

STORMWATER DRAINAGE MANUAL



JULY 2025

CITY OF BEEBE, AR

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1. Submittal Procedures

1.1 General

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

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

Exceptions where no stormwater management and drainage plan are required according to the City of Beebe Municipal Code:

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

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

Except as provided herein, the updated Stormwater Drainage Manual shall be in full force and effect on October 27, 2025.

1.1.1 Additional Regulatory Requirements

Arkansas Department of Environmental Quality

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

- Small construction sites (disturbing one acre or more and less than five acres) have automatic coverage under the Construction Stormwater General Permit. Under automatic coverage for small sites, it is not necessary to submit any documents to ADEQ and there is no fee. However, the automatic Notice of Coverage (NOC) must be posted at the site prior to commencing construction and a Stormwater Pollution Prevention Plan (SWPPP) must be prepared and made available at the site prior to commencing construction.
- Large Construction Sites (disturbing five acres or more) must submit a Notice of Intent (NOI), a Stormwater Pollution Prevention Plan (SWPPP) and pay a fee to the Arkansas Department of Environmental Quality (ADEQ) in order to obtain coverage for discharges of stormwater associated with construction activity at any site or common plan of development or sale that will result in the disturbance of five (5) or more acres of total land area. Additional information may be found at: <https://www.adeg.state.ar.us/water>

U.S. Army Corps of Engineers

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

Floodplain Development Permits

Any development within or bordering a Special Flood Hazard Area, as portrayed on FEMA Flood Insurance Rate Maps (FIRMs) is required to obtain a Floodplain Development Permit.

1.2 Submittal Procedures

1.2.1 Conceptual Drainage Review

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

1.2.2 Preliminary Review

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

1.2.3 Final Drainage Review

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

1.2.4 Online Submittal Procedures

The building permit application is available online at: beebe.portal.iworq.net/portalhome/beebe, or by following links from the City of Beebe main website at : beebeark.org.

1.3 Plans and Specifications Requirements

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

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

Plan Drawings

- Maximum horizontal scale 1 inch = 100 feet

Profile Drawings for drainage and storm systems

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

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

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

1.3.1 Title Sheet

The Title Sheet shall include:

1. The designation of the project which includes the nature of the project, the name of the development, city, and state.
2. Project number.
3. Index of Sheets.
4. Vicinity maps showing project location in relation to streets, railroads, and physical features. The location map shall have a north arrow and appropriate scale.
5. A project control benchmark identified as to the location and elevation with notation referencing City monument(s) used to establish the Project Benchmark.
6. Reference to the horizontal and vertical datum for the project.
 - a. The horizontal datum shall be NAD83, Arkansas State Plane, North Zone.
 - b. The vertical datum shall be North American Vertical Datum of 1988 (NAVD88).
7. The name and address of the owner of the project and the name and address of the engineer preparing the plans.
8. Floodplain statement identifying the FIRM panel, date, and flood zone; and,
9. Engineer's seal (every sheet).

1.3.2 General Layout Sheet

The general layout sheet shall include:

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

8. Floodplain Boundaries with mapped BFE, if available
9. Name of each utility within or adjacent to the project area.
10. Standard notes.
11. If more than one general layout sheet is required, a match line should be used to show continuation of coverage from one sheet to the next.

1.3.3 Other Requirements for Plans and Specifications

Other requirements for Plans and Specifications include:

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

1.4 Grading and Drainage Report Requirements

1.4.1 Preliminary Grading and Drainage Report

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

1.4.2 Final Grading and Drainage Report

A Final Grading and Drainage Report shall be included in the Permit Application. Computer model input summary tables and output result tables shall also be provided as part of the Final Grading and Drainage Report.

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

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

1.5 Electronic As-Builts

The City of Beebe requires as-built plans and information submitted from the Engineer of Record with final plats; request for certificates of occupancies on building permits; and following street and drainage infrastructure construction projects. Plans and information should be provided on public and private stormwater drainage systems installed and/or modified.

Final approval of final plat shall not be given until the Planning & Zoning Commission receives an electronic copy of the Stormwater Drainage Features As-Built, in PDF .pdf and Autocad .dwg file format. The As-Built Plan drawings shall be in State Plane Arkansas North Zone coordinates, with the North American Datum 1983 with units as survey feet. The As-Built drawings shall have the stormwater features drawn in a separate layer in AutoCAD so the features can be easily separated from other layers in the drawing.

2. Stormwater Criteria, Planning, and Regulation

2.1 Stormwater Sizing Criteria

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

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

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

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

Sizing Criteria	Description
Downstream Flood Protection	Provide peak discharge control of the 10-year storm event such that the post-development peak rate is 90% or less of the pre-development rate. Extend analysis through the zone of influence based on the 10% rule for required projects.
Level of Service	Provide a level of service for the storm drainage system based on the 10-year storm. Analyze and provide a safe 100-year overflow path for all developments.

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

2.1.1 Water Quality

Stormwater management and drainage plans must comply with the Statewide General Stormwater Permit, and all applicable Arkansas State permits regarding stormwater quality.

2.1.2 Downstream Flood Protection

Downstream overbank flood protection shall be provided by controlling the post-development peak discharge rate to not exceed 90% of the pre-development rate for the 10-year, 24-hour return frequency storm event. The NOAA Atlas 14 rainfall depth that correspond to this event is 5.75 inches. Please note that this depth is subject to change once NOAA releases their Atlas 15 rainfall depths.

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

Determining the Overbank Flood Protection Volume

- *Peak-Discharge and Hydrograph Generation:* The hydrograph methods provided in Chapter 3 of this Drainage Manual shall be used to compute the peak discharge rate and runoff for the 10-year, 24-hour storm. The runoff hydrographs shall be routed through the proposed detention/retention structures using appropriate software or methodology as outlined in Chapter 7.

2.1.3 Level of Service

The stormwater system (inlets, storm pipes) shall be designed to meet the 10-year Level of Service and cross drainage (cross culverts, bridges) shall be designed to meet the 25-year Level of Service. The Design Storm is based on the fully developed watershed conditions.

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

Determining the Adequacy of the Stormwater Management System

- *On-site Storm System Sizing:* The SCS TR-55 hydrograph method provided in Chapter 3 shall be used to compute peak discharge rate and runoff for the design storm.
- *Downstream Analysis:* Peak discharges at downstream locations shall be checked and evaluated for any increase in peak flow above pre-development conditions. The downstream check shall extend to the point where the developed site area comprises no more than 10% of the total drainage area checked. If the post-developed discharges at the downstream checkpoints exceed pre-development conditions, additional mitigation measures shall be required.
- *System Check:* As a final check, the 100-year, 24-hour storm event shall be routed through the drainage system and stormwater management facilities to determine the effects on the facilities, adjacent property, and downstream, and to confirm adequacy of finished floor elevations for structures. Emergency spillways for structural stormwater controls should be designed to safely pass the fully developed 100-year flow. A positive overflow pathway shall be provided that conveys the flow without damaging structures, utilities, or infrastructure.

3. Determination of Storm Runoff

3.1 General

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

Table 3.1 Hydrology Methodology.

Methodology (Section)	Watershed Size
Rational Method (3.3)	Less than 40 acres Only for structure sizing, not allowed for storage sizing
Unit Hydrograph (SCS) Method (3.4)	Up to 200 acres.
Runoff Routing (HEC-HMS) Method (3.5)	Greater than 1,000 acres.

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

3.2 Precipitation Data

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

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

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

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

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

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

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

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

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

Storm Event	2-year	5-year	10-year	25-year	50-year	100-year
Depth (in)	4.13	5.01	5.75	6.81	7.64	8.50

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

The rainfall intensity is calculated using the formula below:

$$I = \frac{b}{(t_c + d)^e} \quad \text{Eq. 3.1}$$

Where: I = precipitation intensity (inches per hour)

t_c = time of concentration (minutes)

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

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

The majority of drainage sub-basins within the City of Beebe have a relatively short time of concentration. In general, basins with computed times of concentration in excess of 60 minutes (maximum) should be subdivided to create smaller sub-basins for more accurate computation of peak discharge.

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

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

3.3 Rational Method – ONLY FOR STRUCTURE SIZING (NOT ALLOWED FOR STORAGE REQUIREMENTS ANALYSIS)

The Rational Method can be used to estimate stormwater runoff peak flows for the design of gutter flows, drainage inlets, storm pipes, culverts, and existing roadside ditches. It is most applicable to small, highly impervious areas. The Rational Method was not developed for storage design or any application where a more detailed routing procedure is required and shall not be utilized for such.

