PLANNING AND DESIGN DRAINAGE CRITERIA

A. General

The Drainage Criteria included in this section are for the purpose of providing a set of guidelines for planning and designing storm drainage facilities in the City of Fredericksburg, Texas and within its extraterritorial jurisdiction. These criteria will be used by the Department of Public Works, other City Departments, consulting engineers employed by the City, and engineers for private developments in the City.

B. Rational Method for Peak Storm Flows

The formula to be used for calculating peak storm flows for drainage areas less than 200 acres shall be the Rational Method, in which:

$$Q = CIA, \text{ where}$$

Q - is the peak storm flow at a given point in cubic feet per second (cfs)

C - is the runoff coefficient that is equal to the ratio that the peak rate of runoff bears to the average rate (intensity)of rainfall;

I - is the average intensity of rainfall in inches per hour for a storm duration equal to the time of travel for run off to flow from the farthest point of the drainage area to the design point in question;

A - is the drainage area tributary to the design point, in acres.

Note: For drainage areas greater than 200 acres, peak storm flows shall be determined based on a flow routing analysis using detailed hydrographs such as the Soil Conservation Service hydrologic methods that are available in such computer programs as TR-20, HEC-1, etc.

C. Runoff Coefficient

The runoff coefficient (C) shall consider the slope of the terrain, the character of the land use, the length of overland flow and the imperviousness of the drainage area and shall be determined based on ultimate land development. The run-off coefficient for the appropriate land used shall be as follows:

Commercial	0.90
Industrial	0.70
Single Family Residential	0.55
Multi-Family	0.75
Parks and Open Space	0.35
Schools, Churches, etc.	0.75

D. Rainfall Intensity-Frequency

The rainfall intensity-frequency curves which are shown on PLATE 1 are plotted from data by the State Department of Highways and Public Transportation Hydraulic Manual, District 14.

The intensity (I) in the formula Q = CIA, is determined from the curves by arriving at a time of concentration for the subject drainage area and adapting a storm frequency upon which to base the design of drainage improvements.

1. Time of Concentration

The time of concentration, which is the longest time of travel for runoff to flow from any point of the subject drainage area to the design point, consists of the time required for runoff to flow overland plus the time required to flow in a street gutter, storm drain, open channel or other conveyance facility. A minimum time of concentration of fifteen (15) minutes shall be used for Single Family Residential, Parks and Open Space areas and a minimum time of concentration of ten (10) minutes shall be used for Commercial, Industrial, Multi-Family Residential, School and Church areas. A nomograph, shown on PLATE 2, is attached for estimating the time of concentration.

2. Storm Frequency

Required design storm frequencies for storm drainage improvements in the City of Fredericksburg are shown in the following table.

Type of Facility	Design Frequency (years)
*Storm Sewer Systems	25
*Culverts, Bridges, Channels, and Creeks	100

* The drainage system shall be designed to carry those flows greater than the 25-year frequency up to and including a 100-year frequency within defined rights-of-way or drainage easements.

E. Area

The drainage area used in determining peak storm flows shall be calculated by subdividing a map into the watersheds within the basin contributing storm water runoff to the system. Areas shall be determined by planimetering or digitizing.

F. Spread of Water

During the design storm, the quantity of storm water that is allowed to collect in the streets before being intercepted by a storm drainage system is referred to as the "spread of water". In determining the limitations for carrying storm water in the street, the ultimate development of the street shall be considered. The use of the street for carrying storm water shall be limited to the following:

SPREAD OF WATER

Major thoroughfares (divided)	- One traffic lane on each side to remain clear.
Thoroughfares (not divided)	- Two traffic lanes to remain clear.
Collector streets (36' - 40')	- One traffic lane to remain. clear.
Residential streets (24' - 36')	- Six-inch depth of flow at curb

PLATE 3, Capacity of Triangular Gutters, applies to all street widths having a straight cross slope varying from one-eighth inch per foot to one-half inch per foot which are the minimum and maximum allowable street cross slopes.

PLATE 4 and PLATE 5, Capacity of Parabolic Gutters, applies to streets with parabolic crowns.

