STORMWATER DESIGN MANUAL

CITY OF WICHITA FALLS, TEXAS
PUBLIC WORKS DEPARTMENT
1300 SEVENTH STREET
WICHITA FALLS, TEXAS 76301

Approved by City Council ~~Russell Schreiber, P.E., Public Works Director~~

Effective Date: ________________________, 2024 ~~2011~~

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1.0 INTRODUCTION:

1.1 Purpose and Scope

The development of this design manual is authorized by the City of Wichita Falls' Stormwater Management Ordinance (the "Stormwater Ordinance") to protect and provide for the safety and welfare of the general public and to mitigate flood damage to private and public property within the City and its extraterritorial jurisdiction. This manual establishes standard principles and practices, design guidance, and a framework for incorporating effective and environmentally sustainable stormwater management into the development and construction processes within the jurisdiction of the City and is intended to encourage a greater uniformity in developing plans for stormwater management systems. The design methodology, procedures, factors, formulae, and graphs described in the following pages are intended to serve as guidelines for the analysis of drainage matters for local government review and approval purposes; however, the responsibility for the adequacy and effectiveness of the actual design remains with the design engineer and sound engineering judgment must always be applied. Users of this manual should be knowledgeable and experienced in the theory and application of drainage engineering principles. Any deviation from the requirements of this manual must be approved by the Director of Public Works.

2.0 DEVELOPMENT PROCESS:

2.1 Predevelopment Conference

Prior to beginning any design and concept layout for a development project within the City, the developer and his/her design engineer(s) are strongly advised to schedule, in advance, a Predevelopment Conference with the City Engineering Division. Appointments can be made by calling the City Engineer at (940) 761-7477. General information as to the project location, site size, intended use, and other relevant factors should be given when scheduling. The developer, developer's engineer and City Engineering staff can then meet and discuss specific drainage and infrastructure issues for the project site prior to beginning the design process. Available City electronic information such as topographic maps and existing hydrologic and hydraulic studies can be obtained. Additionally, the meeting will serve to review the City's expectations for studies and plans to be prepared and submitted for City review and approval in keeping with this manual during the design and approval process.

2.2 Concept/Preliminary Study

Once concept plans for a project are prepared, the design engineer shall provide to the City Engineering Division a written report that describes and documents the background, intent and methodology intended to be used along with preliminary plans and sufficient analyses to indicate that the requirements of the Stormwater Ordinance can be addressed. For a subdivision, this submission would be commonly referred to as a master drainage plan. This plan and study shall be prepared by a Professional Engineer licensed in the State of Texas with demonstrated knowledge of the study of drainage issues and proficiency with drainage analysis and modeling tools and should include, but is not limited to:

- a preliminary plat and/or overall concept development plan for phased projects;
- background topographic data for the site including off-site drainage area(s) (existing City topographic data may be used for this submission);
- depiction of existing FEMA floodplain and floodway lines on the site and drainage area;
- any proposed revisions to the FEMA floodplain;

- initial hydrologic analyses to assess the stormwater impact of the proposed development;
- approximate structure, pond, and conveyance sizes and proposed flow line grades;
- indications that appropriate easements can be provided, dedicated or obtained; and
- other information that will assist City staff in determining if the project can reasonably comply with the Stormwater Ordinance.

If the design engineer intends to claim that there will be no downstream impact from the development, then a complete downstream assessment as later described including all necessary documentation, analysis and background materials shall be provided for review and approval by the City at this stage.

After appropriate review, the City Engineering Division will notify the design engineer of any comments, suggested revisions and its conditional consent to proceed with final plans. This does not constitute tacit approval of the project plans but simply indicates that the initial proposal appears to be conceptually viable. Department of Public Works approval of a preliminary plat is contingent upon this conditional consent.

2.3 Final Study/Construction Plans

After conditional consent for a project is obtained, the design engineer shall provide to the City Engineering Division final construction plans and analyses that indicate and document the specific improvements that will fully address the requirements of the Stormwater Ordinance. This plan and study data shall be prepared by a Professional Engineer licensed in the State of Texas with demonstrated knowledge of the study of drainage issues and proficiency with drainage analysis and modeling tools and shall include, but are not limited to:

- project specific on-site and off-site grading, drainage and/or detention plans with details of proposed improvements intended to provide compliance with the Stormwater Ordinance including the 100 year hydraulic grade line surfaces being specifically noted on the profile sheets;
- surveyed topographic data for the site and pertinent off-site features and verified topographic data for related drainage area(s);
- depiction of existing FEMA floodplain and floodway lines on the site and drainage area;
- final hydrologic and hydraulic analyses and calculations to document the stormwater impact of the proposed development;
- final downstream assessment as described in Section 3.2 hereof;
- specific structures, pond, and conveyance sizes along with profiles and representative cross sections of drainage channels;
- copies of required permits obtained including all application data as submitted to the approving agency;
- specific identification of required phased improvements within the proposed development sequence;
- signed easement documents with metes and bounds descriptions and/or a copy of the final plat with dedications noted; and
- other information that will assist City staff in determining that the project complies with the Stormwater Ordinance.

The City's approval of the final study and construction plans is a precondition of obtaining approvals from the Department of Public Works for final plats or for building permits. Approval will be evidenced by the signature of the City Engineer or his/her designee on the cover page of the plan set and by a separate letter detailing any other matters required to comply with the Ordinance, this Stormwater Manual, or for approval and acceptance of the drainage improvements.

2.4 Construction Review and Engineer Certification

2.4.1 Public Drainage Improvements

The City Engineering Division shall provide on-going inspection of construction activities and all work related to drainage improvements that are being constructed for City acceptance and maintenance. The City inspector will keep a log of construction site visits and the project contractor will be required to maintain a red-lined set of project drawings indicating any variations noted between plans and actual construction. Upon completion of the drainage improvements, the design engineer shall, as deemed appropriate, field verify: flow line grades of structures and conveyances, measurements of structures, and actual volumes of storage facilities. The engineer shall then produce an as-built set of project drawings using the red-lined plan set from the Contractor, notes kept by the City Inspector, appropriate verified grades and volumes, and other information known to the engineer. Two hard copy sets of the as-built drawings and an electronic file of the drawings in AutoCAD format shall be provided to the City. The as-built plans and any required maintenance bond shall be provided to the City in satisfactory format prior to the City's acceptance of the improvements.

2.4.2 Privately Maintained Drainage Improvements

Upon completion of project site improvements that will be privately maintained, such as a commercial site detention pond, the design engineer shall field verify the as-built flow line grades of structures and conveyances; obtain actual measurements of structures, and verify actual volumes of storage facilities and shall provide a detention facilities agreement in form and substance as provided by the City, a copy of which is included in the Appendix to this Manual, which includes:

- the notarized signature of the fee simple owner of the subject property;
- an Exhibit "A" containing a reduced copy of the as-built plan of the drainage facility with a sealed and signed certification by the engineer as to the volume and release rate of the facilities; and
- an Exhibit "B" containing the metes and bounds description of the area comprising the detention facility.

The City Engineering Division's approval of the project for a certificate of occupancy will be contingent upon receipt and acceptance of the signed and sealed detention facilities agreement.

3.0 DESIGN CRITERIA:

3.1 Hydrology

3.1.1 Hydrologic Methods

The following methods are approved to support hydrologic site analysis for the design methods and procedures included in this Manual:

- Rational Method
- Modified Rational Method
- SCS Unit Hydrograph Method
- Snyder's Unit Hydrograph Method

Table 3.1 lists the hydrologic methods and the circumstances for their use in various analysis and design applications. Table 3.2 provides some limitations on the use of several methods.

In general the Rational Method is recommended for small highly impervious drainage areas such as parking lots and roadways draining into inlets and gutters.

Table 3.1 Applications of the Recommended Hydrologic Methods					
Method	Related Section	Rational Method	SCS Method	Modified Rational	Snyder's Unit Hydrograph
Gutter Flow and Inlets	Section 3.3.1	✓			
Closed Conduits	Section 3.3.2	✓	✓		✓
Storage Facilities	Section 3.3.3		✓	✓	✓
Open Channels	Section 3.3.4	✓	✓		✓
Culverts	Section 3.3.5	✓	✓		✓
Bridges	Section 3.3.6		✓		✓
Energy Dissipation	Section 3.3.7	✓	✓		✓

Table 3.2 Constraints on Using Recommended Hydrologic Methods		
Method	Size Limitations[1]	Comments
Rational	0 – 200 acres	Method can be used for estimating peak flows and the design of small site or subdivision storm sewer systems.
Modified Rational	0 – 200 acres	Method can be used for estimating runoff volumes for storage design.
Unit Hydrograph (SCS)[2]	0-2000 acres	Method can be used for estimating peak flows and hydrographs for all design applications.
Unit Hydrograph (Snyder's)[3]	1 sq. mile and larger	Method can be used for estimating peak flows and hydrographs for all design applications.