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

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

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

$$Q = CiA \quad \text{Eq. 3.2}$$

Where: Q – Peak flow (cfs).

C – Dimensionless runoff coefficient.

i – Rainfall intensity (in/hr).

A – Drainage area (acres).

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

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

Land Use Description	Slope, %	Hydrologic Soil Group		
		A/B	C	D
Lawns				
	0-2	0.15	0.25	0.35
	2-7	0.25	0.35	0.40
	> 7	0.30	0.40	0.45
Forest				
	0-2	0.23	0.28	0.33
	2-7	0.31	0.36	0.41
	> 7	0.36	0.41	0.46
Unimproved areas				
Meadow		0.20-0.4	0.25-0.45	0.30-0.55
Row crops		0.25-0.6	0.35-0.75	0.40-0.80
Business				
Downtown areas		0.7	0.8	0.9
Neighborhood areas		0.5	0.6	0.7
Residential				
8 lots / acre		0.67	0.71	0.76
4 lots / acre		0.46	0.52	0.61
3 lots / acre		0.40	0.47	0.57
2 lots / acre		0.35	0.43	0.54
Suburban (1 lot / acre)		0.30	0.38	0.50
Multi-units, detached		0.70	0.75	0.80
Multi-units, attached		0.75	0.80	0.85
Apartments		0.65	0.70	0.75
Industrial				
Light areas		0.60	0.75	0.85
Heavy areas		0.80	0.85	0.90
Parks, cemeteries		0.25	0.35	0.45
Schools, Churches		0.70	0.75	0.80
Railroad yard areas		0.20	0.35	0.50
Asphalt & Concrete Pavements, Roofs.			0.95	
Brick Pavement or Gravel (compacted subgrade)			0.85	
Graded or no plant cover				
	0-2	0.25	0.30	0.35
	2-7	0.40	0.50	0.60
	> 7	0.50	0.65	0.80

- Selection of design rainfall intensity in the Rational Method shall be based on the required design frequency and calculated using the IDF equation (Eq. 3.1) provided in Section 3.2.
- Drainage area computations for runoff estimation should be based on the best available data. Where more recent and more detailed site-specific topographic data is not available, the most recent publicly available topographic contour data should be used (www.pagis.org).

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

$$Q = C_f C_i A \quad \text{Eq. 3.3}$$

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

Table 3.5 Frequency Factors (C_f) for Rational Equation.

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

Note: $C_f * C$ shall not exceed 1.0

3.4 Unit Hydrograph (SCS) Method

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

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

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

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

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

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

Rainfall amounts for 24-hour storm durations with recurrence intervals of 10-, 25-, and 100- years. The appropriate rainfall distribution for the City of Beebe is Type II.

3.4.1 Time of Concentration and Travel Time

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

$$T_c = T_s + T_{sc} + T_{ch} \quad \text{Eq. 3.4}$$

Where: T_c = Time of Concentration

T_s = Sheet Flow Time

T_{sc} = Shallow Concentrated Flow Time

T_{ch} = Channel Flow Time

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

a. Sheet Flow

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

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

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

L = flow length (ft)

P_2 = 2-year, 24-hour rainfall (in)

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

b. Shallow Concentrated Flow

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

$$T_{sc} = \frac{L_f}{3600 V} \quad \text{Eq. 3.6}$$

Where: L_f = Flow length (ft)

V = Velocity (ft/sec)

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

$$\text{Paved} \quad V = 20.33(S)^{0.5} \quad \text{Eq. 3.7}$$

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

Where: V = Velocity (ft/sec)

S = watercourse slope (ft/ft)

c. **Channel Flow Time**

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

$$T_{ch} = \frac{L_f}{3600 V} \quad \text{Eq. 3.9}$$

Where: L_f = Flow length (ft)

V = Velocity (ft/sec)

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

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

Where: Q = total discharge in cubic feet per second

n = coefficient roughness

A = cross-sectional area of conduit in feet

R = hydraulic radius of channel in feet

S = slope of energy line in feet per foot

V = velocity in feet per second

3.4.2 Runoff Factor

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

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

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

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

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

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

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

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

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

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

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

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

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

3.5 Runoff Routing (HEC-HMS) Method

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

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

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

3.6 Regression Equations

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

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

Table 3.6 TR-55 Runoff Curve Numbers¹ (CN)

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

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

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

Source: USDA Technical Release 55 (TR-55)

3.7 Runoff Analysis Software

Computer software shall be used for stormwater runoff analyses in conformance with design criteria to meet the design standards of the City of Beebe and this Drainage Criteria Manual. Software such as WinTR-55, HEC-HMS, HEC-RAS, EPA-SWMM, XPSWMM, Infoworks ICM, Bentley StormCAD, HydroCAD, Bentley Pond-Pack, Autodesk Storm and Sanitary Analysis or comparable may be used for runoff analyses. Use of two-dimensional (2-D) models, with approval by the City Engineer or Acting Consultant, may be used when necessitated by site conditions with complicated overland flow paths or other special circumstances. Within Special Flood Hazard Areas, FEMA-approved hydrologic models should be used. A list of FEMA- approved software can be found on their website: <https://www.fema.gov/flood-maps/products-tools/numerical-models>

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

3.8 Rain-on-Mesh

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

3.8.1 Excess Precipitation

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

3.8.2 Two-Dimensional Infiltration

For the two-dimensional infiltration method, infiltration losses are calculated within the hydraulic model. A direct rainfall hyetograph is applied to the 2-D mesh and infiltration is subtracted based on the spatial infiltration layer within the model. The infiltration method used within the 2-D model should be the SCS Curve Number method. Other methods such as Deficit & Constant or Green-Ampt may be used with

approval by the City Engineer or Acting Consultant. The infiltration layer is developed by combining the land use layer and the soils layer. To properly account for directly connected impervious area, the land use layer should not use a composite curve number. Instead, the land use layer should include the impervious percentage and the runoff Curve Number for the pervious area only. This is especially important in highly developed, urban areas where runoff will occur at the very beginning of storms due to impervious areas that are directly connected to the storm runoff system.

3.9 Phased Construction

To provide for the most efficient design of stormwater structures, all proposed phased developments shall install (within Phase 1 of the proposed development) the necessary improvements to meet the requirements of this drainage manual for the full build-out conditions. Stormwater post development peak flows must be calculated assuming at a minimum that the entire acreage owned by the developer matches the proposed Phase 1 development type and density. Variances to this requirement will be accepted only if the Developer provides written documentation to the City that a proportion of the acreage will not be developed and specifies the cover type and hydrologic conditions that will be maintained for those undeveloped areas.

4. Storm Drainage System Design

4.1 General

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

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

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

Ditches, channels, and swales are prohibited for new construction. Modifications of existing Roadside ditches, channels and swales to handle proposed peak flows are governed by design criteria and guidance covered in Chapter 5, *Open Channel Design*.

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

4.2 Storm System Design Requirements

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

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

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

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

4.2.1 Design Storm

The design storm for the storm drainage system is the 10-year event. The storm drainage system refers to the infrastructure that collects local runoff which includes inlets, gutters, existing roadside ditches, swales, and underground pipe. The design storm for cross drainage is the 25-year event. However, if a street is the only means of ingress/egress then the cross drainage shall be sized for the 100-year storm regardless of street classification. Cross drainage refers to a pipe, culvert, or bridge that conveys water from one side of a roadway to another.

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

Table 4.1 Design Storm for Street Classification

Street Classification	Design Storm	
	Cross Drainage	Storm System
Principal Arterials	25-year	10-year
Major and Minor Arterials	25-year	10-year
All other streets	25-year	10-year

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

4.3 Street and Roadway Gutters

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

Manning's equation is integrated for an increment of width across the section. The resulting gutter equation is:

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

Where: Q = flow rate

$K_u = 0.56$

n = Manning's coefficient (Table 4.2)

S_x = Cross slope (ft/ft)

S_L = Longitudinal slope (ft/ft)

T = Width of flow (spread) (ft)

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

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

Source: HEC-22

4.3.1 Permissible Spread of Water

Inlets shall be installed at low points and at such intervals to provide the appropriate clear traffic lane per street classification in each direction based upon peak discharges from the 10-year design storm. Minimum lane clearance requirements are provided in Table 4.3. All computations for the 10-year design storm and 100-year, 24-hour storm shall be provided.

