will affect endangered species, then the applicant must receive clearance from Fish and Wildlife Services and/or the National Marine Fisheries Service to obtain permit coverage.* In this case, the operator must initiate formal or informal Endangered Species Act consultation with the Fish and Wildlife Service. This process is described in detail in Addendum A.

After permit eligibility has been determined, preparation of the Storm Water Pollution Prevention Plan may begin.

10.4 Preparation of the Storm Water Pollution Prevention Plan

The Storm Water Pollution Prevention Plan (SWPPP) is the document(s) that defines the measures to be employed to prevent the release of pollution from the construction site. The SWPPP consists of two components, a narrative description of the project, and a drawing of the site with proposed improvements and pollution reduction methods shown. The SWPPP shall be included within the construction documents and reviewed for compliance by the City of Arkadelphia for all developments either for or constructed within the planning jurisdiction of the City of Arkadelphia. The SWPPP shall be prepared by a Profession Engineer licensed in the state of Arkansas.

The SWPPP identifies the techniques that the operator will use to reduce site erosion and sediment loss and manage construction-related wastes. It identifies the maintenance procedures that the operator will perform to preserve the efficiency of the techniques used. The SWPPP must clearly describe the control measures, the timing and sequence of implementation, and which permittee (contractor) is responsible for implementation of the control measures.

Unlike many construction documents, the SWPPP is very likely to change during the course of construction due to variations in construction techniques and/or site conditions. These modifications should be made by the original preparer of the SWPPP or someone else experienced in the design of erosion and sediment control systems in order to maintain the effectiveness of the original SWPPP design.

The SWPPP is not submitted to the ADEQ as part of the NOI; instead it must be available onsite or nearby for inspection by EPA or ADEQ personnel, state and/or local jurisdiction staff, and the public upon request.

In preparing the SWPPP the following information must be presented (detailed explanations of each item are presented in Section 6):

1. Site Description
   - A. Description of Construction
   - B. Project Sequencing
   - C. Total and Disturbed Areas of Site
   - D. Estimate of Runoff Coefficient (C) Before and After Construction
   - E. General Location Map
   - F. Site Map With Drainage and Layout Information
   - G. Location and Description of Any Discharge Associated with Industrial Activity Other Than Construction
   - H. Name of Receiving Waters
   - I. If Applicable, Information on Wetlands
   - J. Copy of Permit Requirements
   - K. If Applicable, Information on Listed Species or Critical Habitat Affected by Activity
   - L. If Applicable, Information on Historic Places Affected by Activity

2. Controls To Be Used Onsite
   - A. Construction, Erosion and Sediment Controls
     1. Stabilization Practices
     2. Structural Practices
   - B. Post-Construction Storm Water Management Controls
     1. Flow and Pollutant Reduction Practices
     2. Velocity Dissipation Devices
   - C. Other Controls
     1. Solid Material Discharge
     2. Offsite Sediment Tracking
     3. Compliance with State and Local Requirements for Waste Disposal
     4. Construction and Waste Materials Storage
     5. Pollutant Sources from Construction Support Activities
     6. Protection Measures for Listed Species or Critical Habitat
   - D. Compliance with State and Local Requirements for Sediment and Erosion Control

3. Maintenance Procedures for Control Measures

4. Inspection Requirements

5. Planned Non-Storm Water Discharges

Outlines for both the narrative and the drawings are included in Section 6 of this manual to ensure that each of the above issues is addressed in the SWPPP. A form is also presented for the narrative portion to simplify preparation of the document. An example SWPPP can be downloaded from ADEQ at the following web site:

www.adeq.state.ar.us/water/npdes/stormwater/construction.htm

10.5 Notice of Intent

The Notice of Intent (NOI) is the primary document used by the ADEQ to monitor and enforce compliance with the NPDES permitting requirements. The NOI is to be submitted after preparation of construction plans and SWPPP at least 48 hours prior to the beginning of construction activities at the site. Unless notified by ADEQ during the 48 hour period after submission, the NOI is considered acceptable and construction activities, including implementation of the SWPPP, can proceed under assumed coverage of the NPDES general permit.

