City of Killeen
Drainage Design Manual

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SECTION 1.0 DRAINAGE POLICY

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1.0 DRAINAGE POLICY

1.1 GENERAL

This Manual represents the application of accepted principles of stormwater drainage engineering and is a working supplement to basic information obtainable from standard drainage handbooks and other publications on drainage design. The policy statements of this Chapter provide the underlying principles by which all drainage facilities shall be designed. The application of the policy is facilitated by the technical criteria contained in the remainder of the Manual.

1.2 CITY OF KILLEEN DRAINAGE POLICY

1.2.1 Application

The City of Killeen's (City) drainage policy shall govern the planning and design of drainage infrastructure within the corporate limits of the City and within all areas subject to its extra territorial jurisdiction, as required. Definitions, formulae, criteria, procedures and data in this manual have been developed to support this policy. Methods not discussed in this manual may be used if they are acceptable industry standard methods. If any condition requiring some additional measure of protection is identified during design or construction, the design engineer shall make provisions within the design. All plans must be signed and sealed by a Professional Engineer licensed in the State of Texas. Any deviations from this manual due to special circumstances are acceptable if they meet the same levels of safety identified in this manual and are approved by the Director of Public Works or his/her designee.

1.2.2 General

A. Stormwater runoff peak flow rates for the 25- and 100-year frequency storms shall not cause increased adverse inundation of any building or roadway surface.

B. Street curbs, gutters, inlets, and storm sewers shall be designed to intercept, contain, and transport all runoff from the 25-year frequency storm, with a maximum overtopping the curb of zero (0) inches and total depth of flow does not exceed 6 inches anywhere between the curb faces during this same frequency storm.

C. In addition to B above, the public drainage system shall be designed to convey those flows from greater than the 25-year frequency storm up to and including the 100-year frequency storm within defined public rights-of-way less 4 feet on each side or drainage easements.

D. When stormwater detention is provided, stormwater runoff peak flow rates shall not be increased at any point of discharge for the 25-year storm frequency event. The 100-year storm event shall be passed from the detention facility through an emergency spillway such that the flow from that event shall not overtop the facility or cause damage to the downstream facilities.

1.2.3 Drainage Flow in Streets

No concentrated point discharges directly into streets will be allowed unless approved by the Public Works Director or his/her designee.

Streets shall have a minimum cross slope of 2% to properly remove water from the pavement surface. Lowering the height of the street crown may be allowed for the purposes of obtaining additional hydraulic capacity. In this case, the crown elevation shall be no lower than 2 inches below the top of curb.

1.2.4 Street Cross Flow

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. When street cross flow occurs from one curb line to the opposing curb line, the depth of flow shall not exceed 6 inches of depth at any point within the street. This policy requires the use of concrete valley gutters to convey runoff across the street. At points of concentration other than intersections, cross-flows shall be contained within underground storm conduit.

In the event that underground storm drainage is not practical, cross flow is allowed. The crown shall be removed and a concrete valley shall be required to convey the runoff across the street. Cross flow shall not exceed 6 inches of depth within the concrete valley or between curb faces.

1.2.5 Allowable Flow of Water through Intersections

As the stormwater flow approaches a residential or marginal access street intersection, inlets shall be required if the depth of flow exceeds 6 inches at any portion of the street intersection. Concrete valley gutters shall be used to convey stormwater flow through intersections. In the case of tee intersections designed in sump conditions, the Engineer shall demonstrate that the depth of stormwater will not exceed 6 inches at any point within the intersection measured from the flowline of the valley gutter. Inlets in such cases shall not be installed within the curb radius of the intersection.

1.2.6 Drainage System

A. Construction plans for proposed reinforced concrete box culverts, bridges and related structures may be adaptations of the current Texas Department of Transportation (TxDOT) Standards.

B. For bridges and culverts in residential and marginal access streets, runoff from the 100-year frequency flow shall not produce a headwater elevation at the roadway greater than either 6 inches above the roadway crown elevation or 3 inches above any top of upstream curb elevation, whichever is lower.

C. For bridges and culverts in streets other than a residential and marginal access street, runoff from the 100-year frequency storm shall not produce a headwater elevation at the roadway greater than 3 inches above the roadway crown elevation or 3 inches above any top of upstream curb elevation, whichever is lower.

D. All drainage facilities (including but not limited to headwalls, open channels, storm sewers, area inlets, and detention, retention, and water quality controls and their appurtenances) shall comply with the following requirements, unless otherwise noted in this section.

1. Storm sewer inlets and gutter transitions shall be designed to avoid future driveways and to avoid conflicts with standard water and wastewater service locations. No utilities shall be allowed to cross through a storm sewer inlet or culvert. No utilities shall be allowed to cross under a new storm sewer inlet. In the case of retrofitting an existing storm sewer system where the relocation of an existing utility is not practical, the Public Works Director or his/her designee may allow the existing utilities to cross under a storm sewer inlet.

2. Drainage channels and detention ponds that are to be maintained by the public (City) shall be contained within drainage easements. A minimum 10-foot wide drainage easement for access shall be provided for drainage channels and detention ponds. Ramps no steeper than 5 feet horizontal to 1 foot vertical shall be provided to allow access to drainage channels and detention ponds. The minimum bottom width for a trapezoidal channel with vegetative side slopes shall be 4 feet. V-ditches are only allowed with side slopes no steeper than 4 feet horizontal to 1 foot vertical.

3. Detention ponds shall be designed with adequate area around the perimeter for access and maintenance. The said area shall be a minimum of 7 feet wide for ponds with depths of 5 feet or less (back slopes included) and a minimum of 15 feet wide for ponds over 5 feet deep or with back slopes in excess of 5 feet high. The said area shall not slope more than 10%. Privately owned parking lot detention areas will not require a perimeter area for access and maintenance.

4. Velocity dissipation shall be accomplished by the use of rock riprap or concrete riprap with formed concrete dissipaters. Rock or stone riprap shall be allowed

with a minimum D50 of 12-inch diameter rock or stone or per TxDOT standards, whichever is larger.

5. Stormwater conveyance between lots (crossing blocks) shall be avoided as much as possible. If necessary, stormwater conveyance between lots shall be underground storm drains or flumes, located entirely on one lot or split between lots, laid along an alignment that retains the conveyance measure within the dedicated drainage easement. Storm drains along rear of residential lots (through back yards) shall be avoided as much as possible. Drainage easements for storm drains shall be a minimum of 15 feet in width or 1.5 times the depth of the storm drain, whichever is greater. The drainage easement for a flume shall be equal to 10 feet or the width of the flume, whichever is greater. Fences shall not cross or be constructed within drainage easements. Fences may cross easements with underground facilities provided the design engineer can illustrate how conveyance for the 100-year storm event is unobstructed, and if approved by the Director of Public Works or his/her designee. No part of a residential or commercial structure may be constructed in or overhang into a drainage easement.

6. Bedding of storm sewer shall be to 6 inches above the top of pipe or to current Public Works Standards (whichever is greater).

7. Storm drains shall be reinforced concrete pipe (RCP), ASTM C76, minimum Class III, and minimum 18-inch diameter. The design Engineer shall provide load analysis to the Director of Public Works or his/her designee as appropriate to demonstrate that class of pipe used is sufficient for the loading conditions. Higher strength pipes shall be used where loadings warrant such. Storm drains shall have a minimum of 2 feet of cover in unpaved areas and a minimum of 1.5 feet of cover from bottom of the subgrade in paved areas. If minimum cover requirements cannot be attained, the design Engineer shall use higher strengths pipes sufficient for the loading conditions.

8. The use of High Density Polyethylene (HDPE) shall be allowed up to 48 inches in diameter in unpaved areas outside of City streets. All cross street storm drainage conduit shall be RCP unless approved by the Public Works Director or his/her designee. HDPE sizes larger than 48 inches in diameter may be used if approved by the Public Works Director or his/her designee.

9. Junction boxes and manholes shall be reinforced concrete. Junction boxes in lieu of manholes shall be provided where any pipe opening exceeds 37 inches in

diameter and where the distance from the outside surfaces of any two pipes entering a manhole is less than 1 foot, measured along the inside of the manhole.

10. Prefabricated wyes, mitered angle fittings, and pipe size reducers shall be allowed in lieu of junction boxes and manholes for all changes in alignment 45 degrees or less. Changes in alignments greater than forty-five-degrees require a manhole or junction box.

11. Channels

    a. **Concrete Channels:** Concrete channels shall be of sufficient cross section and slope (minimum 0.5%) as to fully contain design flows and facilitate self cleaning. Outfalls shall enter major collector drainage ways and watercourses at grade or be designed and constructed with adequate concrete aprons, energy dissipaters, or similar features.

    b. **Vegetated Channels:** Vegetated channels shall have sufficient grade but with velocities that will not be so great as to create erosion. Velocities shall be submitted as the average channel velocity for the full channel cross section. Side slopes shall not be steeper than 3 feet horizontal to 1 feet vertical for channels 4 feet or less in depth and no steeper than 4 feet horizontal to 1 feet vertical in all other channels to allow for future growth and to promote slope stability. All slopes shall be hydro-mulched, sodded, or seeded with approved grass, grass mixtures, or ground as indicated in the City of Killeen's standard details or in TxDOT Standard Specification 164 for use in the Waco District season in which they are applied. All earthen channels must have vegetation established prior to acceptance by the City of Killeen. If vegetation cannot be adequately established prior to the desired acceptance date, the channel side slopes and bottoms shall be fully lined with erosion protection matting prior to acceptance. Such erosion control matting shall be accompanied by an engineering design. Matting shall be pre-seeded or all channel slopes shall be droll-seeded with approved grass mixtures prior to application of all matting.

    c. Watercourses shall not be modified without consent of applicable state and federal agencies and authorization from the Public Works Director or his/her designee.

12. Discharge from storm sewer outfalls shall not cause channel, bluff, or stream bank erosion. If the storm drain discharges to an open drainage facility (as determined by the City), the design Engineer shall demonstrate acceptable nonerosive conveyance to that drainage facility, appropriate energy dissipation at the outfall

and a stable headwall. No outfalls shall be allowed to discharge on the slope of the receiving channel without adequate erosion control measures.

13. If the development is located such that there is considerable drainage from potentially developable upstream areas, the developer may request participation by the City for the cost of over sizing of elements of the overall drainage system for ultimate upstream development. The City shall consider these requests on a case by case basis. Final determination of any cost sharing will be determined by the City Council and City Manager through a development agreement as outlined in Section 26-85 of City Code of ordinances, as amended.

1.2.7 Computations

A. Computations to support all drainage designs shall accompany all submittals to the City. The computations shall be in such form as to allow for timely and consistent review and also to be made a part of the permanent City record for future reference. Computation shall include the impact of the proposed development to the downstream properties adjacent to the drainage resulting from the 100-year event. All computations submitted shall be certified by a Professional Engineer licensed in the State of Texas. The Engineer shall provide the report to the City in both hard copy and a scanned electronic portable document file (pdf) with the proper seal, signature, and date.

B. **Determination of Runoff:** Numerous methods of rainfall-runoff computation are available on which the design of storm drainage and flood control systems may be based. The Rational Method shall be an acceptable means of computing runoff for drainage areas of 200 acres or less when designing streets, storm drainage systems, channels, and culverts. When the drainage area exceeds 200 acres in size, the Natural Resources Conservation Service (NRCS, formerly the Soil Conservation Service [SCS]) hydrologic methods (available in TR-55, or HEC-HMS) or comparable accepted methodology shall be used.

C. **Detention Pond Storage Determination:** A flow routing analysis using detailed hydrographs shall be applied for all detention pond designs. The NRCS hydrologic methods (available in TR-55, HEC-HMS, HEC-RAS, and the Hydrologic Engineering Center [HEC]) hydrologic methods may be used for areas of 200 acres or more.

1.2.8 Stormwater Detention

Pre-developed peak flows generated from the 25-year frequency storm shall not be increased. The peak flows from the 25-year storm shall be mitigated within a development or a common plan of development with release rates equal to, or less than the flows generated from the site for the 25-year storm event when the site was in its existing (natural) state. If a downstream detention pond is proposed for multiple developments, then all drainage easements and rights-of-way shall be in place to properly convey the runoff to said detention pond. Detention ponds shall be designed such that the 100-year storm will not overtop the structure. The design engineer shall design an emergency spillway system that will safely discharge the 100-year storm without damage to the downstream property.

The Public Works Director or his/her designee shall have the authority to waive the requirement for on-site detention, provided that at least one of the following conditions are met:

A. Discharge from the development will be received by an approved regional stormwater management facility. Under this provision, the applicant shall demonstrate that the peak, post-developed runoff generated from the 100-year storm can be conveyed downstream to the regional facility and not adversely impact any downstream properties. An adverse impact shall be:

    1. any impact that causes an inundation, or an increased inundation, of any building structure, roadway, or improvement

    2. downstream erosion and/or sedimentation, or an increase in erosion and/or sedimentation

B. The development is adjacent to a defined watercourse that has sufficient capacity to convey the site's post-developed peak discharge from the 100-year storm event without creating an adverse impact on any other upstream or downstream properties. The discharge in the watercourse shall be determined by using the 100-year storm event with the post-developed site for the development and the remainder of the watershed in an existing conditions state. The determination of ultimate build-out state within the watershed shall be based upon the identified land uses in the current adopted comprehensive Plan or Drainage Master Plan for the City.

C. The development is located such that on-site detention may worsen downstream conditions in the watershed. In such cases, the design Engineer shall demonstrate that conveyance or a combination of detention and conveyance will provide a safer condition. Available capacity downstream shall not be considered as sufficient justification to waive detention.

D. The applicant can demonstrate that there will be no adverse effect to downstream properties, or if changes made by the applicant to downstream properties can mitigate future adverse affects.

1.2.9 Floodplain Management

A. City of Killeen

In all cases where floodplain delineation is required, the determination of the base flood profile shall consider all existing property development within the contributing drainage basin up to the point of consideration. It is the responsibility of the design Engineer to determine existing developed conditions within the drainage basin based upon best available data. Best available data may include but is not limited to the Comprehensive Plan, the current adopted Drainage Master Plan, and the current Bell County FIS.

All new construction and substantial improvements of buildings (structures) within a floodplain shall follow the requirements as outlined in City Ordinance Chapter 12 and as amended and the National Flood Insurance Program (NFIP) administered by the Federal Emergency Management Agency (FEMA). In addition, all base flood event calculations shall be performed using computer software and methodologies accepted by the NFIP.

If development activities are proposed that will result in an increase of one (1) foot or more in the base flood profile anywhere within the community, the development permit applicant shall file a complete flood elevation study (through hydrologic and hydraulic analyses) with the Floodplain Administrator of the City of Killeen. If the proposed development will result in the modification of any FEMA-delineated regulatory flood hazard boundaries within the community, this flood elevation study shall comply with all requirements of the NFIP.

B. Federal Emergency Management Agency

1. FEMA maintains and approves or denies all proposed and physical revisions and amendments to the Digital Flood Insurance Rate Maps (DFIRMs) that depict regulatory floodplain and floodway boundaries within the City and its extraterritorial jurisdiction. The base flood hazards depicted on these DFIRMs are based on modeled conditions of development associated with the current effective Flood Insurance Study.

C. Coordination of City of Killeen and FEMA Floodplain Delineations

1. If the base flood hazards depicted on the current regulatory DFIRM are proposed to be changed due to development activities that will alter existing mapped conditions, then the following requirements shall apply:

a. The development permit applicant shall provide to the City evidence of FEMA-receipt of an application for a Conditional Letter of Map Revision (CLOMR).

b. If a final plat is approved before it is determined that a FEMA-approved Letter of Map Revision is necessary or desired for a proposed subdivision, the developer shall provide to the City a letter certifying that all CLOMR data has been submitted to FEMA for review before the Director of Public Works or his/her designee will release any phase of subdivision construction plans to the developer for construction.

c. Prior to issuance of a building permit for any lot affected by a proposed change to the current regulatory DFIRM, the applicant shall provide to the City evidence of final FEMA acceptance of the CLOMR submitted under (1a) or (1b) above.

2. If a CLOMR is not required before development begins but a Letter of Map Revision (LOMR) or a Letter of Map Amendment (LOMA) is required to update mapped conditions, then the following requirements shall apply:

   a. The development permit applicant shall provide to the City evidence of FEMA-receipt of an application for a LOMR or LOMA.

   b. Before issuance of a building permit for any lot affected by a proposed change to the current regulatory DFIRM, the applicant shall provide to the City evidence of final FEMA acceptance of the LOMR or LOMA submitted under (2a) above.

3. The development permit applicant shall bear the cost of all professional engineering services required to develop the application; respond to City and FEMA review comments; and obtain permit approval. The development permit applicant shall bear the cost of all fees, established by FEMA, associated with review and disposition of a CLOMR, LOMR, or LOMA.

1.2.10 Lot Grading

A. For plat applicants, all site developments must provide a site grading and drainage plan that includes drainage computations, detention of runoff (if required), and a detailed site grading plan that does not adversely affect adjacent lots, property, or downstream property. The grading plan shall include arrows indicating the direction of runoff for each lot, and all lots must have positive drainage away from structures.

B. For construction plan submittals, all items in section 1.2.10.A of this manual must be re-submitted and reflect any revisions to the design since the plat application. All lots that drain front to rear must be identified in the construction plan submittal. Finished floor elevations shall be a minimum of 1 foot above the average top of curb elevation fronting the lot (1.5 feet above the average edge of pavement where no curb is present) and a note shall be placed on the construction plans indicating that lots must meet this requirement. An elevation certificate will not be required to meet this requirement. Where practical, all lots shall be graded from rear to front at which point the drainage shall be intercepted by the street. If the minimum 1-foot requirement cannot be met due to land slope, topography or existing trees, alternate grading plans may be utilized. In these instances, it shall be demonstrated to the satisfaction of the Public Works Director or his/her designee that grading from front to rear would be more reasonably adaptable to the existing topography.

C. Finished floor elevations shall be shown for all lots adjacent to or encroaching upon the FEMA designated 100-year floodplain. Finished floor elevations shall be a minimum of 2 feet above the applicable base flood elevation per City Ordinance Chapter 12, as amended.

D. Lot-to-lot drainage is prohibited except in residential developments where one lot may drain onto one adjacent lot to the rear. Residential lots may drain to a side lot swale or flume only if such feature drains directly to public right-of-way or a protected drainage easement or watercourse. The cumulative stormwater runoff onto any single residential lot may not exceed the cumulative stormwater runoff generated from a total of two residential lots. Flumes are not subject to the channel freeboard requirements, as long as the 100-year runoff will be conveyed within the drainage easement that encompasses the flume.

E. The applicant for a building permit for a lot that is graded from front to rear shall prepare a detailed site grading plan that includes elevations for all corners of the subject lot, all corners of the downstream lot, the finished floor slab elevation, final contours, flow arrows, swales, and any modifications to side yard or rear yard fencing to facilitate removal of runoff from the subject lot.

1.2.11 Erosion Control

Rock berms, silt fences, sedimentation basins, stabilized construction entrances/exits, and similar recognized erosion and sediment control Best Management Practice (BMP) techniques shall be employed to prevent point source sedimentation loading of downstream facilities. Erosion control measure shall be provided along all disturbed areas adjacent to city maintained facilities. Such measures must be installed prior to city acceptance and must be maintained until final stabilization is achieved on the property. Such installations shall comply with current Texas Commission on Environmental Quality (TCEQ) requirements. Additional measures may be required by the Public Work Director or his/her designee during and after construction if erosion or sediment damage is documented as a violation of Texas Pollution Discharge Elimination System (TPDES) regulations or the City's Illicit Discharge Prevention and Storm Water Protection Ordinance.

1.3 DEFINITIONS

All terms and abbreviations used in the text are presented in the glossary of this Manual. If there are any conflicts between the terms provided in this Manual and the terms provided in the City of Killeen Code of Ordinances, the Code of Ordinances shall control.

SECTION 2.0 DETERMINATION OF STORM RUNOFF

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2.0 DETERMINATION OF STORM RUNOFF

2.1 GENERAL

If continuous records of the amounts of runoff from urban areas were as readily available as records of precipitation, they would provide the best source of data on which to base the design of storm drainage and flood protection systems. Unfortunately, such records are available in very few areas in sufficient quantity to permit an accurate prediction of the stormwater runoff. The accepted practice, therefore, is to relate runoff to rainfall, thereby providing a means for predicting the amount of runoff to be expected from urban watersheds at given recurrence intervals.

Numerous methods of rainfall runoff computations are available on which the design of storm drainage systems may be based. The method chosen is dependent upon the Engineer's technical familiarity and the size of the area to be analyzed. Within the chosen method, the Engineer will be responsible for making assumptions as to the development characteristics of the study area.

2.2 EFFECTS OF URBANIZATION

It has long been recognized that urban development has a pronounced effect on the rate of runoff from a given rainfall. Urbanization reduces the storage capacity of a watershed. This reduction of a watershed's storage capacity is a direct result of the elimination of porous surfaces, small ponds, and holding areas. This comes about by the grading and paving of sites, streets, drives, parking lots, and sidewalks, and by construction of buildings and other facilities characteristic of urban development. The result on hydraulic efficiency of a given drainage area is illustrated graphically on Figure 2-1 in Appendix A of this Manual, which is a plot of the runoff rate versus time for the same storm with two different stages of watershed development.

2.2.1 Design Assumptions for Storm Flow Analysis

A. When analyzing an upstream area for channel design purposes, existing conditions shall be used. When analyzing drainage infrastructure for the proposed development, the design Engineer shall assume the proposed development to be fully developed. Urbanization of the full watershed is required if agreements are in place to convey runoff through a development to a downstream facility. Zoning maps, future land use maps, and master plans should be used as aids in establishing the anticipated surface character of the ultimate development. The selection of design runoff coefficients and/or percent impervious cover factors are explained in the following discussions of runoff calculation.

B. An exception to paragraph A above may be granted if the channel is immediately downstream of a regional detention pond and written approval is obtained from the Public Works Director or his/her designee.

C. In designing a storm sewer system within a residential subdivision, full development of interior tracts without detention shall be assumed.

D. In designing a storm sewer system within a commercial or multifamily subdivision, 25-year storm flows can, at the Engineer's discretion, reflect the flow reduction anticipated by future detention ponds. This applies exclusively to the flows generated by those properties contained within the subdivision. Provisions for conveyance of the 100-year undetained flows within the right-of-way or drainage easements still apply.

E. In the event the Engineer desires to incorporate the flow reduction benefits of existing upstream detention ponds, the following field investigations and hydrologic analysis will be required:

1. A field survey of the existing physical characteristics of both the outlet structure and ponding volume. Any departure from the original Engineer's design must be accounted for. If a dual use for a detention facility exists (e.g., parking lot), then this too should be accounted for.

2. A comprehensive hydrologic analysis that simulates the attenuation of the contributing area ponds. This should not be limited to a linear additive analysis but rather a network of hydrographs, which considers incremental timing of discharge and potential coincidence of outlet peaks.