[1] Size limitation refers to the drainage basin for the stormwater management facility (e.g., culvert, inlet).
[2] This refers to SCS methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.
[3] This refers to the Snyder's methodology included in many readily available programs (such as HEC-HMS or HEC-1) that utilize this methodology.

If local stream gage data are available, these data can be used to develop peak discharges and hydrographs. The user is referred to standard hydrology textbooks for statistical procedures that can be used to estimate design flood events from stream gage data.

Note: It must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex and too little data are available on the factors influencing the rainfall-runoff relationship to expect exact solutions.

3.1.2 Rainfall Estimation

Rainfall intensities for the City of Wichita Falls are based on "National Oceanic and Atmospheric Administration's (NOAA) Atlas 14" and ~~have historically been provided in that Intensity Duration Frequency Curve prepared by Forrest and Cotton Engineers in January 1965 which may be used for hydrologic analysis within the City. Values~~ may ~~also~~ be calculated using the following formula:

$$\mathbf{i = b/(t_c + d)^e} \qquad \textbf{(3.1)}$$

where i is inches per hour and t_c is the rainfall duration in minutes. The parameters b, d and e for storm frequencies of 2 year through 100 year events for Wichita County, Texas are shown in Table 3.3:

Table 3.3 Parameters for Formula 3.1 – Wichita County, Texas

Event	e	b	d
2 yr.	0.7967 ~~803~~	42.6883 ~~51~~	9.6027 ~~9.4~~
5 yr.	0.7998 ~~84~~	58.4039 ~~62~~	10.5369 ~~8.7~~
10 yr.	0.8020 ~~795~~	71.1754 ~~76~~	11.2081 ~~8.7~~
25 yr.	0.8050 ~~792~~	89.6041 ~~88~~	12.1319 ~~8.7~~
50 yr.	0.8074 ~~797~~	104.7833 ~~104~~	12.9142 ~~8.7~~
100 yr.	0.8105 ~~792~~	122.2694 ~~114~~	13.7855 ~~9.4~~

3.1.3 RATIONAL METHOD

3.1.3.1 Introduction

When using the Rational Method some precautions should be considered:

- In determining the C value (runoff coefficient based on land use) for the drainage area, hydrologic analysis should take into account any future changes in land use that might occur during the service life of the proposed facility.
- Since the Rational Method uses a composite C and a single t_c value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis (e.g., if the impervious areas are segregated from the pervious areas), then the basin should be divided into sub-drainage basins.
- The formulae and tables included in this section are given to assist the engineer in applying the Rational Method. The engineer should use sound engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate adjustments are appropriate.
- The Rational Method should not be used for calculating peak flows downstream of bridges, culverts, or storm sewers that may act as restrictions causing storage that impacts the peak rate of discharge.

3.1.3.2 Equations

The Rational Formula is expressed as follows:

$$Q = CIA \qquad (3.2)$$

where:

Q	=	maximum rate of runoff (cfs)
C	=	runoff coefficient representing a ratio of runoff to rainfall per Table 3.6
I	=	average rainfall intensity for a duration equal to the t_c (in/hr)
A	=	drainage area contributing to the design location (acres)

3.1.3.3 Time of Concentration

Use of the Rational Formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). The time of concentration consists of an overland flow time to the point where the runoff is concentrated or enters a defined drainage feature (e.g., open channel) plus the time of flow in a closed conduit or open channel to the design point.

For each drainage area, the distance is determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. In urban areas, the length of overland flow distance should realistically be no more than 50 – 100 feet.

Table 3.5 gives recommended minimum and maximum times of concentration based on land use categories. The minimum time of concentration should be used for the most upstream inlet (minimum inlet time). Computed downstream travel times will be added to determine times of concentration through the system. For anticipated future upstream development, the time of concentration should be no greater than the maximum.

Table 3.5 Times of Concentration

Land Use	Minimum (minutes)	Maximum (minutes)
Residential Development	15	30
Commercial and Industrial	10	25
Central Business District	10	15

3.1.3.4 Runoff Coefficient (C)

Table 3.6 provides certain runoff coefficients for the Rational and Modified Rational Methods. The design engineer may also calculate and submit a site specific C value by using the actual site areas and percentages of different land uses within the site being considered rather than arbitrarily using the values from Table 3.6. Clear documentation of the C value determination shall be submitted for review and approval by the City Engineering staff.

If it is important to locate a specific land use within the drainage area, then another hydrologic method should be used where hydrographs for each individual drainage area can be generated and routed through the drainage system.

It may be that using only the impervious area from a highly impervious site (and the corresponding high C factor and shorter time of concentration) will yield a higher peak runoff value than by using the whole site. This should be checked particularly in areas where the overland portion is grassy (yielding a long t_c) to avoid underestimating peak runoff.

Table 3.6 Runoff Coefficient Values

Description of Area	Runoff Coefficients (C)
Lawns:	
Sandy soil, flat, <2%	0.10
Sandy soil, average, 2 - 7%	0.15
Sandy soil, steep, > 7%	0.20
Clay soil, flat,<2%	0.17
Clay soil, average, 2 - 7%	0.22
Clay soil, steep, > 7%	0.35
Agricultural (cultivated)	0.30
Mesquite Pasture	0.25
Streams, Lakes, Water Surfaces	1.00
Business/Commercial/Industrial:	calculate
Residential:	
Single Family (6 lots/ac)	0.55
Single Family (4 lots/ac)	0.50
Single Family (3 lots/ac)	0.45
Single Family (2 lots/ac)	0.40
Single Family (1+ acre lots)	calculate
Multi-Family Projects	calculate
Parks, cemeteries	0.25
Playgrounds	0.35
Railroad yard areas	0.40
Streets:	
Asphalt and Concrete	0.95
Brick	0.85
Drives, walks, and roofs	0.95
Gravel areas	0.50

Graded or no plant cover:	
Sandy soil, flat, 0 - 3%	0.20
Sandy soil, flat, >3%	0.25
Clayey soil, flat, 0 - 3%	0.25
Clayey soil, average, >3%	0.35

3.1.4 MODIFIED RATIONAL METHOD

3.1.4.1 Introduction

For drainage areas of *less than 200 acres*, a modification of the Rational Method can be used for the estimation of storage volumes for detention calculations.

The Modified Rational Method is a procedure based on the Rational Method by which hydrographs are developed rather than only a peak flow. The hydrographs determined by the Modified Rational Method are based on the assumption that runoff begins and increases linearly to the peak volume of runoff. The time in which the peak is reached is the time of concentration (t_c). The peak is maintained for the storm duration and then linearly decreases to zero. The duration (horizontal axis) for both the rising and falling limbs of the inflow hydrograph equal t_c and the peak flow of the hydrograph is maintained for the storm duration. A triangular hydrograph results when the storm duration (t_d) is equal to t_c and represents the same peak flow as calculated by the Rational Method. When t_d is increased beyond t_c, the hydrograph takes a trapezoidal shape as shown in Figure 3.1 below. As t_d is lengthened, the peak flow decreases, but the volume of runoff, the area under trapezoid, increases. An allowable release rate is set (Q_a) based on pre-development conditions. The allowable release rate increases linearly until it reaches the receding limb of the inflow hydrograph. The t_d is varied incrementally until the storage volume (shaded gray area) is maximized. This method is normally an iterative process which can be done by hand or on a spreadsheet. Readily available software programs such as Bentley (Haestad) Pond Pack© Modified Rational Method "I" use this same methodology. Downstream analysis is not possible with this method as only approximate graphical routing takes place.

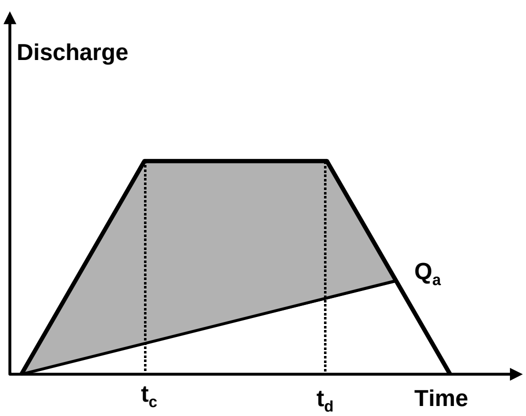


Figure 3.1 Modified Rational Definitions

3.1.4.2 Design Equations

The allowable release rate can be determined from:

$$Q_a = C_a\, i\, A \qquad (3.3)$$

where:

Q_a = allowable release rate (cfs)

C_a = predevelopment Rational Method runoff coefficient

i = rainfall intensity for the corresponding time of concentration (in/hr)

A = area (acres)

The Modified Rational Method should be used for basins with fairly homogeneous land use and flow paths. Consideration should also be given to increasing the C factor for higher intensity ($\geq$25 year) storms because infiltration and other abstraction losses have a proportionally smaller effect on runoff during such events. Care should be exercised in the calculation of the C factor and time of concentration used to determine the Q_a to avoid oversizing the outlet device and thus reducing available storage.

