Chapter 9
STORMWATER DESIGN

9.1 Hydrology Methods

Table 9-1 shows the various hydrologic computation methods that can be used to compute peak flows in the City of Knoxville. The NRCS Unit Hydrograph method is specifically cited for drainage computations in the Knoxville Stormwater and Street Ordinance (Sections 22.5-21 and 22.5-33), using 24-hour Type II rainfall distribution and AMC II soil conditions. The NRCS method is used to compute a peak flow for sizing stormwater conveyances or to generate a hydrograph for the purposes of detention routing.

Table 9-1
Hydrology Design Methods

Method	Drainage Area	Time of Concentration	Impervious	Design Storms
Rational	< 50 acres	-- any --	0% to 100%	2 to 500 years
NRCS Unit Hydrograph	-- any --	-- any --	CN 40 to 98	1 to 500 years
TVA Regression Equations	> 230 acres	N / A	< 75%	2 to 500 years
USGS Regression Equations	> 135 acres	N / A	< 75%	2 to 100 years

Uses for the various hydrologic computation methods are discussed in the Knoxville BMP Manual. The NRCS Unit Hydrograph method shall be used for all design calculations, but other methods may be consulted for sizing stormwater conveyances (particularly if conservative values and assumptions are used). For equations, consult the Knoxville BMP Manual at these locations:

- NRCS Unit Hydrograph — ST-10, Detention Computations
- NRCS Unit Hydrograph — ST-11, Detention Example for Spreadsheet
- NRCS Unit Hydrograph — ST-12, Detention Example for HEC-1 & HEC-HMS
- Rational, TVA, USGS — ST-13, Other Hydrologic Computations

The NRCS Unit Hydrograph shall be used with average antecedent moisture conditions (AMC II) and Type II rainfall distribution, as specified by the NRCS Technical Release 55 (TR-55) publication from June 1986. The NRCS was formerly called the Soil Conservation Service (SCS), part of the United States Department of Agriculture. The TR-55 publication (Urban Hydrology for Small Watersheds) is the principal technical reference to be downloaded from NRCS: http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-tr55.html

9.2 Functional Design of Stormwater Drainage Systems

In selecting the design frequency storm, the following criteria (listed in the order of being progressively more restrictive) will be used:

- Longitudinal side drains shall be designed for a 10-year frequency flood, providing that no residential or commercial structures are flooded by a 100-year flood.
- Roadway cross-drains for all local streets and collector streets shall be designed for a 25-year frequency flood, providing that no structures are flooded by a 100-year flood.
- Roadway cross-drains for arterial streets or a higher street classification shall be designed for a 50-year flood, provided that no structures are flooded by a 100-year flood.
- All bridges, structures, or embankments in floodways designated as part of the Federal Flood Insurance Study shall be designed to pass a 500-year frequency flood without raising the existing 500-year flood profile.

In instances where the contributing drainage area is 200 acres or greater, the 100-year design storm shall also be computed and analyzed, even though the design storm is a lesser storm event. The 100-year design storm will typically be carried by a combination of the installed stormwater drainage system and some form of overland relief flow, typically streets and roads which can temporarily store or convey excess stormwater runoff. Another form of overland relief flow, commonly used in residential subdivisions, are the drainage swales between houses and through neighborhoods. The 100-year design storm (for areas greater than 200 acres) must be contained within a permanent drainage easement or public right-of-way. State highways and interstates are generally capable of passing 100-year design storm safely with a minimum freeboard of 1 feet.

9.3 Design of Open Channels

Manning's equation is the principal means for determining flow capacity and velocity in open channels. Knoxville BMP Manual contains guidance, equations, and coefficients for designing open channels in ES-22 (Channel Linings) and ES-23 (Riprap). Table ES-22-4 lists permissible velocities for various channel lining materials and types of grass. Grass channels are often designed using retardance classifications to determine Manning's roughness coefficient based on the product of hydraulic radius times velocity, an iterative process for which spreadsheets are useful. Grass channels should be analyzed for mowed conditions (when velocity is greatest) and unmowed conditions (when flow depth is greatest) to verify adequacy of design.

A minimum 6" freeboard for the design storm is required for ditches and open channels that are adjacent to streets and roads. This will help to ensure that the pavement subgrade is not repeatedly inundated by smaller design storms, which can eventually lead to damage or failure of the pavement subgrade. When two open channels join together, some form of riprap or other armored surface may be provided. Consult ES-25 (Outlet Protection) in the Knoxville BMP Manual for typical energy dissipators and stilling basins.