G. Storm Sewer Design

Storm water in excess of that allowed to collect in the streets shall be intercepted in inlets and conveyed in a storm sewer system. Storm sewer capacity shall be calculated by the Mannings formula --

$$Q = AV, \text{ and}$$
$$Q = \frac{1.486}{n} AR^{2/3}S^{1/2}$$

where

Q is the discharge in cubic feet per second;
A is the cross-sectional area of the conduit in square feet;
V is the velocity of flow in the conduit in feet per second;
R is the hydraulic radius in feet, which is the area of flow divided by the wetted perimeter ($R = \frac{A}{W}$);
S is the slope of the hydraulic gradient in feet per foot;
n is the coefficient of roughness.

The recommended roughness coefficients to use in the design of a storm sewer system are as follows:

Type of Storm Drain	Manning's Coefficient
Concrete Box Culvert	0.015
New Concrete Pipe	0.013
Standard, unpaved, with or without bituminous coating corrugated metal pipe	0.024
Paved invert, 25% of periphery paved corrugated metal pipe	0.021
Paved invert, 50% of periphery paved corrugated metal pipe	0.018
100% paved and bituminous coated corrugated metal pipe	0.013

In the design of the storm sewer system, the elevation of the hydraulic gradient of the storm sewer shall be a minimum of 0.5 feet below the elevation of the adjacent street gutter.

Storm sewer pipe sizes shall be so selected that the average velocity in the pipe will not exceed 15 feet per second nor less than 3 feet per second. The minimum grade recommended for storm sewer pipe is 0.30%.

Closed storm sewer systems shall be installed in all areas where the quantity of storm runoff is 300 cubic feet per second, or less. A closed storm sewer system may be constructed when the quantity exceeds 300 cfs, at the discretion of the City.

Hydraulic gradients shall be calculated and lines drawn for each storm sewer.

A Pipe Flow Chart for circular conduits 15 inches in diameter through 144 inches in diameter is shown on PLATE 6.

H. Head Losses

Head losses for wyes and pipe size changes will be calculated by the formulas:

$$\text{Where } \frac{(V_2^2)}{2g} - \frac{(V_1^2)}{2g} = HL$$

and, V_1 is upstream velocity
V_2 is downstream velocity

Head losses and gains for manholes, bends and junction boxes will be calculated as shown on PLATE 7.

The basic equation for most cases, where there is both upstream and downstream velocity, takes the form as set forth below with the various conditions of the coefficient "Kj" shown on PLATE 7.

$$Hj = \left(\frac{V_2^2}{2g}\right) - Kj \left(\frac{V_1^2}{2g}\right)$$

Hj = Junction or structure head loss in feet;
V_1 = Velocity in upstream pipe in fps;
V_2 = Velocity in downstream pipe in fps.
Kj = Junction or structure coefficient of loss;
g = 32.2 fps

In the case where the inlet is at the very beginning of a line or the line is laid with bends, the equation becomes the following without any velocity of approach.

$$Hj = Kj \left(\frac{V_2^2}{2g}\right)$$

PLATE 8, Hydraulic Computations for Storm Drains, is attached for use in the design of a storm sewer system.

I. Open Channel Design

Storm water runoff in excess of that allowed to collect and be conveyed in the streets in developed areas and runoff in undeveloped areas may be carried in grass lined, concrete lined or weathered rock open channels (not in the street right-of -way). Earthen, non-vegetated or unlined open channels are not acceptable. Open channel capacity shall be calculated by the Manning's Formula, and roughness coefficients shall be as follows:

Type of Lining	Roughness Coefficient "n"	Maximum Permissible Mean Velocity
Earth (Bermuda Grass)	0.035	6 ft. per sec.
Concrete Lined	0.015	15 ft. per sec.
Weathered Rock	0.030	10 ft. per sec.

Open channels shall be constructed with a trapezoidal cross-section and shall have side slopes no steeper than 3:1 when grass lined and 1.5:1 when lined with concrete.

A right-of -way for all channels of sufficient width shall be dedicated to provide for excavation of the open channel of proper width, plus ten feet on each side to permit ingress and egress for maintenance. Additional width may be considered if sanitary sewer mains are proposed to follow the channel alignment.

J. Culvert Design

At locations of stream or open channel crossings with proposed roadway improvements, it is sometimes necessary to receive and transport storm water under the roadway in culverts. The quantity of flow shall be determined by the appropriate method, and the friction loss through of the culvert shall be calculated by Manning's Formula.

Design of culverts shall include the determination of upstream backwater conditions as well as downstream velocities and flooding conditions. Consideration shall be given to the discharge velocity from culverts, and the limitations specified on PLATE 9 are allowed.

