Drainage Criteria Manual

The City of Lubbock, Texas

November 2019

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1.0 INTRODUCTION

In 2018, the City Council of Lubbock, Texas authorized an update of the Drainage Criteria Manual (DCM) that was originally published in 1997. The DCM was originally established in order to provide guidance and establish minimum criteria to be used for drainage analysis associated with the development/construction of land within the City. The ultimate goal of these criteria is to lessen the probability of flood damages due to changes in land use and storm flow routing that occurs as a result of land development/construction. The majority of flood damages have historically occurred in areas adjacent to playas and playa overflow routes; however, flood damages in all portions of the City have occurred.

Due to updates to technology and methodologies used to analyze hydrologic and hydraulic conditions, it was determined that an update of this manual was warranted. The update was intended to capture current best practices in the land development/construction industry, while continuing to provide sufficient criteria to decrease the likelihood of flooding due to land development/construction.

The majority of this manual is oriented toward the analysis and design of various drainage improvements likely to be encountered in real estate development/construction. The intent of the manual is to facilitate responsible development/construction while protecting the health, safety and welfare of the existing community. In return, Developers can expect a common basis for review of their drainage proposals, and a more expedited review of plats and plans.

This Drainage Criteria Manual is adopted and becomes effective on November 19, 2019. Freese and Nichols, Inc. coordinated the revisions to the 1997 criteria, along with Halff Associates Inc. and Hugo Reed and Associates, Inc. as part of the Stormwater Master Plan update.

1.1 DEFINITIONS

The following definitions are intended to provide clarification regarding some of the most commonly used terms within this manual.

Developer – The Developer is that person ultimately legally responsible for the submittal of all drainage plans and plats, and all of the information contained therein, and for the proposed development/construction being in compliance with the DCM. The Developer shall solicit the services of a Professional Engineer licensed in the State of Texas where required to provide sufficient analysis and design, and to provide seals and signatures for all items required to be sealed by a Professional Engineer.

Development - Any manmade change in improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, excavation, drilling operations or storage of equipment or materials.

Downstream Assessment – An analysis of the impact of a proposed development on downstream properties related to increased flooding and erosion potential due to upstream development/construction.

Professional Engineer – A person licensed to engage in the practice of engineering in the State of Texas.

Thoroughfare Plan – The Thoroughfare Plan is a graphic representation of the existing and proposed street system of the City of Lubbock and the surrounding area. The Lubbock Thoroughfare Plan represents the transportation element of the Lubbock Comprehensive Plan.

Master Thoroughfare Plan (MTP) – The Master Thoroughfare Plan is a long-range plan for major transportation improvements within the City of Lubbock. The purpose of the MTP is to identify the locations and types of roadway facilities that will be necessary to accommodate the ultimate development/construction of the City's thoroughfare network.

Master Drainage Plan (MDP) – The Master Drainage Plan provides an analysis of the hydrologic and hydraulic conditions within the City based on future fully developed (FFD) land use. The MDP is adopted and approved by the City Council and is intended to provide a consistent basis for future hydrologic and hydraulic analyses, including FFD land use assumptions through the City, FFD peak flows during various storm events, and playa peak water surface elevations. The MDP includes areas located within the limits of the City of Lubbock, as well as certain areas within the City of Lubbock's extraterritorial jurisdiction (ETJ).

Rule 12 Plat – A plat of a routine nature approved by the chairman of the Planning and Zoning Commission on recommendation of the Director of Planning with input from other departments and utilities. The name refers to Rule Number 12 in the Planning and Zoning Commission rules of order that governs such plats.

Rule 15 Plat – A plat that meets the conditions of a Rule 12 Plat and also includes a request for delay of water, sewer or paving, or a right-of-way/easement closure within the plat. The name refers to Rule Number 15 in the Planning and Zoning Commission rules of order that governs such plats.

1.2 PURPOSE AND NEED

The purpose of this criteria manual is to establish standard principles and practices for the analysis, design, and construction of storm drainage facilities within the City of Lubbock, Texas and within its extraterritorial jurisdiction (ETJ). The design factors, formulas, graphs, and procedures described in the following pages are intended to serve as guidelines for the solution of drainage problems involving the volume and rate of flow, method of collection, storage, conveyance, and disposal of stormwater and erosion protection from stormwater flows. Ultimate responsibility for actual design, however, remains with the Professional Engineer. Sound engineering judgment shall always be applied. Users of this manual should be knowledgeable and experienced in the theory and application of stormwater engineering. Any variance from the requirements of this manual may be proposed to the City Engineer or designee for consideration. Prior approval by the City Engineer of any variance shall be required in order for the development/construction project to proceed.

Creation of the original Drainage Criteria Manual was initiated in 1994 when the City Council authorized its development in order to lessen the probability of flood damages near playas, playa overflow routes, and areas of concentrated flows in flat terrain encountered during the development/construction process.

This manual updates the criteria from the original manual, which was adopted in 1997. New criteria are needed to reflect the changes that have occurred in community standards, technology, and environmental regulations that impact stormwater management. The primary motivation for this new manual is to promote responsible development and to guide the community in drainage policy and criteria so that new development does not increase flooding, erosion, or water quality problems.

1.3 ENABLING ORDINANCES

Based upon the recommendation of the City Engineer, Engineering Department, Stormwater Master Plan team and Stakeholder Committee, the Lubbock City Council has adopted the Lubbock Drainage Criteria Manual by resolution <Resolution No. & Date> The Subdivision Ordinances (Chapters 22, 30, and 38) of the Lubbock Code of Ordinances support the criteria contained herein. The Drainage Criteria Manual is applicable to hydrologic and hydraulic analysis, design and construction within the City of Lubbock and the City's ETJ.

1.4 MASTER DRAINAGE PLAN COORDINATION

The City of Lubbock has a Master Drainage Plan (MDP) that covers many areas of the City. The MDP is a living document that includes City developed or accepted hydrologic and hydraulic data to establish future fully developed watershed and flooding conditions. When available, this information is to be utilized to establish future fully developed condition regulatory data. Proposed changes to these models or data shall be submitted to the City for review prior to acceptance. The following data is available for some (not all) areas of the City:

- Hydrologic Data
  - Land use
  - Composite Curve Numbers
  - Subbasin Delineations
  - Future Fully Developed Discharges
  - Hydrologic Models
- Hydraulic Data
  - Playa initial water surface elevations
  - Playa peak water surface elevations
  - Overflow routes
  - Thoroughfare discharge capacity
  - Hydraulic Models

The MDP will be updated periodically to reflect new developments/construction within the City. It should be consulted during the development consultation meeting and the preliminary drainage analysis process for appropriate initial conditions and analysis assumptions.

1.5 ACKNOWLEDGEMENTS

A Stakeholder Committee made up of technical users and representatives from the Lubbock community provided review, comment, and input throughout development of this manual. This committee was created through the nomination of representatives of a spectrum of interests, education, experience, and expertise. In conjunction with Halff Associates Inc., Hugo Reed and Associates, Inc., and Freese and Nichols, Inc., those citizens who gave freely of their time and talent to enhance the content of the Drainage Criteria Manual are listed below.

Lubbock Chamber of Commerce Representative

Mr. Rey Carrasco, P.E., Kimley Horn and Associates, Inc.

Lubbock County Representative

Ms. Jennifer Davidson, P.E, CFM, Director of Public Works

Mr. Brent Hogan, Director of Sanitation

Texas Society of Professional Engineers South Plains Chapter Representatives

Mr. Butch Davis, P.E., Parkhill, Smith & Cooper, Inc.

Mr. Cory Dulin, P.E., AMD Engineering, LLC

West Texas Home Builders Association/Developer's Council Representatives

Mr. Chris Berry, Betenbough Homes

Mr. Thomas Payne, Epic Properties

Mr. Jason Swofford, P.E., Hugo Reed and Associates, Inc.

The staff provided personnel support and offered valuable experience-based input into the Drainage Criteria Manual development. Those City staff members that participated in the manual development are as follows:

Mr. Michael G. Keenum, P.E, CFM

Mr. John Turpin, P.E.

Mr. Michael McKay, P.E., CFM

Mr. Steven Nelson, P.E, CFM

2.0 DRAINAGE SUBMITTALS, REQUIREMENTS AND PROCESSES

2.1 INTENDED USE

The purpose of this chapter is to provide an overview of the required calculations and supporting data to be submitted to the City of Lubbock Engineering Department. The drainage review process will be discussed, as well as the individual submittal requirements for all types of submittals. This information is intended to guide all engineering calculations related to public stormwater infrastructure and the impact of development/construction within the City of Lubbock and its ETJ. This document shall be used to assist the Developer with the preparation of required submittal data for any type of development/construction disturbing an area equal to or greater than 0.5 acres within the City of Lubbock and its ETJ. Drainage submittal is not required for a single R1 lot within a previously studied subdivision.

2.2 APPLICABILITY

The Drainage Criteria Manual is applicable for development and redevelopment within the City of Lubbock and the City's ETJ. The criteria in this manual apply to any development, construction project or redevelopment that requires a plat, a re-plat or a building permit. Sites that do not meet the applicability requirements will not require drainage analyses.

New development or redevelopment which are located in critical or sensitive areas, or as identified through a watershed study or plan, may be subject to additional performance and/or regulatory criteria. Furthermore, these sites may need to utilize certain structural controls in order to protect a special resource or address certain drainage problems identified for a drainage area or watershed.

There may be times when this criteria cannot be met due to special circumstances and a variance may be considered under the review of a Professional Engineer and consideration by the City Engineer. A variance may be considered for specific and unique drainage problems. Often studies require unique computer modeling capabilities, specific professional expertise, an accounting for unusual circumstances or providing special interpretation of field measured data. A variance may also be considered in instances or circumstances where there is substantial room for creative and innovative solutions.

If an existing site has been cleared and/or graded within the prior five years of the date of the Developer's initial application submittal, the Developer shall consider the land conditions prior to the clearing and grading to be the existing site conditions.

ADOPTION OF STANDARDS

For projects which have an accepted drainage analysis, including phased developments/construction which have some existing constructed phases after the adoption of the drainage criteria in 1997, findings in accepted studies will remain valid. The applicability of the current drainage criteria is presented below in the Applicability of the Drainage Criteria Manual Adoption Language.

Preliminary and Final Drainage Analyses, as well as drainage design calculations accepted by the City of Lubbock after the adoption of the City's Drainage Criteria Manual on 1997 shall be considered valid when:

- *The proposed project is a phase of a multi-phase development/construction that has a valid preliminary plat.*
- *The drainage infrastructure of the proposed phase will connect directly to drainage infrastructure of a phase of the same development/construction with drainage infrastructure designed and constructed based on the standards in previous versions of the City's Drainage Criteria Manual.*
- *All proposed drainage studies submitted after the adoption date of this Drainage Criteria Manual not meeting the criteria above shall use the current Drainage Criteria Manual standards and will be valid for a period of time that is concurrent with the accepted preliminary or final plat for the project.*

If a proposed development increases the volume or changes the character of site runoff, a drainage analysis submittal will be required and the user should refer to the Preliminary Drainage Analysis Checklist.

For applicable sites, the building permit process shall require a drainage review of the Final Drainage Analysis to ensure that the site runoff is consistent with existing runoff patterns or has been appropriately addressed.

2.3 ACCEPTABLE SOFTWARE

The following table includes acceptable hydrologic and hydraulic modeling packages which are acceptable to the City of Lubbock:

Hydrology Software	Hydrologic Features	Hydraulic Features	2D Flow	Storm Drains
HEC-HMS	**X**			
HEC-RAS		**X**	**X**	
ICPR	**X**	**X**	**X**	**X**

The preferred computer software for hydrologic and hydraulic calculations, including open channels, culverts, closed conduit stormwater systems, and flow in streets and alleys, is Streamline Technologies Interconnected Channel Pond Routing (ICPR). In situations where a unit hydrograph method is required, the use of an acceptable computer software is required by the City. For FEMA submittals, refer to the FEMA website for a list of acceptable software programs.

For all types of analyses discussed above, the use of other computer software and/or hand calculations may be permitted on a case-by-case basis by the City Engineer. In these cases, model input and results shall be provided in a tabular format acceptable to the City in order for the City Engineer to be able to independently verify the results. It is the responsibility of the Developer to defend results that do not match closely with the City's results.

2.4 MINIMUM FINISHED FLOOR REQUIREMENTS

The City of Lubbock has established requirements for the finished floor elevations adjacent to playas, playa overflow routes, street and alley drainage, and open channels. FEMA also has established requirements for minimum finished floor elevations. For certain criteria listed below, the City has adopted higher standards that provide an increased level of safety above FEMA criteria. In instances where City criteria has a higher standard, City criteria will control. Both City and FEMA finished floor requirements are provided in Table 2.1 below.

Table 2-1: City of Lubbock Finished Floor Requirements

Drainage Feature	Minimum City of Lubbock FFD Finished Floor Elevation Requirements[1,2,3]
Overflow Playa	2' above lake overflow elevation OR 1' above 100-year water surface elevation, whichever is greater
Non-Overflow Playa	1' above 500-year water surface elevation
Detention/Retention Basin[4]	2' above 100-year water surface elevation (1' above basin crest elevation)
Channel	1' above 100-year water surface elevation
Street Drainage	18" above highest adjacent gutter elevation or 12" above top of crown
Alley Drainage	City Engineer approval required to use alleys to set finished floor elevations
Area Located within Special Flood Hazard Area (SFHA)	

[1] All elevations provided in table are based on FFD hydrologic conditions, unless otherwise noted

[2] If more than one category above applies to the proposed development/construction, that which results in the greatest minimum FFE shall be used to establish the minimum FFE.

[3] There are times when this criteria cannot be met, and the Developer's Engineer shall provide material for the City Engineer to consider a variance.

[4] Freeboard is for overtopping not for finished floor elevation determination.

2.5 SUBMITTAL REVIEW PROCESS

The City strongly recommends that the Developer schedule a development consultation meeting with the City Engineer during the planning phase of a proposed construction project or development. This meeting allows for a general discussion regarding the proposed development/construction and associated drainage, wherein the global plan can be discussed, potential issues identified, and submittal requirements established.

It is the responsibility of the Developer to provide all necessary calculations and designs described herein. The Developer shall provide the City the data, calculations, and designs necessary to demonstrate that the design does not adversely impact the surrounding or downstream property, and that it meets local, state, and federal rules, regulations, and requirements.

During the development/construction application process, there are numerous development/construction activities which are subject to review by the Engineering Department. Submittal of an application for each of these activities shall include the appropriate items from the checklists provided in the sections below. The checklists are general guidelines that are applicable to the majority of proposed developments/construction; however, they are not to be considered all-inclusive. The City Engineer may require information in addition to that included in the checklists below and may also exclude some of the information in the check lists if it is not applicable to a particular project. Submittal of a complete package with all required calculations and documentation will facilitate the efficient review and approval of the package. All hydrologic analyses submitted in support of a Preliminary or Final Drainage Plan shall be completed assuming future fully developed (FFD) conditions, unless otherwise specifically stated.

In instances where a Developer desires to request approval of alternate practices or methodologies, a variance request may be submitted to the City Engineer for review and consideration. A standard form is provided for variance requests. Variance requests shall include engineering calculation and justification for the requested modification to the standard practices and design criteria contained within this manual.

A listing of each of the possible applications is described below. Submittal forms are provided for each submittal type in the appendix.

Table 2-2: City of Lubbock Submittal Forms

Submittal Type	General Submittal Requirements	Notes
Redevelopment Permit Submittal	Downstream Assessment	Document that verifies proposed changes will not negatively impact existing drainage structures or surrounding properties
FEMA Letter of Map Change Submittal	FEMA Elevation Certificate with licensed surveyor or engineer signature and seal Completed MT-1 or MT-2 Forms per FEMA requirements Elevation certificate required on all new building construction Floodplain Development Permit	Application must receive acceptance and signature from the City's Floodplain Official prior to submittal to FEMA
Playa Cut/Fill Submittal	Certified As-Built Cut/Fill Plan Application Fee Plasticity Index Certified testing, compaction and density results Location maps for density samples	Application shall comply with Ordinance 38.07 and 38.08
Preliminary Drainage Analysis Checklist (Map)	FEMA SFHAs and City MDP floodplains, Available MDP information (listed in Section 1.4) Contours Sub-basin information for all contributing areas Flow Paths Playa elevation and overflow information Existing and Proposed Drainage Features Changes in impervious cover from MDP, Zoning or Comprehensive Plan Runoff rate and direction for all existing drainage features	General drainage concepts with sufficient data to determine compliance with City criteria Required for all plats except: A. Replat with no change to drainage patterns, impervious cover or drainage facilities B. Administrative or Minor Plat, with City Engineer concurrence Submit Concurrent with Preliminary Plat, Acceptance Required for Preliminary Plat Approval

Submittal Type	General Submittal Requirements	Notes
Preliminary Drainage Analysis Checklist (Plan)	Downstream Assessment (refer to Section 6.0)	If the proposed development will result in a change to the land use or impervious cover shown in the MDP and Future Land Use Plan, a composite hydrologic calculation will be required to confirm that the impervious cover of the overall drainage area to the playa will remain the same. If the impervious cover will be increased from that shown in the MDP, mitigation to offset the increase shall be proposed.
	FEMA SFHAs and City MDP floodplains, Available MDP information and needed LOMCs	
	Backup calculations and reports, including sufficient information for both existing and FFD hydrologic conditions to update the MDP.	Submit Concurrent with Preliminary Plat, Acceptance Required for Preliminary Plat Approval
	Limits and capacity calculations for playa overflow routes	
	Playa reclamation areas and calculations	
	Proposed mitigation measures for hydrologic or hydraulic impacts	
	Requested variances	
	Developer acknowledgement letter for compliance with all local, state and federal regulations	
	Engineer's Seal	
Final Drainage Analysis Checklist (Map)	Accepted Preliminary Drainage Map	Acceptance required for Final Plat Approval
	Updates to preliminary drainage map	Reference Section 2.4 for finished floor requirements
	Required finished floor elevations	

Submittal Type	General Submittal Requirements	Notes
Final Drainage Analysis Checklist (Plan)	Accepted Preliminary Drainage Plan	Calculations for 2-, 25- and 100-year events
	Updates to Preliminary Drainage Plan	
	Floodplain Development Permit	Capacities shall be calculated based on flow depths on the lower side of streets where curb elevations differ by more than two-tenths of a foot (0.2’) at a transverse section. Peak flow depths during the 100-year event shall be limited to 12 inches above the gutter elevation.
	Supporting Calculations for all analyses	
	Calculations at each sub-basin, offsite drainage and points of confluence	
	Street and alley 2-year and 100-year flow information and maximum capacity	
	Drainage channel 100-year flow, flow depth and freeboard, supporting calculations	Normal depth and uniform flow calculations may be performed for minor ditches, carrying less than 20 cfs in 100-year discharge
	Storm sewer pipe and inlet calculations	
	Culvert sizing, flow and capacity information	Other items may be required by the City Engineer, due to unusual conditions specific to a particular project.
	Proposed discharges to an unstudied playa	
	Sediment and erosion control BMPs	
	Requested variances	
	Developer acknowledgement letter for compliance with all local, state and federal regulation	
	Engineer's Seal	

Submittal Type	General Submittal Requirements	Notes
Drainage Improvements Documentation Checklist	Digital and hard copy submittals Plan and profile for all drainage, paving, roadway and alley elements. Hydraulic grade lines Requested variances Indicate “as constructed” elevations, alignments, sizes, materials, and other pertinent information as may be required to reflect conditions in-field after project completion.	Minor drainage elements less than 75' in length do not require submittal
Drainage Improvements Documentation	Final drainage map and analysis shall be updated to indicate any changes, such as revised minimum recommended finish floor elevations, necessary as a result of actual construction. Engineer's Seal	
Variance Request	Identification of specific requirement and its location in the DCM for which a variance is requested Detailed explanation of reason(s) for request for variance, including all necessary backup calculations Explanation of ramifications to overall design of variance being requested. Include hydrologic and hydraulic implications on and offsite	City Engineer will be point of contact for all variance requests
Information Request/Update		

2.6 MAINTENANCE AND EASEMENTS

Maintenance of stormwater facilities, including playas, is required to ensure that they function as designed. Maintenance considerations shall be included during the final design phase of a proposed development/construction, and easements shall be accounted for in all proposed land development/construction designs.

2.6.1 Facilities Maintenance Agreements

All drainage improvements constructed within a development/construction, and any existing or natural drainage systems to remain in use, shall require a Facilities Maintenance Agreement that identifies responsible parties for maintenance. Both public and private maintenance responsibility shall be negotiated between the City and the Developer and documented in the agreement. The agreement shall be written such that it remains in force upon sale or transfer of the property. The Facilities Maintenance Agreement shall be complete and accepted by the City prior to the acceptance of final construction plans.

Publicly maintained facilities are those that will be maintained in perpetuity by the City. These include facilities located within dedicated easements and constructed to the standards contained in this manual. Privately maintained facilities are those which will be maintained either by the Developer, or by a private landowner or HOA who takes over ownership of the facility from the Developer. These include facilities located on private property that only handle private water, and detention or retention ponds, dams and retaining walls adjacent to channels, and other stormwater controls which collect public water, as well as drainageways not constructed to City standards but which convey public water.

All public and private constructed drainage improvements shall be wholly contained within a drainage easement and designed and constructed in accordance with this manual.

2.6.2 Maintenance Access/Easement Requirements

Adequate maintenance access and easements shall be provided for all stormwater facilities. This includes both physical space for access, as well as appropriate easements. Maintenance of stormwater facilities may include mowing vegetation, repairing facilities, addressing erosion and/or sedimentation problems, and maintaining the design hydraulic capacity and function of the existing drainage improvements as shown on the approved development/construction plans for the development/construction.

Easements for storm drainage facilities shall be provided at locations containing proposed or existing drainageways and playas or reservoirs. Where natural channels exist, the minimum right-of-way will be dedication of the floodway. Where a man-made channel is proposed the entire channel, including maintenance access will be dedicated.

All proposed development/construction shall comply with the following maintenance access and easement requirements:

Maintenance easements shall be shown on plat or dedicated by separate instrument if they fall outside the plat boundary.

Maintenance easements shall be of sufficient width to allow ingress and egress of maintenance equipment and personnel necessary to properly maintain the stormwater facility (refer to specific easement width requirements below).

The Developer shall provide a maintenance plan for any detention/retention facility as part of the design.

The maintenance plan shall indicate the ingress and egress locations to enter and maintain the detention/retention facility, maintenance roles and responsibilities, contact information for the party responsible for the maintenance, and a maintenance schedule.

2.6.3 Drainage Easements

<u>CHANNEL</u>

Easements for storm drainage facilities shall be provided at locations containing proposed or existing drainageways and playas or reservoirs. Where natural channels exist, the minimum right-of-way will be dedication of the floodway. Where a man-made channel is proposed the entire channel, including maintenance access will be dedicated.

Where drainage easements for surface conveyance are incorporated into the final plat, such easements shall not be infringed upon by buildings or other surface features that will reduce the conveyance capacity within the easement. Easements will be evaluated from the standpoint of required capacity, outlet conditions, alignment, inlet conditions, flow velocity, and effects on the water surface profile upstream and downstream of the proposed easement. Drainage easements must wholly contain the water surface.

Storm drainage easements along proposed or existing man-made channels shall provide sufficient width for the required channel, as well as clearance from fences and space for utility poles, while still allowing the ingress and egress of maintenance equipment.

A drainage easement shall be required in cases where an upstream property owner discharges onto a downstream property that does not have sufficient capacity to convey stormwater with its existing infrastructure. If a permanent drainage easement cannot be obtained, but the downstream property owner is amenable to temporary stormwater discharges from the upstream property owner until downstream infrastructure is constructed, a temporary easement shall be written documenting the agreed-upon discharge allowances.