The operator(s) of the site is required to submit the NOI and is ultimately responsible for the effective reduction of pollution and sediment loss from the site. An NOI must be in place for the site throughout the time the site is in a disturbed condition (except when a site has been temporarily stabilized and transferred to the homeowner).

A copy of the NOI form is included in Appendix C of this manual. Detailed instructions for completion of the NOI can be within the permit application. Additional copies can be obtained from the ADEQ. Completed NOIs should be submitted to ADEQ at the designated address shown on the back of the form.

10.6 Notice of Termination

The Notice of Termination (NOT) provides notification to ADEQ that the site has been stabilized in accordance with the requirements of the general permit and that construction on the site is completed or another operator has assumed control (and submitted an NOI). Upon submission of the NOT, the operator loses the authority to discharge storm water under the conditions of the general permit.

As stated in the general permit, the NOT for the site must be submitted within 30 days of one or more of the following conditions being met: 1) final stabilization of the site is achieved; 2) another operator has taken control over all areas that have not been finally stabilized, or 3) for residential construction only, temporary stabilization has been completed and the residence has been turned over to the homeowner.

Final stabilization means that either: 1) all soil disturbing activities have been completed and a uniform perennial vegetative cover with a density of 70% of the native background vegetative cover has been established for all areas outside of paved areas or building limits, or equivalent other stabilization measures (riprap, gabions, geotextiles, etc.) have been employed; 2) for individual lots in residential construction projects, final stabilization as defined in 1) above is achieved, or temporary stabilization including perimeter controls for an individual lot have been implemented, the home has been transferred to the homeowner, and the homeowner has been instructed on the benefits of final stabilization; or 3) for construction on land used for agricultural purposes, the disturbed land is returned to its preconstruction agricultural use. The section on BMP's in this manual outlines techniques and design criteria for stabilization measures.

A copy of the NOT form is included in Appendix 3 of this manual: Detailed instructions for completion of the NOT can be found on the back of the form. Additional copies can be obtained from ADEQ. Completed NOTs should be submitted to ADEQ at the designated address shown on the back of the form.

10.7 Enforcement

The City of Arkadelphia reserves the right to at any time suspend construction activities until the responsible party is in compliance with the SWPPP. The City of Arkadelphia can warrant additional control measures not indicated on the SWPPP to ensure compliance is guaranteed and pollution is controlled. A Certificate of Occupancy will not be issued for any structures within the development until the site has been stabilized in accordance with the requirements for the general permit and the notice of termination has been filed.

10.8 Inspection Requirement

Qualified personnel (provided by the permittee or cooperatively by multiple permittees) shall inspect disturbed areas of the construction site that have not been finally stabilized, areas used for storage of materials that are exposed to precipitation , structural control measures, and locations where vehicles enter or exit the site, at least once every fourteen (14) calendar days and within 24 hours of the end of a storm event of 0.5 inches or greater.

Based on the results of the inspection, the SWPPP shall be modified as necessary to include additional or modified BMPs designed to correct identified problems. Revisions to the SWPPP shall be completed within 7 calendar days following the inspection. A report summarizing the scope of the inspection, name(s) and qualifications of personnel making the inspection, the date(s) of the inspection, and major observations relating to the implementation of the SWPPP shall be made and retained as part of the SWPPP for at least three

10.9 Stabilization Requirement For Inactive Areas

During construction, some areas that are disturbed may be inactive for extended periods of time. The general permit addresses this issue by requiring that areas that are inactive for periods longer than 14 days must be stabilized through the use of seeding, mulching, sod, geotextiles or vegetative buffer strips. If it is anticipated that construction will resume within 21 days from when activities ceased, stabilization is not required. Proper sequencing and phasing of operations can minimize the need for temporary stabilization.