K. Stormwater Detention Pond Design

The basic concept underlying the use of stormwater detention ponds (SDP) involves providing temporary storage of stormwater runoff so that peak rates of runoff can be reduced. Runoff is released from storage at a controlled rate which cannot exceed the capacities of the existing downstream drainage systems or the predeveloped peak runoff rate of the site, whichever is less.

The solid lined hydrograph shown in Figure K-1 represents a storm runoff event without stormwater detention, while the dashed line hydrograph depicts the same event with stormwater detention. The peak flow of the undetained hydrograph could exceed the capacity of the downstream conveyance system and thereby cause surcharging and flooding problems. With the introduction of the SDP facility, the solid-lined hydrograph is spread over a longer time period and its peak is reduced. The area between the two (2) curves to the left o f their intersection represents the volume of runoff that is temporarily stored or detained in the SDP facility.

Stormwater detention ponds may be of two (2) basic types: On-site and Regional. In general, on-site ponds are those which are located off-channel and provide stormwater detention for a particular project of development. Regional ponds are designed to provide stormwater detention in conjunction with other improvements on a watershed-wide basis. The performance and safety criteria in this section apply to all ponds which provide management of peak rates of stormwater runoff, regardless of type.

PERFORMANCE CRITERIA FOR ON-SITE SDP's

1. On-site SDP's are further classified as either small or large, as follows:

ON-SITE SDP POND CLASS	DRAINAGE AREA
Small	<25 acres
Large	25-64 acres

For design purposes, any pond with a drainage area larger that 64 acres shall be classified as a regional pond.

2. On-site SDP ponds shall be designed to reduce post-development peak rate of discharge to existing pre-development peak rates of discharge for the 2-, 10-, 25- and 100-year storm

events at each point of discharge from the project or development site. In addition, the capacity of the existing downstream systems must be considered in determining the need for managing the 100-year storm event. For the post-development hydrologic analysis, any off-site areas which drain to the pond shall be assumed to remain in the existing developed condition.

3. The Modified Rational Method (MRM) may be used for the design of small on-site ponds only. The maximum contributing drainage area to a pond designed with the MRM is 10 acres when using this equation.

PERFORMANCE CRITERIA FOR REGIONAL SDP's

1. Regional SDP's are classified as small or large, based on the following criteria:

REGIONAL POND CLASS	**IMPOUNDED VOLUME, AC-FT**
Small	0-150
Large	>150 acres

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

2. Performance criteria for regional detention ponds shall be determined by the City on a project-by-project basis. The determination shall be based on a preliminary engineering study prepared by the project engineer.

SAFETY CRITERIA FOR SDP's

All ponds shall meet or exceed all specified safety criteria. Use of these criteria shall in no way relieve the engineer of the responsibility for the adequacy and safety of all aspects of the design of the SDP.

1. The spillway, embankment, and appurtenant structures shall be designed to safely pass the design storm hydrograph with the freeboard shown in the table below. All contributing drainage areas, including on-site and off-site area, shall be assumed to be fully developed. Any orifice with a dimension smaller than or equal to twelve (12) inches shall be assumed to be fully blocked.

DETENTION POND CLASS		**DESIGN STORM EVENT**	**FREEBOARD TO TOP OF EMBANKMENT, FT.**
On-site:	Small	100 year	0
	Large	100 year	1.0
Regional:	Small	100 Year	2.0
	Large	100 year	*

*Design storm event and required freeboard for large regional ponds shall be determined in accordance with Chapter 299 of the Texas Administrative Code (Dam Safety Rules of the Texas Natural Resource Conservation Commission).

2. All SDP's (except small on-site ponds) shall be designed using a hydrograph routing methodology. The Modified Rational Method (MRM) may be used only for contributing drainage areas less than ten (10) acres.

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

TOTAL HEIGHT OF EMBANKMENT, FT.	**MINIMUM TOP WIDTH, FT.**
0-6	4
6-10	6
10-15	8
15-20	10
20-25	12
25-35	15

4. The constructed height of an earthen embankment shall be equal to the design height plus the amount necessary to ensure that the design height will be maintained once all settlement has taken place.

This amount shall in no case be less than five (5%) percent of the total fill height. All earthen embankments shall be compacted to 95% of maximum density.