Table 4.3 Flow Spread Limits for Inlets

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

4.3.2 Flow Bypass

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

4.3.3 Minimum and Maximum Velocities

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

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

4.4 Storm Drain Inlets

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

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

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

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

4.4.1 Classification

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

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

Table 4.4 Stormwater Inlet Types

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

4.4.2 Inlets in Sumps

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

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

$$Q_i = C_w L_w d^{1.5} \quad \text{Eq. 4.2}$$

Where: Q_i = inlet capacity (CFS)

C_w = weir discharge coefficient

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

d = flow depth (ft)

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

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

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

Definition of terms:

L = length of curb opening

a = depth of curb depression

h = height of curb opening

A_o = clear opening area

d = depth of water at curb opening

W = width of grate

W^* = lateral width of recessed section

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

$$Q_i = C_o A_o \sqrt{2gd} \quad \text{Eq. 4.3}$$

Where: Q_i = inlet capacity (cfs)

C_o = orifice coefficient

A_o = orifice area (ft²)

g = gravitational acceleration (32.2 ft/sec²)

d = characteristic depth (ft) defined in Table 4.6

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

Orifice Inlet Types	C_o	A_o	Orifice equation valid for
Curb opening inlet or recessed curb opening inlet	0.67	hL	$d_i^* > 1.4h$
Vane Grate Inlet	0.67	Clear opening area	$d^{**} > 1.79(A_o/L_w)$

* d_i = depth of water at curb opening

** d = depth of water over grate

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

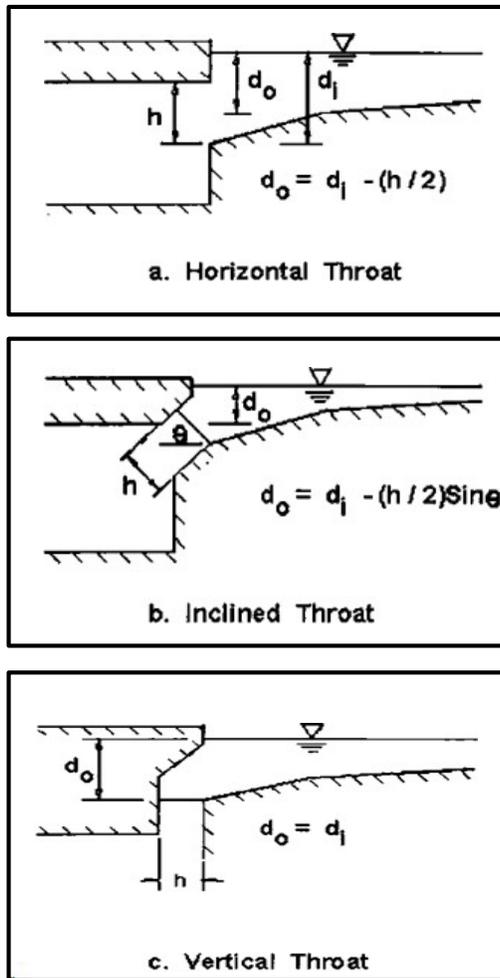


Figure 4.1. Curb-Opening Inlets

4.4.3 Inlets on Grade

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

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

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

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

4.4.4 Combination Inlets

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

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

4.4.5 Plan and Calculation Submittals

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

4.5 Flow in Storm Drains

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

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

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

4.5.1 Hydraulic Grade Line

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

designed for the 10-year storm, then the tailwater elevation shall be based on the 10-year storm elevation in the receiving channel.

4.5.2 Roughness Coefficients

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

4.5.3 Manhole Location

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

4.5.4 Minor Head Losses at Structures

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

$$h_j = K_j \frac{|V_2^2 - V_1^2|}{2g} \quad \text{Eq. 4.4}$$

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

V_1 = Velocity in upstream pipe and feet per second.

V_2 = Velocity in downstream pipe and feet per second.

K_j = Junction or structure coefficient of loss.

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

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

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

Table 4.7 Manning's n-values for conduits.

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

Source: Chow, 1959

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

Table 4.8 Minor Loss Coefficients.

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

Table 4.9 Minor Loss Coefficients for Junctions.

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

Source: Brazoria County, Texas Drainage Manual

4.5.5 Minimum Grades

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

Table 4.10 Minimum slope required to produce scouring velocity.

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

4.5.6 Utilities

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

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

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

4.5.7 Easements

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

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

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

5. Open Channel Design

5.1 General

The City of Beebe requires new construction to be designed with curb, gutter, and inlet systems that convey stormwater to drainage pipe or culverts (closed system). Once stormwater is channelized into drainage pipe or culverts it shall not transition to open channel until reaching an existing open channel system. Proposed construction may empty into existing channel systems that require modification. This section provides an overview of open channel modification design criteria and methods.

5.1.1 Considerations for Use of Open Channels

Intermittent alternating reaches of opened and closed systems are prohibited. All new construction shall be in closed systems until reaching the design outlet.

5.1.2 Open Channel Types

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

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

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

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

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

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

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

5.2 Design Criteria

Open channels shall be designed to the following criteria:

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

5.2.1 Velocity Limitations

The final design of engineered open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for earthen materials categories are presented in Table 5.1. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 5.1. Erosion Control Matting may be used if designed and constructed in accordance with manufacturer's specifications subject to the limitations provided in this manual.

Table 5.1 Maximum velocities for comparing lining materials.

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

Source: AASHTO Model Drainage Manual, 1991.

* Based on manufacturer specifications and subject to approval by City Design Review Engineer.

Table 5.2 Maximum velocities for vegetative channel linings.

Vegetation Type	Slope Range (%) ¹	Maximum Velocity ² (ft/s)
Bermuda grass	0-10	5
Bahia		4
Tall fescue grass mixtures ³	0-10	4
Kentucky bluegrass	0-5	6
Buffalo grass	0-10	5
	>10	4
Grass mixture	0-5 ¹	4
	5-10	3
Annuals ⁴	0-5	3
Sod		4
Staked sod		5

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

5.2.2 Channel Cross Sections

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

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

$$\text{Freeboard (ft)} = 1.0 + 0.025 vD^{1/3} \quad \text{Eq. 5.1}$$

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

D = depth of flow (ft)

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

$$H = v^2 ((T + b)/2gR_c) \quad \text{Eq. 5.2}$$

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

v = velocity of flow (ft/s)

T = top width of water surface (ft)

b = bottom width of channel (ft)

g = acceleration of gravity (32.2 ft/s²)

R_c = mean radius of bend (ft)

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

5.2.3 Channel Drops

Sloped drops shall have roughened faces and shall be no steeper than 2:1. They shall be adequately protected from scour and shall not cause an upstream water surface drop that will result in high velocities upstream. Downcutting and lateral cutting just downstream from the drops is a common problem which must be protected against. FHWA's HEC-14 manual and programs like HY-8 can be used to calculate scour potential and design energy dissipators.

5.2.4 Baffle Chutes

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

Baffle chutes may be used in channels where water is to be lowered from one level to another. The baffle piers prevent undue acceleration of the flow as it passes down the chute. The baffled apron shall be designed for the full discharge design flow and shall be protected from scouring at the lower end. A stilling basin shall be added where appropriate based on velocities. Refer to FHWA's HEC-14 manual for baffle design.

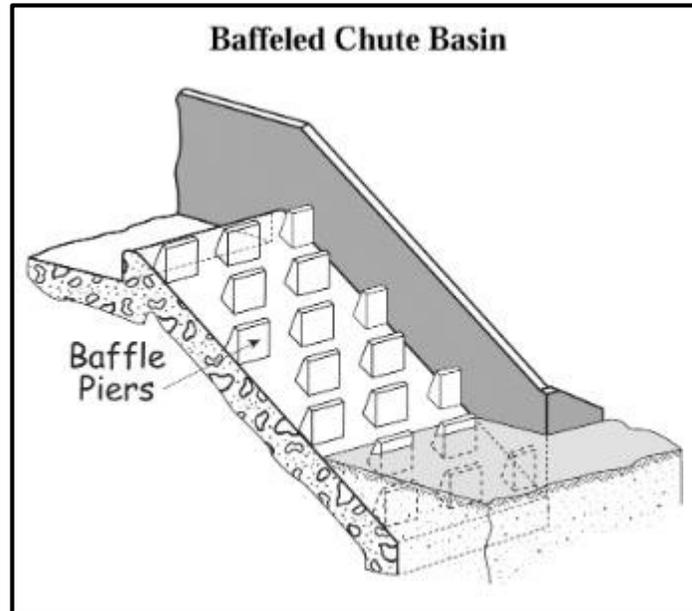


Figure 5.1. Example of Baffle Chute Basin

5.3 Computation and Software

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

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

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

Table 5.3 Channel report output file.

