CITY OF HORSESHOE BAY
DRAINAGE CRITERIA MANUAL

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1 INTRODUCTION

1.1 Background and Authorization

Horseshoe Bay City, a home rule city, has legal responsibility for the regulation, funding, and maintenance of its public drainage facilities. Previous versions of the City's subdivision ordinance required adherence with the drainage criteria manual of the City of Austin. In recognition of the unique drainage conditions associated with this community, the City of Horseshoe Bay is hereby adopting this document as drainage criteria and design requirements applicable within the City Limits of Horseshoe Bay and within the City's Extraterritorial Jurisdiction (ETJ).

Drainage and management of storm water and runoff from existing and new developed areas requires the application of a sound and consistent set of policies and methodologies for the design and acceptance of proposed storm drain systems for proposed roadways, subdivisions, and commercially developed areas. To that end, the City of Horseshoe Bay, Texas adopts these policies and design criteria, in its capacity as the governing body of the City of Horseshoe Bay, Texas and the area within its Extraterritorial Jurisdiction (ETJ).

Based upon the recommendation of the City Engineer, Development Services Department, Stormwater Master Plan team and Stakeholder Committee, the City of Horseshoe Bay has adopted this Drainage Criteria Manual. The Subdivision Ordinance 2021-39: Amending Chapter 10 Subdivision Ordinance of the City of Horseshoe Bay 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 Horseshoe Bay.

1.2 Purpose and Need

The major watersheds draining through the City of Horseshoe Bay to Lake LBJ extend upstream of the City's ETJ. As a result, the City cannot enforce total regulatory and funding control over the upstream reaches of watersheds, which are outside their legal jurisdiction. However, the City believes it is important to maintain uniform and consistent watershed management practices throughout the portions of the watersheds, where legal authority exists. These practices allow for entire watersheds to be analyzed. They further allow for regulation to ensure that drainage systems within the City's jurisdiction consider the health, safety and welfare of citizens and visitors of the City of Horseshoe Bay. Finally, this manual will help further coordinate and disseminate consistent drainage requirements for all future development in the City.

The purpose of this drainage manual is to establish standards and practices for the design and construction of drainage systems within the City of Horseshoe Bay. The design factors, formulas, graphs, and procedures are intended for use as minimum engineering requirements in the solution of drainage problems involving determination of the quantity, rate of flow, extents of inundation, method of collection, storage, and conveyance of storm water. The guidelines described herein will also allow for, but not require, public construction and maintenance access for the continued operation of the City's drainage system.

Methods of evaluation, analysis, or design other than those indicated herein may be considered in cases where experience clearly indicates that they are preferable. Variations from the practices established herein may be considered by City staff, following the requirements presented in section 1.4.2 of this Manual.

1.3 Design Responsibility

The requirements and methods included in this Manual are intended to outline the minimum design and analysis effort required for drainage infrastructure within the City of Horseshoe Bay. The designer may choose to apply more detailed methods to obtain a more efficient design, or it may be necessary to apply other more involved methods to produce an appropriate design for specific projects. Additionally, the designer is responsible for expanded studies, such as geotechnical and environmental investigations and any other studies that may be needed as a basis for sound design. In any case, the recommendations and requirements in this Manual do not relieve the designer of full professional responsibility for the information and resulting constructed improvements. The full responsibility for all designs, plans, and specifications will rest with the design professional who produced them. The City's review of calculations, documents, or construction shall not in any way construe or convey liability for adequacy of the design or construction to City staff.

The Developer shall solicit the services of a professional engineer licensed in the State of Texas to provide sufficient analysis and design, and to provide dates, seals, and signatures for all items required to be sealed by a professional engineer.

1.4 Objectives

1.4.1 Submittal Requirements and Acceptance Process

All developments greater than one single family residential structure or one duplex residential structure shall be required to submit a Preliminary and Final Drainage Plan to the City's Development Services Department. The submittal requirements and approval process for the drainage plan is outlined in the following sections.

The first step in the review and approval process for a proposed development shall be to submit a Preliminary Drainage Plan to the City's Development Services Department. The plan shall demonstrate that adverse drainage or flooding conditions will not be created as a result of the development. The Preliminary Drainage Plan shall define the method of conveying rainfall runoff from the development to the appropriate drainage outfall. This will include, as applicable, sheet flow paths, storm drain design, outlet design, detention design, and addressing 100-year floodplain issues.

The Preliminary Drainage Plan shall show the following as a minimum:

1. Name, address, and phone number of the owner and the licensing and contact information for the engineer preparing the plan.

2. Submittal and re-submittal dates.

3. A scaled drawing of any existing drainage facilities and any proposed improvements on an Architectural D Size sheet, at a minimum scale of 1" = 200'.

4. Vicinity map and legend.

5. A primary benchmark referenced to a N.A.V.D. benchmark with elevation, datum, year of adjustment, and description.

6. North arrow on all sheets oriented upward or to the right, where feasible.

7. Location and dimensions of all lot lines, property lines, rights-of-way, existing and proposed drainage easements, and all other easement lines, with type designated.

8. Contour lines at one-foot intervals covering the entire development including offsite elevations 100 feet around perimeter.

9. Erosion and Sedimentation Control Plan. Erosion control measures shall be established prior to beginning construction on a site. All erosion control devices shall be inspected a minimum of every week and after each rainfall event. Any identified maintenance needs shall be completed promptly at no cost to the City of Horsehoe Bay.

10. Cross-section of existing and/or proposed detention facility, swales, and ditches.

11. Drainage area boundaries for the project area, including off-site areas.

12. Location of all drainage conveyance adjacent to or crossing the development as determined by recent (less than 12 months old) ground survey.

13. Stream alignment shall be shown continuously to 200 feet upstream and to the point of adequate outfall downstream of development.

14. Detention tabulations, including the detention volume required and the detention volume provided. (Detention calculations shall be completed as outlined in Section 5.)

15. Limits of the floodway and the 100-year floodplain, scaled from the current FIRM, if applicable.

16. Location of all planned drainage improvements.

17. Location of existing pipelines and/or any other underground features and structures.

18. Hydrology and hydraulic calculations to demonstrate no adverse impacts.

The second step in the review and approval process for a proposed development is to submit a Final Drainage Plan to the City's Development Services Department, demonstrating that the analysis and design are consistent with these criteria and that adverse drainage or flooding conditions will not be created as a result of the development. The Final Drainage Plan shall be filed and accepted by the City's Development Services Department prior to commencement of construction.

The Final Drainage Plan shall include all the items in the Preliminary Drainage Plan as well as the following as a minimum:

1. The specific deliverables required under each section of this manual
2. Seal of the responsible registered professional engineer on all plans and technical reports

1.4.2 Variances

All variances from the requirements included in this manual shall be approved by the Director of Development Services. A grant of an alternative material, design, or method of construction shall not affect nor relieve the engineer of the obligation and responsibility of such material, design, or method of construction for the intended purposes. In the event that specific circumstances dictate requirements not already included in this Manual, it shall be the responsibility of the engineer to provide the additional information as deemed necessary by the Director of Development Services in writing for review. Written acceptance of variance shall be required prior to submittal of calculations or commencement of construction utilizing alternative methodologies or materials.

1.5 Definitions and Acronyms

Abutment
A structure that supports the lateral load of an arch or span.

Annual Exceedance Probability (AEP)
The probability of exceedance in a given year.

Attenuation
The reduction of the peak of a hydrograph, causing the shape to become flat and wide.

Backwater
Water that is backed up or slowed compared to the average, natural flow. This phenomenon can be caused by temporary obstructions or an opposing current.

Bankfull
The elevation at which the water level stage just begins to overfill the confines of the hydraulic structure (channel, detention basin, roadside ditch, etc.), and flows into the floodplain.

Berm
A flat, raised land surface bordering a body of water. Also termed embankment.

Calibration
The determination and subsequent alterations that account for the differences between true values and values being supplied by models or other instrumentation.

Channel
An open conveyance system capable of moving drainage water through a watershed. May also be described as a river, creek, or ditch.

Confluence
The intersection and convergence of two channels.

Conduit
An enclosed pipe or box, usually concrete, used to convey stormwater underground from one location to another

Contour
A graphic line on a map depicting a specific elevation.

Depression Storage
Water contained in natural low points in the land surface.

Design Storms
A defined hyetograph and total precipitation that represent the estimated runoff for a given hypothetical storm specified by the Drainage Regulatory Entity.

Detention
A volume of available space used to temporarily contain excess stormwater runoff to reduce peak downstream discharge for a specified length of time.

Development
The improvement or subdivision of a tract of land exclusive of land being used for agricultural purposes. Improvement of land includes grading, paving, building of structures, or otherwise changing the runoff characteristics of the land.

Development Services Department
The person or department responsible for review of proposed construction documents and regulation of drainage criteria within the City of Horseshoe Bay

Discharge
The amount of flow produced from a specified rainfall event. Also described as flow or runoff.

Drainage Area
The geographic limits of land contributing to flow at a given location. Also termed as watershed.

Duration
The total length of time for a single rainfall event.

Easement
A legally bind tract of privately owned property reserved for a specified use.

Erosion
A change in geometric configuration caused by loss of existing soil.

Evapotranspiration
The loss of water due to evaporation from soil and water surfaces and the transpiration from plants.

FIRM
Flood Insurance Rate Map. A map generated by FEMA depicting the limits of special flood hazard areas. The limits of flood hazard shown on a FIRM are based on a Flood Insurance Study (FIS).

Floodplain

The area outside the banks of a channel where floodwaters flow when they exceed the capacity of the channel. Normally, the floodplain is immediately adjacent to a channel, but may extend laterally for a significant distance.

Floodway

The geographical limits of encroachment at which a 100-year floodplain may be encroached without increasing water surface elevations more than 1-foot throughout the length of the stream.

Freeboard

The vertical distance between the top edge of a hydraulic structure and the water surface it is containing.

Frequency

The statistical probability of storm recurrence expressed as a percentage. Also known as recurrence interval.

Gabion

A wire cage binding together rocks, concrete, cement, soil, or other material used to stabilize water conveyance channels and shorelines.

Hydraulic

Relating to the physical behavior or properties of runoff from a given rain event.