3. (Note that under no circumstances will the previously approved construction plans of the upstream ponds suffice as an adequate analysis. While the responsibility of the individual site or development plans rests with the Engineer of record, any subsequent engineering analysis must assure that all the incorporated ponds work collectively.):

2.3 METHOD OF ANALYSIS

Numerous methods of rainfall-runoff computation are available on which the design of storm drainage and flood control systems may be based. The Rational Method and the Variable Rainfall Intensity Method are accepted as adequate for drainage areas totaling 200 acres or less. For larger drainage systems, the SCS hydrologic methods (available in HEC-HMS, or the Tabular/Graphical methods) shall be used. The method of analysis shall remain consistent when drainage areas are combined and the method that applies to the largest combined drainage area should be used. Table 2-1 is to be used as a guide in determining some of the applicable methods for calculating storm runoff. The Engineer can use other methods, but must have their acceptability approved by the Public Works Director or his/her designee.

Table 2-1
Storm Runoff Calculation Methods

Contributing Area	Runoff Methods
Less than 200 acres	Rational or VRIM[1] SCS Tabular/Graphical[2]
200–400 acres	SCS Tabular/Graphical[3] HEC-HMS or FEMA-approved models
Greater than 400 acres	SCS HEC-HMS or FEMA-approved models

[1]Variable Rainfall Intensity Method (VRIM) in Section 2.4.5.

[2]SCS, Tabular/Graphical and HEC-HMS Methods in Section 2.5.4.

[3]It is recommended that the hand calculated SCS Tabular Method not be used for areas greater than 400 acres due to the rigorous nature of the calculations and likelihood of error.

2.4 RATIONAL METHOD

The Rational Method is based on the direct relationship between rainfall and runoff and is expressed by the following equation:

$$Q_p = CiA \qquad \text{(Equation 2-1)}$$

where:

Q_p is defined as the peak runoff in cubic feet per second (cfs). Actually, Q_p is in units of inches per hour per acre. Since this rate of in-ac/hr differs from cfs by less than 1% (1 in-ac/hr = 1.008 cfs), the more common units of cfs are used.

C is the coefficient of runoff representing the ratio of peak runoff rate "Q_p" to average rainfall intensity rate "I" for a specified area "A."

i is the average intensity of rainfall in inches per hour for a period of time equal to the time of concentration (t_c) for the drainage area to the point under consideration.

The following basic assumptions are associated with the Rational Method:

A. The storm duration is equal to the time of concentration.

B. The computed peak rate of runoff to the design point is a function of the average rainfall rate during the time of concentration to that point.

C. The return period or frequency of the computed peak flow is the same as that for the design storm.

D. The necessary basin characteristics can be identified and the runoff coefficient does not vary during a storm.

E. Rainfall intensity is constant during the storm duration and spatially uniform for the area under analysis.

2.4.1 Runoff Coefficient (C)

The proportion of the total rainfall that will reach the drainage system depends on the degree of imperviousness of the surface and the slope and storage characteristics of the area. Fully impervious surfaces, such as asphalt pavements and roofs of buildings, will be subject to approximately 100% runoff (regardless of the slope). On-site inspections and aerial photographs may prove valuable in estimating the nature of the surfaces within the drainage area.

The runoff coefficient “C” in the Rational Formula is also dependent on the character of the soil. The type and condition of the soil determines its ability to absorb precipitation. The rate at which a soil absorbs precipitation generally decreases as the rainfall continues for an extended period of time. The soil infiltration rate is influenced by the presence of soil moisture (antecedent precipitation), the rainfall intensity, the proximity of the groundwater table, the degree of soil compaction, the porosity of the subsoil, and ground slopes.

It should be noted that the runoff coefficient “C” is the variable of the Rational Method that is most difficult to precisely determine. A reasonable coefficient must be chosen to represent the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception, all of which affect the time distribution and peak rate of runoff.

Tables 2-2 and 2-3 presents recommended ranges for “C” values based on composite land use and surface types.

2.4.2 Time of Concentration

Time of concentration is defined as the time associated with the travel of runoff between any two given points within a studied drainage area. Runoff from a drainage area usually reaches a peak at the time when the entire area is contributing, in which case the time of concentration is the time for a drop of water to flow from the most remote point in the watershed to the point of interest. However, runoff may reach a peak prior to the time the entire drainage area is contributing. Selection of the appropriate storm intensity to the analyzed time of concentration will be the controlling factor in determining the peak runoff rate for a given watershed. Sound engineering judgment should be used to determine the time of concentration. The time of concentration to any point in a storm drainage system is a combination of the sheet flow (overland), the shallow concentrated flow, and the channel flow, which includes storm sewers. The minimum time of concentration for any area shall be 6 minutes when using the NRCS TR-55 method as discussed in the following sections. Other industry standard methods may be used to determine the time of concentration such as the Kerby-Kirpich method as outlined in the TxDOT Hydraulic Design Manual and as amended.

Table 2-2

Composite Land Use Runoff Coefficients (C)

Description of Area	Return Period					
	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
Park and Open Spaces						
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, Over 7%	0.37	0.40	0.42	0.46	0.49	0.53
Single Family Residential Land Use						
Estates greater than 20,000-sq. foot lots						
Flat, 0–2%	0.32	0.34	0.36	0.41	0.44	0.48
Average, 2–7%	0.38	0.41	0.44	0.49	0.52	0.56
Steep, Over 7%	0.42	0.45	0.48	0.53	0.56	0.60
10,000 to 20,000-sq. foot lots						
Flat, 0–2%	0.38	0.41	0.44	0.48	0.51	0.56
Average, 2–7%	0.44	0.47	0.51	0.55	0.58	0.63
Steep, Over 7%	0.47	0.51	0.54	0.58	0.62	0.66
7,500 to 10,000-sq. foot lots						
Flat, 0–2%	0.44	0.47	0.50	0.55	0.58	0.62
Average, 2–7%	0.49	0.52	0.56	0.60	0.64	0.68
Steep, Over 7%	0.52	0.55	0.58	0.63	0.66	0.71
5,000 to 7,500-sq. foot lots						
Flat, 0–2%	0.50	0.54	0.56	0.61	0.64	0.69
Average, 2–7%	0.54	0.58	0.61	0.65	0.69	0.74
Steep, Over 7%	0.56	0.60	0.63	0.68	0.71	0.76
Multiple Family Residential Land Use						
Low Density (4 stories or less)	0.54	0.58	0.61	0.65	0.69	0.74
Medium Density (7 stories or less)	0.56	0.60	0.63	0.68	0.71	0.76
High Density (more than 7 stories)	0.59	0.63	0.66	0.71	0.75	0.80
Commercial Land Use						
Limited and General Office Building Sites	0.63	0.67	0.70	0.75	0.79	0.84
Shopping Center Sites	0.67	0.71	0.74	0.79	0.83	0.88
Neighborhood Business Districts	0.67	0.71	0.74	0.79	0.83	0.88
Office Parks	0.67	0.71	0.74	0.79	0.83	0.88
Central Business District Sites	0.74	0.79	0.82	0.87	0.91	0.96
Industrial Land Use						
Limited (service station, restaurant)	0.67	0.71	0.74	0.79	0.83	0.88
General (auto sales, convenience storage)	0.67	0.71	0.74	0.79	0.83	0.88
Heavy (surface parking, warehousing)	0.74	0.79	0.82	0.87	0.91	0.96

Table 2-3
Surface Type
Runoff Coefficient (C)

Character of Surface	Return Period					
	2 Years	5 Years	10 Years	25 Years	50 Years	100 Years
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95
Concrete	0.75	0.8	0.83	0.88	0.92	0.97
Poor Condition*						
Flat, 0–2%	0.32	0.34	0.37	0.40	0.44	0.47
Average, 2–7%	0.37	0.40	0.43	0.46	0.49	0.53
Steep, over 7%	0.40	0.43	0.45	0.49	0.52	0.55
Fair Condition**						
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53
Good Condition***						
Flat, 0–2%	0.21	0.23	0.25	0.29	0.32	0.36
Average, 2–7%	0.29	0.32	0.35	0.39	0.42	0.46
Steep, over 7%	0.34	0.37	0.40	0.44	0.47	0.51
Cultivated						
Flat, 0–2%	0.31	0.34	0.36	0.40	0.43	0.47
Average, 2–7%	0.35	0.38	0.41	0.44	0.48	0.51
Steep, over 7%	0.39	0.42	0.48	0.48	0.51	0.54
Pasture/Range						
Flat, 0–2%	0.25	0.28	0.30	0.34	0.37	0.41
Average, 2–7%	0.33	0.36	0.38	0.42	0.45	0.49
Steep, over 7%	0.37	0.40	0.42	0.46	0.49	0.53
Forest/Woodlands						
Flat, 0–2%	0.22	0.25	0.28	0.31	0.35	0.39
Average, 2–7%	0.31	0.34	0.36	0.40	0.43	0.47
Steep, over 7%	0.35	0.39	0.41	0.45	0.48	0.52

Source: R.L. Rossmiller, "The Rational Formula Revisited"; City of Austin, Watershed Engineering Division.

*Grass cover less than 50% of the area.
**Grass cover on 50 to 75% of the area.
***Grass cover larger than 75% of the area.

A. **Sheet Flow (Overland Flow).** Sheet flow is shallow flow over land surfaces, which usually occurs at the headwaters of streams. The Engineer should realize that sheet flow occurs for only very short distances in urbanized conditions. Urbanized areas are assumed to have a maximum sheet flow of 100 feet or less. The time of concentration for sheet flow can be computed using Manning's kinematic equation (Equation 2-2):

$t_c = 0.007(nL)^{0.8}/(P_2^{0.5}\ s^{0.4})$ (Equation 2-2)

where:

t_c = Time of concentration in hours

L =Sheet flow length of the reach in feet

n = Overland flow Manning's n (see Table 2-4)

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

s = Slope of the ground in feet per foot (ft/ft)

B. **Shallow Concentrated Flow.** After a maximum of 100 feet sheet flow becomes shallow concentrated flow. The time of concentration for shallow concentrated flows can be computed from equation 2-3 as follows:

$t_c = L/(3600Ks^{0.5})$ (Equation 2-3)

where:

t_c = Time of concentration in hours

L = Shallow concentrated length of the reach in feet

K =16.13 for unpaved surface, 20.30 for paved surface

s = Slope of the ground in ft/ft

C. **Channel or Storm Sewer Flow.** The velocity in an open channel or a storm sewer not flowing full can be determined by using Manning's Equation. Channel velocities can also be determined by using backwater profiles. Usually, average flow velocity is determined assuming a bank-full condition. The details of using Manning's equation and selecting Manning's "n" values for channels can be obtained from Section 6 of this Manual.

For full flow storm sewer conditions (pressure flow) the following equation should be applied:

$$V = Q/A \qquad \text{(Equation 2-4)}$$

where:

V = Average velocity, feet per second (ft/s)

Q = Design discharge, cfs

A = Cross-sectional area, ft^2

Table 2-4
Manning "n" for Overland Flow for use in the NRCS Method

Surface Description	Manning's n
Smooth surface (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	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:	
Light underbrush	0.40
Dense underbrush	0.80

Source: NRCS Urban Hydrology for Small Watersheds TR-55 (NRCS 1986).

2.4.3 Rainfall Intensity (i)

Rainfall intensity (i) is the average rainfall rate in inches per hour, and is selected on the basis of design rainfall duration and design frequency of occurrence. The design duration is equal to the time of concentration for the drainage area under consideration. The design frequency of occurrence is a statistical variable, which is established by design standards or chosen by the Engineer as a design parameter.

The selection of the frequency criteria is necessary before applying any hydrologic method. Storm drainage improvements shall be designed to intercept and carry the runoff from a 25-year frequency storm, with an auxiliary or overflow system capable of carrying a 100-year frequency storm.

The rainfall intensity used in the rational method is read from the intensity-duration-frequency curves based on the selected design frequency and design duration.

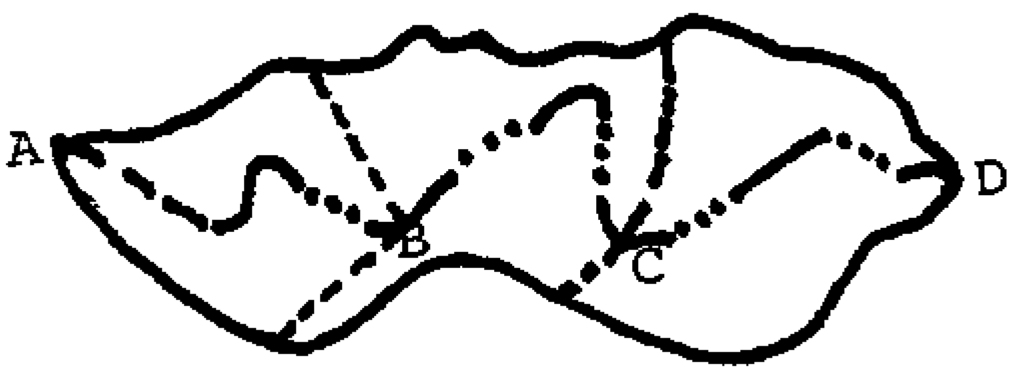
The Killeen intensity-duration-frequency curves are shown on Figure 2-2 in Appendix A of this Manual. The intensity-duration-frequency curves and the intensity-duration equations are applicable for all design frequencies shown and for storm durations from 5 minutes to 4 hours. The intensity-duration-frequency equations for this area are available in the TxDOT Hydraulic Design Manual, as amended. They are required for use in determining peak flows by the Rational Method or other appropriate methods.

2.4.4 Drainage Area (A)

The size (acres) of the watershed needs to be determined for application of the Rational Method. The area may be determined through the use of maps, supplemented by field surveys where topographic data has changed or where the contour interval is too great to distinguish the direction of flow. The drainage divide lines are determined by street layout, lot grading, structure configuration and orientation, and many other features that are created by the urbanization process.

Example 2-1

An urbanized watershed is shown on the following figure. Three types of flow conditions exist between the most distant point in the watershed and the outlet. The calculation of time of concentration and travel time in each reach is as follows:



Reach	Description of Flow	Slope (%)	Length (feet)	Drainage Area (Acre)	"n" Value
A to B	Sheet flow (grass lawn)	4.5	100	3	0.41
B to C	Shallow concentrated flow (curb & gutter)	2.0	840	20	0.015
C to D	Storm drain with inlets (3-ft-	1.5	1,200	30	0.015

	diameter pipe)				

For reaches A-B and B-C, the time of concentration can be calculated from equations 2-2 and 2-3.

t_c (A-B) $= 0.007(0.41*100)^{0.8}/(3.4^{0.5}*0.045^{0.4})$

$= 0.007(19.51)/0.5334$

$= 0.256$ hrs * 60 min. = 15.4 min.

t_c (B-C) $= 840/(3600*20.30*0.02^{0.5})$

$= 840/10335$

$= 0.0813$hrs. * 60 min. = 4.9 min.

The flow velocity in reach C-D needs to be calculated from Manning's Equation, using the assumption of full pipe flow, as follows:

V (C-D) $= (1.49/n)\ R^{0.67}s^{0.5}$

$= (1.49/n)\ (D/4)^{0.67}s^{0.5}$

$= (1.49/0.015)\ (3/4)^{0.67}\ (0.015)^{0.5}$

$= 10.0$ ft/s

The runoff coefficients (C) for the 3 areas are given as follows for the 100-year storm. The time of concentration (tc) is calculated by dividing the length by the velocity.

Reach	**Length (feet)**	**Velocity (fps)**	**t_c (min)**	**C**	**Area (acre)**
A-B	100	-	15.4	0.41	3
B-C	840	-	4.9	0.85	20
C-D	1200	10.0	2.0	0.81	30
			22.2		53

The intensity (i) of the 100-year storm (from Figure 2-2 in Appendix A of this Manual) for 22.2 minutes = 7.8 inches per hour.

The composite runoff coefficient (C) = (0.41 x 3 + 0.85 x 20 + 0.81 x 30)/53= 0.80

Thus the peak flow Qp = CiA = 0.80 x 7.8 in/hr x 53 acre = 331 cfs

2.4.5 Variable Rainfall Intensity Method

The Variable Rainfall Intensity Method, also known as the Modified Rational Method, is one of the methodologies that uses the peak flow (Qp) calculated from the Rational Method to develop synthetic storm hydrographs. The detailed information on this method can be found in the Bibliography, Item 2-5 of this Manual. The following example illustrates the application of the variable rainfall intensity method technique in constructing a 10-year design storm hydrograph.

Example 2-2

Variable Rainfall Intensity Method

Given: A drainage area, when fully developed, will have the following characteristics:

Drainage area = 100 acres Runoff coefficient C = 0.45

Design rainfall frequency: 10 year

Bell County rainfall intensity-duration-frequency curves (Figure 2-2 in Appendix A of this Manual)

Time of concentration = 40 minutes

Find: The 10-year design storm and resulting flood hydrograph

Solution: The solution is outlined in Table 2-5, which shows the development of the design 10-year frequency storm distribution, and Table 2-6, which shows the computation of the design 10-year flood hydrograph.

The computation procedures for Table 2-5 are explained as follows:

Column 1: Duration (minutes) = length of storm

Column 2: Rainfall Intensity read from Figure 2-2 in Appendix A of this manual corresponding to the duration time in Column 1

Column 3: Accumulated Depth (inches) = total precipitation for storm of specified duration (from Table 2-6)

Column 4: Incremental Depth (inches) = difference in total precipitation between specified duration and duration of 5 minutes less than specified duration (e.g., P35 minutes – P30 minutes)

Column 5: Incremental Intensity (inches/hour) = Incremental Depth (inches) x (60 minutes/hour)/(5 minutes)

Table 2-5
Development of a 10-Year Frequency Storm

Duration (Min) (1)	**Intensity (In/Hr) (2)**	**Accumulated Depth (In) (3)**	**Incremental Depth (In) (4)**	**Incremental Intensity (In/hr) (5)**
5	8.64	0.034	0.34	0.41
10			0.36	0.43
15	6.16	0.108	0.038	0.46
20			0.04	0.48
25	5.00	0.19	0.04	0.48
30			0.05	0.60
35	4.30	0.29	0.05	0.60
40			0.06	0.72
45	3.73	0.41	0.06	0.72
50			0.07	0.84
55	3.33	0.56	0.08	0.96
60			0.09	1.08
65	3.00	0.76	0.11	1.32
70			0.13	1.56
75	2.74	1.07	0.18	2.16
80			0.24	2.88
85	2.50	1.67	0.36	4.32
90			0.72	8.64
95	2.32	2.89	0.5	6.0
100			0.29	3.48
105	2.17	3.38	0.20	2.4
110			0.15	1.8
115	2.05	3.65	0.12	1.44
120			0.1	1.2
125	1.94	3.83	0.08	0.96
130			0.08	0.96
135	1.85	3.98	0.07	0.84
140			0.06	0.72
145	1.77	4.09	0.05	0.60
150			0.05	0.60
155	1.69	4.19	0.05	0.60
160			0.04	0.48
165	1.62	4.27	0.04	0.48
170			0.04	0.48
175	1.56	4.34	0.03	0.36
180			0.03	0.36
185	1.50	4.38		0.36

Table 2-6 illustrates the computed 10-year flood hydrograph for the drainage area described in Table 2-5. Referring to Table 2-6, the columns are identified and computed as follows:

Column 1: Time (minutes) = time from the beginning of the storm.

Column 2: i (inches/hour) = incremental intensities (from Table 2-5).

Column 3: Sum (i) = summation of all incremental intensities to the specified time.

Column 4: "Sum" (i lagged) = column 3 displaced a total time equal to the time of concentration for the area producing this hydrograph.

Column 5: (3) - (4) = column 3 - column 4.

Column 6: I_{tc} = column 5 divided by the number of time increments in the time of concentration for the area producing this hydrograph. This column expresses the average intensity over a period of time equal to the time of concentration for the area producing this hydrograph, as measured at the specified chronological time.

Column 7: Q (cfs) = column 6 x "C" x A (for the area producing this hydrograph). This column is for the rising limb calculation.

Column 8: Time Folded revised times and flows for falling limb of hydrograph; falling limb is mirror image of rising limb, but expanded to twice the length. Intermediate values can be linearly interpolated from neighboring values, because 5-minute increments doubled to 10-minute increments leave out intervening values.

The computations were stopped in column 7 when the rising limb of the hydrograph reached its peak value. At this point, the time scale can be folded as shown in column 8. Doubling the time increments for the falling limb serves to double the volume that would have been under that portion of the runoff hydrograph. The volume under the entire discharge hydrograph will be three times that under the rising limb.

With this assumption, the volume of runoff expressed as a percentage from an area with a runoff coefficient of 0.45 becomes approximately 67.5% rather than 45% of the rainfall. In this procedure, the C value from the Rational Method formula represents the ratio of the peak runoff to the average rainfall intensity rate for a period equal to the time of concentration and not a simple runoff to rainfall ratio.

Table 2-6
Runoff Computations from a 100-acre Area with a Time of Concentration of 40 Minutes and C = 0.45

Time (min) (1)	**I_{10} (in/hr) (2)**	**Sum I_{10} (3)**	**Sum I_{10} (Lagged 40 min) (4)**	**Time (3) – (4) (5)**	**I_{40} (in/hr) (6)**	**Q (cfs) (7)**	**Folded (min) (8)**
0							330
5	0.41	0.41		0.41	0.05	2.3	320
10	0.43	0.84		0.84	0.10	4.5	310
15	0.46	1.3		1.3	0.16	7.2	300
20	0.48	1.78		1.78	0.22	9.9	290
25	0.48	2.26		2.26	0.28	12.6	280
30	0.6	2.86		2.86	0.36	16.2	270
35	0.6	3.46		3.46	0.43	19.3	260
40	0.72	4.18		4.18	0.52	23.4	250
45	0.72	4.9	0.41	4.5	0.56	25.2	240
50	0.84	5.7	0.84	4.9	0.61	27.4	230
55	0.96	6.7	1.3	5.4	0.67	30.1	220
60	1.08	7.8	1.78	6.0	0.75	33.7	210
65	1.32	9.1	2.26	6.8	0.85	38.2	200
70	1.56	10.7	2.86	7.8	0.97	43.6	190
75	2.16	12.8	3.46	9.3	1.16	52.2	180
80	2.88	15.7	4.18	11.5	1.44	64.8	170
85	4.32	20.0	4.9	15.1	1.89	85.1	160
90	8.64	28.7	5.7	23.0	2.87	129.1	150
95	6.0	34.7	6.7	28.0	3.5	157.5	140
100	3.48	38.1	7.8	30.3	3.8	171.0	130
105	2.4	40.5	9.1	31.4	3.92	176.4	120
110	1.8	42.3	10.7	31.6	3.95	177.7	(peak)
115	1.44	43.8	12.8	31.0	3.87	174.1	

2.5 SOIL CONSERVATION SERVICE METHODS

The SCS hydrologic methods have been widely used by Engineers and Hydrologists for analyses of small urban watersheds. These methods resulted from extensive analytical work using a wide range of statistical data concerning storm patterns, rainfall-runoff characteristics and many hydrologic observations in the United States. The SCS utilizes a 24-hour storm duration, which is considered to be acceptable for the Killeen area. It should be noted that if the SCS storms are applied, the Type III distribution should be used.