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

5.3.1 Manning's n Values

Recommended Manning's n values for artificial channel linings are given in Table 5.4. The Values in Table 5.4 are based off the flow depth of the channel. For natural channels, earthen channels, and various types of vegetation, Manning's n values should be estimated using experienced judgment and based on the information in Table 5.5. Additional details are provided in the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS 84-204, 1984.

Table 5.4 Manning's roughness coefficients (n) for artificial lined channels.

Category	Lining Type	Depth Ranges		
		0-0.5 ft	0.5-2.0 ft	>2.0 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.023	0.020	0.020
	Rock Cut	0.045	0.035	0.025
Temporary*	Woven Paper Net	0.016	0.015	0.015
	Jute Net	0.028	0.022	0.019
	Fiberglass Roving	0.028	0.022	0.019
	Straw with Net	0.065	0.033	0.025
	Curled Wood Mat	0.066	0.035	0.028
	Synthetic Mat	0.036	0.025	0.021
Gravel Riprap	1-inch D50	0.044	0.033	0.03
	2-inch D50	0.066	0.041	0.034
Rock Riprap	6-inch D50	0.104	0.069	0.035
	12-inch D50	----	0.078	0.040

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

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

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

Type of Channel and Description	Minimum	Normal	Maximum
Natural Streams - Minor streams (top width at flood stage < 100 ft)			
1. Main Channels			
a. Clean, straight, full stage	0.025	0.030	0.033
b. Same as above, but some stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Clean, winding, but some weeds and some stones	0.035	0.045	0.050
e. Same as 4, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as 4, but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
2. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
a. Bottom: gravels, cobbles, few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070
3. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land, tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160

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

Type of Channel and Description	Minimum	Normal	Maximum
5. EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds/plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

5.3.2 Channel Discharge - Manning's Equation

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

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

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

$$S = [Qn / (1.49 AR^{2/3})]^2 \quad \text{Eq. 5.5}$$

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

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

A = cross-sectional area (ft²)

R = hydraulic radius A/P (ft)

P = wetted perimeter (ft)

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

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

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

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

Where practical, unlined channels should have sufficient gradient, depending upon the type of soil, to provide velocities that will be self-cleaning but will not cause erosion. Lined channels, drop structures, check dams, or concrete spillways may be required to control erosion that results from the high velocities of large volumes of water. Channel velocities in man-made channels shall not exceed those specified in Tables 5.1 and 5.2. Where velocities exceed specified velocities, riprap, pavement, or other approved erosion protection measures shall be required. As minimum protection to reduce erosion, all open channel slopes shall be seeded or sodded expeditiously after grading has been completed.

5.4 Channel Lining Design

5.4.1 Vegetative Design

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

5.4.2 Riprap Design

Where the use of riprap is allowed by the City Design Review Engineer, riprap sizing shall be determined based on maximum anticipated channel velocities. Adequate erosion protection shall be provided for the design configurations. For example, if riprap will extend into a stream with higher water surface elevations and/or velocities, i.e., at a pipe outfall going into a creek, then the riprap must be sized to resist the forces of the higher flow in the creek. When rock riprap is used, the need for an underlying filter material must be evaluated. The filter material may be either a granular blanket or plastic filter cloth.

Isbash Equation

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

$$V_c = C [2g \left(\frac{\gamma_s - \gamma_w}{\gamma_w} \right)]^{0.50} (D_{50})^{0.50} \quad \text{Eq. 5.6}$$

Where: V_c = critical velocity (ft/s)

C = 0.86 for high turbulence

C = 1.20 for low turbulence

g = 32.2 ft/s²

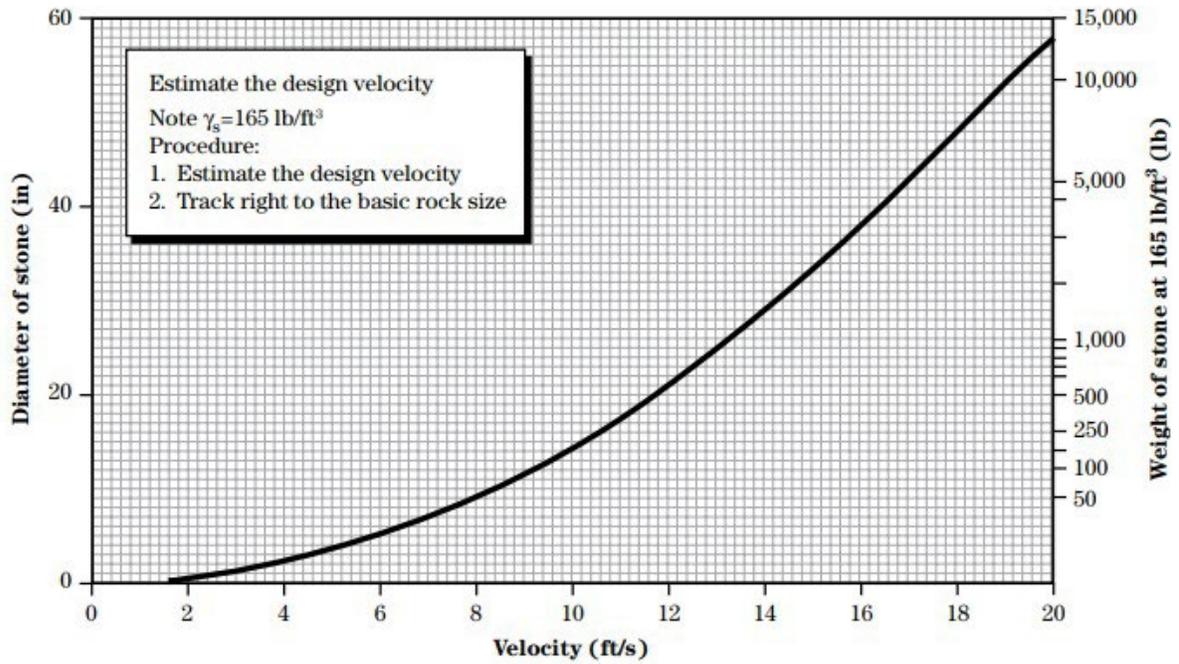
γ_s = stone density (lb/ft³)

γ_w = water density (lb/ft³)

D_{50} = median stone diameter (ft)

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

Figure TS14C-5 Rock size based on Isbash curve



(210-VI-NEH, August 2007)

Figure 5.3. Riprap sizing curve.

6. Culvert Hydraulics

6.1 General

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

6.1.1 Criteria for Use of Culverts

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

Table 6.1 Culvert and Bridge Sizing Requirements

Roadway Classification	Design Storm Event	Minimum Freeboard (Culvert)	Minimum Freeboard (Bridges)
City Streets	25-year	1 foot	1 foot
All other crossings (driveways, alleys, etc)	10-year	1 foot	1 foot

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

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

6.2 Design Criteria

6.2.1 Velocity Requirements

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

6.2.2 Energy Dissipators

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

- **Rock-Protected Outlets**

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

- **Other Energy-Dissipating Structures**

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

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

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

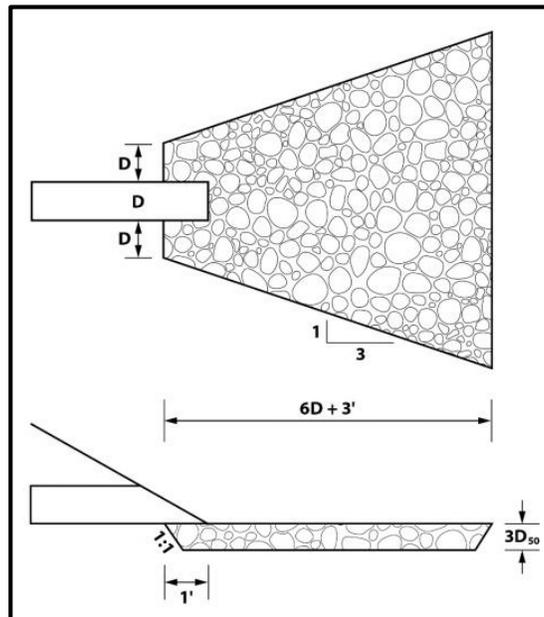


Figure 6.1. Typical Rock Protection Placement

Table 6.2 Outlet Protection Material Size

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

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

6.2.3 Length and Slope

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

6.2.4 Headwater Limitations

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

The following headwater criteria apply to culvert design:

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

6.2.5 Tailwater Considerations

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

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

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

6.2.6 Culvert End Treatments

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

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

Headwalls

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

Wingwalls

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

Aprons

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

Flared End Sections

Flared End Sections are a common end treatment for arch and round pipes suitable for many site applications. In some cases, additional erosion control or energy dissipators may be required depending on velocities.