Hydraulic Grade Line

A line representative of the flow energy, it is a graphical representation of the water surface elevation at any point of an open channel.

Hydrologic

Relating to the quantity of runoff produced from a given rain event.

Hydrograph

Graphical representation of rate of flow over a period of time.

Hyetograph

Graphical representation of rainfall intensity over a period of time.

Impervious Cover

Surfaces that do not absorb rainfall. Also termed impervious area or impervious surface.

Infiltration
The permeation of water into the soil under the ground's surface.

Interception
Precipitation captured by buildings, leaves, etc., before it reaches the land surface.

Inundation
The condition of being flooded.

Intensity
The amount of rainfall experienced over a given time period. Usually expressed in inches per hour.

Manning's N-Value
A dimensionless coefficient used to describe the relative roughness of a surface in contact with runoff. The value is used in hydraulic calculations

NAVD
North American Vertical Datum. A relative indicator of elevation, with a given year of determination. Used for developing benchmarks from which to describe vertical elevations

Outfall
The point of discharge from a channel or conduit into another drainage conveyance location.

Outlet Structure
Structure usually composed of pipes, weirs, spillways, and/or pumps designed to drain a detention or retention facility.

Overbank
A geological deposit of silt or other sediment on a floodplain caused by the overtopping of water.

Overland Flow
The flow of runoff over the land surface, not through a channel or other designated conveyance system.

Parameters
A numerical representation of characteristics of modeled events and locations.

Peak Flow
Rate of flow at the highest point of a hydrograph. Also termed as peak discharge.

Peak Runoff

The maximum runoff capacity for design of a hydraulic structure meant to carry or detain the runoff.

Ponding
The volume of rainfall runoff that is unable to move downstream by gravity.

Rainfall Loss Rate
The portion of the total amount of rainfall that is included in a hydrologic runoff calculation over a given period of time.

Recurrence Interval
The reciprocal of AEP. A return period that marks the average interval of time between the given flood occurrences.

Riprap
Rock, loose stone, or other material used to prevent erosion of shorelines, stream beds, and other channels.

Roughness Coefficient
A dimensionless value used in hydraulic calculation to approximate the impact of different types of physical characteristics within a channel or floodplain

Routing
The alteration of the shape and timing of a runoff hydrograph as it moves downstream through a drainage system.

Runoff
Excess rainfall which runs off the land and which is defined as the rainfall minus the losses.

Runoff Coefficient
A constant used to describe the expected amount of runoff produced from a given rainfall amount.

Scour
Erosion near the base of structures caused by the fast movement of water.

Side Slopes
The angle of the side of a channel. Expressed in the change in horizontal dimension over the change in vertical dimension.

Slope Paving
Smooth concrete placed inside a drainage channel to prevent erosion.

Swale
Natural or manmade shallow channel with gradual side slopes.

Time of Concentration
The travel time of a single particle of water from the farthest point of the watershed to the point of interest.

Unit Hydrograph
The base level for defining a hydrograph from a given watershed. It is defined by the surface runoff due to one inch of rainfall excess (rainfall minus loss) applied uniformly over the watershed in a specified time interval.

Unsteady Flow
Change in a flow hydrograph over time through the creek or channel or other drainage conduit

Variance
A written request to the Development Services department to modify the design standards provided in this document. A variance request must be approved in writing by the Development Services department prior to use of modified requirements.

Watershed
A defined area where all overland flow runoff is conveyed to the same outlet. Similar terms include drainage area, basin, or drainage basin.

Weep Holes
Small openings in the structural siding to allow for water to drain from within the structure.

2 HYDROLOGY

2.1 General

In Horseshoe Bay, two methods of calculation are acceptable, based on the size of the contributing drainage area to the point of analysis. Small, local drainage systems may be analyzed using Rational Method calculation of peak discharges for a specific time of concentration. For areas greater than 100 acres, a unit hydrograph calculation, using SCS Methodology, shall be employed to determine both peak runoff and a time-varying runoff hydrograph for a given drainage area. Because of its versatility and accuracy, the widely used computer program HEC-HMS is recommended as the primary tool for modeling storm runoff hydrographs in Horseshoe Bay. HEC-HMS can be downloaded at no cost from the USACE website. If the engineer wishes to use an alternative design technique or software program, the City's Development Services Director shall provide written acceptance of the alternative methodology prior to submittal of calculations to the City.

For purposes of planning and design of new facilities, fully developed watershed conditions shall be used in calculation of discharges, unless otherwise specifically noted in this document. All proposed drainage facilities shall be analyzed for the 2-year, 5-year, and 100-year return event storms.

2.2 Peak Flow Determination

The Rational Method calculation can be used to determine peak discharges for drainage areas of 100 acres or less. All larger drainage areas require calculation of discharge using hydrograph methods. For detention calculations, the 25-year storm shall also be required.

2.2.1 Rational Method (100 acres or less)

The Rational Method represents an accepted method for determining peak storm runoff rates for small drainage areas that have a drainage system unaffected by complex hydrologic situations such as the confluence of multiple drainage areas, storage basins, and watershed transfers (overflows) of storm runoff. This widely used method provides satisfactory results for storm drain and basic roadside ditch design.

The Rational Method, Equation 2-1, is based on a direct relationship between rainfall and runoff, and is expressed by the following equation:

$$Q = CiA \tag{2-1}$$

Where,

Q	=	Peak rate of runoff in cubic feet per second (cfs);
C	=	Dimensionless coefficient of runoff representing the ratio of peak discharge per acre to rainfall intensity;
i	=	Average intensity of rainfall in inches per hour for a period of time equal to the longest time of concentration from the upstream end of the drainage area to the point of interest (in./hr.);
A	=	Area contributing runoff to the point of interest during the critical time of concentration (ac.).

Basic assumptions associated with the Rational Method are:

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

2. The frequency or recurrence interval of the peak discharge is equal to the frequency of the average, uniform rainfall intensity associated with the critical time of concentration (duration).

3. The storm duration is equal to the critical time of concentration

4. The ratio of runoff to rainfall, C, is constant for the entire storm duration

5. Rainfall intensity is constant for the entire storm duration.

6. The contributing area is the area that drains to the point of interest within the critical time of concentration.

2.2.1.1 Runoff Coefficient (C)

In relating peak rainfall rates to peak discharges, the runoff coefficient "C" in the Rational Formula, Equation 2-1, is dependent on the characteristics of the drainage area's surface.

Coefficients for specific surface types shall be used to develop a composite runoff coefficient based in part on the percentage of different types of surfaces in the drainage area. Table 2-1 presents values for the runoff coefficient "C" for land use types in Horseshoe Bay.

Table 2-1 Typical Average Values for Impervious Cover and Runoff Coefficient

Land-use Type	Impervious (%)	Runoff Coefficient
Commercial/Multifamily	85	0.85
Airport	95	90
Airport Mixed Use	85	0.85
Detention (Wet or Dry)	95	0.90
Major Thoroughfares	90	0.90
Open Space-Row	0	0.60
Open Space-Pastureland	0	0.40
Undeveloped-Wooded/forested	0	0.20
Open Space-Parks/Golf Course/Green Space	5	0.35
Residential* 1/8 Acre	65	0.65
Residential* 1/4 Acre	50	0.60
Residential* 1/3 Acre	40	0.50
Residential* 1/2 Acre	35	0.45
Residential* (1 Acre)	30	0.40
Residential* (2 Acre)	25	0.35
Residential* (≥ 5Acre)	10	0.20
Airport Mixed Use (Residential w/ Hangar)	60	0.6

*Note: Residential Values include impervious cover for internal roadways

2.2.1.2 Rainfall Intensity (i)

Rainfall intensity (i) is the average rainfall rate in inches per hour for a basin or sub-basin and is selected based on design rainfall duration and design frequency of occurrence. In a Rational Method calculation (eq. 2-1), the design duration is equal to the time of concentration (T_C) for all portions of the drainage area that contribute flow to the point of interest. Time of concentration shall be calculated using the United States Department of Agriculture (USDA) Natural Resource Conservation Service (NRCS) method as described in Technical Release 55 (TR-55), and the Equation (2-2):

$$T_c = T_{\text{sheet flow}} + T_{\text{shallow conc.flow}} + T_{c\text{hannel flow}} \quad (2\text{-}2)$$

The time of concentration flow path and supporting calculations shall be submitted to support the selected values. The frequency of occurrence is a statistical variable which is established by design standards.

The time of concentration to any point in a storm drainage system is a combination of the “inlet time” and the “time of flow in the conduit”. The inlet time is the time for water to flow over land to the storm sewer inlet. Inlet time decreases as the slope and the pervious cover of the surface increases. Inlet time increases as the watercourse distance increases and as retention by the contact surfaces increases. A maximum inlet time of concentration is 15 minutes. In the absence of a closed pipe and inlet system, Tc for initial sheet flow can be calculated with the following equation:

$$\mathrm{T}_{sf} = \frac{0.007\ (\mathrm{nL})^{0.8}\ (60)}{(P_2)^{0.5}\ (\mathrm{So})^{0.4}} \quad (2\text{-}3)$$

Where,

T_{sf}	=	Sheet flow time (min);
n	=	Manning roughness coefficient;
L	=	Flow length (ft) (50-ft Maximum);
P2	=	2-Yr, 24 Hr Rainfall (inches)
So	=	Slope of land surface (ft/ft)

Average velocities for estimating travel time for overland (shallow concentrated) flow can be calculated using Figure 2-1. The inlet time shall be determined by direct computation using the following formula, with a maximum inlet time of 15 minutes:

$$T_{\mathrm{SC}} = \frac{D_F}{60V} \quad (2\text{-}4)$$

Where,

T_{SC}	=	Shallow Concentrated flow time (min);
D_F	=	Flow distance (ft);
V	=	Average velocity of design discharge (ft/sec).

The time of flow in a closed conduit or open channel is the quotient of the length of the conduit or open channel and the velocity of flow as computed using the hydraulic characteristics of the conduit at design discharge.