The SCS methods can be applied to urban drainage areas of any size. A brief explanation of the runoff curve numbers, the tabular and graphical methods, and the HEC-HMS method are introduced in this Section. The Supplemental Section 2.6 for the SCS hydrology includes the rainfall-runoff relationship and the dimensionless Unit Hydrograph. For detailed information, the user is referred to the following SCS publications:

NEH-4: "Hydrology," Section 4, *National Engineering Handbook*

HEC-HMS: *Hydrologic Modeling System, Technical Manual*

TR-55: *Urban Hydrology for Small Watersheds*

TP-149: *A Method for Estimating Volume and Rate of Runoff in Small Watersheds*

2.5.1 Left Blank Intentionally

2.5.2 Soil Conservation Service Runoff Curve Numbers

The SCS has developed an index, the runoff Curve Number, to represent the combined hydrologic effect of soil type, land use, agricultural land treatment class, hydrologic condition, and antecedent soil moisture. These watershed factors have the most significant impact in estimating the volume of runoff, and can be assessed from soil surveys, site investigations, and land use maps.

The curve number is an indication of the runoff producing potential of the drainage area for a given antecedent soil moisture condition, and it ranges in value from 0 to 100. The SCS runoff curve numbers are grouped into three antecedent soil moisture conditions—Antecedent Moisture Condition I, Antecedent Moisture Condition II, and Antecedent Moisture Condition III. Values of runoff curve numbers for all three conditions may be computed following guidelines in "Hydrology, Section 4," National Engineering Handbook. Antecedent Moisture Condition I is the dry soil condition and Antecedent Moisture Condition III is the wet soil condition. Antecedent Moisture Condition II is normally considered to be the average condition.

However, studies of hydrologic data indicate that Antecedent Moisture Condition II is not the average throughout Texas. Instead, investigations have shown that the average condition ranges from Antecedent Moisture Condition I in west Texas to between Antecedent Moisture Condition II and Antecedent Moisture Condition III in east Texas. The values given in Table 2-7 are for an Antecedent Moisture Condition II. If it is desired to change to an Antecedent Moisture Condition I or III, the adjustments given in TR-55 or "Hydrology, Section 4," *National Engineering Handbook* should be used.

The SCS has classified more than 4,000 soils into four hydrologic groups, identified by the letters A, B, C, and D to represent watershed characteristics.

Group A: (Low runoff potential). Soils having a high infiltration rate even when thoroughly wetted and consisting chiefly of deep, well-drained to excessively drained sands or gravels.

Group B: Soils having a moderate infiltration rate when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse texture.

Group C: Soils having a slow infiltration rate when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soil with moderately fine to fine texture.

Group D: (High runoff potential). Soils having a very slow infiltration rate when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface and shallow soils over nearly impervious material.

The list of most soils in United States along with their hydrologic soil classification is given in the TR-55 publication. The minimum infiltration rates for the four soils groups are:

Group	Minimum Infiltration Rate (inch/hour)
A	0.30–0.45
B	0.15–0.30
C	0.05–0.15
D	0.00–0.05

Table 2-7 lists the curve numbers for the four soil groups under various land uses, land treatment, and hydrologic conditions. In order to determine the soil classifications in the Killeen area, the SCS *Soil Survey of Bell County and/or Coryell County, Texas,* should be used.

Table 2-7
SCS Runoff Curve Numbers for Urban Areas and Agriculture Lands

Cover Description	Average % Impervious Area [1]	Curve Numbers for Hydrologic Soil Group			
Cover type and Hydrologic Condition		**A**	**B**	**C**	**D**
Fully Developed Urban Areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)		68	79	86	89
Poor condition (grass cover 50%)		49	69	79	84
Fair 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 storms sewers (excluding right-of-way)		98	98	98	
Paved open ditches (including right-of-way)		83	89	92	
Gravel (including right-of-way)		76	85	89	
Dirt (including right-of-way)		72	82	87	
Urban districts:					
Commercial	85	89	92	94	95
Business Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (townhouse)	65	77	85	90	92
¼ acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
½ 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
Agricultural Lands					
Grasslands, or range- Continuous forage for grazing[2]	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80

Table 2-7 (Cont'd)

Cover Description		Curve Numbers for Hydrologic Soil Group			
Cover type and Hydrologic Condition	Average % Impervious Area [1]	A	B	C	D
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[3]	Poor Fair Good	48 35 30	67 56 48	77 70 65	83 77 73
Woods – grass Combination (orchard or tree farm)[4]	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods[5]	Poor Fair Good	45 35 30	66 60 55	77 73 70	83 79 77
Farmsteads-buildings, lanes, driveways, and surrounding lots		59	74	82	86

Source: Soil Conservation Service. TR-55: *Urban Hydrology for Small Watersheds.*

[1]The average percent impervious area shown was used to develop the composite curve numbers. Other assumptions are as follows: Impervious areas area directly connected to the drainage system, impervious areas have a curve number of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

[2]Poor: Less than 50% ground cover or heavily grazed with no mulch.

Fair: 0 to 75% ground cover and not heavily grazed.

Good: Greater than 75% ground cover and lightly or only occasionally grazed.

[3] Poor: Less that 50% ground cover

Fair: 50 to 75% ground cover

Good: Greater than 75% ground cover

[4]Curve numbers shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the curve numbers for woods and pasture.

[5]Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.

Fair: Woods are grazed but not burned, and some forest litter covers the soil.

Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

2.5.3 Time of Concentration

The procedures for estimating time of concentration for the SCS method are described in SCS Technical Release 55 (TR-55) as discussed in Section 2.5.3 above. Three types of flow (sheet flow, shallow concentrated flow, and channel flow) are considered.

In SCS hydrograph analysis, the time of concentration is the time from the end of excess rainfall to the point of inflection on the falling limb of the hydrograph. The time of concentration determines the shape of the runoff hydrograph. Times of concentration are required for the existing and developed conditions to adequately model the impact of the development on stormwater runoff. In general, times of concentration for the developed condition should be calculated based on conservative assumptions concerning the increased hydraulic efficiency expected with an ultimate developed condition. For instance, while sheet flow for existing conditions is typically limited to 100 feet, sheet flow for developed conditions should be limited to 50 feet.

2.5.4 Peak Flow Calculation

The SCS has presented several methods for computing runoff hydrographs for drainage areas. The Tabular, Graphical, and HMS methods are considered acceptable for the Killeen area. The parameters required to calculate the hydrograph are the rainfall distribution, runoff curve numbers, time of concentration, and drainage area.

A. **Tabular Method.** The Tabular Method can be used to develop composite flood hydrographs at any point within a watershed by dividing the watershed into subareas. The method is useful for watersheds where runoff hydrographs are needed for nonhomogeneous areas (i.e., the watershed is divided into homogeneous sub-areas). It is especially applicable for estimating the effects of land use change in just a portion of the watershed. It should be noted that the tables in the TR-55 publication for the tabular method are based on the SCS 24-hour rainfall distributions. The Engineer shall apply those tables corresponding to a Type III rainfall distribution for the Killeen area.

   The basic requirement for use of the Tabular Method is the tabular discharge values for the different types of storm distributions. The tabular discharge values in cfs/mi^2/in (cubic feet of discharge per second per square mile of watershed per inch of runoff) are given in TR-55 for a range of times of concentration from 0.1 to 2 hours and reach travel times of 0 to 3 hours. The discharge values were developed from the HEC-HMS program by computing hydrographs for a 1-square-mile drainage area at selected times of concentration and routing them through stream reaches with the range of travel times indicated.

   The other input needed to develop the composite flood hydrograph includes the total runoff volume (Q_v) and the drainage area (A_m). The equation for calculating the flow at any time is:

   $$q = q_t A_m Q_v \qquad \text{(Equation 2-6)}$$

   where:

q = Hydrograph ordinate at hydrograph time t, cfs

q_t = Individual value read from the tabular discharge tables, cfs/mi^2/in

A_m = Drainage area of individual subwatershed, mi^2

Q_v = Total runoff volume, inches

The composite flood hydrograph is obtained by summation of the individual subarea hydrographs at each time step. For measuring runoff from a nonhomogeneous watershed, the subdivision of the watershed into relatively homogeneous subareas is required. For additional information regarding the Tabular Method, SCS publication TR-55 should be consulted.

B. **Graphical Method.** As in the Tabular Method, the Graphical Method is based on hydrograph analyses using the HEC-HMS computer program. The Graphical Method provides a determination of peak discharge only. If a hydrograph is needed or watershed subdivision is required, use the Tabular or HEC-HMS Method. TR-55 lists in detail the limitations of the Graphical Method and the Engineer should be well aware of these before proceeding. The input requirements for the Graphical Method are as follows:

1. t_c (hrs)
2. Drainage Area (mi^2)
3. Type III rainfall distribution
4. 24-hour, rainfall (in)
5. CN

The peak discharge equation for the graphical method is:

$q_p = q_u A_m Q$ (Equation 2-7)

where:

*q_p = peak discharge (cfs)

q_u = unit peak discharge (csm/in)

A_m = drainage area (mi^2)

Q = runoff (in)

*Note the published SCS equation also has an Fp factor for pond and swamp conditions. This has been omitted since it is not applicable to the Killeen area.

For additional information regarding the Graphical Method, SCS publication TR-55 should be consulted.

C. **HEC-HMS Method.** The HEC-HMS method is a computer program that develops runoff hydrographs for a watershed. The input information includes drainage area, time of concentration, SCS curve number, a specific rainfall distribution, and the antecedent soil moisture condition.

The HEC-HMS program was developed by the SCS to assist in the hydrologic evaluation of flood events for use in analysis of water resource projects. Besides developing the runoff hydrograph from any synthetic or natural storm rainfall, the program provides the capability to route, add, store, divert, or divide hydrographs to convey floodwater from watershed headwaters to watershed outlets.

The program uses the procedures described in SCS National Engineering Handbook "Hydrology, Section 4," except for the reach routing procedures. The modified Attenuation-Kinematic routing method is used for reach routing. Uniform rainfall depth and distribution over time are assumed over a subarea, groups of subareas, or the whole watershed.

2.6 SUPPLEMENTAL SECTION: SOIL CONSERVATION SERVICE HYDROLOGY

2.6.1 Rainfall-Runoff Relationship

The SCS has developed a rainfall-runoff relationship to calculate the total runoff volume for a single storm. Based on the relationship between rainfall, runoff, and retention (i.e., the rain not converted to runoff), an arithmetic equation for a storm without any initial abstraction can be expressed as:

$$F/S = Q/P \qquad \text{(Equation S-1)}$$

where:

Q = Actual runoff volume

P = Rainfall (P is equal or greater than Q)

F = Actual retention after runoff begins

S = Potential maximum retention after runoff begins (S is equal to or greater than F)

The retention, S, is a constant for a particular storm because it is the maximum that can occur under the existing conditions if the storm continues without limit. The retention F varies because it is the difference between P and Q at any point on the mass curve, or:

$F = P - Q$ (Equation S-2)

The actual runoff (Q) can be solved as:

$Q = P^2/(P+S)$ (Equation S-3)

which is a rainfall-runoff relationship in which the initial abstraction is zero.

If an initial abstraction (l_a) greater than zero is considered, the amount available for runoff is $P - l_a$ instead of P. By substituting ($P - l_a$) for P in equation S-1, the following equation results. The new arithmetic expression becomes:

$F/S = Q/(P-I_a)$ (Equation S-4)

where $F \leq S$, and $Q \leq (P - l_a)$. The total retention for a storm consists of l_a and F. The total potential maximum retention (as P gets very large) consists of 1_a and S.

The actual runoff is:

$Q = ((P-l_a)+S)$ (Equation S-5)

The initial abstraction (l_a) is a function of land use, treatment and condition, interception, infiltration, depression storage, and antecedent soil moisture. An empirical analysis performed by the SCS found that the initial abstraction is estimated as:

$l_a = 0.2S$ (Equation S-6)

Thus, the runoff volume (Q) can be obtained from the volume of precipitation (P) and potential maximum retention (S) as follows:

$Q = (P - 0.2\ S)^2/(P + 0.8S)$ (Equation S-7)

Empirical studies indicate that S is a function of the curve number as follows:

$S = (1000/CN)-10$ (Equation S-8)

Therefore, the runoff volume can be determined as a function of precipitation volume and curve number.

2.6.2 Soil Conservation Service Dimensionless Unit Hydrograph

To estimate the peak discharge and establish a runoff hydrograph in the SCS methods, the concept of a dimensionless unit hydrograph is applied. The SCS dimensionless unit hydrograph was derived from analysis of a large number of unit hydrographs developed using gage data from watersheds of a wide range in size and geographical location. The dimensionless unit hydrograph has ordinate values expressed in a dimensionless ratio q/q_p and abscissa values of t/T_p, where q_p is the peak discharge at time T_p and q is the discharge at time t. Figure 2-3 in Appendix A of this Manual shows the shape of the dimensionless unit hydrograph. The mass curve is also illustrated on Figure 2-3 in Appendix A of this Manual with coordinates of Q_a/Q vs t/t_p, in which Q_a is the accumulated volume at time t, and Q is the total volume. Table S-1 lists dimensionless discharge ratios and mass curve ratios for dimensionless time ratios for use in calculating unit hydrographs and mass curves.

The curvilinear unit hydrograph can be approximated by an equivalent triangular unit hydrograph, as shown by dotted lines on Figure 2-3 in Appendix A of this Manual. The area under the rising limb (before time T_p) of the two unit hydrographs are the same. The time base of the dimensionless unit hydrograph is 5 times the time-to-peak (T_p), while the time base of the triangular unit hydrograph is only 2.67 times the time-to-peak (T_p). The transformation of curvilinear unit hydrograph to triangular unit hydrograph provides a solution for the peak flow.

A. **Derivation of Peak Flow.** The area under the triangular unit hydrograph on Figure 2-3 in Appendix A of this Manual equals the volume of direct runoff Q, which can be calculated by:

   $Q = q_p (T_p + T_r)/2$ (Equation S-9)

   where:

   Q = Direct runoff, inches

   T_p = Time to peak, hours

   T_r = Recession time, hours

   Q_p = Peak discharge, inches per hour

   The runoff Q derived from this equation is the same as estimated by Equation S-7. By Equation S-9, the peak discharge q_p can be solved as:

   $q_p = 2Q/(T_p + T_r)$ (Equation S-10)

   Let, $K = 2/(1 + T_r/T_p))$ (Equation S-11) therefore, $q_p = KQ/T_p$ (Equation S-12)

where:

Q = Direct runoff, inches

T_p = Time to peak, hours

T_r = Recession time, hours

q_p = Peak discharge, inches per hour

In making the conversion from inches per hour to cfs and defining the equation variables in terms ordinarily used, including drainage area (A) in square miles and time (T) in hours, equation S-12 becomes the general equation:

$q_p = (645.33\ KAQ)/T_p$ (Equation S-13)

Where q_p is peak discharge in cfs and the conversion factor 645.33 is the rate required to discharge 1 inch of excess rainfall from 1 square mile in 1 hour.

The relationship of the triangular unit hydrograph, shows that $T_r = 1.67\ T_p$ and gives $K = 0.75$ by Equation S-11. Then substituting into equation S-13 gives:

$q_p = 484A\ Q/T_p$ (Equation S-14)

Because the volume under the rising side of the triangular unit hydrograph is equal to the volume under the rising side of the curvilinear dimensionless unit hydrograph on Figure 2-3 in Appendix A of this Manual, the constant 484, or peak rate factor, is applied for calculation of the peak discharge for the dimensionless unit hydrograph.

Table S-1
Ratios for SCS Dimensionless Unit Hydrograph and Mass Curve

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Q_a/Q)
0.0	0.000	0.001
0.1	0.030	0.001
0.2	0.100	0.006
0.3	0.190	0.012
0.4	0.310	0.035
0.5	0.470	0.065
0.6	0.660	0.107
0.7	0.820	0.163
0.8	0.930	0.228
0.9	0.990	0.300
1.0	1.000	0.375
1.1	0.990	0.450
1.2	0.930	0.522
1.3	0.860	0.589
1.4	0.780	0.650
1.5	0.680	0.700
1.6	0.560	0.751
1.7	0.460	0.790
1.8	0.390	0.822
1.9	0.330	0.849
2.0	0.280	0.871
2.2	0.207	0.908
2.4	0.147	0.934
2.6	0.107	0.953
2.8	0.077	0.967
3.0	0.055	0.977
3.2	0.040	0.984
3.4	0.029	0.989
3.6	0.021	0.993
3.8	0.015	0.995
4.0	0.011	0.997
4.5	0.005	0.999
5.0	0.000	1.000

Source: Soil Conservation Service. *TR-55 Urban Hydrology for Small Watersheds*

SECTION 3 STREET FLOW

3.0 STREET FLOW 3-1

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3.0 STREET FLOW

3.1 GENERAL

The location of inlets and permissible flow of water in streets must be related to the extent and frequency of interference to traffic and the likelihood of flood damage property for the 25- and 100-year frequency storms. Interference to traffic is regulated by design limits of the spread of water into traffic lanes. Flooding of surrounding property from streets is controlled by grading and the designated design storm. Conveyance provisions for the 100-year storm must also be made within defined right-of-way and easements.

3.1.1 Interference Due to Flow in Streets

Water that flows in a street, whether from rainfall directly onto the pavement surface or overland flow entering from adjacent land areas, will flow in the gutters of the street until it reaches an overflow point or sump, such as a storm sewer inlet. As the flow progresses downhill and additional areas contribute to the runoff, the width of flow will increase and progressively encroach into the traffic lane. On streets where parking is not permitted, as with many arterial streets, flow widths exceeding one traffic lane become a traffic hazard. Field observations show that vehicles will crowd adjacent lanes to avoid curb flow.

As the width of flow increases, it becomes impossible for vehicles to operate without moving through water in an inundated lane. Splash from vehicles traveling in the inundated lane obscures the vision of drivers of vehicles moving at a higher rate of speed in the open lane. Eventually, if width and depth of flow become great enough, the street loses its effectiveness as a traffic-carrier. During these periods, it is imperative that emergency vehicles such as fire trucks, ambulances, and police cars be able to traverse the street by moving along the crown of the roadway.

3.1.2 Interference Due to Ponding

Storm runoff ponded on the street surface because of grade changes or because of crown slope has a substantial effect on the street-carrying capacity. The manner in which ponded water affects traffic is essentially the same as for curb flow; that is, the width of spread into the traffic lane is critical. Ponded water will often completely halt all traffic. Ponding in streets has the added hazard of producing erratic and dangerous driver responses.

3.1.3 Street Cross Flow

Whenever storm runoff, other than limited sheet flow, moves across a traffic lane, a serious and dangerous impediment to traffic flow occurs. Cross flow is only allowed in case of superelevation of a curve or overflow from the higher gutter on a street with cross fall. When street cross flow occurs, the design engineer shall use a concrete valley gutter to convey the runoff across the street. Cross flow from the higher elevation to the lower elevation gutter should be eliminated.

3.1.4 Allowable Flow of Water Through Intersections

As the stormwater flow approaches an arterial or collector, an inlet is required if more than 5 cfs for the 25-year storm enters the intersection. As stormwater flow approaches a residential or marginal access intersection, an inlet is required if the maximum depth of water exceed 6 inches measured from the flowline of the valley gutter. In either situation, the inlet shall not be placed inside a curb return.

3.1.5 Valley Gutter

Concrete valley gutters are essential in diminishing the deterioration of pavements and shall be required at all local street intersections with cross flow regardless of slope. At the intersection of two arterial streets, a valley gutter cannot be used. At the intersection of two collector streets or local streets, a valley gutter shall be installed when cross flow slope occurs across the intersection.

3.2 PERMISSIBLE SPREAD OF WATER

The flow of water in gutters of various streets of different categories shall be limited by the values found on Table 3-1. These clear widths at the crown of the roadway or at the high point on a divided roadway are necessary to provide access for vehicles in the event of an emergency. Equation 3-1 may be used to determine the spread of gutter flow for a specific street width and flow depth.

$$\text{Spread} = W/2\ [(W^2/4)\ 30y_oW^2/(30 + W)]^{1/2} \qquad \text{(Equation 3-1)}$$

where:

W = Street width, feet

y_0 = Water depth in the gutter, feet

Table 3-1
Minimum Clear Widths for Roadway Design Due to Gutter Flow*

Roadway Type	Proposed Usage	Minimum Clear Width (feet)
1. Local Street	a. Residential	0
	b. Commercial/Industrial	0
2. Collector	a. Marginal Access	0
	b. Commercial/Industrial	12
	c. Major 4 Lanes	24
	5 Lanes	24
	4 Lanes Divided	12 (each way)
	6 Lanes Divided	12 (each way)
3. Arterial	a. 4 Lanes, Undivided	24
	b. 3 Lanes, One way	12
	c. 4 Lanes, One way	24
	d. 4 Lanes, with continuous left turn lane	24
	e. 4 Lanes, Divided	12 (each way)
	f. 6 Lanes, Divided	12 (each way)
	g. 8 Lanes, Divided	24 (each way)

3.3 DESIGN METHOD

3.3.1 Gutter Flow Velocities

To ensure scouring velocities for low flows, the gutter shall have a minimum slope of 0.005 feet per foot (0.5%).

3.3.2 Straight Crowns

Flow in gutters on straight crown pavements is normally assumed to be uniform, with Manning's Equation being used to determine the flow. However, because the hydraulic radius assumption in the Manning's Equation is not able to adequately describe the hydraulic characteristics of the gutter cross section, modification of the equation is necessary to accurately compute the flow. The modified Manning's Equation is:

$$Q_o = 0.56(z/n)S_o^{1/2}Y_o^{8/3} \qquad \text{(Equation 3-2)}$$

where:

Q_o = Gutter discharge, cfs

z = Reciprocal of the crown slope, ft/ft

S_o = Street or gutter slope, ft/ft

n = Roughness coefficient

Y_o = Depth of flow in gutter, feet

The nomograph on Figure 3-1 in Appendix A of this Manual provides a direct solution for flow conditions in triangular channels. For a concrete pavement gutter, a Manning's Roughness Coefficient equal to 0.015 is recommended. For gutters with slope less than 1%, a Manning's Roughness Coefficient of 0.02 is recommended to account for sediment accumulation.

3.3.3 Parabolic Crowns

Flows in parabolically crowned pavement are calculated from a variation of Manning's Equation, which assumes steady flow in a prismatic open channel. However, this equation is complicated and difficult to solve for each design case.

To provide a means of determining the flow in the gutter, generalized gutter flow equations for combinations of parabolic crown heights, curb splits, and street grades of different street widths have been prepared. All of these equations have a logarithmic form.

Note: The street width used in this section is measured from face of curb to face of curb.

