DRAINAGE CRITERIA MANUAL

GEORGETOWN UTILITY SYSTEMS

CITY OF GEORGETOWN

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1. INTRODUCTION

1.1 Purpose and Intent

The purpose of this Drainage Criteria Manual is to establish drainage design procedures and standards for stormwater management systems within the jurisdiction of the City of Georgetown. Recommended design procedures and criteria are presented for conducting hydrologic and hydraulic evaluations. Although the intention of the manual is to establish uniform design practices, it neither replaces the need for engineering judgment nor precludes the use of information not presented. Other accepted engineering procedures may be used to conduct hydrologic and hydraulic studies if approved by the City of Georgetown.

1.2 Basic Design Objectives

In general, the stormwater management system shall be designed and constructed in a manner which promotes the development of a network of both natural and built drainage ways throughout the community and so as to:

A. Retain natural flood plains in a condition that minimizes interference with flood water conveyance, flood water storage, aquatic and terrestrial ecosystems, and ground and surface water.

B. Reduce exposure of people and property to the flood hazard and nuisance associated with inadequate control of run off.

C. Systematically reduce the existing level of flood damages.

D. Ensure that corrective works are consistent with the overall goals of the City.

E. Minimize erosion and sedimentation problems and enhance water quality.

F. Protect environmental quality, social well being and economic stability.

G. Plan for both the large flooding events and the smaller, more frequent flooding by providing both major and minor drainage systems.

H. Minimize future operational and maintenance expenses.

I. Reduce exposure of public investment in utilities, streets and other public facilities (infrastructure).

J. Minimize the need for rescue and relief efforts associated with flooding and generally undertaken at the expense of the public.

K. Acquire and maintain a combination of recreational and open space systems utilizing flood plain lands.

L. Preserve natural drainage patterns and limit the amount of impervious cover so as to prevent erosion, maintain infiltration and recharge of local seeps and springs, and attenuate the harm of contaminants collected and transported by stormwater. Overland sheet flow shall be maintained wherever possible and the dispersion of run-off back to sheet flow shall be a primary objective of drainage design for the subdivision as opposed to concentration of flows in storm sewers and drainage ditches.

1.3 Future Revisions

The Manual is intended to be dynamic, and will be reviewed and updated to keep abreast of developments in urban storm water management. It is the intent of the City of Georgetown to periodically issue revisions to the Manual which incorporate new data, methods or criteria and other information as is deemed appropriate.

END OF SECTION

2. GENERAL DRAINAGE POLICY

2.1 General Requirements

General Requirements related to drainage are summarized as follows:

It shall be the responsibility of the subdivider to design and construct a system for the collection and conveyance of all stormwater run off flowing onto and generated within the subdivision in accordance with:

A. Any specific or general requirements of the City of Georgetown;

B. The City of Georgetown Drainage Criteria Manual;

C. Chapter 15-44, Flood Damage Prevention, of the Georgetown Municipal Code;

D. Good Engineering Practices;

E. City-approved plans, including any regional stormwater plans; and

F. The principles of stormwater law established by the Texas Water Code.

2.2 Design Requirements

The General Design Requirements for stormwater management systems are listed below:

A. The storm drainage system shall be separate and independent of any sanitary sewer system and its use shall not interfere with the operation and maintenance of road networks or utility systems.

B. Each lot, site and block within the subdivision shall be adequately drained as prescribed in the City's Drainage Criteria Manual and the City's Construction Standards and Specifications for Roads, Streets, Structures, and Utilities. Any use of retaining walls or similar construction shall be indicated on the preliminary plat and the Director of Utility Services may require construction plans for such structure.

C. No approval shall be issued which would permit building within a regulatory floodway of any stream or water course. The City may, when it deems necessary for the protection of the health, safety or welfare of the present and future population, prohibit the subdivision and/or development of any property, which lies within a designated regulatory floodway of any stream or watercourse.

D. No lot or building site shall derive sole access to a public street across a waterway unless such access shall be constructed to remain open under design storm conditions as prescribed in the City's Drainage Criteria Manual.

E. Areas subject to inundation under design storm conditions shall be indicated with the minimum floor elevation of each lot so affected on a certified copy of the preliminary plat submitted for filing. The appropriate final action authority for plat approval may, when it deems necessary for the protection of the health, safety or welfare of the present or future populations, place restrictions on the subdivision, regarding the design and use of areas within a drainageway. The final action authority shall not approve any subdivision of land within the floodplain of any stream or watercourse unless the applicant demonstrates that the subdivision and all development anticipated therein will comply with the requirements of this Code.

F. Design of all drainage facilities, including streets, inlets, storm sewers, outfall, culverts and ditches, shall conform to the City's Drainage Criteria Manual and the City's Construction Specifications and Standards.

G. Projected runoff rates for the design of drainage facilities shall be based on the expected ultimate developed state of the upstream contributing area. Said ultimate developed state shall be based on the maximum intensity allowable under existing zoning as applicable, the Comprehensive Plan, and approved plans within the contributing area.

H. Design of major drainage ways through a subdivision and major structures such as box culverts or bridges across a major drainage channel shall be coordinated with the requirements of the Williamson County Health District when any portion of the subdivision lies outside the City limits, and when applicable, a letter requesting a local flood plain map amendment from the Federal Emergency Management Agency (FEMA) shall be provided prior to final construction plan approval.

2.3 General Drainage Policies

The following policies apply to all drainage design and construction criteria within the jurisdiction of the City of Georgetown.

A. Acceptance of the design of all stormwater facilities are subject to the approval of the Drainage Engineer.

B. The criteria and guidelines presented in this Manual are minimum requirements.

C. The developer and his engineer shall bear full responsibility for the adequacy of design. Approval by the City of Georgetown in no way relieves the developer and his engineer of their responsibility.

D. Prior to the design of any channel improvement, storm sewer, stormwater detention facility, or any other drainage feature, the Drainage Engineer shall be consulted regarding preferred flood control strategies for the watershed(s) within which the alterations are to occur.

END OF SECTION

3. HYDROLOGY

3.1 Introduction

This Chapter presents required methodologies for the determination of design flow rates for stormwater runoff. Although numerous engineering methods exist for calculating peak flow rates and generating hydrographs, this Manual requires consistent methodologies in order to expedite development review and standardize hydrologic design and analysis. The omission of other hydrologic methods does not preclude their use; however, the City of Georgetown must approve alternative methods on a case by case basis.

A basic understanding of hydrologic processes is essential for the proper and accurate application of the methods contained in this Chapter. Further information can be obtained from textbooks on engineering hydrology and the references cited at the end of this Chapter.

3.2 Required Technical Information to be Submitted for Review

All plans must be prepared under the supervision of a registered professional engineer licensed to practice in the State of Texas. The engineer shall affix his seal and signature to each plan sheet and any reports or calculations submitted to support his plans. Appropriate hydrologic design calculations and data to be submitted for city review include, at a minimum:

A. A vicinity map indicating a north arrow, boundary lines of the site, and other information necessary to locate the project site.

B. A drainage area map with scale and north arrow shown, and including the following:

1. Existing and proposed topography of the site at two-foot minimum interval contours (for delineating large off-site contributing areas, a separate drainage map with USGS contours is acceptable).
2. Physical improvements on the site, including existing structures and proposed development.
3. Existing and proposed subbasin areas labeled with acreage and identification.
4. Time of concentration flow paths for existing and proposed conditions.

C. If the Rational Method (see Section 3.5) is used, one drainage table for existing conditions and one for proposed conditions, showing the following for each subbasin:

1. Drainage area number or identification.
2. Drainage area.
3. Times of concentration.
4. Rainfall intensities for the 2, 10, 25, and 100-year frequency storms.
5. Runoff "C" values.
6. Peak flow rates for the 2, 10, 25, and 100-year storm events

Backup calculations must be provided for the times of concentration and runoff "C" values used in the tables.

D. If the Soil Conservation Service (SCS, see Section 3.6) methods are used, one drainage table for existing conditions and one for proposed conditions, showing the following for each subbasin:

1. Drainage area
2. Times of concentration
3. Runoff curve numbers
4. Peak flow rates for the 2, 10, 25, and 100-year frequency storm events
5. Existing and proposed drainage easements
6. If detention and/or other stormwater control facilities are existing or planned, their location must be shown and stage/storage and stage/discharge tables must be included.

Backup calculations must be provided for the times of concentration and runoff curve numbers used in the tables.

E. A drainage report, including the following sections as required:

1. Project Overview
2. Existing Hydrologic Condition Analysis
3. Proposed Hydrologic Conditions Analysis
4. Detention Analysis and Design
5. Conveyance Systems Analysis and Design

The engineering report should be a comprehensive supplemental report containing all technical information and analysis necessary for the proposed project. This report should contain all of the calculations, conceptual design analysis, reports and other information used in the design of the project.

Section 1. Project Overview

The project overview should provide a general description of the existing conditions on the site, the proposed development, the area of the site, the size of the improvements, and a summary of the pre-developed and post-developed drainage conditions of the site.

Section 2. Existing Site Hydrology

This section should include a discussion of assumptions and site parameters used in analyzing the existing site hydrology. Each subbasin acreage, C values or runoff curve numbers, and times of concentration used to determine existing flow characteristics, along with basin maps, graphics and exhibits for each subbasin affected by the development should be included.

Each subbasin contained within, or flowing through, the site should be individually labeled. The subbasin labels must match the labels used in the hydrologic computations (if the subbasin is "A", the calculation set must be labeled subbasin "A").

Section 3. Developed Site Hydrology

This section should include a brief narrative, mathematical and graphical presentation of parameters selected and values used for the developed site conditions, including subbasin acreage, C values or curve numbers, and times of concentration.

Section 4. Detention Analysis and Design (if required)

This section should include all assumptions, calculations, equations, references, storage/volume tables, stage/discharge tables, graphs and any other aids necessary to clearly show results and methodology used to determine the storage facility volumes. A clear sequence of how the storage facility size was determined should be provided. The location of the detention facility with contours necessary to calculate the storage volumes available from zero to the maximum head, and location and sizes of all outlet structures must be shown on a topographic map.

Section 5. Conveyance System Analysis and Design

This section should present the detailed analysis of any existing conveyance systems and the analysis and design of the proposed stormwater collection and conveyance system for the development. This information should be presented in a clear, concise manner that can be easily followed, checked and verified. All pipes, culverts, inlets, channels, swales, and other stormwater conveyance appurtenances must be clearly labeled and correspond to the engineering plans.

The minimum information should include street, inlet and pipe flow tables. Verification of capacity and performance should be provided for each element of the conveyance system. Show design velocities and flows for all drainage facilities within the development as well as those off-site areas affected by the development.

3.3 Stormwater Runoff Calculation Methods

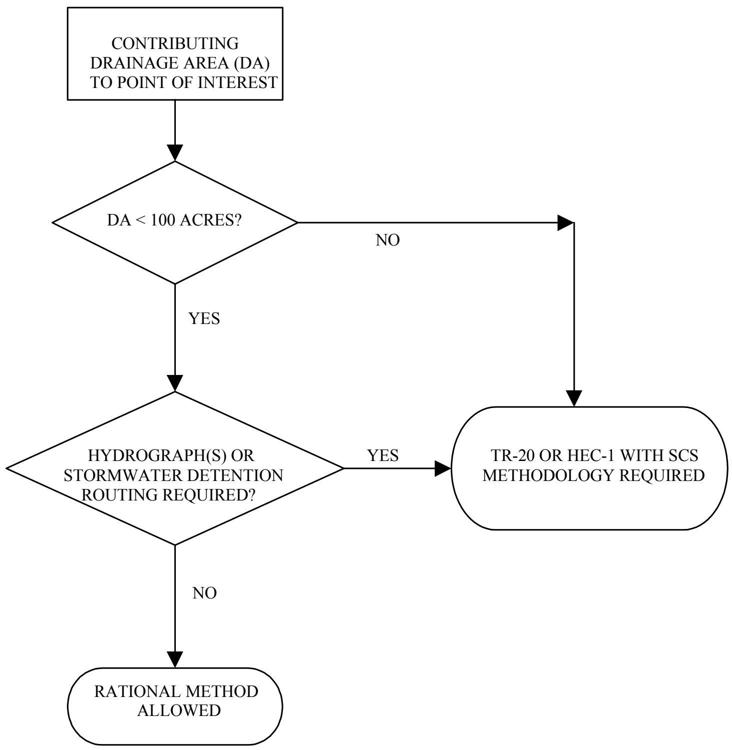
Two methods are acceptable for use in stormwater runoff calculations:

A. For contributing areas less than one-hundred (100) acres where no runoff hydrographs are required, the Rational Method may be used. The Rational Method should be used primarily for minor channel and storm sewer system design.

B. For all other situations, the U.S. Army Corps of Engineers HEC-1[3] or National Resource Conservation Service (NCRS) TR-20[4] computer programs using Soil Conservation Service (SCS) methodology must be used. These methods should be used for detention pond design, major channel design and analysis, determination of peak flow rates for floodplain modeling, and hydrologic channel routing.

The flow chart presented in Figure 3-1 provides additional direction for choosing the appropriate method for calculating stormwater runoff.

FIGURE 3-1 DECISION TREE FOR DETERMINING HYDROLOGIC ANALYSIS METHOD



3.4 Rational Method

When properly applied, the Rational Method provides satisfactory estimates for peak runoff rates from small areas with insignificant storage effects. The method is best suited for, and should primarily be used for the estimation of peak runoff rates for inlet and piped storm drainage system design.

The theory of the rational method is that if a rainfall of constant intensity begins instantaneously over the area of interest and continues indefinitely, the rate of runoff at a given point will increase until all of the upstream area is contributing runoff (until the time of concentration is reached).

The Rational Method incorporates the following basic assumptions:

A. The rainfall intensity is constant over the entire drainage area and for the duration of the storm. In reality, rainfall intensity varies with both time and location, but for small areas this assumption is reasonable.

B. The rate of runoff from any constant rainfall intensity is a maximum when the rainfall lasts as long or longer than the time of concentration. Since the rainfall occurs at a constant rate, the entire drainage area will be contributing and the peak runoff rate will occur once the time of concentration has elapsed.

C. The fraction of rainfall that becomes runoff is independent of the rainfall intensity or volume. This requires selecting a runoff coefficient (C) that is appropriate for the return period storm, land use and soil conditions.

3.4.1 Rational Method Equation

The peak discharge estimated by the Rational Method depends on four variables: time of concentration, rainfall intensity, runoff coefficient, and drainage area. Equation 3-1 is the common form of the Rational Method Equation.

EQUATION 3-1 RATIONAL METHOD EQUATION[1]

$$Q_p = CiA$$

Q_p - Flowrate (cfs) (Note: The conversion from 1 in/hr/ac = 1.008 cfs and is normally neglected.)

C - Runoff Coefficient as described in Section 3.5.2 (dimensionless)

i - Constant rainfall intensity as described in Section 3.5.3 (in/hr)

A - Contributing drainage area (ac)

3.4.2 Runoff Coefficient (C)

The fraction of the total rainfall that becomes runoff is dependent on the topography, land use, vegetation, soil type, and antecedent moisture condition of the soil. The runoff coefficient, C, must account for all of these factors. Additionally, rainfall intensity and duration as well as the basin characteristics influence the runoff coefficient. As the rainfall depth increases, the effect of rainfall abstractions decreases, and the runoff coefficient becomes larger. If land use varies within a contributing area, a weighted composite runoff coefficient C_w can be calculated. Runoff Coefficients in this Manual are based on 100-year frequency runoff coefficients taken from the City of Austin Drainage Criteria Manual. For other frequency storm events, the 100-year coefficient is multiplied by a scaling factor. Due to the limited rainfall losses for completely impervious cover, no scaling factor is applied to these surfaces. The scaling factors, and runoff coefficients for a range of storm events and land uses are presented in Table 3-1.

TABLE 3-1 RUNOFF COEFFICIENTS FOR THE RATIONAL METHOD

	Frequency Storm (yr)						
	2	5	10	25	50	100	500
	Runoff Coefficient Scaling Factor (based on 100 yr Storm Coefficients)						
Land Cover	0.67	0.73	0.78	0.86	0.92	1.00	1.18
Asphalt	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Concrete/Roof	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Grass, 0-50% cover, 0-2% slopes	0.32	0.34	0.37	0.40	0.43	0.47	0.55
Grass, 0-50% cover, 2-7% slopes	0.36	0.39	0.41	0.45	0.49	0.53	0.63
Grass, 0-50% cover, 7+% slopes	0.37	0.40	0.43	0.47	0.51	0.55	0.65
Grass, 50-75% cover, 0-2% slopes	0.28	0.30	0.32	0.35	0.38	0.41	0.48
Grass, 50-75% cover, 2-7% slopes	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Grass, 50-75% cover, 7+% slopes	0.36	0.39	0.41	0.45	0.49	0.53	0.63
Grass, 75%+ cover, 0-2% slopes	0.24	0.26	0.28	0.31	0.33	0.36	0.42
Grass, 75%+ cover, 2-7% slopes	0.31	0.34	0.36	0.39	0.42	0.46	0.54
Grass, 75%+ cover, 7+% slopes	0.34	0.37	0.40	0.44	0.47	0.51	0.60
Cultivated Land, 0-2% slopes	0.32	0.34	0.37	0.40	0.43	0.47	0.55
Cultivated Land, 2-7% slopes	0.34	0.37	0.40	0.44	0.47	0.51	0.60
Cultivated Land, 7+% slopes	0.36	0.40	0.42	0.46	0.50	0.54	0.64
Pasture/Range, 0-2% slopes	0.28	0.30	0.32	0.35	0.38	0.41	0.48
Pasture/Range, 2-7% slopes	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Pasture/Range, 7+% slopes	0.36	0.39	0.41	0.45	0.49	0.53	0.63
Forest/Woodlands, 0-2% slopes	0.26	0.29	0.30	0.33	0.36	0.39	0.46
Forest/Woodlands, 2-7% slopes	0.32	0.34	0.37	0.40	0.43	0.47	0.55
Forest/Woodlands, 7+% slopes	0.35	0.38	0.41	0.45	0.48	0.52	0.61

3.4.3 Rational Method Time of Concentration

The time of concentration is defined as the time period required for the entire drainage area to begin contributing runoff at the point of interest. It is the time of travel for water falling on the hydraulically most remote point in the drainage area to the point of calculation. Determining the longest time of concentration for a drainage area is a trial and error process. The travel time for a flow path from the most physically remote point may not be as long as another path with lesser slope and more surface roughness.