3.1.5 SCS METHOD

3.1.5.1 Application

The SCS method can be used for both the estimation of stormwater runoff peak rates and the generation of hydrographs for the routing of stormwater flows, thus it can be used for most design applications. It is assumed that most users of the SCS methodology will use a computer program such as HEC-HMS therefore this manual does not attempt to include the equations and concepts utilized as the ~~metholοlgy~~ methodology is adequately described in the HEC-HMS *User's Manual* and *Technical Reference Manual.*

3.1.5.2 Runoff Factor (CN)

The SCS method uses a combination of soil conditions and land uses (ground cover) to assign a runoff factor to each area. Soils data can be obtained from a site specific geotechnical report or from the County Soils Survey information available on-line at http://soils.usda.gov/.

Average antecedent soil moisture conditions (AMC II) are recommended for most hydrologic analysis. Table 3.8 gives standard curve number values for a range of land uses.

When a drainage area has more than one land use, a composite curve number can be calculated based upon percentages of land uses within a basin

Table 3.8 Runoff Curve Numbers[1]

Cover Description		Curve numbers for hydrologic soil groups			
Cover type and hydrologic condition	*Average percent impervious area[2]*	**A**	**B**	**C**	**D**
Cultivated Land:					
Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land:					
Poor condition		68	79	86	89
Good condition		39	61	74	80
Meadow:					
Good condition		30	58	71	78
Wood or forest land:					
Thin stand, poor cover		45	66	77	83
Good cover		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved; curbs and storm drains (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89

Table 3.8 Runoff Curve Numbers[1]

Cover Description		Curve numbers for hydrologic soil groups			
Cover type and hydrologic condition	*Average percent impervious area[2]*	**A**	**B**	**C**	**D**
Urban districts:					
Commercial and business	85%	89	92	94	95
Industrial	72%	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town house)	65%	77	85	90	92
1/4 acre	38%	61	75	83	87
1/3 acre	30%	57	72	81	86
1/2 acre	25%	54	70	80	85
1 acre	20%	51	68	79	84
2 acres	12%	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)		77	86	91	94

[1] Average runoff condition, and $I_a = 0.2S$

[2] The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

[3] CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combinations of open space cover type.

3.1.5.3 Urban Modification of the SCS Method

Connected Impervious Areas

The CNs for various land cover types were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

1. Pervious urban areas are equivalent to pasture in good hydrologic condition, and
2. Impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions do not appear applicable, refer to the graphical chart provided by SCS to compute a composite CN.

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, refer to the graphical charts provided by SCS to compute a composite CN.

3.1.5.4 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another within a watershed, through the various components of the drainage system. Time of concentration (t_c) is computed by summing all the travel times for consecutive components of the drainage conveyance system from the hydraulically most distant point of the watershed to the point of interest within the watershed.

Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = L/3600V \qquad (3.4)$$

where:

T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours

Sheet Flow

Sheet flow can be calculated using the following formula:

$$T_t = \frac{0.42\,(nL)^{0.8}}{60\,(P_2)^{0.5}(S)^{0.4}} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}(S)^{0.4}} \qquad (3.5)$$

where:

T_t = travel time (hr)
n = Manning roughness coefficient
L = flow length (ft),
P_2 = 2-year, 24-hour rainfall
S = land slope (ft/ft)

Shallow Concentrated Flow

After 50 to 100 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this type of flow can be graphically determined from information provided in the SCS manual or can be computed from the following equations.

Unpaved $V = 16.13(S)^{0.5}$ **(3.6)**

Paved $V = 20.33(S)^{0.5}$ **(3.7)**

where:

V = average velocity (ft/s)
S = slope of hydraulic grade line (watercourse slope, ft/ft)

After determining average velocity, use Equation 3.6 to estimate travel time for the shallow concentrated flow segment.

<u>Open Channels</u>

- Open channels are assumed to begin where surveyed cross sections have been obtained, where visible on aerial photographs, where identified by the local municipality, or where stream designations appear on USGS quadrangle sheets.
- Manning's Equation or water surface profile information can be used to estimate average flow velocity.
- Average flow velocity for travel time calculations is usually determined for bank-full elevation assuming low vegetation winter conditions.

Manning's Equation is

$$V = (1.49/n)\,(R)^{2/3}\,(S)^{1/2} \qquad (3.8)$$

where:

V = average velocity (ft/s)
R = hydraulic radius (ft) and is equal to A/P_w
A = cross sectional flow area (ft^2)
P_w = wetted perimeter (ft)
S = slope of the hydraulic grade line (ft/ft)
n = Manning's roughness coefficient for open channel flow

After average velocity is computed using Equation 3.10, T_t for the channel segment can be estimated using Equation 3.6.

<u>Limitations</u>

- Equations in this section should not be used for sheet flow longer than 50 feet for impervious surfaces.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate t_c.
- A culvert or bridge can act as detention structure if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert or bridge.

3.1.5.5 Hydrologic Stream Routing

The routing methods selected for use in Wichita Falls are the Modified Puls and the Muskingum-Cunge 8 point section methods.

3.1.6 SNYDER'S UNIT HYDROGRAPH METHOD

The Snyder method estimates a peak discharge and a time to the peak of the unit hydrograph. It also estimates shape parameters. Rainfall runoff models, such as HEC-1, will typically complete the unit hydrograph based on assumed parameters and relationships. Typically, two parameters are needed to develop the Snyder Unit Hydrograph:

- T_L - lag time and
- C_p - shape factor, also expressed as C_p640.

The following equation to compute lag time should be used:

$$\mathbf{T_L = C_T\ (L{*}L_{CA}/S^{0.5})^{0.38}} \qquad \textbf{(3.9)}$$

T_L = Lag Time (hr)
C_T = coefficient
L = hydraulic length of the watershed along the longest flow path (mi)
L_{CA} = hydraulic length along the longest watercourse from the point under consideration to a point opposite the centroid of the drainage basin (mi)
S = weighted slope of the basin (ft/mi), measured from the 85% to the 10% points along the longest stream path in the basin.

The value **C_T** is a dimensionless parameter that is typically assumed to be consistent for various areas of the state. It can be estimated from neighboring areas or calibrated for the whole or portions of the basin, and then applied to multiple subbasins within the watershed.

Note that there are multiple forms of the Snyder equation for **T_L.** Some use ft/ft for the slope and some do not include the slope at all. If a regional **C_T** value is used, verify that the same equation was used in the study within which it was developed. Values generally range from about 0.7 up to about 3.0 though values outside that range have been calibrated.

The shape factor **C_p** reflects the sharpness of the hydrograph. High values, up to about 500, reflect a rapidly responding basin with a sharp peaked hydrograph. Low values, such as 250, generally reflect a flatter, slow responding basin with a longer, flatter hydrograph. These values are generally divided by 640 and entered into HEC-HMS as the **C_p** value ranging from about 0.4 to 0.8.

3.2. Downstream Assessment

3.2.1 Introduction

The assessment should extend from the outfall of a proposed development to a point downstream where the discharge from a proposed development no longer has a significant impact on the receiving stream or storm drainage system. The assessment should be a part of the preliminary and final plans, and should include the following properties:

- Hydrologic analysis of the pre- and post-development on-site conditions
- Drainage path which defines extent of the analysis.
- Capacity analysis of all existing constraint points along the drainage path, such as existing floodplain developments, underground storm drainage systems culverts, bridges, tributary confluences, or channels
- Offsite undeveloped areas are considered as “full build-out” for both the pre- and post-development analyses
- Evaluation of peak discharges and velocities for three (3) 24-hour storm events
  - 2-year storm
  - 10-year storm; and

    - 100-year storm
- Separate analysis for each major outfall from the proposed development

Once the analysis is complete, the designer should ask the following three questions at each determined junction downstream:

- Are the post-development discharges greater than the pre-development discharges?
- Are the post-development velocities greater than the pre-development velocities?
- Are the post-development velocities greater than the velocities allowed for the receiving system?

These questions should be answered for each of the three storm events. The answers to these questions will determine the necessity, type, and size of non-structural and structural controls to be placed on-site or downstream of the proposed development.

3.2.2 Downstream Hydrologic Assessment

Common practice requires the designer to control peak flow at the outlet of a site such that post-development peak discharge is equal to or less than pre-development peak discharge. It has been shown that in certain cases this does not always provide effective water quantity control downstream from the site and may actually exacerbate flooding problems downstream. The reasons for this have to do with (1) the timing of the flow peaks, and (2) the total increase in volume of runoff.

Due to a site's location within a watershed, there may be very little reason for requiring flood control from a particular site. In certain circumstances where detention is in place or a master drainage plan has been adopted, a development may receive or plan to receive less that ultimate developed flow conditions from upstream. This might be considered in the detention needed and its influence on the downstream assessment. Any consideration in such an event would be with the prior approval of the City Public Works Director. This section outlines a suggested procedure for determining the impacts of post-development stormwater peak flows and volumes that are required as part of a developer's stormwater management site plan.