9.4 Design of Curb and Grate Inlets

Use of City of Knoxville standard inlets or TDOT standard inlets is required within all public rights-of-way or drainage easements. This allows easy repair or replacement of any damaged grates and curbs using standard off-the-shelf items. Use of standard inlets on private property is encouraged for reasons of structural reliability, ease of maintenance, common availability and standardized installation methods. See City of Knoxville standard details COK-10 and COK-11 (on the City Engineering Division webpage) for frames/grates/inlets at locations with 24" wide concrete gutter, with the required "No Dumping - Drains to River" stormwater message.

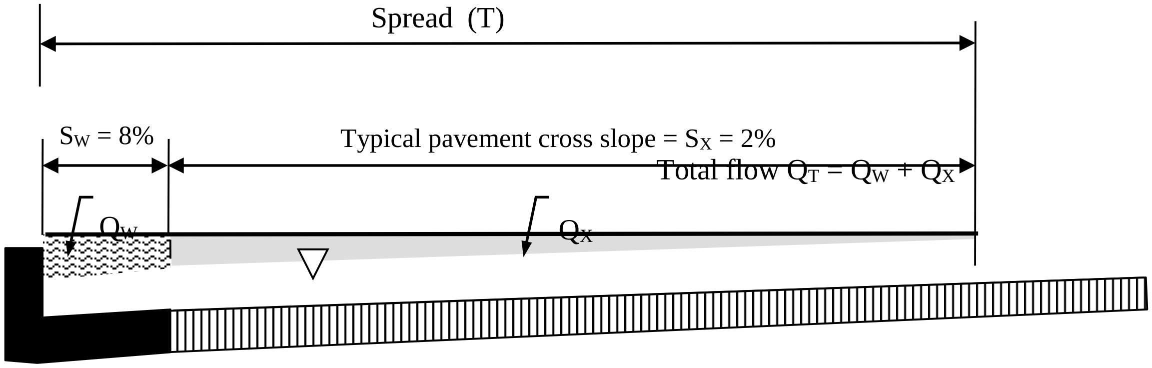
The designer must locate street inlets to quickly drain stormwater from paved surfaces, keeping streets passable and safe for vehicular traffic. Street inlets must be spaced and located in a manner to carefully balance vehicle safety, drainage system capacity, economics and efficiency. Maximum inlet spacing is generally 300 feet unless proven otherwise by computations. Inlets should be located at uphill corners of each street intersection to prevent sheetflow of stormwater. The basic geometry of stormwater flow along curbs is a thin shallow triangular cross-sectional area. If the section contains curb and gutter, then the stormwater flow is a composite shape formed by both concrete and asphalt surfaces, for which Manning's equation is still applicable. See Figure 9-1 for basic geometric considerations in computing gutter flow depth and velocity.

Based upon the longitudinal slope of the gutter and the cross slope of the street, the gutter flow will spread across the street. The spread impacts vehicular traffic in a negative way, causing vehicles to hydroplane or to pull in one direction. Basic references for computing spreads, inlet capacities, and interception rates for curb and grate inlets are FHWA Hydraulic Engineering Circular No. 12, *Drainage of Highway Pavements* (March 1984), or FHWA Hydraulic Engineering Circular No. 22, *Urban Drainage Design Manual* (November 1996). Both references can be downloaded in Acrobat format at the FHWA website:

(http://www.fhwa.dot.gov/bridge/hydpub.htm)

Detailed inlet computations are usually not required for local residential streets and alleys, except at sag locations where potentially inadequate inlets could flood nearby houses and buildings. Slow design speeds on local streets usually minimize the impact of spread and hydroplaning, although local streets do tend to have steeper approach slopes for intersections. Typical considerations for inlet design include:

1. Place inlets at all sag locations and other depressed areas to ensure positive drainage. Compute flow capacity of the inlet by ponding stormwater to a depth not higher than top of curb. Use orifice equation (with actual open area of grate) and weir equation (open perimeter of grate) to determine flow capacity of inlet. Ensure that ponded water does not flood nearby structures, buildings, or houses. Flanking inlets, at an offset distance of 25' or 50', are desirable in sag locations with large flow rates. A combination inlet (curb + grate opening) is less likely to clog at sag locations.



Curb and Gutter Section (composite flow)

Figure 9-1
Street Pavement Drainage Section

2. Place inlets at street intersections to prevent stormwater from flowing across a street or entrance. This is particularly important wherever a local street intersects a larger street, such as a collector or arterial. Valley gutters across street intersections are not allowed, unless specifically requested for very short dead-end streets or cul-de-sacs.
3. Select standard grates and avoid using grates with unusual shapes or configurations, such as beehive or raised designs. Install standard sizes of manholes and inlet tops. Grate openings should be oriented to allow for safe bicycle traffic.
4. Curb and grate inlets have a much higher interception rate for streets with relatively flat grades. Therefore, all other considerations being equal, inlets along steep sections of roadway are not as effective in intercepting stormwater.
5. Maintain a minimum curb and gutter longitudinal slope of 0.5% if possible to keep positive drainage. When designing a flat stretch of street, the street designer may incorporate a gently rolling vertical profile to maintain positive drainage (along with placement of additional inlets).