STORM DRAIN

The location of the storm drain pipe shall be within the City's street right-of-way whenever possible. For storm drain pipe not located within the street right-of-way, the owner, Developer or other responsible party shall provide an offsite easement so that the City can have access for maintenance purposes. Additional drainage easement criteria is located in Section 4.3.2.

PLAYAS

The location for drainage easements associated with playa lakes, whether overflow or non-overflow will be based on the limits of storage required in accordance with Chapter 5, Playas.

2.7 OTHER AGENCY PERMITS

It is the responsibility of the Developer to confirm that the proposed development/construction complies with all applicable federal, state and local regulations, and that all required permits are obtained prior to commencement of development/construction. These may include, but are not limited to, dam safety, water rights, and environmental regulations. A letter signed by the Developer must be with the preliminary and final drainage plans acknowledging that the proposed facilities comply with all applicable federal, state and local regulations.

3.0 HYDROLOGY

3.1 GENERAL

This section contains the minimum hydrologic design criteria required for the design of storm drainage infrastructure. The design factors, formulas, procedures and figures described are intended to serve as guidelines. Responsibility for the actual design remains with the Developer. Deviation from the requirements of these standards shall be approved by the City Engineer. All hydrologic analyses within the City of Lubbock shall be completed assuming future fully developed (FFD) conditions, unless otherwise specifically stated.

Preliminary and Final Drainage Analyses, as well as drainage design calculations accepted by the City of Lubbock after the adoption of this Drainage Criteria Manual shall be considered valid when:

A. The proposed project is a phase of a multi-phase development that has a valid preliminary plat.

B. The drainage infrastructure of the proposed phase will connect directly to drainage infrastructure of a phase of the same development with drainage infrastructure designed and constructed based on the standards in previous versions of the City's Drainage Criteria Manual.

3.1.1 Required Design Frequencies

The required design frequencies for each type of design feature discussed in this Drainage Criteria Manual are provided in Table 3.1 below.

Table 3-1: Required Design Frequencies by Design Type

Design Type	**Minimum Design Frequency Requirements***		
Unpaved Alley	100-year		
Paved Alley	100-year		
Storm Drains (Inlets on Grade)	5-year		
Storm Drains (Inlets in Sump)	25-year**		
Channels	100-year		
Playa Lake (Overflow)	100-year		
Playa Lake (Non-Overflow)	500-year		
Bridges	100-year		
Streets	100-year		
Flumes	100-year		
Culverts			
Street Classification***	**Up to 1/2 Square Mile Drainage Area**	**Over 1/2 Square Mile Drainage Area**	**Playa Overflow Route Included**
Type R-1A Residential	25-year	50-year	100-year
Type R-1 Residential	25-year	50-year	100-year
Type R-2 Residential	25-year	50-year	100-year
Type C-1 Collector	25-year	50-year	100-year
Type I Industrial	50-year	100-year ****	100-year ****
Type T-1 Thoroughfare	50-year	100-year ****	100-year ****
Type T-2 Thoroughfare	50-year	100-year ****	100-year ****
Type E-1 Expressway (Including Frontage Roads)	50-year	100-year	100-year
Type E-2 Expressway (Including Frontage Roads)	100-year	100-year	100-year

Source: City of Lubbock criteria.
*Maximum street flow depths shall meet the requirements of Chapter 4.
**Positive overflow shall be provided for the 100-year event.
*** Refer to current City of Lubbock *Master Thoroughfare Plan (MTP)* for definitions of street classifications. The level of protection listed will govern where topographic conditions, inlet and outlet conditions and street profile grades permit the installation of the appropriate culvert dimensions. Where conditions are not conducive to achieving the listed level of protection, Developer shall coordinate with the Engineering Department to determine an acceptable level of protection.
****Outside lane, on upstream side, can be inundated by culvert headwater.

3.1.2 Time of Concentration

Regardless of the hydrologic method used to determine peak discharge for a given design or study, the time of concentration (t_c) for the drainage basin shall be calculated using either the SCS method or Kerby-Kirpich method.

SCS Method

The SCS method (SCS TR-55 1986) separates the flow through the drainage area into sheet flow, shallow concentrated flow, and open channel flow. The t_c is the sum of travel times for these three types of flow. The time of concentration overall flow path and sheet flow path shall be shown on the submittal.

A. Sheet Flow: The maximum allowable length for sheet flow is 300’ for undeveloped drainage areas, and 100’ for developed areas. When selecting the Manning’s roughness coefficient for sheet flow, consider cover to a height of about 0.1’. This is the only part of the plant cover that will obstruct sheet flow. The T_t in minutes for sheet flow is determined using the following equation:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}}$$

Where:

T_t = travel time (hr)

n = Manning’s roughness coefficient as shown in Table 3.2 (dimensionless)

L = sheet flow length (ft)

P_2= 2-year, 24-hour rainfall (2.80 in)

S = slope of hydraulic grade line, ground slope (ft/ft)

Table 3-2: Sheet Flow Manning's Roughness Values

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils	
Residue cover less than 20%	0.06
Residue cover greater than 20%	0.17
Grass	
Short Prairie Grass	0.15
Dense Grasses	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods	
Light Underbrush	0.40
Dense Underbrush	0.80

Source: Table 3-1 SCS TR-55 1986.

B. Shallow Concentrated Flow: Shallow concentrated flow begins where sheet flow ends. A projected slope should be established along the flow line for the shallow concentrated flow length. The T_t in minutes for shallow concentrated flow is determined by the following equation:

$$T_t = \frac{L}{3600v}$$

Where:

T_t = travel time (hr)

L = shallow concentrated flow length (ft)

S = slope of hydraulic grade line, ground slope (ft/ft)

v = velocity (ft/s)

velocity in unimproved (unpaved) areas $=16.1345*S^{0.5}$

velocity in improved (paved) areas $=20.3283*S^{0.5}$

Table 3-3: Shallow Concentrated Flow Velocity Based on Slope

Slope [ft/ft]	Velocity for Unimproved [ft/s] (Unpaved Areas)	Velocity for Improved [ft/s] (Paved Areas)
0.001	0.510	0.643
0.002	0.722	0.909
0.003	0.884	1.113
0.004	1.020	1.286
0.005	1.141	1.437
0.006	1.250	1.575
0.007	1.350	1.701
0.008	1.443	1.818
0.009	1.531	1.929
0.010	1.613	2.033
0.012	1.767	2.227
0.014	1.909	2.405
0.016	2.041	2.571
0.018	2.165	2.727
0.020	2.282	2.875
0.025	2.551	3.214
0.030	2.795	3.521
0.035	3.018	3.803
0.040	3.227	4.066
0.045	3.423	4.312
0.050	3.608	4.546
0.060	3.952	4.979
0.070	4.269	5.378
0.080	4.564	5.750
0.090	4.840	6.098
0.100	5.102	6.428

C. Open Channel Flow: Open Channel Flow occurs when the runoff is located within a defined channel, street section, or in some cases, a closed storm system. The T_t for open channel flow is determined using the following equation:

$$T_t = \frac{L}{3600V}$$

$$v = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Where:

T_t = travel time (hr)

v = average velocity (ft/s)

R = A/P = hydraulic radius (ft)

A= cross sectional flow (ft^2)

P=wetted perimeter (ft)

S = slope of the hydraulic grade line, channel slope (ft/ft)

n = Manning's roughness coefficient (dimensionless)

Kerby-Kirpich Method

The Kerby-Kirpich method offers an alternative means of calculating the time of concentration for a watershed (TxDOT Hydraulic Design Manual 2016). With this method, the overall time of concentration is divided into two components: overland flow and channel flow. The Kerby equation is used to calculate overland flow, and the Kirpich equation is used to calculate channel flow.

This method is most appropriate for watersheds where the overland flow component of the time of concentration is anticipated to be an important component of the overall time of concentration. The maximum overland flow length allowed by the Kerby method is 1,200 feet, which is considerably longer than the maximum allowed by the SCS method. There is no shallow concentrated flow component to the Kerby-Kirpich method.

The Kerby-Kirpich method for estimating t_c is applicable to watersheds ranging from 0.25 square miles to 150 square miles, with main channel lengths between 1 and 50 miles, and main channel slopes between 0.002 and 0.02 (ft/ft) (Roussel et al 2005).

The main channel slope is computed as the change in elevation from the watershed divide to the watershed outlet divided by the curvilinear distance of the main channel (primary flow path) between the watershed divide and the outlet.

The Kerby equation for calculation of time of concentration for overland flow is:

$$t_{ov} = K(LN)^{0.467}\, S^{-0.235}$$

Where:

t_{ov}= overland flow time of concentration (min)

K = units conversion coefficient, in which K=0.828 for traditional units and K=1.44 for SI units

L = the overland flow length (in feet or meters as dictated by K)

N = a retardance coefficient as shown in Table 3.4 (dimensionless)

S = the slope of terrain conveying the overland flow (ft/ft)

Table 3-4: Kerby Equation Retardance Coefficient Values

Generalized Terrain Description	Dimensionless Retardance Coefficient (N)
Pavement	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough packed surfaces	0.20
Pasture, average grass	0.40
Deciduous forest	0.60
Dense grass, coniferous forest, or deciduous forest with deep litter	0.80

Source: Table 4-5 TxDOT Hydraulic Design Manual 2016.

The Kirpich equation for calculation of time of concentration for channel flow is:

$$t_{ch} = KL^{0.770}\, S^{-0.385}$$

Where:

t_{ch}= channel flow time of concentration (min)

K = units conversion coefficient, in which K = 0.0078 for traditional units and K = 0.0195 for SI units

L = the channel flow length (in feet or meters as dictated by K)

S = the slope of terrain conveying the main channel flow (ft/ft)

Alternative methods for calculating the time of concentration may be presented to the City on a case by case basis if deemed more appropriate.

Regardless of methodology used, the minimum total time of concentration for any residential area shall be 15 minutes.

3.2 METHODOLOGY

There are numerous methods available for rainfall-runoff computations on which the design of stormwater drainage and flood control plans may be based. For the City of Lubbock, there are two methods which will be considered acceptable for design and analysis within certain, specified limits:

A. Rational Method

The Rational Method may be used to determine the runoff generated from a property when the contributing drainage area at the outfall from the proposed development (and any contributing areas upstream of the proposed development) is less than 160 acres.

B. Unit Hydrograph Method

A unit hydrograph method shall be used to determine the runoff generated from a property with a contributing drainage area greater than 160 acres. The City Engineer may require developments with contributing drainage areas less than 160 acres to use a unit hydrograph method.

3.2.1 Rational Method

The Rational Method formula for computing peak runoff rates is as follows:

$$Q = C\ i\ A$$

Where:

Q = runoff rate (cfs)

C = runoff coefficient (dimensionless)

i = rainfall intensity (in/hr)

A = drainage area (acre)

Runoff Coefficient

The proportion of the total rainfall that will reach the design point depends on the imperviousness of the surface, the surface slope, the ponding characteristics of the area and the design storm event. Future fully developed (FFD) runoff coefficients shall be based on the future land use map, which shall serve as a basis for information stored in the MDP. If the proposed land use differs from that shown in the MDP, the Developer may be required to submit sufficient information for the City to be able to update the MDP and a downstream assessment may be required.

Table 3.5 provides guidelines for runoff coefficients for typical land use within the City; however, a weighted runoff coefficient may be used for the design if it is more representative of site conditions.

A lower runoff coefficient may be used if sustainable elements, as defined in Section 9 of this manual, are included in the design. The Developer shall notify the City Engineer of the design intent and provide the necessary data, calculations and design to support the desired runoff coefficient. All sustainable designs are subject to approval by the City Engineer.

Table 3-5: Guidelines for Runoff Coefficients

Classification	**Average Percent Impervious Area**	**2-10 Year Slope**		**25-Year Slope**		**50-Year Slope**		**100-Year Slope**	
		0-2%	**2-7%**	**0-2%**	**2-7%**	**0-2%**	**2-7%**	**0-2%**	**2-7%**
1-Acre Residential	20	0.37	0.44	0.41	0.49	0.44	0.53	0.46	0.56
Single Family Residential	49	0.53	0.59	0.59	0.64	0.64	0.71	0.66	0.74
Duplex/ Townhome	69	0.65	0.68	0.72	0.75	0.78	0.82	0.81	0.85
Apartments	75	0.69	0.71	0.76	0.78	0.83	0.85	0.86	0.89
Commercial*	99	0.82	0.83	0.90	0.91	0.98	1	1	1
IDP Industrial Park	79	0.70	0.72	0.73	0.76	0.79	0.83	0.82	0.86
Public Use (e.g., schools)	36	0.46	0.52	0.51	0.57	0.55	0.62	0.58	0.65
Open Space/Parks	7	0.29	0.38	0.32	0.42	0.35	0.46	0.36	0.48
Playas (regulatory boundary)	100	1	1	1	1	1	1	1	1

[1] The City Engineer can allow, at his/her option, that a lower percent impervious area be used for proposed property. Lower runoff coefficients, or lower percentage imperviousness, must be justified by landscaping plans and construction of those landscaping plans.

*Property zoned as Land Use Code Garden Office shall use Commercial classification.

Intensity-Duration-Frequency Calculations

The rainfall intensity (i) shall be based on the Texas Department of Transportation Hydraulic Design Manual, dated July 2016. The equation used to determine the intensity values for various storm events and durations is provided below:

$$i = \frac{b}{(T_c + d)^e}$$

Where:

i = rainfall intensity (in/hr)

T_c = time of concentration (min)

Refer to the Texas Department of Transportation (TxDOT) Intensity Duration Frequency coefficient spreadsheet (EBDLKUP.xls – current version), linked from the TxDOT Hydraulic Design Manual, Section 12: Rational Method, Rainfall Intensity subsection for for the appropriate coefficients to use to calculate rainfall intensity for various return events.

3.2.2 Unit Hydrograph Method

The use of a unit hydrograph method shall be based upon standard and accepted engineering principles normally used in the profession subject to the approval of the City Engineer. The acceptable method is the Soil Conservation Service (SCS) Unit Hydrograph Method. Alternative unit hydrograph methodologies may be proposed by the Developer and approved or denied by the City Engineer on a case by case basis.

Circumstances that may require the use of a unit hydrograph method, even if the contributing drainage area is less than 160 acres, include open channels, playa lakes, playa overflows, floodplain reclamation, or creation of regional detention/retention facilities. The City requires FFD conditions be used for all post-development modeling. NFIP peak flow rates shall only be used in specific situations, as the flow rates are generally based upon existing watershed conditions. Situations which allow the use of NFIP peak flow rates include cases of fully developed subbasins, and urban redevelopment projects that will not involve changes that would affect the hydrologic calculations.

Curve Number

Selection of a curve number for a drainage area is based on the hydrologic soil group, the percentage of impervious cover, the condition of vegetative cover and the antecedent moisture condition of the drainage area. "Impervious Cover" shall be as defined in Chapter 22.07 Municipal Drainage Utility of the City Code of Ordinances. AMC II curve numbers will be used for analyses within the City of Lubbock and are provided in Table 3.6 below.

Table 3-6: AMC II Curve Numbers

Zoning	Average Percent Impervious Area	AMC II Curve Number for Soil Group			
		A	B	C	D
1-Acre Residential	20	51	68	79	84
Single Family Residential	49	71	81	87	90
Duplex/Townhome	69	81	87	91	93
Apartments	75	84	89	92	94
Commercial*	99	98	98	98	98
IDP Industrial Park	79	89	92	94	95
Public Use (e.g., schools)	36	60	74	83	87
Open Space/Parks	7	43	64	76	81
Playas	100	98	98	98	98

[1] The City Engineer can allow, at his/her option, that a lower percent impervious area be used for proposed property. Lower runoff coefficients, or lower percentage imperviousness, must be justified by landscaping plans and construction of those landscaping plans.
*Property zoned as Land Use Code Garden Office shall use Commercial classification.

Rainfall Depth-Duration Data

Rainfall depths for various return period and storm duration combinations are provided in Table 3.7.

Table 3-7: Point Rainfall Depth Duration

Return Period	Depth for Duration								
	30-Min (Inches)	1-Hour (Inches)	2-Hours (Inches)	3-Hours (Inches)	6-Hours (Inches)	12-Hours (Inches)	24-Hours (Inches)	48-Hours (Inches)	72-Hours (Inches)
2-Year	1.07	1.36	1.63	1.79	2.08	2.38	2.72	3.16	3.46
5-Year	1.44	1.82	2.18	2.38	2.74	3.12	3.54	4.10	4.47
10-Year	1.75	2.22	2.65	2.90	3.33	3.77	4.25	4.89	5.32
25-Year	2.19	2.78	3.33	3.65	4.19	4.71	5.27	6.00	6.50
50-Year	2.53	3.21	3.87	4.25	4.89	5.47	6.08	6.87	7.40
100-Year	2.90	3.67	4.45	4.91	5.65	6.28	6.94	7.78	8.34
500-Year*	3.96	5.03	6.10	6.72	7.69	8.48	8.38*	10.20	10.90

Source: NOAA Atlas 14 Volume 11 (2018)
Source for 500-Year, 24-Hour event value: Technical Paper 40, U.S. Dept. of Commerce and NOAA Technical Memorandum NWS HYDRO-35.

SCS Rainfall – Runoff Relationship

To estimate the peak discharge and establish a runoff hydrograph using the SCS Method, the concept of a dimensionless unit hydrograph is applied. A peaking factor of 484 will be used for all analyses within Lubbock. The lag time for each watershed is calculated from the time of concentration (T_c), using the following equation:

$$T_{LAG} = 0.6 \times T_c$$

3.3 HYDROLOGIC MODEL SUBMITTAL REQUIREMENTS

Submittals of hydrologic models and supporting calculations shall be in accordance with Chapter 2 *Drainage Submittal Requirements and Process*. Sufficient information shall be submitted so that the City Engineer is able to complete a thorough review of the hydrologic calculations.

4.0 HYDRAULICS

4.1 HYDRAULIC DESIGN OF STREETS

This section contains minimum storm drainage design criteria to be employed on projects utilizing street drainage in the City of Lubbock. The arrangement, character, extent, and location of streets shall be considered in relation to existing and planned streets, to topographical conditions, to drainage in and through the proposed and adjacent subdivisions, to public convenience and safety, and in appropriate relation to the proposed uses of land to be served by such streets. Due consideration will be given to the effect of street drainage on traffic.

4.1.1 Street Classifications and Typical Sections

Street classification definitions, as well as typical sections, may be found in the City of Lubbock *Master Thoroughfare Plan (MTP)*.

Maximum and Minimum Street Grades

Minimum street grades should not be less than 0.002 feet per foot (0.2 percent). Grades less than 0.002 feet per foot usually result in inadequate drainage of the gutters and often have inadequate flow capacities. Minimum and maximum street grades for each street type should conform to the City of Lubbock *MTP*. If there is a discrepancy between this manual and the *MTP*, the *MTP* shall control.

Capture of Flow in Gutter

All stormwater shall be drained from the streets to the gutter. This shall be accomplished through sheet flow perpendicular to the roadway. Depressions in the crowns of streets that hold water, or inverted street crowns, are not acceptable. The gutter shall convey the stormwater to a point of release or to a stormwater inlet.

4.1.2 Street Flow Calculations

When calculating the peak discharge of stormwater runoff to determine street flow depths, the peak discharge shall be determined as discussed in Chapter 3, Hydrology.

Straight crown street capacities shall be hydraulically designed using Manning's equation, modified for gutter flow:

$$Q_o = 0.56\left(\frac{z}{n}\right) S_o^{1/2} Y_o^{8/3}$$

Where:

Q_o = gutter discharge (cfs)

z = inverse of the cross slope (ft/ft)

S_o = street or gutter slope (ft/ft)

n = Manning's roughness coefficient (0.02 for Lubbock streets)

Y_o = depth of flow in gutter (ft)

Use of this equation is limited to a single triangular gutter cross section with a straight street crown.

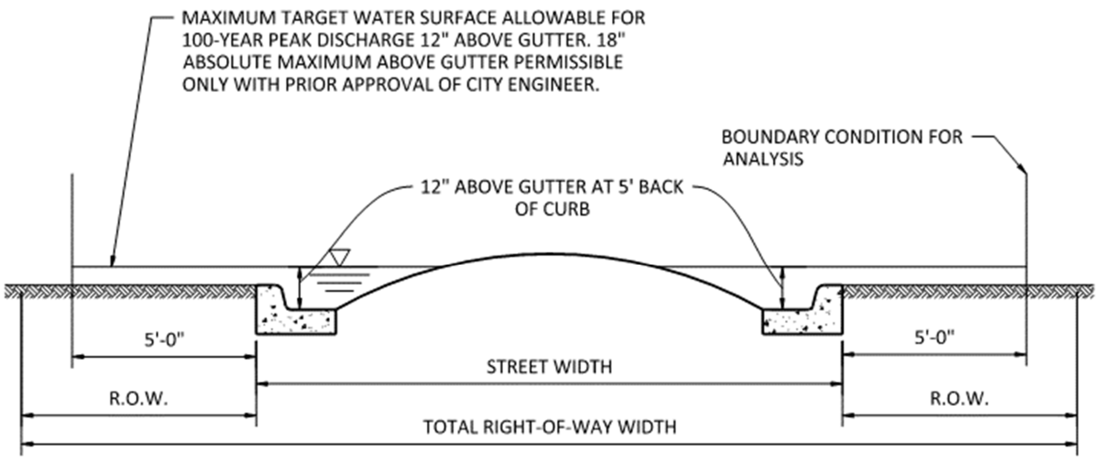
The Developer may also use appropriate hydraulic modeling software to determine the straight crown street capacities in lieu of the equation provided above. Sufficient information shall be provided so that the City Engineer is able to verify the calculations.

Maximum Allowable Flow Depth

The street under consideration must be capable of conveying the 100-year peak discharge using the boundary conditions shown in Figure 4-1. Where the gutter elevations vary by 0.2' or less from one side of the street to the other, depth of flow and minimum finished floor elevations shall be determined based on the higher of the two gutter elevations. Differences in gutter elevations of more than 0.2' require approval by the City Engineer. Refer to Section 3.1.1 for minimum design frequencies for streets.

Finished floor elevations shall be in conformance with Chapter 2 of this manual and the Lubbock Code of Ordinances. For property that is zoned as other than single-family residential use, the adjacent alley shall be paved for drainage to another point of pavement release. Refer to alley design criteria in this section and the Lubbock Code of Ordinances and building code for alley paving requirements.

Where hydrologic and hydraulic calculations show that maximum allowable depths of flow will be exceeded, the Developer shall incorporate design features to reduce the flow depth. Such features can include storm drains, division of flow across more than one street, grading to alter a localized



NOTES:

1. CONDITIONS SHOWN ARE FOR SYMMETRICAL STREET SECTION (EQUAL ELEVATION OF CURB AND GUTTER BOTH SIDES OF STREET). ANALYSIS MUST BE BASED ON ACTUAL STREET SECTION.
2. CAPACITY CALCULATIONS SHALL BE BASED ON MTP TYPICAL SECTIONS.
3. FLOW DEPTHS GREATER THAN 12" ABOVE GUTTER FLOW LINE WILL REQUIRE APPROVAL FROM THE CITY ENGINEER.

TYPICAL CROWNED STREET SECTION FOR
100-YEAR STORMWATER CAPACITY ANALYSIS

N.T.S.

FIGURE 4-1

Figure 4.1: Typical Crowned Street Section for 100-Year Stormwater Capacity Analysis

development's drainage pattern, detention/retention basins, or other solutions acceptable to the City Engineer.

Figure 4-2 shows the typical relationship of the maximum allowable street and alley water surface elevation with a building first-floor elevation.