3.2.3 Methods for Downstream Evaluation

The downstream assessment is a tool by which the impacts of development on stormwater peak flows and velocities are evaluated downstream. The assessment should consider the zone of influence of the proposed development and shall extend from the outfall of the development to a point downstream where the discharge no longer has a significant impact upon the receiving stream or storm drainage system

Typical steps in a downstream assessment include:

1. Determine the outfall location of the site and the pre- and post-development site conditions.
2. Collect data for the stormwater facilities within the zone of influence, such as reviewing other studies and obtaining as-built plans. Based on this information, document whether or not the downstream facilities were designed for build out conditions for all property upstream and whether there are any know problems downstream such as road overtoppings, historical structure flooding, etc. If there are no such downstream problems and if the downstream systems are documented as being designed for build out conditions upstream, then this information shall be presented for the approval and consent of the City Public Works Director. If these criteria are not satisfied, then the assessment must continue.

3. Using a topographic map determine a preliminary lower limit of the zone of influence (approximately 10% point).
4. Using a hydrologic model determine the pre-development peak flows and velocities at each junction beginning at the development outfall and ending at the next junction beyond the “10% zone of influence” point. The 10% zone of influence can be considered to be the point where the drainage area controlled by the detention or storage facility comprises 10% of the total drainage area. For example, if a structural control drains 10 acres, the zone of influence ends at the point where the total drainage area is 100 acres or greater. Undeveloped off-site areas are modeled as “full build-out” for both the pre- and post-development analyses. The discharges and velocities are evaluated for the three design storms.
5. Change the land use on the site to post-development conditions and rerun the model.
6. Compare the pre- and post-development peak discharges and velocities at the downstream end of the model. If the post-developed flows are higher than the pre-developed flows for the same frequency event, or the post-developed velocities are higher than the allowable velocity of the downstream receiving system, extend the model downstream. Repeat steps 3 and 4 until the post-development flows are less than the pre-developed flows, and the post-developed velocities are below the allowable velocity. Allowable velocities are given in Table 3.13 in Section 3.3.4, Open Channels.
7. If shown that no peak flow increases occur downstream, and post-developed velocities are allowable, then the control of the flood protection volume can be waived by the City Director of Public Works.
8. If peak discharges are increased due to development, or if downstream velocities are erosive, one of the following options are required.
   - Provide an acceptable design to reduce the flow elevation and/or velocity through channel or flow conveyance structure improvements downstream; or
   - Design an on-site structural control facility such that the post-development flows do not increase the peak flows, and the velocities are not erosive, at the outlet and the determined junction locations.

3.3. HYDRAULICS

3.3.1 Streets

Gutter Flow

The City has chosen to calculate gutter depth using a straight crown cross section.

Design guidance on gutter flow hydraulics may be obtained from the Federal Highway Administration’s *Urban Drainage Design Manual*, HEC-22.

Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56/n]\ S_x^{5/3}\ S^{1/2}\ T^{8/3} \qquad (3.10)$$

where:

Q = gutter flow rate, cfs

S_x = pavement cross slope, ft/ft

n = Manning's roughness coefficient

S = longitudinal slope, ft/ft

T = width of flow or spread, ft

Manning's n Table

Table 3.9 Manning's n Values for Street and Pavement Gutters	
Type of Gutter or Pavement	Manning's n
Concrete gutter, troweled finish	0.014
Concrete gutter with Smooth adjoining pavement Rough (broom finish) adjoining pavement	 0.015 0.018
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

Stormwater Inlets

Inlets used for the drainage of pavement surfaces can be divided into three major classes:

- Grate Inlets – These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.
- Curb-Opening Inlets – These inlets are vertical openings in the curb covered by a top slab.
- Combination Inlets – These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

The City of Wichita Falls requires the use of curb type inlets. However, grate inlets may be allowed in certain design situations with the prior approval of the City Public Works Director. The City of Wichita Falls requires that any design using grate inlets must reduce that calculated capacity of the grate by 50% due to the probability of debris clogging the inlet.

Inlets may be classified as being on a continuous grade or in a sump. Overflow provisions shall be provided in sump locations to handle excess stormwater flows that may exceed curb height in the event of a storm exceeding the design conditions or if the inlet were to clog. These overflow provisions shall not adversely affect adjoining private property.

Design guidance for all inlet types of inlet hydraulics may be obtained from the Federal Highway Administration's *Urban Drainage Design Manual*, HEC-22, and from AASHTO's Model Drainage Manual.

3.3.2 Storm Sewer (Closed Conduit) Systems

Closed conduit systems may be composed of different lengths and sizes of conduits (system segments) connected by appointment structures (system nodes). Segments are most often circular pipe, but can be a box or other enclosed conduit. The following requirements shall be applied to the design of storm sewers:

- The minimum acceptable pipe size is 18" inside diameter
- Manholes or junction boxes shall be provided at all changes in horizontal direction or slope, changes in pipe diameters, or pipe intersections with a maximum spacing on long pipe runs of 1000 feet.
- Preformed wyes may be used only for single leads from an inlet to the main line.

- Approved piping materials for conduits are:

  (1) Reinforced Concrete Pipe ("RCP") with pipe class determined by depth of cover and loading conditions installed per City Details,
  (2) High Density Polyethylene ("HDPE") with a smooth interior may be used only when a minimum of 24" of cover from finish grade to the top of pipe is provided and the pipe shall be properly embedded in strict accordance with manufacturers' specifications with graded gravel. Any HDPE used within five feet (5') of a street pavement edge or under street sections shall be encased with flowable fill on all sides per City details.
  (3) Poly Vinyl Chloride ("PVC") pipe with a smooth interior surface is allowable. Pipe stiffness shall be a minimum of 46 with actual pipe class determined by depth of cover calculations with pipe not to exceed 5% deflection.

Capacity Calculations

A closed conduit may be under pressure or at other times the conduit may flow partially full; however, the usual design assumption is that the conduit is flowing full but not under pressure. Under this assumption the rate of head loss is the same as the slope of the pipe (S_f=S), in ft/ft.

The hydraulic capacity of storm drain pipes for gravity and pressure flows shall be determined by the following equation:

$$\mathbf{V = (1.486/n)\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(3.11)}$$

where:

V = mean velocity of flow, ft/s

R = the hydraulic radius, ft - defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/WP)

S = the slope of hydraulic grade line, ft/ft

n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$\mathbf{Q = (1.486/n)\, A\, R^{2/3}\, S^{1/2}} \qquad \mathbf{(3.12)}$$

where:

Q = rate of flow, cfs

A = cross sectional area of flow, ft^2

For pipes flowing full, the area is $(\pi/4)D^2$ and the hydraulic radius is D/4, so, the above equations become:

$$\mathbf{V = [0.590\, D^{2/3} S^{1/2}]/n} \qquad \mathbf{(3.13)}$$
$$\mathbf{Q = [0.463\, D^{8/3} S^{1/2}]/n} \qquad \mathbf{(3.14)}$$

where:

D = diameter of pipe, ft
S = slope of the pipe = S_f hydraulic grade line, ft/ft

The Manning's Equation can be written to determine friction losses for storm drain pipes as:

$$\mathbf{H_f = [0.453\, n^2 V^2 L]/[R^{4/3}]} \qquad \mathbf{(3.15)}$$

$$\mathbf{H_f = [(2.87\, n^2 V^2 L]/[D^{4/3}]} \qquad \mathbf{(3.16)}$$

$$\mathbf{H_f = [(185 n^2 (V^2/2g) L]/[D^{4/3}]} \qquad \mathbf{(3.17)}$$

where:

H_f = total head loss due to friction, ft (S_f x L)

n = Manning's roughness coefficient

D = diameter of pipe, ft

L = length of pipe, ft

V = mean velocity, ft/s

R = hydraulic radius, ft

g = acceleration of gravity = 32.2 ft/sec^2

Table 3.10 Manning's Coefficients for Storm Drain Conduits
(HEC 22, 2001)

Type of Culvert	Roughness or Corrugation	Manning's n
Concrete Pipe	**Smooth**	**0.013**
Concrete Boxes	**Smooth**	**0.013**
HDPE	**Smooth**	**0.010**
Polyvinyl chloride (PVC)	**Smooth**	**0.010**

***NOTE: The Manning's n values indicated in this table were obtained in the laboratory and are supported by the provided reference. These numbers should be considered as the best possible for the pipe type. Actual field values for culverts may vary depending on the effect of abrasion, corrosion, deflection, and joint conditions.**

Minimum Grades and Desirable Velocities

The minimum allowable velocity for closed conduits flowing full is 2.0 fps. The minimum slopes are therefore calculated by the modified Manning's formula:

$$\mathbf{S = [(nV)^2]/[2.208R^{4/3}]} \qquad \textbf{(3.18)}$$

where:

S = the slope of the hydraulic grade line, ft/ft

n = Manning's roughness coefficient

V = mean velocity of flow, ft/s

R = hydraulic radius, ft (area divided by wetted perimeter)

For circular conduits flowing full but not under pressure, R=D/4, and the hydraulic grade line is equal to the slope of the pipe. For these conditions Equation 3.20 may be expressed as:

$$\mathbf{S = 2.87(nV)^2/D^{4/3}} \qquad \textbf{(3.19)}$$

For a minimum velocity of 2.0 fps, the minimum slope equation becomes:

$$\mathbf{S = 11.48(n^2/D^{4/3})} \qquad \textbf{(3.20)}$$

where:

D = diameter, ft

Maximum Velocities

Maximum velocities in storm drains should not exceed 15 fps. However, the outfall velocity shall not exceed the velocity of the receiving channel for the same storm event.