Generally, as runoff travels along a flow path, it progresses from sheet, or overland flow, to shallow concentrated flow, and finally to open channel or storm drain flow. The time of concentration is the sum of travel times for each of these components.

The minimum time of concentration for any drainage area with the Rational Method is five (5) minutes.

3.4.3.1 Sheet (Overland) Flow

Sheet flow runoff occurs over plane surfaces in depths less than approximately 0.1 feet. This type of flow generally occurs for very short distances in the headwaters of a stream. Using sheet flow lengths in excess of 300 feet in rural areas and 150 feet in urban areas should be based on sound engineering judgment. Equation 3-2 has been developed for sheet flow of less than 300 feet.

EQUATION 3-2 SHEET FLOW TRAVEL TIME[2]

$$T_t = \frac{Ln}{42\sqrt{s}}$$

T_t - Travel time for sheet flow (min)

L - Length of sheet flow path (ft)

n - Manning's roughness coefficient for sheet flow (seeTable 3-2)

s - Ground slope along flow path (ft/ft)

3.4.3.2 Shallow Concentrated Flow

After a short distance, sheet flow usually becomes shallow concentrated flow. Shallow flow lengths are normally less than 1000 feet. Equation 3-3 is used for the calculation of travel time for this component.

EQUATION 3-3 SHALLOW CONCENTRATED FLOW EQUATION[2]

$$\mathrm{T_t} = \frac{\mathrm{Ln}}{60\sqrt{\mathrm{s}}}$$

T_t - Travel time for sheet flow (min)

L - Length of sheet flow path (ft)

n - Manning's roughness coefficient for sheet flow (see Table 3-2)

3.4.3.3 Open Channel or Storm Drain Travel Time

Open channel flow is assumed to begin where channels become evident on topographic maps or are visible on aerial photos. The velocity of flow in open channels, within pipes or other conduits can be calculated using Manning's equation (see Chapter 6). For open channels, the velocity should be based on flow depths to the top of the main channel (bankfull conditions). For storm drains, the velocity should be based on full flow capacity. Once the flow velocity is calculated, the travel time can be obtained from Equation 3-4.

EQUATION 3-4 OPEN CHANNEL FLOW TRAVEL TIME[6]

$$T_t = \frac{L}{60V}$$

T_t - Travel time for channel flow (min)

L - length of open channel or storm drain flow (ft)

V - velocity calcualated from Manning's equation (ft/s)

TABLE 3-2 MANNING'S ROUGHNESS COEFFICIENTS FOR SHEET AND SHALLOW CONCENTRATED FLOW – RATIONAL METHOD[2]

Manning's "n"	Condition
0.016	Concrete (rough or smoothed finish)
0.02	Asphalt
0.1	0-50% vegetated ground cover, remaining bare soil or rock outcrops, minimum brush or tree cover
0.2	50-90% vegetated ground cover, remaining bare soil or rock outcrops, minimum- medium brush or tree cover
0.3	100% vegetated ground cover, medium- dense grasses (lawns, grassy fields etc.) medium brush or tree cover
0.6	100% vegetated ground cover with areas of heavy vegetation (parks, green- belts, riparian areas etc.) dense under- growth with medium to heavy tree growth

3.4.4 Rainfall Intensity

The rainfall intensity, *i*, in the Rational Method represents the average rainfall intensity for a duration equal to the time of concentration for the drainage area. The intensity varies with each specific frequency or return period storm. Intensity-duration-frequency (IDF) curves are generated from historical rainfall frequency duration data. Equation 3-5 is a mathematical representation of the IDF curves to be used for the City of Georgetown, and coefficients for specific frequency rainfall events are shown in Table 3-3.

EQUATION 3-5 INTENSITY DURATION FREQUENCY EQUATION[2]

$$i = \frac{a}{(t+b)^{c}}$$

i - Average rainfall intensity (in/hr)

t - time of concentration, also equal to storm duration (min)

a, b, c - coefficients for specific rainfall frequencies (see Table 3-3)

TABLE 3-3 INTENSITY-DURATION-FREQUENCY EQUATION COEFFICIENTS[2]

Storm Frequency	a	b	c
2-year	106.29	16.81	0.9076
5-year	99.75	16.74	0.8327
10-year	96.84	15.88	0.7952
25-year	111.07	17.23	0.7815
50-year	119.51	17.32	0.7705
100-year	129.03	17.83	0.7625
500-year	160.57	19.64	0.7449

3.4.5 Drainage Area

Engineering judgment must be used when determining drainage areas for the Rational Method. Drainage areas should be checked to insure that higher peak flow rates do not result from smaller, more impervious portions of the overall drainage area. The effects of development on drainage divides, flow paths, and times of concentration must also be considered.

3.5 Soil Conservation Service Methodology

The methodology developed by the Soil Conservation Service (SCS), currently the National Resource Conservation Service (NCRS) is well known and has been widely used by engineers and hydrologists for the analysis of small urban watersheds. This Section provides information on input parameters, but does not fully explain the methodology, theory or details. Further information can be found in the HEC-1[3] and TR-20[4] users manuals and the SCS National Engineering Handbook, Section 4 (NEH-4)[5] and TR-55[6] publications.

The SCS method uses the same basic data as the Rational Method: drainage area, a runoff curve number, time of concentration, and rainfall. However, the SCS method also considers initial and uniform rainfall losses, and the time distribution of rainfall.

3.5.1 SCS Rainfall Runoff Relationship

A relationship between precipitation and runoff, or excess rainfall, was developed by the SCS from experimental data. The basic SCS proportionality relationship between rainfall and runoff is shown in Equation 3-6.

EQUATION 3-6 SCS RAINFALL RUNOFF RELATIONSHIP[5]

$$\frac{F}{S} = \frac{Q}{P - I_a}$$

F - Actual rainfall lost to surface storage, interception, and infiltration (in)

S - Potential maximum rainfall losses (in)

Q - Excess rainfall, or runoff (in)

P - Total Precipitation (in)

I_a - *Initial abstration ,losses due to storage, interception and infiltration prior to the start of runoff (in)*

An empirical relationship between the initial abstractions and the potential maximum retention developed by the SCS is shown in Equation 3-7.

EQUATION 3-7 SCS INITIAL ABSTRACTION AND MAXIMUM RETENTION RELATIONSHIP[5]

$$I_a = 0.2S$$

REARRANGING EQUATION 3-6 AND SOLVING FOR THE RUNOFF:

EQUATION 3-8 SCS RAINFALL RUNOFF EQUATION[5]

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}$$

Q - Excess rainfall, or runoff (in)

The SCS also defined a physically based runoff coefficient, the curve number, as shown in Equation 3-9.

EQUATION 3-9 SCS CURVE NUMBER DEFINITION[5]

$$CN = \frac{1000}{(10 + S)}$$

or

$$S = \frac{1000}{CN} - 10$$

CN - SCS runoff coefficient (see Section 3.5.3)

These relationships allow the excess rainfall, or runoff, to be estimated from precipitation volume and curve number.

3.5.2 SCS Hydrologic Soil Groups

Precipitation losses due to infiltration are predominantly a function of soil type and land use. Most urban areas are only partially covered by impervious surfaces: the soil remains an important factor in runoff estimates. The SCS has classified over 4,000 soils into four hydrologic soil groups based on infiltration rates. In order to determine the soil classification for a specific area, the SCS Soil Survey for Williamson County should be consulted. The four hydrologic soil groups are described as follows:

- **Group A** – Soils with low runoff potential due to high infiltration rates, even when saturated. These soils consist primarily of deep sands or gravels.
- **Group B** – Soils with moderate runoff potential due to moderate infiltration rates, even when saturated. These soils consist mainly of deep, well-drained soils moderately fine to moderately coarse textures such as sandy loams.
- **Group C** – Soils with slow infiltration rates and moderately high runoff potential. These soils often have a layer near the surface that impedes infiltration. Examples are clay loams, shallow sandy loams, and soils high in clay content.
- **Group D** – Soils with high runoff potential and very slow infiltration rates. Examples are soils with high swelling potential, high water tables, claypan or a clay layer near the surface, and shallow soils over nearly impervious parent material.

If the SCS maps do not appear to accurately represent the soils for a proposed project, it is the responsibility of the design engineer to assure that the actual soil types that exist on the site are properly mapped.

3.5.3 Runoff Curve Number (RCN)

The SCS has developed relationships between land use, soil type, vegetation cover, interception, infiltration, surface storage, and runoff. These relationships have been characterized by a single runoff coefficient called the "runoff curve number" (RCN). This runoff curve number represents the runoff potential of an area. Curve numbers describe average conditions that are useful for design purposes. Table 3-4 through Table 3-7 provide a list of runoff curve numbers from the TR-55 publication.

ABLE 3-4 RUNOFF CURVE NUMBERS FOR URBAN AREAS[6]

Cover type and hydrologic condition	Average percent impervious area	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.):					
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
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1 - to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
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					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

- Values in table are for average runoff condition and Ia = O.2S.
- The average percent impervious area shown was used to develop the composite RCN's. Other assumptions are: (1) impervious areas are directly connected to the drainage system, (2) impervious areas have a RCN of 98, and (3) pervious areas are considered equivalent to open space in good hydrologic condition.

TABLE 3-5 RUNOFF CURVE NUMBERS FOR CULTIVATED AGRICULTURAL LAND[6]

Cover type	Treatment	Hydrologic condition	A	B	C	D
Fallow	Bare soil	-	77	86	91	94
	Crop residue	Poor	76	85	90	93
	cover(CR)	Good	74	83	88	90
Row	Straight row (SR)	Poor	72	81	88	91
Crops		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C+CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured &	Poor	66	74	80	82
	terraced(C&T)	Good	62	71	78	81
	C&T+CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR+CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C+CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+CR	Poor	60	71	78	81
		Good	58	69	77	80
	Close-seeded SR	Poor	66	77	85	89
	or broadcast	Good	58	72	81	85
	Legumes or C	Poor	64	75	83	85
	Rotation	Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

- Values in table are for average runoff condition and **Ia** = O.2S.
- Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
- Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%), and (e) degree of roughness.
- Poor: Factors impair infiltration and tend to increase runoff
- Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

TABLE 3-6 RUNOFF CURVE NUMBERS FOR OTHER AGRICULTURAL LANDS[6]

Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range- continuous forage for grazing	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow- continuous grass, Protected from grazing and generally mowed for hay	--	30	58	71	78
Brush--brush-weed-grass mixture, With brush the major element	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30	48	65	73
Woods--grass combination (orchard or tree farm)	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads--buildings, lanes, driveways, and surrounding lots	--	59	74	82	86

- Values in table are for average runoff condition and **Ia** = 0.2S.
- Pasture: Poor: <50% ground cover or heavily grazed with no mulch
  Fair: 50 to 75% ground cover and not heavily grazed Good: > 75% ground cover and lightly or only occasionally grazed
- Meadow: Poor: <50% ground cover
  Fair: 50 to 75% ground cover
  Good: > 75% ground cover
- Woods/grass: RCN's shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from RCNtS for woods and pasture
- Woods: Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning
  Fair: Woods grazed but not burned, and some forest litter covers the soil
  Good: Woods protected from grazing, litter and brush adequately cover soil

TABLE 3-7 RUNOFF CURVE NUMBERS FOR ARID AND SEMI-ARID RANGELANDS[6]

Cover type	Hydrologic condition	A	B	C	D
Herbaceous--mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen-mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper--pinyon, juniper, or both.. grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass Understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub--major plants Include saltbush, greasewood, creosote-bush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

- Values in table are for average runoff condition and $^{1}a = 0.2S$.
- Hydrologic Condition: Poor: <30% ground cover (litter, grass, and brush overstory)
  Fair: 30% to 70% ground cover
  Good: > 70% ground cover
- Curve numbers for Group A have been developed only for desert shrub.

3.5.4 Antecedent Moisture Condition (AMC)

Runoff curve numbers are also dependent on the soil moisture present when the rainfall begins. Runoff curve numbers in this Manual apply for normal antecedent moisture conditions (AMC II). The NEH-4 publication provides guidelines for adjusting curve numbers for wet (AMCIII) or dry (AMC I) conditions. To standardize hydrologic design, AMC II conditions must be used unless prior approval is obtained from the City of Georgetown.

3.5.5 SCS Drainage Areas

The proper selection of homogeneous drainage subbasins is necessary to obtain an accurate hydrograph analysis. Significant differences in land use within a given basin must be addressed by dividing the basin area into subbasin areas of similar land use and/or runoff characteristics.

For example, a drainage basin consisting of a concentrated residential area and a large meadow area should be divided into two subbasins. Hydrographs should be computed for each subbasin area and summed to form the total runoff hydrograph for the basin. By analyzing predominantly pervious and impervious areas separately the errors associated with averaging these land uses are minimized and the true shape of the runoff hydrograph is better approximated.

3.5.6 SCS Hydrograph Analysis

To obtain a realistic and consistent hydrologic analysis from one development site to the next, the City of Georgetown requires that all developments use the SCS hydrograph analysis methods presented in this section for drainage planning and design. Hydrograph analysis utilizes the standard plot of runoff flow versus time for a given design storm, thereby allowing characteristics such as peak, volume, and timing to be considered in the design of drainage facilities.

A unit hydrograph, as used in both TR-20 and HEC-1, is a linear transfer function model that assumes runoff proportional to rainfall excess. The unit hydrograph reflects the response of the drainage area to one inch of excess rainfall over a specified time interval. A more detailed treatment of the unit hydrograph can be found in the references provided, and in standard textbooks on engineering hydrology.

Some basic assumptions in using the unit hydrograph theory are:

A. The excess rainfall has a constant intensity within the effective duration.

B. The excess rainfall is uniformly distributed over the drainage area.

C. The duration of direct runoff resulting from an excess rainfall of given duration is constant.

D. The unit hydrograph reflects all of the combined physical characteristics of the drainage area.

E. The runoff hydrograph is a linear model; the ordinates are directly proportional to the volume of runoff.

From experimental data on gaged watersheds, the SCS created a dimensionless unit hydrograph by plotting the fraction of runoff over the peak runoff versus the fraction of time over the time to peak response.

EQUATION 3-10 SCS UNIT HYDROGRAPH PARAMETERS[5]

$$\frac{Q}{Q_{peak}} \text{ versus } \frac{time}{time_{peak}}$$

$$Q_{peak} = 484 \frac{A}{time_{peak}}$$

$$time_{peak} = \left(\frac{\Delta t}{2}\right) + t_{lag}$$

A - area (sq. mi)

Δt - unit hydrograph duration (hr)

*t_{lag} - basin lag (hr) approximately 0.6 * time of concentration*

The SCS NEH-4 publication provides more information on the dimensionless unit hydrograph.

3.5.7 SCS Method Time of Concentration

The time of concentration is defined as the time period required for the entire drainage area to begin contributing runoff at the point of calculation. The time of concentration is calculated as the time of travel for water falling on the hydraulically most remote point in the drainage area to the outlet. Determining the longest time of concentration for a drainage area is a trial and error process. The travel time for a flow path from the most physically remote point may not be as long as another path with less slope and more surface roughness.

Generally, as runoff travels along a flow path, it progresses from sheet, or overland flow, to shallow concentrated flow, and finally to open channel or conduit flow. The time of concentration is the sum of travel times for each of these components. To help standardize drainage design, all calculations for time of concentration using the SCS method must use the TR-55 procedures listed below. The minimum time of concentration with the SCS method is six (6) minutes.

3.5.7.1 Sheet (Overland) Flow

Sheet flow can be defined as flow over plane surfaces and usually occurs in the headwater of streams. A Manning's "n" is used as an effective roughness coefficient that includes the effects of drag, raindrop impact, and obstacles such as litter and rocks. These "n" values are for very shallow flow depths of less than 0.1 feet. Equation 3-11 presents the equation for calculating SCS sheet flow travel time, and Table 3-8 presents Manning's "n" values to be used in the equation. It should be noted that the equation is valid for a maximum sheet flow length of 300 feet. This value should not be used as the default, as normally sheet flow lengths are much shorter, and should be determined from field investigation and engineering judgment. Using sheet flow lengths in excess of 300 feet in rural areas and 150 feet in developed areas will require prior approval of City of Georgetown staff.

EQUATION 3-11 SCS METHOD SHEET FLOW TRAVEL TIME[6]

$$T_t = \frac{\left[0.42 * (nL)^{0.8}\right]}{(P_2)^{0.5} * (S)^{0.4}}$$

T_t - Travel time (min)

n - Manning's roughness coefficient

L - Length of flow path (ft)

P_2 - 2-year, 24-hour rainfall = 4.2 in

S - Ground slope along flow path (ft/ft)

3.5.7.2 Shallow Concentrated Flow

After a short distance, sheet flow normally becomes shallow concentrated flow. The average velocity for flow of this type can be estimated depending on the surface roughness.

EQUATION 3-12 SCS SHALLOW CONCENTRATED VELOCITY[6]

Unpaved $V = 16.1345(S)^{0.5}$

Paved $V = 20.3282(S)^{0.5}$

V - Average velocity (ft/s)

S - Ground slope along flow path (ft/ft)

3.5.7.3 Open Channel or Storm Drain Flow

The velocity for flow in open channels, within pipes or other conduits can be calculated using Manning's equation (see Chapter 6). For open channels, the velocity should be based on depths of flow to the top of the main channel (bankfull conditions). For storm drains, the velocity should be based on full flow capacity. Once the flow velocity is calculated, the travel time can be obtained from Equation 3-13:

EQUATION 3-13 OPEN CHANNEL FLOW TRAVEL TIME[6]

$$T_t = \frac{L}{60V}$$

TABLE 3-8, MANNING'S "N" VALUES FOR SHEET FLOW

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated Soils:	
Residue Cover <20%	0.06
Residue Cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information compiled by Engman (1986).
[2] Includes species such as weeping lovegrass, blue grass, buffalo grass, blue grama grass, and native grass mixtures.
[3] When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

3.5.8 Design Storms

The City of Georgetown uses the City of Austin three (3) hour design storm for all SCS method hydrology. If the drainage area time of concentration exceeds this duration, the three hour storm may not be appropriate. A 24-hour storm with TP-40 rainfall depths is recommended for these basins, but final approval of the alternate design storms must be approved by the City of Georgetown. Prior approval is also required if it is desired to reduce rainfall depths due to areal extent. Incremental and cumulative design storm precipitation values for the three hour storm are presented in Table 3-9 and Table 3-10, respectively.