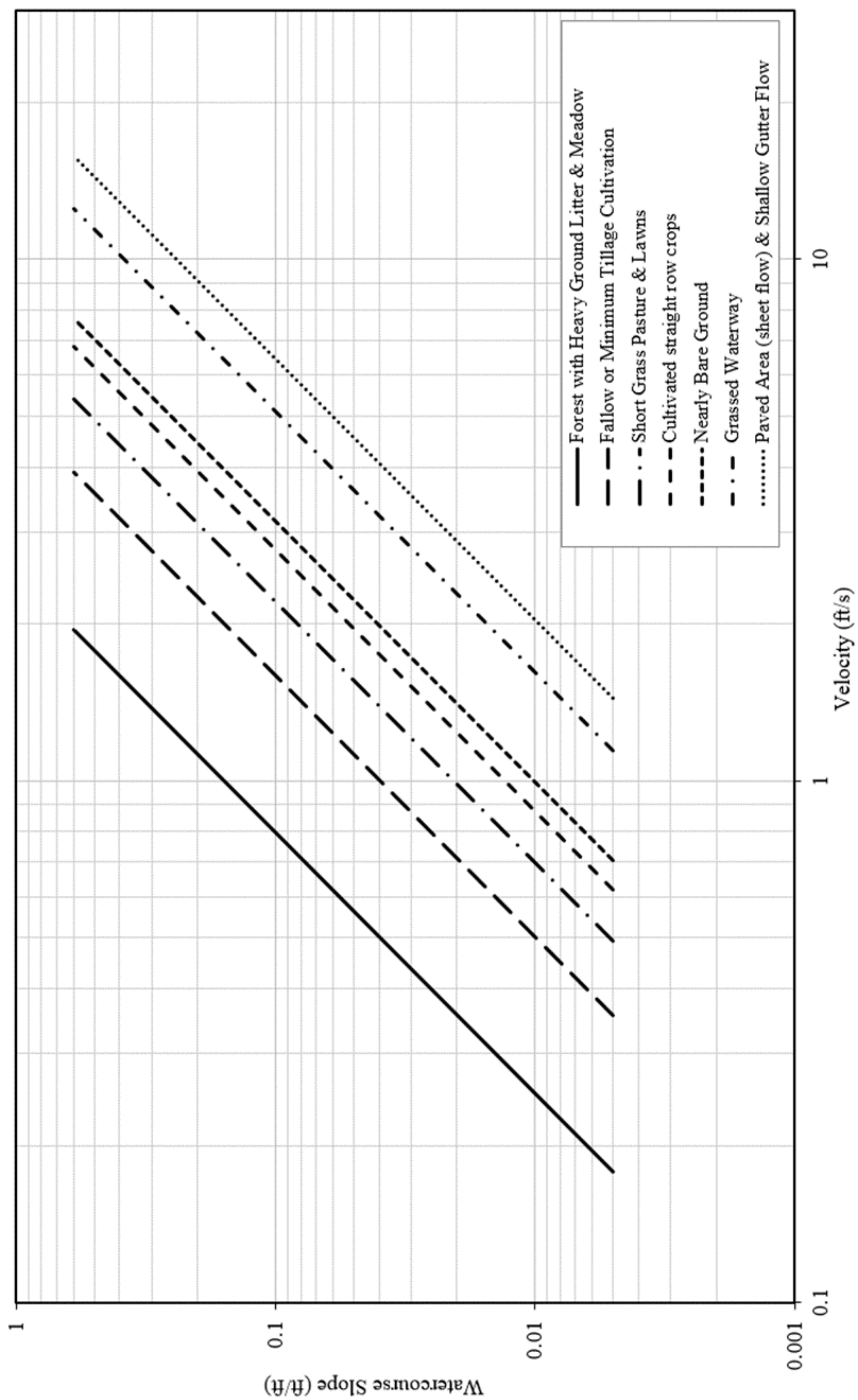

Figure 2-1 Average Velocities for Estimating Travel Time for Overland Flow

Total rainfall depths for various storm events and durations, based on Atlas 14 rainfall statistics, are provided in Table 2-2. To obtain intensity values from these tables, the appropriate depth value for a given time of concentration must be converted into inches per hour: divide the selected value by the corresponding time of concentration in minutes and multiply by 60 minutes per hour. For time of concentration values between the times provided in the table, a linear interpolation of rainfall values shall be used.

Table 2-2 Partial Duration Precipitation Frequency Estimates by Annual Exceedance Probability

	100%	50%	20%	10%	4%	2%	1%	0.2%	0.1%
	1-Year	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year	1000-Year
5-min:	0.409	0.494	0.628	0.744	0.91	1.04	1.19	1.35	1.57
10-min:	0.652	0.789	1	1.19	1.46	1.67	1.9	2.15	2.49
15-min:	0.819	0.989	1.25	1.48	1.81	2.08	2.36	2.67	3.11
30-min:	1.15	1.39	1.75	2.07	2.52	2.88	3.26	3.7	4.32
60-min:	1.49	1.8	2.29	2.71	3.32	3.8	4.33	4.93	5.83
2-hr:	1.79	2.21	2.84	3.41	4.24	4.92	5.69	6.59	7.96
3-hr:	1.96	2.45	3.17	3.84	4.83	5.65	6.6	7.72	9.45
6-hr:	2.26	2.87	3.75	4.59	5.85	6.92	8.16	9.66	12
12-hr:	2.58	3.31	4.34	5.34	6.86	8.16	9.69	11.5	14.4
24-hr:	2.93	3.78	4.99	6.15	7.93	9.46	11.3	13.4	16.7
2-day:	3.36	4.35	5.76	7.1	9.15	10.9	12.9	15.3	18.9
3-day:	3.67	4.73	6.26	7.71	9.89	11.8	13.9	16.4	20.1
4-day:	3.91	5.01	6.62	8.12	10.4	12.3	14.5	17	20.7

Source: NOAA Atlas 14 Volume 11 Version 2, September 2018, Partial Duration Series Rainfall Depths (Latitude: 30.5447°, Longitude: -98.3671°)

2.2.2 Drainage Area (A)

The size and shape of the drainage area must be determined. The area may be determined using topographic maps and supplemented by field surveys. A maximum 2-foot contour interval is required on all drainage area maps. A drainage area map shall be provided for each project. The entire drainage area contributing to the drainage system and the drainage subarea contributing to each point of interest shall be identified. Drainage area maps shall contain topographic information far enough outside the study area to confirm that project drainage divides are shown correctly.

2.2.3 Hydrograph Methodology

When a contributing drainage area to the point of analysis is greater than 100 acres, the USDA Soil Conservation Service (SCS) Unit Hydrograph Method shall be used for determining rates and volumes of stormwater runoff. This methodology shall be used with HEC-HMS software unless otherwise approved in writing by the City's Development Services Director.

2.2.3.1 Precipitation

Drainage systems sized with hydrograph methodology shall be designed for a 24-hour storm. Rainfall values shall be based on NOAA Atlas 14 Point Precipitation Frequency Estimates. Table 2-2 shall be used to populate precipitation information in the hydrologic software.

2.2.3.2 Rainfall Distribution

The hydrograph shape shall be developed using the SCS Type II 24-hour storm distribution, with an intensity position of 50% of the rainfall time.

2.2.3.3 Lag Time

Lag time is required for unit hydrograph methodology. Lag time is defined as the different in time between the peak of the rainfall event and the peak discharge of the resulting hydrograph. Lag Time shall be defined by the equation:

$$T_{\mathrm{L}} = 0.6\, T_C \qquad (2\text{-}5)$$

Where,

T_L	=	Lag Time (hr);
T_c	=	Time of concentration (hr).

Time of Concentration methodology described in section 2.2.1.2.

2.2.3.4 Runoff Curve Number

The runoff potential for a drainage area shall be expressed in the form of a Curve Number value (CN), related to the impervious cover of the area and the hydrologic properties of the underlying soil. Soils are classified in the NRCS National Engineering Handbook in Groups A through D. This information may be obtained digitally through the NRCS Soil Survey Geographic Database SSURGO, https://websoilsurvey.nrcs.usda.gov/ and should be used to obtain soils information for the subject drainage areas. Composite curve numbers should be used to represent runoff potential, based on the information in Table 2-3. Additional impervious percentage should not be included in the hydrologic calculation.

2.2.3.5 Flood Routing

Flood routing is an iterative process that determines the flood wave travel time and attenuation using hydrologic or hydraulic routing methods. The preferred method of hydrologic routing for Horseshoe Bay is Modified Puls method, which requires an available steady flow hydraulic (HEC-RAS) model to determine volumetric storage in the channel reaches. Discharges values for the hydraulic model should range, at a minimum, from 10% to 150% of 100-yeardischarge in 10% increments (10%, 20%, 30%, etc.). The cross-sections must define the entire floodplain storage available at various water levels. However, on the effective flow area of the cross-section is used to establish the proper discharge-water level relationship. Where Modified Puls routing is not feasible, other methodology may be approved in writing by the City’s Development Services Director prior to submittal of calculations.

Table 2-3 Runoff Curve Numbers (CN)

Cover Description	CN for hydrologic soil groups			
	A	B	C	D
Cultivated Land:				
Without conservation treatment	72	81	88	91
With conservation treatment	62	71	78	81
Pasture or range land:				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow:				
Good condition	30	58	71	78
Wood or forest land:				
Thin stand, poor cover	45	66	77	83
Good cover	25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc.)				
Poor condition (grass cover <50%)	68	79	86	89
Fair condition (grass cover 50% to 75%	49	69	79	84
Good condition (grass cover >75%)	39	61	74	80
Impervious areas:				
Paved; curbs and storm drains (excluding right-of-way)	98	98	98	98
Paved; open ditches (including right-of-way)	83	89	92	93
Gravel (including right-of-way)	76	85	89	91
Dirt (including right-of-way)	72	82	87	89
Urban districts:				
Commercial and business	89	92	94	95
Industrial	81	88	91	93
Residential districts by avg lot size:				
1/8 acre or less (town house)	77	85	90	92
1/4acre	61	75	83	87
1/3 acre	57	72	81	86
1/2 acre	54	70	80	85
1 acre	51	68	79	84
2 acres	46	65	77	82
Developing urban areas and newly graded areas (previous areas only, no vegetation)	77	86	91	94

Source: USDA-NRCS, Technical Release-55, Urban Hydrology for Small Watersheds.