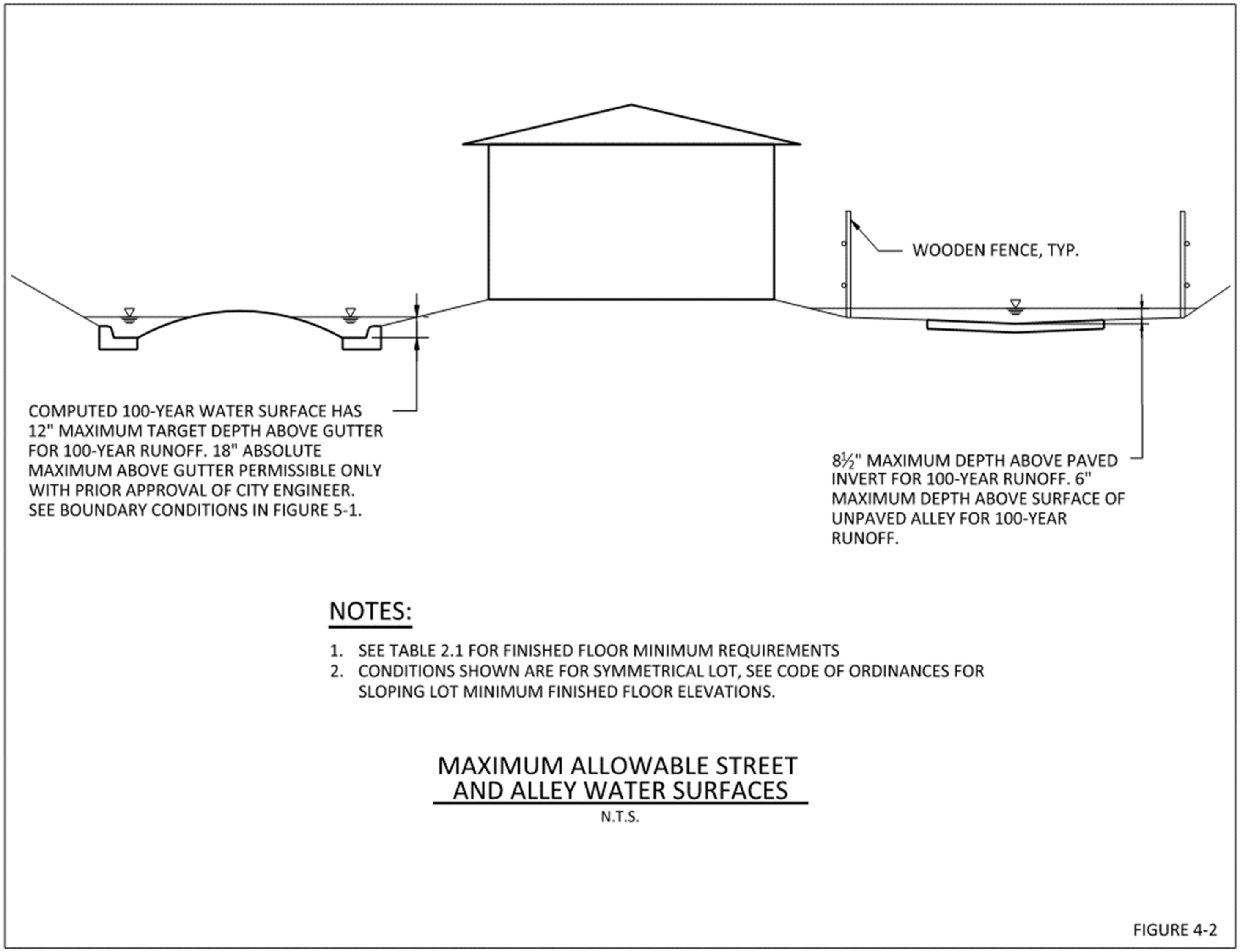


Figure 4.2: Maximum Allowable Street and Alley Water Surfaces

4.1.3 Alley Flow Calculations

Two types of alleys occur throughout the City of Lubbock: unpaved (or unimproved) and paved (improved). Typically, the unpaved alley is flat in cross section and the paved alley has a "V" cross section within the pavement section itself. Regardless of type, alleys with a slope steeper than two percent shall be stabilized. Stabilization method shall be approved by City Engineer.

Unpaved (Unimproved)

A typical unpaved alley section is shown in Figure 4-3. Although the alley may not be truly flat in shape, for hydraulic analysis purposes it can be considered as flat in design calculations.

The stormwater runoff should be contained within a 15-foot width within the alley, as this is considered to be the effective flow width. The maximum allowable depth of flow in the alley for the 100-year storm event is six inches above ground level. Depths of flow greater than six inches require prior approval from the City Engineer.

Open channel methods are used to determine the alley's capacity. The following equation can be used to calculate the capacity of an unpaved alley for normal depth:

$$Q = 168 * S^{0.5}$$

Where:

Q = alley capacity (cfs)

S = ground slope (ft/ft)

If grades flatter than 0.2 percent exist, a step-backwater analysis will be required.

Paved (Improved)

A typical paved alley section is shown in Figure 4-4. The following equation can be used to calculate the capacity of a paved (improved) alley for normal depth:

$$Q = 354 * S^{0.5}$$

Where:

Q = alley capacity (cfs)

S = ground slope (ft/ft)

When checking the total capacity of the paved alley for the 100-year storm event, the maximum allowable depth is six inches above the edge of pavement. This translates into eight and one-half inches above the alley invert. Depths of flow greater than six inches above the edge of the alley pavement require prior approval from the City Engineer.

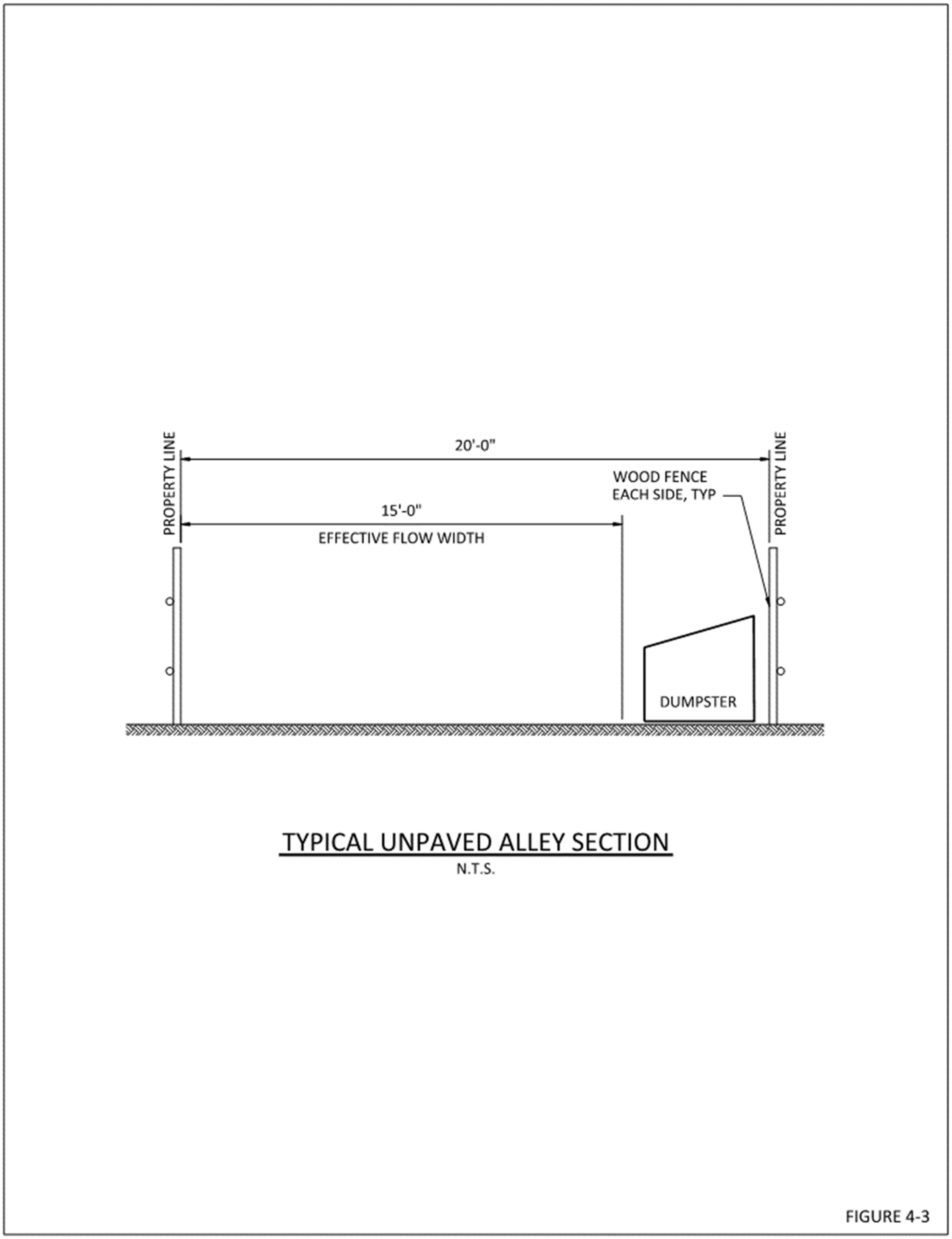


Figure 4.3: Typical Unpaved Alley Section

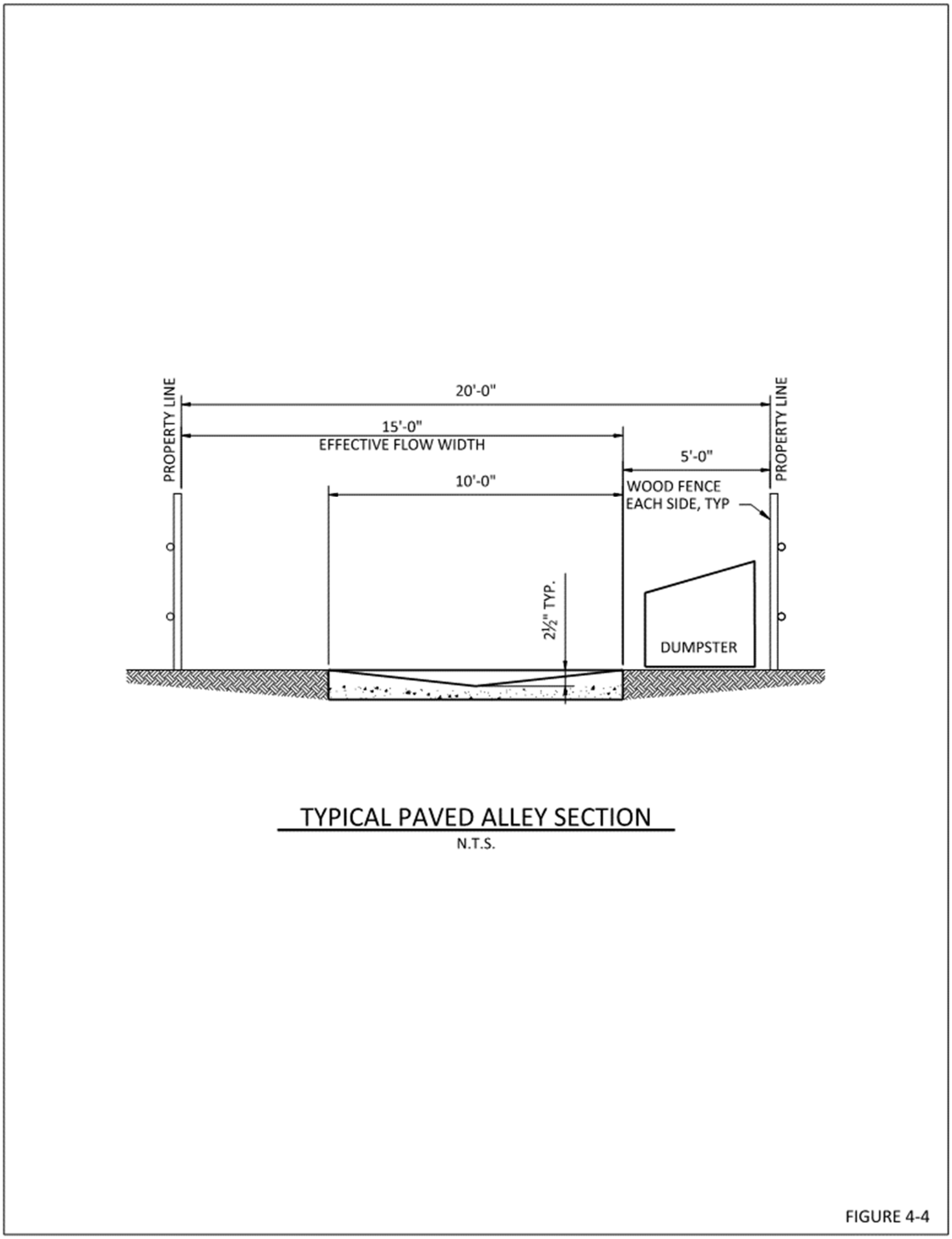


Figure 4.4: Typical Paved Alley Section

4.1.4 Flow at Intersections

Where storm drains are in use, and wherever possible, inlets should be placed upstream from an intersection to prevent large amounts of runoff water or persistent amounts of nuisance water from running through the intersection. The inlets should be located on the approach street to an intersection and in alleys where necessary to prevent water from entering these intersections in amounts which would cause the allowed street capacity to be exceeded, or which could freeze in winter creating an unsafe travel condition (nuisance water).

The use of valley gutters to convey stormwater across an intersection is subject to the following criteria:

A. A thoroughfare shall not be crossed with a valley gutter. It is preferred that thoroughfares be crossed with a concrete dip section. Concrete dip sections are described in detail below. The minimum width of the concrete dip shall be coordinated with the City Engineer.

B. Wherever feasible, a collector street shall not be crossed with a valley gutter except for valleys with very shallow side slopes that can be easily crossed at the street's posted speed limit. Valley gutters shall not be less than 10 feet in width in any case.

C. Valley gutters perpendicular to the main street shall only be permitted at cross-street locations, unless otherwise approved by the City Engineer.

A concrete dip is a section of roadway in which the roadway crown is transitioned such that flow may cross the roadway. Concrete dips shall comply with the following criteria:

A. The dip shall be concrete pavement for the length of roadway necessary to convey the design flow for the cross-drainage.

B. A concrete dip shall maintain appropriate roadway grades and slopes considering the posted speed limit.

C. A concrete dip section is required on thoroughfare roadways and preferred on collector roadways.

D. A concrete dip shall have a minimum width equivalent to the roadway width from face of curb to face of curb.

E. Design flows in a concrete dip at a thoroughfare or collector shall comply with controlling design criteria prescribed elsewhere in this manual.

4.1.5 Surface Outfalls to Channels and Lakes

A sawtooth curb and gutter may be used to route stormwater from the gutter to an open channel or a drainage flume. This is a desirable feature where storm drains are not feasible and the stormwater runoff must be routed away from the street by surface means. Refer to the City of Lubbock Design Standards and Specifications for examples of typical sawtooth curb and gutter construction, as well as flume construction. Potential applications of a sawtooth curb and gutter include conveyance of runoff into a playa lake, overflow sections of playa lakes, or conveyance to open channels. If a flume is used to convey water into a playa, the flume shall be designed to convey and fully contain the design flow specified in Table 3.1.

Figure 4-5 depicts a typical application of sawtooth curb and gutter with drainage flumes.

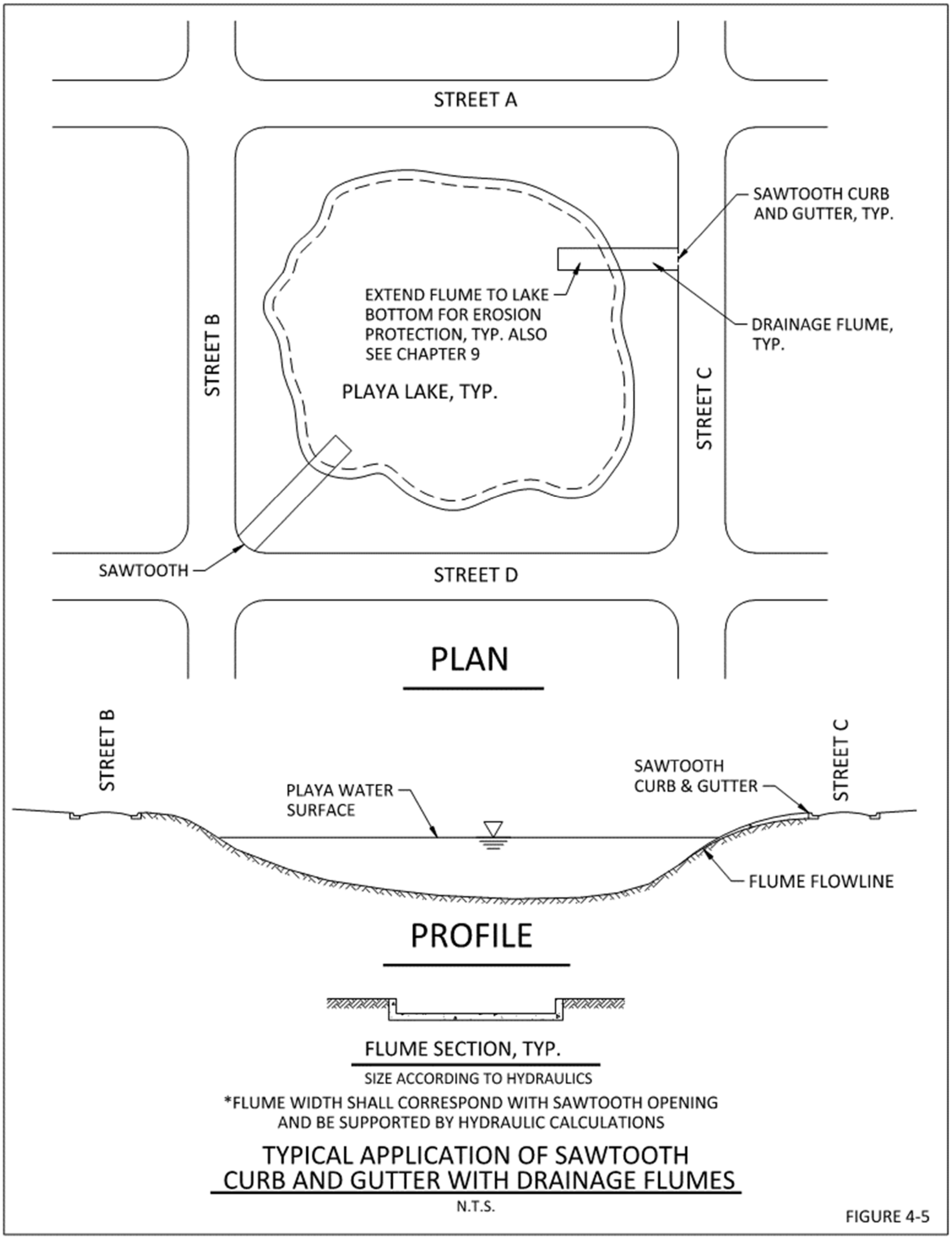


Figure 4.5: Typical Application of Sawtooth Curb and Gutter with Drainage Flumes

4.2 INLETS

There are three types of inlets are commonly used by the City of Lubbock: curb inlets, grate inlets and ditch inlets. Inlets may be located in streets, ditches, medians or ponding areas. The following guidelines shall be used in the design of inlets to be located in streets:

A. Design and location of inlets shall take into consideration pedestrian and bicycle traffic, as well as openings in the curb line. Inlets located in streets must be designed so that they will not conflict with traffic.

B. Minimum design frequency for inlets is listed in Section 3.1.1.

C. Only curb inlets will be allowed at street sumps or low points. Grate inlets and slotted drains are not acceptable because of debris collection.

D. All curb inlets incorporate a standard 3-inch depression.

E. Grated inlets in street gutters are discouraged from use due to their increased tendency to clog, problems with replacement and increased pavement maintenance in the immediate vicinity of the inlet. Use of a grated inlet instead of a curb inlet will require prior approval by the City Engineer.

F. Where curb or grate inlets are not feasible, use of a slotted drain is acceptable only for nuisance water control.

G. Design and installation of all inlets shall be in accordance with the City of Lubbock Design Standards and Specifications.

4.2.1 Curb Inlets at Sump

Unsubmerged curb inlet openings located at a sump, or low point, operate as rectangular weirs with a coefficient of discharge of 3.0. The capacity of a sump curb inlet is determined according to the following equation:

$$Q = 3.0h^{1.5}L$$

Where:

Q = capacity of curb inlet at sump (cfs)

h = head at the inlet, a + Y_o (ft)

L = length of opening through which water enters the inlet (ft)

a = gutter depression at the inlet (ft)

Y_o = depth of water above the normal gutter flow line (ft)

Curb inlets at sumps have a tendency to collect debris at their entrances. To account for blockage of flow into the inlet by this debris, the calculated inlet capacity *shall be reduced by 50 percent*.

When the water depth at the curb inlet opening exceeds 1.4h (h is the height of the inlet opening in feet) then the inlet opening acts as an orifice, and capacity may be determined by this equation:

$$Q = 0.67A\left[2g\left(d_i - {}^{h}\!/_{2}\right)\right]^{0.5}$$

Where:

Q = capacity of the inlet (cfs)

A = area of the inlet opening (ft^2)

H = height of the inlet opening (ft)

g = acceleration due to gravity, 32.2 (ft/sec^2)

d_i = depth of water above the inlet opening invert, a + Y_0 (ft)

Designing for a curb inlet to operate under orifice flow conditions is not usually desirable; however, Lubbock's surface drainage systems occasionally make such situations unavoidable. The depth of water at the curb face for orifice flow conditions usually covers street driving lanes with an unacceptable depth of water. Design of inlets with the orifice equation, along with the increased depth at the curb, will require approval by the City Engineer.

4.2.2 Curb Inlets on Grade

The hydraulic design of a curb inlet on grade is accomplished through the use of the following equation:

$$Q/_{L} = 0.7\left[\frac{1}{H_1 - H_2}\right]\left[(H_1)^{5/2} - (H_2)^{5/2}\right]$$

Where:

Q/L = inlet capacity per foot of opening (cfs/ft)

H_1 = a + Y_o (ft)

H_2 = a = gutter depression (ft)

Y_o = depth of water above the normal gutter flow line (ft)

Refer to Table 4.10 in the appendix for the City of Lubbock Curb Inlet Flow Calculation Table.

4.2.3 Grate Inlets on Grade

The hydraulic design of a grate inlet on grade is dependent on the ratio of frontal flow to total gutter flow, the ratio of side interception to frontal interception, and the velocity at which splash-over occurs. In lieu of these calculations, manufacturer information may be provided.

The ratio of frontal flow to total gutter flow (E_o) for a straight cross-slope is:

$$E_o = \frac{Q_w}{Q} = 1 - \left(1 - W/_{T}\right)^{2.67}$$

Where:

Q = total gutter flow (cfs)

Q_w = flow in width (cfs)

T = total spread of water in the gutter (ft)

W = width of grate (ft)

The ratio of side flow (Q_s) to total gutter flow is defined as:

$$Q_s/_{Q} = 1 - Q_w/_{Q} = 1 - E_o$$

With Q, Q_w and E_o the same as defined earlier. The ratio of frontal flow intercepted to total frontal flow (R_f) is:

$$R_f = 1 - 0.09(v - v_o)$$

Where:

v = velocity of flow in the gutter (ft/s)

v_o = gutter velocity where splash-over first occurs (ft/s)

This ratio is equivalent to the frontal flow interception efficiency.

The ratio of side flow intercepted to total side flow, or side flow interception efficiency is expressed as:

$$R_s = 1/\left[1 + \left(\frac{0.15v^{1.8}}{S_x L^{2.3}}\right)\right]$$

Where:

R_s = ratio of side flow intercepted to total side flow

v = velocity of gutter flow (ft/s)

S_x = pavement cross slope (ft/ft)

L = length of grate (ft)

The efficiency, E, of a grate is found by:

$$E = R_f E_o + R_s(1 - E_o)$$

Where the terms are defined as in previous equations. The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\left[R_f E_o + R_s(1 - E_o)\right]$$

A grate inlet on grade located in a street should be of the curved vane or tilted vane type.