TABLE 3-9, CUMULATIVE DESIGN STORM PRECIPITATION VALUES[2]

Time (Minutes)	2- Year	5- Year	10- Year	25- Year	50- Year	100-Year
5	0.01	0.03	0.03	0.04	0.05	0.06
10	0.03	0.05	0.07	0.09	0.11	0.12
15	0.04	0.08	0.11	0.14	0.17	0.19
20	0.06	0.11	0.15	0.19	0.23	0.27
25	0.08	0.15	0.19	0.25	0.29	0.34
30	0.97	0.18	0.24	0.31	0.37	0.43
35	0.12	0.22	0.29	0.38	0.44	0.51
40	0.15	0.27	0.35	0.45	0.53	0.61
45	0.17	0.32	0.41	0.53	0.62	0.72
50	0.21	0.37	0.48	0.62	0.07	0.84
55	0.25	0.44	0.56	0.72	0.83	0.97
60	0.30	0.51	0.65	0.84	0.95	1.13
65	0.35	0.60	0.76	0.98	1.08	1.31
70	0.43	0.71	0.89	1.14	1.24	1.53
75	0.53	0.86	1.07	1.36	1.43	1.80
80	0.69	1.06	1.31	1.65	1.68	2.17
85	0.61	1.39	1.67	2.19	2.00	2.72
90	1.48	2.03	2.39	3.01	2.50	3.71
95	1.84	2.46	2.89	3.53	3.41	4.42
100	2.03	2.72	3.18	3.88	4.06	4.87
105	2.15	2.89	3.38	4.13	4.46	5.18
110	2.24	3.02	3.53	4.32	4.74	5.43
115	2.31	3.12	3.65	4.47	4.95	5.63
120	2.36	3.20	3.75	4.60	5.13	5.79
125	2.41	3.27	3.83	4.71	5.27	5.94
130	2.44	3.33	3.91	4.80	5.40	6.06
135	2.47	3.38	3.98	4.89	5.51	6.17
140	2.49	3.43	4.04	4.97	5.60	6.27
145	2.52	3.47	4.09	5.03	5.68	6.37
150	2.54	3.51	4.14	5.10	5.75	6.45
155	2.56	3.54	4.19	5.16	5.82	6.53
160	2.58	3.57	4.23	5.21	5.89	6.61
165	2.59	3.60	4.27	5.26	5.95	6.68
170	2.61	3.63	4.31	5.31	6.01	6.74
175	2.62	3.66	4.34	5.36	6.06	6.81
180	2.64	3.68	4.37	5.40	6.11	6.86

TABLE 3-10, INCREMENTAL DESIGN STORM PRECIPITATION VALUES[2]

Time (Minutes)	2- Year	5- Year	10- Year	25- Year	50- Year	100- Year
0	0.000	0.000	0.000	0.000	0.000	0.000
5	0.013	0.025	0.034	0.044	0.052	0.061
10	0.014	0.027	0.036	0.047	0.056	0.064
15	0.015	0.029	0.038	0.050	0.058	0.068
20	0.017	0.031	0.041	0.053	0.062	0.073
25	0.018	0.034	0.044	0.057	0.067	0.077
30	0.020	0.037	0.047	0.061	0.072	0.083
35	0.023	0.040	0.051	0.067	0.077	0.089
40	0.025	0.044	0.057	0.073	0.085	0.098
45	0.029	0.049	0.063	0.080	0.094	0.108
50	0.034	0.056	0.070	0.089	0.104	0.119
55	0.040	0.054	0.079	0.101	0.117	0.135
60	0.048	0.075	0.092	0.117	0.135	0.154
65	0.059	0.090	0.109	0.138	0.159	0.181
70	0.076	0.112	0.134	0.168	0.192	0.219
75	0.104	0.146	0.172	0.214	0.244	0.275
80	0.153	0.205	0.238	0.291	0.329	0.369
85	0.254	0.324	0.368	0.540	0.494	0.549
90	0.540	0.640	0.720	0.820	0.910	0.990
95	0.356	0.437	0.494	0.520	0.648	0.713
100	0.193	0.253	0.290	0.352	0.398	0.443
105	0.124	0.141	0.200	0.247	0.281	0.316
110	0.088	0.127	0.151	0.189	0.216	0.244
115	0.067	0.100	0.121	0.151	0.175	0.198
120	0.053	0.082	0.100	0.127	0.146	0.167
125	0.043	0.069	0.086	0.109	0.126	0.144
130	0.036	0.060	0.075	0.096	0.111	0.124
135	0.031	0.052	0.066	0.085	0.099	0.113
140	0.027	0.047	0.059	0.076	0.089	0.102
145	0.024	0.042	0.054	0.069	0.081	0.094
150	0.021	0.038	0.050	0.064	0.075	0.087
155	0.019	0.035	0.046	0.060	0.069	0.080
160	0.017	0.032	0.042	0.055	0.065	0.074
165	0.016	0.030	0.040	0.051	0.061	0.070
170	0.015	0.028	0.037	0.048	0.057	0.066
175	0.014	0.026	0.035	0.046	0.054	0.062
180	0.013	0.024	0.033	0.043	0.051	0.059

3.6 Channel Hydrograph Routing

Many methods exist for the routing of flood hydrographs through a reach to account for the attenuation effects of channel storage. The use of a channel routing method not included in this Manual must be approved by the City of Georgetown on a case by case basis.

Unnecessary channel routing should be avoided. The travel time through a downstream subbasin must exceed five (5) minutes before channel routing will be allowed.

3.6.1 TR-20 Channel Routing Methods

The approved channel routing methods for use in the TR-20 program include the modified Att-Kin and Muskingum methods, with the modified Att-Kin method recommended.

The theory and procedures for using the modified Att-Kin method are given in the TR-20 user's manual, and information on the Muskingum method can be found in most hydrology texts, such as Chow[7].

3.6.2 HEC-1 Channel Routing Methods

The approved channel routing methods for use in the HEC-1 program include the Muskingum and Muskingum Cunge methods, with the Muskingum Cunge method recommended.

The theory and procedures for using the Muskingum and Muskingum Cunge methods are found in most hydrology texts, such as Chow[7].

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[1] Kuichling, E., The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts, Trans. Am. Soc. of Civil Engineers, vol. 20, pp. 1-56, 1889.

[2] Drainage Criteria Manual. City of Austin, Texas, March 1996.

[3] HEC-1 Flood Hydrograph Package User's Manual. U.S. Army Corps of Engineers, September 1990.

[4] TR-20, Project Formulation – Hydrology. Soil Conservation Service (currently National Resource Conservation Service), May 1982.

[5] National Engineering Handbook, Section 4 – Hydrology. Soil Conservation Service (currently National Resource Conservation Service), March 1985.

[6] TR-55, Urban Hydrology for Small Watersheds. Soil Conservation Service (currently National Resource Conservation Service), June 1986.

[7] Chow, V.T., Applied Hydrology, 1988.

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4. REGIONAL STORMWATER MANAGEMENT PROGRAM

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REGIONAL STORMWATER MANAGEMENT PROGRAM PARTICIPATION NOT CURRENTALLY AVAILBLE THROUGH THE CITY OF GEORGETOWN

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5. DETENTION

5.1 Introduction

Stormwater detention facilities provide temporary storage for the increased stormwater runoff resulting from development. Urbanization alters the hydrology of a watershed by improving its drainage efficiency, reducing its surface infiltration and reducing its storage capacity. This results in increased peak discharge and runoff volume which can adversely affect downstream floodplains and accelerate erosion processes. Under favorable conditions, the attenuation of peak discharge can mitigate the impact to downstream residents and drainage systems. This Chapter presents the general design criteria for detention facilities in the City of Georgetown.

The most effective stormwater management results from planning on a watershed-wide basis. For this reason, a determination of the feasibility of participation in the Regional Stormwater Management Plan is available from the City of Georgetown before or during the preliminary design of all proposed projects (see Chapter 4).

The City of Georgetown encourages innovative designs that allow recreational and community uses in addition to stormwater detention. An attempt should be made in the development planning stages to investigate the possibility of multi-use detention facilities.

5.2 Detention Requirements

A. Peak runoff rates leaving the project must be controlled to levels below existing conditions for the 2-, 10-, 25- and 100-year design storms.

B. Stormwater runoff from a proposed project must produce no significant impact to the adjacent downstream property.

C. Where no conveyance system exists at the adjacent downstream property line and the discharge was previously unconcentrated flow, the runoff must be conveyed across the downstream property(ies) as sheet flow at depths no greater than existing conditions, or within drainage easements secured from the downstream owners and recorded with Williamson County prior to drainage plan approval.

D. Peak runoff control is not required for a proposed project in the following situations:

1. The proposed project site post-developed peak runoff rate for the 100-year design storm is calculated to be less than 0.5 cfs greater than the existing 100-year peak runoff rate, OR
2. The fully developed project proposes to construct less than 2,500 square feet of impervious cover, OR
3. The proposed project has obtained approval to participate in the Regional Stormwater Management Program (see Chapter 4).

Waiver of the detention requirement for any reason does not relieve the project owner of responsibility under civil law to adjacent and downstream property owners.

5.3 Required Technical Information to be Submitted for Review

Detailed hydraulic calculations shall be provided for all detention facilities. Stage-discharge-storage tables must be presented with all outlet structure components, such as orifice, weir, and outlet conduit flows clearly indicated. All assumptions and coefficient values must be included.

All plans, specifications, and reports documenting detention facilities should contain the following:

A. Stage-Discharge-Storage tables with stage values (at intervals not exceeding one foot) ranging from the bottom flowline of the structure to the maximum elevation in the pond.

B. The configuration (dimensions and flowline) and stage-discharge relationship (equation) for each outflow opening (such as orifices, weirs, and conduits).

C. All coefficients and assumptions included in the analysis.

5.4 Design Standards and Criteria

5.4.1 General Criteria

A. Detention facilities must meet or exceed the criteria in this Manual. The use of these criteria does not relieve the design engineer of the responsibility for the adequacy and safety of all aspects of the design of the detention pond.

B. All detention facilities must be analyzed with the hydrograph methods and routing procedures described in this Manual.

C. Detention facilities which measure greater than six feet in height are subject to Title 30, Texas Administrative Code (TAC 30) Chapter 299. The height of a detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment) including the natural stream channel. Chapter 299 classifies dam sizes and hazard potential and specifies required failure analyses and spillway design flood criteria.

D. No outlet structure from a detention facility shall be designed to discharge concentrated flow onto a street or sidewalk.

E. Private parking lots, paved storage areas, and other paved surfaces may be considered as part of the detention volume for the post-developed peak rates of runoff provided that:

   1. runoff is not detained in the parking lot for storms up to the two (2) year frequency to avoid nuisance flooding; AND

2. the maximum depth of water detained can not exceed one (1.0) foot at any location in the parking lot; AND
3. the minimum gradient of the parking lot subject to flooding is one (1) percent for asphalt and 0.5 percent for concrete surfaces.
4. Ponds in parking lots must place signs in areas of detention indicating its use as a stormwater management facility.

5.4.2 Performance Criteria

Detention facilities are classified by contributing drainage area, as follows:

TABLE 5-1, POND CLASSIFICATION

Pond Class	Drainage Area
Minor	< 20 acres
Medium	20 $\leq$ 64 acres
Major	> 64 acres

5.4.2.1 Minor and Medium Class Ponds

Minor and Medium class ponds must meet the standard criteria for peak runoff control as set forth in this Manual. Off-site areas draining to these detention facilities may be assumed to remain in the existing development condition for peak flow management control. However, an emergency spillway must be designed to pass the fully developed 100-year peak flow rate while maintaining the required freeboard.

5.4.2.2 Major Class Ponds

For major class ponds, the possibility of dam failure due to overtopping of the dam embankment must be considered. Downstream flood damage and loss of life must be evaluated and, if a significant hazard exists, the dam must be adequately designed to prevent such hazards. Specific dam criteria for storm events with frequencies in excess of the 100-year frequency shall be established by the City of Georgetown on a case by case basis. These criteria should be established in the preliminary design phase. Major class ponds located on natural watercourses must assume fully developed contributing drainage areas.

5.4.3 Safety Criteria

A. The spillway, embankment, and outlet structures from each detention facility must be designed to safely pass the required design storm with the freeboard shown in Table 5-2.

TABLE 5-2, REQUIRED FREEBOARD FOR DESIGN STORM

Pond Class	Design Storm	Required Freeboard
Minor	100-year	0.5 ft
Medium	100-year	1.0 ft
Major	*	*

* The required design storm event and freeboard will be determined by the City of Georgetown on a case by case basis, and may include a dam break analysis based on the principles outlined in Chapter 299 of Title 30, Texas Administrative Code.

The minimum top width for earthen embankments is shown in Table 5-3.

TABLE 5-3, MINIMUM REQUIRED TOP WIDTH FOR EARTHEN EMBANKMENTS

Total Height of Embankment, ft.	Minimum Top Width, ft.
0.0 to 4.0	6.0
4.1 to 8.0	8.0
8.0 to 12.0	10.0
12.1 +	12.0

B. Vegetated earthen embankment side slopes must be no steeper than 3H:1V (3 horizontal to 1 vertical). Proposed earthen embankments with a constructed height greater than 10 feet will require analysis and construction specifications by a professional civil-geotechnical engineer.

C. Concrete vertical retaining walls shall be designed by a professional civil-structural engineer.

D. A chain link fence is required along the perimeter of the pond for facilities with side slopes greater than 3H:1V. The fence shall be a minimum of four (4) feet in height. Access gates to the pond shall be 16 feet in width consisting of two (2) swinging sections eight (8) feet in width. Other fencing materials may be approved by the City of Georgetown on a case by case basis.

E. Chain link fences a minimum of four (4) feet in height shall be constructed on all wingwalls and headwalls greater than three (3) feet in height.

F. Earthen pond bottom slopes must be one (1) percent or greater. If an earthen low flow channel is provided, a bottom slope of 0.8% will be allowed, provided all points in the pond drain to the low flow channel at a minimum of 1.0%. The use of concrete low flow channels within earthen detention facilities is not recommended.

G. The minimum bottom slope for a concrete pond must be 0.5% or greater.

H. Spillways must be stable and non-erosive.

5.4.4 Erosion Control

Adequate erosion control and revegetation must be accomplished during and following construction of the pond. Specific requirements are identical to those required for open channels. These requirements are discussed in Chapter 6, Section 6.5.

5.5 Detention Analysis and Design Process

The process for the design and analysis of a typical detention pond is described below.

A. Calculate Runoff Hydrographs for existing conditions (refer to Chapter 3 for more information on the steps described below):

1. Identify and delineate the overall drainage basin for each discharge point from the proposed project under existing conditions.
2. Identify existing land uses.
3. Identify existing soil types using SCS soil survey or on-site evaluation.
4. Identify existing drainage features such as streams, conveyance systems, detention facilities, etc.
5. Select and delineate pertinent subbasins based on existing conditions:
6. Select homogeneous (having generally consistent soils, topography, and land use) subbasin areas.
7. Select separate subbasin areas for on-site and off-site drainage.
8. Select subbasin areas for major drainage features.
9. Determine runoff parameters for each subbasin under existing conditions:
10. Determine contributing acreage.
11. Identify pervious and impervious areas.
12. Select SCS runoff curve numbers.
13. Compute time of concentration.
14. Determine allowable release rates for the appropriate design storm frequencies:
15. Compute existing condition hydrographs for each design storm frequency from each subbasin.
16. For subbasins which drain to existing detention ponds or are conveyed for significant distances in open channels, use storage or channel hydrograph routing to calculate the attenuated versions of the hydrographs.
17. Sum the appropriate subbasin hydrographs to obtain the total existing runoff hydrographs for each design storm.
18. The appropriate release rate for each design storm frequency under developed conditions is the peak flow rates calculated from the existing runoff hydrographs.

B. Repeat the above steps for the proposed developed site conditions.

C. Design and size the detention facility:

1. Compute developed condition runoff hydrographs.
2. Sum the appropriate subbasin hydrographs to obtain the total developed runoff hydrographs for each design storm frequency.
3. Use the developed runoff hydrographs and an iterative process to size the detention facility to meet the allowable release rates.
4. Allow adequate provisions for maintenance to insure the facility does not detract from the value of adjacent properties.
5. Always design with safety, maintenance, and aesthetics in mind.

5.6 Preliminary Detention Pond Sizing

Many methods exist for estimating required detention storage volume. One common method for the preliminary sizing of detention facilities is outlined in the SCS TR-55 publication. The TR-55 method requires a peak inflow rate and a peak outflow rate along with watershed characteristics to compute the approximate storage. The volume required is based on Equation 5-1.

EQUATION 5-1, APPROXIMATE REQUIRED DETENTION VOLUME[1]

$$V_s = V_r\left[C_0 + C_1\left(\frac{q_o}{q_i}\right) + C_2\left(\frac{q_o}{q_i}\right)^2 + C_3\left(\frac{q_o}{q_i}\right)^3\right]$$

V_s - *Storage Volume required for given frequency storm (ac. ft.)*

V_r - *Runoff Volume for developed conditions and given frequency storm (ac. ft.)*

C_0, C_1, C_2, and C_3 - *SCS coefficients that vary for Type I, II, and III storms(see Table 5-4)*

q_o - *pre-developed peak flow rate for given frequency storm (cfs)*

q_i - *developed peak flow rate for given frequency storm (cfs)*

Although designed for the SCS graphical or tabular peak flow rate method and a twenty-four (24) hour storm distribution, reasonable results for the preliminary sizing of a detention pond can be obtained with this method. Table 5-4 presents the coefficients used in the SCS method.