9.5 Design of Storm Drainage Systems

Manning's equation is typically used to compute non-pressurized flow in pipes and storm drainage systems where inlets and headwalls are closely spaced to allow atmospheric pressure throughout the entire system. Computations for each pipe should be performed systematically (such as in a table) and include the drainage area, design flow, velocity, capacity, diameter or size, slope, length, construction material, upstream and downstream inlets, etc. Computations should also include one or more maps and drawings to show drainage areas, impervious surfaces, slopes, land cover, paths for computing time of concentration, and any offsite areas that contribute flow. Minimum size diameter of storm drainage pipes is 15 inches. For allowable types of pipe see Policy 16 (Stormwater Pipe Materials) in Appendix A.

Computation of the hydraulic grade line (HGL) may be required by the Engineering Department, particularly if pipes are designed without excess capacity, pipes are placed at steep slopes with high velocities, or if there are excessive deflection angles in the stormwater drainage system. Excessive velocities should be avoided to prevent HGL problems and the potential for erosion. Minimum design velocities should be at least 3 feet per second to ensure that a storm drainage system has some capability for self-cleaning (typical target slope is 1% or greater).

9.6 Design of Culverts

A culvert is a single drainage pipe, not part of an enclosed system, which has a pipe or box opening as the inlet condition. Allowable flow within culverts are subject to inlet control, outlet control, or some combination of the two controls. Culvert design is performed using FHWA Hydraulic Design Series No. 5, *Hydraulic Design of Highway Culverts* (September 1985), which can be downloaded at http://www.fhwa.dot.gov/bridge/hydpub.htm as an Adobe Acrobat document. The flow equations for a culvert are complex, such that design nomographs and procedures in Hydraulic Design Series No. 5 are the most common method of solution.

Considerations in culvert design include analysis of open channels at both ends of the culvert, potential for storage or channel routing, and design of energy dissipators and outlet protection. Head loss can be reduced by using headwalls, wingwalls, mitered slopes, and tapered inlets; refer to Hydraulic Design Series No. 5 for more details concerning culvert design. In general, reinforced concrete pipe (RCP) is more hydraulically efficient than corrugated metal pipe (CMP). Considerations for determining the allowable headwater are potential for upstream property damage, road overtopping, erosion potential, human safety, and whether wingwalls and headwalls are designed as part of the culvert. Minimum size diameter for culverts is 15 inches. See Policy 16 (Stormwater Pipe Materials) in Appendix A for allowable culvert materials.

9.7 Hydraulic Grade Line Computations

Due to short pipe lengths of typical drainage systems, normal pipe flow from Manning's equation only occurs in the middle section of the various pipes. Minor energy losses (directional changes, bend losses, inlet expansion and contraction effects) may cause the hydraulic grade line to be higher than the crown of pipe or the top of inlet. This can occur even if velocities and flows from Manning's equation appear to be acceptable and seem to indicate adequate pipe capacity.

If the hydraulic grade line is above the top of inlet, then stormwater escapes from the inlet and flows down the street. This may lead to unsafe road conditions, local flooding, displaced grates or manhole lids, etc. Hydraulic grade line computations are usually not necessary in instances where the drainage designer chooses the next larger size of pipe than necessary to convey design flows. Where the hydraulic grade line is deemed to be critical by the Engineering Director or his representative, the HGL shall be computed using the form in Table 9-2 or equivalent method.

HGL computations must be performed by a registered engineer using principles of hydrology and hydraulics, and basic formulas such as conservation of momentum and energy, continuity of flow, and types of flow classification. Open channel flow is preferred, since pressure flow allows less margin of safety and also creates more stress on pipe joints. Start at the outlet for the entire drainage system and compute the HGL at the next upstream junction(s), by repeating the procedure in columns 1 through 22 for each incoming pipe to determine the next HGL point.