Hydraulic Grade Line

All drainage plans prepared for review by the City shall include hydraulic grade lines indicated on the profile views for the system. The energy grade line (EGL) represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head. The 10-year storm hydraulic grade line for a closed conduit system shall be contained within the closed conduit system.

$$\mathbf{E = V^2/2g + p/v + z} \qquad \textbf{(3.21)}$$

where:

E	=	Total energy, ft
$V^2/2g$	=	Velocity head, ft (kinetic energy)
p	=	Pressure, lbs/ft^2
v	=	Unit weight of water, 62.4 lbs/ft^3
p/v	=	Pressure head, ft (potential energy)
z	=	Elevation head, ft (potential energy)

Bernoulli's Law expressed between points one (1) and two (2) in a closed conduit accounts for all energy forms and energy losses. The general form of the law may be written as:

$$V_1^2/2g + p_1/v + z_1 = V_2^2/2g + p_2/v + z_2 - H_f - \Sigma H_m \quad \textbf{(3.22)}$$

where:

H_f	=	Pipe friction loss, ft
ΣH_m	=	Sum of minor or form losses, ft

An in-depth presentation of the EGL and HGL calculations for a closed conduit system is provided in the Federal Highway Administration's ***Urban Drainage Design Manual***, HEC-22 to which reference is herein made.

Storm Drain Outfalls

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the stormwater. The procedure for calculating the hydraulic grade line through a storm drainage system begins at the outfall. Therefore, consideration of outfall conditions is an important part of storm drain design.

Storm drain outfalls shall include a headwall structure and a minimum 10 foot-long concrete apron with turned down footing at a transition into an earthen channel. However, the maximum velocity exiting the outfall cannot exceed the allowable velocity for the receiving channel. Refer to Table 3.12 for allowable velocities.

Several aspects of outfall design must be given serious consideration. These include the flowline or invert (inside bottom) elevation of the proposed storm drain outlet, tailwater elevations, the need for energy dissipation, and the orientation of the outlet structure.

The flowline or invert elevation of the proposed outlet should be equal to or higher than the flowline of the outfall. If this is not the case, there may be a need to pump or otherwise lift the water to the elevation of the outfall.

Energy dissipation may be required to protect the storm drain outlet. A minimum 10 foot-long concrete apron shall be installed at the storm drain outlet into another conveyance. The outfall velocity shall not exceed that of the receiving stream or the maximum velocities provided in Table 3.12. Protection may be required at the outlet to prevent erosion of the outfall bed and banks. Riprap aprons or energy dissipators should be provided if high velocities are expected.

The **orientation of the outfall** is another important design consideration. Where practical, the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce turbulence and the potential for excessive erosion. If the outfall structure cannot be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel, care must be taken to avoid erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed on the bank. Alternatively, an energy dissipator structure could be used at the storm drain outlet.

The tailwater depth or elevation in the storm drain outfall must be considered carefully. Evaluation of the hydraulic grade line for a storm drainage system begins at the system outfall with the tailwater elevation. The starting point for the hydraulic grade line determination should be either the design tailwater elevation or the average of critical depth and the height of the storm drain conduit, $(d_c + D)/2$, whichever is greater.

Coincidental Occurrence

If the outfall channel is a river or stream, it may be necessary to consider the joint or coincidental probability of two hydrologic events occurring at the same time to adequately determine the elevation of the tailwater in the receiving stream. The relative independence of the discharge from the storm drainage system can be qualitatively evaluated by a comparison of the drainage area of the receiving stream to the area of the storm drainage system. For example, if the storm drainage system has a drainage area much smaller than that of the receiving stream, the peak discharge from the storm drainage system may be out of phase with the peak discharge from the receiving watershed. Table 3.11 provides a comparison of discharge frequencies for coincidental occurrence for the 2-, 5-, 10-, 25-, 50-, and 100-year design storms. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system based on the expected coincident storm frequency on the outfall channel. For example, if the receiving stream has a drainage area of 200 acres and the storm drainage system has a drainage area of 2 acres, the ratio of receiving area to storm drainage area is 100 to 1. From Table 3.11 and considering a 10-year design storm occurring over both areas, the flow rate in the main stream will be equal to that of a five year storm when the drainage system flow rate reaches its 10-year peak flow at the outfall. Conversely, when the flow rate in the main channel reaches its 10-year peak flow rate, the flow rate from the storm drainage system will have fallen to the 5- year peak flow rate discharge. This is because the drainage areas are different sizes, and the time to peak for each drainage area is different.

Table 3.11 Frequencies for Coincidental Occurrences
(TxDOT, 2002)

Area ratio*	2-year design		5-year design	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	2	1	5
	2	1	5	1
1,000:1	1	2	2	5
	2	1	5	2
100:1	2	2	2	5
	2	2	5	5
10:1	2	2	5	5
	2	2	5	5
1:1	2	2	5	5
	2	2	5	5
Area ratio*	**10-year design**		**25-year design**	
	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	10	2	25
	10	1	25	2
1,000:1	2	10	5	25
	10	2	25	5
100:1	5	10	10	25
	10	5	25	10
10:1	10	10	10	25
	10	10	25	10
1:1	10	10	25	25
	10	10	25	25
Area ratio*	**50-year design**		**100-year design**	
	Main Stream	Tributary	Main Stream	Tributary

Table 3.11 Frequencies for Coincidental Occurrences (TxDOT, 2002)				
10,000:1	2	50	2	100
	50	2	100	2
1,000:1	5	50	10	100
	50	5	100	10
100:1	10	50	25	100
	50	10	100	25
10:1	25	50	50	100
	50	25	100	50
1:1	50	50	100	100
	50	50	100	100

*The Area ratio is the ratio of the overall drainage area of the receiving stream to the drainage area of the facility being evaluated.

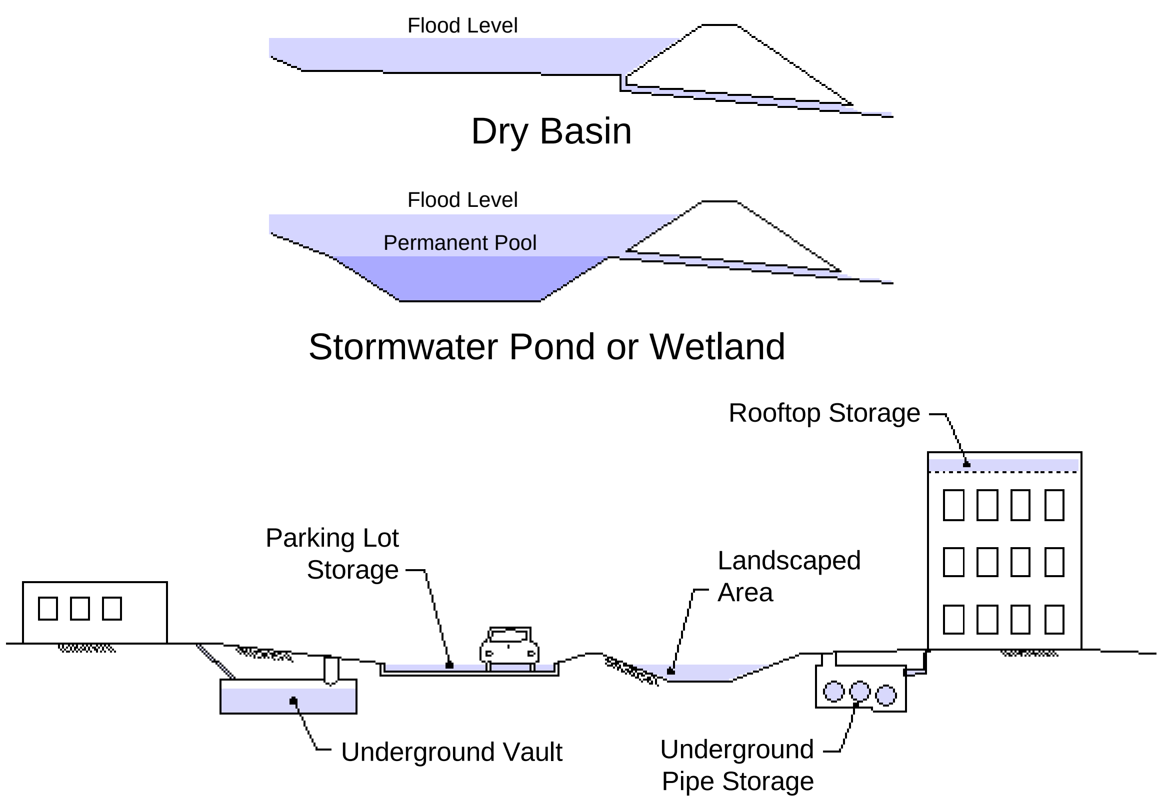
There may be instances in which an excessive tailwater causes flow to back up the storm drainage system and out of inlets and manholes, creating unexpected and perhaps hazardous flooding conditions. The potential for this should be considered. Flap gates placed at the outlet can sometimes alleviate this condition; otherwise, it may be necessary to isolate the storm drain from the outfall by use of a pump station.