4.2.4 Ditch Inlet at Sump

Ditch inlets located at low points or sumps operate under either weir flow or orifice flow conditions. For weir flow conditions, the flow rate through the grating is described by the equation:

$$Q = 3.0LH^{1.5}$$

Where:

Q = inlet flow rate (cfs)

3.0 = weir discharge coefficient (dimensionless)

L = perimeter length open for flow (ft)

H = depth of water above inlet (ft)

Note that the perimeter length open for flow must be reduced if one side of the inlet is against an obstruction. To allow for blockage by trash and debris, the size of the grate inlet selected, with respect to area, *shall be increased by 100 percent*. The weir flow regime for a grate inlet usually occurs at water depths above the inlet of 0.4 feet or less.

4.3 CLOSED CONDUIT STORM DRAIN DESIGN

The purpose of this section is to establish the hydraulic design of storm drains and their appurtenances in a storm drainage system. Usually, unsteady and non-uniform flow exists within a storm drain and the flow is either open channel gravity flow (conduit not flowing full) or surcharged flow (hydraulic grade line exceeds the top of pipe putting the storm drain in a pressure flow condition). It is important that the hydraulic grade line for both conditions not interfere with the connecting inlets for the design storm.

4.3.1 Hydraulic Grade Line Calculations

The design procedure will vary according to the expected operation of the storm drain. Under gravity flow conditions, or open channel operation, the design calculation will proceed from downstream to upstream in sequence. This is normally the calculation sequence where sufficient elevation difference is available to build storm drains at relatively steep grades, and pipe inverts at manholes can also be sufficiently staggered to offset junction losses. For surcharged storm drain conditions, or full conduit pressure flow, the design calculations proceed from the outlet in a step-backwater sequence to the most upstream inlet. The surcharged condition describes the flow regime most likely to be encountered in Lubbock.

In some instances, the flow regime for a storm drain may be surcharged flow conditions in the downstream reaches and gravity flow conditions in the upstream reaches. Such a flow transition occurs when the hydraulic grade line elevation decreases from a level above the top of the conduit to a level below the top of the conduit.

When the design is for a surcharged condition, then calculations begin at the outlet for the system and the upstream conduit reaches have not yet been sized and velocities not yet determined. For surcharged conditions, a velocity will be calculated by dividing the discharge by the cross-sectional area. The engineer also has the option of performing iterations of the design in order to more accurately determine velocities for travel time calculations.

The peak discharge for design of any point in the storm drain system is determined by the controlling time of concentration to that point. The engineer should check to be sure that a smaller area to each inlet, with a corresponding shorter time of concentration, does not result in a greater peak discharge that must be accommodated. If it does occur, then use the reduced area of consideration and the greater peak discharge for design.

Head loss calculations shall be completed based on the following criteria:

A. Adjustments are made in the HGL whenever the velocity in the trunk main changes due to conduit size changes or discharge changes. Laterals in partial flow must be designed appropriately and the partial flow velocity shall be used in the calculations.

B. In determining the HGL for the lateral, begin with the hydraulic grade of the trunk line at the junction plus the HL due to the velocity change. Where the lateral is in full flow, the hydraulic grade is projected along the friction slope calculated using Manning's Equation.

C. HL losses or gains for wyes, pipe size changes, and other velocity changes will be calculated by the following formulas:

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

Where:

H_L = head loss or gain (ft)

v_1 = upstream velocity (ft/s)

v_2 = downstream velocity (ft/s)

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

H_L for pipe in full flow at manholes, bends, and inlets, where the flow quantity remains the same, shall be calculated as follows:

$$H_L = K_j \left[\frac{v^2}{2g}\right]$$

Where:

H_L = head loss or gain (ft)

v = velocity in the lateral (ft/s)

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

K_j = coefficient of loss as shown in Table 4.7 in the Appendix (dimensionless)

Head losses or gains at manholes and junction boxes where there is an increase in flow quantity shall be calculated as follows:

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

Where:

H_L = head loss or gain (ft)

v_1 = upstream velocity (ft/s)

v_2 = downstream velocity (ft/s)

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

K_j = coefficient of loss as shown in Table 4.7 in the Appendix (dimensionless)

4.3.2 Design Criteria

There are several general rules to observe when designing storm sewers:

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 configuration.

B. Never discharge the stormwater of a larger conduit into a smaller conduit, even though the smaller conduit may have a higher capacity due to a steeper slope.

C. Where connections are made of different pipe sizes, match the soffits of the pipes rather than the flow lines. The only exceptions shall be where wyes or tees are used. Where wyes or tees are used, they shall be plant-manufactured to the manufacturer's standard details. Such standard manufacturing processes may preclude matching the soffits of different sizes of pipe.

D. The minimum diameter for pipe storm drains in the City of Lubbock shall be 24 inches regardless of whether the storm drain is a trunk line or a lateral line.

E. The conduit of the storm drain system shall be placed at not less than the grade that maintains a minimum 2.5 ft/s velocity when the conduit is flowing full.

F. For all pipe junctions, other than a manhole or inlet structure, the angle of intersection should not be greater than 60 degrees. This includes discharges into channels and box sections. Recommended junction loss values are shown in Table 4.7 in the Appendix for various junction configurations.

G. No pipe having an outside dimension greater than the inside dimension of a connected box section shall be allowed to discharge into that box section.

H. The preferred storm drain configuration does not have the trunk storm drain pipe running through an inlet box. The inlet box should discharge into the trunk sewer through a wye connection or manhole. Where a storm drain does go through an inlet box, the inlet box shall be treated as a junction box and the hydraulic grade line, including losses, shall not interfere with the inlet box opening for the design storm event.

I. The design hydraulic grade line of a storm drain system shall not be higher than 6 inches above the adjacent street gutter elevation. Variation for hydraulic grade lines that are higher than 6 inches above gutter elevation will be evaluated on a case-by-case basis by the City Engineer; however, extenuating physical site conditions must be the cause for the variation.

Minimum Grades and Velocities

Storm drains shall be designed to maintain velocities which prevent the deposition of suspended solids in the storm drain conduit. Storm drains shall be designed at a grade which results in a *minimum velocity of 2.5 feet per second* when the storm drain conduit is flowing full. The *minimum practical grade for construction purposes shall not be less than 0.0010 feet per linear foot (0.10 percent),* even if a flatter grade still results in a velocity of 2.5 feet per second or greater. Tables 4.7 and 4.8 in the appendix contain minimum design grades, for box and circular pipe of various materials and diameters, to achieve the 2.5 foot per second criteria for normal depth flow. Table 4.1 contains Manning's n values for various material types. The engineer is responsible for submitting minimum grade analysis for alternate geometries when those storm drain conduits are proposed.

Where storm drains of box cross section are used, the engineer shall carefully evaluate the effect of available box sections on the capacity of the storm drain. The box cross section normally has an equal or larger dimension for the horizontal span than for the vertical rise. Precast boxes can be constructed to American Society of Testing and Materials (ASTM) standards, American Association of State Highway and Transportation Officials (AASHTO) standards and to standards of the Texas Department of Transportation (TxDOT). TxDOT also has cast-in-place box section construction details available, but their use is the responsibility of the Developer.

Maximum Grades and Velocities

Limiting the maximum velocity in a storm drain conduit is important since a high velocity can possibly erode the conduit's flow line at an excessive rate. The suspended solids which are carried with the stormwater runoff act as abrasives on the conduit's interior surfaces at higher velocities. *The City of Lubbock has established the maximum permissible velocity for trunk storm drains as 15 feet per second*. No maximum velocity has been established for laterals which serve only one inlet.

The maximum grade on which storm drains should be designed and constructed is that grade which corresponds to the 15 feet per second velocity limit.

Manhole Spacing

The maximum manhole spacing for straight, uniformly sloped storm drains should not exceed 500 feet. Where tees, wyes or bends are located, access to the storm sewer through either an inlet or a manhole shall be possible from both the upstream and downstream directions. Access to the storm drain shall be located not further than 150 feet in either the upstream or downstream direction from the tee, wye or bend.

Manhole Conflicts

Manhole conflicts shall be designed in accordance with the City of Lubbock Design Standards and Specifications.

Location and Size of Easements

For storm drain pipe not located within the street right-of-way, the owner, Developer or other responsible party shall provide an offsite easement so that the City can have access for maintenance purposes. The minimum offsite easement width provided shall not be less than the following:

> 20-foot easement width for circular pipe diameters or arch pipe spans less than 36 inches.
>
> 30-foot easement width for circular pipe diameters or arch pipe spans greater than or equal to 36 inches.
>
> Easements for storm sewers constructed of box cross-sectional shapes will be determined by the City Engineer on an individual, site-specific basis.

Permissible Pipe Materials

Pipe within City of Lubbock right-of-way or easements shall be reinforced concrete pipe (RCP). Refer to the City of Lubbock Standard Details and Specifications for other acceptable pipe material and applications.

4.3.3 Overflow Routes

An overflow route shall be defined for all street sump locations and T-intersections and kept clear to a point of outfall to an acceptable drainageway. The purpose of providing an overflow route is to allow for surcharges from the storm sewer system to be safely directed to a drainageway. Such an overflow area could consist of additional space between building structures and channelization of runoff. One example situation is shown in Figure 4-6. Any emergency overflow area shall be designed for adequate hydraulic capacity for the 100-year storm runoff without exceeding an elevation six inches below the finished floor elevations of adjacent buildings.

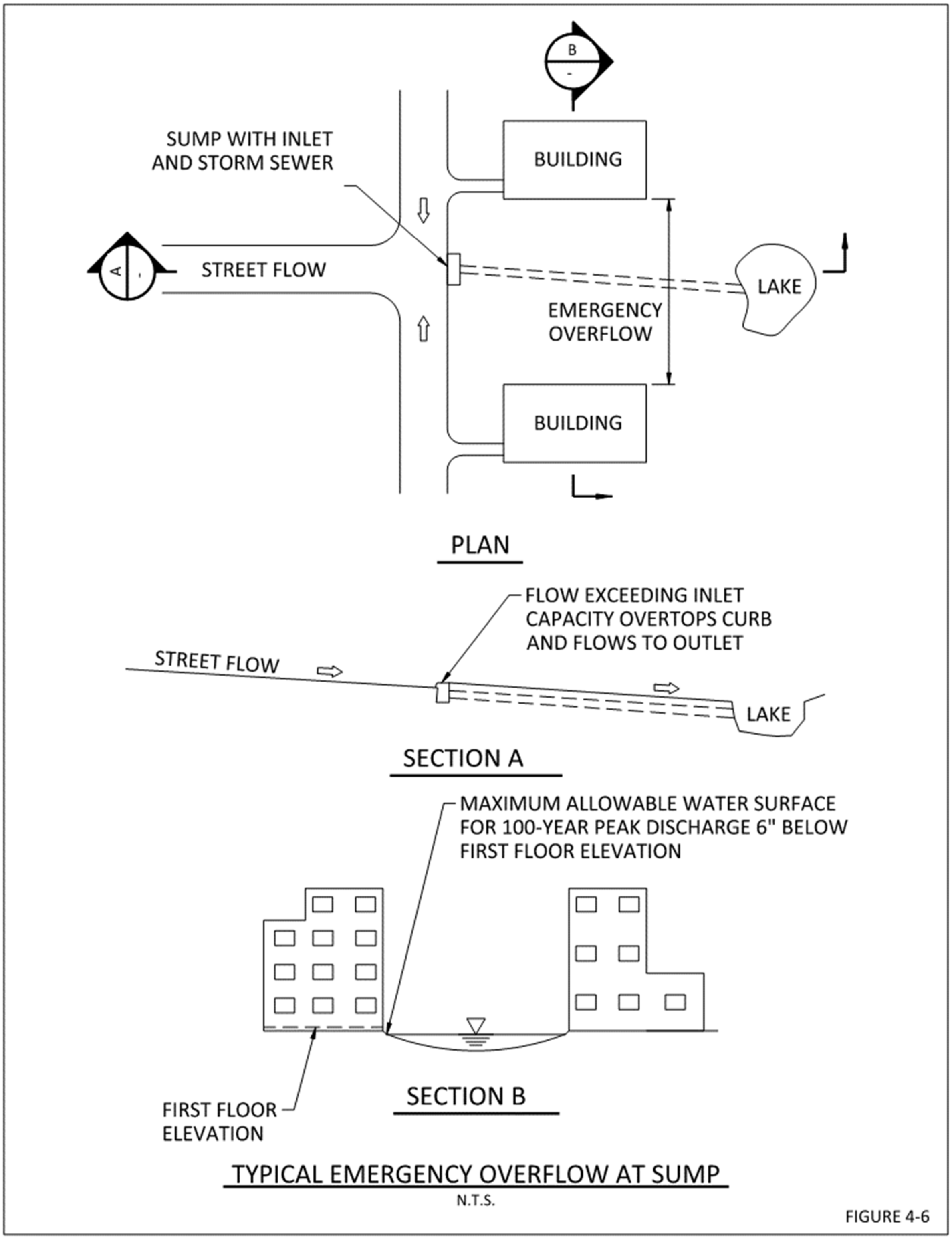


Figure 4.6: Typical Emergency Overflow at Sump

4.3.4 As-Built Drawing Submittal

As-built drawings will be required prior to issuance of a Certificate of Occupancy or Residential Final Inspection, whichever applies. Specific submittal requirements are provided in Chapter 2.

4.4 OPEN CHANNEL DESIGN

Channel design involves the determination of the channel cross-section required to accommodate a given design discharge. The hydraulic design of a flood control channel improvement must include the consideration of a number of factors, such as: Safety; efficiency (optimize conveyance and minimize operation and maintenance); reliability; cost effectiveness (initial, operational, maintenance, and replacement cost); all with an appropriate concern for environmental and social aspects. The design parameters for natural and engineered channels are discussed in the following paragraphs.

The general classifications for open channels include: (1) Natural channels, which include all water courses that have been carved by nature through erosion; and (2) Engineered channels, which are constructed or existing channels that have been significantly altered by human effort. Engineered channels can be lined with grass, concrete, mortared rocks, rock riprap or other materials.

4.4.1 Methodology

There are several terms used to describe flow in open channel hydraulics. These terms are steady flow, unsteady flow, uniform flow, varied flow, subcritical flow, critical flow and supercritical flow. Most of the problems in stormwater drainage involve uniform, gradually varied or rapidly varied flow situations. In this section, the basic equations and computational procedure for uniform, gradually varied and rapidly varied flows are presented.

Steady Flow

Flow in an open channel is said to be “steady” if the depth of flow does not change or if it can be assumed to be constant during the time interval under consideration. In most open channel problems and design, it is necessary to study only steady flow conditions.

Unsteady Flow

Flow is considered “unsteady” if the depth changes with time.

Uniform Flow

Open channel flow is said to be "uniform" if the depth of flow is the same at every section of the channel. A uniform flow may theoretically be steady or unsteady, depending on whether or not the depth changes with time. Steady uniform flow is the fundamental type of flow treated in open channel hydraulic design.

For a given channel condition of roughness, discharge and slope, there is only one possible depth for maintaining a uniform flow. The depth is referred to as normal depth. The Manning's Equation is used to determine the normal depth for a given discharge.

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 each computation is usually adequate and useful and, therefore, necessary for planning and design.

Varied Flow

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.

In gradually varied flow, the flow remains constant and the streamlines are parallel. The most common example of gradually varied flow in urban drainage system occurs in the backwater of bridge openings, culverts, storm drain inlets and channel constructions. Under these conditions, gradually varied flow will be created, and the flow depth will be greater than normal depth in the channel. Backwater programs such as HEC-RAS are generally intended to be used in analyzing steady, gradually, varied flow in natural or man-made channels.

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 system are dams, side-spill weirs, weirs and spillways of detention basins.

Subcritical, Critical and Supercritical Flow

Flowing water contains potential and kinetic energy. The relative values of 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.

$$H = d + \frac{v^2}{2g}$$

Where:

H = specific energy head (ft)

d = depth of flow (ft)

v = average channel flow velocity (ft/s)

F = acceleration due to gravity, 32.2 (ft/s^2)

When depth of flow is plotted against specific energy for a given channel discharge at a section, the resulting curve shows that 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:

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

Where:

d_c = critical depth (ft)

V = average channel flow velocity (ft/s)

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

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

$$Fr = \frac{V}{(gd)^{0.5}}$$

Where:

Fr = Froude Number (dimensionless)

V = average channel flow velocity (ft/s)

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

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 a 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.

$$Fr = \left[\frac{Q^2 B}{g A^3}\right]^{0.5}$$

Where:

Fr = Froude Number (dimensionless)

Q = channel discharge (cfs)

B = top width of channel (ft)

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

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

In general, improved channels should be designed with subcritical slopes to avoid the instability that occurs at or near critical depth and the relatively high velocities encountered in supercritical flow regimes. Section 4.3.3 Design Criteria includes additional discussion on concrete lined channels.

The determination of critical slope, critical depth, and related hydraulic parameters can be computed using the noted equations or available computer software for hydraulic design.

Manning's Equation and Roughness Coefficients

Careful attention must be given to the design of drainage channels to provide adequate capacity and allow for minimum maintenance. The hydraulic characteristics of open channels shall be determined by using Manning's Equation, commonly expressed as:

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

Where:

Q = channel discharge (cfs)

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

R = hydraulic radius of channel, A/WP (ft)

WP = wetted perimeter of channel flow (ft)

S = slope of energy gradient (ft/ft)

n = Manning's coefficient of channel roughness (dimensionless)

Manning's Roughness Coefficient

The Manning's 'n' value or coefficient is a parameter that represents the "roughness" or resistance to flow that the water flowing in a channel will encounter. The more vegetation or obstructions to flow that occur, the higher the 'n' value.

Natural Streams

Lubbock does not have any natural channels except for the Double Mountain Fork of the Brazos River, Blackwater Draw, Yellow House Canyon Draw, and small draws that drain into these streams. Manning's roughness coefficients for natural streams are determined by field observations, serial photographs, and experiences of the hydraulic engineer. The 'n' values for the above-mentioned streams used in the Flood

Insurance Study of Lubbock vary from 0.015 to 0.090 for the channel portion and from 0.015 to 0.080 for the overbank areas. For new studies submitted for review, the City Engineer will approve the Manning's coefficient used for the analysis.

Improved Channels

Since any channelization will be a man-made nature, Manning's roughness coefficients have been developed from research and experience for open channel designs. These design roughness coefficients of Manning's 'n' values, are contained in Table 4.1. The bare earth roughness coefficient should be used only for checking the maximum velocity in the channel.

Table 4-1: Manning's (n) Roughness Coefficients

Type of Channel Description	Minimum	Normal*	Maximum
Concrete or asphalt streets	0.012	0.020	0.023
Paved Alleys	0.018	0.020	0.03
Unpaved Alleys	0.035	0.045	0.050
Reinforced concrete pipes	0.011	0.013	0.020
Corrugated metal pipes	0.017	0.024	0.030
Concrete lined channel	0.011	0.013	0.015
Dry rubble or riprap lined channel	0.023	0.033	0.036
Gabion basket/mattress lined channel	0.025	0.030	0.035
Earth channel maintained,			
Bare Earth[1]	0.023	0.025	0.030
Straight and uniform with short grass and few weeds	0.030	0.035	0.040
Earth channel not maintained,			
Bare Earth[1]	0.025	0.028	0.033
Clean Bottom, Brush on Sides	0.040	0.050	0.080
Undeveloped			
Pasture short grass	0.025	0.030	0.035
Pasture high grass	0.030	0.035	0.050
Cultivated area with mature row crop	0.025	0.035	0.045
Cultivated area with mature field crop	0.035	0.040	0.050
Light brush and trees	0.035	0.055	0.080

Source: Chow, Ven Te. *Open Channel Hydraulics*, 1959.
Note: The values under the column marked Normal* shall be used in all design in the City of Lubbock, unless detailed analysis, approved by the City Engineer, proves otherwise.
[1] For maximum velocity check only. Grass lined values shall be used for design capacity.

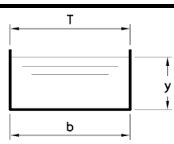
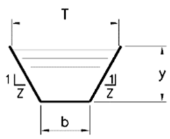
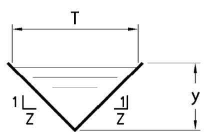
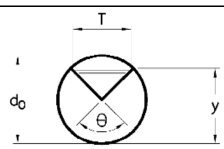
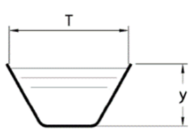
4.4.2 Design Criteria

All channels shall be designed for the 100-year, 24-hour peak discharge flood without levees and without out-of-bank flow, except where the out-of-bank flow is contained within a greenbelt or within the limits of adjacent, parallel streets and alleys. The minimum bottom width of any improved channel will be 10 feet, for maintenance purposes. The design of proposed channel improvements in Lubbock will include submittal of hydraulic design calculations for small improved channel (less than 200’ in total length). For larger improved channels, a hydraulic model will be required, to verify the design water surface profiles. For overflow routes between playa lakes, a hydraulic model will always be required regardless of dimensions. In addition, all concrete-lined channels will require a hydraulic model. Approved hydraulic models and additional related requirements are contained in Chapter 2: Drainage Submittals, Requirements and Processes.

Geometric Elements of Channel Section

Geometric elements are properties of a channel section that can be defined entirely by the geometry of the section and the depth of flow. These elements are very important and are used extensively in flow computations. They are shown on Table 4.2.

Table 4-2: Geometric Elements

Section	Area *A*	Wetted Perimeter *P*	Hydraulic Radius *R*	Top Width *T*	Hydraulic Depth *D*	Section Factor *Z*
 RECTANGLE	by	$b + 2y$	$\frac{by}{b + 2y}$	b	y	$by^{1.5}$
 TRAPEZOID	$(b + zy)y$	$b + 2y\sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}}$	$b + 2xy$	$\frac{(b + zy)y}{b + 2xy}$	$\frac{[(b + zy)y]^{1.5}}{\sqrt{b + 2xy}}$
 TRIANGLE	zy^2	$2y\sqrt{1 + z^2}$	$\frac{zy}{2\sqrt{1 + z^2}}$	$2zy$	$\frac{1}{2} y$	$\frac{\sqrt{2}\ zy^{2.5}}{2}$
 CIRCLE	Θ = radians $\frac{1}{8}(\Theta - \sin\Theta)d_o^2$	$\frac{1}{2}\Theta d_o$	$\frac{\frac{1}{4}(1 - \sin\Theta)\,d_o}{\Theta}$	$(\sin\frac{1}{2}\Theta)\,d_o$ or $2\sqrt{y(d_o - y)}$	$\frac{1}{8}\ \frac{(\Theta - \sin\Theta)}{(\sin\frac{1}{2}\Theta)}\ d_o$	$\frac{\sqrt{2}(\Theta - \sin\Theta)^{1.5}\,d_o^{2.5}}{32(\sin\frac{1}{2}\Theta)^{0.5}}$
 PARABOLA	$\frac{2}{3} Ty$	$T + 8/3\ (y^2/T)$*	$\frac{2T^2y^*}{3T^2 + 8y^2}$	$\frac{3}{2}\frac{A}{y}$	$\frac{2}{3} y$	$2/9\sqrt{6}\ Ty^{1.5}$

Natural Channels

The ideal open channel is a stabilized water course developed by nature over time, characterized by stable bed and banks. The benefit of such a channel are:

A. Velocities are usually low, resulting in longer concentration times and lower downstream peak discharges. Available channel storage can decrease peak flows.

B. Maintenance needs are usually low because the channel is somewhat stabilized.

C. The channel provides a desirable green belt and recreational area adding significant social benefits.

Many natural channels have mild slopes, are reasonably stable, and are not in a state of serious degradation or aggradation. However, if a natural channel is to be used for carrying storm runoff from an urbanizing area, the altered nature of the runoff peaks and volumes from urban development may cause scour and erosion. Some on-site modification of the natural channel may be required to assure a stabilized condition.