6.2.7 Size and Material Selection

Reinforced concrete pipe (RCP) is required under Highway and City Streets including under curbs. Sections where other material is used must be approved by the City Design Review Engineer. High-density polyethylene pipe (HDPE) may be used in areas, not requiring RCP. Galvanized CMP is not permissible and shall not be used in any culvert design. Coated CMP and HDPE flared end sections are prohibited within right of ways and drainage easements.

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

6.3 Design Procedure

6.3.1 Flow Type

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

Inlet Control

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

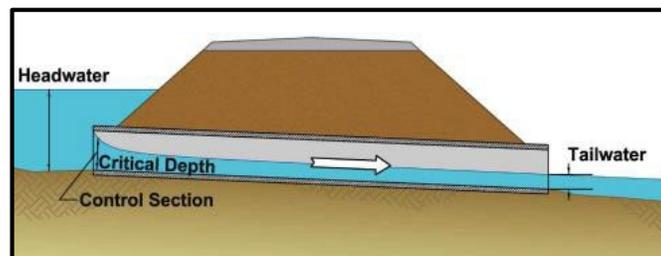


Figure 6.2. Typical Inlet Control Flow Section

Outlet Control

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. All geometric and hydraulic control characteristics, including all factors governing inlet control, water surface elevation at the outlet, and barrel characteristics, play a role in determining the culvert's capacity. Figure 6.3 depicts two typical outlet control flow sections.

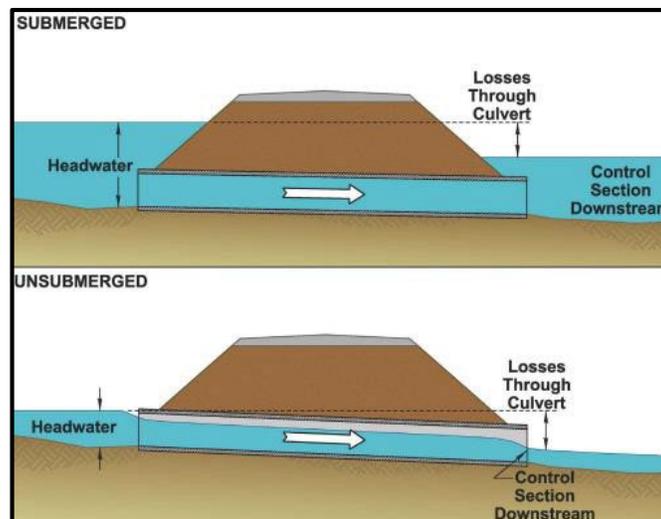


Figure 6.3. Typical Outlet Control Flow Sections

6.4 Design Software

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

6.4.1 Design Procedure

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

Step 1. – List Design Input Data.

Q = discharge (cfs)	L = culvert length (ft)
S = culvert slope (ft/ft)	TW = tailwater depth (ft)
V = velocity for trial diameter (ft/s)	K _e = inlet loss coefficient
Material Type	HW = allowable headwater depth (ft)

Step 2. – Determine Trial Size.

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

Step 3. – Calculate HW for Inlet and Outlet Control

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

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

$$HW = H + h_0 - LS \quad \text{Eq. 6.1}$$

Where: $h_0 = \frac{1}{2}$ (critical depth + D) or tailwater depth, whichever is greater.

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

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

If inlet control governs, then the design is complete, and no further analysis is required.

If outlet control governs and the HW is unacceptable, select a larger trial size and repeat the steps. Unless material or entrance conditions change, the inlet control conditions for the larger pipe does not need to be rechecked.

Step 5. – Check Potential Scour.

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

6.4.2 Multi-barrel Installations

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

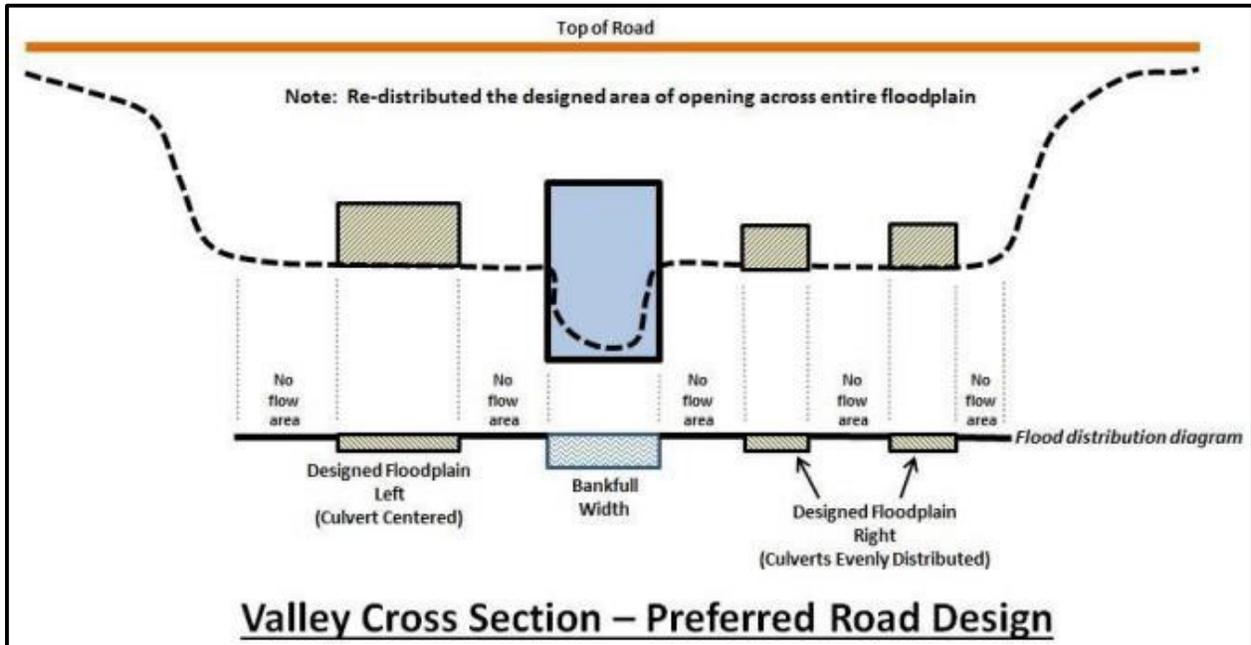


Figure 6.4. Road bisecting the floodplain with floodplain culverts.

Table 6.3 Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient K_e ¹
Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)]	0.2
Mitered to conform to fill slope	0.7
Flared End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal¹	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
Flared End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides) Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

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

7. Stormwater Detention

7.1 General

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

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

7.2 Method of Evaluation

Pre-development and post-development runoff shall be calculated to evaluate the use of stormwater detention. Runoff shall be evaluated for the 10-, 25-, and 100-year 24-hour storm events. All areas shall be evaluated using the SCS TR-55 Method.

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

7.2.1. Flood Routing

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

7.3 Detention Volume

Software such as HEC-HMS, Hydraflow Hydrographs, and others have capabilities to route hydrographs through detention basins.

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

The final design of a detention facility requires three items:

- an inflow hydrograph
 - a stage vs. storage curve
 - a stage vs. discharge curve
1. To check the capacity of a basin with a known volume, use the methods described in the previous sections.
 - a. Develop an inflow hydrograph for the storm in question.
 - b. Develop the stage-storage and stage-discharge curves for the basin.
 - c. Route the storm through the basin to determine the outflow hydrograph. Check the peak of the outflow hydrograph to ensure that it does not exceed the allowable value (90% of pre-development peak flow). Also, check the peak storage volume to ensure that it does not exceed the capacity of the basin.
 2. Analyzing a known basin utilizing the methods developed in the previous sections is relatively straightforward. However, determining the required size of a proposed basin is an iterative process, and can be quite time consuming without a method to develop a preliminary volume estimate. TR-55 provides a method for determining quick estimates of detention basin volumes.
 - a. Figure 7.1 relates two ratios: peak outflow to peak inflow (q_o/q_i) and storage volume to runoff volume (V_s/V_r). The value for q_i is determined by the peak of the inflow hydrograph. The value for q_o is normally dictated by the allowable release rate. The volume of runoff can be calculated by the SCS method or tabular hydrograph method. The relationships in Figure 7.1 were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data.
 - b. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that:
 - 1) Each stage requires a design storm and a computation of the storage required for it.
 - 2) The discharge of the upper stage(s) includes the discharge of the lower stage(s).
 - c. The brevity of the procedure allows the designer to examine many combinations of detention basins. When combined with the Tabular Hydrograph Method, the procedure's usefulness is increased. Its principal use is to develop preliminary indications of storage adequacy.