3.3.3 Storage Design

General Storage Concepts

Storage of stormwater runoff within a stormwater management system is essential to providing the extended detention of flows for water quality protection and downstream streambank protection, as well as for peak flow attenuation of larger flows for flood protection. Runoff storage can be provided within an on-site system through the use of structural stormwater controls and/or nonstructural features and landscaped areas. Figure 2.1 illustrates various storage facilities that can be considered for a development site.

Figure 3.3.2 Examples of Typical Stormwater Storage Facilities



General Design Criteria

- Outlet rates and design storms are defined in the City's Stormwater Ordinance;
- HEC-HMS shall be used for large project designs (ponds with a drainage area of 200 acres or more);
- A primary outlet device and corresponding storage volumes must be designed for discharge of 2, 10 and 100 year storms;
- Stage-storage curve or table for the proposed storage facility shall be provided for all detention designs;
- Stage-discharge curve or table for all outlet control structures shall be provided for all detention designs;
- Ponds ~~may not be located~~ in existing drainage ways must be constructed with pilot channels to facilitate pond maintenance;
- A secondary outlet device (emergency spillway) shall be provided at all facilities and designed to pass the 100-year storm. A minimum of 6" of freeboard is required at all earthen dams or where erosion may occur from overtopping.
- Maximum design WSEL shall be at least 12" below the finish floor elevations of nearby structures
- All storage facilities must be able to drain by gravity;
- Earthen ponds shall have a minimum 0.5% slope across the flow line of the pond bottom and have minimum side slopes of 4:1 or flatter for residential detention ponds, and 3:1 or flatter for commercial detention ponds;
- Permanent vegetation shall be established in all earthen ponds. In those ponds to be accepted for City maintenance, the vegetation shall be a drought tolerant blend containing bermudagrass which is actively growing; covering the pond floor and side slopes with a 15 foot-wide belt around the top of berm; and having no bare spots greater that one square foot in size.
- Retention ponds shall have a minimum, normal conservation pool depth of two feet unless the pond is intended to be stocked in which case at least 25% of the conservation pool depth shall be five feet deep or more. Measures shall be provided to ~~insure~~ ensure that aerobic pond conditions are maintained. Only that volume existing above the normal conservation pool elevation shall be considered as storage. Discharge/outlet devices shall be designed as with a detention pond.
- The maximum depth of ponding in parking areas shall be 18"; however, the developer must clearly ~~indentify~~ identify the area of potential ponding over 12" deep with signage and assumes all liability for vehicle damage;
- Underground systems must ~~have~~ be designed with adequate manway access for cleaning and must be able to drain by gravity;
- Rooftop system designs must also include the signed and sealed certification by a currently licensed Texas professional engineer that the entire structure (primary and secondary framing) has been properly designed to accommodate the additional stormwater loadings and that the building envelope has been designed to properly protect the interior from water intrusion as a result of the rooftop detention.

General Storage Design Procedures

The design procedures for all structural control storage facilities are the same whether or not they include a permanent pool of water. In the latter case, the permanent pool or spillway elevation is taken as the "bottom" of storage and is treated as if it were a solid basin bottom for routing purposes.

It should be noted that the location of structural stormwater controls is very important as it relates to the effectiveness of these facilities to control downstream impacts. In addition, multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system, which could decrease or increase flood peaks in different downstream locations. Refer to Section 3.2 for the requirements of a Downstream Assessment.

3.3.4 Open Channels

Open Channel Types

The three main classifications of open channel types according to channel linings are vegetated, flexible, and rigid. Vegetated linings include grass with mulch, sod and lapped sod, and wetland channels. Stone riprap and some forms of flexible man-made linings or gabions are examples of flexible linings, while rigid linings are generally concrete or rigid block.

Vegetative Linings – Vegetation, where practical, is the most desirable lining for an artificial channel. It stabilizes the channel body, consolidates the soil mass of the bed, checks erosion on the channel surface, provides habitat, and provides water quality benefits.

Conditions under which vegetative cover only may not be acceptable include but are not limited to:

- High velocities
- Standing or continuously flowing water
- Lack of regular maintenance necessary to prevent growth of taller or woody vegetation
- Lack of nutrients and inadequate topsoil
- Excessive shade

Proper soil preparation, seeding, mulching, watering and any other work necessary to the establishment of healthy vegetation shall be provided. Channels shall not be accepted for City maintenance until vegetation is fully established on a minimum of 90% of the channel bottom and side slopes.

Pilot Channels - Man-made earthen channels with longitudinal slopes of less than 0.5% or that serve an area where consistent low flows are or may become prevalent shall be provided with a pilot channel per City Engineering standards.
Flexible Linings – Rock riprap, including rubble and gabion baskets, is the most common type of flexible lining for channels. It presents a rough surface that can dissipate energy and mitigate increases in erosive velocity. These linings are usually less expensive than rigid linings and have self-healing qualities that reduce maintenance. However, they may require the use of a filter fabric depending on the underlying soils, and the growth of grass, weeds, and trees may present maintenance problems.

Rigid Linings – Rigid linings are generally constructed of concrete and used where high flow capacity is required. Higher velocities, however, create the potential for scour at channel lining transitions and channel headcutting.

Manning's n Values

The choice of Manning's n value can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process.

Recommended Manning's n values for man-made channels with unlined, rigid, gabion and riprap linings are given in the following Table 3.12For natural channels, Manning's n values should be estimated using experienced judgment and information presented in publications such as the *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*, FHWA-TS-84-204, 1984, FHWA HEC-15, 1988, or Chow, 1959.

Table 3.12 Manning's Roughness Coefficients for Design of Open Channels

Lining Type	Manning's n	Comments	Max. Velocity
Grass Lined	0.035	.	6
Concrete Lined	0.015		15
Gabions	0.030		10
Rock Riprap	0.040	$n = 0.0395 d_{50}^{1/6}$ where d_{50} is the stone size of which 50% of the sample is smaller	10
Grouted Riprap	0.028	FWHA	10

Uniform Flow Calculations

Design Aids

This manual does not attempt to provide an exhaustive review of open channel design. Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration Hydraulic Design Standard manuals have numerous design charts or nomographs to aid in the design of rectangular, trapezoidal, and triangular open channel cross sections. In addition, design charts for grass-lined channels have been developed. Numerous software programs are available for calculating open channel flows. All submissions of design data to the City must clearly define which programs were used for analysis.

Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels. Most packaged drainage software utilizes these basic formulae:

$$\mathbf{v = (1.49/n)\, R^{2/3} S^{1/2}} \qquad \textbf{(3.23)}$$

$$\mathbf{Q = (1.49/n)\, A\, R^{2/3} S^{1/2}} \qquad \textbf{(3.24)}$$

$$\mathbf{S = [Q_n/(1.49\, A\, R^{2/3})]^2} \qquad \textbf{(3.25)}$$

where:

v = average channel velocity (ft/s)

Q = discharge rate for design conditions (cfs)

n = Manning's roughness coefficient

A = cross-sectional area (ft^2)

R = hydraulic radius A/P (ft)

P = wetted perimeter (ft)

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

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are assumed to be equal.

For a more comprehensive discussion of open channel theory and design, see the reference USACE, 1991/1994.

3.3.5 Culvert Design

Overview

A *culvert* is a short, closed (covered) conduit that conveys stormwater runoff under an embankment or away from the street right-of-way. The primary purpose of a culvert is to convey surface water, but properly designed it may also be used to restrict flow and reduce downstream peak flows.

The hydraulic and structural designs of a culvert must be such that minimal risks to traffic, property damage, and failure from floods prove the results of good engineering practice and economics. For economy and hydraulic efficiency, engineers should design culverts to operate with the inlet submerged during flood flows, if conditions permit. Design considerations include site and roadway data, design parameters (including shape, material, and orientation), hydrology (flood magnitude versus frequency relation), and channel analysis (stage versus discharge relation).

Design Criteria

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following list of design recommendations should be considered for all culvert designs as applicable.