10.10 Sediment Basin Requirement

The general permit states that for common drainage areas that serve an area of 10 or more acres that are disturbed at one time, a sediment basin shall be provided where attainable until final stabilization of the site occurs. As stated in the BMP, the required volume for the sediment basin must provide storage for the calculated volume of runoff from a 2-year, 24-hour storm for each acre of drainage area that for the sediment basin is 3600 cubic feet of storage area per acre disturbed. Sediment basins shall be designed and constructed to the minimum standards provided in the BMP section of this manual.

By phasing development and the amount of land disturbed at one time, the size of the basin can be reduced or eliminated entirely. However, if necessary, sediment basins provide excellent temporary and permanent storm water treatment and can serve as an amenity to the site. Where a sediment basin with the above storage requirements is not attainable, smaller sediment basins and/or sediment traps may be used. However, at a minimum, silt fences or equivalent controls are required on all sideslopes and downslope boundaries of the site.

10.11 Storm Water Management Measures

As part of the SWPPP, storm water management measures must be addressed to reduce pollutants in storm water runoff from the site once construction is complete and the development is occupied or placed in operation. Although sometimes referred to as “post-construction” controls, best management practices to control the quality of storm water runoff from developed areas need to be considered during the earliest stages of planning for the project. Practices such as reducing the amount of impervious surface, open drainage swales, extended detention wet ponds, and others should be given consideration. Appropriate measures must be incorporated into project plans and the SWPPP. Note that the permittee is only responsible for design and installation of the management measures, and maintenance of the measure(s) during construction (prior to final stabilization of the site).

Specific techniques listed in the permit include storm water detention (dry sedimentation basins), retention structures (extended detention wet ponds), measures to allow for infiltration (trenches, open drainage swales), and velocity dissipation. Other techniques are discussed in the other manuals in this series.

10.12 Coverage of Support Activities

This permit also authorizes storm water discharges from support activities (e.g., concrete or asphalt batch plants, equipment staging yards, material storage areas, excavated material disposal areas, borrow areas) provided:

a) The support activity is directly related to a construction site having NPDES permit coverage for discharges of storm water associated with construction activity;

b) The support activity is not a commercial operation serving multiple unrelated construction projects by different operators, and does not operate beyond the completion of the construction activity at the last construction project it supports; and

c) Appropriate controls and measures are identified in a Storm Water Pollution Prevention Plan covering the discharges from the support activity.

All discharges of storm water from ready-mix concrete batch plants covered as a support activity must also comply with the following limitations:

a) pH – between 6.0 and 9.0 standard units;
b) Oil and Grease – 15 mg/l as a daily maximum; and
c) Total Suspended Solids – 65 mg/l as a daily maximum.

10.13 Spill Notification

The general permit allows for storm water discharge from construction sites only. Discharges of other substances from construction activities or from operations on a site during construction are not permitted. In the event of a spill of a hazardous substance, the operator is required to notify the National Response Center (NRC) at (800) 424-8802 to properly report the spill. In addition, the operator shall submit a written description of the release (including the type and the approximate amount of material released), the date of the release, the circumstances of the release, and the steps to be taken to prevent future spills to the EPA Regional Office. In addition, the SWPPP must be revised within 14 calendar days after the release to reflect the release, stating the information above along with modifications to minimize the possibility of future occurrences.

If fuels, oils or other substances are to be present on site, it is imperative that closed containers be provided along with containment areas for large quantity spills. Hazardous chemicals include fertilizers, paints, oils, grease, pesticides, and fuels, along with other construction chemicals. While much of this manual focuses on the sediment and erosion control aspects of the SWPPP, the potential for damaging pollution from chemicals is great. Provisions must be provided to address potential pollution through the use of the BMPs presented along with compliance of OSHA and other regulatory requirements.

10.14 Retention of Records

As part of the general permit, the SWPPP and supporting documentation must be retained for a period of 3 years after the completion of the project. This is to protect the operator(s) of the site from future claims concerning water quality and measures implemented at the site. It is recommended that each of the operator(s) maintain a copy of the SWPPP for the 3-year period.