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

the parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant.

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

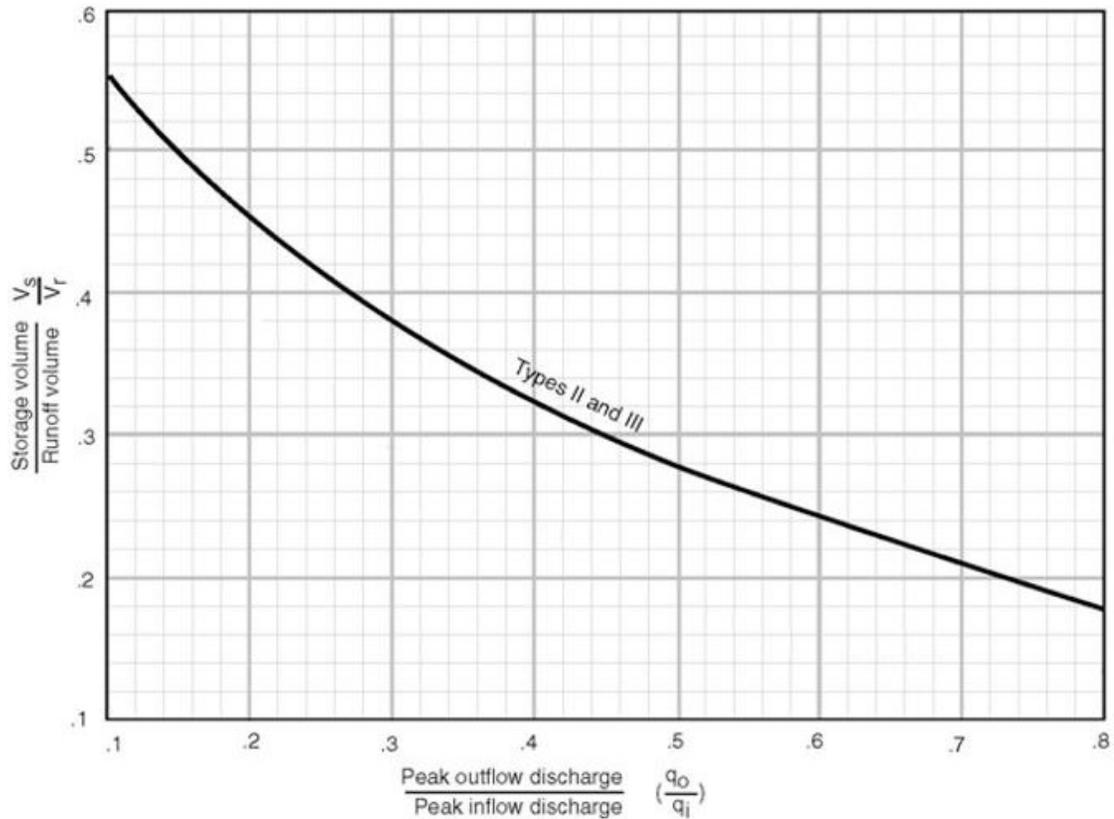


Figure 7.1. Approximate Detention Basin Routing for Type II and III

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

7.4 Structural Controls Appropriate for Detention

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

7.4.1 Stormwater Ponds

Stormwater ponds (also referred to as *retention ponds*, *wet ponds*, or *wet extended detention ponds*) are constructed stormwater retention basins that have a permanent (dead storage) pool of water

throughout the year. They are categorized in this Manual as water quality structural controls and can meet the intent of the water quality criteria, however; they also can provide detention storage to meet the other stormwater criteria (Section 2.1).

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

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

7.4.2 Stormwater Wetlands

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

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

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

7.4.3 Dry Detention / Dry ED Basins

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

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

7.4.4 Multi-purpose Detention Areas

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

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

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

7.4.5 Underground Detention

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

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

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

7.5 Detention Design Criteria

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

7.5.1 Design Procedure

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

Step 1 Perform preliminary calculations to evaluate detention storage requirements for the hydrographs as described above.

Step 2 Determine the physical dimensions necessary to hold the estimated volume from Step 1. From the selected shape determine the maximum depth in the pond. Develop the stage-storage curve for the detention basin.

Step 3 Select the desired type of outlet and size the outlet structures based on allowable discharges for the design storm events, beginning with outlet structure sizing for the smaller events to the extreme flood event and taking into consideration the tailwater in the receiving stream. The estimated peak stage for each storm event (10-, 25-, and 100-year) will occur for the maximum associated volume from Step 2. The outlet structure(s) should be sized to convey the allowable discharge for the corresponding stage for each flood event. The outfall structure shall be designed with appropriate erosion prevention measures.

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

Step 5 Evaluate whether the routed post-development peak discharges from the design storms exceed 90% of the existing pre-development peak discharges. If so, then revise the dimensions of the pond or outlet device geometry accordingly and repeat Steps 2 through 4 until the post-development peak discharges do not exceed the existing pre-development peak discharges for the watershed.

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

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

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

7.5.2 Detention Design Standards

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

7.5.2.1 General

Detention facilities shall be located within the parcel limits of the project under consideration. No detention or ponding will be permitted within public road right-of-ways. Location of detention facilities immediately upstream or downstream of the project will be considered by special request if proper documentation is submitted with reference to practicality, feasibility, and proof of ownership or right-of-use of the area proposed. Orifices shall be provided to limit outflows in accordance with design requirements and not to exceed 90% of the pre- development discharges,

7.5.2.2 Dry Detention / Dry ED Basins

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

7.5.2.3 Stormwater Ponds

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

Maximum side slopes for the fluctuating area of stormwater ponds shall be 1-foot vertical to 3-foot horizontal (3:1) unless provisions are included for safety, stability, and ease of maintenance. Safety railing or other safety measures such as a shallow shelf shall be provided for ponds located in residential areas. All stormwater ponds shall include a sediment forebay at the inflow to the basin to allow heavier

sediments to drop out of suspension before runoff enters the permanent pool. Sediment forebays shall be located at each point where piping or other conveyances discharge into the stormwater pond. Forebays shall be located such that they are accessible by maintenance equipment. Forebays shall be designed with adequate depth (preferably 4 to 6 feet to dissipate turbulent inflow - lesser design depths may be justified with supporting velocity computations) and volume to dissipate the energy of incoming stormwater flows and allow coarse-grained sediments and particulates to settle out of the runoff. The sediment forebay should be sized to accommodate 0.25 inches of runoff per contributing on-site impervious acre of drainage area and should allow flow to exit the forebay at non-erosive velocities. The forebay may be included as part of the required volume for detention with permanent pools.

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

7.5.2.4 Low Impact Development Practices

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

7.5.2.5 Other Methods

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

7.5.2.6 Outlet Works

Detention facilities shall be provided with effective outlet works. Flows shall be limited to 90% of the pre-development rates for the design storm events.

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

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

7.5.2.7 Discharge Systems

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

7.5.2.8 Ownership of Stormwater Detention Ponds

Ownership of stormwater detention ponds shall be vested in the property owner (POA).

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

7.5.2.9 Easements

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

7.5.2.10 Maintenance

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

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

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

These items include:

- 7.5.2.10.1 Minor dirt and mud removal,
- 7.5.2.10.2 Outlet cleaning,
- 7.5.2.10.3 Mowing,
- 7.5.2.10.4 Herbicide spraying (in strict conformance with the City's policies and procedures),
- 7.5.2.10.5 Litter control, and
- 7.5.2.10.6 Forebay cleaning (where applicable)

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

7.5.3 Downstream Hydrologic Assessment

7.5.3.1 Introduction

The purpose of the overbank flood protection and extreme flood protection criteria is to protect downstream properties from increases in flood hazard due to upstream development. These criteria require the designer to control peak flow at the outlet of a site such that post-development peak discharge equals 90% of the pre-development peak discharge. In certain cases, this does not always provide effective water quantity control downstream from the site and may exacerbate flooding problems downstream. The reasons for this have to do with the timing of the flow peaks, and the total increase in volume of runoff. This section outlines the procedure for determining the impacts of post-development stormwater peak flows and volumes on downstream flows. For sites less than 40 acres a downstream assessment

is not required, however the site still must meet the 90% of pre-development discharge rates.