- **Storm Frequency**
  - Refer to the Stormwater Ordinance. Culverts must pass a minimum of a 10 year event but are also subject to depth of water restrictions over the roadway.
- **Velocity Limitations**
  - Culverts are limited to velocities of 15 fps; however, the maximum allowable velocity of the downstream conveyance shall not be exceeded.
- **Debris Control**
  - In designing debris control structures, it is recommended that the Hydraulic Engineering Circular No. 9 entitled Debris Control Structures be consulted.
- **Headwater Limitations**
  - Governed by depth over roadway limitation of the Stormwater Ordinance.
- **Tailwater Considerations**
  - Flows must be kept in dedicated easements and at least 12" below downstream structures.
- **Culvert Inlets**
  - Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient Ke, is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 3.13.
- **Inlets with Headwalls**
  - Headwalls may be used for a variety of reasons, including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shortening the length of the required structure. Headwalls are required for all culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall.
  - This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.
- **Wingwalls and Aprons**
  - Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.
- **Improved Inlets**
  - Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance of the culvert.
- **Material Selection**
  - Reinforced concrete pipe (RCP), pre-cast and cast in place concrete boxes are recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. High-density polyethylene (HDPE) pipe may also be used if encased in flowable fill as specified by City details. Table 3.14 gives recommended Manning's n values for different materials.
- **Culvert Skews**
  - Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.
- **Weep Holes**
  - Weep holes are sometimes used to relieve uplift pressure on headwalls and concrete rip-rap. Filter materials should be used in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels through the fill

embankment. The filter materials should be designed as an underdrain filter so as not to become clogged and so that piping cannot occur through the pervious material and the weep hole.

- **Outlet Protection**
  - Culvert discharges shall be treated as a storm drain outfall.
- **Environmental Considerations**
  - Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.
- **Safety Considerations**
  - Roadside safety should be considered for culverts crossing under roadways. Guardrails or safety end treatments may be needed to enhance safety at culvert crossings. The AASHTO roadside design guide should be consulted for culvert designs under and adjacent to roadways.

Table 3.13 Inlet Coefficients

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

[1] Although laboratory tests have not been completed on K_e values for High-Density Polyethylene (HDPE) pipes, the K_e values for corrugated metal pipes are recommended for HDPE pipes.

* Note: "End Section conforming to fill slope", made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

Source: HDS No. 5, 2001

Table 3.14 Manning's n Values		
Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.012
	Good joints, rough walls	0.016
	Poor joints, rough walls	0.017
Concrete Box	Good joints, smooth finished walls	0.012
	Poor joints, rough, unfinished walls	0.018
High Density Polyethylene (HDPE)	Smooth Liner	0.011
	Corrugated	0.020
Polyvinyl Chloride (PVC)		0.011

Source: HDS No. 5, 2001

Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, 2001, HDS No. 5, pages 201 - 208.

Comprehensive Design Guidance

Comprehensive design discussions and guidance may be found in the Federal Highway Administration, National Design Series No. 5, document entitled Hydraulic Design of Highway Culverts, Second Edition, published in 2001. This document is available from the National Technical Information Service as Item Number PB2003102411*DL. (http://www.ntis.gov/search.htm) Search for this document using the Item Number.

3.3.6 Bridge Design

The following subsections present considerations related to the hydraulics of bridges. It is generally excerpted from Chapter 9 of the Texas Department of Transportation (TxDOT) Hydraulics Design Manual dated March 2009.

Design Recommendations

The design of a bridge should take into account many different engineering and technical aspects at the bridge site and adjacent areas. Bridges shall be designed to pass a 100-year event with 12" of freeboard between the calculated 100-year water surface elevation and the lowest structural member.

Loss Coefficients

The contraction and expansion of water through the bridge opening creates hydraulic losses. These losses are accounted for through the use of loss coefficients. Table 3.15 gives recommended values for the Contraction (K_c) and Expansion (K_e) Coefficients.

Table 3.15 Recommended Loss Coefficients for Bridges		
Transition Type	Contraction (K_c)	Expansion (K_e)
No losses computed	0.0	0.0
Gradual transition	0.1	0.3
Typical bridge	0.3	0.5
Severe transition	0.6	0.8

3.3.7 ENERGY DISSIPATION

General Criteria

Erosion problems at culvert, pipe and engineered channel outlets are common. Determination of the flow conditions, scour potential, and channel erosion resistance shall be standard procedure for all designs.

Energy dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream area channel system.

Energy dissipator designs will vary based on discharge specifics and tailwater conditions.

Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence.

Recommended Energy Dissipators

For many designs, the following outlet protection devices and energy dissipators provide sufficient protection at a reasonable cost:

- Riprap apron
- Riprap outlet basins
- Baffled outlets
- Grade Control Structures

Refer to the Federal Highway Administration Hydraulic Engineering Circular No. 1, ***Hydraulic Design of Energy Dissipators for Culverts and Channels***, for the design procedures of energy dissipators.

APPENDIX

DETENTION FACILITY AGREEMENT

STATE OF TEXAS §
COUNTY OF WICHITA §

KNOW ALL MEN BY THESE PRESENTS

This agreement made this the _____day of__________, 20__, by and between the City of Wichita Falls, Texas, hereinafter "City", acting by and through its City Manager, and ____________ ~~<<Owner>>~~, hereinafter "Owner". The term "Owner" shall include the above named owner, its successors and assigns.

W I T N E S S E T H:

WHEREAS, Owner is the owner of certain real property located in the corporate limits of the City, more fully described as ~~<<Legal Description>>~~____________________ and incorporated herein by reference (the "Owner Tract"); also known as ~~<<Street Address>>~~ _______________________ and,

WHEREAS, the Owner and the City desire that the development of the Owner Tract be in accordance with applicable storm water runoff regulations of the City, designed to promote the health, safety and general welfare of the citizens of the City;

NOW, THEREFORE, in consideration of the covenants contained herein and other good and valuable consideration, the receipt and sufficiency of which are hereby acknowledged, the City and Owner hereby agree as follows:

ARTICLE ONE

In consideration of the City plat and site approval of the Owner Tract, Owner hereby agrees to construct, maintain and repair a certain Detention Facility to be constructed on a portion of the Owner Tract, identified in Exhibits "A & B", attached hereto and made a part hereof for all purposes (the "Detention Facility").

ARTICLE TWO

Owner shall construct, maintain and repair the Detention Facility in a condition sufficient to provide storm water detention in accordance with the regulations of the City in effect on the date of this agreement. The Detention Facility and site grading shall be completed in accordance with Site and Grading Plans submitted by Owner and approved by City and shall be completed prior to

City's issuance of a Certificate of Occupancy for any building constructed on the Owner Tract. The Owner shall not allow any structure nor allow any modification within the limits of the Detention Facility which will adversely affect the performance of the facility. In the event the Owner shall subdivide the Owner Tract into two (2) or more parcels which use the Detention Facility, the owner of each resulting tract shall have the right to perform the maintenance necessary to retain the functionality of the Detention Facility. The maintenance obligation shall be a covenant running with the Owner Tract; provided, however, that in the event any owner conveys its interest in the Owner Tract, such conveying owner shall be released from any and all obligations under this agreement arising after the date of such conveyance. City shall have the right to inspect the Detention Facility at all reasonable times to ensure compliance with this agreement and Owner hereby grants City access to and across the Owner Tract for this purpose. In the event Owner fails to fully perform its obligations under this agreement to maintain the Detention Facility, and such failure continues for thirty (30) days after receipt by Owner of written notice from the City to Owner, City shall have the right to perform the necessary maintenance and receive full reimbursement from the Owner for the reasonable expenses incurred by City in connection therewith. Any notice, request, demand or other communication to be given to the Owner hereunder shall be in writing and shall be deemed to be delivered: if sent by mail, three (3) days following deposit in a U.S. Postal Service receptacle, postage prepaid, as certified mail, return receipt requested; or by (prepaid) national overnight courier service (e.g., FedEx, Airborne, UPS, Express Mail, etc.), addressed as set forth below:

To Owner: ____________________ ~~Owner Name~~
____________________ ~~Owner Address~~
____________________ ~~Owner City, State, Zip~~

To City: City of Wichita Falls, Texas
(Attn: City Manager)
1300 7th Street
P. O. Box 1431
Wichita Falls, Texas 76301.

Either party may, at any time, or from time to time, designate in writing a substitute address for that above set forth and thereafter all notices to such party shall be sent to such substitute address.

ARTICLE THREE

Owner agrees to indemnify and hold harmless the City, its officers, agents and employees from all suits, actions or claims, and from all liability and damages for any and all injuries or damages arising solely from or as a result of Owner's negligence in the performance or failure to perform its obligations under this agreement.

ARTICLE FOUR

Approval of this agreement by the City shall not create any financial obligation of the City, nor does such approval indicate approval of the appropriateness, adequacy or engineering of the Detention Facility.

IN TESTIMONY WHEREOF, the parties have caused this instrument to be executed on the date shown above.

Owner

By:

Name

Title

NOTE: PLEASE COMPLETE APPROPRIATE ACKNOWLEDGEMENT ONLY

STATE OF TEXAS §

CORPORATE ACKNOWLEDGMENT

COUNTY OF WICHITA §

BEFORE ME, the undersigned authority, a Notary Public in and for the State of Texas, on this day personally appeared ______________________________, known to me to be the person and officer whose name is subscribed to the foregoing instrument and acknowledged to me that same was the act of ______________________________, a ~~<<State of entity's formation>>~~ ______________________________ [corporation, limited liability corporation, or limited partnership], as the ~~<<title of officer or agent>>~~ ________________, and that he executed same for the purposes and consideration therein expressed and in the capacity therein stated.