Column 2	For the HGL table, the principal categories are manhole or drop inlet. Also note whether the invert of the structure is shaped with a round half-section at the proper invert elevations, in order to efficiently convey water and reduce energy losses. This information affects the total head loss adjustment in column 19.
Column 3	For the most downstream junction (i.e., the drainage system outfall), find the outlet WSE from backwater computations, outlet channel flow depth, or by assuming 0.8 times the pipe diameter/height. Determine outlet WSE for other inlets by using inlet WSE (column 21) for the downstream inlet.
Columns 7-8	The friction slope S_F is computed from the equation shown at the bottom of Table 9-2. Manning's roughness coefficient (n), the cross-sectional flow area (A), and the hydraulic radius (R) should be already known from stormwater drainage computations to size the pipes and culverts.
Columns 9-10	The loss H_O is equal to contraction loss from flow entering the outlet pipe.
Columns 11-18 **	For a given inlet, there may be more than one incoming pipe. List all the incoming pipes using one row each. Do not compute H_i or H_Δ using lateral pipes carrying less than 10% of peak flow of largest incoming pipe.
Columns 11-14	** Compute the value of Q_iV_i for each pipe. The pipe with the largest value of Q_iV_I is used to compute the head loss assigned to the inlet (H_i), which is equivalent to expansion losses of flow leaving the pipe.
Columns 15-17	** Enter deflection angle for each incoming pipe, when compared to the outlet pipe. A value of 0º means that there is no bend between the incoming pipe and the outlet pipe. Then compute K for each angle, by interpolating from values at the bottom of Table 9-2. The largest value of H_Δ will be used to determine H_L in column 19.
Columns 18-19	Add the values for the individual components of junction loss (H_O, H_i, H_Δ) to compute the total junction loss, H_L. Adjust the value of H_L in column 18 based on either one or both conditions. Multiply by 1.3 if the junction is a drop inlet that receives more than 10% of its total flow from the surface. And/or multiply by 0.5 if the junction has a shaped invert with a smooth half-diameter channel formed into the bottom for each incoming pipe to the outlet. The embedded channel must be durable and continuous.
Columns 20-21	Column 20 = column 8 + column 19. (H_T: Total head loss) Column 21 = column 3 + column 20. (Inlet HGL or WSE) Compare column 21 to column 22 to see if the drainage system overflows the top of manhole or the catch basin lip. Use the answer in column 21 as the outlet HGL (column 3) for the next upstream junction.

9.8 Analysis of Downstream System

The Knoxville Stormwater and Street Ordinance requires that discharge from a developed site (typically a stormwater detention basin) must be routed to an existing natural or manmade stormwater channel with adequate capacity. Calculations must be submitted that show the capacity of the receiving stormwater channel to handle the 2-year and 10-year design storms. The routing calculations must extend at least as far as the second downstream street crossing or to a blue-line stream. Routing calculations must be extended even further downstream, if the Engineering Director or his representative has reasonable concern about the capacity of a downstream stormwater channel based on scientific or engineering evidence.

The first reason for analysis of the downstream system is to ensure that known flooding problems are not exacerbated. Stormwater detention basins are always designed so that the peak flow discharge is not increased. This means that the immediate downstream receiving channel, if it currently has adequate capacity, will continue to be adequate. However, if the stormwater detention basin causes a longer duration for peak or near-peak flows (as shown in Figure 10-1), then flooding could occur in locations where it did not occur before.

The second reason for analysis of the downstream system is to determine any backwater effects on the detention outlet structure and embankment. In most situations, the design engineer assumes inlet control conditions for the detention basin control structure, which must be verified to ensure that the detention basin operates as designed.

Analysis of the downstream system will usually include flow capacity and velocity for existing and proposed flow conditions, using Manning's equation at a minimum, but could potentially include backwater routing effects with a computer program.

Table 9-2
Hydraulic Grade Line Computation Form

Structure Location		Outlet HGL or WSE	Pipe Loss					Junction Losses										Adjust H_L	Total Head Loss	Inlet HGL or WSE	Rim Elev or Curb Elev
			Downstream pipe or culvert			Friction Loss		Exit Loss		Entrance Loss				Bend Loss			Total Jnct Loss	Drop Inlet (x 1.3)			
#	Type		Dia	L_O	Q_O	S_F	H_F	V_O	H_O	Q_i	V_i	Q_iV	H_i	Δ	K	H_Δ	H_L	Shaped Inlet (x 0.5)	H_T		
----	----	----	ft	ft	cfs	%	ft	fps	ft	cfs	fps	----	ft	---	---	ft	ft	ft	ft	----	----
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22

Outfall: Determine outlet WSE by backwater computations, or using outlet flow depth of 0.8 x pipe diameter or height.

Peak flow and velocity (columns 6, 9, 11 and 12) are normally part of storm drainage computations.

$S_F = Q^2 n^2 / (2.208\ A^2 R^{4/3})$ or determined by charts.

$H_F = L\ S_F$

$H_O = 0.25\ (V_O^2/2g)$

$H_i = 0.35\ (V_i^2/2g)$

$H_\Delta = K\ (V_i^2/2g)$

$H_L = H_O + H_i + H_\Delta$

Then adjust H_L based on inlet

90º	K = 0.70	45º	K = 0.47
75º	K = 0.64	30º	K = 0.35
60º	K = 0.56	15º	K = 0.19