TABLE 5-4, COEFFICIENTS FOR TR-55 PRELIMINARY DETENTION POND SIZING[1]

Rainfall Distribution	C_0	C_1	C_2	C_3
I, IA	0.660	-1.76	1.96	-0.730
II, III	0.682	-1.43	1.64	-0.804

5.7 Storage Routing of Hydrographs

The continuity equation for reservoir routing can be expressed as:

Inflow Volume – Outflow Volume = Change in Storage Volume

Equation 5-2 shows the numerical form of this equation:

EQUATION 5-2, CONTINUITY EQUATION FOR STORAGE ROUTING[2]

$$\left[\frac{I_1 + I_2}{2}\right] - \left[\frac{O_1 + O_2}{2}\right] = \frac{\Delta S}{\Delta t} = S_2 - S_1$$

I - Inflow at time 1 and time 2 (cfs)

O - Outflow at time 1 and time 2 (cfs)

S - Storage at time 1 and time 2 (cu ft)

t - Time step or interval, 2-1 (s)

Equation 5-3 eliminates the Δt variable and rearranges Equation 5-2:

EQUATION 5-3, NUMERICAL STORAGE ROUTING[2]

$$I_1 + I_2 + 2S_1 - O_1 = O_2 + 2S_2$$

All terms on the left hand side of the equation are known from the inflow hydrograph and the storage and outflow values from the previous time step. The unknowns O_2 and S_2 can be solved iteratively by the TR-20 and HEC-1 programs from given stage-storage and stage-discharge relationships.

5.7.1 Stage-Storage Table

A stage-storage table defines the relationship between the depth of water and storage volume for a given pond. The steps below explain this relationship.

A. Develop topographic contours for the proposed pond and planimeter (or otherwise compute) the area enclosed by each contour.

B. Calculate the average area between each contour, as shown in Equation 5-4 or Equation 5-5.

EQUATION 5-4, AVERAGE AREA BETWEEN CONTOURS (AVERAGE END METHOD)

$$\text{Average Area}_{1,2} = \frac{(\text{Area of Contour}_1) + (\text{Area of Contour}_2)}{2}$$

EQUATION 5-5, AVERAGE AREA BETWEEN CONTOURS (CONIC METHOD)

$$\text{Average Area}_{1,2} = \frac{(\Delta \text{ elevation between contours})}{3}\left(A_1 + A_2 + \sqrt{A_1 + A_2}\right)$$

C. Calculate the volume between contours by multiplying the average area between contours by the difference in elevation.

EQUATION 5-6, VOLUME BETWEEN CONTOURS

$$\text{Volume Between Contours}_{1,2} = (\text{Average Area}_{1,2})(\text{Elevation}_2 - \text{Elevation}_1)$$

D. Calculate the total storage below each contour. This is the sum of the volumes computed in the previous step up to the contour in question.

5.7.2 Stage-Discharge Table

A Stage-Discharge table defines the relationship between the depth of water and the outflow from a given pond. The table should clearly indicate all outflow structure components such as orifice, weir, and outlet conduit flows.

A prime consideration in developing a stage-discharge table is downstream water surface elevation (tailwater) on the outflow structure. This tailwater elevation may affect the discharge capacity of the outflow structure and must be considered in determining the outflow versus stage relationship. The design of pond outlet structures is discussed in the following section.

5.8 Outlet Control Structures

The detention facility outlet restricts flow to a controlled rate to meet runoff criteria for a proposed project. Although many types and combinations of outlet works are possible, most consist of a combination of weirs and orifices in conjunction with outlet pipes.

5.8.1 Orifice Flow

An orifice in a detention pond is usually the primary outlet where a regulated amount of water can flow through. An orifice can be defined as an opening in a thin plate with a closed perimeter and of regular shape through which water flows. The opening of the outlet can be round, square, rectangular or any other convenient shape.

When the size of the orifice is large when compared to the water depth, the opening is unsubmerged, and orifice equations are not accurate. Until the ratio of water depth to orifice diameter is greater than 1.2, more accurate results are possible using inlet control nomographs for culverts published by the Federal Highway Administration (see Chapter 8).

Equation 5-7 presents the basic orifice equation.

EQUATION 5-7, ORIFICE DISCHARGE[3]

$$Q = C_d A \sqrt{2gH}$$

Q - discharge through orifice (cfs)

C_d - orifice discharge coefficient

A - orifice cross-section area (sq. ft)

g - gravitational constant (32.2 ft/s^2)

H - head from orifice centerline to water surface (ft)

The minimum orifice diameter (or dimension for a rectangular orifice) is 12 inches. Openings of less than twelve (12) inches are very susceptible to clogging and are not recommended.

An opening in a thick wall (a wall whose width is more than roughly three orifice diameters) and pipes used as pond outlets should be analyzed with culvert analysis methods of Chapter 8. Factors creating an outlet control condition must always be considered in the design of detention facilities.

5.8.2 Culverts as Outlet Structures

Refer to the procedures and equations in Chapter 8 when using culverts as outlets to detention ponds.

5.8.3 Weir Flow

Weirs are often used as the primary overflow control device. The edge or surface over which the water flows is called the crest. The overflowing sheet of water is called the nappe. Weirs are classified as either sharp-crested or broad crested. A sharp-crested weir has a thin crest and sharp upstream edge such that the nappe will spring clear of the crest in a free-fall condition on the downstream side of the weir (assuming no tailwater submergence). Spillways constructed in earthen embankments should be analyzed as broad-crested weirs. Equation 5-8 is the standard weir flow equation.

EQUATION 5-8, DISCHARGE OVER A WEIR[3]

$$Q = C_w L H^{1.5}$$

Q - *weir flow rate (cfs)*

C_w - *Weir Coefficient*

L - *horizontal length of weir crest (ft)*

H - *head above weir crest elevation (ft)*

The value of the weir coefficient is dependent on many parameters, but to standardize design, a value of 3.0 should be used in Equation 5-8 for sharp-crested rectangular weirs unless the use of a different value is justified. See Brater and King[3] for more discussion.

Spillways constructed in earthen embankments should use Equation 5-8 with Table 5-5 values unless the use of a different value is justified.

TABLE 5-5, BROAD CRESTED WEIR COEFFICIENT[3]

Description	C_w
1 ft. Wide	3.00
5 ft. Wide	2.70
15 ft. Wide	2.60

Triangular, or V-notch weir discharge can be calculated with Equation 5-9.

EQUATION 5-9, V-NOTCH WEIR DISCHARGE[3]

$$Q = C_v \tan\left(\frac{\theta}{2}\right) H^{2.5}$$

Q - *weir discharge (cfs)*

C_v - *v-notch weir coefficient*

θ - *angle of v-notch (degrees)*

H - *head above apex of notch (ft)*

5.8.4 Trash Racks

Although not required for minor or medium facilities, trash racks should be considered if excessive debris is expected, or if the outlet presents a potential hazard to children. The following guidelines should be followed when designing trash racks:

A. For outlets less than 24 inches, the net opening of the trash rack should have no less than ten times the area of the outlet opening.

B. For outlets larger than 24 inches, the net open area of the trash rack should have no less than four times the area of the outlet opening.

C. The spacing between the openings on the trash rack should be smaller than the smallest dimension of the outlet.

D. For large outlets, the spacing of the openings on the trash rack should be six inches or less to prevent a child from passing through the rack.

E. Racks should be sloped at about 3H:1V to 5H:1V to facilitate cleaning. This allows floating debris to “ride up” the rack as the pond level rises.

F. Racks should be hinged or bolted to the outlet structure to allow removal for maintenance and repair.

[1] TR-55, Urban Hydrology for Small Watersheds. Soil Conservation Service (currently National Resource Conservation Service), June 1986.

[2] Chow, V.T., Applied Hydrology, 1988.

[3] King, H.W. and Brater, E.F., Handbook of Hydraulics, 6th Edition, 1976.

END OF SECTION

6. OPEN CHANNELS

6.1 Introduction

The hydraulic design of a channel is of primary importance to insure that flooding and erosion problems are not aggravated or created. This Chapter summarizes methodologies, procedures, and criteria to be used in the hydraulic analysis of most design problems in the Georgetown area. If methodologies and parameters not discussed in this Manual are justified, prior approval of the City of Georgetown is required.

6.2 General Requirements

A. The floodplain limits for the twenty-five (25) and one hundred (100) year storm event shall be determined for water courses draining twenty (20) or more acres. Floodplain calculations for these storm events shall utilize generally recognized backwater computational methods and actual field channel and overbank configurations.

B. No importation of fill material or channel modifications shall be undertaken within the area of the one hundred (100) year floodplain without written approval of the Floodplain Coordinator. Such approval shall be based upon certified engineering data and calculations furnished by the applicant.

C. All development activity within the regulatory floodplain must comply with City and Federal Emergency Management Agency (FEMA) floodplain management regulations.

6.3 Required Technical Information to be Submitted for Review

Appropriate hydraulic design calculations and data to be submitted for city review includes, at a minimum:

A. A topographic map of the subject drainage basin showing proposed drainage areas and proposed channel locations.

B. Design flow rate calculations carried out as described in this Chapter.

C. For design using the Manning’s Equation, a listing of the following parameters:

1. Flow depth (ft)
2. Channel slope (ft/ft)
3. Channel flow (cfs)
4. Manning’s “n” value
5. Channel side slopes (H:V)
6. Channel bottom width (ft)

D. Existing and proposed channel cross sections with locations delineated on the drainage map.

6.4 Channel Design Criteria

Open channels can be classified as natural or constructed. Natural channels are defined as those which have occurred naturally due to the flow of surface waters, or those that although constructed or modified by human activity have taken on the appearance of a natural channel. The cross section of a natural channel ordinarily consists of a main channel that conveys low flows and a floodplain that transports flood flows. They may vary hydraulically along each channel reach and should be left in their natural condition wherever feasible in order to maintain the natural hydrologic functions and wildlife habitat benefits from established vegetation. Constructed channels are man-made and generally have uniform geometric cross sections.

Constructed channels should be designed to resemble natural channels when possible. This design approach helps to reduce channel erosion and therefore, is more stable and requires less maintenance. It is worth noting that most natural streams will not remain stable during urbanization without some man-made improvements.

Channels with a contributing drainage area of less than twenty (20) acres at the section in question are further classified as minor channels, while those with contributing areas of over twenty (20) acres are classified as major channels.

6.4.1 Minor Channel Design Criteria

For natural or constructed channels with a contributing area of less than twenty (20) acres, with grass or other flexible lining (for concrete channels, see Section 4.3.5), the following criteria apply:

A. Freeboard - Channels must be designed to convey the 100-year frequency storm peak flow with a minimum freeboard of one-half (0.5) foot.

B. Geometry - Constructed channel cross sections are normally triangular (v-shaped) or trapezoidal in shape.

C. Bottom Width - For trapezoidal channels, a minimum bottom width of six (6) feet is required.

D. Side slopes - For earthen channels, side slopes must be 3H:1V or flatter.

E. Longitudinal Slope - The minimum longitudinal slope is one (1.0) percent. If topographic constraints or the use of a natural channel require a flatter slope, prior approval from the City of Georgetown must be obtained.

F. Shear Stress - The required lining for a channel is based on the shear stress generated by the 25-year frequency peak flow rate. Maximum shear stresses for various linings are shown in the Georgetown Construction Standards and Specifications.

G. Bends - The minimum radius of curvature for channel bends is two times the 100-year peak flow rate top width (but not less than 100 feet), and the maximum bend angle is ninety (90) degrees.

H. Intersection Angles - The angle of intersection between a side and main channel should be 15 to 45 degrees (angle measured between channel centerlines on upstream side of point of intersection).

I. Flow rates should remain subcritical at all sections along the channel, with the exception of drop or other energy-dissipating structures.

6.4.2 Major Channel Design Criteria

For constructed channels with a contributing area of twenty (20) acres or more, with grass or other flexible lining (for concrete channels, see section 4.3.5), the following criteria apply:

A. Freeboard - Channels must be designed to convey the 100-year frequency storm peak flow with a minimum freeboard of one (1) foot. The entire channel area, including freeboard, must be contained within a drainage easement or right of way.

B. Geometry - See Section 6.4.3.

C. Side slopes - See Section 6.4.3.

D. Longitudinal Slope - The minimum channel slope is 0.08 percent. If topographic constraints require a flatter slope, prior approval from the City of Georgetown must be obtained.

E. Shear Stress - The required lining for a channel is based on the shear stress generated by the 25-year frequency peak flow rate. Maximum shear stresses for various linings are shown in the Georgetown Construction Standards and Specifications.

F. Bends - The minimum radius of curvature for channel bends is two times the 100-year peak flow rate top width (but not less than 100 feet), and the maximum bend angle is ninety (90) degrees.

J. Intersection Angles - The angle of intersection between a side and main channel should be 15 to 45 degrees (angle measured between channel centerlines on upstream side of point of intersection).

K. Flow rates should remain subcritical at all sections along the channel, with the exception of drop or other energy-dissipating structures.

6.4.3 Composite Channels

For major constructed channels, a composite cross section is required. The design of the channel must meet the following requirements:

A. Channel lining must be grass or other flexible lining material.

B. Low flow (or Inset) channel

1. At a minimum, the low flow channel must have the capacity to convey the fully developed six-month storm.
2. The maximum storm to be conveyed within the low flow channel is the fully developed two-year storm.

3. The maximum side slopes for the low flow channel are 3H:1V, unless the low flow channel is lined in accordance with Section 6.6.

C. Main Channel

1. The maximum storm to be conveyed within the main channel is the fully developed ten (10) year storm.
2. The main channel transverse slope must be a minimum of two (2) percent towards the low flow channel.
3. The maximum side slopes for the main channel are 3H:1V, unless the main channel is lined in accordance with Section 4.6.

D. Flood Channel

1. The low flow, main channel, and flood channel area must have the capacity to convey the fully developed one hundred (100) year storm and provide the required freeboard.
2. The flood channel transverse slope must be a minimum of two (2) percent and a maximum of ten (10) percent towards the main channel.
3. The maximum side slopes for the flood channel are 3H:1V.

6.4.4 Modification of Major Natural Channels

It is the goal of the City of Georgetown to protect natural watercourse areas to the extent feasible, while not imposing an unreasonable burden on the development of land. If a major natural channel is to be modified to convey stormwater runoff, the main channel and immediate vegetation area should be left undisturbed, and the overbank conveyance capabilities improved by excavating the floodplain area. To this end, the following criteria must be followed when modifying major natural channels:

A. No modification of the existing natural channel is allowed within the fully developed 2-year floodplain (other than erosion protection and channel stability measures).

B. No concrete or other rigid linings are permitted in natural channels.

C. Stabilization techniques such as soil reinforcing mattress, bio-revetment, large boulders, and other flexible methods should be incorporated in potential areas of erosion.

In areas of low topographic relief and extremely wide floodplains, the City of Georgetown may approve alternate designs, provided the main channel area is left in a natural state. Alternate designs must be approved on a case by case basis, and it is recommended that the City of Georgetown be contacted as early as practicable in the design process.

6.4.5 Concrete Channels

The following are minimum requirements to be used in the design of all concrete lined channels:

A. All concrete channels should be trapezoidal in shape with a minimum bottom width of three (3) feet.

B. The maximum side slopes for a concrete channel are 2V:1H

C. A minimum freeboard of 1.5 feet above the one hundred (100) year storm elevation is required.

D. The maximum velocity for concrete lined channels is ten (10) feet per second.

Supercritical flow in an open channel in urbanized areas should be avoided due to the hazards associated with high velocity flows. Also, high sediment bed loads can increase the effective "n" value, shifting flows from supercritical to subcritical, resulting in over-bank flooding. Because of field construction conditions, a Manning's "n" value lower than 0.015 is not allowed.

6.5 Open Channel Hydraulics

Open channel flow refers to any flow that occupies a defined channel and has a free surface. This Section covers basic hydraulic principles used in typical open channel flow design and analysis. While important concepts are presented, detailed theory is not discussed. Texts such as Chow[1] or French[2] should be consulted for further information.

6.5.1 Flow Classification

Identifying the type of flow is an important factor in open channel hydraulics, as many design equations are developed for specific flow classes. Traditional flow types encountered in open channel hydraulics are discussed below.

6.5.1.1 Steady vs. Unsteady Flow

Steady flow occurs when the discharge, depth, and velocity at a specific section are constant with time. Unsteady flow refers to conditions that change with time at a specific point or section. For example, if at any specified location the velocity is changing with time, the flow is unsteady.

6.5.1.2 Uniform vs. Nonuniform Flow

Uniform flow requires that the discharge, depth, and velocity are all constant with distance along the channel. The channel slope, energy or friction slope, and water surface slope are all equal for uniform flow. The depth of uniform flow is called normal depth, and will be discussed further with Manning's equation. Nonuniform flow is characterized by changing depth and velocity with distance. If these changes take place over considerable lengths, such that vertical accelerations can be neglected, it is termed gradually varied flow. Uniform flow equations are often used for very short distance intervals within a gradually varied flow. Rapidly varied flow produces abrupt changes in depth and velocity over very short distances. Rapidly varied flow usually involves wave phenomena which cannot be calculated with uniform flow equations.

6.5.1.3 Subcritical vs. Supercritical Flow

The flow in open channels may also be classified according to the energy level in the flow itself. This can be represented by the Froude number. The Froude number represents the ratio of inertial forces to gravity forces, and is presented in Equation 6-1:

EQUATION 6-1, FROUDE NUMBER[1]

$$F_r = \frac{v}{\sqrt{g d_m}}$$

F_r - Froude number

D_m - hydraulic depth (ft)

The hydraulic depth, d_m, is calculated with Equation 6-2:

EQUATION 6-2, ,HYDRAULIC DEPTH[1]

$$d_m = \frac{A}{T_w}$$

A - cross sectional flow area (ft^2)

T_w - Free surface top width (ft)

A Froude number of less than 1.0 defines the subcritical range of flow and is characterized by low velocities and high depths, and is found primarily on hydraulically mild slopes. Supercritical flow has a Froude number greater than 1.0, and is characterized by high velocities and low depths, and occurs primarily on hydraulically steep slopes. Critical flow occurs when the Froude number is equal to 1.0. The depth at critical flow is a function of the discharge and channel geometry only. For a given discharge and cross section, only one critical depth exists. Equation 6-3 presents the general equation for critical depth for any channel shape:

EQUATION 6-3, CRITICAL DEPTH[1]

$$\frac{Q_c^2}{g} = \frac{A_c^3}{T}$$

Q_c - critical discharge (cfs)

A_c - cross sectional area at critical depth (ft^2)

T - free water surface top width at critical discharge (ft)

Formulas for various geometric cross sections can be derived from Equation 6-3. For example, critical depth in a rectangular conduit can be calculated with Equation 6-4:

EQUATION 6-4, CRITICAL DEPTH FOR RECTANGULAR CONDUIT

$$d_c = \sqrt[3]{\frac{q^2}{g}}$$

d_c - critical depth (ft)

q - discharge per unit width of rectangular channel (cfs/ft)

For a circular conduit section, the following equations allow determination of the area, top width, and wetted perimeter, respectively.