Grass-Lined Channels

Grass channels are the most desirable of the various types of improved channels for the following reasons:

A. The grass can stabilize the body of the channel.

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

C. The grass will check the erosion and control the movement of soil particles along the channel bottom.

Key parameters in grass-lined channel design include permissible velocity, roughness coefficient, side slope, curvature, bottom width, and freeboard. The roughness coefficient for bare earth should only be used for checking maximum permissible velocity (Table 4.3). Since bare earth channels will normally collect sediment and encourage undesirable vegetative growth, they will not be permitted for proposed improvements, unless written approval is obtained from the City Engineer. *If a variance is granted, the capacity of proposed bare earth channels will be determined using the grass-lined Manning's coefficient.*

Table 4-3: Permissible Velocities for Grass-Lined Channels

<table>
<tr><th rowspan="2">Cover</th><th rowspan="2">Slope Range (%)</th><th colspan="2">Permissible Velocity (ft/s)</th></tr>
<tr><th>Erosion-Resistant Soils</th><th>Easily Eroded Soils</th></tr>
<tr><td rowspan="3">Bermuda grass</td><td>0-5</td><td>8</td><td>6</td></tr>
<tr><td>5-10</td><td>7</td><td>5</td></tr>
<tr><td>>10</td><td>6</td><td>4</td></tr>
<tr><td rowspan="3">Buffalo grass, Kentucky bluegrass, smooth brome, blue grama</td><td>0-5</td><td>7</td><td>5</td></tr>
<tr><td>5-10</td><td>6</td><td>4</td></tr>
<tr><td>>10</td><td>5</td><td>3</td></tr>
<tr><td rowspan="3">Grass mixture</td><td>0-5</td><td>5</td><td>4</td></tr>
<tr><td>5-10</td><td>4</td><td>3</td></tr>
<tr><td colspan="3">Do not use on slopes steeper than 10%</td></tr>
<tr><td rowspan="2">Lespedeza sericea, weeping love grass, ischaemum (yellow bluestem), kudzu, alfalfa, crabgrass</td><td>0-5</td><td>3.5</td><td>2.5</td></tr>
<tr><td colspan="3">Do not use on slopes steeper than 5%, except for side slopes in combination channel</td></tr>
<tr><td rowspan="2">Annuals - Used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudan grass</td><td>0-5</td><td>3.5</td><td>2.5</td></tr>
<tr><td colspan="3">Use on slopes steeper than 5% is not recommended</td></tr>
</table>

Note: The values apply to average, uniform stands of each type of cover. Use velocities exceeding 5 feet per second only where good covers and proper maintenance can be obtained.
*Source: U.S. Soil Conservation Service [41]

Erosion Mitigation

Regardless of channel velocities, all improved grass-lined or bare earth channels will be required to have a concrete low-flow section to prevent erosion caused by meandering low flows. Additional channel lining, such as concrete or geotextile may be required for velocities exceeding six (6) feet per second.

A. Design Methods: Small roadside-type ditches may be designed using normal depth, uniform flow methods as long as the flow rate does not exceed 100 cubic feet per second and the flow area does not exceed 20 square feet. Ditches and channels which exceed these limits must be designed with the standard-step method and will require the submittal of a HEC-RAS (or equivalent) hydraulic method.

B. Velocity: The maximum permissible velocity for grass-lined channels is six (6) feet per second for the 100-year storm. To verify the channel velocities, the engineer shall use uniform depth (for small channels) and a HEC-RAS (or equivalent) hydraulic model (for large channels and for playa-to-playa overflow channels).

Note: Flow velocities shall not exceed six (6) feet per second for the 100-year storm where exiting a riprap section or culvert apron back onto grass-lined or bare earth channels. Appropriate energy dissipater designs shall be used if these velocities are exceeded.

C. Roughness Coefficient: The roughness coefficient selected for grass-lined channels is based on the degree of retardance of vegetation. Table 4.1 provides Manning's 'n' for grass-lined channel design.

D. Slope: The slope of the channel should be 0.002 feet per foot (0.2 percent) or greater. A maximum slope has not been established, however, velocities exceeding six (6) feet per second may require channel lining such as concrete or geotextile.

E. Side Slopes: Side slopes for earthen or grass-lined channels shall be seven (7) horizontal to one (1) vertical side slopes or flatter if the slopes are to be maintained by the City. Where the City Parks Department is to be the primary maintained organization, then both the City Engineer and the Director of Parks and Recreation must both concur in steeper channel side slopes.

Vegetation on grassed-lined channel side slopes must be established in a manner approved by the City.

F. Bottom Width: The minimum bottom width for a trapezoidal channel will be ten (10) feet for maintenance purposes.

G. Freeboard: The design water surface in a grass-lined channel for the 100-year storm event will be contained within the excavated channel cross section without the use of levees, dikes, or berms. Freeboard is not required for subcritical flow. Freeboard will be verified by HEC-RAS (or equivalent) modeling for large channels and playa-to-playa overflow routes.

H. Intersections: The intersection of one open channel with another should not be made with an intersection angle of 90 degrees. The angle between the channels should not exceed 15 degrees. Such an intersection angle should be accomplished by curving one of the channels upstream of the intersection.

I. Curvature: The center line curvature shall have a minimum radius of three times the top width of the channel unless analysis is furnished that the channel cross section has sufficient freeboard to contain the super elevated water surface.

J. Adjacent Roadways, Alleys, and Greenbelts: See City of Lubbock Design Standards and Specifications for design criteria.

If the distance from a street driving lane edge (lip of gutter pan), to the top of a channel side slope (for side slope of 3H:1V or steeper) is less than 20 feet, then a metal beam guard fence must be installed.

If the distance from the edge of an alley to the top of a channel side slope (for side slope of 3H:1V or steeper) is less than 10 feet, then a post-and-cable barrier system must be installed.

Although desirable, neither a street nor alley is required to be constructed to parallel the entire length of the channel (not necessary for maintenance access).

If a greenbelt is incorporated into the overall plan for development, as a channel or as a buffer zone around a channel, then the greenbelt shall be landscaped, <u>yet still retain the required hydraulic capacity</u>. Landscaping requirements and maintenance responsibilities will be decided at the plat stage. If the City is expected to assume maintenance of a greenbelt, then the Developer is responsible for obtaining a letter from the City Engineer stating such.

Concrete-Lined Channels

Concrete lined channels are designed to protect the channel from the erosive potential of high velocities. In addition to concrete-lined channels, other methods may be proposed to combat erosive velocities in channels and should be submitted for review.

Concrete-lined channels may be needed in channel reaches where the velocities are excessive or where the flow characteristics require such use.

A. Design Method: The standard-step (HEC-RAS or equivalent) method is required for design and analysis.

B. Velocity: In concrete lined channels the probability of supercritical flow is greatly increased. For channel velocities of greater than six (6) feet per second, concrete lining will be required.

C. Supercritical Flow Considerations: Improved channels with supercritical slopes will not be allowed for proposed projects, unless the City Engineer issues a written variance, in special cases, supported by technical and economic data. If a supercritical design is allowed by the City, the Developer must take care to ensure against the possibility of unanticipated hydraulic jumps forming in the channel in considering the 100-year storm. Flow with a Froude number equal to or greater than one (1) is unstable and should be avoided. If supercritical flow does occur, then freeboard and superelevation must be determined and included in the design.

All channels carrying supercritical flow shall be continuously lined with reinforced concrete within the supercritical flow regime.

Improved channels will have design depths that avoid the potential instability problem of flow near critical depth. Therefore, the design flow depth (d) shall be greater than 1.1 times the critical depth for subcritical flow, and less than 0.9 times critical depth for supercritical flow.

D. Freeboard: Channel freeboard of six (6) inches shall be provided for the 100-year storm.

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.

The freeboard shall be measured above the water surface.

A. Side Slopes: For concrete-lined channel sections the concrete portion of the side slopes may not be steeper than two horizontal to one vertical. Side slopes steeper than two to one will be evaluated on a case-by-case basis by the City Engineer.

B. Intersections: The intersection of one open channel with another should not be made with an intersection angle of 90 degrees. The preferred angle would be 15 degrees or less. Intersection angles can be accomplished by curving one of the channels upstream of the intersection and/or constructing a tapered junction wall.

C. Channel Bottoms: Channels will have a minimum of 10 feet bottom width and will have a slight “V” depression in the invert. No low flow channels from one playa to another will be allowed without an inverted bottom.

D. Channel Intersections with Roadway: The concurrence of the City Engineer is required for using a street as the outlet for a channel.

If a situation arises where a channel outlet discharges into a street, then the street alignment must continue the channel alignment. A channel that creates a “T” intersection with the street will not be permitted.

The channel peak discharge must not exceed the street conveyance capacity at the depth limits. If the peak discharge exceeds the street conveyance capacity, then the channel must be extended to an acceptable discharge location.

Also, if the channel velocity exceeds the street flow velocity, then energy dissipation features are required in the channel such that its exit velocity is at or less than the street flow velocity.

Rock-Lined Channels

Rock-lined channels are constructed from ordinary rock riprap or wire enclosed riprap (gabions). The rock lining permits a higher design velocity and therefore a steeper design slope than for grass-lined channels. Rock linings are also used for erosion control at culvert/storm drain outlets, at sharper channel bends, at channel confluences, and at locally steepened channel sections. Incorrectly designed rock-lined channels can result in excessive maintenance requirements. Correct sizing and bedding are essential to good performance. Maintenance responsibility should be a part of any drainage plan utilizing rock-lined channels.

If the project constraints dictate the use of a riprap or gabion lining, such use shall be allowed only upon approval of the City Engineer. Riprap for the purposes of local erosion control is permitted only if vegetation is unsuitable.

Channel linings constructed from ordinary riprap, grouted riprap, or wire encased rock (gabions) to control channel erosion have been found to be cost effective. Situations for which riprap linings might be appropriate are: 1) where major flows, such as the 100-year flood are found to produce channel velocities in excess of allowable non-eroding values; 2) where channel side slopes must be steeper than 4H:1V; 3) for low flow channels, and; 4) where rapid changes in channel geometry occur, such as at channel bends and transitions.

The City Engineer must approve the use of rock-lined or gabion-lined improved channels. All hydraulic structures shall be designed in such a way as to prevent erosion and conform to Section 9.2.

Other channel types may be approved by the City Engineer on a case-by-case basis, and sufficient design calculations and/or vendor information must be submitted such that the design can be verified.

Acceptable computation submittal formats for all hydrologic and hydraulic analyses are discussed in detail in Chapter 2.

Easement and Right-of-Way Requirements

The following criteria apply to maintenance access to channels:

The access ramp shall not be less than 20 feet wide.

The driving surface of the access ramp shall not exceed a slope of fifteen (15) horizontal to one (1) vertical.

Maintenance ramps shall provide access to all portions of the channel.

Maintenance ramps shall not infringe on the channel cross section. Additional right-of-way will be provided at access ramps.

Access roads for channels shall not be less than 20 feet wide adjacent to the top bank on both sides of the channel.

Minimum Access Road Width

The access road shall not be less than 10 feet on one side of the channel. Variances from this access requirement requires the City Engineer's approval.

Maintenance Access Ramps to Channel Bottom

A. Minimum Width – The access ramp shall not be less than 20 feet wide.

B. Maximum Slope – The driving surface of the access ramp shall not exceed a slope of fifteen (15) horizontal to one (1) vertical.

C. Maintenance ramps shall provide access to all portions of the channel.

D. Maintenance ramps shall not infringe on the channel cross section. Additional right-of-way will be provided at access ramps.

Channel Drop Structures

Refer to Section 4.7: Energy Dissipation for information related to channel drop structure design.

Drainage Flumes

Drainage flumes may be used to convey stormwater from a watershed if sheet flow is not desirable or easily obtainable. Figure 4-5 shows one typical application of a flume to convey stormwater runoff from a street to a playa lake. The Developer is referred to "Chapter 5: Playas" for additional criteria where playa lakes are involved.

4.5 CULVERT DESIGN

The purpose of a culvert is to provide a means of passing a design storm discharge beneath a roadway or similar structure without causing excessive backwater or overtopping of the structure as well as preventing the creation of excessive downstream velocities. The use of culverts and their design locations will depend on the available depth for installation, inlet and outlet conditions, general topography, upstream and downstream land use, development layout, flow rates and the level of protection that might be attainable. It should be noted that the criteria contained in this chapter applies only to those culverts to be placed in public right-of-way and those culverts within private property drainage easements whose headwater depth might cause unacceptable inundation of public right-of-way. The Lubbock Engineering Department will review proposed culverts on a case-by-case basis and determine the feasibility or the creation of an adverse situation based on the circumstances.

Culvert design shall be such that energy losses and discharge velocities remain within allowable limits. The design storm discharge shall be determined by hydrologic methods set forth in "Chapter 4: Hydrology."

As a minimum, culverts shall be designed to pass the discharge from a particular year frequency storm based on the street classification for which the culvert is being designed. Table 3.1 lists the design storm frequencies to be used for various street classifications and drainage areas where culverts are associated with public rights-of-way. The culverts shall also be designed to meet the design peak flow rates which are determined using fully developed watershed conditions as well as at the peak discharge guidelines and the other requirements established in "Chapter 3: Hydrology."

4.5.1 Methodology

Headwalls and endwalls shall be used on culverts to control the erosion and scour resulting from excessive velocities and turbulence, to prevent the adjacent soil from sloughing into the culvert waterway opening and to prevent movement of a culvert due to hydraulic pressures. Headwalls and endwalls with or without wingwalls and aprons shall be constructed of reinforced concrete, and in accordance with engineering drawings as required by the physical conditions of the particular installation.

Conditions at Entrance

The entrance geometry of a culvert is a controlling factor in the amount of flow that a culvert may pass. The design of culverts shall involve the consideration of energy head losses that may occur at the entrance. Entrance head losses may be determined by the following equation:

$$h_e = C_e\,(v_2^2 - v_1^2)/2g$$

Where:

h_e = Entrance Head Loss (ft)

v_2 = velocity of flow in culvert (ft/s)

v_1 = velocity of flow approaching culvert (ft/s)

C_e = entrance loss coefficient as shown in Table 4.4 (dimensionless)

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

Table 4-4: Entrance Loss Coefficients

Type of Structure and Design of Entrance	Coefficient C_e
Pipe, Concrete	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls: socket end of pipe (groove end)	0.2
Headwall or headwall and wingwalls: square-edge	0.5
Headwall or headwall and wingwalls: rounded (radius=1/12D)	0.2
Mitered to conform to fill slope	0.7
End-section conforming to fill slope*	0.5
Pipe or Pipe-Arch, Corrugated Metal**	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls: square-edge	0.5
Mitered to conform to fill slope	0.7
End-section conforming to fill slope*	0.5
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls): square-edged on 3 edges	0.5
Headwall parallel to embankment (no wingwalls): rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel: square-edged at crown	0.4
Wingwalls at 30° to 75° to barrel: crown edge rounded to radius of 1/12 barrel dimension	0.2
Wingwalls at 10° to 25° to barrel: square-edged at crown	0.5
Wingwalls parallel (extension of sides): square-edged at crown	0.7

Source (Pipe, Concrete and Box, Reinforced Concrete): American Concrete Pipe Association, *Concrete Pipe Design Manual*, 2011.

Source (Pipe or Pipe-Arch, Corrugated Metal): American Concrete Pipe Association. *Concrete Pipe Handbook*, 1980.

*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 have a superior hydraulic performance.

**Corrugated Metal Pipe is included in the table for analysis purposes only. Corrugated metal pipe shall not be used for new design projects.

Types of Headwalls

Several types of headwall entrances may be used for culverts. The approach velocity is the determining factor in the type of headwall or entrance structure chosen. Typical headwall types and conditions are as follows:

A. Parallel Headwall – constructed parallel with the roadway above and provide stability against embankment erosion. They should be used in locations where the approach velocities in the channel are below 6 feet per second, and headwater pools are permitted.

B. Flared Wingwalls – help to further retain and protect the embankment by acting as a funnel or guide for the flow. They should be used where the approach velocities in the channel are between 6 and 12 feet per second, along with the addition of a concrete apron.

C. Skewed Wingwalls – similar to flared wingwalls, except the wingwalls are skewed toward the direction of flow relative to the orientation of the culvert. They should be used where the approaching flow is not parallel with the culvert.

D. Parallel Wingwalls – oriented parallel with the direction of flow through the culvert and are normally used to reduce the wingwall length required to protect the embankment. Parallel wingwalls will typically adversely affect the hydraulic operation of the culvert and should only be used on special non-hydraulic adaptations of culverts where drainage is a minor factor. Headwall designs based on the dimensions and reinforcement requirements of Texas Department of Transportation (TxDOT) design standards will be suitable for construction as long as the remaining requirements (such as velocity limits) of this section are also satisfied.

Debris Fins

For culvert structures which consist of more than one box culvert, and where the possibility of debris flow exists, debris fins may be required by the City Engineer. The debris fin is an extension of the interior walls of multiple box culverts. The thickness of the extended wall shall be designed to meet structural requirements as well as limit interference with the flow capacity of the culvert.

The debris fin shall be the same height as the wall at the structure and shall slope away from the culvert such that the length of the debris fin is one and one-half (1.5) times the height of the wall of the culvert.

Safety End Treatments

Culverts will have safety end treatments installed to protect the traffic on the crossing roadway and where necessary to comply with Texas Department of Transportation (TxDOT) criteria. For isolated instances where the culvert installation does not fit one of the categories below, the City Engineer shall specify the culvert end treatment to be used. The end treatments shall consist of one of the following:

A. For circular or arched culverts less than or equal to 24 inches in diameter, where the vertical distance from the roadway edge to the stream flowline of the channel is less than 3 feet, the ends of the culvert shall be sloped with the embankment at a slope not greater than 7 feet horizontal to 1 foot vertical, or

B. For circular or arched culverts which do not meet the criteria above or cannot be sloped as required, and for box culverts, metal beam guard fence shall be installed in accordance with TxDOT Standards, or

C. Culverts may also use pipe runners or metal gratings designed after TxDOT standards. To use pipe runners or metal gratings as safety end treatments in lieu of metal beam guard fence, the end of the top of the culvert barrel must be at least 12 feet from the edge of the nearest uncurbed traffic lane.

Selection of Culvert Size and Flow Classification

The flow in culverts is controlled in one of two ways: (1) Inlet Control or (2) Outlet Control. With Inlet Control, the discharge rate is independent of the length of pipe, the slope, or the roughness of the pipe wall. The discharge is dependent only upon the depth of the headwater elevation above the invert at the entrance, the pipe size and the entrance geometry. With Outlet Control, the discharge rate is affected by all hydraulic factors upstream of the outlet. These factors are headwater elevation, entrance geometry, pipe size, wall roughness, barrel length and slope. Tailwater elevation will be a factor if the tailwater depth is greater than the critical depth.

Submittals of models and culvert design calculations shall be in accordance with Chapter 2.3. Sufficient information shall be submitted so that the City Engineer is able to complete a thorough review of the design calculations.

4.5.2 Design Criteria

Developer shall refer to the City of Lubbock Design Standards and Specifications for applicable culvert design parameters. Culvert materials shall comply with requirements for storm drain pipe, provided in the *Permissible Pipe Materials* subsection in Section 4.3.2.

Erosion and Sedimentation Control

Outlet protection of culverts shall be employed to prevent scour and erosion damage to the structure, stream bed and embankment. Endwalls are the downstream companions of headwalls and can duplicate the embankment and culvert protection features of headwalls. Endwalls and associated structures should provide an efficient flow transition from potentially higher velocity pipe flow to the less concentrated stream or ditch flow. The designed structure required for this transition thus depends on an examination of the expected exit velocity from the culvert and the ability of the surrounding soils or earth to withstand the erosive aspects of the flow. Velocities in culverts are normally limited to the maximum allowed in the downstream channel. However, the concentrated flow exiting from a culvert can still produce scour and eddy currents that can undermine the endwall structures and damage the stream bed and the embankment.

Structures that attempt to decrease the erosive qualities of exiting flow include riprap, aprons, endwalls and wingwalls. The minimum outlet protection used will involve either riprap or an apron and will be based on the discharge velocity.

A. For outlet velocities below 6 feet per second, riprap protection consisting of a 4-inch minimum thickness of concrete or alternate approved material will be required. The length shall be as computed using the equation shown below, and the rip-rap width should cover at least two pipe diameters on each side of the outlet.

B. Aprons – For outlet velocities above 6 feet per second, a minimum 6-inch thick reinforced concrete apron structure with a toewall will be required. The minimum length of the apron in the culvert design, or the length determined from the following equation, whichever is greater:

$$L = {}^{(VD)}/_{5}$$

Where:

L = Length of Apron (ft)

V = Discharge Velocity (ft/s)

D = Height of Box Culvert or Diameter of Pipe Culvert (ft)

C. Endwalls – Examples of acceptable endwall design may be found in the City of Lubbock Design Standards and Specifications. The endwall size should be sufficient to support the culvert end, and act as a retaining wall for the surrounding soils if necessary. To avoid scour behind the endwall, the face should extend as high as the expected tailwater depth, and the endwall face width should preferably be wider than the low-water downstream channel. Also, at a minimum, the endwall should extend into the stream bed below the invert to a depth below the frost line. This type of installation may also require downstream streambed protection in the form of concrete riprap.

D. Wingwalls – Wingwalls can be used as retaining walls, and/or to contain erosive eddy currents at higher tailwater depths. If high tailwater depths are expected, the installation of an apron is also required to reduce the hazard of undercutting the wingwalls. The wingwall length shall match the designed apron, while the orientation of the wingwalls is generally determined by site conditions.

Where discharge velocities exceed 12 feet per second and the hydraulic characteristics of the culvert cannot be modified to reduce the discharge velocities, energy dissipaters shall be used to control downstream erosion. Refer to Section 4.7 for additional information on energy dissipation.

4.6 BRIDGE DESIGN

4.6.1 Methodology

A hydraulic analysis of any proposed bridge shall be completed to determine the impacts to water surface elevation and velocities. The analysis shall be completed using appropriate software as identified in Section 2.3. Fully developed flows should be used in all hydraulic analyses.

The contraction and expansion of water through bridge openings results in hydraulic losses. These losses are accounted for in the hydraulic model through the implementation of contraction and expansion coefficients. Recommended values for the contraction and expansion coefficients are provided in Table 4.5 below.

Table 4-5: Contraction and Expansion Coefficients

Transition Type	Contraction (K_c)	Expansion (K_e)
Gradual Transition	0.1	0.3
Typical Bridge	0.3	0.5
Severe Transition	0.6	0.8

4.6.2 Design Criteria

The following design criteria should be adhered to for the hydraulic design of all bridges.

A. Bridges should be designed to pass the design storm event with a minimum of two feet of freeboard between the proposed low chord elevation of the bridge and the anticipated water surface elevation. Refer to Section 3.1.1 for minimum design frequency events for bridges.

B. No increases to the fully developed 100-year water surface elevation are allowed upstream of the bridge.

C. Excavation of the natural channel is not allowed as compensation for loss of conveyance.

4.6.3 Scour Analysis

A scour analysis for all proposed bridges shall be completed in conjunction with the hydraulic analysis of the structure. Such analysis shall be completed according to the following guidelines:

A. TxDOT guidelines in “Evaluating Scour at Bridges” (HEC-18).

B. Bridge foundations will be designed to withstand the scour depths for either the 100-year flood, or a smaller flood if it will result in scour depths deeper than the 100-year flood.

C. An analysis of abutment scour does not need to be completed; however, abutments shall be protected against potential scour through use of flexible revetment, where possible, or hard armoring as necessary.

D. Check the bridge foundations against the scour depth associated with the 500-year flood. This flood event is considered an extreme event, and the factor of safety on the bridge foundations shall be greater than or equal to 1.0.