3 OPEN CHANNEL HYDRAULICS

3.1 General

This Chapter summarizes the practical considerations, technical principles, and minimum criteria required for design of open channels. When a design approach is to be used that is not covered in this manual, the City's Development Services Director shall provide written acceptance of the alternative methodology prior to submittal of calculations to the City.

3.2 Design Considerations

Where feasible, the design engineer should consider taking advantage of naturally occurring drainage paths when locating and designing open channels. In situations where the use of a natural drainage alignment is not feasible, a grass-lined (vegetated) channel is an acceptable alternative, subject to the design criteria provided in this document. When the use of existing natural features is not possible, sufficient documentation shall be provided to justify their infeasibility.

Where development occurs adjacent to or surrounding an existing drainage channel, easement or right-of-way shall be dedicated of sufficient size to accommodate a vegetated channel for fully developed upstream watershed conditions.

The following design considerations shall be evaluated for all open channels in Horseshoe Bay:

1. Maintain creek overbank storage;
2. Maintain natural conditions;
3. Follow the natural drainage course;
4. Avoid crossing drainage divides;
5. Avoid tight channel bends;
6. Minimize conflicts with existing buildings, homes, utilities, and contaminated sites; and
7. Minimize the number of property owners affected by proposed channel construction.

3.3 Minimum Design Criteria

The following minimum requirements shall be provided for designed open channels:

1. Open channels shall be designed to accommodate the fully developed conditions 100-year, 24-hour storm, with one foot of freeboard.
2. Finished floor elevations for buildings adjacent to a channel shall be a minimum of 2-feet above the 100-year fully developed conditions water surface elevation.
3. Channel slopes must be revegetated immediately after construction to minimize erosion. Vegetation must achieve 80% coverage prior to City acceptance. Earthen channels may not remain uncovered or unvegetated for more than 72 hours after construction.
4. The minimum invert slope for an open channel shall be 0.5 percent. Channel bottom slopes should have sufficient grade to prevent significant siltation, but grades should not be so large as to create velocity or erosion. Maximum acceptable velocities are outlined in section 3.4.4. Greater velocities require permanent erosion protection and written approval from the City's Development Services Director.
5. Constructed open channels shall have a minimum bottom width of 10-feet, with side slopes no steeper than 4:1 (H:V). A geotechnical analysis, sealed by a Texas registered professional engineer shall be required to support a request for steeper side slopes. When steeper side slopes are allowed by written permission of the City's Development Services Director, an 8-foot-wide access ramp shall be provided for future maintenance access. Access ramp shall have a grade no steeper than 25%.
6. Drainage conduit outfalls into an open channel shall be located no more than 6-inches above the flowline of the receiving stream. Outfall velocity may not exceed 6 feet per second. Outfalls shall be angled in the downstream direction to minimize erosion potential. Permanent erosion protection shall surround all drainage conduit outfalls. Erosion protection material shall be rock rip rap unless otherwise approved in writing by City's Development Services Director.
7. A drainage easement shall be dedicated encompassing the top width of the channel plus a continuous 15-foot-wide section adjacent to one side of the channel for maintenance purposes. Where design includes side slopes steeper than 4:1, an 8' wide access ramp shall be provided to accommodate maintenance equipment. Loss of storage volume due to access ramp shall be accounted for in computations.
8. Approval must be obtained for all future utility lines crossing an open channel. All utility manholes shall be located outside the drainage easement or right-of-way. All utility lines shall be at least 4-feet under the surface of the finished ground elevation throughout the entire right-of-way, unless approved in writing by the City's Development Services Director.

3.4 Analysis Requirements

Channel hydraulic analyses are required to size proposed channel improvements, and to demonstrate new construction results in no adverse impacts to flood risks. Adverse impacts are defined as an increase in water surface elevation greater than 0.10 ft, velocity increase as described in section 3.4.4, or a net loss of storage volume below the 100-year water surface elevation.

3.4.1 Drainage Study

A separate drainage study shall accompany the design of any development that includes an option channel, whether existing or proposed. The following information shall be supplied:

1. A vicinity map that identifies the full scope of improvements.

2. A detailed map of the area and proposed channels with all pertinent physiographic information including topographic information, rights-of-way, drainage easements, drainage facilities, and floodplain information.

3. A watershed map showing the existing and proposed drainage area boundary along with all subarea delineations and all areas of existing or proposed development.

4. Discharge calculations specifying methodology and key assumptions used including discharges at key locations.

5. Hydraulic calculations specifying methodology used. All assumptions and calculated values of the design parameters must be clearly stated.

6. A profile of the subject reach which includes the following:

    a. All pertinent water surface profiles. This will minimally include the design storm event. For large channels, this will also include the 100-year frequency floods for both existing and proposed channel conditions.

    b. All existing and proposed bridge, culvert, and pipeline crossings.

    c. The location of all tributary and drainage confluences.

    d. The location of all hydraulic structures (e. g. dams, weirs, drop structures, etc.)

    e. Existing and proposed flowlines with flow direction arrows.

    f. Natural ground elevations at the right-of-way.

7. A map delineating existing and proposed design flood extents.

8. A map delineating existing and proposed rights-of-way.

9. Typical existing and proposed cross-sections.

10. The relevant surveying benchmark, elevation, datum and year of adjustment.

11. A soils report which addresses ground water, erosion, and slope stability (newly constructed channels only).

12. All hydraulic modeling data (original digital format).

3.4.2 Analysis Software

In general, the latest version of HEC-RAS shall be used for any newly developed hydraulic models. Software programs other than HEC-RAS will be considered on a case-by-case basis. The City's Development Services Director shall provide written acceptance of the alternative software program prior to submittal of calculations to the City.

3.4.3 Return Event

Channels shall be analyzed for the 2-Year, 5-Year and 100-Year storm to evaluate water surface elevation and velocity impacts due to proposed improvements. Proposed projects shall not result in a net reduction in flood carrying capacity for the three return-events analyzed.

3.4.4 Channel Velocity

The maximum allowable velocities for earthen channels is 6 feet per second for sandy soils, 8 feet per second for clay soils, and 10 feet per second for rocky soils. For expected velocities higher than these values, additional erosion protection measures shall be required.

Proposed projects may not increase velocities more than 5% over existing velocities. Any variance requests to these velocity criteria must be supported by a geotechnical investigation prepared by a Texas licensed professional engineer.

3.4.5 Manning's N Values

The Manning's N roughness values presented in Table 3-1 shall be used for channel analysis in the City of Horseshoe Bay:

Table 3-1 Manning's n for Channels

Description	N-Value
Earthen Channel:	
No vegetation	0.030
Maintained, grass-lined	0.035
Irregular weed and grass	0.040
Gabion or constructed block	0.040
Light brush on banks	0.050
Channels not maintained, weeds and brush uncut:	
Dense weeds, high as flow depth	0.080
Clean bottom, brush on sides	0.050
Dense brush, high stage	0.100
Pasture, no brush:	
Short grass	0.030
High grass	0.035
Cultivated areas:	
No crop	0.030
Mature row crops	0.035
Mature field crops	0.040
Brush	0.050
Buildings	0.120
Fenced areas:	
Chain Link	0.080
Wrought Iron	0.080
Wood/Solid	0.120
Trees:	
Dense willows, summer, straight	0.150
Cleared land with tree stumps, no sprouts	0.040

3.4.6 Hydraulic Losses

Head losses at transitions and bends also need to be taken into consideration. These minor losses should be accounted for during channel design efforts. Incorporate head losses into hydraulic profile computations for channel bends when the:

- Radius of curvature is less than three times the channel top width, and

- Average channel velocity is greater than 4 feet per second for the 100-year storm event.

The equations 3-1 and 3-2 can be used to calculate head loss across transitions and across a bend, respectively.

Transition Head Loss Equation

$$h_T = C\frac{\left(V_2{}^2 - V_1{}^2\right)}{2g} \tag{3-1}$$

Where,

h_T	=	Head loss across the transition (ft)
C	=	Empirical expansion or contraction coefficient; see Table 3-2
V_2, V_1	=	Average channel velocity of the downstream and upstream sections, respectively (fps)
G	=	Acceleration of gravity (32.2 ft/sec^2)

Table 3-2 Contraction and Expansion Coefficients

Transition	Coefficient	
	Contraction	Expansion
Gradual or warped	0.1	0.3
Bridge sections; wedge; straight-lined	0.3	0.5
Abrupt or square-edged	0.6	0.8

Bend Head Loss Equation

$$\mathrm{h_B} = \mathrm{c_f}\left(\frac{\mathrm{V}^2}{2\mathrm{g}}\right) \tag{3-2}$$

Where,

h_B	=	Head loss in feet
c_f	=	Coefficient of resistance; see Table 3
V	=	Average channel velocity in feet per second
g	=	Acceleration due to gravity (32.2 feet/sec^2)

Table 3-3 Coefficient of Resistance

Radius of Curvature Divided by Channel Top Width	C_f
Between 1.5 and 3.0	0.2
Between 1.0 and 1.5	0.3

3.4.7 Unsteady and 2D Channel Analysis

Occasionally, site conditions may exist such that a full hydrograph should be evaluated within a channel. Another potential condition includes significant 2-dimensional sheet flow patterns, such as flooding around buildings or at the confluence of channels, requiring Unsteady 2D modeling to properly analyze flooding conditions. Where a condition exists, which may require unsteady or 2D analysis, the engineer shall consult with the City's Development Services Director and receive approval in writing prior to submitting the advanced detailed calculation methods.

3.5 Channel Crossings

3.5.1 Culverts

3.5.1.1 Design Frequency

To maximize the efficiency of a culvert crossing, it is acceptable to design culverts such that the upstream side of the culvert (headwater) is submerged. Culverts shall be sized to carry a 100-year fully developed conditions discharge with 1-foot of freeboard to the edge of pavement or back of curb.

Driveway culverts shall be designed to convey runoff from the 5-year fully developed conditions discharge.

Impacts from culvert placement shall not increase water surface elevations more than 0.10 feet over existing conditions. Proposed culverts shall not cause flooding to existing properties or structures. The 100-year water surface in an open channel and surrounding a culvert shall be contained with a drainage easement.