EQUATION 6-5, CRITICAL DEPTH AREA FOR CIRCULAR CONDUIT

$$A = \frac{D^2}{g}\left[2\cos^{-1}\left(1-\frac{2d}{D}\right)-\sin\left(2\cos^{-1}\left(1-\frac{2d}{D}\right)\right)\right]$$

EQUATION 6-6, CRITICAL DEPTH TOP WIDTH FOR CIRCULAR CONDUIT

$$T = D\sin\left(\cos^{-1}\left(\frac{2d-D}{D}\right)\right)$$

EQUATION 6-7, WETTED PERIMETER FOR CIRCULAR CONDUIT

$$WP = D\cos^{-1}\left(1-\frac{2d}{D}\right)$$

A - cross sectional area of flow (ft^2)

T - free water surface top width at critical discharge (ft)

WP - wetted perimeter (ft)

d - depth of flow (ft)

D - diameter of pipe (ft)

6.5.1.4 Continuity

The Continuity equation is the expression of conservation of mass in fluid mechanics. For steady flow in open channels, the continuity principle is:

EQUATION 6-8, CONTINUITY PRINCIPLE[1]

$$Q = A_1V_1 = A_2V_2$$

Q - discharge (cfs)

A - flow cross sectional area (sq. ft.)

V - mean cross sectional velocity (ft/s)

Subscripts 1 and 2 refer to successive cross sections

6.5.1.5 Energy

The total energy in open channel flow at any point, also called the total head, is the sum of potential, pressure, and kinetic energies. The energy conservation principle states that the sum of these energies does not change along the channel, only the distribution among the individual energy components. As shown in Equation 6-9, the total energy at a given point in the flow is equal to:

EQUATION 6-9, TOTAL ENERGY[1]

$$H = z + y\cos\theta + \frac{P}{\gamma} + \alpha\left(\frac{v^2}{2g}\right)$$

H - total energy head (ft)

z - elevation head (ft)

P - pressure (lbs/sq ft)

γ - unit weight of water (lb/cu. Ft)

α - velocity distribution coefficient

v - average velocity(ft/s)

y - depth of flow normal to channel bed (ft)

$\cos\theta$ - angle between channel slope and horizontal datum

g - gravitational constant (ft/s^2)

The velocity distribution coefficient α is used to account for variations in velocity throughout a cross section. Because the velocity distribution in a channel varies from a maximum in the main channel to almost zero in overbank areas, the average velocity head is not a true measure of the kinetic energy in the flow. A weighted average of the kinetic energy head is obtained by multiplying the average velocity head by the velocity distribution coefficient. The velocity distribution coefficient is determined from the flow rate and velocity for subareas in the cross section (i.e. left overbank, channel, and right overbank).

EQUATION 6-10, VELOCITY DISTRIBUTION COEFFICIENT[1]

$$\alpha = \frac{\left[Q_1 v_1^2 + Q_2 v_2^2 + \ldots Q_n v_n^2\right]}{Q\overline{v^2}}$$

$Q_{1,2..n}$ - flow rate in subarea 1,2…n of cross section (cfs)

$V_{1,2,\ldots n}$ - velocity in subarea 1,2…n of cross section (ft/s)

V = average velocity at cross section (ft/s)

Some simplifying assumptions can be made to Equation 6-9. For most channel slopes encountered in practice, the angle between the channel slope and the horizontal datum is small enough to neglect it in the computations. Hydrostatic pressure distribution is also a reasonable assumption for mild slopes, which cancels the pressure term equation. The velocity distribution coefficient is approximately 1.0 for prismatic channels, eliminating it from the equation for most constructed channels. With these simplifications, the total energy at a point can then be expressed in the form of :

EQUATION 6-11, TOTAL ENERGY IN MILD SLOPE PRISMATIC CHANNELS[3]

$$H = z + y + \left(\frac{v^2}{2g}\right)$$

For natural channels, the pressure term and angle between bed slope and datum are normally neglected. The energy or head loss between two sections can then be expressed with by Equation 6-12, the energy equation:

EQUATION 6-12, ENERGY EQUATION[3]

$$z_1 + y_1 + \alpha\left(\frac{v_1^2}{2g}\right) = z_2 + y_2 + \alpha\left(\frac{v_2^2}{2g}\right) + H_L$$

H_L - sum of all energy losses between section 1 and 2 (ft)

Subscripts 1,2 - parameter value at upstream and downstream section

When the horizontal datum is taken as the channel invert, the energy head relative to the channel bottom is the Specific Energy, expressed as Equation 6-13:

EQUATION 6-13, SPECIFIC ENERGY[1]

$$E + y + \alpha\left(\frac{v^2}{2g}\right)$$

E - Specific Energy (ft)

6.5.1.6 Hydraulic Jump

A hydraulic jump is a rapidly varied flow phenomenon where open channel flow changes abruptly from supercritical flow at a relatively shallow depth (less than critical depth) to subcritical flow at a greater depth (greater than critical depth). The identification of possible locations of hydraulic jumps is important because of the potential for unexpected surcharge or channel scour. Channel designs should always be analyzed for the occurrence of hydraulic jumps.

6.5.2 Manning's Equation

Normal depth is defined as the depth of uniform flow under a constant discharge. In uniform flow, friction and gravity forces in the direction of flow are equal but acting in opposite directions. At normal depth, the channel slope, hydraulic grade line slope, and energy grade line slope are all numerically equal and parallel to each other. Normal depth is a function of discharge, channel size, shape, and slope, and frictional resistance to flow. Manning's equation is an empirical formula used to evaluate the effects of friction and resistance in open channels. Most uniform flow computations are performed with Equation 6-14, Manning's equation:

EQUATION 6-14, MANNING'S EQUATION[1]

$$V = \left(\frac{1.49}{n}\right)R^{2/3}S^{1/2}$$

n - Manning's roughness coefficient

R - hydraulic radius = A/P (ft)

P - wetted perimeter, the perimeter of the channel that is in direct contact with water (ft)

S - slope of the energy grade line = channel slope for uniform flow (ft/ft)

Combining the continuity and Manning's equations, the uniform flow rate in a given cross section is given by:

EQUATION 6-15, MANNING'S EQUATION FOR DISCHARGE

$$Q = \left(\frac{1.49}{n}\right) AR^{2/3} S^{1/2}$$

The friction slope is a theoretical value which describes the slope of the energy grade line. For gradually varied flow, Manning's equation is used to calculate the friction slope at a given channel section as shown in Equation 6-16:

EQUATION 6-16, MANNING'S EQUATION FOR FRICTION SLOPE[1]

$$S_f = \left(\frac{n^2 V^2}{2.22 R^{4/3}}\right)$$

or

$$S_f = \left(\frac{Qn}{R^{2/3} A}\right)^2$$

S_f - *Friction slope (ft/ft)*

In channel analysis, channel properties are often grouped into a single term called the channel conveyance, as shown in Equation 6-17:

EQUATION 6-17, CONVEYANCE[3]

$$K = \frac{1.49}{n} AR^{2/3}$$

Manning's equation is then written as:

EQUATION 6-18, MANNING'S CONVEYANCE

$$Q = KS^{1/2}$$

6.5.2.1 Steep Slope vs. Mild Slope

If normal depth is less than critical depth for a given channel slope, it is considered hydraulically steep. Hydraulically mild slopes have normal depths greater than critical depth for a given section and discharge.

6.5.3 Manning's "n" Values

Manning's "n" values for design purposes should be based on field inspection for existing channels. Channel and overbank sections may need to be subdivided to represent differences in roughness across the section. Design channels should conform with Table 6-1 in general, although other values may be used when justified.

TABLE 6-1, MANNING'S ROUGHNESS COEFFICIENTS

6.5.4 Channel Description	6.5.5 Typical Manning's "n" Value
Man Made Maintained Grass Lined	.040
Man Made Unmaintained Grass Lined	.055
Concrete Lined	.015
Rock Riprap Lined	.045
Overbanks and Existing Unimproved Channels	See References 1 and 2

6.6 Erosion Protection

Erosion and local scour can result in channel degradation, in undermining and structural failure, or in the loss of channel bed materials and damage to channel linings. Excessive suspended sediment in streams may result in environmental impacts, as well as creating aesthetic and maintenance problems. Erosion protection is required where permissible shear stresses are exceeded for the 25-year storm event.

6.6.1 Shear Stress (Unit Tractive Force)

Channel shear stress, or unit tractive force, is defined as a shear force acting on the submerged portion of the channel bed and side slopes. This force acts in the direction of flow. The value of this shear stress can be calculated from Equation 6-19:

EQUATION 6-19, SHEAR STRESS[1]

$$\tau = \omega y s$$

τ - *average boundary shear stress (lb/sq ft)*

ω - *unit weight of water (62.4 lb/cu ft)*

s - *friction slope = bed slope for uniform flow conditions (ft/ft)*

Refer to the Georgetown Construction Standards and Specifications for permissible shear stresses for various channel lining materials.

6.7 Water Surface Profiles

When water surface profiles are required, the Corps of Engineers HEC-2[4] or HEC-RAS[5] software programs must be utilized for the calculations. The following sections present guidelines and criteria for use in water surface profiling.

6.7.1 Cross Sections

Channel cross sections must extend beyond the 100-year water surface elevation a reasonable distance. The purpose of including elevations beyond the 100-year water surface elevation is to avoid a design which creates ponding adjacent to the channel. A reasonable distance depends on the adjacent terrain, but in no case shall it be less than 25 feet.

Other guidelines for the placement of cross sections include the following:

A. Placed perpendicular to the direction of flow.

B. Locations should be representative of average conditions within stream reach.

C. Located at appreciable changes in cross sectional area, roughness, or gradient.

D. Needed at all control sections.

E. Adjacent cross sections should not overlap.

F. Needed immediately above and below the confluence on a main stream and immediately above the confluence on a tributary.

G. More cross sections are generally required:

  1. In urban as opposed to rural areas.
  2. In areas of steeper slopes.
  3. On smaller streams.

H. Spacing of cross sections has a direct impact on computed results. Additional cross sections may be required between two sections:

  1. If the computed energy slope decreases by more than 50%.
  2. If the computed energy slope increases by more than 100%.
  3. If the flow distribution appears unreasonable.

6.7.2 Starting Water Surface Elevations

For the design of open channels, starting water surface elevations at the channel mouth will generally be based on the normal depth (slope-area method in HEC-RAS or HEC-2) in the design channel.

For actual flood profiles or flood plain delineation, the water surface elevation from the outfall channel should be projected horizontally upstream until it intersects with the flood profile on the design channel. An assumption that the peaks occur at the same time will generally produce a conservative flood profile. Otherwise, an analysis of coincident flow may be conducted to determine the flow in the outfall channel at the time the peak flow occurs on the design channel.

6.7.3 Head Losses

Manning's equation (Section 6.5.2) is used to estimate the energy or head losses due to channel friction and resistance. Other sources of losses in open channels include confluences, transistions, bends, bridges, culverts, and drop structures. When computing water surface profiles, the engineer must include the significant sources of head loss. The following sections present the hydraulic criteria for analyzing nonfrictional head losses.

Roughness factors that are representative of unmaintained conditions should be used for the analysis of water surface profiles. Roughness factors representative of maintained conditions should be used to determine channel velocities.

6.7.4 Transitions

Transitions in channels should be designed to create a minimum of flow disturbance thereby minimizing energy loss. Transitions generally occur at bridges or culverts, and where cross sections change size or geometry.

All angles of transition should be less than 12 degrees (20 feet in 100 feet). If supercritical flow conditions are encountered, superelevation and hydraulic jumps must be considered.

Expansion and contraction losses must be accounted for in backwater computations. Transition losses are usually computed using the energy equation (Eq. 6-12) and are expressed in terms of the change in velocity head from downstream to upstream of the transition. Equation 6-20 is the equation used to compute this head loss between cross sections.

EQUATION 6-20, TRANSITION HEAD LOSS[3]

$$h_l = c\left[\left(\frac{v_2^2 - v_1^2}{2g}\right)\right]$$

H_l - head loss (ft)

c - expansion or contraction coefficient

v_1 - average channel velocity of upstream section (ft/s)

v_2 - average channel velocity of downstream section (ft/s)

g - gravitational constant (32.2 ft/s^2)

Typical transistion loss coefficients are given in Table 6-2:

TABLE 6-2, TRANSITION LOSS COEFFICIENTS[3]

	Coefficient	
Transition Type	**Contraction**	**Expansion**
Gradual or warped	0.10	0.30
Bridge sections, wedge or straight lined	0.30	0.50
Abrupt or Squared End	0.60	0.80

When computing the backwater profile through a transition, engineering judgment must be used in selecting the reach lengths. As a guideline, the velocity should not change more than 50 percent between two cross sections. Smooth transitions require fewer computation steps than the abrupt transitions.

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[1] Chow, V.T., Open Channel Hydraulics, 1959.

[2] French, R.H., Open Channel Hydraulics, 1985.

[3] Hydrologic Engineering Center, U.S. Army Corps of Engineers, HEC-RAS Hydraulic Reference Manual, 1997.

[4] Hydrologic Engineering Center, U.S. Army Corps of Engineers, HEC-2 User's Manual, 1990.

[5] Hydrologic Engineering Center, U.S. Army Corps of Engineers, HEC-RAS User's Manual, 1997.

END OF SECTION

7. STORM SEWERS

7.1 Introduction

Before a watershed is developed, runoff follows the natural topography. As sheet flow accumulates into shallow concentrated streams, the streams run together to form larger streams finally flowing to a common outfall. Grading for roads and lots disrupts the natural flow paths. Storm sewers accommodate this disruption by removing runoff from road surfaces so that traffic is not impeded.

Storm sewers remove roadway runoff through a network of inlets, conduits, and junctions which flow to a common outlet. Well designed storm sewers should generally follow natural pre-construction flow paths without crossing watershed boundaries. Inevitably, the layout of roads will modify the shape of the watershed, but this should be minimized.

Curb inlets are the most common facility used for capturing roadway runoff, but a number of other inlet types are often used including drop inlets, grate inlets, and slotted drains.

Water which ponds in sags or gutter flow which extends out into traffic presents a hydroplaning hazard to traffic. In addition, puddles which sit on the pavement cause cracking and failure, reducing the life of the pavement.

The following chapter describes the City of Georgetown requirements and processes for designing a storm sewer system.

7.2 Required Technical Information to be Submitted for Review

All plans and specifications for storm sewer systems shall contain the following documentation:

A. full gutter street capacity for each unique street segment (width/cross slope/longitudinal slope);

B. drainage area boundaries to each inlet;

C. 25- and 100-year peak flow quantities to each inlet;

D. hydraulic calculations at each inlet, including 25- and 100-year:

1. gutter depth,
2. ponded width,
3. Q intercepted,
4. Q passed; and
5. sewer plan and profile showing 25-year hydraulic grade line (and 100-year hydraulic grade line if the conduit is flowing full in the 25 year event), 25- and 100-year peak flow and velocity.

7.3 Design Requirements

7.3.1 Streets

Curb and gutter street sections for the City of Georgetown have parabolic crowns as described in the City of Georgetown Construction Specifications and Standards. Parabolic crowns differ from straight crown sections in that gutter flow is pushed toward the curb reducing the extent of ponding at any given depth. As a result, the gutter flow capacity at any given depth is also reduced.

For purposes of design,

A. Inlets shall be placed at all low points.

B. Inlets shall be placed and sized such that 25-year peak flows are contained within the 6-inch gutter and 100-year peak flows are contained within the street right-of-way.

C. All ponded widths for the 25-year peak flows shall be checked, and reduced where special situations require.

D. 100-year peak flow depths shall be less than 6 inches where flow over a driveway access or other curb cut results in flooding.

E. Concrete valley gutters shall be installed where gutter flow crosses a street.

F. Cross flow shall be limited to 4 cfs for the 25-year peak flow.

7.3.2 Storm Sewer Pipes

Storm sewer systems which follow the natural drainage topography usually result in the smallest required lengths and sizes of pipe. The pipes shall be sized according the following criteria:

A. Pipe sizes shall not decrease from upstream to downstream.

B. Junctions shall match pipe soffits whenever pipes of different sizes come together if possible.

C. Pipes shall intersect at a maximum angle of 45°, except at manholes.

D. The 25-year hydraulic grade line shall remain 6 inches below the gutter flowline of the inlet.

E. Pipes shall be designed for a minimum velocity of 3 ft/s except near outfalls where mild slopes are used to reduce outlet velocities.

F. Pipes, other than short lateral runs, shall be designed for a 100-year flow velocity of no greater than 20 ft/s,

G. Storm sewer outfalls with an outlet velocity for the 25-year storm greater than 6 ft/s shall have a structurally reinforced concrete apron with energy dissipaters and a 1'-6" minimum toe-wall. Storm sewer outfalls with an outlet velocity for the 25-year storm less than 6 ft/s shall provide rock riprap protection or alternate approved material. In these cases outlet pipe shall be protected according to typical concrete riprap at pipe detail in the current City of Georgetown Construction Specifications and Standards Manual.

H. Storm sewer lines shall be designed such that outfalls, or manholes provide access to within 500 ft of any location along the line for all pipes.

I. Storm sewer pipes shall have adequate cover to prevent any structural compromise of either the pipe or the surface material but will under no circumstances have less than 12 inches of cover.

7.4 Design Process

The following steps outline the iterative process for designing a storm drain system. Careful planning and calculations will minimize the number of iterations, but often a number of the steps must be repeated before producing a sound design.