4.7 ENERGY DISSIPATION

Energy dissipaters are used to dissipate excessive kinetic energy in flowing water that could promote erosion. An effective energy dissipater must be able to retard the flow of fast-moving water without damage to the structure or to the channel below the structure. Energy dissipation often involves a hydraulic jump, which occurs when supercritical flow has its velocity reduced to a subcritical range. There is no ordinary means of having a smooth transition from supercritical flow to subcritical flow, therefore, there is a drastic change in water surface profile. The hydraulic jump is characterized by a steep upward slope of the profile with a great turbulent action. A poorly formed hydraulic jump is called an undular jump.

Drop structures may be used in channels to provide energy dissipation in a controlled section and lower the velocity of the flow. The function of a drop structure is to reduce channel velocities by allowing for flatter upstream and downstream channel slopes. Design of a drop structure may include stilling basins, transitions, baffle chutes, or other structures. Their shape, size, and other features vary widely depending upon the function to be served on a specific project.

The flow velocities in the upstream and downstream channels of the drop structure need to satisfy the permissible velocities allowed for channels. The design parameters for the sloping channel drop and the vertical channel drop are given below. Note that all proposed drop structures and energy dissipaters must include approved hydraulic models for the downstream to upstream analysis in order to verify approximate locations of hydraulic jumps, critical depth, and related phenomena.

Sloping Channel Drop

A. Approach Apron: A minimum ten (10) foot long concrete riprap apron should be constructed immediately upstream of the drop to protect against the increasing velocities and turbulence, which results as the water approaches the sloping portion of the drop structure. The same riprap and bedding design should 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, usually with vertical side walls. The length, L, of the chute depends upon the hydraulic characteristics of the channel and drop. For a unit 100-year discharge, q, of 30 cubic feet per second per foot, L would be about 15 feet, that is, about ½ of the q value. L should not be less than ten (10) feet, even for low q values.

C. Downstream Apron: The length of the downstream apron shall be sized according to Table 4.6 and shall be constructed of reinforced concrete or riprap depending on structural requirements. The steepest allowable side slope for the riprap stilling basin is 4:1. The riprap would extend up the side slopes to a depth equal to one (1) foot above the normal depth projected stream from the downstream channel.

Table 4-6: Length of Downstream Apron

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

Vertical Channel Drops

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

A description of the drop structure and the design procedure, going from upstream to downstream, is given below.

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

B. Approach Apron: A minimum ten (10) foot long concrete riprap apron is provided upstream of the drop to protect against the increasing velocities and turbulence which result as the water approaches the vertical drop.

A. Chute Apron: The riprap stilling basin is designed to force the hydraulic jump to occur within the basin and is designed for essentially zero scour. A detailed analysis of the hydraulic jump is required to determine the length of the riprap stilling basin. The steepest allowable side slope for the riprap stilling basin is 4:1. The riprap would extend up the side slopes to a depth equal to one (1) foot above the normal depth projected stream from the downstream channel.

B. The maximum fall allowed at any one drop structure is four (4) feet from the upper channel bottom to the lower channel bottom.

Impact-type energy dissipaters 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 impact-type energy dissipaters.

Other energy dissipaters 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 are this type of dissipater, where energy is diffused as flow plunges into a pool of water.

Generally, the impact-type of energy dissipater is considered to be more efficient than the hydraulic jump-type. Also, the impact-type energy dissipater results in smaller and more economical structures.

The design of energy dissipaters 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 Dissipaters.* Two impact-type energy dissipaters are briefly explained here.

Baffled Apron/Impact Basin (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 tailwater forms a pool into which the flow discharges. The chutes are constructed on a slope that is 2:1 or flatter and extends below the channel bottom. Backfill is placed over one of more bottom rows of baffles to restore the original streambed elevation. When scour of 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 patter.

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

C. 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.

D. Chute Entrance – The flow entering into the chute should be well distributed laterally across the width of the chute. The velocity should be well below the critical velocity, which can be calculated as Vc = (gq)^(1/3) for a rectangular channel.

E. Chute Design – The chute is usually constructed on a 2:1 slope. The upstream end of the chute floor should be joined to the horizontal floor by a curve to prevent excessive vertical contraction of the flow.

Baffled Outlet

Baffled outlets are used to dissipate the discharge energy from flow in a pipe or culvert through the creation of a hydraulic jump. 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.

5.0 PLAYAS

5.1 PLAYA DEFINITION

Playas are natural ground surface depressions which retain surface water runoff but lack the ability to sufficiently drain after a storm event. Some playas permanently retain surface runoff water, others are ephemeral, and still others have permanent water levels due to perched or shallow ground water conditions. The only two natural causes of significant water loss from a playa are evaporation to the atmosphere and infiltration through the playa bottom to the soil subsurface. Any other type of water loss must be man-made, such as pumped withdrawal for irrigation, pumped withdrawal to a separate point of discharge or withdrawal through a gravity storm drainage system.

The City of Lubbock has identified playa systems in its National Flood Insurance Study (FIS) and its Master Drainage Plan (MDP). A playa system is a group of playas and their subbasin drainage areas that, provided each one overflowed to the next downstream playa, would exhibit a continuity of flow independent of any other playa system. The playas systems assigned identifying labels by the City are labeled alphabetically, beginning with System A. The letter "L" has not been used as a playa system identifier because the City's geographic information system uses that identifying letter to retrieve digital information on individual playas. Detailed playa and playa system information is contained in the MDP and shall be referenced during the development process.

The overall drainage boundary around a playa system is referred to as that system's drainage basin, while the drainage area to an individual playa is referred to as a subbasin. Each playa system subbasin in Lubbock's MDP and FIS has been assigned a unique alphanumeric label and each playa has also been assigned a unique numeric label. The drainage area of a subbasin contributing to a playa shall not be increased or decreased by development surface grading activities without a detailed impact analysis and request for variance to the City Engineer. It is recommended that the Developer confirm with the City Engineer all subbasin boundaries adjacent to the proposed development early in the development planning process.

The naturally-occurring playa systems as defined by the natural topography shall be maintained through post-developed conditions. Stormwater runoff transfers from one playa system to a separate playa system are not permissible without detailed analysis to ensure no rise in water surface elevations, and acceptance by the City Engineer.

In the event that a playa pump station is proposed as part of a development, it will be subject to review and approval of the City Engineer. Any playas that are part of a proposed development and not owned by the City at the time of the development must meet the requirements laid out in Ordinance Chapter 38.06, Playa Lakes Development and Ownership, in order to be accepted by the City.

5.2 PLAYA CLASSIFICATIONS

The City of Lubbock has classified each playa into one of two categories, Overflow and Non-Overflow, dependent on the overflow characteristics of the playa. Playas are classified according to the characteristics described below.

Non-Overflow Playa

A. Storage volume is sufficient to completely contain the combined runoff from its subbasin's initial condition runoff, its subbasin's FFD condition 100-year, 24-hour runoff, as well as the overflow volume contributed to it from upstream playas.

B. The playa may experience overflow discharge during a 500-year, 24-hour storm event.

Overflow Playa

A. Storage volume is NOT sufficient to completely contain the combined runoff from its subbasin's initial condition runoff, its subbasins FFD 100-year, 24-hour runoff, and the overflow volume contributed to it from upstream playas.

B. The calculated water surface elevation in the playa assuming the above-listed runoff volumes is greater than the playa's natural overflow elevation. The water surface elevation may be determined by routing flood hydrographs through the playa.

5.3 OVERFLOW ROUTES

Overflow routes for playas are those conveyance areas that allow discharges from one playa to flow to the next downstream playa. Overflow routes for playas shall not be altered to the extent that the hydrologic routing sequence of a post-developed playa system is different from that sequence existing prior to development. In other words, a playa's overflow discharge cannot be redirected to a different receiving playa within the same playa system.

The natural paths of overflow routes between playas shall be followed to the maximum extent possible through post-development. This not only follows the natural course of the runoff water but is also economically effective. As an example, if a proposed street is planned to convey potential overflow from one playa to another, then that street shall follow the natural path that the concentrated flow follows in pre-development conditions.

5.4 PLAYA REQUIREMENTS

The City of Lubbock recognizes that development activities around playa overflow discharge areas and along overflow routes will have impacts upon the conveyance characteristics of these features. The City shall be able to evaluate the suitability and adequacy of design elements associated with any proposed changes, construction, or other reconfiguration of these features. An adequate analysis of the effects of all proposed changes on playa overflow discharge areas and along overflow routes shall be prepared and submitted to the City Engineer for review and approval.

5.4.1 Non-Overflow Playas

The following requirements apply to all development around Non-Overflow Playas:

A. At least one overflow route shall be identified and kept clear of obstructions to allow for adequate conveyance in situations in which emergency drawdown discharges are necessary. An overflow route shall be a street, public right-of-way, or drainage easement.

B. Minimum finished floor elevations shall be set in accordance with Chapter 2.

C. Maximum target flow depth in streets shall be 12 inches above the gutter elevation during the 100-year event, as detailed in Chapter 4. In areas subject to playa backwater during the 100-year event, the maximum depth shall be calculated two ways: 1) Assuming local street drainage, and 2) Assuming playa backwater effects. Final gutter elevations shall be determined based on the higher of the two.

5.4.2 Overflow Playas

The following requirements apply to all development around Overflow Playas:

A. Overflows shall not be obstructed by development, nor shall development encroach into the overflow area.

B. The overflow elevation of a playa shall not be altered without analysis.

C. Where an Overflow Playa exhibits more than one overflow route at the 100-year, 24-hour peak water surface elevation, the post-development overflows and route conveyances shall maintain the same proportional discharges as existed prior to development. Overflows shall not be obstructed to reduce the number of overflow directions from a playa. For example, if a playa had two overflows where one overflow carried 60% of the peak discharge and the other overflow carried 40% of the peak discharge, then the same 60-40 peak discharge proportion shall be maintained in post-development conditions.

D. Sometimes, adjacent playas receive sufficient volumes of stormwater runoff to raise the water surface elevation in the playas high enough that two or more playas achieve a common water surface elevation and act as a single playa. For this reason, multiple playas that are predicted to achieve a common water surface elevation so that they act as a single unified playa shall not be hydraulically isolated from one another by man-made construction or development. Any man-made hydraulic connection (culverts, storm drains, etc.) shall have sufficient capacity to minimize the water surface differential at 100-year, 24-hour peak flow rates from one playa to another.

E. Minimum finished floor elevations shall be set in accordance with Chapter 2.

F. Maximum target flow depth in streets shall be 12 inches above the gutter elevation during the 100-year event, as detailed in Chapter 4. In areas subject to playa backwater during the 100-year event, the maximum depth shall be calculated two ways: 1) Assuming local street drainage, and 2) Assuming playa backwater effects. Final gutter elevations shall be determined based on the higher of the two.

G. The post-development conveyance capacity of an overflow section of a playa shall be equal to or greater than the capacity of the naturally occurring overflow section. A request for variance with supporting documentation shall be submitted to the City Engineer for any proposed encroachment into an overflow area demonstrating that the conveyance capacity and overflow route of the playa will not be decreased. This documentation shall be reviewed and approved by the City Engineer.

5.5 VOLUMETRIC MITIGATION

The City of Lubbock recognizes that some reclamation and/or alteration of playas can be allowed, within limits, so that viable development can take place and appropriate layouts of those developments can be achieved. The limits within which playa reclamation and/or alteration will be allowed are detailed in the following sections.

5.5.1 Runoff Storage Preservation

The natural storage volume below the 100-year, 24-hour peak water surface elevation shall be maintained or increased for Overflow Playas. The natural storage volume below the 500-year, 24-hour peak water surface elevation shall be maintained or increased for Non-Overflow playas. The appropriate water surface elevation for the volume balance determination shall be obtained from the most current version of the City's MDP. Storage volumes shall be maintained by offsetting all proposed fill within the playa below the above referenced water surface elevations with an equal amount of excavation below this elevation.

For Overflow Playas, the area of allowable reclamation below the 100-year, 24-hour peak water surface elevation shall not exceed 30% of the playa surface area at the 100-year, 24-hour peak water surface elevation. For example, if the surface area of a playa at its overflow elevation is 60 acres, then the allowable reclamation area is 30% of 60 acres, or 18 acres. This type of reclamation is illustrated in Figure 5-1.

For Non-Overflow Playas that retain the 500-year, 24-hour storm event, the allowable reclamation area shall not exceed 30% of the playa surface area at the 500-year, 24-hour water surface elevation. The total storage volume below the 500-year, 24-hour water surface elevation shall be maintained. This scenario is illustrated in Figure 5-2. If a Non-Overflow Playa exhibits overflow characteristics for the 500-year, 24-hour storm event, then the allowable reclamation area shall be 30% of the surface area at the overflow elevation.

The allowable 30% recovery area does <u>not</u> include ancillary areas within the public rights-of-way of streets and/or alleys. In no case shall the overflow elevation of a playa be altered, nor shall encroachments into overflow area be allowed that will decrease the allowable discharge through the overflow.

The City allows the operation of constant level playas wherein a constant water surface elevation is maintained. Such playas can be aesthetically pleasing, contribute to the quality of life and enhance the value of a development or recreational facility. The playa's natural storage capacity, however, shall be maintained, and the constant water level operations shall not encroach into that storage capacity.

5.5.2 Side Slopes

Where playas incorporate reclamation areas as discussed above, or where playas are altered or reconfigured to increase storage, the proposed fill shall not be placed on a slope steeper than 7 feet horizontal to one foot vertical (7H:1V). Proposed excavation shall be accomplished at a cut slope of no steeper than 7H:1V. Similar to open channel design discussed in Chapter 4, side slopes steeper than 7H:1V may be permitted on a case-by-case basis by the City Engineer. Such an exception shall require geotechnical investigations and analysis to demonstrate that steeper slopes are appropriate. Playa excavation shall not occur within 10 feet of any adjacent property line or public right-of-way.

5.5.3 Aesthetics

The City of Lubbock encourages the incorporation of aesthetically pleasing features into the alteration of playas. If a playa is altered from its natural shape, the new playa configuration shall fit the development layout and topography through the use of gradual curves and transitions. Square and rectangular pits are not desirable. Playa alterations shall meet the aesthetic requirements in Ordinance Chapter 38.06, Playa Lakes Development and Ownership. The final playa configuration is subject to approval by the City Engineer prior to construction.

5.5.4 Variances

Deviations from the volumetric mitigation requirements discussed in the previous sections may be considered on a case-by-case basis. The Engineer is allowed to provide alternative methods of playa reclamation if able to demonstrate that the proposed methods adequately address safety concerns, water surface elevation requirements, discharge requirements, overflow conveyance requirements, and minimize potential erosion problems. Variances shall require the consideration and approval of the City Engineer.

Cut/fill plans for playa reclamation shall be in accordance with Chapter 2 of this manual, as well as the City's Subdivision Regulations in the Code of Ordinances. Erosion control around playas is addressed is Chapter 9 of this manual.

5.6 PLAYA ANALYSIS

Hydrologic and hydraulic analyses of playas and playa systems are complex and require specific assumptions. In order to maintain consistency, the following sections provide guidance as to the appropriate methodologies and assumptions to be used in the analysis of playas and playa systems in the City of Lubbock.

5.6.1 Hydrology Calculations

Allowable hydrologic methods to determine stormwater runoff quantities are discussed in detail in Chapter 3 of this manual. All analyses involving a playa or playa system require the use of a hydrograph model to accurately represent the attenuation effects of the playa, as well as evaluate the timing of peak inflows and outflows from the playa.

The model may involve a single playa and its subbasin, a playa sub-system or an entire playa system, depending on the direction given by the City Engineer. The minimum storm event analyzed for Overflow Playas shall be the 100-year, 24-hour storm event. The minimum storm event analyzed for Non-Overflow Playas shall be the 500-year, 24-hour storm event. The City Engineer may also require that the 500-year, 24-hour storm event be analyzed for Overflow Playas in certain instances. When available, the hydrologic model from the MDP shall be used as the basis for hydrologic calculations; otherwise, the best available data shall be used.

5.6.2 Storage Calculations

Playa storage calculations shall be completed using the average end area method based on topography data that meets the accuracy requirements described in Chapter 2. If bathymetry data is available and meets the accuracy requirements of Chapter 2, it may be used to calculate storage volumes in a playa below the water surface. Where a natural playa is to be altered, reclaimed or reconfigured, the elevation-area-storage relationship for both the natural playa condition and the changed playa condition shall be analyzed.

5.6.3 Initial Playa Condition

A playa will exhibit fluctuations in the volume of water being stored with time. The volume of water in a playa at any given time is dependent on inflow to the playa versus evaporation and infiltration from the playa. When available, the initial playa condition shall be taken directly from the MDP for use in the hydrologic and hydraulic analyses. If the initial playa condition for the playa(s) of interest is not available in the MDP, the following guidelines shall be used to determine the most appropriate initial playa condition. The Engineering Department shall be consulted to confirm the initial playa condition assumptions and achieve consensus on other assumptions necessary in the hydrologic and hydraulic analyses.

Scenario 1: Playa appears as though it is normally dry, and there is no apparent "normal pool" elevation as evidenced by a well-defined dry-land vegetation line or an erosion line caused by wave action.

Initial Condition Assumption: A volume of water equivalent to 1.4 inches of runoff from the playa subbasin shall be placed into the dry playa. The resulting water surface elevation shall be used as the initial playa condition for the hydrologic analysis.

Scenario 2: Playa appears dry, or a reduced water surface elevation caused by an extended lack of runoff is apparent, but there is evidence of a normal or frequently realized pool elevation, such as a well-defined dry-land vegetation line or a wave-action bank erosion line.

Initial Condition Assumption: A volume of water equivalent to 1.4 inches of runoff from the playa subbasin(s) shall be placed in the dry playa. The resulting water surface elevation shall be compared to the dry-land vegetation line or a wave-action bank erosion line, and the higher elevation used as the initial playa condition.

Scenario 3: Playa maintains a relatively consistent normal pool elevation, as evidenced by a dry-land vegetation line, historic pool elevation information, photographic information, ground water elevations or other appropriate indicators.

Initial Condition Assumption: A volume of water equivalent to 1.4 inches of runoff from the playa subbasin(s) shall be placed in the dry playa. The resulting water surface elevation shall be compared to the dry-land vegetation line or a wave-action bank erosion line, and the higher elevation used as the initial playa condition.

Scenario 4: Playa is drained to an established elevation by a gravity storm drain, or other reliable means.

Initial Condition Assumption: The outflow invert elevation shall be used as the initial playa water surface elevation. The City Engineer shall be consulted for applicability of this assumption on a case-by-case basis.

Scenario 5: Evidential data suggests that the playa is typically full, with a pool elevation equal to the overflow elevation, or the 1.4 inches of direct runoff volume equals or exceeds the available playa storage volume at the overflow elevation.

Initial Condition Assumption: The overflow elevation shall be used as the initial playa water surface elevation. The excess volume from the 1.4 inches of runoff that exceeds available playa storage volume at the overflow elevation shall be discarded. This volume shall not be transferred as overflow volume to the next downstream playa.

For the scenarios described above, all subbasins that contribute runoff to the playa under consideration shall be included in the volume calculation, unless otherwise directed by the City Engineer.

The proposed initial water surface elevation in the playa shall be reviewed and approved by the City Engineer. The City Engineer has the authority to require a higher starting water surface elevation if evidence supports such an assumption. Once the initial playa condition has been established and approved by the City Engineer, complete final calculations and/or final computer models shall be submitted to the City so that the City can update the MDP to reflect the most current data available.

5.6.4 Hydrologic Playa Routing

As noted previously, hydrograph computer models are required where the hydrologic flood routing involves a playa. Such flood routing takes into account the relationship of the rate of inflow to a playa, the elevation-storage characteristics of the playa under consideration, and the elevation-discharge relationship of the playa. Based on these relationships, it is possible to determine the peak water surface elevation in the playa during various storm events.

5.7 PLAYA EROSION

The City of Lubbock has historically experienced sheet and rill erosion around playas, along with gully erosion at the edges and downstream ends of drainage flumes. Such erosion has caused the City to incur costs from the standpoint of maintenance around playas, regardless of whether those playas are designated as stormwater retention areas or as parks.

For developments around a playa where there is cut, fill or other disturbance of the playa from its natural condition, the Developer will need to incorporate measures to control sheet and rill erosion on the disturbed areas. The types of sheet and rill erosion control measures are dependent on the drainage features at each playa and upon consultation with the City Engineer. Refer to Chapters 5 and 9 of this manual, as well as the City's MS4 Permit for guidance regarding typical erosion control measures around playas.

The Developer can propose gully erosion control measures other than those presented in the City's Standard Details and Specifications; however, any such measure will require the review and approval of the City Engineer. The designs for gully erosion control measures shall wholly contain the peak discharge from the 100-year, 24-hour storm event where the runoff is from the local playa sub-basin.