A. **Streets Without Curb Split.** Curb split is the vertical difference in elevation between curbs at a given street cross section. The gutter flow equation for parabolic crown streets without any curb split is:

$$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 \qquad \text{(Equation 3-3)}$$

where:

Q = Gutter flow, cfs

S_0 = Street grade, ft/ft

y_0 = Water depth in the gutter, feet

K_0, K_1, K_2 = Constant coefficients shown in Table 3-2 for different street widths

Table 3-2
Coefficients for Equation 3-3, Streets without Curb Split

Street Width* (feet)	Coefficients		
	K_0	K_1	K_2
30	2.85	0.50	3.03
36	2.89	0.50	2.99
40	2.85	0.50	2.89
44	2.84	0.50	2.83
48	2.83	0.50	2.78
60	2.85	0.50	2.74

Source: City of Austin, Watershed Engineering Division

*Note: The street width is measured from face of curb to face of curb (FOC-FOC).

B. **Streets with Curb Split – Higher Gutter.** The gutter flow equation for calculating the higher gutter flows is as follows:

$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3(C_S)$ (Equation 3-4)

where:

Q = Gutter flow, cfs

S_0 = Street grade, ft/ft

Y_0 = Water depth in the gutter, feet

C_S = Curb split, feet

K_0, K_1, K_2, K_3 = Constant coefficients shown in Table 3-3 for different street widths

Table 3-3
Coefficients for Equation 3-4, Streets with Curb Split – Higher Gutter

Street Width (feet)	Coefficients				Curb Split Range (feet)
	K_0	K_1	K_2	K_3	
30	2.85	0.50	3.03	-0.131	0.0–0.6
36	2.89	0.50	2.99	-0.140	0.0–0.8
40	2.85	0.50	2.89	-0.084	0.0–0.8
44	2.84	0.50	2.83	-0.091	0.0–0.9
48	2.83	0.50	2.78	-0.095	0.0–1.0
60	2.85	0.50	2.74	-0.043	0.0–1.2

Source: City of Austin, Watershed Engineering Division

C. **Streets with Curb Split – Lower Gutter.** The gutter flow equation for the lower gutter is:

$\log Q = K_0 + K_1 \log S_0 + K_2 \log y_0 + K_3(C_S)$ (Equation 3-5)

where:

Q = Gutter flow, cfs

S_0 = Street grade in ft/ft

y_0 = Water depth in the gutter in feet

C_S = Curb split in feet

K_0, K_1, K_2, K_3 = Constant coefficients shown in Table 3-4 for different street widths

Table 3-4
Coefficients for Equation 3-5, Streets with Curb Split – Lower Gutter

Street Width (feet)	**Coefficients**				**Curb Split Range (feet)**
	K_0	**K_1**	**K_2**	**K_3**	
30	2.70	0.50	2.74	-0.215	0.0-0.6
36	2.74	0.50	2.73	-0.214	0.0-0.8
40	2.75	0.50	2.73	-0.198	0.0-0.8
44	2.76	0.50	2.73	-0.186	0.0-0.9
48	2.77	0.50	2.72	-0.175	0.0-1.0
60	2.80	0.50	2.71	-0.159	0.0-1.2

Source: City of Austin, Watershed Engineering Division

Crown heights for different street widths are calculated by the following equation:

Crown Height (feet) = 0.5 + [(W 30)/120] (Equation 3-6)

where:

W = street width, feet

D. **Parabolic Crown Location.** The gutter flow equation presented for parabolic crowns with split curb heights is based on a procedure for locating the street crown. The procedure allows the street crown to shift from the street center line toward the high one-fourth (¼) percentage point of the street in direct proportion to the amount of curb split. The maximum curb split occurs with the crown at the one-fourth (¼) percentage point of the street. The maximum allowable curb split for a street with parabolic crowns is 0.02 foot per foot of street width.

Example: Determination of Crown Location

Given: 0.4-foot design split on 30-foot-wide street

Maximum curb split = 0.02 x street width = 0.02 x 30 feet = 0.6-foot Maximum

Movement = ¼ street width for 30-foot street = ¼ x 30 feet = 7.5 feet

Split Movement = (Design split x w/Maximum Split x 4)

= (0.4 x 30/0.6 x 4) = 5 feet

Curb splits that are determined by field survey, whether built intentionally or not, should be considered when determining the capacity of the curb flow.

Special consideration should be given when working with cross sections that have the pavement crown above the top of curb. When the crown exceeds the height of the curb the maximum depth of water is equal to the height of the curb, not the crown height. It should be noted that a parabolic section where the crown equals the top of curb will carry more water than a section that has a crown above the top of curb.

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4.0 INLETS

4.1 GENERAL

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

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

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

A. Because grated curb inlets have an increased tendency to clog and are difficult to repair, an open curb inlets shall be used unless prior approval is given by the Public Works Director or his/her designee.

B. Minimum transition for recessed inlets shall be 10 feet.

C. All curb inlets (whether in a sump or on grade) shall incorporate a standard 5-inch depression. Unless otherwise approved in writing by Public Works Director or his/her designee, all curb inlets shall be a minimum of 10 feet in length.

D. When recessed inlets are used, they shall not decrease the width of the sidewalk. Use of recessed inlets must be approved by the Public Works Director or his/her designee for all streets.

E. Design and location of inlets shall take into consideration pedestrians. In addition inlets shall be designed to assure safe passage of bicycles.

F. Inlet design and location must be compatible with the criteria established in Section 3 of this Manual.

G. The use of slotted drains is discouraged except in instances where there is no alternative. If used, the manufacturer's design guidelines shall be followed.

4.2 INLET CLASSIFICATIONS

Inlets are classified into two major groups: (1) inlets in sumps where flow contributes from two or more sides (Type S); and (2) inlets on grade (Type G). The following list references the various inlet types (see figures 4-1 through 4-7 in Appendix A of this Manual for further guidance).

Inlets in Sumps	
(1) Curb Opening	Type S-1
(2) Grate*	Type S-2
(3) Combination (Grate and Curb Opening)*	Type S-3
(4) Area Without Grate	Type S-4
Inlets on Grade	
(1) Curb Opening	Type G-1
(2) Grate*	Type G-2
(3) Combination (Grate and Curb Opening)*	Type G-3

Recessed inlets are identified by the suffix (R), i.e., S-1(R).

* For the flow capacity through the grate inlets, the Engineer should check appropriate vendor catalog.

4.3 STORM INLET HYDRAULICS

4.3.1 Inlets in Sumps

Inlets in sumps are inlets at low points with gutter flow contributing from two or more sides. The capacity of inlets in sumps must be known in order to determine the depth and width of ponding for a given discharge. Sump inlets should be designed using Figure 4-8 in Appendix A of this Manual for an unsubmerged inlet or Figure 4-9 in Appendix A of this Manual for submerged conditions, regardless of what depth of depression exists at the inlet.

A. Curb Opening Inlets (Type S-1) and Area Inlet Without Grate (Type S-4).

   Unsubmerged curb opening inlets (Type S-1) and area inlets without grates (Type S-4) in a sump function as rectangular weirs with a coefficient of discharge of 3.0. Their capacity shall be based on the following equation:

   $Q = 3.0h^{1.5}L$ (Equation 4-1)

   where:

   Q = Capacity of curb opening inlet or of area inlet, cfs

   h = Head at the inlet, feet, $= a + Y_0$

   L = Length of opening through which water enters the inlet, feet

   Figure 4-8 in Appendix A of this Manual provides for direct solution of the above Equation 4-1.

Curb opening inlets and drop inlets in sumps have a tendency to collect debris at their entrances. For this reason, the calculated inlet capacity shall be reduced by 10% to account for clogging.

B. Grate Inlets (Type S-2).

An area inlet with a grate (Type S-2) in a sump functions as an orifice with a coefficient of discharge of 0.60. Therefore, the orifice equation becomes:

$Q = 4.82Ah^{0.5}$ (Equation 4-2)

where:

Q = Capacity, cfs

h = Depth of flow at inlet, feet

A = Area of grate opening, square feet

The curves shown on Figure 4-9 in Appendix A of this Manual provide for direct solution of Equation 4-2.

Area inlets with grates in sumps have a tendency to clog from debris, which becomes trapped by the inlet. For this reason, the calculated inlet capacity of a grated area inlet shall be reduced by 50% to account for clogging. Because clogging necessitates maintenance, grate inlets in sumps are discouraged.

C. Combination Inlets (Type S-3).

The capacity of a combination inlet Type S-3 consisting of a grate and curb opening in a sump shall be considered to be the sum of the capacities obtained from figures 4-8 and 4.9 in Appendix A of this Manual. When the capacity of the gutter is not exceeded, the grate inlet accepts the major portion of the flow.

Combination inlets in sumps have a tendency to clog and collect debris at their entrances. For this reason, the calculated inlet capacities shall be reduced by their respective percentages indicated previously (which are 10% for a curb opening and 50% for grate inlets).

D. Recessed Inlets in Sumps (Type S-1(R), Type S-3(R)).

Recessed inlets can be either curb opening or combination types. The clogging factors shall remain the same for recessed or nonrecessed inlets.

4.3.2 Inlets on grade With Gutter Depression

A. **Curb Opening Inlets on Grade (Type G-1).**

The capacity of a depressed curb inlet should be determined by use of figures 4-10 and 4-11 in Appendix A of this Manual. Because the inlet is on a slope and there is no grate to catch debris, the majority of the debris will be carried downstream; therefore, no reduction for clogging is necessary.

B. **Grate Inlets on Grade (Type G-2).**

The depression of the gutter at a grate inlet decreases the flow past the outside of the grate. The effect is the same as that caused by the depression of a curb inlet.

The bar arrangements for grate inlets greatly affect the efficiency of the inlet. In order to determine the capacity of a grate inlet on grade, the appropriate vendor data must be checked (see Bibliography, Item 4-3 of this Manual).

Grate inlets have a tendency to trap debris such as leaves and litter being carried by the gutter flows. This causes traffic problems from ponding water and requires maintenance. A reduction factor of 30% to account for clogging shall be applied.

C. **Combination Inlets on Grade (Type G-3).**

Combination inlets (curb opening plus grate) have greater hydraulic capacity than curb opening inlets or grate inlets of the same length. Generally speaking, combination inlets are the most efficient of the three types of inlets on grade presented in this Manual. The basic difference between a combination inlet and a grate inlet is that the curb opening receives the carry-over flow that passes between the curb and the grate. The reduction factor for clogging of this type of inlet shall be 0% for the curb opening and 35% for the grate inlet.

D. **Recessed Inlets on Grade (Type G-1R, G-3R).**

Capacities for recessed inlets on grade shall be calculated as 0.75 times the capacity for nonrecessed inlets. The clogging factors shall remain the same for the various types of inlets.

4.3.3 Example 4-1

Given: Parabolic crown street width = 30 feet

Cross Slope = 0 ft/ft

Street Grade = 5%

Q_a in one gutter = 12 cfs

Find: Capacity of a 10-foot curb inlet on grade (Type G-1) with a 5-inch gutter depression

Step 1: From Equation 3-3 (Section 3 of this Manual) depth of flow in gutter is y_0= 0.43 foot, of 5.1 inches

Step 2: Enter Figure 4-10 with y_0 = 0.43 foot and a = 5 inches and find corresponding Q_a/L_a = 0.90

Step 3: Compute L_a = 12/0.90 = 13.33

Step 4: Compute L/L_a = 10/13.33 = 0.75

Step 5: Enter Figure 4-11 (in Appendix A of this Manual) with L/L_a=0.75 and a/y = 0.98 and find corresponding Q/Q_a = 0.84

Step 6: Determine Q from Q/Q_a

Q = 0.84 (12)= 10.1 cfs

Step 7: Determine Q_{pass}

Q_{pass} = 12-10.1 = 1.9 cfs

Step 8: The by-pass flow is 1.90 cfs

4.4 INLET SYSTEM LAYOUT

The following section of this Manual is intended to provide a general step by step procedure for the layout of an inlet system utilizing the information that has been provided in chapters 3 and 4. This information is in no way a requirement for design and is provided solely as an aid for the design Engineer.

4.4.1 Preliminary Design Considerations

A. Prepare a drainage map of the entire area to be served by proposed drainage improvements. Contour maps serve as excellent drainage area maps when supplemented by field observation.

B. Outline the drainage area for each inlet in accordance with present and future street development. Show all existing underground utilities.

C. Make a tentative layout of the proposed storm drainage system, locating all inlets, manholes, mains, laterals, ditches, culverts, etc.

D. Establish the design rainfall frequency.

E. Establish the minimum inlet time of concentration.

F. Establish the typical cross section of each street.

G. Establish the permissible spread of water on all streets within the drainage area.

H. Indicate each drainage area, the size of area, the direction of surface runoff by small arrows, and the coefficient of runoff for the area.

4.4.2 Inlet System Design

Determining the size and location of inlets is largely a trial and error procedure. Based on criteria outlined in chapters 2, 3, and 4 of this Manual, the following steps will serve as a guide to the procedure to be used:

Step 1: Beginning at the upstream end of the project drainage basin, outline a trial subarea and calculate the runoff from it.

Step 2: Compare the calculated runoff to allowable street capacity. If the calculated runoff is greater than the allowable street capacity, reduce the size of the trial subarea. If the calculated runoff is less than street capacity, increase the size of the trial subarea. Repeat this procedure until the calculated runoff equals the allowable street capacity. This is the first point at which a portion of the flow must be removed from the street. The percentage of flow to be removed will depend on street capacities versus runoff entering the street downstream.

Step 3: Record the drainage area, time of concentration, runoff coefficient and calculated runoff for the subarea. This information shall be recorded on the plans or in tabular form similar to that shown in Table 4-1 shown at the end of Section 4.4.3.

Step 4: If an inlet is to be used to remove water from the street, determine and record the inlet size, amount of intercepted flow and amount of flow carried over (bypassing the inlet).

Step 5: Continue the above procedure for other subareas until a complete system of inlets has been established. Account for carry-over from one inlet to the next.

Step 6: After a complete system of inlets has been established, modification should be made to accommodate special situations such as point sources of runoff, and variation of street alignments and grades.

Step 7: Record information as in steps 3 and 4 above for all inlets.

Step 8: After the inlets have been located and sized, the inlet pipes can be designed (see Section 5 of this Manual).

4.4.3 Inlet Flow Calculation Table

An example of a calculation table for inlet flow design is shown in Table 4-1 of this Manual.

The following is an explanation of each column in Table 4-1:

Column 1: Inlet number. All inlets are classified with a designated number.

Column 2: Drainage area number. List all numbers of the drainage areas that drain stormwater into inlet number in Column 1.

Column 3: The corresponding discharge from the drainage areas in Column 2.

Column 4: The carry-over flow (Q_{pass}) in this column is the quantity of water that has passed by the last preceding inlet to the inlet under consideration.

Column 5: The total run-off, Q_a, is the run-off from Column 3 plus the carry-over from preceding drainage areas.

Column 6: The slope, S, expressed in percentage, is obtained from established grade lines as shown on the plan-profile sheets, or from specified data.

Column 7: Gutter depression.

Column 8: The water depth, Y_0, in the gutter is expressed in feet. "Y_0" can be determined from Equation 3-1 or Figure 3-1 (in Appendix A of this Manual) for the straight crown streets and determined from equations 3-3, 3-4, or 3-5 for the parabolic crown streets.

Column 9: The value of the ponded width is the product of the water depth (in Column 7) and the reciprocal of the cross slope (z) in the Equation 3-2. The ponding width must be kept within the maximum permissible ponded limit of the streets.

Column 10: The reduction factor for each inlet as specified in Section 4.3.0 of this Manual.

Column 11: Q_a/L_a is read from Figure 4-10 in Appendix A of this Manual by the gutter depression and gutter flow depth.

Column 12: L_a is calculated from Q_a divided by the value in Column 11. L_a represents the length of an inlet for 100% interception.

Column 13: Length of the inlet L.

Column 14. The ratio of L/L_a.

Column 15: The ratio of gutter depression (in feet) to water depth in the gutter (in feet).

Column 16. The ratio of Q/Q_a. The value is read from Figure 4-11 in Appendix A of this Manual.

Column 17: Q is the flow intercepted by the inlet of length L.

Column 18: The carry-over flow (Q_{pass}) is the result of Q_a-Q.

Column 19: This column is used to specify the inlet information.

Table 4-1 Inlet Flow Calculation Table

INLET FLOW CALCULATION TABLE																		
Inlet Number	Drainage Area No	Q (cfs)	Q Pass (cfs)	Q Total (Qa) (cfs)	Slope (%)	Q (in)	Yo (ft)	Ponded Width (ft)	R.F. (%)	Oa/La	La (ft)	Length (ft)	L/La	a/y_o	Q/Qa	Q (cfs)	Q Pass (cfs)	Remark
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

Source: City of Austin Drainage Criteria Manual

SECTION 5.0 STORM DRAINS

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5.0 STORM DRAINS

5.1 GENERAL

The purpose of this section is to consider the hydraulic aspects of storm drains and their appurtenances in a storm drainage system. Hydraulically, storm drainage systems consist of conduits (open or enclosed) in which unsteady and nonuniform flow will occur. The design storm shall be the 25-year storm with provisions made for the 100-year storm as noted in Chapter 3 of this Manual.

5.2 DESIGN GUIDELINES

The following guidelines shall be observed in the design of storm drain system components to be located in public right-of-way or public drainage easements in order to promote proper operation and to minimize maintenance of those systems:

A. Select pipe size and slope so that the velocity of flow will increase progressively, or at least will not appreciably decrease, at inlets, bends, or other changes in geometry or alignment.

B. A larger pipe shall not discharge into a smaller one even if the capacity of the smaller pipe may be greater due to a steeper slope.

C. No proposed pipe having a diameter greater than 50% of the minimum dimension of an existing box culvert shall be allowed to discharge into that box culvert. Exceptions shall be justified by structural engineering analysis or manufacturer design data.

D. The 25-year hydraulic grade line shall remain at or below the allowable water surface elevation at the inlet throat.

5.3 DESIGN PARAMETERS

5.3.1 Minimum Grades

Storm drains should operate with velocities of flow sufficient to prevent deposition of solid material. The controlling velocity is near the bottom of the conduit and is considerably less than the mean velocity. Storm drains should be designed to have a minimum velocity of 2.5 feet per second (fps).

5.3.2 Maximum Velocities

Maximum velocities in conduits are important because of the possibility of excessive erosion of the storm drain pipe material. Table 5-1 lists the maximum velocities allowed.

Table 5-1
Maximum Velocity in Storm Drains

Type	Maximum Permissible Velocity
Storm Drains (inlet laterals)	No limit
Storm Drains (trunk)	15 fps

5.3.3 Minimum Diameter

Pipes that are to become an integral part of the public storm sewer system shall have a minimum diameter of 18 inches.

5.3.4 Roughness Coefficients

The coefficients of roughness listed in Table 5-2 are for use in Manning's Equation.

Table 5-2
Roughness Coefficients "n" for Storm Drains

Materials of Construction	Minimum Design Coefficient
Concrete	0.015
Corrugated-Metal Pipe	0.024
Plain or Coated Paved Invert (Asphalt)	 0.020
Plastic Pipe Smooth Perforated	 0.010 0.020

5.4 FLOW IN STORM DRAINS

All storm drains shall be designed by the application of the Continuity Equation (Equation 5-1) and Manning's Equation (Equation 5-2) either through the appropriate charts and nomographs, or by direct solution of the equations as follows:

5.4.1 Flow Equation Method

$Q = AV$ and (Equation 5-1)

$Q = (1.49/n)AR^{2/3}S^{1/2}$ (Equation 5-2)

where:

Q = Pipe Flow, cfs

A = Cross-sectional area of flow, ft^2

V = Velocity of flow, ft/s

n = Coefficient of roughness of pipe

R = Hydraulic radius = A/W_p, feet

S = Friction slope in pipe, ft/ft

W_p = Wetted perimeter, feet

5.4.2 Nomograph Method

Nomographs for determining flow properties in circular pipe, elliptical pipe, and pipe-arches are given here as figures 5-1 through 5-9 in Appendix A of this Manual. The nomographs are based upon a value of "n" of 0.012 for concrete. The charts are self-explanatory, and their use is demonstrated by the following examples in this Section.

For values of "n" other than 0.012, the value of Q should be modified by using the following formula:

$$Q_c = 0.012\ Q_n/n_c$$

where:

Q_c = Flow based upon n_c

n_c = Value of "n" other than 0.012

Q_n = Flow from nomograph based on n = 0.012

This formula can be used in two ways. If n_c = 0.015 and Q_c is unknown, use the known values to find Q_n from the nomograph, and then use the formula to convert Q_n to the required Q_c. If Q_c is one of the known values, use the formula to convert Q_c (based on n_c) to Q_n (based on n = 0.012) first, and then use Q_n and the other known values to find the unknown variable on the nomograph.

Example 5-1:

Given: Slope = 0.005 ft/ft

d = depth of flow =1.8 feet

D = diameter = 36 inches

n = 0.018

Find: Discharge (Q)

First determine d/D = 1.8 feet/3.0 feet = 0.6. Then enter Figure 5-1 (in Appendix A of this Manual) to read Q_n = 34 cfs. From the formula, Q_c = 34 (0.012/0.018) = 22.7 cfs.

Example 5-2:

Given: Slope = 0.005 ft/ft

D = diameter = 36 inches

Q = 22.7 cfs

n = 0.018

Find: Velocity of flow (ft/s)

First convert Q_c to Q_n so that nomograph can be used. Using the formula Q_n = 22.7 (0.018)/(0.012) = 34 cfs, enter Figure 5-1 (in Appendix A of this Manual) to determine d/D = 0.6. Now enter Figure 5-3 (in Appendix A of this Manual) to determine V = 7.5 ft/s.

5.5 HYDRAULIC GRADIENT

In storm drain systems flowing full, all losses of energy are a function of resistance of flow in pipes or by interference with flow patterns at junctions. These losses must be accounted for by their accumulation along the system from its tailwater elevation at the outlet to its upstream inlet. The purpose of determining head losses is to include these values in a progressive calculation of the hydraulic gradient. In this way, it is possible to determine the hydraulic gradient line that will exist along the storm drain system. The hydraulic gradient line shall be computed and plotted for all sections of a storm drain system flowing full or under pressure flow. The determination of friction loss and minor loss are important for these calculations.