A. Place inlets at each sump or low point.

B. Place inlets to prevent gutter flow from crossing streets when 25-year peak flows exceed 4 cfs.

C. Calculate the full gutter flow capacity for each unique street segment (each street width and grade combination).

D. Place inlets along street segments to prevent 25-year peak flows from exceeding street capacity and 100-year peak flows from exceeding right-of-way capacity. Based on some approximation of the 100-year runoff for your site, estimate how large an area can contribute to any given point on your street. For example, if typical areas from your site produce about 8 cfs per acre and your street capacity is 12 cfs, then place inlets along the street so that no more than 1.5 acres contribute to any inlet from one side.

E. Calculate the gutter hydraulics (flow depth/ponded width) at each inlet and size the inlet. At some locations the inlet will not capture all the contributing runoff. If ponding depths are too great or the inlet passes too much flow to an adjacent drainage area, inlet locations must be rearranged to reduce the contributing drainage area.

F. Connect the inlets with a pipe network that follows the natural topography from upstream to downstream.

G. Size each conduit in the network from upstream to downstream starting at the inlets and working downstream to each junction.

H. At each junction, size the downstream segment based on the total contributing drainage area and the overall time of concentration.

I. Calculate the tailwater and design the outlet pipe slope and outfall to minimize scour.

J. Calculate the 25-year and 100-year hydraulic grade line (HGL) starting at the outlet tailwater elevation and working upstream to each inlet (the 25-year HGL is not required if the 100-year is within the conduit).

K. Check the HGL to ensure that each inlet functions properly and street capacities are not exceeded.

7.5 Gutter Hydraulics

The hydraulic characteristics of gutter flow are calculated to determine how much flow streets can convey and how often inlets must be placed to prevent flooding. The most common method for describing open channel flow is Manning's Equation. The hydraulic radius (flow area / wetted perimeter) fails to properly describe the hydraulic characteristics of the sloping triangular cross section for gutter flow. The majority of the flow occurs with relatively low friction near the curb. The shallow area which extends out into the street provides a very small portion of the overall conveyance. When the hydraulic radius of the gutter section is taken as a whole, Manning's equation underestimates the total gutter flow. Izzard[1] developed a modified version of Manning's equation by integrating the flow rate for each incremental area across a triangular section. Equation 7-1 shows the modified version of Manning's equation for a straight crown section.

EQUATION 7-1, MANNING'S EQUATION FOR STRAIGHT CROWN GUTTER FLOW

$$Q = \frac{0.557 \frac{ft^{0.333}}{\sec} \cdot S_o^{0.5} \cdot y_o^{2.667}}{n \cdot S_x}$$

Q - *Flowrate (cfs)*

S_o - *Channel Slope (ft/ft)*

y_o - *Normal Depth (ft)*

S_x - *Gutter Section Transverse Slope (ft/ft)*

n - *Channel Roughness*

It is often convenient to solve for flow depth directly given a flowrate and street geometry. Equation 7-2 shows a re-arrangement of Equation 7-1 in terms of flow depth.

EQUATION 7-2, DIRECT SOLUTION FOR DEPTH BASED ON MANNING'S EQUATION FOR STRAIGHT CROWN GUTTER FLOW

$$y_o = \left[\frac{Q \cdot n \cdot S_x}{0.557 \frac{ft^{0.333}}{\sec} \cdot S_o^{0.5}} \right]^{0.375}$$

City of Georgetown streets have a parabolic crown. The flow characteristics of a parabolic crown gutter are similar to a straight crown gutter, but the cross sectional area and ponded width for flow at any given depth is different. Equation 7-3 displays Izzard's variation of Manning's equation in terms of flow area and wetted perimeter instead of transverse slope and flow depth.

EQUATION 7-3, MANNING'S EQUATION FOR PARABOLIC CROWN GUTTER FLOW

$$Q = \frac{1.769 \frac{ft^{0.333}}{\sec} \cdot A^{1.667} \cdot S_o^{0.5}}{n \cdot P^{0.667}}$$

Q - *Flowrate (cfs)*

S_o - *Channel Slope (ft/ft)*

A - *Flow Area (ft^2)*

P - *Wetted Perimeter (ft)*

n - *Channel Roughness*

The following set of relationships describes flow area and top width for parabolic crowns based on flow depth.

EQUATION 7-4, POLYNOMIAL EQUATION FOR STATION-ELEVATION CHARACTERISTICS OF A PARABOLIC CROWNED STREET SECTION

$$y(x) = a \cdot x^2 + b \cdot x$$

y(x) - *Street Elevation for a Zero Datum at the Lip of Gutter (ft)*

x - *Street Section Station (ft)*

a - *Polynomial Equation Coefficient (ft^{-1})*

b - *Polynomial Equation Coefficient (-)*

EQUATION 7-5, POLYNOMIAL EQUATION COEFFICIENT - A

$$a = \frac{g_2 - g_1}{2 \cdot W}$$

a - *Polynomial Equation Coefficient (ft^{-1})*

g_1 - *Transverse Street Grade at Gutter (ft/ft)*

g_2 - *Transverse Street Grade at Centerline (ft/ft)*

W - *Street Width from Face of Curb to Centerline (ft)*

EQUATION 7-6, POLYNOMIAL EQUATION COEFFICIENT - B

$$b = g_1$$

b - *Polynomial Equation Coefficient (-)*

g_1 - *Transverse Street Grade at Gutter (ft/ft)*

EQUATION 7-7, PONDED WIDTH FOR PARABOLIC CROWNED STREETS

$$T = \frac{-b + \sqrt{b^2 + 4 \cdot a \cdot y_o}}{2 \cdot a}$$

T - Extent of Ponding from the Face of Curb (ft)

y_o - *Normal Depth (ft)*

a - Polynomial Equation Coefficient (ft^{-1})

b - Polynomial Equation Coefficient (-)

EQUATION 7-8, FLOW AREA FOR PARABOLIC CROWNED STREETS

$$A = y_o \cdot T - \tfrac{1}{2} b \cdot T^2 - \tfrac{1}{3} a \cdot T^3$$

A - Flow Area (ft^2)

T - Extent of Ponding from the Face of Curb (ft)

y_o - *Normal Depth (ft)*

a - Polynomial Equation Coefficient (ft^{-1})

b - Polynomial Equation Coefficient (-)

For a given ponded width, the wetted perimeter for flow in a parabolic gutter is very close to the same wetted perimeter for a straight crowned street. The following relationship shown in Equation 7-9 approximates wetted perimeter as two legs of a right triangle.

EQUATION 7-9, WETTED PERIMETER

$$P = y_o + \sqrt{y_o^2 + T^2}$$

P - Wetted Perimeter (ft)

T - Extent of Ponding from the Face of Curb (ft)

y_o - *Normal Depth (ft)*

Calculating gutter hydraulics for a straight crown street is straight forward because one can solve directly for flow depth. Parabolic gutter calculations require an iterative solution where depth (i.e. Area and Wetted Perimeter) is adjusted until Equation 7-3 produces the desired flowrate. Due to the complexities involved in calculating parabolic gutter hydraulics, it is acceptable to utilize Equation 7-2 to calculate gutter flow depth and ponding width.

7.6 Inlet Hydraulics

Inlets are placed where flow must be removed from road surfaces. Examples include places where a transition would cause flow to travel in an unwanted direction (i.e. across a travel lane), to reduce ponded widths, and at low points. Once inlet locations have been established, the inlet must be sized so that it removes an adequate volume of flow. The following sections describe the use and capacity of curb inlets and grate inlets.

Alternative inlet types may be considered where their use and the engineering methods used to evaluate them are justified to the City of Georgetown.

7.6.1 Curb Inlet on Grade

Curb inlets offer high hydraulic capacity with very little clogging potential. They work well both on grade as well as at the low point of a sag vertical curve (sump). Curb inlets on grade operate most efficiently when part of the flow bypasses the inlet. Therefore, if there are no ponding or inlet capacity problems downstream, it is good design practice to select an inlet slightly shorter than the length required for total capture. Equation 7-10 determines the length inlet required to capture all flow approaching the inlet.

EQUATION 7-10, CURB OPENING LENGTH REQUIRED FOR TOTAL FLOW CAPTURE ON GRADE[2]

$$L_r = \frac{0.6\frac{ft^{2.58}}{\sec}\cdot Q^{0.42} S_o^{0.3}}{(n\cdot S_e)^{0.6}}$$

L_r - *Required Curb Inlet Length (ft)*

Q - *Flowrate (cfs)*

S_o - *Longitudinal Street Slope (ft/ft)*

n - *Manning's Roughness Coefficient*

S_e - *Effective Cross (Transverse) Slope (ft/ft) {see equations that follow}*

The throat of a curb inlet functions as a side flow weir. The rate of flow which spills into the curb inlet depends on the depth of flow at the inlet, the transverse slope of the street, and the forward momentum of the flow. Typically curb inlets include a gutter depression at the throat to increase the hydraulic capacity. Equation 7-10 accounts for the gutter depression in the term S_e, effective cross slope. Equation 7-11and the following relationships measure the effective cross slope. These relationships were taken from the Federal Highway Administration's H.E.C. No. 12[2] and modified as needed to account for a parabolic crowned street section.

EQUATION 7-11, EFFECTIVE CROSS SLOPE[2]

$$S_e = S_x + S_w E_o$$

S_e - *Effective Cross Slope (ft/ft)*

S_x - *Gutter Section Transverse Slope (ft/ft) {i.e. the transverse slope at the gutter for a parabolic crowned street}*

S_w - *Depressed Section Cross Slope (ft/ft)*

E_o - *Ratio of Flow Approaching Gutter Depression (cfs/cfs)*

EQUATION 7-12, DEPRESSED SECTION CROSS SLOPE

$$S_w = \frac{a}{W_i}$$

S_w - *Depressed Section Cross Slope (ft/ft)*

a - *Depth of Depressed Section (ft)*

W_i - *Width of Depressed Section (ft)*

EQUATION 7-13, CURB INLET ON GRADE - FLOW CAPTURE[2]

$$Q_i = Q\left(1-\left(1-\frac{L}{L_r}\right)^{1.8}\right)$$

Q_i - *Intercepted Flow (cfs)*

Q - *Total Gutter Flow (cfs)*

L - *Actual Curb Inlet Length (ft)*

L_r - *Required Curb Inlet Length (ft)*

7.6.2 Curb Inlet in a Sump

Curb inlets at the low point in a road (sump) operate as a weir at low water depths and as an orifice at greater water depths. The hydraulic capacity is even greater than curb inlets on grade because there is no gutter flow momentum tending to carry the flow past the opening. But sag inlets are more critical in their operation in that the water has nowhere else to go.

In critical locations where failure could be hazardous, it is good engineering practice to place one or more relief inlets to either side of the sag inlet. It is best to place such relief inlets far enough up-grade so that the elevation of the opening is 4 to 6 inches higher than the sag inlet. This minimizes the clogging potential of the relief inlet so that it can remove excess runoff if the primary inlet fails.

The City of Georgetown requires runoff from the 100-year storm to remain within the right-of-way . As a result, curb inlets in a street section sag may operate in both the weir flow and orifice regimes. The orifice flow relationship may be useful for area (table-top) inlets or curb inlets in different locations.

EQUATION 7-14, WEIR FLOW CAPACITY OF A CURB INLET IN A SUMP[2]

$$Q = 2.3\frac{ft^{0.5}}{\sec}\cdot\left(L+1.8\cdot W_i\right)\cdot y_o^{1.5}$$

for `$y_o < 0.85$ ft

Q - *Flowrate (cfs)*

L - *Curb Inlet Length (ft)*

W_i - *Gutter Depression Width (ft)*

y_o - *Ponded Depth at Lip of Gutter(ft)*

Equation 7-15 shows Equation 7-14 rearranged to solve directly for ponded depth.

EQUATION 7-15, DIRECT SOLUTION FOR PONDED DEPTH FOR WEIR FLOW AT A CURB INLET IN A SAG[2]

$$y_o = \left(\frac{Q}{2.3 \frac{ft^{0.5}}{sec} \cdot \left(L + 1.8 \cdot W_i \right)} \right)^{0.667}$$

EQUATION 7-16, ORIFICE FLOW CAPACITY OF A CURB INLET IN A SUMP

$$Q = 0.67 \cdot (0.46 ft) \cdot L \cdot \sqrt{2 \cdot g \cdot (y_o + 0.23 ft)}$$

for `$y_o > 0.85$ ft

- Q - *Flowrate (cfs)*
- L - *Curb Inlet Length (ft)*
- g - *Gravitational Constant (32.16 ft^2/sec)*
- y_o - *Ponded Depth at Lip of Gutter(ft)*

EQUATION 7-17, DIRECT SOLUTION FOR PONDED DEPTH FOR ORIFICE FLOW AT A CURB INLET IN A SAG

$$y_o = \frac{1}{2 \cdot g} \cdot \left(\frac{Q}{0.67 \cdot (0.46 ft) \cdot L} \right)^2 - 0.23 ft$$

(note: negative results indicate a depth lower than the lip of gutter and should be recalculated as weir flow)

7.6.3 Grate Inlet on Grade

In most cases a curb inlet operates more efficiently on grade than a grate inlet, but there are some situations where curb inlets are not feasible. A common application for grate inlets on grade are on bridge decks. The City of Georgetown should be contacted before designing a grate inlet on grade. The following equations describe design of a grate inlet on grade for situations which require such a configuration.

EQUATION 7-18, GRATE INLET FLOW CAPTURE[2]

$$Q_i = Q\left(R_f \cdot E_o + R_s \cdot (1 - E_o) \right)$$

- Q_i - *Intercepted Flow (cfs)*
- Q - *Total Gutter Flow (cfs)*
- R_f - *Ratio of Intercepted Frontal Flow to Total Frontal Flow (cfs/cfs)*
- R_s - *Ratio of Intercepted Side Flow to Total Side Flow (cfs/cfs)*
- E_o - *Ratio of Flow Approaching Gutter Depression to Total Flow(cfs/cfs)*

EQUATION 7-19, RATIO OF FRONTAL FLOW APPROACHING GRATE FOR A PARABOLIC CROWNED STREET MODIFIED FROM EQ. 7 IN REF. 2

$$E_o = 1 - \left(1 - \frac{W_i}{T}\right)^{2.90}$$

E_o - *Ratio of Flow Approaching Gutter Depression (cfs/cfs)*

W_i - *Grate Inlet Width (ft)*

T - *Extent of Ponding from the Face of Curb (ft)*

The flow intercepted by a grate inlet on grade depends on how fast the flow moves over the inlet and the configuration of the inlet bars. Equation 7-20 describes the fraction of frontal flow intercepted.

EQUATION 7-20, RATIO OF FRONTAL FLOW INTERCEPTED BY INLET[2]

$$R_f = 1 - 0.09\,ft^{-1}(v_o - v_s)$$

R_f - *Ratio of Intercepted Frontal Flow to Total Frontal Flow (cfs/cfs)*

v_o - *Average Gutter Flow Velocity (ft/sec)*

v_s - *Initial Velocity at Which Flow Splashes over the Grate (ft/sec)*

Equation 7-21 describes the velocity at which some of the frontal flow passes over the grate of an inlet. Equation 7-21 is unique to a parallel bar grate with 1.2 inch spacing. For other grate types please refer to H.E.C. No. 12[2].

EQUATION 7-21, SPLASH VELOCITY FOR GRATE INLET[3]

$$v_s = 1.762\,\frac{ft}{sec} + 3.110 \cdot L\,\frac{1}{sec} + 0.4505 \cdot L^2\,\frac{1}{ft \cdot sec} + 0.0333 \cdot$$

v_s - *Initial Velocity at Which Flow Splashes over the Grate (ft/sec)*

L - *Length (parallel to curb) of Grate Inlet (ft)*

EQUATION 7-22, RATIO OF SIDE FLOW INTERCEPTED BY INLET[2]

$$R_s = \frac{1}{\left(1 + \dfrac{0.15\,ft^{0.5}sec^{1.8} \cdot v_o^{1.8}}{S_x \cdot L^{2.3}}\right)}$$

R_s - *Ratio of Intercepted Side Flow to Total Side Flow (cfs/cfs)*

v_o - *Average Gutter Flow Velocity (ft/sec)*

S_x - *Gutter Section Transverse Slope (ft/ft) {i.e. the transverse slope at the gutter for a parabolic crowned street}*

L - *Length (parallel to curb) of Grate Inlet (ft)*

7.6.4 Grate Inlet in a Sump

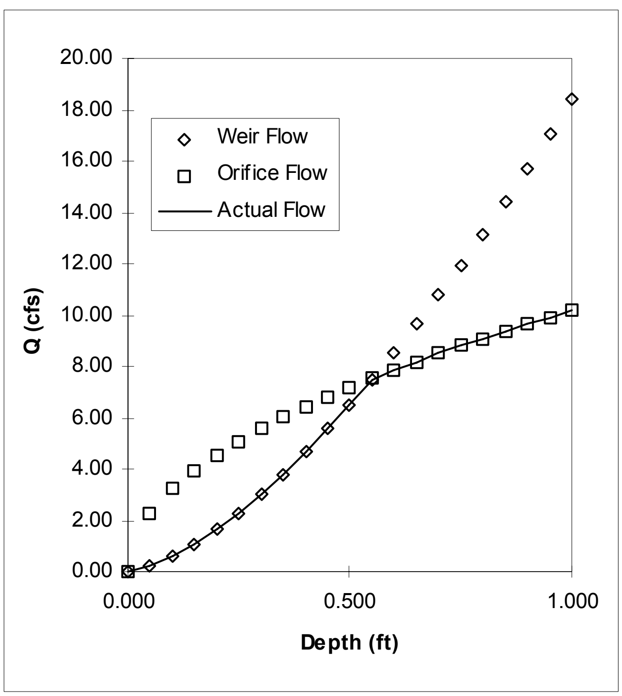
Grate inlets in a sump, like curb inlets, operate as a weir at low water depths and as an orifice at greater water depths. Grate inlets are often placed at the bottom of drainage swales or local low points. They may also be used where curb inlets will not fit.

Due to the propensity for clogging, design methods for grate inlets include a safety factor. In the weir flow regime, debris has little effect on flow spilling over the side of the inlet; but when ponding covers the inlet, the force of water tends to hold debris against the grate reducing flow capacity. The orifice equations below include a safety factor which reduce the flow area by 10%. While this safety factor adds some conservatism to design, the engineer should exercise judgment when placing grate inlets. Some locations may be prone to constant clogging, and a 10% safety factor is insufficient. In clogging prone locations a curb inlet or area (table top) inlet should be used instead.