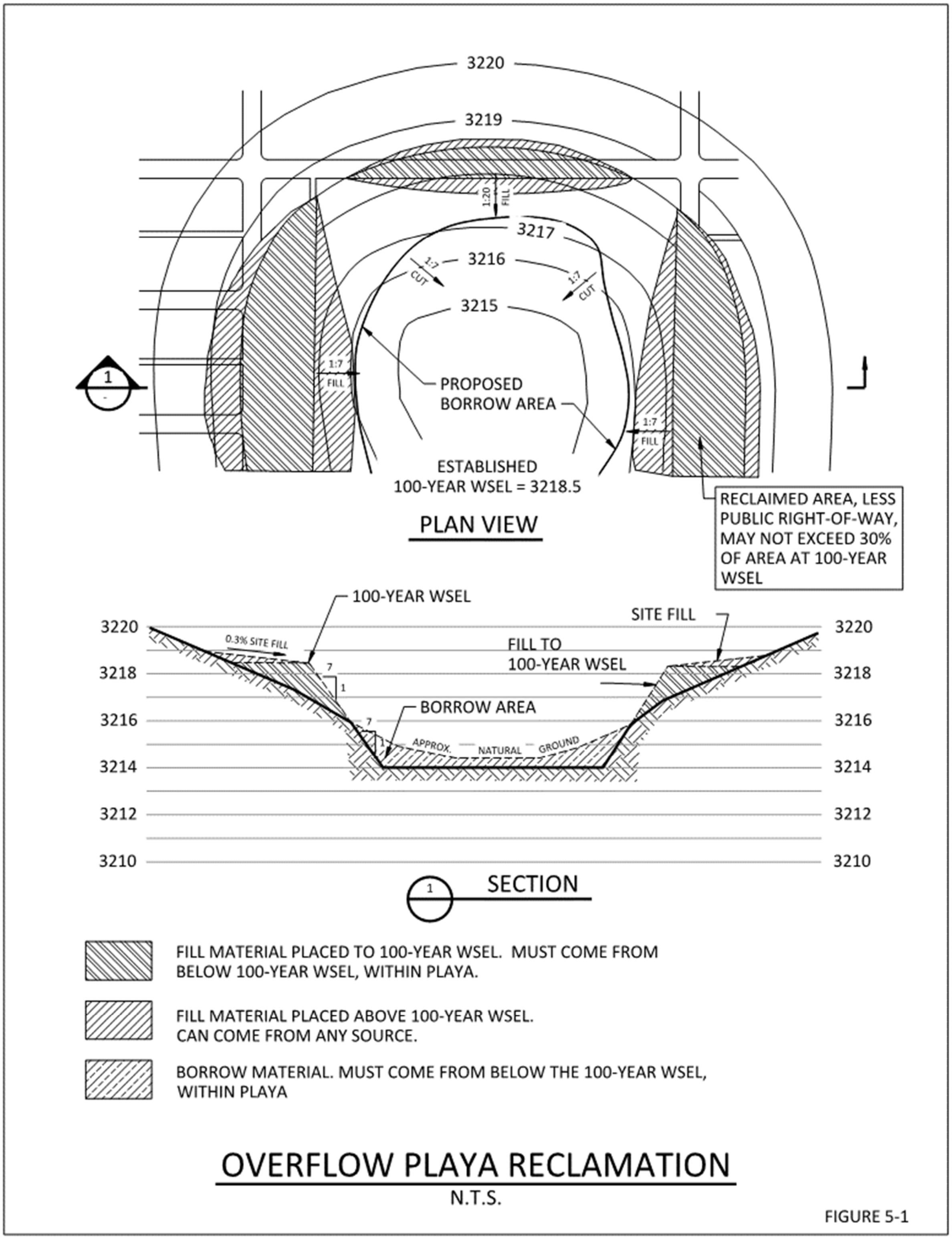


Figure 5.1: Overflow Playa Lake Reclamation

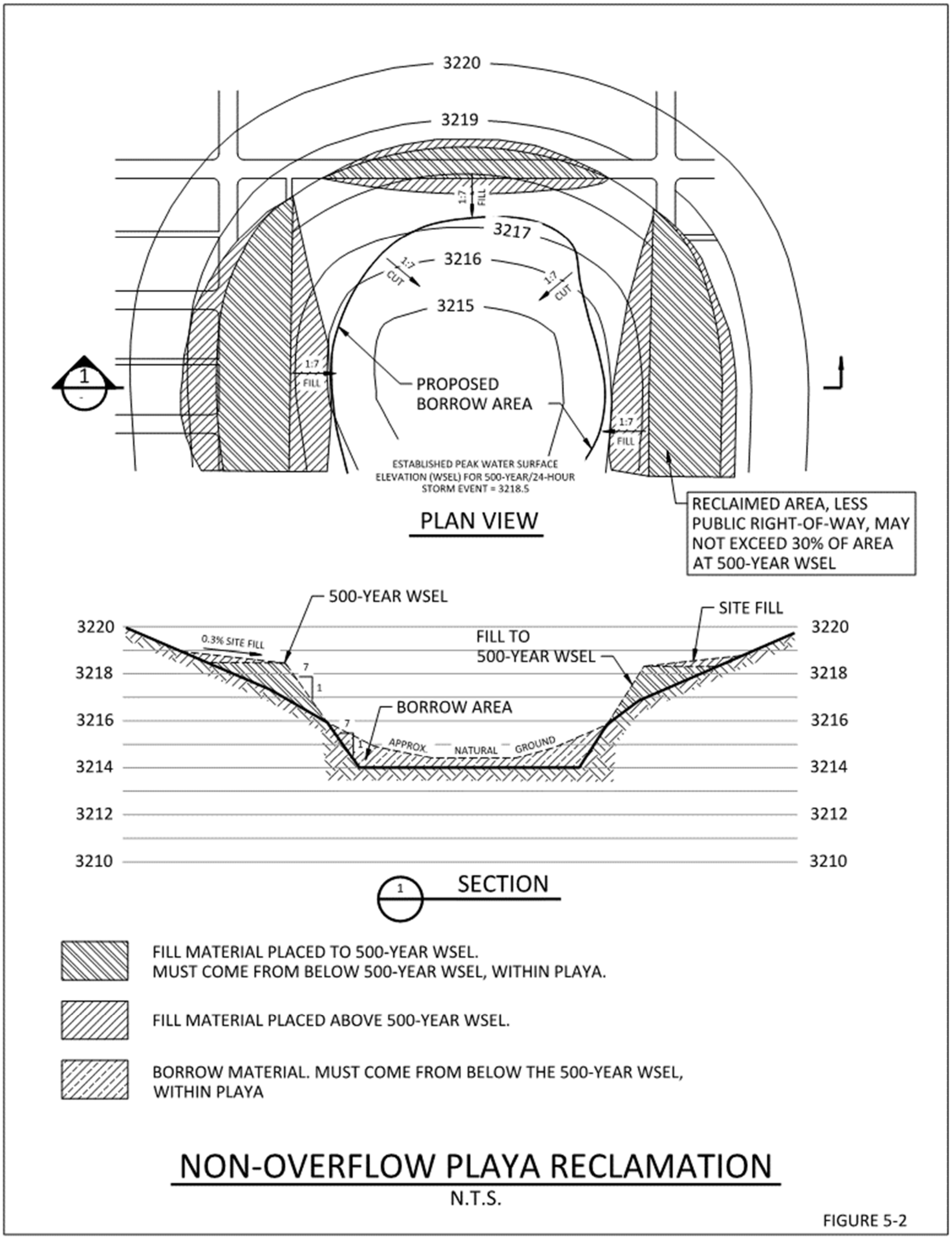


Figure 5.2: Non-Overflow Playa Lake Reclamation

6.0 DOWNSTREAM ASSESSMENT

The purpose of a Downstream Assessment is to determine the potential impacts to downstream areas due to proposed development within the watershed upstream. Increased flooding potential, insufficient downstream storm drainage infrastructure, and increases in erosion are all evaluated as part of the assessment. The ultimate goal of such an analysis is to identify the necessary mitigation measures, either in the proposed upstream development or in the downstream area, necessary to offset any such impacts. The importance of the Downstream Assessment is particularly evident for larger sites or developments that have the potential to dramatically impact downstream areas. The cumulative effect of smaller sites, however, can be just as significant.

The Downstream Assessment shall extend from the locations where increased discharges leave a proposed development to a point downstream as defined below in 6A, 6B or 6C. This point shall meet one of the following criteria, whichever results in the shortest distance:

A. The next downstream playa. Should the analysis indicate that impacts from the proposed development may propagate further downstream than the next downstream playa, the Downstream Assessment shall be extended to that point at which no hydraulic impacts are indicated.

B. The next downstream development with a final drainage analysis that has been accepted by the City. All boundary conditions from the proposed site (including peak flow and discharge location) must match the assumed inflow to the downstream development, as approved in the final drainage analysis report.

C. A point downstream from the proposed development at which there are no hydraulic impacts indicated, even if this distance is shorter than that determined in A or B above. The Developer must provide supporting calculations demonstrating that it is appropriate to end the Downstream Assessment at this point, and the City Engineer must approve.

Projects involving development or other reconfiguration within playa overflow routes require special consideration during a Downstream Assessment. As discussed in Chapter 1, in many cases, updated hydrologic and hydraulic models developed to predict future fully developed (FFD) conditions within various playa watershed systems will be available in the MDP. These models shall be used as the basis for the new analysis. Complete final calculations and/or final computer models shall be submitted to the City upon approval of the Final Drainage Report by the City. The final approved calculations and/or computer models will be used by the City to update the MDP to reflect the most current data available.

Downstream Assessments shall be completed assuming the following combinations of hydrologic and hydraulic conditions:

A. Existing hydrologic and hydraulic conditions on proposed site with existing hydrologic and hydraulic conditions on all remaining subbasin(s), both upstream and downstream, included in the analysis. This establishes the baseline scenario.

B. Existing hydrologic and hydraulic conditions on proposed site, with FFD hydrologic conditions on all portions of upstream watershed draining to proposed site, and existing hydrologic and hydraulic conditions on all downstream subbasin(s).

C. FFD hydrologic and hydraulic conditions on proposed site and all portions of upstream watershed draining to proposed site, with existing hydrologic and hydraulic conditions on all downstream subbasin(s).

D. FFD hydrologic and hydraulic conditions on proposed site, existing hydrologic and hydraulic conditions on the upstream watershed, with existing hydrologic conditions on all downstream subbasin(s).

The Developer shall mitigate for hydrologic impacts resulting from the proposed development, which are quantified as the difference between Scenarios B and C, above. Hydraulic improvements downstream of the proposed site must be designed with sufficient capacity to convey FFD flows from the proposed site only (Scenario D); however, the width of the easement for the downstream improvements must be large enough that FFD flows from the entire upstream watershed may be contained (Scenario C). This width should be calculated assuming an earthen, trapezoidal channel.

Downstream Assessments shall be prepared and submitted to the City with the preliminary drainage plan. The assessment shall demonstrate that the development will produce no adverse impacts. No adverse impacts may include, but are not limited to:

A. No new or increased flooding (0.00’ increase in peak water surface elevation) of existing insurable (FEMA) structures (habitable buildings).

B. No increases in water surface elevations for the 2-, 25-, and 100-year storm events and no more than 5% increase in discharge is allowed. Dry lane and gutter capacity requirements set forth in Chapter 5 shall also be met. This criteria does not apply to FEMA floodplains.

C. Post-development channel velocities shall not be increased such that they are greater than the maximum allowable channel velocities. Refer to Chapter 4 for maximum allowable channel velocities. Exceptions to these criteria require a certified geotechnical/geomorphologic study that provides documentation that a higher velocity will not increase erosion.

D. No increases in downstream discharges caused by the proposed development that, in combination with existing discharges, exceed the existing capacity of the downstream storm drainage system or drainage easement. This criteria does not apply to FEMA floodplains.

E. If a playa is present within the area included in the Downstream Assessment, no increases in FFD playa water surface elevation, no impact to any existing structures within the area inundated by the FFD 100-year playa pool, and no increase in FFD water surface elevations and floodplain configuration along downstream playa overflow routes. An increase in water surface elevation is defined as 0.01 feet.

Downstream Assessment may be waived for any of the following conditions:

A. Sites proposing detention or retention and showing no increase in discharges.

B. Sites which will not increase runoff coefficient from existing conditions.

In the event that the Downstream Assessment indicates adverse impacts as described above, mitigation measures shall be required to mitigate the impact of increased discharges from the site. Mitigation measures may include either acquisition of off-site easement, construction of off-site improvements, detention or retention. Figure 6-1 shows this process. Refer to Chapter 7 for an explanation of detention and retention basins, as well as design criteria for each.

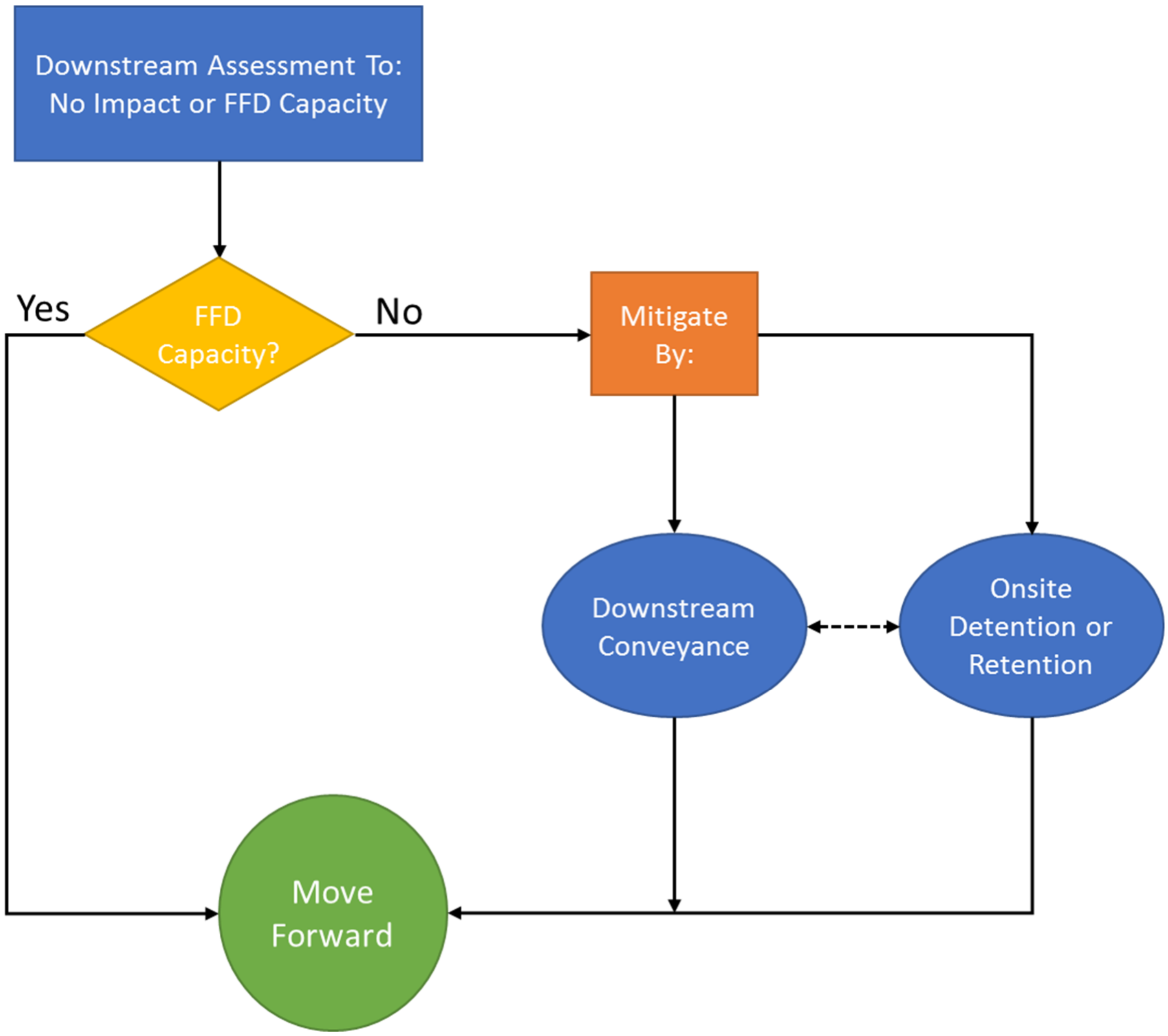


Figure 6.1: Downstream Assessment Process Flow Chart

7.0 DETENTION AND RETENTION

Stormwater detention and retention basins are used to temporarily impound stormwater runoff, thereby reducing peak discharge rates. Detention may be required for a proposed development if a Downstream Assessment indicates that the development may cause adverse impacts downstream without some form of discharge mitigation. A detailed discussion regarding situations where detention or retention may be required is provided in Chapter 6.

A detention basin has a free discharge point at the lowest elevation of the basin and is designed to drain completely following a storm event and remain dry between storm events. A retention basin may or may not have an outlet structure. A retention basin with an outlet has a free discharge point above the bottom of the basin and will typically store water below this elevation. Retention storage is regained only through evaporation and/or infiltration. This additional retention storage shall not be considered when calculating discharge mitigation due to storage without approval from the City Engineer. Detention storage may be available above the retention elevation. Detention/retention basins may improve water quality by allowing some sediment to settle out of the stormwater.

On-site detention/retention ponds may be used to provide flow rate (discharge) mitigation when required. Retention basins may also be used to provide volumetric mitigation.

Maintenance of all detention/retention basins requires the periodic removal of debris and sediment. Without maintenance, a basin may become unsightly, a social liability, or ineffective as a detention or retention basin. A Facilities Maintenance Agreement is required for all detention/retention basins. Maintenance is discussed in greater detail in Chapter 6. Also included in Chapter 2 is guidance regarding Downstream Assessments, and how they should be used to determine the need for detention/retention on a proposed project site.

7.1 METHODOLOGY

There are two acceptable methods for designing detention and retention basins in the City of Lubbock. For detention/retention basins with no upstream detention/retention basins, and with a single drainage area of 25 acres or less, the Modified Rational Method may be used, as described in Section 7.3. If the above criteria are not met, the Unit Hydrograph Method shall be used. The hydrologic model shall extend to the appropriate point downstream of the basin, as determined based on the Downstream Assessment completed in compliance with Section 2.4. Future fully developed conditions shall be assumed for all hydrologic design calculations, unless otherwise specifically stated.

7.2 MODIFIED RATIONAL METHOD CALCULATIONS

The Modified Rational Method is allowed for watersheds of 25 acres and less; however, this method is not acceptable for basins in series. The Modified Rational Method uses the peak flow calculating capability of the Rational Method in conjunction with assumptions about inflow and outflow hydrographs to compute an approximation of storage volumes for simple detention calculations.

The following equations are used to calculate the required volume:

$$Inflow = T_d * Q * 60$$

Where:

T_d = storm duration (min)

Q = flow rate calculated for given T_d (cfs), calculated using Rational Method as described in Section 3.2.1

$$Outflow = 0.5 * (T_c + T_d) * Q_0 * 60$$

Where:

T_c = time of concentration for proposed conditions (min)

T_d = storm duration (min)

Q_0 = existing conditions flow rate (cfs)

$$V = \frac{(Inflow - Outflow)}{43{,}560}$$

Where:

V = required storage volume (acre-ft)

This process of estimating inflow, outflow and required detention volume is iterated assuming multiple storm durations in order to find the critical storm duration that maximizes the required volume. In the case that this method is used to determine the required storage volume in a retention basin, the required storage volume determined from the calculations above shall be added to the proposed retention volume to determine the total volume required in the basin.

An example table for submittal of Modified Rational Method calculations in an acceptable format is provided in the appendix. If it is determined by the engineer that the Unit Hydrograph method is more appropriate to determine sizing for a given detention or retention basin, refer to Section 3.2.2 for detailed information regarding use of this method.

7.3 DESIGN CRITERIA

Figures 7-1 and 7-2 illustrate the key elevations and components related to detention and retention basins.

Detention and retention basins shall be designed based upon the following minimum criteria:

A. Detention shall be provided for the 2-, 25-, and 100- year design storms based on the results of the Downstream Assessment completed in accordance with Section 2.4. Sites without a Downstream Assessment will be required to provide detention to limit discharges to pre-development runoff rates. Should the Downstream Assessment results demonstrate that downstream facilities are adequate and on-site detention is not required, fully developed off-site conditions shall be taken into account for the on-site design facilities.

B. All detention/retention facilities shall demonstrate an acceptable outfall location. Acceptable outfall locations in the City of Lubbock include playas, streets with sufficient drainage capacity, or drainage easements. If it is not possible to obtain a drainage easement or to discharge into a playa or street with sufficient drainage capacity, post-development discharges shall mimic pre-development discharges for 2-, 25-, and 100-year events in flow rate, velocity and location, as defined in Section 6.0 Downstream Assessment.

C. Earthen embankments may be used to temporarily or permanently impound surface water for the purpose of detention/retention. They shall be designed by a Professional Engineer, licensed in the State of Texas, and shall be constructed according to specifications required based on geotechnical investigations of the site, as well as all regulatory requirements. The steepest side slope permitted for a vegetated embankment is 7 horizontal to 1 vertical (7H:1V); however, steeper side slopes may be permitted by the City Engineer on a case-by-case basis if hard-armoring of the slope is proposed. If the proposed embankment meets the definition of a dam according to Texas Administrative Code, Title 30, Part I, Chapter 299 (TAC §299) effective January 2009, or most current version, it shall be required to comply with all Texas Commission on Environmental Quality (TCEQ) dam safety criteria.

D. The effective crest of any detention/retention embankment shall be a minimum of one foot above the 100-year water surface elevation. Development will be permitted between the 100-year water surface elevation and the detention/retention structure crest; however, finished floor elevations of properties surrounding the basin shall comply with the requirements in Chapter 2.

E. Detention facilities shall be designed with an overflow spillway to protect the embankment in the event that the primary outfall ceases to function as designed, and/or in the event of a larger storm event than that used to design the primary outfall. The overflow spillway shall be designed to pass a minimum of the 100-year flood event while still providing for the minimum one foot of freeboard between the water surface elevation and the crest of the embankment. The overflow spillway shall not engage in a 25-year or more frequent event.

F. Excavation only retention facilities are not required to have an overflow spillway. In this instance, the retention basin shall be designed to contain the 100-year flood event while still providing for the minimum one foot of freeboard between the water surface elevation and the crest of the embankment. The initial water surface elevation in the basin shall be determined in accordance with Section 5.6.3. If it is desired to assume a lower initial water surface elevation, water balance calculations, including evaporation calculations, shall be submitted and approved by the City Engineer to support the use of the lower elevation.

G. Retention facilities that include construction of an embankment above existing ground shall comply with all requirements for detention basins.

H. The detention basin bottom shall be designed to provide positive drainage to the outlet, with a minimum slope of 0.2%.

I. Parking lots may be used to provide temporary detention of stormwater; however, the maximum allowable depth of ponded water shall be six inches during the 100-year, 24-hour event. No fire lanes may be located within the portion of a parking lot designated as detention storage.

J. A Facilities Maintenance Agreement is required for all detention/retention basins. The maintenance plan shall be developed in accordance with Section 2.5.

Detention/retention basins shall comply with all applicable federal, state and local regulations. These may include, but not be limited to, dam safety, water rights, and environmental regulations. Developer shall submit a statement signed by the Developer at the time of preliminary drainage plan submittal acknowledging that the proposed facilities will comply with all applicable federal, state and local regulations.