3.5.1.2 Design Considerations

1. Culvert minimum pipe size shall be 18-inches in diameter. Culvert minimum box dimensions shall be 2 feet by 2 feet.

2. Concrete culverts shall be designed using a minimum Manning's N value of 0.015. Higher values may be considered to account for sedimentation or blockage.

3. Corrugated metal pipe (CMP) and (High Density Polyethylene) HDPE pipe shall not be allowed under roadways or in public rights-of-way. HDPE pipe may utilize in areas that do

not accommodate vehicular loading. In all cases, the pipe shall be completely contained within a drainage easement.

4. Culverts shall span the entire width of the crossing facility, typically a roadway or railroad embankment.

5. Culverts shall be aligned with the centerline of the existing/natural channels, ditches, swales, and other low areas to ensure maximum hydraulic efficiency and minimal erosion.

6. Culverts shall be aligned so that the outfall is pointing towards the downstream end of the receiving channel.

7. Culverts shall be aligned to avoid severe (greater than 90 degrees) channel bends.

8. Culvert locations and alignments shall be such that adequate access for maintenance can be achieved.

9. Culverts shall be aligned to avoid conflicts with existing buildings, homes, pipelines, and contaminated sites.

10. Culverts shall be aligned to minimize the number of property owners affected.

11. The maximum skew angle for box culverts is 45 degrees.

12. Headwalls and endwalls shall be utilized for culverts greater than 36 inches in diameter to control erosion and scour, anchor the culvert against lateral pressures, and ensure bank stability.

13. All headwalls shall be constructed of reinforced concrete.

14. Headwalls shall be functionally monolithic with the culvert conduit and must generally be parallel with the alignment of the crossing roadway.

3.5.1.3 Erosion

Structural erosion protection shall be added downstream of the culvert based on the culvert exit velocity. Erosion protection shall continue downstream to the point where the engineer can re-establish normal flow characteristics.

3.5.1.4 Driveway Culverts

Driveway culverts shall be sized assuming full flow conditions using Manning’s formula (Eq. 3-3)

$$Q = \frac{1.49}{n} A R^{2/3} S_o^{1/2} \tag{3-3}$$

Where,

Q	=	Discharge in the culvert (cfs)
n	=	Manning's 'n' value for the pipe
A	=	Cross-sectional area of the conduit (sf)
R	=	Hydraulic Radius of the conduit (ft)
S	=	Slope of the pipe (ft / ft)

3.5.2 Bridges

3.5.2.1 Bridge Design Considerations

Bridges should be designed per current AASHTO LRFD Bridge Design Specifications with Interims. HL-93 Loading shall be used for bridges. Bridge designs shall be sealed by a Texas licensed professional engineer with experience in bridge design.

Bridges and bents constructed on existing or interim channels shall be designed to accommodate the ultimate planned channel section with minimum structural modifications.

Existing vegetation should be incorporated into the overall bridge plan. Where practicable, trees and shrubs should be left intact even within the right-of-way. Minimizing vegetation removal also tends to control turbulence of the flow into, though, and out of the bridge and mitigate erosion potential.

3.5.2.2 Bridge Location and Orientation Guidelines

Newly constructed bridges must be designed to completely span the existing or proposed channel such that the channel will pass under the bridge without modification. Energy losses due to flow transitions shall be minimized.

The bridge shall be centered on the main channel portion of the entire floodplain. This may cause an eccentricity in location with respect to the entire stream cross section but allows for better accommodation of the usual and low flows of the stream.

The bridge waterway opening shall be designed to provide a flow area sufficient to maintain the through-bridge velocity for the design discharge no greater than the allowable through-bridge velocity. The headers and interior bents shall be oriented to conform to the streamlines at flood stage. Standard skew values of 15 degrees, 30 degrees, and 45 degrees should be used where feasible.

The piers and the toe of slope of the header must be located away from deep channels, cuts, and high velocity areas to avoid scour problems or interference with stream low flows.

3.5.2.3 Freeboard

At a minimum, bridges shall be designed to pass the fully developed 100-year design flow without causing backwater problems, structural damage, or erosion.

The low chord of all bridges shall be located at least one foot above the 100-year flood elevation, or at the level of natural ground, whichever is higher. More freeboard may be appropriate for bridges over streams that are prone to heavy debris loads, such as large tree limbs.

3.5.2.4 Scour Analysis

A scour analysis utilizing HEC-RAS is required for all new bridges, replacements, and widenings. Where a scour analysis indicates high depths of potential contraction scour, measures to withstand the scour are required. Alternative methods for computing scour shall be approved in writing by the City's Development Services Director prior to submitting calculations.

3.5.2.5 Approach Embankments

Embankment protection shall be required for approach section channel velocities greater than 6.0 feet per second. Existing vegetation shall be maintained as much as possible surrounding bridge construction to minimized erosion potential.

3.5.2.6 Abutments

Bents and abutments shall be aligned parallel to the longitudinal axis of the channel to minimize obstruction of flow. Bents shall be placed as far away from the channel centerline as possible and if possible, should be eliminated entirely from the channel bottom.

Protective measures are required at each of the following instances:

- Header slopes
- Deep toe walls
- Vertical abutment walls
- Sheet pile toe walls
- Deep foundations of piles or drilled shafts

To prevent embankment failure from undermining by contraction scour, a toe wall must be extended below the level of expected scour. Stone is generally preferred to concrete for erosion protection near abutments.

3.5.2.7 Hydraulic Analysis

A hydrologic and hydraulic analysis is required for designing all new bridges over waterways, bridge widening, bridge replacement, and roadway profile modifications that may adversely impact the floodplain even if no structural modifications are necessary.

Peak flow rates or hydrographs for the design storm event shall be determined using the hydrologic methodologies outlined in Chapter 2.

The hydraulic analysis should include the following:

- Determination of the backwater associated with each alternative profile and waterway opening(s)

- Determination of the effects on flow distribution and velocities for the 2-year, 5-year and 100-year storm events.

- Existing and proposed condition water surface profiles for design and check flood conditions

- A scour analysis

The latest version of HEC-RAS shall be used to complete the hydraulic analysis. The use of other software programs must be approved in writing by the City's Development Services Director prior to submittal of calculations or design drawings.

4 LOCAL DRAINAGE SYSTEM HYDRAULICS

4.1 General

This Chapter summarizes the requirements for design of local drainage systems within the City of Horseshoe Bay. All development within the City shall be accommodated within enclosed drainage systems (closed system), bar ditches, or roadway rights-of-way. Local drainage shall be adequately conveyed to an open channel with adequate capacity to carry the developed discharge as designated in the city's Drainage Master Plan, or to Lake LBJ. All drainage systems shall be contained within drainage easements or rights-of-way sized to carry the fully developed 100-year, 24-hour discharge to a point of adequate outfall, with adequate points of access.

Developments requiring the construction of street infrastructure shall design street curb and gutter and closed stormwater systems as necessary to convey the fully developed 100-year storm event between the curbs. Major arterials must maintain one lane of traffic open in each direction during the designed fully developed 100-year storm event.

When a design approach is to be used that is not covered in this manual, the City's Development Services Director shall provide written acceptance of the alternative methodology prior to submittal of calculations to the City.

4.2 Closed Systems Design Considerations

4.2.1 Design Flow Frequency Criteria

Local closed system drainage design shall accommodate a 5-year storm with calculated hydraulic grade lines (HGL) below the roadway gutter line. The 100-year storm shall be contained within a drainage easement or roadway right-of-way. A clear overflow route, including a dedicated drainage easement shall be maintained from the system outfall to an open channel with adequate capacity to carry the developed discharge as designated in the city's Drainage Master Plan, or to Lake LBJ.

4.2.2 Finished Floor Elevations

Finished floor elevations shall be a minimum of 2-feet above the adjacent 100-year HGL. All proposed drainage systems shall maintain a maximum HGL of 2-feet below the lowest adjacent grade of existing habitable structures. Code of Federal Regulations (CFR) Chapter 65 FEMA documentation shall be consulted for the definition of lowest adjacent grade.

Unless superseded by specific requirements of the appropriate Drainage Regulation Entity, the following specific criteria and requirements shall apply to the design and construction of storm sewer systems.

4.2.3 Design Requirements

The following minimum criteria shall be utilized:

1. All storm sewer and appurtenant construction shall conform to the Texas Department of Highway and Public Transportation Construction Specifications and all subsequent revisions or approved equal.

2. All storm sewer, excluding outfalls, shall be constructed with reinforced concrete pipe, or approved equal, unless approved in advance in writing by the City's Development Services Director.

3. Storm sewer design calculations may be performed using a manual spreadsheet design or a proprietary design software. In any case, design calculations shall be included in the storm sewer construction plans to confirm design results.

4. Calculation of the hydraulic grade line for design conditions in a specific branch of storm sewer shall proceed upstream from the water surface elevation at the outfall location. The HGL for the design storm shall be included on the profile sheets for the storm sewer.

5. The storm sewer system shall be designed to convey runoff from the design storm event without causing the hydraulic grade line to exceed the gutter flow line in the street.

6. The minimum diameter of a storm sewer pipe shall be 18-inches.

7. Pipe sizes shall increase in the downstream direction, regardless of additional capacity developed by increased grade.

8. Pipe soffit (inside top) elevations shall match whenever practical. Where there is a connection of different conduit sizes on the trunk line, the soffit elevations, rather than the flow line elevations, shall be approximately the same. The soffit elevation of the incoming pipe may be offset (increased) by an amount equal to the headloss at the structure where the conduits meet.

9. The minimum velocity to be allowed in a section of storm sewer flowing full shall be 3 feet per second. The maximum velocity shall be 15 feet per second. Refer to Section 3 for storm sewer outfall velocities and erosion protection.

10. All cast-in-place concrete storm sewers shall follow the alignment of the right-of-way or easement. The Minimum easement width of 15-feet shall be centered in the storm Sewer.

11. For all storm sewers having a cross-sectional area equivalent to a 42-inch inside diameter pipe or larger, soil borings with logs shall be made along the alignment of the storm sewer at intervals not to exceed 500-feet and to a depth not less than 3-feet below the flowline of the sewer. The required bedding of the storm sewer as determined from these soil borings shall be shown in the profile of each respective storm sewer. The design engineer shall inspect the open trench and may authorize changes in the bedding indicated on the plans. Such changes shall be shown on the record drawings and, along with soil boring logs. All bedding and subsequent revisions shall be constructed as specified in the Texas Department of Highways and Public Transportation Specifications or approved equal.