Equation 7-23 and Equation 7-25 describe the inlet capacity for operation in the weir flow and orifice flow regimes respectively. To calculate capacity, take the lowest resultant flowrate produced by both equations.

Equation 7-24 and Equation 7-26 describe the ponding depth for operation in the weir flow and orifice flow regimes respectively. To calculate ponding depth, take the highest resultant depth produced by both equations.

FIGURE 7-1, WEIR FLOW / ORIFICE FLOW TRANSITION



EQUATION 7-23, WEIR FLOW CAPACITY OF A GRATE INLET IN A SUMP

$$Q = 2.3\tfrac{ft^{0.5}}{\sec}\cdot\left(2\cdot L + 2\cdot W_i\right)\cdot y_o^{1.5}$$

Q - *Flowrate (cfs)*

L - *Grate Length (ft)*

W_i - *Grate Width (ft)*

y_o - *Ponded Depth Above Grate(ft)*

Equation 7-24 shows Equation 7-23 rearranged to solve directly for ponded depth.

EQUATION 7-24, DIRECT SOLUTION FOR PONDED DEPTH FOR WEIR FLOW AT A CURB INLET IN A SAG

$$y_o = \left(\frac{Q}{2.3\tfrac{ft^{0.5}}{sec}\cdot\left(2\cdot L + 2\cdot W_i\right)}\right)^{0.667}$$

EQUATION 7-25, ORIFICE FLOW CAPACITY OF A GRATE INLET IN A SUMP

$$Q = 0.60\cdot A_i\cdot\sqrt{2\cdot g\cdot y_o}$$

Q - *Flowrate (cfs)*

A_i - *Curb Inlet Length (ft)*

g - *Gravitational Constant (32.16 ft^2/sec)*

y_o - *Ponded Depth at Lip of Gutter(ft)*

EQUATION 7-26, DIRECT SOLUTION FOR PONDED DEPTH FOR ORIFICE FLOW AT A GRATE INLET IN A SAG

$$y_o = \frac{1}{2\cdot g}\cdot\left(\frac{Q}{0.60\cdot A_i}\right)^2$$

7.7 Conduit Hydraulics

A pipe network joins all the inlets in a drainage area and carries the flow to a common outfall. The pipes must be sized and placed so that they can convey all the flow received by the inlets. Storm sewer networks are designed using Manning's equation to size each pipe for the contributing inlets and Manning's equation in conjunction with a backwater method to calculate the hydraulic grade line.

The pipe flowlines are typically set to follow the general topography of the overlying ground with some minimum cover. In places the pipe must be lowered even further in the ground to avoid conflicts or maintain gravity flow.

The following sections describe how to size a pipe and how to measure the hydraulic grade line of a system after the pipes have been sized.

7.7.1 Manning's Equation for Conduits

Storm sewer conduits are usually circular or box shaped. Other sizes including elliptical and arched shapes are also available, but the hydraulics for these shapes are not included in this discussion.

Storm sewer flow for all conduit shapes is calculated using Manning's equation shown in Equation 7-27.

EQUATION 7-27, MANNING'S EQUATION FOR CONDUITS

$$Q = \frac{1.486 \frac{ft^{0.333}}{\sec} \cdot A^{1.667} \cdot S_o^{0.5}}{n \cdot P^{0.667}}$$

Q - *Flowrate (cfs)*

S_o - *Channel Slope (ft/ft)*

A - *Flow Area (ft^2)*

P - *Wetted Perimeter (ft)*

n - *Channel Roughness*

Table 7-1 shows suggested roughness values for channel and conduit materials.

TABLE 7-1, MANNING'S ROUGHNESS FOR STORM SEWER DESIGN

Material	n
Pavement Gutter	0.015
Concrete Conduit	0.013
Corrugated Metal Conduit	0.024
See Specs for Special Conduit Mat'ls	-

Flow area and wetted perimeter are calculated based on the conduit shape. Equation 7-28 hrough 7-31 describes the hydraulic parameters for circular and box conduits.

EQUATION 7-28, CIRCULAR FLOW AREA

$$A = \frac{R^2}{8}\left[2\cos^{-1}\left(1-2\frac{y_o}{R}\right) - \sin\left(2\cos^{-1}\left(1-2\frac{y_o}{R}\right)\right)\right]$$

A - *Flow Area (ft^2)*

R - *Pipe Rise (ft)*

y_o - *Flow Depth (ft)*

EQUATION 7-29, CIRCULAR WETTED PERIMETER

$$P = R\cos^{-1}\left(1-2\frac{y_o}{R}\right)$$

P - *Wetted Perimeter (ft^2)*

R - *Pipe Rise (ft)*

y_o - *Flow Depth (ft)*

EQUATION 7-30, BOX FLOW AREA

$$A = y_o \cdot S_b$$

A - *Flow Area (ft^2)*

S_b - *Box Span (ft)*

y_o - *Flow Depth (ft)*

EQUATION 7-31, BOX WETTED PERIMETER

$$P = 2 \cdot y_o + S_b \qquad \text{or}$$

$$P = 2 \cdot R_b + 2 \cdot S_b$$

P - *Wetted Perimeter (ft)*

S_b - *Box Span (ft)*

R_b - *Box Rise (ft)*

y_o - *Flow Depth (ft)*

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[1]Izzard, C.F., “Hydraulics of Runoff from Developed Surfaces” Proc. Highway Research Board, Volume 26, p. 129-150, Highway Research Board, Washington, D.C., 1946.

[2] Johnson, F.L, F. Chang, “Drainage of Highway Pavements,” Hydraulic Engineering Circular No. 12, Federal Highway Administration, March 1984.

[3] “Hydraulic Volume - Design Manual,” Texas Department of Transportation, October 2001.

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8. CULVERTS AND BRIDGES

8.1 Introduction

This chapter addresses the hydraulic aspects of culvert and bridge design. Culverts are normally used to convey runoff through an embankment, while bridges are structures constructed over an obstacle (such as a waterway) to allow the continuation of a thoroughfare (such as a road).

8.2 General Criteria

A. All bridge and culvert structures shall be designed to carry and/or contain the upstream runoff from a twenty-five (25) year storm.

B. Run-off from the one hundred (100) year storm shall not top the road surface at bridge or culvert crossings for an arterial or collector street crossing and shall not exceed a depth of six (6) inches on a local street crossing.

C. All bridge and culvert structures shall be designed such that the structural integrity of the roadway shall be maintained during the one hundred (100) year storm event.

8.3 Culverts

A complete theoretical analysis of the hydraulics of a particular culvert installation is time consuming and difficult. Flow conditions vary from culvert to culvert and also over time for any given culvert. The barrel of the culvert may flow full or partly full depending on upstream and downstream conditions, barrel characteristics, and inlet geometry.

Based on research sponsored by the Federal Highway Administration, simplified procedures for culvert design have been developed. The basic approach is to analyze a culvert for various types of flow and then design for the control which produces the minimum performance.

8.3.1 Culvert Hydraulics

The hydraulic capacity of a culvert depends not only on the physical features of the culvert, but also on the type of flow through the culvert.

The concepts of inlet and outlet control are used to help simplify the analysis of culvert hydraulics. Inlet control occurs when the flow capacity of the culvert entrance is less than the flow capacity of the culvert barrel. The control section for culverts operating under inlet control is located just inside the entrance of the culvert. The water surface passes through critical depth at or near this location, and the flow regime immediately downstream is supercritical. For inlet control, the required upstream energy is computed by assuming that the culvert inlet acts as an orifice or a weir, depending on the submergence of the entrance. Therefore, the geometry of the culvert entrance governs the inlet control capacity. Outlet control flow occurs when the culvert flow capacity is limited by high tailwater downstream or by the flow capacity of the culvert barrel. The required head for outlet control is computed by performing an energy balance through the culvert from downstream to upstream.

8.3.1.1 Inlet Control

With inlet control, the culvert capacity depends on the headwater depth, barrel shape, cross-sectional area, and entrance conditions. The entrance can be either submerged or unsubmerged, while the outlet is always unsubmerged. With the entrance unsubmerged, weir flow exists. If the entrance is submerged, the discharge can be computed using the orifice equation.

The assumption of inlet control is that the flow passes through critical depth near the culvert inlet and transistions into supercritical flow. If the flow remains as low flow through the length of the barrel, then inlet control is assumed to be valid. If the flow goes through a hydraulic jump inside the barrel that fully develops the entire cross sectional area of the culvert, it is assumed that this condition will cause the pipe to pressurize over the entire length of the culvert barrel, and an outlet control answer should be used.

8.3.1.2 Outlet Control

With outlet control, culvert capacity depends on the tailwater depth, headwater depth, shape, slope, length, and roughness of the conduit. Tailwater depths should be determined by computing a backwater profile up to the culvert location. If the entrance and exit are both submerged, pressure flow conditions exist and the discharge is independent of the slope.

Computation of discharges or head losses through a culvert under outlet control can be obtained by applying the Bernoulli equation. For full culvert flow, the equation should be applied at points immediately upstream and downstream of the culvert. A partially full culvert requires that the equation be applied at points immediately upstream and a short distance downstream of the entrance. In both cases, the energy required for a given flow rate through a culvert is composed of three major parts: a velocity head, an entrance loss, and a friction loss.

Culvert analysis involves computing the inlet and outlet control headwater elevations for given discharge. These elevations are compared, and the larger of the two is used as the controlling headwater elevation. Tailwater effects are taken into consideration when calculating these elevations. If the controlling headwater elevation overtops the roadway embankment, an overtopping analysis is done in which the flow distribution is balanced between the culvert discharge and the surcharge over the roadway.

8.3.2 Computing Headwater Under Inlet Control

Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. Inlet control is most likely when the culvert configuration is on a hydraulically steep slope ($d_c > d_n$,). The control section of a culvert operating under inlet control is located just inside the entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Depending on conditions downstream of the culvert inlet, a hydraulic jump may occur in the culvert.

Under inlet control, hydraulic characteristics downstream of the culvert do not affect the culvert capacity. The upstream water surface elevation and the inlet geometry represent the major flow controls. The inlet geometry includes the barrel shape, the cross sectional area, and the inlet edge.

While inlet control is more likely under a steep slope situation, it can also occur under mild slope conditions. When the tailwater is lower than critical depth, inlet control is likely for a short culvert.

For the unsubmerged condition the culvert acts as a weir. One of the two forms of Equation 8-1 can be used to estimate the inlet control headwater depth for this condition.

EQUATION 8-1, UNSUBMERGED ENTRANCE INLET CONTROL EQUATIONS[1]

Form (1)

$$H_w = D\left[\frac{H_c}{D} + K\left(\frac{Q}{AD^{0.5}}\right)^M - 0.5S\right]$$

Form (2)

$$H_w = DK\left[\frac{Q}{AD^{0.5}}\right]^M$$

H_w - Inlet control headwater (ft)

D - Height or span of culvert (ft)

H_c - specific head at critical depth (ft)

Q - culvert discharge (cfs)

A - cross sectional area of culvert (sq ft)

K, M - constants from Table 8-1

The specific head, H_c, for form (1) is calculated with Equation 8-2:

EQUATION 8-2, SPECIFIC HEAD[1]

$$H_c = d_c + \frac{v_c^2}{2g}$$

H_c - specific head (ft)

d_c - critical depth (ft)

v_c - velocity at critical depth (ft/s)

g - acceleration due to gravity (32.2 ft/s^2)

TABLE 8-1, INLET CONTROL CONSTANTS

FHWA CHART NO.	SHAPE & MATERIAL	MONOGRAPH SCALE	INLET EDGE DESCRIPTION	UNSUBMERGED K	M	SUBMERGED C	Y
1	Circular	1	Square edge w/ headwall	0.0098	2	0.0398	0.67
	Concrete	2	Groove end w/ headwall	0.0078	2	0.0292	0.74
		3	Groove end projecting	0.0045	2	0.0317	0.69
2	Circular	1	Headwall	0.0078	2	0.0379	0.69
	CMP	2	Mitered to slope	0.0210	1.33	0.0463	0.75
		3	Projecting	0.0340	1.50	0.0553	0.54
3	Circular	A	Beveled ring, 45 degree bevels	0.0018	2.5	0.03	0.74
		B	Beveled ring, 33.7 degrees bevels*	0.0018	2.5	0.0243	0.83
8	Rectangular	1	30 to 75 degree wingwall flares	0.026	1	0.0385	0.81
	Box	2	90 to 15 degree wingwall flares	0.061	0.75	0.04	0.8
		3	0 degree wingwall flares	0.061	0.75	0.0423	0.82
9	Rectangular	1	45 degree wingwall flares d= .0430	0.51	0.667	0.0309	0.8
	Box	2	18 to 33.7 degree wingwall flares d= .0830	0.486	0.667	0.0249	0.83
10	Rectangular	1	90 degree headwall w/ 3/4" chamfers	0.515	0.667	0.0375	0.78
	Box	2	90 degree headwall w/ 45 degree bevels	0.495	0.667	0.0314	0.82
		3	90 degree headwall w/ 33.7 degree bevels	0.486	0.667	0.0252	0.865
11	Rectangular	1	3/4" Chamfers: 45 degree skewed headwall	0.522	0.667	0.0402	0.73
	Box	2	3/4" Chamfers: 30 degree skewed headwall	0.553	0.667	0.0375	0.79
		3	3/4" Chamfers: 15 degree skewed headwall	0.545	0.667	0.04505	0.68
		4	45 degree bevels: 10-45 degree skewed headwall	0.498	0.667	0.0327	0.75
12	Rectangular	1	45 degree non-offset wingwall flares	0.497	0.667	0.0339	0.803
	Box	2	18.4 degree non-offset wingwall flares	0.493	0.667	0.0361	0.806
	3/4" Chamfers	3	18.4 degree non-offset wingwall flares 30 degree skewed barrel	0.495	0.667	0.0386	0.71
13	Rectangular	1	45 degree wingwall flares- offset	0.497	0.667	0.0302	0.835
	Box	2	33.7 degree wingwall flares- offset	0.495	0.667	0.0252	0.881
	Top Bevels	3	18.4 degree wingwall flares- offset	0.493	0.667	0.0227	0.887
16-19	CM Boxes	1	90 degree headwall	0.0083	2.0	0.0379	0.69
		2	Thick wall projecting	0.0145	1.75	0.0419	0.64
		3	Thin wall projecting	0.0340	1.5	0.0496	0.57
29	Horizontal	1	Square edge with headwall	0.0100	2.0	0.0398	0.67
	Ellispe	2	Groove end with headwall	0.0018	2.5	0.0292	0.74
	Concrete	3	Groove end projecting	0.0049	2.0	0.0317	0.69
30	Vertical	1	Square edge with headwall	0.0100	2.0	0.0398	0.67
	Ellispe	2	Groove end with headwall	0.0018	2.5	0.0292	0.74
	Concrete	3	Groove end projecting	0.0095	2.0	0.0317	0.69
34	Pipe Arch	1	90 degree headwall	0.0083	1.5	0.0496	0.57
	18" Corner	2	Mitered to slope	0.0300	2.0	0.0463	0.75
	Radius CM	3	Projecting	0.034	2.0	0.0496	0.53
35	Pipe Arch	1	Projecting	0.0296	1.5	0.0487	0.57
	18" Corner	2	No Bevels	0.0087	2.0	0.0463	0.75
	Radius CM	3	33.7 degree bevels	0.003	2.0	0.0496	0.53
36	Pipe Arch	1	Projecting	0.0296	1.5	0.0487	0.57
	31" Corner		No Bevels	0.0087	2.0	0.0361	0.66
	Radius CM		33.7 degree bevels	0.003	2.0	0.0264	0.75
40-42	Arch CM	1	90 degree headwall	0.0083	2.0	0.0379	0.69
		2	Mitered to slope	0.030	2.0	0.0463	0.75
		3	Thin wall projecting	0.034	1.5	0.0496	0.57
55	Circular	1	Smooth tapered inlet throat	0.5340	0.5500	0.0196	0.8900
		2	Rough tapered inlet throat	0.5190	0.6400	0.0289	0.9000
56	Elliptical	1	Tapered inlet- beveled edges	0.5360	0.6220	0.0368	0.8300
	Inlet Face	2	Tapered inlet- square edges	0.5035	0.7190	0.0478	0.8000
		3	Tapered inlet- thin edge projecting	0.5470	0.8000	0.0598	0.7500
57	Rectangular	1	Tapered inlet throat	0.475	0.6670	0.0179	0.97

The unsubmerged equation applies approximately until $Q/AD^{0.5} = 3.5$. A small transition zone exists between the valid ranges for the unsubmerged and submerged inlet control equation, which applies to $Q/AD^{0.5} = 4.0$ and above. The transition zone can be defined empirically by drawing a curve between and tangent to the curves defined by the unsubmerged and submerged equations.

For inlet control and submerged entrance conditions, the culvert begins to operate as an orifice, and the headwater depth can be calculated with Equation 8-3:

EQUATION 8-3, SUBMERGED ENTRANCE INLET CONTROL EQUATION

$$HW_{ic} = Dc\left[\frac{Q}{AD^{0.5}}\right]^2 + Y - 0.5S$$

HW_{ic} - inlet control headwater depth (ft)

Q - culvert discharge (cfs)

D - rise or diameter of culvert (ft)

c, Y - Equation constants

A - cross sectional area of culvert barrel (sq ft)

S - culvert barrel slope (ft/ft)

Values for "c" and "Y" depend on the culvert shape and entrance conditions and are also found in .

8.3.3 Computing Headwater Under Outlet Control

Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening can accept. Outlet control is likely only when the hydraulic grade line inside the culvert at the entrance exceeds critical depth. Therefore, outlet control is most likely when the flow in the culvert is on a mild slope ($d_n > d_c$). It is also possible to experience outlet control with a culvert on a steep slope if high tailwater conditions exist such that subcritical or full flow exists in the culvert.

The total energy required to pass the flow through the culvert barrel must overcome entrance losses, friction losses, and exit losses. Other losses such as bends, junctions and grates must also be included when required.