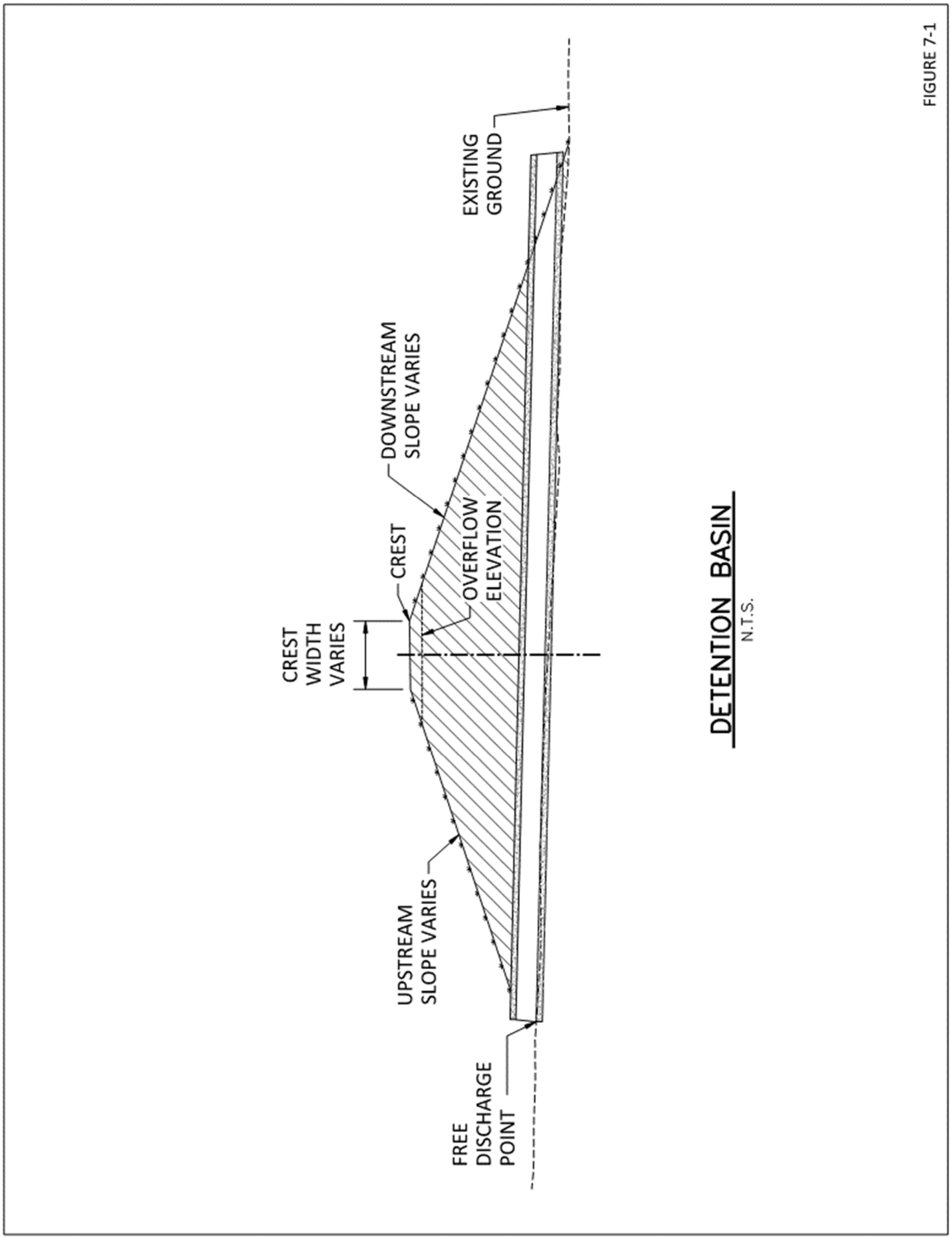


Figure 7.1: Detention Basin

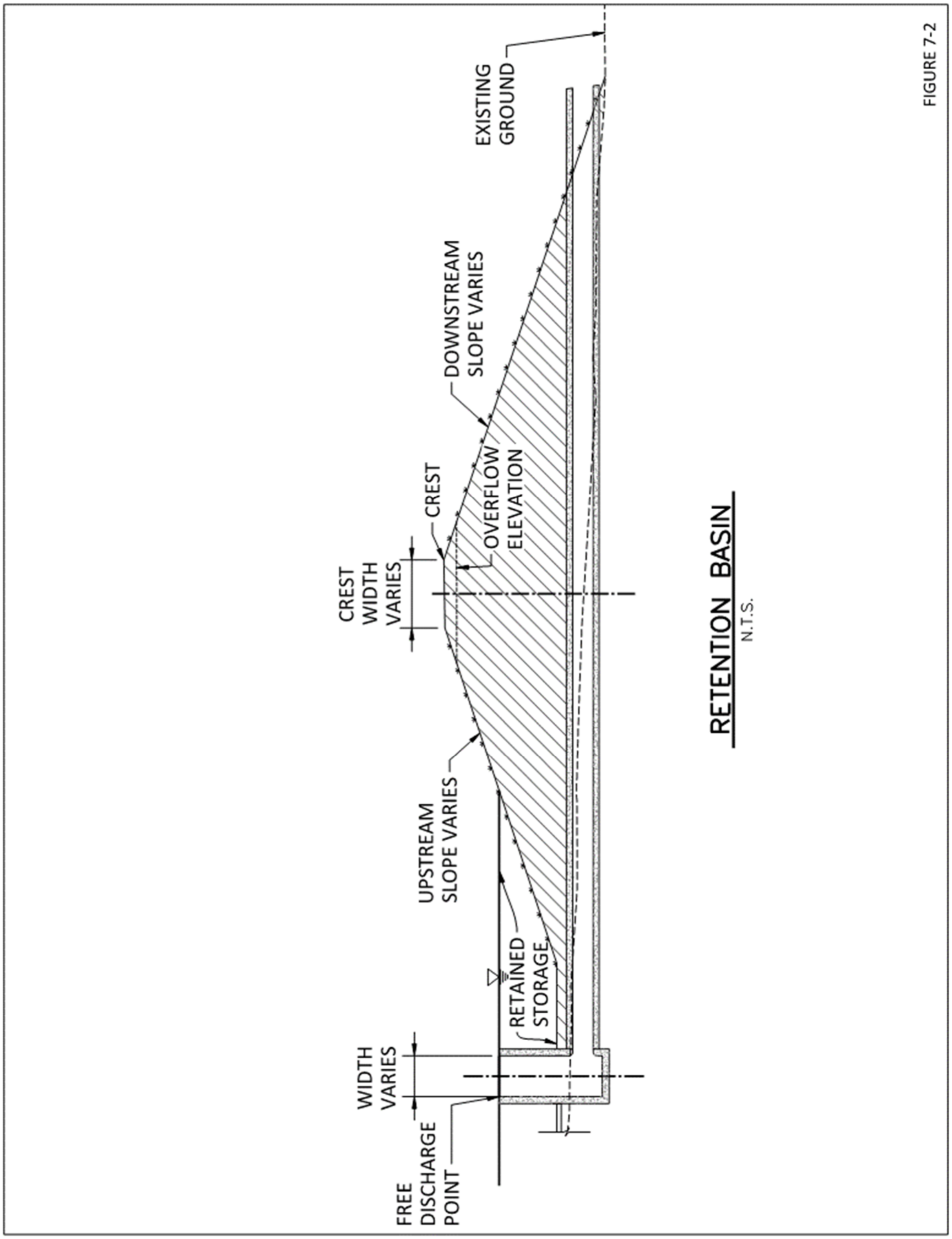


Figure 7.2: Retention Basin

8.0 FLOODPLAIN AND FLOODWAY DEVELOPMENT REQUIREMENTS

The Federal Emergency Management Agency (FEMA) administers the National Flood Insurance Program (NFIP), which enables property owners to purchase flood insurance. In return for making flood insurance available for existing structures, the participating community agrees to regulate new development within Special Flood Hazard Areas (SFHAs). Lubbock is also a participating city in the Community Rating System (CRS), which is a voluntary program for NFIP participating communities. The CRS provides incentives for participating communities to require flood protection above the minimum requirements developed by FEMA.

The specific regulations related to development and the NFIP can be found in the City of Lubbock Code of Ordinances. The design and submittal requirements that are affected by these regulations are discussed in the sections below.

8.1 FLOODPLAIN DEFINITIONS

There are two types of floodplains within the City of Lubbock, each based on different hydrologic assumptions:

> *The "City Floodplains" are the floodplains developed based on future fully developed (FFD) hydrologic conditions. These floodplains and corresponding water surface elevations may be found in the most current version of the MDP.*
>
> *The "FEMA Floodplains" are based on the most current version of the Flood Insurance Study (FIS) and Flood Insurance Rate Maps (FIRMs), as delineated by FEMA. The FIS and FIRMs are developed based on existing conditions land use at the time of the study, which differs from the City Floodplains that are based on FFD land use. FEMA Floodplains shall be used to determine the need for submittal of a LOMR with a proposed development.*

FEMA Floodplains are designated according to varying levels of flood risk. Numerous floodplain designations are used by FEMA; however, the three that will be most commonly involved in development projects within the City of Lubbock are Zone A (areas with a 1% annual chance of inundation developed without detailed analyses, no base flood elevations shown), Zone AE (areas with a 1% annual chance of inundation developed based on detailed analyses, base flood elevations shown), and shaded Zone X (areas with a 0.2% annual chance of inundation developed based on detailed analyses.

The term "floodway" is used to define the main channel of a river or watercourse, as well as the adjacent land areas that shall be reserved for conveyance to allow the discharge of the base flood without cumulatively increasing the water surface elevation by more than one foot. Development within a floodway is generally prohibited; however, exceptions to this policy may be made on a case-by-case basis at the discretion of the City Engineer.

Three procedures are available to change and/or correct an existing FEMA Floodplain. These include a Letter of Map Amendment (LOMA), Letter of Map Revision (LOMR), and a Physical Map Revision (PMR). These are defined as follows:

A. LOMA – Used to remove a specific area or property from a FEMA designated floodplain. A LOMA is completed based on a review of technical information and calculations relating to the property that demonstrates the area or property should not be included in the floodplain.

B. LOMR – Used to modify an effective FIRM. Completed after construction of physical measures that affect the hydrologic or hydraulic characteristics of an area. A LOMR is required when development is proposed in a Zone A in order to establish a base flood elevation (BFE).

C. PMR – An official republication of a FIRM to affect changes to flood insurance zones, floodplain delineations, flood elevations, floodways, and planimetric features.

It is possible to exclude a single structure from the floodplain by placing it on fill with sufficient freeboard above the base flood elevation. This procedure is referred to as a Letter of Map Revision – Fill (LOMR-F). LOMR-Fs shall not be allowed within playa overflow conveyance areas.

A Conditional Letter of Map Revision (CLOMR) may be required at the discretion of the City Engineer prior to construction of proposed development within a floodway.

If a Developer wishes to officially modify a City Floodplain, sufficient information shall be provided to the City such that the floodplains in the MDP can be updated appropriately.

8.2 FLOODPLAIN DEVELOPMENT PERMIT

The development consultation meeting will facilitate a discussion between the City and Developer where the Developer will learn the applicability of a floodplain development permit (FDP) and associated submittal requirements. If any development activities are proposed that would alter the topography of a property located in a FEMA Floodplain, an FDP from the City shall be required prior to commencement of construction. When hydrologic and hydraulic studies are required, a Professional Engineer, registered in the State of Texas, shall perform the analyses in order to obtain the FDP. Two analyses shall be completed for each proposed project: one assuming existing conditions hydrologic conditions, and the second assuming FFD hydrologic conditions.

The Developer shall be required to submit the results of the existing conditions analyses and supporting hydrologic and hydraulic data to the City Engineer. The appropriate FEMA application forms, if required, shall also be submitted to the City concurrent with the Floodplain Development Permit submittal. If the City Engineer accepts the analyses, the existing conditions data, model and application forms will need to be submitted to FEMA by the Developer to update the FIS and FIRMs, if the City determines that a LOMR is necessary.

The Developer shall also be required to submit the results of the FFD conditions analyses and supporting data to the City Engineer. If the City Engineer approves the analyses, the City Floodplains shall be updated by the City upon completion of construction to reflect the revised data. Sufficient data shall be provided by the Developer such that the City Engineer is able to appropriately update the City Floodplains.

An FDP shall be granted by the City Engineer upon review and approval of both the existing conditions and FFD conditions hydrologic and hydraulic analyses. The proposed development shall also meet the requirements provided in Section 1.3.

8.3 FLOODPLAIN DEVELOPMENT REQUIREMENTS

Floodplain alteration shall be allowed only if all the following criteria are met:

A. Alteration of a floodplain shall meet the requirements of Chapter 6 Downstream Assessments.

B. Hydrologic and hydraulic analyses shall include flows generated for existing conditions and FFD conditions for the 2-, 10-, 25-, 50-, 100- and 500-year storm events.

C. Alterations of the conveyance paths and floodplain storage shall not increase the existing or FFD water surface elevations in the flow route being studied for the 2-, 10-, 25-, 50- and 100-year storm events.

D. Alterations shall be completed in compliance with FEMA guidelines.

E. Any alteration of floodplain areas shall not cause any additional expense in any current or projected public improvements, including maintenance.

F. A Floodplain Development Permit shall be required to perform any grading activities on site.

G. Development within a designated floodway is not permitted without FEMA CLOMR approval and proof of no downstream impact.

H. A LOMR shall be required when proposed development occurs within a FEMA Zone A Floodplain, or when proposed development in a Zone AE Floodplain results in a change in Base Flood Elevation. Upon completion of construction, the LOMR shall be submitted to the City Engineer for approval prior to submission to FEMA. Once the City Engineer has signed the required acknowledgement form, it is the responsibility of the Developer to submit the LOMR to FEMA. The Developer shall provide the City Engineer with proof of submission of the LOMR to FEMA. Final acceptance of the development is contingent upon proof of LOMR submission to FEMA. The LOMR process is completed once construction of the proposed development is complete and the LOMR becomes effective with FEMA.

I. A LOMR-F is acceptable if approved by the City Engineer for a single lot development, or as an interim condition for a multi-phase development. In all other cases, final acceptance of the development is contingent upon proof of LOMR submission to FEMA.

J. A CLOMR may be required at the discretion of the City Engineer. This will typically be required in situations where there is a risk of significant change to the floodplain, and it is unclear how much the proposed development will affect water surface elevations. The CLOMR shall be submitted to the City Engineer for approval prior to submission to FEMA. Once the City Engineer has signed the required acknowledgement form, it is the responsibility of the Developer to submit the CLOMR to FEMA. The Developer shall provide the City Engineer with proof of submission of the CLOMR to FEMA. When required, the CLOMR process shall be completed prior to commencement of construction of the proposed development.

K. New residential basements or commercial basements that are not flood-proofed shall not be allowed within Zone A or Zone AE floodplains.

8.4 FLOODWAY CALCULATIONS

Floodway calculations involve modification of the hydraulic model for a channel to determine the allowable encroachments on either side of the channel. Floodways are typically mapped within Zone AE floodplains, where sufficient data is available to develop the floodway boundary. Encroachments are gradually added to the overbank areas of each cross section in order to achieve an increase in water surface elevation of up to one foot. The floodway calculations establish a boundary within which land may not be altered without an approved CLOMR. Development within a designated floodway is not permitted without FEMA CLOMR approval and proof of no downstream impact.

9.0 STORMWATER QUALITY

9.1 GENERAL

The City of Lubbock is a Phase I Municipal Separate Storm Sewer System (MS4) required to comply with its National Pollutant Discharge Elimination System (NPDES) permit and is a co-permittee with Texas Tech University (TTU). The City of Lubbock Engineering Department is the authority responsible for overseeing compliance of the City's Stormwater Management Plan (SWMP).

The policy of the City of Lubbock is to include water quality considerations in planning for storm drainage facilities. Construction activities that discharge stormwater runoff into or adjacent to any surface water of the state are regulated by the state of Texas under the Construction General Permit (CGP) (TXR150000). The governing agency is TCEQ. Construction activities are regulated according to the area of land disturbed. The disturbance of land includes clearing, grading and excavation, including preconstruction activities.

Although the regulations cover a plethora of activities associated with stormwater management and stormwater pollution prevention, this manual will limit itself to those aspects associated with post-construction sediment and erosion control measures. The Developer shall consult with the City Engineer and the City's Code of Ordinances to determine the most current stormwater quality considerations and compliance with the most current MS4 permit. Enforcement actions for non-compliance with the most current MS4 permit shall be as outlined in the City's Code of Ordinances.

Post-Construction Best Management Practices (BMPs) are currently required by the City of Lubbock's MS4 Permit. The items in this section are acceptable BMPs within the City of Lubbock. All others will be considered by the City Engineer on a case-by-case basis. Per Section 22.11.032(f) of the City's Code of Ordinances, the Developer shall incorporate these BMPs to the maximum extent practicable.

9.2 POST-CONSTRUCTION BEST MANAGEMENT PRACTICES

9.2.1 Structural Controls

Post-construction structural controls (BMPs) are those facilities intended to treat stormwater runoff after development of an area is complete. The Developer is referred to the *iSWM™ Technical Manual: Site Development Controls* for an in-depth discussion of available structural BMPs, their relative pollutant removal efficiencies, and appropriate applications of each. Those BMPs which are most applicable within the City of Lubbock are discussed briefly in Table 9.1 below. All constructed structural controls within the City of Lubbock shall have an executed Facilities Maintenance Agreement.

Table 9-1: Sediment and Erosion Controls

Control	Description/Purpose
Detention	May include Dry Detention Basins or Multi-Purpose Detention Areas. Dry Detention Basins are surface facilities intended to provide for the temporary storage of stormwater runoff to reduce downstream water quantity impacts. Multi-Purpose Detention Areas are site areas used for one or more specific activities, such as parking lots and rooftops, which are also designed for the temporary storage of runoff.
Retention	Constructed stormwater retention basins that have an outlet elevation above the flowline of the basin. Basin initial conditions are defined in Chapter 6.
Filter Strips	Provide “biofiltering” of stormwater runoff as it flows across the grass surface. Similar to grass channels, filter strips are not capable of removing large quantities of TSS by themselves; therefore, they are typically only used as a pretreatment measure or as part of a treatment train approach.
Grass Channels	Provide “biofiltering” of stormwater runoff as it flows across the grass surface. Grass channels are not capable of removing large quantities of TSS by themselves; therefore, they are typically only used as a pretreatment measure or as part of a treatment train approach.
Enhanced Swales	Vegetated open channels that are explicitly designed and constructed to capture and treat stormwater runoff within dry or wet cells formed by check dams or other means.
Infiltration Trenches	Excavated trenches filled with stone aggregate used to capture and allow infiltration of stormwater runoff into the surrounding soils from the bottom and sides of the trench.
Rain Harvesting	A container or system designed to capture and store rainwater discharged from a roof. The rain harvesting system consists of a storage container, a downspout diversion, a sealed lid, and an overflow system. Typical rain harvesting systems hold between 50 and 500 gallons of water, and may work in series to provide larger volumes of storage.
Disconnected Downspout	A roof downspout that is located to drain through a filter strip, rain barrel, or other water quality device prior to discharging to receiving system.
Bioretention Areas	Shallow stormwater basins or landscaped areas which utilize engineered soils and vegetation to capture and treat stormwater runoff. Runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil.
Sand Filters	Multi-chamber structures designed to treat stormwater runoff through filtration, using a sand bed as its primary filter media. Filtered runoff may be returned to the conveyance system, or allowed to partially exfiltrate into the soil.

Source: *iSWM™ Technical Manual: Site Development Controls*, April 2010 (Revised September 2014)

9.2.2 Topsoil Salvage and Placement

Where excavation or general land disturbing activity is planned, the existing topsoil shall be salvaged and stockpiled for replacement. If fill is planned, then 6 inches of topsoil under the fill location shall be salvaged prior to fill placement. The final lift on cut and fill areas shall be the placement of salvaged topsoil. The minimum desirable topsoil finished lift thickness is six inches; however, in some cases, the volume of salvaged topsoil is insufficient to cover the entire cut/fill area to this thickness.

Vegetation shall be established as quickly as practicable following completion of topsoil placement to achieve final stabilization of the site. Acceptable vegetation establishment requires coverage of at least 70% of the disturbed area, per the City's Code of Ordinances.

9.3 INSPECTION AND MAINTENANCE

There will be a period of time between substantial completion of a project and final acceptance of the development by the City. During this time, the Developer shall be responsible for inspection and maintenance of structural BMPs; however, the City shall have the right to inspect and maintain all structural BMPs, if deemed necessary. Inspection and maintenance shall be completed by the Developer in accordance with the recommendations outlined in the *iSWM™ Technical Manual: Site Development Controls.* Compliance shall follow procedures described in the City's Code of Ordinances.

Appendix

Table 4.7: Entrance and Junction Loss Coefficients

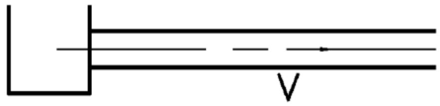
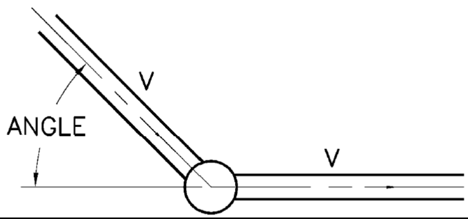
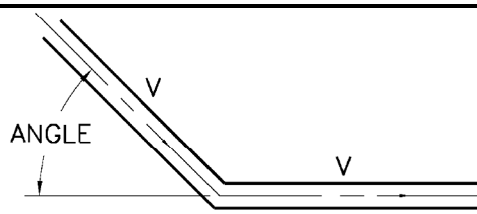
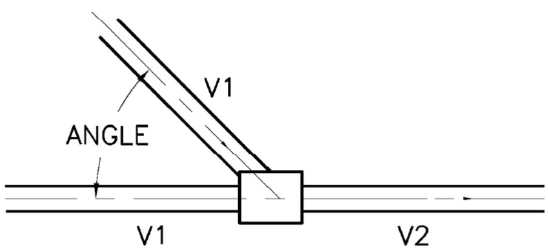
Inlet		
Schematic		**K_j**
		1.25
Manhole at Change in Pipe Direction		
Schematic	**Angle**	**K_j**
	90°	0.55
	60°	0.48
	45°	0.42
	30°	0.30
	0°	0.05
Bend in Pipe		
Schematic	**Angle**	**K_j**
	45°	0.35
	30°	0.20
Manhole		
Schematic	**Angle**	**K_j**
	0°	1.00
	22 ½°	0.75
	45°	0.50
	60°	0.35
	90°	0.25

Table 4.8: Minimum Design Grades for Concrete Box Section
Manning's "n"=0.015

Type	Span (Ft)	Height (Ft)	Min. Grade (Ft/Ft)
Without Interior Haunches	3	2	0.0013
	3	3	0.0010
	4	2	0.0011
	4	3	0.0010
	4	4	0.0010
With Interior Haunches	3	2	0.0014
	3	3	0.0010
	4	2	0.0012
	4	3	0.0010
	4	4	0.0010
All Other Span and Height Combinations			0.0010

Table 4.9: Minimum Design Grades for Circular Pipe

Pipe Diameter (Inches)	Minimum Design Grade in Ft/Ft for Manning's "n"								
	0.010	0.015	0.020	0.021	0.023	0.024	0.026	0.027	0.031
18	0.0010	0.0023	0.0042	0.0046	0.0055	0.0060	0.0070	0.0076	0.0100
21	0.0010	0.0019	0.0034	0.0037	0.0045	0.0049	0.0057	0.0062	0.0081
24	0.0010	0.0016	0.0028	0.0031	0.0038	0.0041	0.0048	0.0052	0.0068
27	0.0010	0.0014	0.0024	0.0027	0.0032	0.0035	0.0041	0.0044	0.0058
30	0.0010	0.0012	0.0021	0.0023	0.0028	0.0030	0.0036	0.0038	0.0051
36	0.0010	0.0010	0.0017	0.0018	0.0022	0.0024	0.0028	0.0030	0.0040
42	0.0010	0.0010	0.0013	0.0015	0.0018	0.0019	0.0023	0.0025	0.0032
48	0.0010	0.0010	0.0011	0.0012	0.0015	0.0016	0.0019	0.0021	0.0027
54	0.0010	0.0010	0.0010	0.0011	0.0013	0.0014	0.0016	0.0018	0.0023
60	0.0010	0.0010	0.0010	0.0010	0.0011	0.0012	0.0014	0.0015	0.0020
66	0.0010	0.0010	0.0010	0.0010	0.0010	0.0011	0.0012	0.0013	0.0018
72	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0011	0.0012	0.0016
78	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0011	0.0014
84	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0013
90	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0012
96	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0011
108	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010
120	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010
132	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010
144	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010	0.0010

Minimum v = 2.5 ft/s at d/D = 0.5 (d/D = 0.5 means conduit is one-half full), normal depth flow.
See Table 4.1 for materials corresponding to "n" values.
Consult manufacturer for available circular pipe sizes.
Engineer to provide minimum grade analysis for arch and elliptical pipe.
Minimum permissible design grade = 0.0010 ft/ft for construction purposes.

Table 4.10: City of Lubbock Curb Inlet Flow Calculation Table

Design Point for Inlet	Inlet Number	Location of Inlet by Station Number	Drainage Area Designation	Drainage Area Size (acres)	Runoff Coefficient (C)	Time of Concentration (Minutes)	100-Year Intensity (in/hr)	100-Year Runoff Q=CIA (cfs)	100-Year Carryover Flow (cfs)	100-Year Total Gutter Flow (cfs)	Percentage of Flow from Lower Station (cfs)	Percentage of Flow from Higher Station (cfs)	100-Year Total Gutter Flow Reaching Lower Side (cfs)	100-Year Total Gutter Flow Reaching Higher Side (cfs)	Longitudinal Slope of Approach Gutter (ft/ft)	Half the Longitudinal Slope of Approach Gutter on Higher Station (ft/ft_	Street Crown Section Type	Roadway Cross Slope (%)	Manning's Roughness Coefficient (n) for Pavement (0.0175 for concrete)

Table 4.11: Storm Drain Calculations

Down-stream Station Number	Upstream Station Number	Distance between Stations (ft)	Designation of Drainage Area(s)	Drainage Area (Acres)	Total Drainage Area (Acres)	Runoff Coefficient "C"	Product of Columns 5 by 7	Total "CA" for the Drainage System	Inlet Time of Concen-tration	Flow Time in the Storm Drain (Min)	Total Time of Concen-tration (Min)	Intensity of Rainfall (in/hr)	100-Year Storm Runoff (cfs)	Proposed Inlet Carryover from Upstream 100-yr Storm	Proposed Inlet carryover in 100-yr Storm	Design Discharge for Storm Drain System (Qpipe) cfs	Enter Selected Pipe Size for Circular Pipe	Enter Selected Width for Box Pipe	Enter Selected Height for Box Pipe	Enter Manning's Roughness Coefficient	Enter Slope of Frictional Gradient	Upstream HGL Before Structure Calc	Beginning Hydraulic Gradient of Line	Velocity Flow Incoming Pipe Junction	Velocity Flow Outgoing Pipe	Velocity Head Col. 25	Velocity Head Col. 26	Head Loss Kj	Col. 27 X Col. 29	Head Loss Structure Min. 0.10'	Design HGL	Invert Ele for Pipe Analyzed Col. 1	Invert Ele for Pipe Analyzed Col. 2	Top of Curb Ele Col. 2	Remarks