5.5.1 Friction Losses

Friction loss is the energy required to overcome the roughness of the pipe and is expressed as:

$$h_f = (29n^2/R^{1.33})(V^2/2g)L \qquad \text{(Equation 5-3)}$$

where:

h_f = Friction loss, feet

n = Manning's Coefficient

L = Length of pipe, feet

R = Hydraulic radius, feet

V = Velocity of flow, ft/s

g = Acceleration due to gravity, 32 ft/s^2

In addition to Equation 5-3, Table 5-3 can be used to determine the friction slope and applied in Equation 5-4.

$$h_f = S_f L \quad \text{(Equation 5-4)}$$

where:

h_f = Friction loss, feet

S_f = Friction slope, feet = $(Q/C)_2$

L = Length of pipe, feet

C = Full flow coefficient from Table 5-3

Q = Discharge, cfs

Table 5-3
Full Flow Coefficient Values for Circular Concrete Pipe

D Pipe Diameter (inches)	A Area (square feet)	R Hydraulic Radius (feet)	Value of C* for			
			n = 0.010	n = 0.011	n = 0.012	n = 0.013
8	0.349	0.167	15.8	14.3	13.1	12.1
10	0.545	0.208	28.4	25.8	23.6	21.8
12	0.785	0.250	46.4	42.1	38.6	35.7
15	1.227	0.312	84.1	76.5	70.1	64.7
18	1.767	0.375	137	124	114	105
21	2.405	0.437	206	187	172	158
24	3.142	0.500	294	267	245	226
27	3.976	0.562	402	366	335	310
30	4.909	0.625	533	485	444	410
33	5.940	0.688	686	624	574	530
36	7.069	0.750	867	788	722	666
42	9.621	0.875	1308	1189	1090	1006
54	15.904	1.125	2557	2325	2131	1967
60	19.635	1.250	3385	3077	2821	2604
66	23.758	1.375	4364	3967	3636	3357
72	28.274	1.500	5504	5004	4587	4234
78	33.183	1.625	6815	6195	5679	5242
84	38.485	1.750	8304	7549	6920	6388
90	44.170	1.875	9985	9078	8321	7681
96	50.266	2.000	11850	10780	9878	9119
102	56.745	2.125	13940	12670	11620	10720
108	63.617	2.250	16230	14760	13530	12490
114	70.882	2.375	18750	17040	15620	14420
120	78.540	2.500	21500	19540	17920	16540
126	86.590	2.625	24480	22260	20400	18830
132	95.033	2.750	27720	25200	23100	21330
138	103.870	2.875	31210	28370	26010	24010
144	113.100	3.000	34960	31780	29130	26890

Source: American Concrete Pipe Association, *Concrete Pipe Design Manual.*

* $C = (1.486/n)AR^{0.667}$

Example 5-3:

Given: Discharge Q = 24 cfs, diameter D = 24 inches, the length of pipe L = 300 feet and n = 0.013

Find: The friction loss h_f

First, from Table 5-3 for D = 24 inches and n = 0.013, the full flow coefficient C = 226

Second, the friction slope $S_f = (Q/C)^2 = 0.011$

The friction loss $h_f = S_f L = 3.3$ feet

5.5.2 Minor Losses

From the point at which stormwater enters the drainage system at the inlet until it discharges at the outlet, it encounters a variety of hydraulic structures such as manholes, bends, enlargements, contractions and other transitions. These structures will cause head losses, which are called “minor head losses.”

The minor head losses are generally expressed in a form derived from the Bernoulli and Darcy-Weisbach Equations:

$h = KV^2/2g$ (Equation 5-5)

where:

h = velocity head loss, feet

K = coefficient for head loss

The following are minor head losses of hydraulic structures commonly found in a storm drainage system.

A. **Junction Losses.** Equation 5-6 is used to determine the head loss at a junction of two pipes, with the various conditions of the coefficient Ki given in Table 5-4.

   $h_j = (V_2^2 - K_j V_1^2)/2g$ (Equation 5-6)

   where:

   V_1 = Velocity for inflowing pipe, ft/s

   V_2 = Velocity for outflowing pipe, ft/s

K_j = Junction or structure coefficient of loss

Table 5-4
Junction or Structure Coefficient of Loss

Cases	Reference Figure	Description of Condition	Coefficient K_j
A	5-10	Manhole on Main Line with 45° Branch Lateral	0.50
B	5-10	Manhole on Main Line with 90° Branch Lateral	0.25
C	5-11	45° Wye Connection or cut-in	0.75
D	5-11	Inlet or Manhole at Beginning of Main Line or Lateral	1.25
E	5-11	Conduit on Curves for 90°* Curve radius = diameter Curve radius = (2 to 8) diameter Curve radius = (8 to 20) diameter	 0.50 0.40 0.25
F	5-11	Bends where radius is equal to diameter 90° bend 60° bend 45° bend 22½° bend Manhole on line with 60° Lateral Manhole on line with 22½° Lateral	 0.50 0.43 0.35 0.20 0.35 0.75

Source: *City of Austin Drainage Criteria Manual*, Department of Public Works, Austin, Texas. January 1977.

*Where bends other than 90 degrees are used, the 90 degree bend coefficient can be used with the following percentage factor applied: 60° Bend - 85%; 45° Bend - 70%; 22½° Bend - 40%

The detailed design information for junction losses can be found in Bibliography of this Manual, Item 5-10.

B. **Bend Losses.** The minor head loss at a bend results from a distortion of the velocity distribution, thereby causing additional shear stresses within the fluid. The bend loss is considered to be that in excess of the loss for an equal length of straight pipe. The equation to compute the bend loss is:

$H_b = K_b V^2/2g$ (Equation 5-7)

The coefficient K_b varies with the angle of the bend. Table 5-4 and Figure 5-11 in Appendix A of this Manual show the different K_b coefficients used in bend losses.

Table 5-5
Values of K for Determining Loss of Head Due to Sudden Enlargement in Pipes, from the Formula $H = K (V^2/2g)$

d_2/d_1	Velocity, V, fps										
	2	**3**	**4**	**5**	**6**	**7**	**8**	**10**	**12**	**15**	**20**
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80
10.0	1.00 1.00	.99 1.00	0.96 0.98	0.95 0.96	0.93 0.95	0.92 0.94	0.91 0.93	0.89 0.91	0.88 0.90	0.86 0.88	0.84 0.86

Source: E.F. Brater and H.W. King, *Handbook of Hydraulics*, 1976.

V = velocity in smaller pipe
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe

C. **Transition Losses.** The head losses resulting from sudden and gradual changes in the cross section or flow direction are included in this category. Four transition losses are discussed here.

1. **Sudden Enlargement.** Table 5-5 shows the coefficients used in the different cases for head losses due to a sudden enlargement.

2. **Gradual Enlargement.** Table 5-6 shows the coefficients for calculating the head loss based on the angle of the cone transition.

3. **Sudden Contraction.** Table 5-7 illustrates the values of coefficients in determining the head loss due to a sudden contraction.

4. **Gradual Contraction.** The head losses due to a gradual contraction are determined by the following equation with a constant head loss coefficient.

$h_g = 0.04\ V^2/2g$ (Equation 5-8)

where:

V = velocity for smaller pipe, fps.

Table 5-6
Values of K for Determining Loss of Head Due to Gradual
Enlargement in Pipes from the Formula $H = K (V^2/2g)$

d_2/d_1	Angle of Cone*													
	2°	**4°**	**6°**	**8°**	**10°**	**15°**	**20°**	**25°**	**30°**	**35°**	**40°**	**45°**	**50°**	**60°**
1.1	0.01	0.01	0.01	0.02	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.20	0.21	0.23
1.2	0.02	0.02	0.02	0.03	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.33	0.35	0.37
1.4	0.02	0.03	0.03	0.04	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.47	0.50	0.53
1.6	0.03	0.03	0.04	0.05	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.54	0.57	0.61
1.8	0.03	0.04	0.04	0.05	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.58	0.61	0.65
2.0	0.03	0.04	0.04	0.05	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.60	0.63	0.68
2.5	0.03	0.04	0.04	0.05	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.62	0.65	0.70
3.0	0.03 0.03	0.04 0.04	0.04 0.04	0.05 0.06	0.08 0.08	0.16 0.16	0.31 0.31	0.40 0.40	0.48 0.49	0.55 0.56	0.59 0.60	0.63 0.64	0.66 0.67	0.71 0.72

Source: E.F. Brater and H.W. King. *Handbook of Hydraulics*, 1976.
* Angle of cone is twice the angle between the axis of the cone and its side.
V = velocity in smaller pipe.
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe.

Table 5-7
Values of K for Determining Loss of Head Due to Sudden
Contraction in Pipe From the Formula $H = K (V^2/2g)$

d_2/d_1	Velocity, V in feet per second										
	2	**3**	**4**	**5**	**6**	**7**	**8**	**10**	**12**	**15**	**20**
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.31
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42
10.0	0.49 0.49	0.48 0.49	0.48 0.48	0.48 0.48	0.48 0.48	0.47 0.47	0.47 0.47	0.46 0.47	0.46 0.46	0.45 0.45	0.43 0.44

Source: E.F. Brater and H.W. King, *Handbook of Hydraulics*, 1976.
V = velocity in smaller pipe
d_2/d_1 = ratio of diameter of larger pipe to diameter of smaller pipe

5.5.3 Hydraulic Gradient Calculation Table

After computing the quantity of storm runoff entering each inlet, the storm drain system required to convey the runoff can be designed. The ground line profile is now used in conjunction with the previous runoff calculations. Table 5-8 can be used to keep track of the pipe design and corresponding hydraulic grade line calculations. Note that the computations begin at the downstream discharge point and continue upstream through the pipe system.

The following is an explanation of each of the columns in Table 5-8:

Column 1: Design Point; this point is the first junction point* upstream.

*"Junction Point" refers to any inlet, manhole, bend, etc., that occurs and causes a minor head loss.

Column 2: Junction point immediately downstream of design point.

Column 3: Distance between junction points 1 and 2.

Column 4: Design discharge as determined in inlet calculations (see Table 4-1).

Column 5: Size of pipe chosen to carry an amount equal to or greater than the design discharge (figures 5-12 and 5-15 in Appendix A of this Manual can be used to determine this).

Column 6: Slope of frictional gradient (Table 5-3 using $(Q/C)^2 = S_f$, can be used to determine this).

Column 7: Elevation of hydraulic gradient (h_g) at upstream end of pipe = elevation of downstream end + Column 6 times Column 3, or elevation at upstream end +(d/D) if pipe is not flowing under pressure flow conditions.

Column 8: Elevation of hydraulic gradient at downstream end of pipe (note: At outfall point, assume h_g is at top of pipe or above if actual tailwater elevation exists).

Column 9: Velocity of flow in incoming pipe at design point (use Q = AV for full flow and figures 5-1 and 5-3 in Appendix A of this Manual for partial flow).

Column 10: Velocity of flow in outgoing pipe at design point.

Column 11: Velocity head loss for outgoing pipe at design point.

Column 12: Velocity head loss for incoming pipe at design point.

Column 13: Head loss coefficients at junction (see figures 5-10 and 5-11 in Appendix A of this Manual).

Column 14: Column 12 times Column 13.

Column 15: Column 11 - Column 14 (Note: For bends and inlets or manholes at the beginning of a line, $V_1 = V_2$. The appropriate K_j value should be used in Column 14 and Column 14 = Column 15.)

Column 16: Column 7 + Column 15.

Column 17: Invert elevation at design point for incoming pipe.

Column 18: Invert elevation at design point for outgoing pipe.

Table 5-8
Hydraulic Computations Storm Sewers

Manholes/ Inlets		Dist.	Discharge	Pipe Size	Friction Grade	Hydra. Gradient Up Stream	Down Stream	V_1 Inflow	V_2 Outflow	$\frac{V_2^2}{2g}$	$\frac{V_1^2}{2g}$	Kf	$\frac{Kf\,V_1^2}{2g}$	h	HGL Design PT.	Inv. In	Inv. Out
from	to	feet	cfs	in	ft/ft	Elev.	Elev.	fps	fps	ft	ft	Constant	ft	ft.	Elev.	Elev.	Elev.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18

5.6 MANHOLES

Manholes provide a very important access point for maintenance purposes. Due to equipment restraints, every point within the storm drain must be a maximum of 500 feet from an access point for drains. Inlets and storm drain outfalls may be considered as access points for maintenance purposes. Access points must be accessible in accordance with the requirements of Section 1.2.6D of this Manual and must provide a maintenance path within the storm drain that has no more than one horizontal bend, with that bend having a deflection of no more than 45 degrees in the direction of the maintenance path, and no vertical bend with a deflection of greater than 5 degrees. Storm drain slope adjustments of less than 5 degrees are not subject to this requirement.

Manholes shall also be located where two or more laterals intersect the main line within 5 feet of each other (see Figure 5-12 in Appendix A of this Manual for examples of possible manhole locations).

5.7 DEPTH OF COVER

Design of storm drains for areas that will or could receive vehicular traffic or that will be subject to other loading, the design Engineer shall use higher strength pipes sufficient for the loading conditions.

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6.0 OPEN CHANNELS

6.1 GENERAL

Open channels for use in a major drainage system have significant advantages related to cost, capacity, multiple use for recreational and aesthetic purposes, and potential for detention storage. Disadvantages include right-of-way needs and maintenance costs. Careful planning and design are needed to minimize the disadvantages and to increase the benefits.

The general classifications for open channels are (1) natural channels, which include all watercourses that have been carved by nature through erosion; and (2) new or altered channels, which are constructed or existing channels that have been significantly altered by human effort. New or altered channels can be lined with grass, concrete, mortared rocks, or other materials. The channels should be designed for the 25-year storm with provisions for the 100-year storm within dedicated easements or right-of-way.

6.1.1 Natural Channels

The ideal natural channel has the following benefits:

A. Flow velocities are usually low, resulting in longer concentration times and lower downstream peak flows

B. The channel stabilized to reduce maintenance

C. The channel provides desirable green space and recreational or aesthetic amenities

6.1.2 New or Altered Channels

Grass-lined channels are the most desirable of the various types of new channels for the following reasons:

A. Properly selected and cultivated grass will stabilize the body of the channel.

B. The grass consolidates the soil mass of the bed.

C. The grass controls the movement of soil particles along the channel bottom.

Concrete-lined channels are designed to protect the channel body from the erosive potential of high velocities. In addition to concrete-lined channels, other methods to combat erosive velocities in channels may be available and should be thoroughly analyzed before submittal to the Public Works Director or his/her designee for design consideration.

6.1.3 Environmental Permits

When a project to modify a natural channel is proposed, the design Engineer must check the requirements of Clear Water Act Section 404, Permits for Dredged or Fill Material. If required, a permit shall be obtained from the U.S. Army Corps of Engineers by the design Engineer. In addition, the City requires the design Engineer to follow City floodplain development permit requirements as described in Chapter 1.2.9 of this manual.

6.2 OPEN CHANNEL HYDRAULICS

An open channel is a conduit in which water flows with a free surface. The classification of open channel flow is made according to the change in flow depth with respect to time and space.

Flow in an open channel is said to be “steady” if the depth of flow does not change over time or if it can be assumed to be constant during the time interval under consideration. The flow is “unsteady” if the depth changes with time.

Open channel flow is said to be “uniform” if the depth of flow is the same at every section of the channel under consideration. A uniform flow may theoretically be steady or unsteady, depending on whether or not the depth changes with time. The establishment of unsteady uniform flow requires that the water surface fluctuate with time while remaining parallel to the channel bottom. Since it is impossible for this condition to occur within a channel, steady uniform flow is the fundamental type of flow treated in open channel hydraulics.

Flow is “varied” if the depth of flow changes along the length of the channel. Varied flow may be either steady or unsteady. Since unsteady uniform flow is rare, the term “unsteady flow” is used to designate unsteady varied flow exclusively.

Varied flow may be further classified as either “rapidly” or “gradually” varied. The flow is rapidly varied if the depth changes abruptly over a comparatively short distance; otherwise, it is gradually varied. Rapidly varied flow is also known as a local phenomenon; an example of which is the hydraulic jump.

With these varying conditions, open channel hydraulics can be very complex, encompassing many different flow conditions from steady uniform flow to unsteady rapidly varied flow. Most of the problems in stormwater drainage involve uniform, gradually varied or rapidly varied flow situations. In this section, the basic equation and computational procedures for uniform, critical, gradually varied and rapidly varied flows are presented.

6.2.1 Uniform Flow

For a given channel condition of roughness, discharge, and slope, there is only one possible depth for maintaining a uniform flow. This depth is referred to as normal depth.

The Manning's Equation is used to determine the normal depth for a given discharge.

$$Q = (1.49/n)AR^{2/3}S^{1/2} \quad \text{(Equation 6-1)}$$

where:

Q = Total discharge, cfs

n = Roughness coefficient

A = Cross-sectional area of channel, ft^2

R = Hydraulic radius of channel, feet (R = A/P)

S = Slope of the frictional gradient, ft/ft

P = Wetted perimeter, feet

Uniform flow is more often a theoretical abstraction than an actuality. True uniform flow is difficult to find in nature or to obtain in the laboratory. The Engineer must be aware of the fact that uniform flow computations provide only an approximation of what will occur, but that such computations are usually adequate and useful and, therefore, necessary for planning.

The computation of normal depth for trapezoidal sections can be performed by using Figure 6-1 in Appendix A of this Manual.

6.2.2 Critical Flow

Flowing water contains potential and kinetic energy. The relative values of the potential and kinetic energy are important in the analysis of open channel flow. The potential energy is represented by the depth of water plus the elevation of the channel bottom above a datum. The kinetic energy is represented by the velocity head, $V^2/2g$. The specific energy or specific head is equal to the depth of water plus the velocity head.

$$H = d + (V^2/2g) \quad \text{(Equation 6.2)}$$

where:

H = specific energy head, ft

d = depth of flow, ft

V = average channel flow velocity, ft/s

g = acceleration of gravity

When depth of flow is plotted against specific energy for a given channel discharge at a section, the resulting curve shows that, at a given specific energy, there are two possible flow depths. At minimum energy, only one depth of flow exists. This is known as the critical depth. At critical depth, the following relationship applies for rectangular sections:

$dc = V^2/g$ (Equation 6.3)

where:

dc = critical depth, ft

V = average channel flow velocity, ft/s

g = acceleration of gravity

The effect of gravity upon the state of flow is represented by a ratio of the inertial forces to gravity forces. This ratio is known as the Froude Number, Fr, and is used to categorize the flow. The Froude Number is defined by Equation 6.4 for a rectangular section.

$Fr = V/(gd)^{0.5}$ (Equation 6.4)

where:

Fr = Froude Number

V = average channel flow velocity, ft per second

g = acceleration of gravity

d = depth of flow, ft

The critical state of flow through a rectangular channel is characterized by several important conditions:

A. The specific energy is a minimum for a given discharge.

B. The discharge is a maximum for a given specific energy.

C. The specific force is a minimum for a given discharge.

D. The velocity head is equal to half the hydraulic depth in a channel of small slope.

E. The Froude Number is equal to 1.0.

If the critical state of flow exists throughout an entire reach, the channel flow is critical and the channel slope is at critical slope, S_c. A flow at or near the critical state is unstable, because minor changes in specific energy, such as from channel debris, will cause a major change in depth.

In the analysis of nonrectangular channels, the Froude Number equation is rewritten. The depth of flow is defined as the cross sectional area divided by the top width.

$$Fr = [Q^2B/gA^3]^{0.5} \qquad \text{(Equation 6.5)}$$

where:

Fr = Froude Number

Q = discharge in channel, cfs

B = top width of channel, ft

g = acceleration of gravity

A = cross sectional area, square feet

It can be shown that Fr = 1 for critical flow. If the Froude Number is greater than 1, the flow is supercritical, but when the Froude Number is less than 1, the flow is subcritical.

6.2.3 Gradually Varied Flow

The most common example of gradually varied flow in urban drainage systems occurs in the backwater of bridge openings, culverts, storm sewer inlets, and channel constrictions. Under these conditions, gradually varied flow will be created and the flow depth will be greater than normal depth in the channel. Backwater techniques would need to be applied to determine the water surface profile.

Calculations of water surface profiles can be accomplished by using standard backwater methods or acceptable computer routines, which take into consideration all losses due to changes in velocity, drops, bridge openings and other obstructions in open channels.

There are several acceptable methods for backwater calculations. The most common hand calculation method for prismatic channels and irregular-uniform channels is the Standard Step Method. The most widely used backwater analysis computer program is HEC-RAS, developed by the U.S. Army Corps of Engineers. This program can compute water surface profiles for natural and new channels.

6.2.4 Rapidly Varied Flow

Rapidly varied flow is characterized by abrupt changes in the water surface elevation for a constant flow. The change in elevation may become so abrupt that the flow profile is virtually broken, resulting in a state of high turbulence. Some common causes of rapidly varied flow in urban drainage systems are side-spill weirs, weirs, and spillways of detention basins.

6.3 MANNING'S ROUGHNESS COEFFICIENTS

6.3.1 Existing and Natural Channels

Because several primary factors affect the roughness coefficient, a procedure has been developed to estimate this value, n. By this procedure, the value of n may be computed by:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m \qquad \text{(Equation 6-6)}$$

where n_0 is a basic n value for a straight, uniform, smooth channel in the natural materials involved, n_1 is a value added to n_0 to correct for the effect of surface irregularities; n_2 is a value for variations in shape and size of the channel cross section; n_3 is a value for obstructions; n_4 is a value for vegetation and flow conditions; and m is a correction factor for meandering of the channel. Proper values of n_0 to n_4 and m may be selected from Table 6-1 according to the given conditions.

In selecting the value of n_1, the degree of irregularity is considered smooth for surfaces comparable to the best attainable for the materials involved; minor for good dredged channels, slightly eroded or scoured side slopes of canals or drainage channels; moderate for fair to poor dredged channels, moderately sloughed or eroded side slopes of canals or drainage channels; and severe for badly sloughed banks of natural streams, badly eroded or sloughed sides of canals or drainage channels, and unshaped, jagged and irregular surfaces of channels excavated in rock.

In selecting the value of n_2, the character of variations in size and shape of cross section is considered gradual when the change in size or shape occurs gradually; alternating occasionally when large and small sections alternate occasionally or when shape changes cause occasional shifting of main flow from side to side; and alternating frequently when large and small sections alternate frequently or when shape changes cause frequent shifting of main flow from side to side.

The selection of the value of n_3 is based on the presence and characteristics of obstructions such as debris deposits, stumps, exposed roots, boulders, and fallen and lodged logs. One should recall that conditions considered in other steps must not be re-evaluated or double-counted in this selection. In judging the relative effect of obstructions, consider the following: the extent to which the obstructions occupy or reduce the average water area, the obstruction characteristics (sharp-edged or angular objects induce greater turbulence than curved, smooth-surfaced objects), and the position and spacing of obstructions transversely and longitudinally in the reach under consideration.

Table 6-1
Computation of Composite Roughness Coefficient for Excavated and Natural Channels
$n = (n_0 + n_1 + n_2 + n_3 + n_4)m$

Channel Conditions		**Values**
n_0 Material Involved	Earth React Fine Gravel Coarse Gravel	0.020 0.025 0.024 0.028
n_1 Degree of Irregularity	Smooth Minor Moderate Severe	0.000 0.005 0.010 0.020
n_2 Relative Effect of Channel Cross Section	Gradual Alternating Occasionally Alternating Frequently	0.000 0.005 0.013
n_3 Relative Effect of Obstructions	Negligible Minor Appreciable Severe	0.000 0.013 0.025 0.050
n_4 Vegetation	Low Medium High Very High	0.008 0.018 0.038 0.075
m Degree of Meandering	Minor Appreciable Severe	1.000 1.150 1.300

Source: V.T. Chow, *Open Channel Hydraulics*, 1959.

In selecting the value of n4, the degree of effect of vegetation is considered in the following way:

A. **Low** for conditions comparable to the following: (a) dense growths of flexible turf grasses or weeds, of which Bermuda and blue grasses are examples, where the average depth of flow is two to three times the height of vegetation, and (b) supple seedling tree switches, such as willow, cottonwood or salt cedar where the average depth of flow is three to four times the height of the vegetation.