The velocity of flow in the barrel is calculated as follows:

EQUATION 8-4, FULL FLOW VELOCITY IN CULVERTS[1]

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

V - average velocity of flow in the culvert barrel (ft/s)

Q - discharge (cfs)

A - full cross sectional area of culvert barrel (sq ft)

The velocity head is:

EQUATION 8-5, VELOCITY HEAD[1]

$$H_v = \frac{V^2}{2g}$$

H_v - velocity head (ft)

V - velocity (ft/s)

g - gravitational constant (32.2 ft/s^2)

The outlet control headwater depth is calculated by balancing energy between the exit and entrance of the culvert, as presented in Equation 8-6:

EQUATION 8-6, OUTLET CONTROL HEADWATER[1]

$$H_w + \frac{v_u^2}{2g} = H_o + \frac{v_d^2}{2g} + H_L$$

H_w - outlet control headwater depth (ft)

v_u - approach velocity (ft/s)

v_d - downstream velocity (ft/s)

g - acceleration due to gravity (32.2 ft/s^2)

H_o - outlet depth (ft)

H_L - sum of all losses including entrance, friction, exit, and other losses (ft)

In most instances, the approach velocity is low, and the approach velocity head can be ignored. Likewise, the velocity downstream of the culvert is normally neglected, and the equation becomes:

EQUATION 8-7, OUTLET CONTROL HEADWATER NEGLECTING VELOCITY HEADS

$$H_w = H_o + H_L$$

The headloss through the culvert, H_L, is expressed in Equation 8-8.

EQUATION 8-8, CULVERT HEAD LOSS[1]

$$H_L = H_e + H_f + H_o$$

H_e - entrance loss (ft)

H_f - friction loss (ft)

H_o - exit loss (ft)

The entrance loss, H_e, is a function of the velocity head in the barrel, and can be expressed as a coefficient times the velocity head.

EQUATION 8-9, ENTRANCE LOSS[1]

$$H_e = k_e\left(\frac{v^2}{2g}\right)$$

TABLE 8-2, ENTRANCE LOSS COEFFICIENTS

INLET TYPE TABLE | ENTR LOSS COEF

	Chart/Scale*	Description	Ci
1	2-3	Circular Corrugated Metal Pipe Projecting	0.9
2	2-2	Circular Corrugated Metal Pipe with Sloping (Mitered) Inlet	0.7
3	2-1	Circular Corrugated Metal Pipe with Headwall	0.7
4	1-3	Circular Concrete Pipe with Grooved End Projecting	0.2
5	1-2	Circular Concrete Pipe with Grooved End and Headwall	0.2
6	3-a	Circular Concrete Pipe with Beveled Ring (1"x1.5" on 24" Pipe, 2"x3" on 48" Pipe, etc.)	0.2
7	3-b	Circular Concrete Pipe with Beveled Ring (2"x3" on 24" Pipe, 4"x6" on 48" Pipe, etc.)	0.2
8	1-1	Circular Concrete Pipe with Square Edge and Headwall	0.5
9	10-1/8-2	Concrete Box with Parallel Wingwalls with 3/4" Chamfers	0.2
10	10-3	Concrete Box with Parallel Wingwalls with 1"/ft Bevels	0.2
11	8-1	Concrete Box with 30° to 75° Wingwalls	0.4
12	8-3	Concrete Box with Parallel Wingwalls	0.7
13	9-1/9-2	Concrete Box with 45° Wingwalls and a Top Bevel (1" Bevel on 2ft Box, 3" Bevel on 6ft Box, etc.)	0.2

*Chart/Scale Numbers as shown in nomographs in HDS 5, FHWA

Friction loss represents the energy required to overcome the roughness of the barrel, and is also a function of the velocity head. Based on Manning's equation, the friction loss is:

EQUATION 8-10, FRICTION LOSS[1]

$$H_f = \frac{29n^2L}{R^{1.33}}\left(\frac{v^2}{2g}\right)$$

n - *Manning's roughness coefficient*

L - *length of the culvert barrel (ft)*

R - *hydraulic radius of the barrel flowing full (ft)*

A - *cross sectional area of barrel (sq ft)*

v - *velocity in the barrel (ft/s)*

The exit loss is normally taken as equal to the full flow velocity head in the barrel, as shown in :

EQUATION 8-11, CULVERT EXIT LOSS[1]

$$H_o = \frac{v^2}{2g}$$

v - *full flow velocity in barrel (ft/s)*

Appropriate loss terms for bends, junctions, and grates should be also be included where applicable. HDS-5[1] provides more information regarding these losses.

The outlet depth, H_o, is determined as follows:

A. For hydraulically steep slopes, if the discharge is greater than the capacity of the conduit calculated with Manning's Equation, the culvert is assumed to flow full and the outlet depth is taken as the higher of the barrel depth, D, and the tailwater depth. Otherwise, the outlet depth is set to the tailwater depth.

B. For hydraulically mild slopes, if critical depth exceeds the barrel depth ($d_c > D$) the outlet depth is taken as the higher of the barrel depth and the tailwater depth. Otherwise, the outlet depth is taken as the higher of critical depth and the tailwater depth.

8.4 Culvert Design

Normally, the process of culvert design is one of trial and error. The following steps must be done until all design criteria are satisfied:

A. Analyze a trial configuration.

B. Compare the results with the design criteria.

C. Adjust the configuration.

D. Perform another analysis.

8.4.1 Design Data

An effective culvert design requires the knowledge of some items and the assumption of others. The items which should be known through data collection or calculation include:

A. Design discharge

B. Design tailwater or tailwater curve

C. Culvert slope

D. Allowable Headwater depth

E. Culvert length

F. Entrance conditions

G. Conduit shape and material

H. Maximum depth or diameter of culvert barrel

8.4.2 Design Procedure for Culverts

The following is a simplified culvert design procedure for a standard culvert configuration. Any change to the configuration considered in the iterative process will influence the flow type. Each new iteration requires a determination of whether there is inlet or outlet control. Nomographs, spreadsheets, or computer programs may be utilized for the culvert analysis computations. Currently, the computer programs accepted by the City of Georgetown are HY-8 (FHWA) and Culvert Master (Haestad Methods). Other programs may be added to this list if approved by the City Drainage Engineer.

A. Pick an initial trial size. One way to pick an initial size is to assume inlet control as follows:

B. Determine the maximum practical rise of culvert (D_{max}) and the maximum allowable headwater depth (HW_{max}). First calculate a trial head using Equation 8-12:

EQUATION 8-12, TRIAL HEAD FOR CULVERT SIZING

$$h = HW_{\max} - \frac{D_{\max}}{2}$$

h – allowable effecitve head (ft)

HW_{max} - allowable headwater depth (ft)

D_{max} - maxumum conduit rise (ft)

C. Use Equation 8-16 (a form of the orifice equation) to determine the required area for the design discharge. This equation assumes an orifice coefficient of 0.6, which may require modification during later iterations.

EQUATION 8-13, TRIAL BOX CULVERT AREA

$$A = 0.21\frac{Q}{h^{0.5}}$$

A - approximate sectional area required (sq ft)

Q - design discharge (cfs)

D. Decide on a culvert shape. For a box culvert:

1. Determine the required width, W, calculated as A/D_{max}.
2. Round W up to the nearest value of standard box culvert widths.
3. Divide W by the largest span S for which W is a multiple to determine the number of barrels, N (i.e., for a 10 foot width, the largest divisor is 5, therefore N = 2).

E. For a circular culvert:

1. Determine the ratio of area required to maximum barrel area as:

EQUATION 8-14, TRIAL CIRCULAR CULVERT SIZE

$$R = \frac{4A}{\pi D_{max}^2}$$

R - Ratio of Area to maximum barrel area

2. Round R to the nearest whole number to get the required number of barrels, N.

For other shapes, provide an approximate size such that the cross-sectional area is approximately equal to A.

F. Determine the design discharge per barrel as Q/N.

G. Proceed with the hydraulic analysis of the trial configuration.

H. Evaluate the trial design.

8.5 Bridges

Bridges are important and expensive hydraulic structures that are vulnerable to damage and failure from flood related causes. In order to minimize the risk of failure, the hydraulic requirements of a stream crossing must be recognized and considered in bridge design and construction.

8.5.1 Bridge Hydraulics

The hydraulic considerations when designing a bridge include the hydraulics of the flow at the bridge itself, the upstream effect of the bridge with regard to increased flooding potential, and the possibility of failure due to scour at the bridge piers and abutments.

Normally, the entire design storm flow passes through the bridge opening without touching any part of the bridge deck. Under this condition the bridge normally has a relatively small effect on the water surface profile. The water surface elevation change created by the presence of the bridge translates into an increased stage upstream of the bridge. This increase in water surface elevation is due to several factors, including the contraction and expansion of the flow as it enters and leaves the bridge section, and the impact and consequent change in momentum caused by the piers and abutments.

There are numerous theoretical and empirical methods for estimating the impact of bridges on water surface profiles. The City of Georgetown requires that the HEC-RAS or HEC-2 computer programs be employed to perform such estimates. The user's manuals for these programs should be consulted for specific requirements and computational procedures employed. However, the following sections provide a preliminary discussion of bridge hydraulics.

8.5.2 Bridge Flow Class

Flow through a bridge can be grouped into three classifications: low flow, pressure flow, and weir flow. The type of flow present is dependent on both the water surface at the downstream side of the bridge and the discharge. Pressure flow occurs when the water surface is above the low chord, but does not overtop the bridge. When flows overtop the bridge deck or top of road, weir flow occurs.

8.5.2.1 Low Flow

Low flow occurs when the water level is below the low chord of the bridge. Figure ** also indicates the four types of low flow that are encountered at bridges. These are labeled Type I, IIA, IIB, and III.

8.5.2.2 Type I Flow

Type 1 flow exists when the water surface through the bridge remains subcritical. This is the type of flow normally encountered in practice. The backwater expression for type I flow is obtained by applying the energy or momentum equation.

8.5.2.3 Type II Flow

For type IIA flow, the unconstricted water surface profile again remains above critical depth throughout the bridge area, but the constricted water surface passes through critical depth in the area of the bridge. Once the constricted water surface passes through critical depth, it becomes independent of conditions downstream (even though the water surface returns to the unconstricted level at section 4).

The constricted water surface for type IIB flow starts out above both the unconstricted water surface and critical depth upstream of the bridge, passes through critical depth in the bridge and downstream of the bridge, and returns to unconstricted depth. The return to unconstricted depth results in a hydraulic jump since the unconstricted depth is greater than critical depth.

8.5.2.4 Type III Flow

Type III flow exists when both the unconstricted and constricted water surface profiles are below critical depth throughout the area of the bridge. Normally, this type of flow is found only in mountainous regions. Theoretically, no backwater would occur for this type of flow, although some undulation of the water surface will occur in the vicinity of the bridge. The energy or momentum equation is used to calculate the water surface through the bridge for this type of flow.

8.5.3 Pressure Flow Computations

Pressure flow occurs when the flow comes into contact with the low chord of the bridge. This creates an orifice flow condition through the bridge. Two types of orifice flow conditions are possible, depending on whether the downstream low chord is in contact with the water.

If only the upstream end of the bridge is in contact with the flow, a sluice-gate type of equation is used:

EQUATION 8-15, SLUICE GATE PRESSURE FLOW FOR BRIDGES[3]

$$Q = C_d A_{bu}\left[2g\left(Y_3 - \frac{Z}{2} + \frac{\alpha_3 V_3^2}{2g}\right)\right]^{1/2}$$

Q - discharge at bridge (cfs)

C_d - coefficient of discharge for pressure flow

A_{bu} - Open area of bridge at section inside bridge upstream (sq ft)

Y_3 - Hydraulic depth at Section 3 (ft)

Z - vertical distance from bridge low chord to mean channel bed elevation (ft)

The discharge coefficient C_d ranges from 0.27 to 0.5, with 0.5 commonly used in practice.

The second type of pressure flow occurs when both the upstream and downstream sides of the bridge are submerged, but the bridge is not overtopped. A standard orifice equation is used for this condition, as shown Equation 8-16:

EQUATION 8-16, STANDARD ORIFICE PRESSURE FLOW FOR BRIDGES[3]

$$Q = CA\sqrt{2gH}$$

C - coefficient of discharge for fully submerged pressure flow

H - vertical distance from the energy grade line elevation upstream and the water surface elevation downstream (ft)

A - area of bridge opening (sq ft)

Values of the discharge coefficient C range from 0.7 to 0.9, with 0.8 commonly used in practice.

8.5.4 Weir Flow Computations

When flow overtops the bridge, a standard weir equation is used to compute the flow:

EQUATION 8-17, BRIDGE WEIR FLOW[3]

$$Q = CLH^{3/2}$$

Q - weir flow over the bridge (cfs)

C - coefficient of discharge for weir flow

L - Effective length of weir flow

H - vertical distance from upstream energy grade line and crest of roadway (ft)

If the weir discharge is independent of tailwater, the value of C ranges from 2.5 to 3.1 for broad crested weirs. A value of 2.6 is commonly used in practice.

8.5.5 Bridge Design

A new bridge should span the existing or design channel so as to minimize energy losses due to contraction and expansion of the flow. The channel section should be carried under the bridge without modification. For channels with wide floodplains, approach embankments can be used provided the embankments do not increase flood levels by encroaching in effective flow areas. Where approach embankments do restrict the effective flow areas, channel improvements must be constructed to compensate for the reduced flow area in the overbank areas.

Bends and abutments must be aligned parallel to the longitudinal axis of the channel in order to minimize obstruction of flow. Standard skew angles should be used (15, 30, 45 degrees, etc.). Headers and interior bents should be oriented to conform with stream lines and flood stage.

Bents should be located outside of the channel bottom if possible. This reduces debris collection and provides for more efficient flow during storm events.

An important aspect of bridge analysis often overlooked is the non-effective flow area upstream and downstream of the bridge. If the bridge section constricts the flow, there will be non-effective flow areas established on both sides of the bridge. Eddies and secondary currents will occur in these areas, but no effective flow occurs. When computing the backwater profile, channel cross sections should be included near the bridge which block out noneffective flow areas.

Average channel velocities usually increase in the vicinity of a bridge. In addition, vegetation rarely grows under bridges. For these reasons, erosion protection is often necessary around bridges.

8.5.5.1 Velocity

As flow moves toward the bridge opening, the velocity increases. The increase in velocity can result in scour along the embankment and through the bridge. Turbulence, eddying and vortices around piers and headers can also cause scour.

The through bridge velocity is a basic sizing criteria for span-type bridges. The waterway opening should be large enough that the 100-year design flow average through-bridge velocity is less than six feet per second. Calculation of the average through bridge velocity is performed with the continuity equation:

EQUATION 8-18, AVERAGE THROUGH-BRIDGE VELOCITY

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

V - Average through-bridge velocity (ft/s)

Q - Design storm discharge (cfs)

A - Cross sectional area of bridge opening conveying runoff (sq ft)

Higher velocities may be acceptable in certain cases, such as where the bridge headers are placed outside of the flood waters. The effects of scour on the stream banks must still be considered in this situation.

If natural stream velocities are greater than six ft/s, the bridge may be sized by spanning the natural stream so as not to cause a restriction.

To reduce bridge costs, in many instances approach embankments are extended into the floodplain. In doing so, the embankments will constrict the flow of the stream during flood stages. Obviously, the greater the free span opening, the less the amount of backwater during the design flood, and vice versa.

The contraction and expansion of flow in the area of the bridge causes a loss of energy, the majority of which occurs in the downstream expansion. This loss of energy is reflected in a rise in the water surface upstream of the bridge. The extent of this rise, termed backwater, is one of the most important criteria in bridge design.

For preliminary sizing of bridge length and elevation, the following steps are suggested:

A. Assume an average through-bridge velocity that is less than the maximum allowable velocity but not lower than the existing velocity at the proposed bridge location.

B. A preliminary cross sectional area can be determined from the Continuity Equation:

EQUATION 8-19, CONTINUITY FOR BRIDGE DESIGN

$$A = \frac{Q}{v}$$

A - Trial cross sectional area of bridge opening (sq ft)

Q - design flood discharge (cfs)

v - assumed velocity (ft/s)

C. Estimate the depth of flow at the cross section where the bridge is to be located with the cross sectional area determined in step 2.

D. Estimate the trial length of the bridge with Equation 8-20:

EQUATION 8-20, TRIAL LENGTH FOR BRIDGE DESIGN

$$L = \frac{A}{D}$$

L - length of bridge (ft)

A - preliminary cross sectional area (sq ft)

D - estimated depth of flow (ft)

E. Position headers in the cross section approximately the length of the bridge apart, and where the bridge open area is maximized.

F. Using water surface profile techniques, determine the cross sectional area below the design discharge within the structure limits.

G. Use the Continutiy Equation to find the average through-bridge velocity for the actual waterway area.

H. If the velocity calculated in step 7 is close to the assumed velocity, the bridge length may be reasonable. This length will usually need to be adjusted to fit standard span length requirements.

If the new velocity is much lower or greater than the allowable maximum velocity, the length should be adjusted as necessary.

I. Determine the low chord for the bridge at the upstream and downstream sides.

J. Estimate the backwater caused by the bridge. The bridge backwater effects cannot exceed one foot upstream of the structure.

K. Determine the potential scour envelope for the bridge.

8.5.6 Bridge Scour

Bridge scour is the result of the erosive force of the water which excavates and carries away material from the bed and banks of channels. For simplicity, scour can be considered to consist of three components:

- Long term aggradation and degradation
- Contraction scour
- Local scour

Long term aggradation and degradation, also called natural scour, can result from lateral channel migration, natural processes within the channel, or some modification to the stream or watershed.

Contraction scour occurs when the flow area of a channel is decreased, either by a natural contraction or a bridge structure. Increased velocities in the contracted section are responsible for contraction scour.

Local scour occurs around piers, abutments, spurs and embankments. It is caused by high local velocities and flow disturbances such as eddies and vortices within the channel.

The rates of scour depend on the following factors:

- Erosive power in the flow
- Resistance to erosion of the material
- Sediment balance into and out of a section

For erosion resistant materials, final equilibrium may not be reached in any one flood event but may develop over a long period. Methods currently available do not specifically accommodate cohesive bed materials or time-dependency. Therefore, scour calculations should be considered as an estimate of the maximum potential scour. Judgment based on experience must be exercised to decide whether a calculated depth is likely for the given site conditions.

[1] Federal Highway Administration, Hydraulic Design of Highway Culverts, HDS No. 5, 1985.

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