12. All storm sewer inlet leads shall be designed in a straight-line alignment. Inlet leads shall be a minimum 18-inch diameter.

4.2.4 Design Tailwater Conditions

Downstream tailwater conditions shall be determined based on the size of the system's drainage area relative to the drainage area of the receiving point. Table 4-1 shows a coincidental peak comparison. The starting tailwater shall be the water surface elevation at the receiving point for the corresponding design return event.

Table 4-1 Frequencies for Coincidental Occurrence

Area Ratio	2-Year design		5-Year design		25-Year design		100-Year design	
	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	2	1	5	2	25	2	100
	2	1	5	1	25	2	100	2
1000:1	1	2	2	5	5	25	10	100
	2	1	5	2	25	5	100	10
100:1	2	2	2	5	10	25	25	100
	2	2	5	5	25	10	100	25
10:1	2	2	5	5	10	25	50	100
	2	2	5	5	25	10	100	50
1:1	2	2	5	5	25	25	100	100
	2	2	5	5	25	25	100	100

4.2.5 Computation of Hydraulic Grade Line

To adequately design the storm sewer system, the hydraulic grade line must be computed in order to determine whether the storm sewer meets the design requirements. Hydraulic grade line calculations for the design storm shall be provided on the construction plans. In addition, a submittal is required confirming 100-year HGL is maintained in an easement or right-of-way to a point of adequate outfall as defined in the opening section of this chapter.

4.2.5.1 Friction Losses

The main source of head loss in a storm sewer system is the friction loss in the conduit. The following two equations were derived from the Manning's equation and can be used to determine the friction loss in the conduit. Equation 4-1 is the friction loss for a circular pipe and Equation 4-2 is the friction loss of any other type of conduit of a known cross-sectional area and hydraulic radius.

$$h_f = L\left[\frac{Qn}{0.4644D^{8/3}}\right]^2 \tag{4-1}$$

$$h_f = L\left[\frac{Qn}{1.496R^{2/3}A}\right]^2 \tag{4-2}$$

Where,

h_f	=	Head loss due to friction along the length of the conduit (ft)
Q	=	Discharge in the conduit (cfs)
L	=	Length of the conduit (ft)
n	=	Manning's 'n' value for the pipe
D	=	Diameter of the pipe (ft)
R	=	Hydraulic Radius of the conduit (ft)
A	=	Cross-sectional area of the conduit (sf)

Table 4-2 provides Manning's "n" values for typical pipe materials accepted for closed conduits in Horseshoe Bay.

Table 4-2 Manning's "n" for typical pipe materials

Material	Roughness Coefficient (n)
Reinforced Concrete Pipe	0.013
Reinforced Concrete Box	0.015
Corrugated Metal	0.024

4.2.5.2 Minor Head Losses

Minor head losses shall be determined at all entrances, structures, and bends. While the head losses at each structure may be minor, the cumulative effect of the combined minor losses to the hydraulic grade line throughout the entire system can be significant.

i. Head Loss at Bends

For flow through a junction such as a manhole where no additional flows are entering into the system and where there is no change in pipe size across the junction, Equation 4-3, along with the loss coefficients provided in Table 4-3 may be used to determine the head loss across the structure.

$$h_m = K\frac{(V)^2}{2g} \tag{4-3}$$

Where,

h_m = Minor head loss across the junction (ft)
K = Loss coefficient
V = Velocity in the conduit (ft/sec)
g = Acceleration due to gravity (ft/sec^2)

Table 4-3 Loss Coefficients for Bends

Type of Junction	Coefficient (K_j)
Straight through manhole	0.05
22.5-degree bend	0.20
45-degree bend	0.35
60-degree bend	0.43

90-degree bend	0.50

ii. Head Loss at Junctions

For junctions where a pipe change occurs, where lateral pipes joins the junction, or where additional flow is introduced with an inlet, Equation 4-4 along with the loss coefficients provided in Table 4-4 may be used to determine the head loss across the structure:

$$h_m = \frac{(V_2)^2}{2g} - K\frac{(V_1)^2}{2g} \qquad (4\text{-}4)$$

Where,

h_m	=	Minor head loss across the junction (ft)
K	=	Loss coefficient
V_1	=	Velocity in the main line upstream of the junction (ft/sec)
V_2	=	Velocity in the main line upstream of the junction (ft/sec)
g	=	Acceleration due to gravity (ft/sec^2)

Table 4-4 Loss Coefficients for Junctions

Type of Junction	Coefficient (K_j)
Inlet on main line	0.50
Inlet on main line with a branch lateral	0.25
Junction or manhole on main line with a 22.5-degree branch lateral	0.75
Junction or manhole on main Line with a 45-degree branch lateral	0.50
Junction or Manhole on main line with 60-degree branch lateral	0.35
Junction or manhole on main line with 90-degree branch lateral	0.25

iii. Entrance Losses

For entrances where flow is entering the storm sewer, Equation 4-3 along with the loss coefficients provided in Table 4-5 may be used to determine the head loss at the entrance:

Table 4-5 Loss Coefficients for Entrances

Type of Entrance	Coefficient (K_j)
Inlet Entrance	1.25
Conduit Projecting from Fill, Socket End (Groove End)	0.20
Projecting from fill, square cut end	0.50
Headwall and wingwalls	
Socket end of pipe (groove-end)	0.20
Square-edge	0.50
Rounded (radius = 1/12D)	0.20
Mitered to conform to fill slope	0.70

4.2.6 Junction Structures

Manholes shall be a minimum of 48-inches in diameter. All storm sewer junction boxes shall have a manhole access at natural ground. Manholes or junction boxes shall be placed at the following locations:

1. At the location of all changes in storm sewer size or cross section.
2. At storm sewer intersections or P.I.'s.
3. At storm sewer slope changes.
4. At street intersections.

5. At all inlet lead intersections with the storm sewer where precast concrete storm sewers are proposed.

6. At maximum intervals measured along the centerline of the storm sewer per Table 4-6 below:

Table 4-6 Manhole Placement Intervals

Pipe Diameter or Height (in)	Maximum Distance (ft)
24	300
30-36	375
45-54	450
60+	900

4.2.7 Inlets

4.2.7.1 Design Criteria

The following specific criteria and requirements shall apply to the design and construction of inlets.

1. All inlets shall be constructed as specified in the Texas Department of Transportation Specifications.

2. All inlets shall be designed to convey the peak flow rate for the 5-year design storm event and meet the following limitations:

3. Depth of ponding shall not exceed the top of curb.

4. For residential streets, the flow spread shall not exceed the center crown of the roadway.

5. For a roadway with two or more lanes in each direction, the flow spread shall maintain at least a 12-feet wide dry travel lane.

6. Inlets shall be spaced so that the maximum travel distance of water in the gutter will not exceed six-hundred feet (600’) one way for residential streets and three-hundred feet (300’) one way for roads with two or more lanes in each direction.

7. Curb inlets shall be located on intersecting side streets to major thoroughfares for all original designs or developments to prevent concentrated storm water flow from crossing traffic lanes. Special conditions warranting other locations of inlets shall be determined on a case-by-case basis.

8. Valley gutters shall not be allowed.

9. All inlets shall be located in public street right-of-way or in easements that will not prohibit future maintenance access.

4.2.7.2 Sump Inlet Hydraulic Calculations

Inlets placed in a local low point of a road profile are termed sump inlets. In this configuration, the flow velocity across an inlet is assumed to be zero. In this case, the standard weir and orifice equations are applicable for determining flow capture capacity.

A curb inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage and inlet capacity is based the lesser of the weir and orifice capacities.

For determining the capacity of a curb inlet located in a sag that is submerged, the standard orifice equation, Equation 4-5, may be used.

$$Q = C_o d_o L\sqrt{2gh} \tag{4-5}$$

Where,

Q	=	Total flow reaching the inlet (cfs)
C_o	=	Orifice coefficient (typically 0.67)
D_o	=	Physical depth of curb opening, including depression depth (ft)
L	=	Length of curb opening inlet (ft)
g	=	Acceleration due to gravity (32.2 $\mathrm{ft/sec^2}$)
h	=	Effective head at center of the orifice throat (ft)

For determining the capacity of a curb inlet located in a sag that is not submerged, the standard weir equation, Equation 4-6, may be used.

$$Q = C_w(L + 1.8W)y^{1.5} \tag{4-6}$$

Where,

Q	=	Inlet capacity (cfs)

C_w = Weir coefficient (typically 3.0 for inlets without curb depression, typically 2.3 for depressed inlets (for English units)
L = Length of the opening which water enters the inlet (ft)
y = Total depth of water or head on the inlet (ft)
W = Gutter depression depth (ft)

4.2.7.3 On-Grade Inlet Hydraulic Calculations

Inlets placed on the sloped portion of a road profile are in an on-grade configuration. Inlets are typically placed on-grade in order to reduce the amount of flow in the gutter and, in turn, the flow spread in the roadway prior to reaching the low point in the road profile.