B. **Medium** for conditions comparable to the following: (a) turf grasses where the average depth of flow is one to two times the height of vegetation, and (b) stemmy grasses, weeds or tree seedlings with moderate cover where the average depth of flow is two to three times the height of vegetation and brush growths, moderately dense, similar to willows 1 to 2 years old, dormant season, alongside slopes of a channel with no significant vegetation along the channel bottom, where the hydraulic radius is greater than 2 feet.

C. **High** for conditions comparable to the following: (a) turf grasses where the average depth of flow is about equal to the height of vegetation, (b) dormant season – willow or cottonwood trees 8 to 10 years old, intergrown with some weeds and brush, where none of the vegetation is in foliage, where the hydraulic radius is greater than 2 feet, and (c) growing season — bushy willows about 1 year old intergrown with some weeds in full foliage alongside slopes, no significant vegetation along channel bottom, where hydraulic radius is greater than 2 feet.

D. **Very high** for conditions comparable to the following: (a) turf grasses where the average depth of flow is less than ½ the height of vegetation, (b) growing season — bushy willows about 1 year old, intergrown with weeds in full foliage alongside slopes, or dense growth of cattails along channel bottom, with any value of hydraulic radius up to 10 or 15 feet, and (c) growing season-trees intergrown with weeds and brush, all in full foliage, with any value of hydraulic radius up to 10 or 15 feet.

In selecting the value of m, the degree of meandering depends on the ratio of the meander length to the straight length of the channel reach. The meandering is considered minor for ratios of 1.0 to 1.2, appreciable for ratios of 1.2 to 1.5, and severe for ratios of 1.5 and greater.

In applying the above method for determining the n value, several things should be noted. The method does not consider the effect of suspended and bed loads. The values given in Table 6-1 were developed from a study of some 40 to 50 cases of small and moderate channels. Therefore, the method is questionable when applied to large channels whose hydraulic radii exceed 15 feet. The method applies only to unlined natural streams, floodways, and drainage channels and shows a minimum value of 0.02 for the n value of such channels. The minimum value of n in general, however, may be as low as 0.012 in lined channels and as low as 0.008 in artificial laboratory flumes.

6.3.2 New or Altered Channels

The Manning's Roughness Coefficients (n) for new or altered channels are shown in Table 6-2. This table does not encompass roughness coefficients for all situations and it is recommended to use other hydraulic references to determine appropriate Manning's Roughness Coefficients such as the HEC-RAS Hydraulic Reference Manual.

Table 6-2
Minimum Roughness Coefficients of New or Altered Channels

Type of Channel and Description	Manning's Coefficients
1. Grass lined	
a. Bermuda (with regular mowing)	.040
b. St. Augustine (with regular mowing)	.045
c. Native grasses and vegetation not mowed regularly	.060
2. Concrete	
a. Concrete lined (rough finish)	.020
b. Concrete lined (smooth finish-culverts)	.015
c. Concrete rip-rap (exposed rubble)	.025
3. Gabion	.035
4. Rock-cut	.025

Sources: V.T. Chow, *Open Channel Hydraulics*. 1959; WRC Engineering, Inc., *Boulder County Storm Drainage Criteria Manual*. 1984.

6.4 DESIGN REQUIREMENTS

Channel design involves the determination of the channel cross-section required to accommodate a given design discharge. The design requirements for open channels are discussed in the sections below and apply to channels or waterways that are proposed to be modified or constructed.

6.4.1 Grass-Lined Channels and Waterways

Key parameters in grass-lined channel or waterway design include permissible velocity, roughness coefficient, side slope, curvature, bottom width, and freeboard. The grass species selected shall be suitable for permanent application based upon the anticipated operation and maintenance of the channel or waterway.

A. **Velocity.** The maximum permissible velocity for the 100-year storm is 6 feet per second and includes all transitions to or from channels and waterways with similar or different materials. In all cases, the velocity for the 100-year storm must be nonerosive. The minimum permissible velocity for the 2-year storm is 2 feet per second.

B. **Roughness Coefficient.** The roughness coefficients selected shall be based on the degree of retardance of vegetation. Table 6-2 provides minimum Manning's Coefficients for channel design. The roughness coefficient shall be adjusted to reflect the relationship between the depth of flow and the typical height of the design vegetation, especially for shallow depths of flow, as well as other factors affecting channel conveyance.

C. **Slope.** A reinforced concrete pilot channel may be required by the Director of Public Works or his/her designee where erosion is a concern and the channel slope is less than 1%. A natural unmodified watercourse within a special flood hazard area will not require a pilot channel.

D. **Side Slopes.** Side slopes shall be 4 to 1 or flatter for channels equal to or over 4 feet deep and 3 to 1 or flatter for channels less than 4 feet deep.

E. **Curvature.** The center line curvature shall have a minimum radius of twice the top width of the 100-year storm flow.

F. **Bottom Width.** The minimum flat bottom width of the channel is 4 feet.

G. **Freeboard.** All grass-lined channels shall be designed to convey the 100-year storm event. The freeboard for the channel shall be the velocity head for the 100-year storm.

6.4.2 Concrete-Lined Channels

Concrete-lined channels may be needed in channel reaches where the velocities are excessive (see Section 6.4.1A of this Manual) or where the channel characteristics require such use.

A. **Velocity.** In concrete-lined channels, the probability of achieving supercritical flow is greatly increased. The designer must take care to insure against the possibility of unanticipated hydraulic jumps forming in the channel in considering the 25- and 100-year storms. Flow with a Froude number equal to 1 is unstable and should be avoided. If supercritical flow does occur, then freeboard and superelevation must be determined. In addition, all channels carrying supercritical flow shall be continuously lined with reinforced concrete.

B. **Roughness Coefficient.** Table 6-2 provides the Manning's Coefficients for concrete-lined channels.

C. **Freeboard.** Adequate channel freeboard shall be provided for the 100-year storm in reaches flowing at critical depth by Equation 6-7 or using the energy grade line, whichever is less.

$H_{FB} = 1.0 + 0.025V (d)^{1/3}$ (Equation 6-7)

where:

H_{FB} = Freeboard height, feet

V = Velocity, ft/s

d = Depth of flow, feet

Freeboard shall be in addition to superelevation, standing waves, and/or other water surface disturbances. Concrete side slopes shall be extended to provide freeboard. Freeboard shall not be obtained by the construction of levees.

D. **Superelevation.** Superelevation of the water surface shall be determined at all horizontal curves, which deviate more than 45 degrees off the projected centerline. An approximation of the superelevation at a channel bend can be obtained from the following equation:

$h = V^2 T_w / g r_c$ (Equation 6-8)

where

h = Superelevation, feet

V = Flow velocity, ft/s

T_w = Top width of channel, feet

r_c = Centerline radius of curvature, feet

g = Acceleration due to gravity, 32.2 ft/s^2

The freeboard shall be measured above the superelevation water surface.

E. **Side Slopes.** Because concrete lined channels do not require slope maintenance, the side slopes may be as steep as vertical with appropriate structural methods applied.

F. **Slope.** The flow line slope of the channel shall be no less than 0.5% and must also be sufficient to produce a velocity for the 2-year storm flow of at least 2 feet per second. Compliance with this requirement must take into account the variation in channel flow due to distributed inflows to the channel. An alternative design may be submitted to the Director of Public Works or his/her designee if existing topography is not conducive to this slope requirement.

6.4.3 Other Channels

Channels composed of materials other than vegetation or concrete shall be designed so that sediment deposition does not occur for the 2-year storm (except for channel drop structures and energy dissipaters) and velocities for the 100-year storm are not erosive, as approved by the Public Works Director or his/her designee.

6.5 CHANNEL DROP STRUCTURES

The function of a drop structure is to reduce channel velocities by allowing for flatter upstream and downstream channel slopes. Two commonly used drop structures are shown on Figure 6-2 in Appendix A of this Manual.

The flow velocities in the upstream and downstream channels of the drop structure must satisfy the permissible velocities allowed for channels. The design parameters for the sloping channel drop and the vertical channel drop are defined in this section. Further design methodology parameter determination for channel drop structures can be found in the Bureau of Reclamation, Hydraulic Design of Stilling Basins and Energy Dissapators.

6.5.1 Sloping Channel Drop

A. **Approach Apron.** A riprap apron shall be constructed immediately upstream of the drop to protect against the increasing velocities and turbulence, which result as the water approaches the sloping portion of the drop structure. The same riprap and bedding design shall be used as specified for the portion of the drop structure immediately downstream of the drop.

B. **Chute.** The chute shall have roughened faces and shall be no steeper than 2:1 (H:V). The length, L, of the chute depends upon the hydraulic characteristics of the channel and drop. For a unit discharge, q, of 30 cfs per foot, L would be about 15 feet, that is, about ½ of the q value. The length should not be less than 10 feet, even for low discharge values.

C. **Downstream Apron.** The length of the downstream apron shall be sized according to Table 6-3 and shall be constructed of reinforced concrete or riprap depending on structural requirements.

Table 6-3
Length of Downstream Apron

Maximum Unit Discharge, q (cfs/ft)	Length of Downstream Apron, L_B (feet)
0–14	10
15	15
20	20
25	20
30	25

Source: City of Austin, Watershed Engineering Division.

6.5.2 Vertical Channel Drops

The design criteria for the vertical channel drop are based upon the height of the drop, and the normal depth and velocity of the channel flow at the approach and exit. The channel shall be prismatic throughout, from the upstream channel through the drop to the downstream channel.

A riprap stilling basin is required below any drop structure. The steepest allowable side slope for the riprap stilling basin is 4:1. The riprap shall extend up the side slopes to a depth equal to 1 foot above the normal depth projected upstream from the downstream channel. The maximum fall allowed at any one drop structure is 4 feet from the upper channel bottom to the lower channel bottom.

A description of the drop structure and the design procedure, going from upstream to downstream, follows and is shown on Figure 6-2 in Appendix A of this Manual.

A. **Approach Channel**: The upstream and downstream channels will normally be grass-lined trapezoidal channels.

B. **Approach Apron:** A riprap apron shall be provided upstream of the drop to protect against the increasing velocities and turbulence, which result as the water approaches the vertical drop.

C. **Chute Apron:** The riprap stilling basin shall be designed to force the hydraulic jump to occur within the basin and designed for essentially zero scour.

6.6 ENERGY DISSIPATORS

Energy dissipators are used to dissipate excessive kinetic energy in flowing water that could promotes erosion. An effective energy dissipator must be able to retard the flow of fast moving water without damage to the structure or to the channel below the structure.

Impact-type energy dissipators direct the water into an obstruction that diverts the flow in many directions and in this manner dissipates the energy in the flow. Baffled outlets and baffled aprons are two types of impact-type energy dissipators.

Other energy dissipators use the hydraulic jump to dissipate the excess head. In this type of structure, water flowing at a higher than critical velocity is forced into a hydraulic jump, and energy is dissipated in the resulting turbulence. Stilling basins exemplify this type of dissipator, where energy is diffused as flow plunges into a pool of water.

Generally, an impact-type of energy dissipator is considered to be more efficient than a hydraulic jump-type. Also, the impact-type energy dissipator results in smaller structures.

The design of energy dissipators is based on the empirical data resulting from a comprehensive series of model structure studies by the U.S. Bureau of Reclamation, as detailed in its book *Hydraulic Design of Stilling Basins and Energy Dissipators, 1984.* Two impact-type energy dissipators are briefly explained here.

6.6.1 Baffled Apron (U.S. Bureau of Reclamation Type IX)

Baffled aprons are used to dissipate the energy in the flow at a drop. They require no initial tailwater to be effective, although channel bed scour is not as deep and is less extensive when the discharges in to a tailwater pool. The simplified hydraulic design of the baffled apron is shown on Figure 6-3 in Appendix A of this Manual. The chutes are constructed on a slope that is 2:1 (H:V) or flatter and extends below the channel bottom. Backfill is placed over one or more bottom rows of baffles to restore the original streambed elevation. When scour or downstream channel degradation occurs, successive rows of baffle piers are exposed to prevent excessive acceleration of the flow entering the channel. If degradation does not occur, the scour creates a stilling pool at the downstream end of the chute, stabilizing the scour pattern.

The general rules of hydraulic design of a baffled apron are as follows:

A. **Design Discharge.** The chute should be designed for the full capacity expected to be passed through the structure. The maximum unit discharge may be as high as 60 cfs per foot for the 100-year storm.

B. **Chute Entrance.** The flow entering into the chute shall be well distributed laterally across the width of the chute. The velocity should be well below the critical velocity, preferably the value shown in the curve D of Figure 6-3 in Appendix A of this Manual. The curve C on Figure 6-3 in Appendix A of this Manual is the critical velocity in a rectangular channel, $V_c = (gq)^{1/3}$.

C. **Chute Design.** The upstream end of the chute floor shall be joined to the horizontal floor by a curve to prevent excessive vertical contraction of the flow. The upstream face of the first row should be no more than 1 foot (vertically) below the high point of the chute.

Based on the results of U.S. Bureau of Reclamation experiments, the greatest tendency to overtop the training walls occurs in the vicinity of the second and third rows of baffles. To prevent this overtopping, a partial baffle (1/3 to 2/3 of the width of a full baffle) must be placed against the training walls in the first row. This will place a space of the same width adjacent to the walls in the second row. Alternate rows are then made identical (i.e., rows 1, 3, 5, 7, etc., are identical; rows 2, 4, 6, 8, etc., are identical). Four rows of baffles are necessary to establish the expected flow pattern at the base of the chute.

The height of the training walls on the chute shall be three or more times the baffle height, measured normal to the chute floor. Several rows of baffle piers are usually constructed below the channel grade to establish full control of the flow. At least one row of baffles should be buried in the backfill, which is used to restore the original bottom topography.

D. **Heights and Spacing of Baffle Pier.** Baffle pier height, H, should be about 0.8 D_c to 0.9 D_c, as shown in Curve B on Figure 6-3 in Appendix A of this Manual. D_c is the critical depth in a rectangular channel and determined by:

$$D_c = (q^2/g)^{1/3} \qquad \text{(Equation 6-9)}$$

Baffle pier widths and spaces should be equal and may be, up to 1.5 H, but no less than H. The slope distance between rows of baffle piers should be 2H.

6.6.2 Baffled Outlet

Baffled outlets are used to dissipate the discharge energy from flow in a pipe. They are normally used at outlets from detention ponds or storm drainage systems. The baffles are intended to decrease the discharge velocities and subsequent erosion of the receiving system.

6.7 STRUCTURE AESTHETICS

The design of hydraulic structures in the urban environment requires an approach not encountered elsewhere because appearance must be an integral part of the design. The treatment of the exterior appearance should not be considered of minor importance.

Parks. Hydraulic structures should not detract from the functionality or safety of an urban park. Furthermore, parks and green belts may later be developed in an urban area in which the structure will play a dominant environmental role.

Play Areas. An important consideration is that drainage structures often are an attraction for children. It is almost impossible to make drainage infrastructure inaccessible to children, and therefore what is constructed should be made as safe as is reasonably possible.

Concrete Surface Treatment. The use of textured and colored concrete presents a pleasing appearance and hides form marks. Exposed aggregate concrete is also attractive, but may require special control of the aggregate used in the concrete. Exposed aggregate concrete structures shall be constructed only with prior approval of the Director of Public Work or his/her designee.

Rails and Fences. The use of rails and fences along concrete walls provides a pleasing topping to an otherwise stark wall, yet provides a safety measure against the hazard of falling from an unprotected wall. Rails and fences shall be constructed as required and in accordance with TAS/ADA regulations.

6.8 SUPPLEMENTAL SECTION

6.8.1 Alternative New Channel Design

The following is a description of the cross-sectional characteristics of an alternative channel design that may be applied at the Engineer's discretion, but is in no way a requirement.

A. A pilot channel designed to carry the 10-year storm shall be calculated with Manning's "n" values in accordance with tables 6-2 and 6-3. This channel is designed to separate the more frequent 10-year storm via an unobstructed pilot channel. Side slopes of the pilot channel shall not exceed 3:1 slope gradient and shall have a bottom width of no less than 6 feet. The remaining cross-sectional area is designed to convey the additional storm flows up to the 100-year storm. This upper platform will accommodate vegetation with minimal maintenance requirements.

B. The existing 100-year channel discharge shall be contained within overbanks on each side of the pilot channel. These overbanks shall be a minimum width of ten (10) feet and have a slope gradient not to exceed 6:1. The overbanks shall be stabilized with native grasses, wildflowers, and woody species appropriate to the riparian habitat. Biodegradable matting shall be applied to encourage revegetation and provide erosion control until such time as the vegetation is fully established. In calculating Manning's "n" values for the overbanks, reference must be made to tables 6-2 and 6-3 with the following assumptions:

1. Heavily wooded and brushy overbanks; and

2. Bank irregularities that will form from occasional, moderate erosion.

Figure 6-4 in Appendix A of this Manual depicts the conceptual idea of this alternative channel design.

SECTION 7.0 CULVERTS/BRIDGES

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7.0 CULVERTS/BRIDGES

7.1 GENERAL

The function of a drainage culvert is to pass the design storm flow without causing excessive backwater or overtopping of the structure and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within allowable limits when selecting a structure that will meet these requirements. The design storm flow shall be determined by the hydrologic methods as set forth in Section 2 of this Manual. The system shall accommodate the runoff from a 100-year frequency storm meeting the limitations for overflows at bridges and culverts set forth in sections 1.2.6C and 1.2.6D of this Manual.

7.2 CULVERT HEADWALLS

7.2.1 General

The normal functions of properly designed headwalls and endwalls are to anchor the culvert in order to prevent movement due to hydraulic and soil pressures, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into the waterway. All headwalls shall be constructed of reinforced concrete and may be either straight-parallel, flared, or warped. They may or may not require aprons, as determined by site conditions. Headwalls should be aligned with the direction of the receiving flow when discharging into a waterway. Precast headwalls and endwalls may be used if all other criteria are satisfied; generally precast headwalls/endwalls are available for smaller culverts (i.e., 18 and 24 inches diameter).

7.2.2 Conditions at Entrance

The hydraulic characteristics of a culvert may be completely changed by the shape or condition at the inlet or entrance. Therefore, design of culverts must involve consideration of energy head losses that may occur at the entrance. Entrance head losses may be determined by the following equation:

$$h_e = K_e(V_2^2 - V_1^2)/2g \qquad \text{(Equation 7-1)}$$

where:

h_e = Entrance head loss, feet

V_2 = Velocity of flow in culvert, ft/s

V_1 = Velocity of flow approaching culvert, ft/s

K_e = Entrance loss coefficient as shown in Table 7-1

g = Acceleration due to gravity

Table 7-1
Values of Culvert Entrance Loss Coefficients

Type of Entrance	Entrance Coefficient, K_e
Pipe	
Headwall (no wingwalls)	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and HDPE)	0.40
Headwall with 45° Wingwalls	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance (no headwall or wingwalls)	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin walls (HDPE)	0.90
Sloping Entrance (no headwall or wingwalls)	
Mitered to conform to slope	0.70
Flared-end section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on sides of opening	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel axis	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of culvert walls)	
Square edged at crown	0.70

Source: WRC Engineering, Inc. Boulder County Storm Drainage Criteria Manual, 1984.

RCP: Reinforced Concrete Pipe

HDPE: High Density Polyethylene
NOTE: The entrance loss coefficients are used to evaluate the culvert or sewer capacity operating under outlet control.

7.2.3 Type of Headwall

The common types of headwall entrances are shown on Figure 7-1 in Appendix A of this Manual, but are not limited to the designs shown there. The following guidelines may be used in the selection of the type of headwall. Approach velocities are measured immediately upstream of the headwall under normal operating conditions.

A. **Parallel Headwall.**

   1. Approach velocities are low (below 6 feet per second).

   2. Backwater pools are permitted.

B. **Flared Headwall.**

   1. Approach velocities are between 6 and 10 ft/s.

   2. Ample right-of-way or easement is available.

   The wings of flared walls must be located with respect to the direction of the approaching flow, not the culvert axis as on Figure 7-1 in Appendix A of this Manual.

C. **Warped Headwall.** Approach velocities are between 8 and 20 feet per second. Warped headwalls are effective with aprons to accelerate flow through the culvert.

7.2.4 Debris Fins

For conditions where a multiple box culvert is required, the upstream face of the structure shall incorporate debris deflector fins to prevent debris buildup. For multiple-pipe culverts, installations of debris fins is optional.

The debris fin is an extension of the interior walls of a multiple-box culvert. The fin thickness shall be designed to satisfy structural requirements and reduce impact and turbulence to the flow.

A debris fin is always constructed to the height of the culvert. A fin length of 1.5 times the height of the box culvert is required. Because the debris fins are subject to the same erosive forces as bridge piers, care must be taken in the design of the footing. A toewall at the upstream end of the debris fin and the apron is recommended.

Figure 7-2 in Appendix A of this Manual depicts the conceptual design for debris deflector fins. It should be noted that alternate types of wingwalls can be used other than the parallel shown on Figure 7-2 in Appendix A of this Manual.

7.3 CULVERT DISCHARGE VELOCITIES

High discharge velocities from culverts can cause eddies or other turbulence, which could damage unprotected downstream properties and roadway embankments. To prevent damage from scour and erosion in these conditions, culvert outlet protection is needed as defined by Table 7-2. This outlet protection is based on the discharge velocity.

Table 7-2
Recommended Outlet Protection

Velocity	Outlet Protection
Below 10 ft/s	Riprap protection or alternate approved material
Above 10 ft/s	Structurally reinforced apron, 6 inch minimum thickness with toe wall

The minimum apron length that provides transition from a culvert outlet to an open channel shall be calculated from the following equation:

$$L = 0.2VD \qquad \text{(Equation 7-2)}$$

where:

L = Apron length, feet

V = culvert discharge velocity, ft/s

D = height of box culvert or diameter of pipe culvert, feet

7.4 SELECTION OF CULVERT SIZE AND FLOW CLASSIFICATION

Laboratory tests and field observations show that there are two major types of culvert flow: (1) flow with inlet control; and (2) flow with outlet control. Under inlet control, the cross-sectional area of the barrel, the inlet configuration or geometry, and the amount of headwater are the factors affecting capacity. Outlet control involves the additional consideration of the tailwater in the outlet channel and the slope, roughness, and length of barrel. Under inlet control conditions, the slope of the culvert is steep enough so that the culvert does not flow full and the tailwater does not affect the flow.

7.4.1 Culvert Hydraulics

A. **Inlet Control Condition.** Inlet control for culverts may occur two ways.

1. **Unsubmerged**: The headwater is not sufficient to submerge the top of the culvert opening and the culvert inlet slope is supercritical. The culvert inlet acts like a weir (Condition A, Figure 7-3 in Appendix A of this Manual).

2. **Submerged**: The headwater submerges the top of the culvert, but the pipe does not flow full. The culvert inlet acts like an orifice (Condition B, Figure 7-3 in Appendix A of this Manual).