7.5.3.2 Reasons for Downstream Problems

Flow Timing

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

Increased Volume

An important impact of new development is an increase in the total runoff volume of flow. Thus, even if the peak flow is effectively attenuated, the longer duration of higher flows due to the increased volume may combine with discharge from downstream tributaries to increase the downstream peak flows.

Figure 7.3 illustrates this concept. The figure shows the pre- and post-development hydrographs from a development site (Tributary 1). The post-development runoff hydrograph meets the flood protection criteria (i.e., the post-development peak flow is equal to the pre-development peak flow at the outlet from the site). However, the post-development combined flow at the first downstream tributary (Tributary 2) is higher than pre-development combined flow. This is because the increased volume and timing of runoff from the developed site increases the combined flow and flooding downstream. In this case, the detention volume needs to be increased to account for the downstream timing of the combined hydrographs to mitigate the impact of the increased runoff volume.

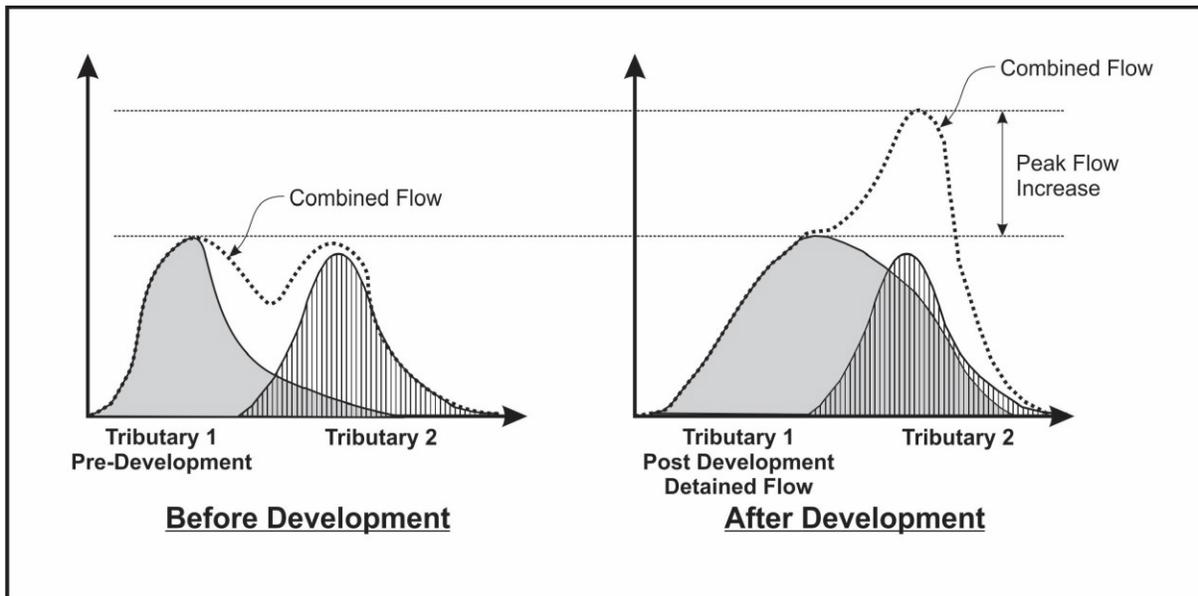


Figure 7.3. Downstream hydrograph comparison of pre and post development of detention structure.

7.5.3.3 The Ten-Percent Rule

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

The ten-percent rule recognizes the fact that a structural control providing detention has a “zone of influence” downstream where its effectiveness can be felt. Beyond this zone, the influence of the

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

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

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

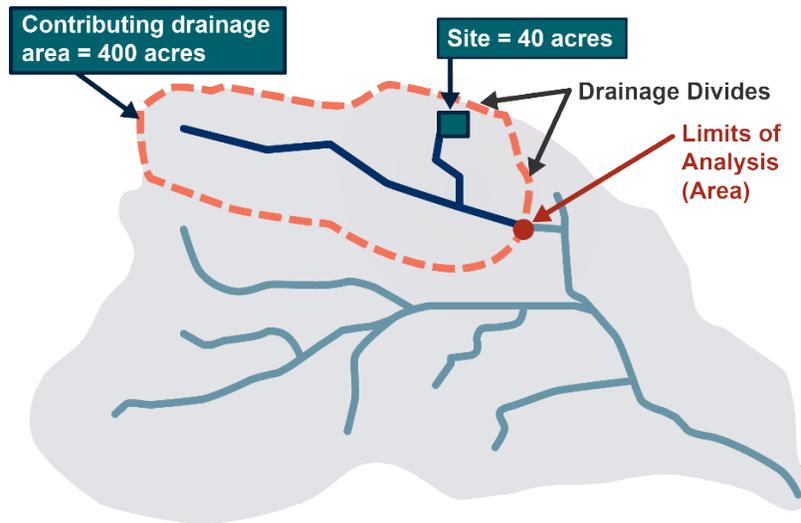


Figure 7.4. Schematic of zone of influence using the 10% rule.

8 Construction Site Stormwater Management

8.1 General

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

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

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

8.2 General Stormwater Permit Coverage

Numeric discharge limits are not imposed by the Statewide general permit at this time. The permit language ensures that those seeking coverage under this general permit will select, install, implement, and maintain BMPs at their construction site and construction support activities located off site that will be adequate and sufficient to meet water quality standards for all pollutants of concern. The DEQ has determined that BMPs, when properly selected, installed, implemented, and maintained, do provide effluent quality that can meet WQS based on 40 C.F.R. §122.44(k).

EROSION CONTROL BMPs

BMP	BMP Considered for project	BMP Used	BMP Not Used	If not used, state reason
EC-1 Scheduling	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-2 Preservation of Existing Vegetation	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-3 Hydraulic Mulch	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-4 Hydroseeding	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-5 Soil Binders	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-6 Straw Mulch	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-7 Geotextiles & Mats	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-8 Wood Mulching	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-9 Earth Dikes & Drainage Swales	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-10 Velocity Dissipation Devices	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-11 Slope Drains	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
EC-12 Stream bank Stabilization	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	

SEDIMENT CONTROL BMPs

BMP	BMP Considered for project	BMP Used	BMP Not Used	If not used, state reason
SE-1 Silt Fence	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-2 Sediment Basin	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-3 Sediment Trap	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-4 Check Dam	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-5 Fiber Rolls	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-6 Gravel Bag Berm	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-7 Street Sweeping and Vacuuming	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-8 Sand Bag Barrier	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-9 Straw Bale Barrier	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-10 Storm Drain Inlet Protection	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
SE-11 Chemical Treatment	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	

WIND EROSION CONTROL BMPs

BMP	BMP Considered for project	BMP Used	BMP Not Used	If not used, state reason
WE-1 Wind Erosion Control	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	

TRACKING CONTROL BMPs

BMP	BMP Considered for project	BMP Used	BMP Not Used	If not used, state reason
TR-1 Stabilized Construction Entrance/Exit	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
TR-2 Stabilized Construction Roadway	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
TR-3 Entrance/Outlet Tire Wash	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	

NON-STORM WATER MANAGEMENT BMPs

BMP	BMP Considered for project	BMP Used	BMP Not Used	If not used, state reason
NS-1 Water Conservation Practices	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-2 Dewatering Operations	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-3 Paving and Grinding Operations	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-4 Temporary Stream Crossing	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-5 Clear Water Diversion	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-6 Illicit Connection/ Discharge	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-7 Potable Water/Irrigation	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-8 Vehicle and Equipment Cleaning	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-9 Vehicle and Equipment Fueling	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-10 Vehicle and Equipment Maintenance	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-11 Pile Driving Operations	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-12 Concrete Curing	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-13 Concrete Finishing	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-14 Material and Equipment Use Over Water	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-15 Demolition Adjacent to Water	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
NS-16 Temporary Batch Plants	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	

WASTE MANAGEMENT AND MATERIALS POLLUTION CONTROL BMPs

BMP	BMP Considered for project	BMP Used	BMP Not Used	If not used, state reason
WM-1 Material Delivery and Storage	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-2 Material Use	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-3 Stockpile Management	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-4 Spill Prevention and Control	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-5 Solid Waste Management	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-6 Hazardous Waste Management	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-7 Contaminated Soil Management	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-8 Concrete Waste Management	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-9 Sanitary/Septic Waste Management	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	
WM-10 Liquid Waste Management	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	