For determining the length of inlet required to capture the total flow in the gutter, Equation 4-7 may be used.

$$L_r = K_c Q^{0.42} S^{0.3} (\frac{1}{nS_e})^{0.6} \qquad (4\text{-}7)$$

Where,

L_r = Length of opening required to intercept total flow in the gutter (ft.)
K_c = Coefficient = 0.6 (English units)
Q = total flow in the gutter (cfs)
S = Longitudinal slope of gutter (ft/ft)
n = Manning's roughness coefficient
S_e = Equivalent cross slope (ft/ft)

Carryover flow is allowed. It must be determined and applied to the next downstream gutter and inlet hydraulic calculations. This carryover flow may be calculated using Equation 4-8

$$Q_{co} = Q(1 - \frac{L_a}{L_r})^{1.8} \qquad (4\text{-}8)$$

Where,

Q_{co} = Carryover flow (cfs)
Q = Total flow in the gutter (cfs)
L_a = Design length of the curb opening (ft)
L_r = Length of opening required to intercept total flow in the gutter (ft)

4.2.8 Gutter Hydraulic Calculations

More frequent inlet spacing, especially for on-grade inlets, may be required to maintain a spread of gutter flow in the roadway less than the maximum allowed ponding widths specified in Section

4.2.7.1. Equation 4-9 may be used to determine the depth of flow in the gutter and Equation 4-10 may be used to determine the flow spread width in the roadway.

$$y = z\left(\frac{QnS_x}{S^{1/2}}\right)^{3/8} \tag{4-9}$$

Where,

y	=	Depth of water in the curb and gutter cross section (ft)
z	=	1.24 for English measurements or 1.443 for metric
Q	=	Gutter flow rate (cfs)
n	=	Manning's roughness coefficient (Table 4-7)
S_x	=	Transverse Slope (ft/ft)
S	=	Longitudinal slope (ft/ft)

$$T = \frac{y}{S_x} \tag{4-10}$$

Where,

T	=	Ponded width of flow
Y	=	Depth of standing water or head on the inlet
S_x	=	Transverse slope (1/z)

Table 4-7 Manning's n-Values for Street Pavement Gutters

Type of Gutter or Pavement		Roughness Coefficient (n)
Asphalt Pavement	Smooth Texture	0.013
	Rough Texture	0.016
Concrete Gutter with Asphalt Pavement	Smooth Texture	0.013
	Rough Texture	0.015
Concrete Pavement	Float Finish	0.014
	Broom Finish	0.016

4.2.9 Bar Ditch Design

A bar ditch is defined as a trapezoidal channel located in a road right-of-way and shares a common edge with a roadway. Open bar ditches shall be contained completely within the road right-of-way and/or road drainage easement.

Bar ditches shall be designed to accommodate the 5-year storm event within the channel. The 100-year storm event may expand to the full roadway right-of-way. The minimum bottom width is 2-feet, with side slopes no steeper than 4:1.

Driveway culverts or other road crossings of a bar ditch shall be analyzed to confirm that the 5-year HGL may be contained in the ditch. Culverts serving a drainage area of 10 acres or less may be analyzed as a stand-alone structure, with a normal depth downstream water surface elevation. For drainage areas greater than 10 acres, a channel backwater analysis shall be performed.

5 DETENTION

5.1 General

Three methods are acceptable for mitigating the impact of increased peak discharge due to increased impervious cover from land development activities:

- No change in impervious cover from existing conditions. Calculations and documentation shall be submitted to substantiate maintained impervious cover values.
- Acquisition of drainage easements from the point of impact to an open channel with adequate capacity to carry the developed discharge as designated in the city's Drainage Master Plan, or to Lake LBJ. Easements shall be of sufficient size to accommodate fully developed conditions discharges.
- Volumetric storage of stormwater runoff is a common mitigation technique used to offset the impact of increased peak discharge due to development. This section of the manual presents requirements for the design of appropriate storm runoff storage facilities.

5.2 Downstream Impact Assessment Requirements

A downstream impact assessment is required for all proposed Major Projects as defined in the City of Horseshoe Bay's Development Guide. For all proposed Major Projects, a downstream impact analysis is required. Analyze using HEC-HMS and HEC-RAS through the entire downstream channel section for the 2-year, 25-year, and 100-year events. Analysis shall be extended downstream to a point where the proposed development does not increase flow rates by more than 0.40 cfs and does not increase water surface elevations more than 0.1 feet for all analyzed storms. In all cases, proposed developments shall not create more than a 5% increase in channel velocities and shall not produce a net loss of storage volume below the 100-year floodplain. Offsite discharges shall be contained within a drainage easement upon leaving the proposed project site to an open channel with adequate capacity to carry the developed discharge as designated in the City's Drainage Master Plan, or to Lake LBJ.

Where a development discharges directly into the Lake LBJ or where a detailed hydrologic timing analysis shows that peak discharges are not increased at any point within the limits of the downstream impact assessment, detention is not required.

5.3 Detention Storage Types

The purpose of detention storage is to hold storm runoff and release it continuously at a rate that mitigates a development's increase in peak discharge. Using a flow-limiting outlet structure, downstream peak discharges can be controlled. The outlet works of a detention facility shall mitigate increased peaks in discharge from 2-year, 25-year, and 100-year return event storms. In cases where the detention basin contributing drainage area is significantly different from the receiving stream drainage area, the coincidental peak table, Table 5-1 shall be used to determine an appropriate downstream water surface elevation.

5.3.1 Retention Storage

In a retention storage facility (wet pond), a constant water surface elevation is maintained below the detention storage volume for aesthetic purposes. When such aesthetic design is included, the volume of storage below the constant water surface elevations (normal pool) shall not be included in the storage volume calculations. The normal pool water surface elevation shall be at least 6-feet deep, measured from the bottom of the pond to the normal pool water surface elevation. The detention outlet for peak discharge mitigation shall be above the normal pool.

A pond aerating pump and fountain system shall be required for all retention storage facilities. Construction plans shall include details of the pump, its capacity, details on construction for affixing the pump, and a detailed operation and maintenance schedule for the pump. Aeration pumps shall be operated minimum of once every 48 hours to maintain proper water quality and to minimize habitat for nuisance pests and vegetation. Aeration pumps shall provide at least 4 hours of continuous operation.

5.3.2 On-line Storage

An on-line storage facility is one in which the total upstream storm runoff volume passes through the retention or detention facility's outflow structure.

5.3.3 Off-channel Storage

An off-channel storage design is one in which storm runoff does not begin to flow into the storage facility until the discharge in the channel reaches some critical value above which unacceptable increases in downstream discharge will occur. An offline facility serves to store only the runoff volume associated with the high flow rate portions of the flood event.

5.4 Design Procedures

5.4.1 For Drainage Areas < 50 Acres

The maximum allowable release rate from the detention facility is the rate of runoff from the drainage area prior to development. A proposed detention pond operating as an isolated facility may be designed using Modified Rational Method hydrologic calculations.

The size of the outlet pipe required to pass the maximum allowable release rate during the 100-year storm must be computed assuming outlet control. Establish a maximum ponding level in the detention facility during the 100-year storm and utilize the receiving stream tailwater at the downstream end of the outlet pipe as described in Section 5.4.3. The outlet pipe shall be configured to mitigate increases in peak discharge for the 2-year and 25-year frequency storms as well.

In certain instances of commercial development, where only a small amount of detention storage is required, a parking lot within the proposed development may be utilized for detention storage. When a parking lot is utilized for detention, the following design requirements shall be achieved:

1. No more than 50% of the parking spaces may be impacted by the detention facility.
2. Within parking spaces, ponding depth may be no more than 8 inches high.
3. Outfall calculations shall be required.
4. Freeboard depth is not required.

Detention ponds designed to operate in series, or in conjunction with other peak discharge timing considerations, shall be sized as described in Section 5.4.2.

5.4.2 For Drainage Areas > 50 Acres

A detailed hydrologic analysis utilizing HEC-HMS or another method that has been accepted in writing in advance by the City's Development Services Director

shall be required to develop inflow and outflow hydrographs at the detention location. The HEC-HMS model shall then be used to verify the size of the facility and the outlet structure and to mitigate the project's impact on downstream flooding conditions. Once existing conditions are established, the impact of the proposed development including the detention facility shall be analyzed for the 2-year, 25-year, and 100-year frequency storm events, utilizing NOAA Atlas 14 precipitation data. The 2-year and 25-year event storms shall be used for determining the sizing of the outflow structure to prevent impacts to downstream properties. The 100-year will be used for sizing the required detention volume and configuring an emergency spillway. The detention facility will be sized to allow an appropriate release rate that will not cause any increase in flood levels downstream.

5.4.3 Design Tailwater Depth

In order to route the inflow hydrograph through the detention facility in the HEC-HMS model, a relationship must be established between the volume of storage in the pond and the corresponding amount of discharge through the outflow structure. In most cases in Horseshoe Bay, this relationship is directly dependent on the elevation of the tailwater at the outlet of the outflow structure. For the purpose of establishing an outflow rating curve, detention facilities that are evaluated using computer models shall use a variable tailwater condition based on the frequency storm being analyzed. The design engineer shall submit calculations to the City's Development Services Director to substantiate the design.

In certain situations where the drainage area for the detention basin is significantly different than the drainage area for the receiving stream at the point of discharge from the detention basin, coincidental peak with the design storm event shall be considered for the determination of tailwater elevation using Table 5-1 A coincidental peak analysis is useful to reduce the starting HGL based on the ratio of the storm system's drainage area to the watershed area of the receiving stream. For the determination of hydraulic gradient and the sizing of storm drain conduits, a tailwater elevation which can be reasonably expected to occur coincident with the design storm event shall be used. Provide calculations for each coincidental occurrence and the more restrictive results based on the area ratio shall be used.

Table 5-1 Frequencies for Coincidental Occurrence

Area Ratio	2-Year design		5-Year design		25-Year design		100-Year design	
	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary
10,000:1	1	2	1	5	2	25	2	100
	2	1	5	1	25	2	100	2
1000:1	1	2	2	5	5	25	10	100
	2	1	5	2	25	5	100	10
100:1	2	2	2	5	10	25	25	100
	2	2	5	5	25	10	100	25
10:1	2	2	5	5	10	25	50	100
	2	2	5	5	25	10	100	50
1:1	2	2	5	5	25	25	100	100

	2	2	5	5	25	25	100	100

5.4.4 Final Sizing of Pond Storage and Outflow Structure

To size detention facilities, a minimum of 6 inches of freeboard is required if the average pond depth is 3 feet or less. For ponds with an average depth greater than 3 feet, a minimum of 12 inches of freeboard shall be maintained during the 100-year storm event.

The maximum side slope allowed is 4 (horizontal):1(vertical) for long term stability and maintenance.

Minimum easement width of 10-feet shall be dedicated around a detention basin to allow for maintenance equipment access.

The volume of water held in public infrastructure (storm drain pipes or roadside ditches) shall not be considered as available detention volume.