Nomographs capacity for several culvert materials, shapes, and inlet configurations under inlet control conditions are presented on figures 7-5 to 7-10 in Appendix A of this Manual. These nomographs were developed empirically by the Bureau of Public Roads, the Federal Highway Administration, and various pipe manufacturers. The nomographs are recommended for use in all inlet-control culvert calculations.

B. **Outlet Control Condition.** Outlet control for culverts may occur three ways.

1. The headwater submerges the culvert opening and the culvert outlet is submerged by the tailwater. The culvert will flow full (Condition A, Figure 7-3 in Appendix A of this Manual).

2. The headwater submerges the culvert opening, but the culvert outlet is not submerged by the tailwater. The culvert may or may not flow full (Condition B or C, Figure 7-3 in Appendix A of this Manual).

3. The headwater is insufficient to submerge the top of the culvert opening. The culvert slope is subcritical and the tailwater depth is lower than critical depth for the culvert (Condition D, Figure 7-3 in Appendix A of this Manual).

The capacity of a culvert for outlet control is calculated using Bernoulli's Equation, which is based on the conservation of energy principle. In the application of this equation, an energy balance is determined between the headwater at the culvert inlet and the tailwater at the culvert outlet. This balance is a function of inlet losses, friction losses, and velocity head (see Figure 7-4 in Appendix A of this Manual).

Bernoulli's Equation is:

$d_1 + V_1^2/2g + LS_0 = TW + h_e + h_f + h_v$ (Equation 7-3)

The sum of the first two terms on the left-hand side of Equation 7-3 is equal to the headwater (HW). That is:

$HW = d_1 + V_1^2/2g$ (Equation 7-4)

Substituting Equation 7-4 into Equation 7-3 and isolating the head losses on the right side results in the following equation:

$HW + LS_0 - TW = h_e + h_f + h_v$ (Equation 7-5)

From Figure 7-4 (in Appendix A of this Manual),

$HW + LS_0 = HL + TW$

Thus the total head loss can be determined from this relationship as shown in Equation 7-6:

$HL = HW + LS_0\ TW$ (Equation 7-6)

Substituting Equation 7-6 into Equation 7-5, the following results:

$HL = h_e + h_f + h_v$ (Equation 7-7)

in which $h_v = V_2/2g$ (Equation 7-8)

For inlet losses, the governing equation is Equation 7-1: $h_e = K_e(V_2^2 - V_1^2)/2g$

From Equation 7-4, the headwater (HW) is above the actual depth by the velocity head of the approaching water. However, with water ponded at the entrance, this velocity head (V_1) is usually considered to be negligible, therefore,

$h_e = K_e V^2/2g$ (Equation 7-9)

where K_e is the entrance loss coefficient, as shown in Table 7-1 and V is the velocity of flow in the culvert.

Friction loss is the energy required to overcome the roughness of the culvert material and is expressed as:

$h_f = (29n^2L/R^{1.33})(V^2/2g)$ (Equation 7-10)

where

n = Manning's coefficient

L = Length of culvert, feet

R = Hydraulic radius, feet

V = Velocity of flow in the culvert, ft/s

Combining equations 7-7, 7-8, 7-9, and 7-10 and simplifying the terms results in the following equation:

HL $= (K_e + 1 + 29n^2L/R^{1.33})V^2/2g$ (Equation 7-11)

Equation 7-11 can be used to calculate directly the capacity of the culvert flowing under outlet condition A or B on Figure 7-3 in Appendix A of this Manual. This is because conditions A and B have tailwater depths at or above the top of the culvert, and conditions C and D have tailwater depths less than critical depth. The method for calculating headwater depth for conditions C and D is discussed in the following section.

C. **Depths of Tailwater and Headwater.** In culverts flowing with outlet control, tailwater is an important factor in computing both the headwater depth and the hydraulic capacity of a culvert. Thus, in many culvert designs, it becomes necessary to determine tailwater depth in the outlet channel.

Much engineering judgment and experience are needed to evaluate possible tailwater conditions during storms. Tailwater is often controlled by a downstream obstruction or by water stages in another stream. A field inspection can be made to check on downstream controls and to determine water stages

An approximation of the depth of flow in a natural stream (outlet channel) can be made by using Manning's equation in the channel with normal flow condition (see Section 6.2.1, Uniform Flow, of this Manual). If the water surface in the outlet channel is established by downstream controls, a backwater analysis is required (see Section 6.2.2, Gradually Varied Flow, of this Manual).

The headwater depth can be calculated by the summation of head loss, tailwater depth, and the elevation difference of the inlet and outlet, as shown in the following equation:

HW $= H + h_0\ LS_0$ (Equation 7-12)

where:

HW = vertical distance from flow line at the entrance to the pool surface, feet

H = head loss, feet (use appropriate nomograph)

h_0 = vertical distance from flow line at the outlet to the hydraulic grade line, feet (in this case h_0 equals TW, measured in feet above the flow line)

S_0 = slope of barrel, ft/ft

L = culvert length, feet

Equation 7-12 has the same form shown in Equation 7-6, which was derived from Bernoulli's Equation. For a tailwater depth equal to or greater than the top of the culvert at the outlet (outlet control conditions A and B on Figure 7-3 in Appendix A of this Manual), h_0 can be set equal to TW and the headwater depth can be found by Equation 7-12. For tailwater elevation less than the top of the culvert at the outlet (outlet control conditions C and D on Figure 7-3 in Appendix A of this Manual), h_0 in Equation 7-12 will be assumed as

h_0 =$(d_c+D)/2$ or TW, whichever is greater (Equation 7-13)

where:

d_c = critical depth in feet (d_c cannot exceed D)

D = height of culvert opening in feet, whichever value is greater.

Headwater depth determined by equations 7-12 and 7-13 becomes increasingly less accurate as the headwater computed by this method falls below the value of $D + (1+K_e)V^2/2g$.

A series of nomographs for various culvert materials and shapes have been developed by the Federal Highway Administration and the various pipe manufacturers. The nomographs include inlet control conditions (figures 7-5 to 7-10 in Appendix A of this Manual) and outlet control conditions (figures 7-11 to 7-17 in Appendix A of this Manual). The critical depth for pipes of different shapes is shown on figures 7-16 to 7-22 in Appendix A of this Manual.

7.4.2 Design Procedures

The Texas Hydraulic System (THYSYS) program developed by the TxDOT can be used for culvert design in addition to help calculate the culvert size and related computations. Design procedures are as follows:

A. **Step 1.** List design data.

   1. Design discharge Q, cfs

   2. Length culvert L, feet

   3. Slope of culvert S_o, ft/ft

4. Allowable headwater depth, which is the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the headwater pool or approach channel upstream from the culvert, feet

5. Allowable flow velocity in natural stream, ft/s

6. Type of culvert for first trial selection, including material, cross-sectional shape and entrance type

B. **Step 2.** Determine the first trial size culvert. Because the procedure given is one of trial and error, the initial trial size can be determined by one of the following ways:

   1. Make an arbitrary selection.

   2. Use an approximating equation such as Q/V = A assuming a V for the trial culvert.

   3. Use inlet control nomographs for the culvert type selected (figures 7-5 to 7-10 in Appendix A of this Manual). If this method is used, HW/D must be assumed. If any trial size is too large because of height restrictions or structure availability, multiple culverts may be used by dividing the discharge equally between the number of barrels used.

C. **Step 3.** Find headwater depth for trial size culvert assuming inlet control or outlet control.

   1. Assuming Inlet Control

      a. Using the trial size from Step 2 above, find the headwater depth HW from the appropriate inlet control nomograph (figures 7-5 to 7-10 in Appendix A of this Manual). HW in this case is found by multiplying HW/D obtained from the nomograph by the height of the culvert, D. Tailwater conditions are neglected in this determination.

      b. If HW is greater or less than the desired results, try another culvert size until HW is acceptable for inlet control before computing HW for outlet control.

   2. Assuming Outlet Control

      a. Determine the depth of tailwater, in feet, for the design flood condition at the outlet.

      b. For a TW elevation equal to or greater than the outlet soffit of the culvert, set ho equal to the TW and find HW by Equation 7-12.

c. For a tailwater elevation less than the outlet soffit of the culvert, find headwater HW by Equation 7-12 and Equation 7-13.

3. Compare the headwaters found in Step 3-1 and Step 3-2 (Inlet Control and Outlet Control). The higher headwater governs and indicates the type of flow control for the given conditions and culvert size selected.

D. **Step 4.** If outlet control governs but the HW is too high, select a larger culvert size and recalculate HW as instructed in Step 3-2. If the previously calculated inlet control governs, the smaller size is satisfactory as determined under Step 3-1.

E. **Step 5.** Compute the outlet velocity for the culvert size selected and determine its compatibility with the criteria of Section 7.3.0 of this Manual. If the computed velocity is too high, go back to Step 2 and select a larger culvert size.

1. If outlet control governs in Step 3-3, the outlet velocity equals Q/A_0, where A_0 is the cross-sectional area of flow in the culvert at the outlet. If d_c or TW is less than the height of the culvert barrel, use A_0 corresponding to d_c or TW depth, whichever gives the greater area of flow. A_0 should not exceed the total cross-sectional area A of the culvert barrel.

2. If inlet control governs in Step 3-3, outlet velocity can be assumed to equal mean velocity in open-channel flow in the barrel as computed by Manning's Equation for the rate of flow, barrel size, roughness, and slope of culvert selected.

F. **Step 6.** Record final selection of culvert with size, type, required headwater, and outlet velocity.

7.4.3 Instructions for Using Nomographs

A. **Inlet-Control Nomographs** (figures 7-5 to 7-10 in Appendix A of this Manual).

1. To determine HW, given Q, and size and type of culvert:

a. Connect with a straightedge the given culvert diameter or height (D) and the discharge Q, or Q/B for box culverts; mark intersection of straightedge on HW/D scale marked (1).

b. If HW/D scale marked (1) represents entrance type used, read HW/D on scale (1). If another of the three entrance types listed on the nomograph is used, extend the point of intersection in (a) horizontally to scale (2) or (3) and read HW/D.

c. Compute HW by multiplying HW/D by D.

2. To determine Q per barrel, given HW, and size and type of culvert.

   a. Compute HW/D for given conditions.

   b. Locate HW/D on scale for appropriate entrance type. If scale (2) or (3) is used, extend HW/D point horizontally to scale (1).

   c. Connect point on HW/D scale (1) as found in step 2.b and the size of culvert on the left scale. Read Q or Q/B on the discharge scale.

   d. If Q/B is read in (c), multiply by B (span of box culvert) to find Q.

3. To determine culvert size, given Q, allowable HW, and type culvert.

   a. Using a trial size, compute HW/D.

   b. Locate HW/D on scale for appropriate entrance type. If scale (2) or (3) is used, extend HW/D point horizontally to scale (1).

   c. Connect point on HW/D scale (1) as found in step 3.b to given discharge and read diameter, height, or size of culvert required for HW/D value.

   d. If D is not that originally assumed, repeat procedure with a new D.

B. **Outlet-Control Nomographs** (figures 7-11 to 7-17 in Appendix A of this Manual).

Outlet control nomographs can be used to solve Equation 7-11 for head H when the culvert barrel flows full for its entire length. They are also used to determine H for some part-full flow conditions with outlet control. These nomographs provide and incomplete solution for HW, because they give only H in the equation $HW = H + h_0 - LS_0$.

1. To determine H for a given circular culvert and discharge Q:

   a. Select appropriate nomograph for type of culvert selected. Find K_e for entrance type from Table 7-1.

   b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales, Conduct the following three steps:

   Step 1. If the n value of the nomograph corresponds to that of the culvert selected, choose the length curve correspond to K_e and locate the starting point at the given culvert length. If a K_e curve is not shown for the selected K_e, proceed to Step 2. If the n value for the culvert selected differs from that of the nomograph proceed to Step 3.

Step 2. For a K_e intermediate between the scales given in the nomograph, connect the given length on adjacent scales by a straight line and select a point on this line spaced between the two chart scales in proportion to the K_e values.

Step 3. For a different roughness coefficient (n1) than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the following equation:

$L_1 = L(n_1/n)^2$ (Equation 7-14)

c. Using a straightedge, connect the point on length scale to the point or size of culvert barrel scale and mark the point of crossing on the “turning line.” (See subsection 7.4.3A.2 of this manual for size considerations for a rectangular box culvert.)

d. Pivot the straightedge on this point on the turning line and connect given discharge rate scale. Read head in feet on the head (H) scale.

e. For values beyond the limit of the chart scales, find H by solving Equation 7-13.

2. To use the box culvert nomograph (Figure 7-13 in Appendix A of this Manual) for full flow for other than square boxes:

a. Compute cross-sectional area of the rectangular box.

b. Connect proper point (see subsection 7.4.3A.1. of this Manual) on length scale to barrel area scale and mark point on turning line.

c. The area scale on the nomograph is calculated for barrel cross-sections with span B twice the height D; its close correspondence with area of square boxes assures it may be used for all sections intermediate between square and B = 2D or B = 0.5D. (For other box proportions, use Equation 7-11 for more accurate results.)

d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale.

7.4.4 Example 7-1

The following example problem utilizes the computation illustrated on Figure 7-1 for a culvert rating curve calculation.

Given:

Culvert size D	= 48 inches RCP
Length L	= 110 feet
Roughness coefficient	= 0.012
Inlet elevation	= 720.0 feet (above mean sea level)
Outlet elevation	= 718.8 feet (above mean sea level)
Slope S_0	= 0.010 ft/ft

Entrance condition (square edge), $K_e = 0.50$

Maximum elevation for embankment = 732.0 feet (above mean sea level)

Find: Culvert rating curve

(Figure 7-1 is used to take the computations for the culvert design.)

Step 1. List the elevations for headwater depths in Column 1. Then put headwater depth and ratio of headwater depth to culvert height (or pipe diameter) in Column 2 and Column 3.

Step 2. Based on the inlet control conditions, the ratio of HW/D is used to find the flows (Q) that are put in Column 4. In this example, Item (B) on Figure 7-5 in Appendix A of this Manual is utilized.

Step 3. For outlet control conditions, the flow rate (Q) in Column 4 is used to determine the head loss (H) in Column 5. In this example, Figure 7-12 in Appendix A of this Manual is utilized.

Step 4. If the tailwater rating curve is available, the tailwater (TW) depth can be entered in Column 6. If the tailwater rating curve is not available, an estimate of the tailwater can be used.

Step 5. If the tailwater depth is less than the diameter of the culvert, columns 7 and 8 should be calculated. If TW is larger than D, the TW value is entered in Column 9 for ho.

Step 6. The critical depth (d_c) is found from figures 7-18 to 7-22 in Appendix A of this Manual, and then used to compute Column 8.

Step 7. The headwater depth (HW) now can be computed from Equation 7-12.

Step 8. Compare the two headwater depth values from Column 2 and Column 10. The controlling headwater depth and type of control are recorded in Column 11 and Column 12, respectively. The calculated elevation is written in Column 14.

Step 9. The rating curve for the culvert can be plotted from the values in Column 4 and Column 13.

To size a culvert crossing, the same approach can be used, with some variation in the basic data. First, a design Q value is selected and the maximum allowable headwater is determined. An inlet type (i.e., headwall) is selected and the invert elevations and culvert slope are estimated based upon site constraints. A culvert type is then selected and first rated for inlet control then outlet control. If the controlling headwater exceeds the maximum allowable headwater, the input data is modified and the procedure repeated until the desired results are achieved.

7.5 HYDRAULIC CONSIDERATIONS IN BRIDGE DESIGN

7.5.1 General

Sections 1.2.6C and 1.2.6D of this Manual state the policies concerning allowable flow over bridge structures. The current policy for overtopping of residential streets is a maximum of 12 inches for the 100-year frequency storm, and for any street other than residential, the allowable maximum is 6 inches for the 100-year frequency storm.

Several hydraulic parameters should be considered in bridge design. Among these considerations should be, but should not be limited to, the following:

A. Channel transitions into and out of the bridge opening

B. Overall length and height of bridge

C. Cross-sectional area of bridge opening

D. Location of the bridge opening relative to the main channel

E. Bridge alignment relative to general flow of main channel (i.e., “skewed” crossing)

F. Number of crossings or bridge openings

G. Other obstructions to flow (i.e., piers, abutments, deck width, and clearances)

H. Design flows that bridge opening must pass

I. Required channel freeboard

7.5.2 Types of Flow for Bridge Design

There are three types of flow caused by bridges (as shown on Figure 7-23 in Appendix A of this Manual):

A. **Type I Flow.** Referring to Item A of Figure 7-23 in Appendix A of this Manual, it can be observed that normal water surface is above critical depth at all points. This has been labeled Type I, or subcritical flow, the type usually encountered in practice. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between cross-sections 1 and 4.

B. **Type IIA Flow.** There are at least two variations of Type II flow that will be described here as Types IIA and IIB. For Type IIA flow, Item B of Figure 7-23 in Appendix A of this Manual, normal water surface in the unconstricted channel again remains above critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at cross-section 4).

C. **Type IIB Flow.** The water surface for Type IIB flow, Item C of Figure 7-23 in Appendix A of this Manual, starts out above both normal water surface and critical depth upstream, passes through critical depth in the constriction and then returns to normal. The return to normal depth can be rather abrupt, as illustrated in Item C of Figure 7-23 in Appendix A of this Manual, taking place in the form of a poor hydraulic jump, because the normal water surface in the stream is above critical depth.

D. **Type III Flow.** In Type III Flow, Item D of Figure 7-23 in Appendix A of this Manual, the normal water surface is below critical depth at all points and the flow throughout is supercritical. This is an unusual case requiring a steep gradient, but such conditions do exist. Theoretically, backwater should not occur for this type, because the flow throughout is supercritical. It is more than likely that an undulation of the water surface will occur in the vicinity of the constriction, as indicated on Item D of Figure 7-23 in Appendix A of this Manual.

   A more thorough and complete discussion of these parameters and preliminary design procedures are presented in Chapters 1 and 11 of Hydraulics of Bridge Waterways by U.S. Department of Transportation, Federal Highway Administration.

7.5.3 Modeling Hydraulic Conditions

The most commonly used backwater program for modeling hydraulic conditions at existing or proposed bridge crossings is the U.S. Army Corps of Engineers HEC-RAS Water Surface Profiles Program. The bridge and culvert routines contained within the program are widely used. A thorough discussion of the application of these bridge routines are presented in the HEC-RAS User's Manual. HEC-RAS software is available from the U.S. Army Corps of Engineers Hydrologic Engineering Center.

SECTION 8.0 STORMWATER MANAGEMENT

8.0 STORMWATER MANAGEMENT 8-1

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8.3.1 General 8-2

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8.0 STORMWATER MANAGEMENT

8.1 GENERAL

Stormwater Management (SWM) programs aimed at controlling the detrimental effects of increased urban runoff generated by development are a top priority in urban planning. More-frequent flooding, increased rates and volumes of runoff, increased stream channel erosion and degradation, increased sedimentation, and increased water pollution are all problems intensified by development. SWM facilities such as detention, retention, extended detention, infiltration, and sedimentation ponds have proven to significantly reduce downstream flooding, reduce sediment and pollutant loads, and provide debris removal, which benefit water quality.

The basic concept of SWM for peak rates of runoff is to provide for a temporary storage of excess stormwater runoff. Runoff is then released at a controlled rate that cannot exceed the capacities of the existing downstream drainage systems, or the predeveloped peak runoff rate of the site, whichever is less.

The solid-lined hydrograph shown on Figure 8-1 in Appendix A of this Manual represents a storm runoff event without SWM, while the dashed-line hydrograph depicts the same event with SWM. The peak flow of the undetained hydrograph could exceed the capacity of the downstream conveyance system and thereby cause surcharging and flooding problems. With the introduction of the SWM facility, the solid-lined hydrograph is spread over a longer time period and its peak is reduced. The area between the two curves to the left of their intersection represents the volume of runoff temporarily stored or detained in the SWM facility.

The City intends to control excess flows through the application of both on-site and regional SWM. Essentially, the distinction between the two approaches is that on-site is generally limited to site-specific criteria, while regional incorporates a basin wide hydrologic analysis.

8.2 REGIONAL STORMWATER MANAGEMENT PROGRAM

8.2.1 General

Regional Stormwater Management (RSM) provides for the planning, design and construction of regional drainage improvements. The cost for construction of RSM facilities may be subsidized through fees paid by developers or through in-kind donations by property owners. The RSM uses a watershed-wide approach to analyze potential flooding problems, to identify appropriate mitigation measures, and to select site locations and design criteria for regional drainage improvements. These improvements include detention and retention ponds, waterway enlargement and channelization, and improved conveyance structures. The RSM allows developers to participate in the program (in lieu of constructing on-site controls) if the resulting use of regional drainage improvements will produce no identifiable adverse impact to other properties due to increased runoff from the proposed development.

The fees charged for participation in the RSM are typically based upon the size of the development and the proposed land use.

The benefits afforded by RSM include the following:

A. A higher level of confidence in the hydrologic analysis is obtained because each RSM improvement's interrelationship within a given basin can be readily determined. This is accomplished by establishing a hydrologic database of the entire basin, and then using this to determine the most hydrologically efficient location for SWM facilities. This procedure takes into consideration the interrelated nature of tributary subareas within a watershed.

B. Maintenance is more responsive due to the City's vested interest and responsibility in the RSM.

C. The cost of construction and the total land required can be considerably less than that needed for comparable on-site SWM.

D. The expended land area in RSM lends itself to other uses (e.g., parks, natural areas, organized sports, etc.).

8.3 STORMWATER MANAGEMENT PONDS

8.3.1 General

SWM ponds may be of two basic types: on-site and regional. In general, on-site ponds are those that are located off-channel and provide SWM for a particular project or development or more than one development. Regional ponds are designed to provide SWM in conjunction with other improvements on a watershed-wide basis. SWM ponds may be further classified as retention or detention ponds and may incorporate water quality controls such as sedimentation, infiltration, or filtration. The performance and safety criteria in this section apply to all ponds that provide management of peak rates of stormwater runoff regardless of type.

8.3.2 Performance Criteria for On-Site SWM Ponds

A. On-site SWM ponds are further classified as either small or large, as follows:

Table 8-1
On-Site SWM Pond Classification

On-Site SWM Pond Class	Drainage Area
Small	<25 acres
Large	>25 acres

For design purposes, any pond with a drainage area larger than 200 acres shall use the regional pond criteria in this manual.

B. On-site SWM ponds shall be designed to reduce post-development peak rates of discharge to existing pre-development peak rates of discharge for the 25-year storm events at each point of discharge from the project or development site. For the post-development hydrologic analysis, any off-site areas that drain to the pond shall be assumed to remain in the existing developed condition.

8.3.3 Performance Criteria for Regional SWM Ponds

A. Regional SWM ponds are classified as small and large, based on the following criteria:

Table 8-2
Regional SWM Pond Classification

Regional Pond Class	Impounded Volume (ac-ft)
Small	0–150
Large	>150

Any regional pond with a height of dam over 15 feet shall be classified as a large regional pond.