GIVEN UNDER MY HAND AND SEAL OF OFFICE this the _____ day of ________________, 20____.

Notary Seal

____

Notary Public

STATE OF TEXAS §

INDIVIDUAL ACKNOWLEDGMENT

COUNTY OF WICHITA §

BEFORE ME, the undersigned authority, a Notary Public in and for the State of Texas, on this day personally appeared ______________________________, known to me to be the person whose name is subscribed to the foregoing instrument and acknowledged to me that he executed same for the purposes and consideration therein expressed.

GIVEN UNDER MY HAND AND SEAL OF OFFICE this the ______ day of ________________, 20____.

Notary Seal

____

Notary Public

City of Wichita Falls, Texas

By: ____________________________

Paul Menzies, Assistant City Manager ~~Darron Leiker, City Manager~~

STATE OF TEXAS §
COUNTY OF WICHITA §

BEFORE ME, the undersigned authority, a Notary Public in and for the State of Texas, on this day personally appeared ~~Darron Leiker~~ Paul Menzies, Assistant City Manager for the City of Wichita Falls, Texas, a municipal corporation, known to me to be the person and officer whose name is subscribed to the foregoing instrument and acknowledged to me that same was the act of said City of Wichita Falls, a Texas municipal corporation, and that he executed same for the purposes and consideration therein expressed and in the capacity therein stated.

GIVEN UNDER MY HAND AND SEAL OF OFFICE this the _____ day of _______________, 20____.

Notary Seal

____

Notary Public

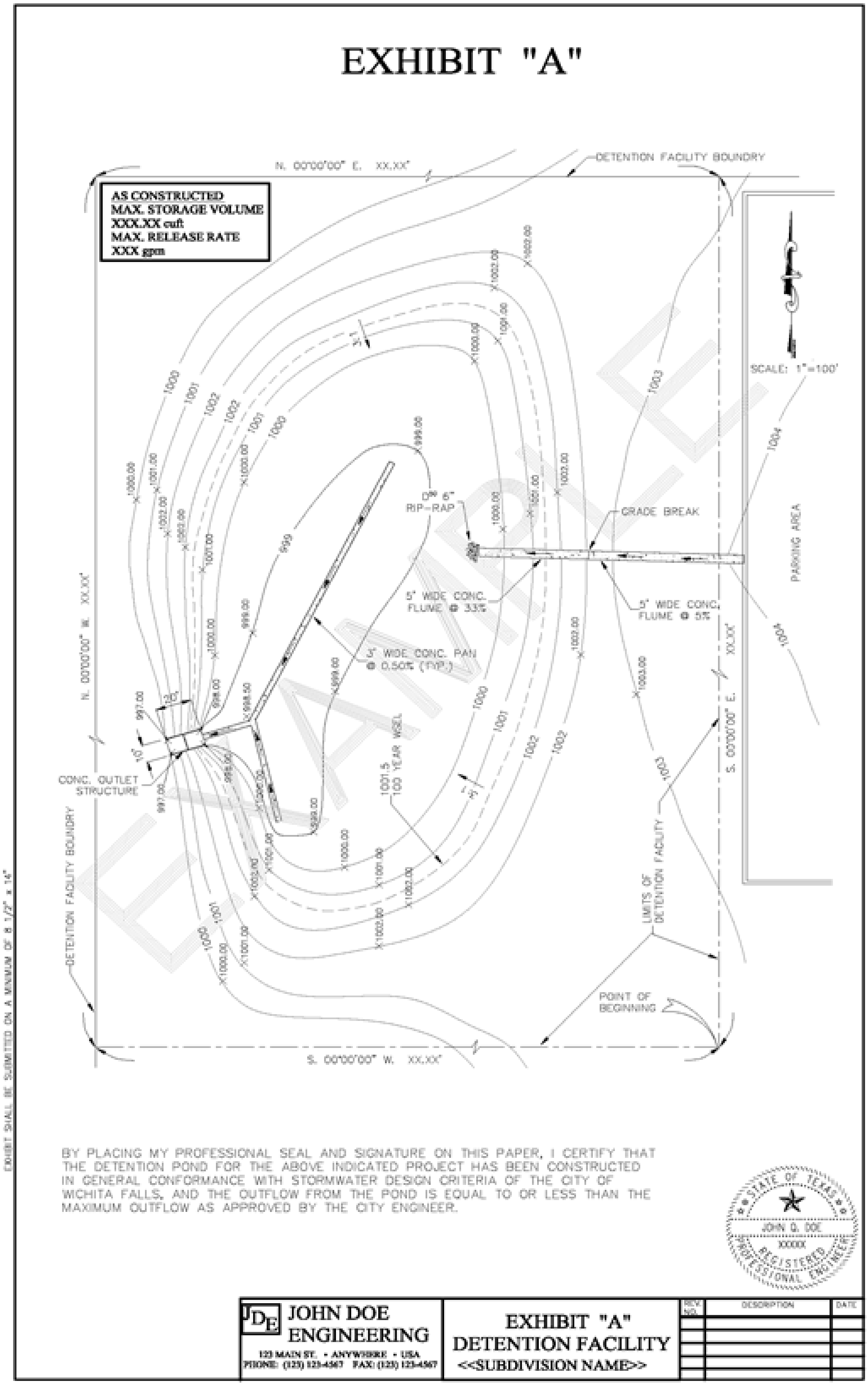


EXHIBIT "A"
AS CONSTRUCTED
MAX. STORAGE VOLUME
XXX.XX cuft
MAX. RELEASE RATE
XXX gpm
N. 00°00'00" E. XX.XX'
DETENTION FACILITY BOUNDRY
SCALE: 1"=100'
D50 6"
RIP-RAP
GRADE BREAK
5' WIDE CONC.
FLUME @ 33%
5' WIDE CONC.
FLUME @ 5%
3' WIDE CONC. PAN
@ 0.50% (TYP.)
PARKING AREA
1001.5
100 YEAR WSEL
CONC. OUTLET
STRUCTURE
DETENTION FACILITY BOUNDRY
LIMITS OF
DETENTION FACILITY
POINT OF
BEGINNING
N. 00°00'00" W. XX.XX'
S. 00°00'00" E. XX.XX'
S. 00°00'00" W. XX.XX'
EXAMPLE
EXHIBIT SHALL BE SUBMITTED ON A MINIMUM OF 8 1/2" x 14"
BY PLACING MY PROFESSIONAL SEAL AND SIGNATURE ON THIS PAPER, I CERTIFY THAT THE DETENTION POND FOR THE ABOVE INDICATED PROJECT HAS BEEN CONSTRUCTED IN GENERAL CONFORMANCE WITH STORMWATER DESIGN CRITERIA OF THE CITY OF WICHITA FALLS, AND THE OUTFLOW FROM THE POND IS EQUAL TO OR LESS THAN THE MAXIMUM OUTFLOW AS APPROVED BY THE CITY ENGINEER.
STATE OF TEXAS
JOHN Q. DOE
XXXXX
REGISTERED
PROFESSIONAL ENGINEER
JOHN DOE
ENGINEERING
123 MAIN ST. • ANYWHERE • USA
PHONE: (123) 123-4567 FAX: (123) 123-4567
EXHIBIT "A"
DETENTION FACILITY
<<SUBDIVISION NAME>>
REV. NO.
DESCRIPTION
DATE

EXHIBIT “B”

DETENTION FACILITY – FIELD NOTES

A TRACT OF LAND MORE COMMONLY KNOWN AS THE DETENTION FACILITY IS A PORTION OF <<LOT, BLOCK, SUBDIVISION NAME>>, AN ADDITION TO THE CITY OF WICHITA FALLS, TEXAS, AS RECORDED IN VOLUME XX, PAGE X, WICHITA COUNTY PLAT RECORDS. THE BOUNDARY OF THE DETENTION FACILITY IS MORE SPECIFICALLY DESCRIBED BY METES AND BOUNDS AS FOLLOWS:

BEGINNING AT A ½ INCH IRON ROD FOR THE MOST EASTERLY SOUTHEAST CORNER OF SAID <<LOT, BLOCK>>, FOR THE SOUTHEAST CORNER AND PLACE OF BEGINNING OF THIS BOUNDARY:

THENCE SOUTH 00° 00’ 00” WEST XX.XX FEET ALONG THE SOUTH LINE OF SAID <<LOT>>. TO A POINT FOR THE SOUTHWEST CORNER OF THIS BOUNDARY;

THENCE NORTH 00° 00’ 00” WEST XX.XX FEET ALONG THE EXISTING PROPERTY LINE TO THE NORTHWEST CORNER OF THIS BOUNDARY;

THENCE NORTH 00° 00’ 00” EAST XX.XX FEET ALONG THE EXISTING PROPERTY LINE TO THE NORTHEAST CORNER OF THIS BOUNDARY;

THENCE SOUTH 00° 00’ 00” EAST XX.XX FEET TO THE PLACE OF BEGINNING AND CONTAINING XX.XX ACRES OF LAND, MORE OR LESS.