Appendix A: Report and Plan Checklists

Final Drainage Report Template and Checklist

The City of Beebe, Arkansas

Project name _____

Engineer of Record _____

Planning Project Number _____

Revision no. _____

Date _____

Submittal should include the following:

1. **PROJECT TITLE & DATE**
2. **PROJECT LOCATION** - Include street address and Vicinity Map.
3. **PROJECT DESCRIPTION** - Brief description of the proposed project.
4. **NAME, ADDRESS, TELEPHONE NUMBER, AND EMAIL** of the owner and developer of the property to be permitted.
5. **NARRATIVE SUMMARY** - The summary shall include a description of the methods used to meet the conveyance, detention, and water quality requirements. This includes at a minimum a description of the target pollutants and treatment train for water quality and a description of the detention strategy used to meet the downstream flood protection requirement. Also include a description of the off-site areas, onsite areas, condition of the downstream receiving areas, existing problems, changes to flows and flow volume, proposed improvements, detention, areas with potential for high pollutant loading, and final conclusions.
6. **EXISTING DRAINAGE AREA MAP** – Existing drainage area map on a 1-inch = 200-foot minimum scale plan drawing, with 2 foot contours (1 foot contours on “flat” sites), that includes: study points at property lines, time of concentration path, bar scale, and the following information:
 - a. Aerial photograph of the project vicinity, covering the project area and the total lands that contribute runoff;
 - b. Existing drainage areas and flow patterns to downstream property line, establishing the study points;
 - c. Upstream and downstream drainage flow paths for all areas that contribute runoff to the existing site or receive runoff from the site. The downstream area(s) shall be shown as necessary to document the receiving conveyance system; and
 - d. Existing land use conditions for the drainage areas that contribute runoff.

7. **SOIL MAP** - Provide the most recent U.S. Soil Conservation Service soils and vegetation information for both the project area and the drainage area that contributes runoff on a separate map from the Existing Drainage Area Map.
8. **PROPOSED DRAINAGE AREA MAP** – Proposed drainage area site map on a 1-inch = 200-foot minimum scale plan drawing, with 2' contours (1 foot contours on “flat” sites), that include: study points, time of concentration path, bar scale, and the following information:
 - a. Proposed drainage areas and flow patterns and, if applicable, natural feature protection areas, green stormwater practice and infiltration areas;
 - b. Upstream and downstream drainage flow paths for all areas that contribute runoff to the proposed development site or receive runoff from the site. The downstream area(s) shall be shown as necessary to document the receiving conveyance system;
 - c. Proposed land use conditions for the development site and drainage areas that contribute runoff; and
 - d. Proposed locations of grading and placement of fill material within the project area and drainage areas that contribute runoff.
9. **WATER QUALITY** – Calculations and documentation indicating the target pollutants and the required water quality treatment volume.
 - a. Provide calculations for each structural control indicating the corresponding level of treatment; and,
 - b. Provide a map showing the impervious area and structural controls
10. **DOWNSTREAM FLOOD PROTECTION** – Provide calculations and documentation indicating that the post-development peak discharge rate does not exceed 90% of the pre-development rate for the 10-year, 24-hour storm event. The calculations shall include the following information:
 - a. A summary table of runoff discharge flows for the 10-year, 25-year, and 100-year, 24- hour storm events for the pre-development and post-development conditions for each study point. The summary shall include the existing and proposed flows along with supporting calculations for all of the discharge points to the receiving system. This includes the flow entering each drainage area and the flow generated within each drainage area on the site (do not separate onsite and offsite flows).
 - b. The effects of the 100-year, 24-hour storm event on the stormwater management system, adjacent property, and downstream facilities and property shall be evaluated. The 100-year flow shall be controlled through the use of structural stormwater controls to protect existing downstream property with no increase in the existing base flood elevation, or calculations shall be provided to

indicate that the on-site conveyance system will safely pass the flow and allow it to discharge into receiving waters where the floodplain is of capacity sufficient to accommodate significant additional discharges without causing damage.

11. **CHECK FOR EXISTING DOWNSTREAM FLOODING** – Describe the existing downstream capacity of each receiving area (study point). Provide documentation of a downstream assessment using the 10% Rule in accordance with Section 7.5.3. Documentation shall include photographs of the existing structures downstream of the development as well as a map showing the locations and distances of downstream structures from the development.
12. **STORMWATER DETENTION DESIGN** – If detention is required, include all computations and backup/support data including:
 - a. Detention basin size requirement computations (using an approved method).
 - b. Release structure design computations including design Water Surface Elevations for the 10-year, 25-year, and 100-year storms.
 - c. Stage-Storage and Stage-Discharge curves for the detention facility.
 - d. A summary hydrograph of the effect of the detention facility for relevant storms, incorporated with bypass.
 - e. Overflow structure(s) size and location(s);
 - f. Outfall structure(s), location(s), and orifice size(s).
 - g. Emergency overflow path.
 - h. Results of downstream analysis.
13. **PAVEMENT DRAINAGE DESIGN** – Include a table listing street classification, width, allowable spread and actual spread for design storm.
14. **STORM SEWER INLET DESIGN** - Include all computations for the design storm. Reference Table 4.3 in Chapter 4 for allowable spread and depth.
15. **INLET DRAINAGE AREA MAP** – Provide a separate map showing the inlet layout and design including the drainage areas. The map should include the proposed design, drainage areas, time of concentration paths, runoff coefficients, and bar scale.
16. **STORM SEWER DESIGN** - Include all computations and hydraulic profiles for the design storm and 100-year, 24-hour storm.
17. **CULVERT DESIGN** - Include all computations, hydraulic profile, and energy transition to channel.
18. **OPEN CHANNEL FLOW MODIFICATION DESIGN** - Include computations for normal depth and velocity. Specify required channel lining if necessary to mitigate velocity impacts on stability.
19. **FEDERAL AND STATE REQUIREMENTS** (if required).

- a. Wetlands determination (if wetlands are present on the site).
 - b. 404 permit required (include letter from USACE as an exhibit).
 - c. NPDES Construction Stormwater "Notice of Intent" (ADEQ)(include as an exhibit if required).
 - d. ANRC permit/review for "dams"(required if a stormwater impoundment qualifies as a dam per ANRC regulations).
 - e. Floodplain Development permit (required if proposed development is located within 100-year regulatory floodplain as defined by FEMA FIRM Panel)
 - f. Other
20. **EXHIBITS** – Attach the following exhibits to the final drainage report.
- a. Grading and drainage construction drawings.
 - b. Landscaping Plan.
 - c. Operations and maintenance plan.
 - d. Letter from USACE if answered Yes to 19.b. above.
 - e. Notice of Coverage (NOC) and completed SWPPP (sites 1 acre or larger).
 - f. Master Drainage Plan (if part of a larger or phased project).

21. The following paragraph with relevant information included:

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

Signed & Sealed by
Professional Engineer

22. ARKANSAS REGISTERED ENGINEER SEAL

Appendix B: Maintenance Agreement

**Inspection and Maintenance Agreement
for**

Address

Beebe, White County, Arkansas

A. GENERAL

In accordance with the requirements stated by the Beebe Stormwater Drainage Manual, this document outlines the aspects of long-term maintenance for the developed site.

Components

The drainage system on-site will be as shown in the submitted documentation. Maintenance of these on-site BMP's will be crucial to the longevity and safety of the developed site and the existing roadway.

B. INSPECTION

Structural Inspection

The drainage system shall be professionally inspected twice a year. The thorough inspections will take place in months of June and October to ensure the culverts have been checked before and after the rainy season (November-May).

Erosion

Along with structural inspection, the drainage system shall be inspected for signs of erosion before and after the rainy season.

Clearance of Debris

During the rainy season, inlets to the drainage system will be inspected once a month for debris such as branches, leaves, trash, and sediment to ensure the culverts function as designed.

C. DOCUMENTATION

A record shall be kept for all structural inspections and major maintenance activities. The report of inspections will be turned in to the City of Beebe annually.

D. RESPONSIBILITY

The property owner will be responsible for the maintenance services.

E. CONCLUSION

As demonstrated above by the topics covered, the Inspections and Maintenance Agreement meets the requirements of the City of Beebe.

I, _____, Licensed Contractor No. _____ in the State of Arkansas, hereby certify that the long-term maintenance and inspection items contained in this agreement have been prepared in accordance with sound engineering practice and principles, and the requirements of the City of Beebe. Further, I hereby acknowledge that the review of the long-term maintenance and inspection plan by the City of Beebe or its representatives cannot and does not relieve me from any professional responsibility or liability.

Contractor