B. Performance criteria for regional ponds shall be determined on a project-by-project basis. The determination shall be based on a preliminary engineering study prepared by the design Engineer.

8.3.4 Safety Criteria for SWM Ponds

All ponds shall meet or exceed all safety criteria specified in this section. Use of these criteria shall in no way relieve the Engineer of the responsibility for the adequacy and safety of all aspects of the design of the SWM pond.

A. The spillway, embankment, and appurtenant structures shall be designed to safely pass the design storm hydrograph with the freeboard shown in Table 8-3. Any orifice with a dimension smaller than or equal to 12 inches shall be assumed to be fully blocked. For all spillways (especially enclosed conduits), the ability to adequately convey the design flows shall take into account any submergence of the outlet, any existing or potential obstructions in the system, and the capacity of the downstream system. For these reasons, enclosed conduit spillways connecting directly to other enclosed conduit systems are discouraged. If used, they shall be justified by a rigorous analysis of all enclosed conduit systems connected to the spillway.

Table 8-3
Detention Pond Freeboard Requirements

Detention Pond Class	Design Storm Event	Freeboard to Top of Embankment (feet)
On-site: Small	25 year	0
Large	25 year	1.0
Regional: Small	100 year	2.0
Large	100 year	*

*Design storm event and required freeboard for large regional ponds shall be determined by a dam break analysis based on the principles outlined in Title 30, Part 1, Chapter 299 of the Texas Administrative Code. The dam break analysis shall be submitted to the Director of Public Works or his/her designee for approval.

B. If an embankment is classified as a dam pursuant to Title 30, Part 1, Chapter 299 of the Texas Administrative Code, all design criteria found in Title 30, Part 1, Chapter 299 of the Texas Administrative Code shall be met, as evidenced by certification by Professional Engineer licensed in the State of Texas.

C. All SWM ponds shall be designed using a hydrograph routing methodology.

D. The minimum embankment top width of earthen embankments shall be as follows:

Table 8-4
Detention Pond Embankment Requirements

Total height of embankment (feet)	Minimum top width (feet)
0–5	7
5–15	15
15–+	*

*To be determined on a case by case basis by the Public Works Director or his/her designee.

E. The constructed height of an earthen embankment shall be equal to the design height plus the amount necessary to ensure that the design height will be maintained once all settlement has taken place. This amount shall in no case be less than 5% of the total fill height. All earthen embankments shall be compacted to 95% of optimum density.

F. Earthen embankment side slopes shall be no steeper than 4 horizontal to 1 vertical. Slopes must be designed to resist erosion, to be stable in all conditions, and to be easily maintained. Earthen side slopes for regional detention facilities shall be designed on the basis of appropriate geotechnical analyses.

G. Detailed hydraulic design calculations shall be provided for all SWM ponds. Stage-discharge rating data shall be presented in tabular form with all discharge components, such as orifice, weir, and outlet conduit flows, clearly indicated. A stage-storage table shall also be provided. In all cases, the effects of tailwater or other outlet control considerations shall be included in the rating table calculations.

H. When designing ponds in series (i.e., when the discharge of one becomes the inflow of another), the Engineer shall submit a hydrologic analysis that demonstrates the system's adequacy. This analysis must incorporate the construction of hydrographs for all inflow and outflow components.

I. No outlet structures from detention, filtration and/or sedimentation ponds, parking detention, or other concentrating structures shall be designed to discharge concentrated flow directly onto arterial or collector streets. Such discharges shall be conveyed by a closed conduit to the nearest storm sewer. If there is no existing storm sewer serving the development, the outlet design shall provide for a change in the discharge pattern from concentrated flow back to sheet flow, following as near as possible the direction of the gutter. If the outflow discharge of the detention is released to a watercourse, the proper erosion control measure shall be implemented to prevent erosion.

J. Storm runoff may be detained within parking lots. However, the Engineer should be aware of the inconvenience to both pedestrians and traffic. The location of ponding areas in a parking lot should be planned so that this condition is minimized. Stormwater ponding on public sidewalks is prohibited and shall be fully contained within the parking lot area to be used for storage. The design Engineer shall minimize concentrated flows at the outfall to prevent erosion.

K. All stormwater pipes discharging into a public storm sewer system shall have a minimum diameter of 18 inches. In all cases, ease of maintenance and/or repair must be assured.

L. All concentrated flows into a SWM pond shall be collected and conveyed into the pond in such a way as to prevent erosion of the side slopes. All outfalls into the pond shall be designed to be stable and nonerosive.

8.3.5 Outlet Structure Design

There are two basic types of outlet control structures: those incorporating orifice flow and those incorporating weir flow. Rectangular and V-notch weirs are the most common outlet types.

Generally, if the crest thickness is more than 60% of the nappe thickness, the weir should be considered broad-crested. The coefficients for sharp-crested and broad-crested weirs vary. The respective weir and orifice flow equations are as follows:

A. **Rectangular Weir Flow Equation** (see Figure 8-2 in Appendix A of this Manual)

$Q = CLH^{3/2}$ (Equation 8-1)

where:

Q = Weir flow, cfs

C = Weir Coefficient

L = Horizontal length, feet

H = Head on weir, feet

B. **V-notch Weir Flow Equation** (see Figure 8-2 in Appendix A of this Manual)

$Q = C_v \tan(0/2)H^{2.5}$ (Equation 8-2)

where:

Q = Weir flow, cfs

C_v = Weir Coefficient

O = Angle of the weir notch at the apex, degrees

H = Head on Weir, feet

C. **Orifice flow** equation (see Figure 8-2 in Appendix A of this Manual)

$Q = C_oA(2gH)^{0.5}$ (Equation 8-3)

where:

Q = Orifice Flow, cfs

C_o = Orifice Coefficient (use 0.6)

A = Orifice Area, square feet

g = Acceleration due to gravity

H = Head on orifice measured from centerline, feet

Analytical methods and equations for other types of structures shall be approved by the Public Works Director or his/her designee prior to use.

In all cases, the effects of tailwater or other outlet control considerations should be included in the rating table calculations.

8.4 DETENTION POND STORAGE DETERMINATION

A flow routing analysis using the detailed hydrograph method previously defined in this Manual shall be applied for all detention pond designs. The SCS hydrologic methods (available in HEC-HMS) and the Hydrologic Engineering Center (HEC) hydrologic methods may be used. The Engineer may use other methods but must have their acceptability approved by the Public Works Director or his/her designee.

SECTION 9.0 EROSION AND SEDIMENT CONTROL

9.0 EROSION AND SEDIMENT CONTROL 9-1

9.1 GENERAL 9-1

9.2 EROSION AND SEDIMENT CONTROL REQUIRED 9-1

9.0 EROSION AND SEDIMENT CONTROL

9.1 GENERAL

The purpose of this section is to provide policy for the protection of land and water resources, so as to minimize the adverse effects of erosion and sedimentation per the City of Killeen's Municipal Separate Storm Sewer System (MS4) Phase II Permit (TXR040010). Additionally, criteria have been fashioned to complement the language of the TPDES General Permit (TXR150000) regulating Stormwater discharges from construction sites and the City of Killeen Code of Ordinance (Chapter 32, Article III).

The conversion of land from its natural state to urban use accelerates the processes of erosion and sedimentation. This accelerated process can negatively impact drinking water supply, aquatic life, and the recreational resource provided by them. As additional development and urban growth takes place in the City, the value of all land and water resources increases. The conservation of these resources is less expensive than their restoration.

Construction related sediment can be a significant pollutant of streams, lakes, ponds, and reservoirs. Sedimentation can also carry pesticides, phosphates and many other chemical pollutants on the soil particles. These pollutants are carried to the waterway on the sediment particle and further reduce the quality of the water.

On most development projects, the highest erosion potential exists between the time when the existing vegetation is removed to begin site work and the completion of construction and final revegetation. There are numerous activities associated with construction and land development that accelerate the rate of erosion. Virtually all of these actions involve the removal of vegetation and/or the movement of the native geologic structure during a construction. The adverse impact upon the site and the environment in general can be reduced if these actions are taken with some thought to the resultant erosion.

The erosion and sediment best management practices (BMP) included in Appendix B provide several methods to address the dual problems of erosion and sedimentation, but are in no way a complete outline of the possible actions to provide adequate control. Any questions concerning the criteria or the use of BMPs not included in this Manual, the design Engineer or Certified Professional shall be directed to the Director of Public Works or his/her designee prior to use at construction sites.

9.2 EROSION AND SEDIMENT CONTROL REQUIRED

The City of Killeen Erosion and Sedimentation Control policy shall govern the planning, design, installation, maintenance, and inspection of temporary and permanent erosion and sedimentation controls associated with development. This policy is the official criteria manual required by the TPDES MS4 Phase II permit, and as such strives to comply with all federal and state permit mandates.

Erosion and sediment BMPs are required for all significant ground disturbance activities and development, conducted with or without a permit, including without limitation commercial, multi-family, single-family, road, utility, park, golf course, water quality basins, and detention basin construction and all other activities utilizing clearing, trenching, grading, or other construction techniques. It is the intent of City of Killeen policy to closely parallel the requirements set forth in the TPDES Construction General Permit (TXR150000), the City of Killeen's MS4 Phase II Permit (TXR040010), and any applicable updates to the state and federal National Pollutant Discharge Elimination System (NPDES) program.

This policy is intended to achieve the following objectives:

- MS4 Phase II Permit compliance.
- TPDES Construction General Permit compliance.
- Minimize the erosion and transport of soil resulting from development activities.
- Minimize sedimentation in streams, creeks, lakes, waterways, storm drains, etc.
- Protect and improve the quality of surface water and maintain and improve the quality and quantity of recharge to groundwater supplies.
- Minimize flooding hazards and silt removal cost associated with excessive sediment accumulation in storm drains and waterways.
- Preserve and protect existing vegetation to the greatest extent possible, particularly native plant and wildlife habitats.
- Provide for revegetation of sites to minimize the negative environmental impacts of construction activity.

Development disturbing greater than 1 acre shall be required to develop and implement a Stormwater Pollution Plan (SWPPP) as outlined in TPDES Construction General Permit (TXR150000) and the City of Killeen's Erosion and Sediment Control Ordinance (Chapter 32, Article III). Under the City of Killeen's Illicit Discharge Ordinance (Chapter 32, Article II), it is illegal to discharge pollutants, including sediment, debris, materials, waste, and other pollutants into the City's MS4.

Developers, Engineers, and Certified Professionals may select an appropriate control method or combinations of methods or structures described in Appendix B and are responsible for both the adequacy and implementation of an effective erosion and sedimentation control plan. The developer, contractor, and/or Engineer are responsible for ensuring proper erosion and sedimentation control until all areas are stabilized following construction activities.

Appendix A

Figures and Diagrams

Appendix B

Erosion and Sediment Control Best Management Practices

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Glossary

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GLOSSARY

Abutment - A wall supporting the end of a bridge or span, and sustaining the pressure of the abutting earth.

Apron - A floor or lining of concrete, timber, or other suitable material located at the inlet or discharge side of hydraulic structures (box culverts, spillways, etc.) designed to protect the waterway from erosion from falling water or turbulent flow.

Backwater - The rise of the water level upstream due to an obstruction or constriction in the channel.

Backwater Curve - The term applied to the longitudinal profile of the water surface in an open channel when flow is steady but nonuniform.

Baffle Chute - A drop structure in a channel or outlet of a pond with baffles for energy dissipation to permit the lowering of the hydraulic energy gradient in a short distance to accommodate topography.

Baffles - Deflector vanes, guides, grids, gratings, or similar devices constructed or placed in flowing water to (1) cause a more uniform distribution of velocities; (2) dissipate energy; (3) divert, guide, or agitate the flow; and (4) mitigate eddy currents.

Calibration - Process of checking, adjusting, or standardizing operating characteristics of instruments and model appurtenances on a physical model or coefficients in a mathematical model. The process of evaluating the scale readings of an instrument in terms of the physical quantity to be measured.

Channel Roughness - The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the Manning coefficient "n" used in the Manning Equation.

Chute - An inclined conduit or structure used for conveying water to a lower level.

Concentrated Flow - Stormwater runoff that moves through an open waterway or channel that is bounded by banks or walls, such as a swale, ditch, creek, river, open pipe, flume or culvert.

Conduit - Any open or closed device for conveying flowing water.

Criteria - A standard or rule on which a judgment or decision is based.

Critical Flow - The state of flow for a given discharge at which the specific energy is a minimum with respect to the bottom of the conduit.

Critical Slope - The minimum slope of a conduit that will produce critical flow

Crown - (1) The highest point on a transverse section of a conduit; (2) or the highest point of a roadway cross section.

Culvert - Pipe or other conduit through which flow passes through an earthen embankment (e.g., road profile).

Curb - A structure located along the edge of a roadway, normally constructed integrally with the gutter, which strengthens and protects the pavement edge and clearly defines the pavement edge to vehicle operators.

Dam - A barrier constructed across a watercourse for the purpose of either temporarily or permanently impounding water.

Design Storm or Flood - The storm or flood that is used as the basis for design (i.e., against which the structure is designed to provide a stated degree of protection or other specified result).

Detention - The storage of storm runoff for a controlled release during or immediately following the design storm.

1. Off-site detention - A detention measure located outside the boundary of the area it serves.

2. On-site detention - A detention measure that is located within the specific site or subdivision it serves.

3. On-stream detention - The temporary storage of storm runoff behind embankments or dams located in a channel.

4. Regional detention - Detention facilities provided to control excess runoff based on a watershed-wide hydrologic analysis.

Drainage Area - The aggregate surface boundary contributing storm runoff to a stream or drainage system at a particular point.

Drop Structures – A transition used to reduce channel velocities by allowing for flatter upstream and downstream channel slopes

Engineer - A person who is duly licensed and registered to engage in the practice of professional engineering in the State of Texas.

Energy Grade Line - A line representing the energy in flowing water. The elevation of the energy line is equal to the summation of elevation of the flow line plus the depth, velocity head, and the pressure head.

Entrance Head - The head required to force flow into a conduit or other structure; it includes both entrance loss and velocity head.

Entrance Loss - Head lost in eddies or friction at the inlet to a conduit, headwall, or structure.

Exceedance Probability - Is the probability that an event will be equaled or exceeded.

Flood Control - The elimination or reduction of flood losses by the construction of flood storage reservoirs, channel improvements, dikes and levees, by-pass channels, or other engineering works.

Floodplain - Geographically the entire area subject to flooding at a given design storm even reoccurrence.

Flume - A narrow shallow rectangular concrete open channel that conveys stormwater runoff, typically less than 18 inches in depth.

Freeboard - The distance between the calculated water surface elevation and the maximum possible physical elevation of a channel or pond, which is provided as an additional factor of safety.

Frequency (of storms, floods) - Average recurrence interval of events over long periods of time. Mathematically, frequency is the reciprocal of the exceedance probability.

Friction Slope - The friction head or loss per unit length of channel or conduit. For uniform flow, the friction slope coincides with the energy gradient. Where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slope and the energy gradient. The friction slope is equal to the bed or surface slope only for uniform flow in uniform open channels.

Froude Number - A flow parameter that is a measure of the extent to which gravitational action affects the flow. A Froude number greater than 1 indicates supercritical flow and a value less than 1 indicates subcritical flow.

Gabion - A wire basket containing rocks that is placed uniformly with others to provide protection against erosion.

Grade - The inclination or slope of a channel, conduit, or natural ground surface, usually expressed in terms of the ratio of vertical rise to horizontal distance.

Gutter - A shallow structure typically constructed of concrete adjacent to a curb for conveying street flow.

Headwall - When properly designed, a feature whose normal function is to anchor a culvert to prevent movement due to hydraulic and soil pressures, to control erosion and scour resulting from excessive velocities and turbulence, and to prevent adjacent soil from sloughing into a waterway opening.

Headwater - (1) The upper reaches of a stream near its sources; (2) the region where groundwaters emerge to form a surface stream; or (3) the headwater depth on the upstream side of a structure (see Entrance Head).

Hydraulic Gradient - A hydraulic profile of the piezometric level of water, representing the sum of the depth of flow and the pressure head. In open channel flow, it is the free water surface.

Hydraulic Jump - An abrupt rise in the water surface, which occurs in an open channel when water flowing at supercritical velocity transitions to subcritical velocity. The transition through the jump results in a marked loss of energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is sometimes used as a means of energy dissipation.

Hydraulics - The science that deals with practical applications of the mechanics of water movement.

Hydrograph - A graph or table showing discharge versus time at a given point on a stream or conduit.

1. Synthetic Hydrograph - Runoff or unit hydrographs that are devised by empirical means (as opposed to derivation based upon natural, measured data).

2. Unit Hydrograph - The direct runoff hydrograph resulting from one inch of precipitation excess distributed uniformly over a watershed for a specified duration.

Hydrology - The science that deals with the processes governing the depletion and replenishment of the water resources of the earth.

Hyetograph - A histogram or graph of rainfall intensity versus time for a storm.

Impervious - A term applied to a material through which water cannot pass, or through which water passes with great difficulty.

Infiltration - The absorption of water by the soil, either as it falls as precipitation, or from a stream flowing over the surface.

Inlet - The inflow point for a storm sewer system that is usually associated with channelized or conduit flow.

Intensity - See Rainfall Intensity.

Invert - The floor, bottom, or lowest portion of the internal cross section of a conduit. Used particularly with reference to sewers, tunnels, and drains.

Lag Time - In hydrograph analysis, lag time is the time from the centroid of the mass of excess rainfall to the peak of the runoff hydrograph.

Manning Coefficient - The coefficient of roughness used in the Manning Equation.

Manning Equation - An equation used to relate velocity, hydraulic radius, and the energy gradient slope of uniform flow in an open channel.

May - A permissive condition. No requirement for design or application is intended.

Must - A mandatory condition. Where certain requirements in the design or application of the guidelines are described with the “must” stipulation, it is mandatory that the requirements be met.

One Hundred (100) Year Storm - Size of storm equaled or exceeded on the average once in 100 years (with given duration), or that storm having a 1% chance of occurring in any given year.

One Hundred (100) Year Flood - Size of flood equaled or exceeded on the average once in 100 years, or has a 1% chance of occurring in any given year. Usually associated with the 100-year storm.

Orifice - An opening with closed perimeter, and of regular form in a plate, wall, or partition, through which water may flow.

Overland Flow - Runoff that is not considered concentrated. See sheet flow.

Peak Flow (Peak Rate of Runoff) - The maximum rate of flow past a particular point for a given storm event.

Policy - A definite course or method of action selected to guide and determine present and future decisions.

Precipitation - Any moisture that falls from the atmosphere, including snow, sleet, rain, and hail.

Prismatic Channel - A channel built with unvarying cross section and constant bottom slope.

Probable Maximum Flood (PMF) - The flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region.

Probable Maximum Precipitation - The critical depth-duration-area rainfall relationship that would result from a storm containing the most critical meteorological conditions considered probable of occurring.

Rainfall Duration - The length of time over which a discrete rainfall event lasts.

Rainfall Frequency - The average recurrence interval of rainfall events, averaged over long periods of time.

Rainfall Intensity - The rate of accumulation of rainfall, usually in inches per hour.

Rational Formula - A traditional means of relating runoff from an area and the intensity of the storm rainfall ($Q = CiA$).

Reach - Any length of river or channel. Usually used to refer to sections that are uniform with respect to discharge, depth, area or slope, or sections between gauging stations.

Recommended - A condition that should be met if it is physically and economically feasible to do so.

Required - A mandatory condition. Where certain requirements in the design or application of the guidelines are described with the “required” stipulation, it is mandatory that they be met.

Recurrence Interval - The average interval of time within which a given event would be equaled or exceeded once. For an annual series (as opposed to a partial duration series), the probability of occurrence in any 1 year is the inverse of the recurrence interval. Thus, a flood having a recurrence interval of 100 years has a 1% probability of being equaled or exceeded in any one year.

Return Period - See Recurrence Interval.

Right-of-way - Land dedicated by a plat or separate instrument for the benefit of the public, typical as an improved thoroughfare.

Riprap (or Revetment) - Form of bank protection, usually rock. Riprap is a term applied to stone, which is dumped rather than placed more carefully.

Routing - Routing is a technique used to predict the temporal and spatial variations of a flood wave as it traverses a river reach or reservoir. Generally, routing techniques may be classified into two categories - hydrologic routing and hydraulic routing.

Runoff - That part of the precipitation that reaches a stream, drain, or sewer.

Runoff Coefficient (C) - A decimal number used in the Rational Formula that defines the runoff characteristics of the drainage area under consideration. It may be applied to an entire drainage basin as a composite representation or it may be applied to a small individual area such as one residential lot.

Sediment - Material of soil and rock origin transported, carried, or deposited by water.

Shall - A mandatory condition. Where certain requirements in the design or application of the guidelines are described with the “shall” stipulation, it is mandatory that the requirements be met.

Sheet Flow - Stormwater runoff that flows downslope over relatively smooth surfaces in the form of a thin, continuous layer that does not vary in depth perpendicular to the direction of flow.

Should - An advisory condition. Where the word “should” is used, it is considered to be advisable usage, recommended but not mandatory.

Soffit - The bottom of the top of a conduit. In a pipe, it is the uppermost point on the inside of the structure. In contrast, the crown is the uppermost point on the outside of the pipe wall.

Soil Conservation Service (SCS) Runoff Curve Number (CN) - Index number used by the SCS as a measure of the tendency of rainfall to run off into streams rather than evaporate or infiltrate.

Steady Flow - Open channel flow is characterized as steady if the depth of flow does not change or if it can be assumed to be constant during the time interval of consideration.

Stilling Basin - Pool of water that is conventionally used as part of a drop structure or other structure to dissipate energy.

Synthetic Hydrograph - See Hydrograph

Tailwater - The depth of flow in a stream immediately downstream of a hydraulic structure.

Time of Concentration - The time associated with the travel of a drop of runoff from an outer point of a given drainage basin along a flow path that best represents the shape of the contributing area.

Total Head - In the flow process, the total energy for a given point that is represented by the summation of $V_2/2g$, p/γ, and z. The units for these three items are foot-pounds force per pound force. It is common practice to lump all these three items together as total head in feet. The item of $V_2/2g$ is called velocity head (in feet) and p/ is the pressure head (in feet).

Trunk Line - The primary collector line of a storm sewer system

Uniform Channel - A channel with a constant cross section and roughness.

Uniform Flow - Open channel flow is said to be uniform if the depth of flow is the same at every section of the channel, for a constant flow.

Unit Hydrograph - See Hydrograph.

Watercourse - A watercourse includes a drainage path or way or the channel of a stream, to include, without limitation, waters in the State or U.S., in which water flows within a defined bed and banks, even though the same may be slight, imperceptible or even absent in places, and originates from a definite source or sources. The water need not always be present and may be intermittent if the latter occurs with some degree of regularity, depending on the characteristics of the sources (i.e.: water is present or flowing during and/or after a rainfall event) or as amended in City Ordinance Chapter 32.

Watershed - The total area contributing storm runoff to a discrete point of discharge (i.e., stream or creek).

Weir - A notch of regular geometry through which water flows.