The minimum allowable diameter for an outflow pipe of a detention facility is 24 inches. For outfall to a roadside ditch, an 18-inch diameter pipe may be utilized upon approval by the City's Development Services Director. The end treatment of an outfall pipe shall be cut to match the receiving stream slope. The outfall pipe embedment shall include a minimum of 1 foot of stabilized sand around the pipe. The outfall flowline shall be no more than 6-inches higher than the receiving stream flowline at the point of connection.

Roadside ditches on county roads may be used as drainage outfalls only when all the following conditions are met:

1. The roadside ditches are the only existing means of drainage.
2. Property being developed drains naturally to the roadside ditches.
3. Runoff after development is limited to the runoff that occurred prior to development.

5.4.5 Allowances for Extreme Storm Events

For all detention ponds greater than 0.5 acres of surface area, an emergency spillway, overflow structure, or swale (collectively referred to as the "emergency spillway") must be sized with the following criteria:

- Set the emergency spillway crest elevation at or above the 100-year pond elevation.
- Size the emergency spillway to carry the 100-year flow so that the 100-year detention storage does not exceed the top-of-bank of the facility.
- Assume that the principal outlet structure is completely blocked.

- Size the emergency spillway assuming a normal pool level or a dry condition at the beginning of the storm.
- A minimum of 12-inches of freeboard above the spillway crest is required for an emergency overflow/extreme event condition.

- If the spillway is not immediately adjacent to a receiving stream, obtain a flowage easement to provide a clear path for conveyance without affecting adjacent property owners.

In places where a dam has been utilized to provide detention, due consideration shall be given to the consequences of a failure. Downstream of any proposed dam and easement shall be dedicated to contain the 100-year storm event in the event of dam failure. If a significant hazard exists, the dam must be adequately designed to prevent such hazards.

In addition, detention facilities with an outfall berm greater than 6 feet in height are subject to Title 30 Texas Administrative Code (TAC) Chapter 299 (Sub chapters A through E, latest edition) and all subsequent changes. The height of a detention facility or dam is defined as the distance from the lowest point on the crest of the dam (or embankment), excluding spillways, to the lowest elevation on the centerline or downstream toe of the dam (or embankment) including the natural stream channel. Subchapters A through E of Chapter 299 classify dam sizes and hazard potential and specify required failure analyses and spillway design flood criteria.

5.4.6 Erosion Controls

The same types of erosion protection required in earthen channels, as outlined in Section 3, shall be incorporated in detention design. This includes proper revegetation and pond surface lining, where necessary. Extra care must be taken to provide proper protection at pipe outfalls into the facility, pond outlet structures, and overflow spillways where excessive turbulence and velocities will encourage erosion.

5.5 Multipurpose Land Use

When a dual use facility is proposed, a joint use agreement is required between the City and the entity sponsoring the secondary use. In all cases, maintenance shall be the responsibility of the private property owner and not the City.

5.5.1 Approval of Facilities

Each stormwater detention facility will be reviewed and accepted only if:

1. The facility has been designed to meet or exceed the requirements contained within this manual; and

2. Provisions are made for the facility to be adequately maintained and a maintenance agreement is completed between the City and the private property owner; and

3. If walking paths or other amenities are anticipated, sufficient details of each improvement shall be provided to City for review and comment. The trail or path geometry and location may be required to utilize special specifications or provide access for maintenance vehicles to cross the facility.

5.5.2 Maintenance

Each development that provides detention shall make provisions to ensure future maintenance of the detention facility. The entity responsible for the maintenance of the facility shall be noted on the plat and plans.

The City's Drainage Staff and Maintenance Staff have the right to inspect the facility and determine if the facility is maintained to designed conditions.

Detention storage shall be wholly contained within a drainage easement. Private maintenance shall be required, and a maintenance agreement shall be executed with the City of Horseshoe Bay. Access by the City shall be allowed at all times. If the private property owner does not properly maintain the facility to its design capacity and function, the City shall have the right, but not the responsibility, to enter the property and maintain the facility. The City reserves the right to lien the property for the cost of the maintenance upon failure of the property owner to perform prompt, adequate maintenance.

5.6 Geotechnical Investigation

Before initiating final design of a detention pond, a detailed soils investigation by a professional geotechnical engineer, licensed in the State of Texas, shall be undertaken. The following minimum requirements shall be addressed:

1. Stability of the basin side slopes for short term and long-term conditions. (If basin depth ≤ 5 feet, a slope stability analysis is not required, however, a geotechnical report is still required to address the other issues.)

2. Stability of the permanent pool side slopes.

3. Evaluation of bottom instability due to excess hydrostatic pressure.

4. Control of groundwater.

5. Identification of dispersive soils.

6. Potential erosion problems.

7. Constructability issues.

8. Evaluation of inflow and outflow structures.

9. If a dam is to be constructed, the following shall be required:
   a. adequate investigation of potential seepage problems through the dam;
   b. attendant control requirements;
   c. availability of suitable embankment material;
   d. stability requirement for the dam itself.

10. Investigation into the potential for structural movement on areas adjacent to the pond may be required. This is mainly due to the induced loads from exiting or proposed structures and methods of controlling it.

5.7 General Requirements for Detention Pond Construction

The structural design of detention facilities is very similar to the design of open channels. For this reason, all requirements from Section 3 pertaining to the design of lined or unlined channels shall also apply to lined or unlined detention facilities.

1. Dry Pond Bottom Design – A pilot channel shall be provided in detention facilities to ensure that proper and complete drainage of the storage facility will occur. Concrete pilot channels shall have a minimum depth of 6-inches, 4:1 side slope and a minimum flowline slope of 1.0%.

2. The bottom slopes of the detention basin shall be graded toward the pilot channel at a minimum slope of 1.0%. Detention basins which make use of a channel section for detention storage may not be required to have a pilot channel but shall be built in accordance with the requirements for open channels as outlined in Section 3.

3. Retention Design – Ponds with a permanent pool shall include a shallow shelf below the normal pool to reduce the risk of falling into the water by running or rolling down the side

slope. The shelf shall be located one foot above the static (normal pool) water surface (normal pool level), have a minimum width of 10 feet, and a cross slope of 0.02 ft/ft.

4. Shallow pools may be used around the edges of deeper pools to support aquatic plants and habitat, and to improve water quality. However, shallow pools alone cannot be used in a pond bottom due to maintenance issues. Wet ponds shall be at least 6-feet deep from the lowest point to the top of bank, measured from the lowest point in a dry basin, or the normal pool water surface elevation.

5. Reinforced concrete pipe used in the outlet structure shall conform to ASTM C-76 Class III with compression type rubber gasket joints conforming to ASTM C-443. Pipes, culverts and conduits used in the outlet structures shall be carefully constructed with sufficient compaction of the backfill material around the pipe structure as recommended in the geotechnical analysis. Generally, compaction density should be the same as the rest of the structure. The use of pressure grouting around the outlet conduit should be considered where soil types or conditions may prevent satisfactory backfill compaction. Pressure grouting should also be used where headwater depths could cause backfill to wash out around the pipe.

6 SINGLE- and TWO-FAMILY RESIDENTIAL DRAINAGE AND GRADING

6.1 General

All residential development is required to consider the impact of the proposed development on existing drainage systems and on the potential for soil erosion.

Horseshoe Bay encourages the use of native vegetation and natural areas in development of residential properties.

6.2 Residential Drainage

1. When an existing residential development proposes improvements that increase impervious cover on the lot by more than 50%, a drainage plan, sealed by a professional engineer licensed in the State of Texas, shall be required. The drainage plan shall meet all requirements set forth in this manual. All relevant items from Section 3.4.1 of this manual shall be included in the drainage plan.

2. When a proposed residential site includes improvements exceeding the impervious cover standards listed in Table 6-1, a drainage plan, sealed by a professional engineer licensed in the State of Texas, shall be required. The drainage plan shall meet all requirements set forth in this manual. All relevant items from Section 3.4.1 of this manual shall be included in the drainage plan.

Table 6-1 Maximum Allowable Impervious Cover

Residential Lot Size (ac)	Impervious Cover (%)
1/8	65
1/4	50
1/3	40
1/2	35
1	30
2	25

3. When improvements on a residential lot include work in an area reserved as an easement, a drainage plan, sealed by a professional engineer licensed in the State of Texas, shall be required. The drainage plan shall meet all requirements set forth in this manual. All relevant items from Section 3.4.1 of this manual shall be included in the drainage plan.

4. Driveway culverts on a residential lot shall be analyzed to confirm that the 5-year HGL may be contained in the ditch. Culverts serving a drainage area of 10 acres or less may be analyzed as a stand-alone structure, with a normal depth downstream water surface elevation. For drainage areas greater than 10 acres, a channel backwater analysis shall be performed.

5. No drainage pipes smaller than 6-inches in diameter shall be utilized to convey drainage on a residential property.

6. Finished floor elevations on all proposed residential properties shall be at or above the higher of the following criteria:

   a. At least 2 feet above the fully developed conditions 100-year water surface elevation of an adjacent creek;
   b. At least 2 feet above the hydraulic grade line of an adjacent public storm drain pipe or top of pipe when no records exist;
   c. At least 2 feet above the lowest perimeter elevation.

6.3 Residential Grading Requirements

1. Erosion control devices shall be in place and fully functional prior to any construction on a residential site.

2. All lot grading for residential development shall meet the requirements of the International Residential Code, latest edition adopted by Horseshoe Bay.

3. All land disturbance areas shall be revegetated immediately after site construction activities are complete to minimize erosion. Erosion control devices shall remain in place until vegetation is established. Vegetation shall achieve 80% coverage prior to issuance of Certificate of Occupancy. Deviation from this requirement requires written approval from the Building Official.

4. Finished grades on a residential lot shall be no steeper than 4:1 (H:V). A geotechnical analysis, sealed by a Texas registered professional engineer shall be required to support a request for steeper side slopes. Grades steeper than 4:1 shall only be allowed by written permission of the City's Development Services Director.

5. Residential driveways shall be no steeper than 8% longitudinal slope and 2% cross slope.

6. Any proposed retaining wall higher than 4’ above ground shall require an engineering foundation design, sealed by a registered professional engineer, licensed in the State of Texas.

APPENDIX 1: Design Checklist

APPENDIX 2: Maintenance Agreement

APPENDIX 3: Infrastructure Plan Checklist with Drainage

APPENDIX 4: Fill Permit