5. Earthen embankment side slopes shall be no steeper than three (3) horizontal to one (1) vertical. Slopes must be designed to resist erosion, to be stable in all conditions and to be easily maintained. Earthen side slopes for regional facilities shall be designed on the basis of appropriate geotechnical analyses.

6. Detailed hydraulic design calculation shall be provided for all SDP's. Stage-discharge rating data shall be presented in tabular form with all discharge components, such as orifice, weir, and outlet conduit flows, clearly indicated. A stage-storage table shall also be provided.

7. When designing SPD's in a series (i.e., when the discharge of one pond becomes the inflow to another), the engineer must submit a hydrologic analysis which demonstrates the system's adequacy. This analysis must incorporate the development of hydrographs for all inflow and outflow components.

8. No outlet structures from SDP's, parking detention, or other concentrating structures shall be designed to discharge concentrated flow directly onto arterial or collector streets. Such discharges shall be conveyed by a closed conduit to the nearest existing storm sewer. If there is no existing storm sewer within 300 feet, the outlet design shall provide for a change in the discharge pattern from concentrated flow back to sheet flow, following as near as possible the direction of the gutter.

9. Stormwater runoff may be detained within parking lots. However, the engineer should be aware of the inconvenience to both pedestrians and traffic. The location of ponding areas in a parking lot should be planned so that this condition is minimized. Stormwater ponding

depths (for the 100-year storm) in parking lots are limited to an average of eight (8") inches with a maximum of twelve (12") inches.

10. All pipes discharging into a public storm sewer system shall have a minimum diameter of twelve (12"). In all cases, ease of maintenance and/or repair must be assured.

11. All concentrated flows into a SDP shall be collected and conveyed into the pond in such a way as to prevent erosion of the side slopes. All outfalls into the pond shall be designed to be stable and non-erosive.

OUTLET STRUCTURE DESIGN

There are two (2) basic types of outlet control structures: those incorporating orifice flow and those incorporating weir flow. Weir flow is additionally broken down into two (2) categories: rectangular and V-notch. In each type, the bottom edge of the weir over which the water flows is called the crest. Sharp-crested and broad-crested weirs are the most common types.

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

1. <u>Rectangular Weir Flow Equation</u> (See Figure K-2)

$Q = CLH^{3/2}$ (Eq. K-1)

where
Q = Weir discharge, cubic feet per second
C = Weir coefficient
L = Horizontal length, feet
H = Head on weir, feet

2. <u>V-notch Weir Flow Equation</u> (See Figure K-2)

$Q = C_v \tan(O/2)H^{2.5}$ (Eq. K-2)

where
Q = Weir Flow, cubic feet per second
C_v = Weir Coefficient
O = Angle of the Weir notch at the apex (degrees)
H = Head on Weir, feet

3. <u>Orifice Flow Equation</u> (See Figure K-2)

$Q = C_O A (2gH)^{0.5}$

Where
Q = Orifice Flow, cubic feet per second
C_O = Orifice Coefficient (use 0.6)

A = Orifice Area, square feet
g = Gravitation constant, 32.2 feet/sec^2
H = Head on orifice measured from centerline, feet

Analytical methods and equations for other types of structures shall be approved by the City prior to use.

DETENTION POND STORAGE DETERMINATION

The overall procedure to be followed in determining the proper storage capacity for a SDP is summarized on the chart in PLATE 10. The method to be used for determining detention pond volume requirements is governed initially by the size of the total contributing drainage area to the pond.

For contributing areas up to ten (10) acres, the Modified Rational Method (MRM) may be used. For contributing areas greater than ten (10) acres, a flow routing analysis using detailed hydrographs must be applied. The Soil Conservation Service hydrologic methods (available in TR-20, HEC-1) can be used. The engineer may use other methods but must have their acceptability approved by the City. These methods may also be used for the smaller areas. The most noticeable difference between the two (2) methods is that the MRM is essentially an approximation of the dynamic routing procedures used in the flow routing analysis using detailed hydrographs. The MRM is also limited in application by the restrictions and assumptions associated with the Rational Method. The following subsections give a more detailed description of these two (2) storage determination methods.

MODIFIED RATIONAL METHOD (MRM)

The MRM is derived from the Rational Method previously described in Section B. This procedure determines the critical storm duration which produces the largest pond storage requirement with respect to the release rate values established for the pond. These release rates can be derived either from pre-development conditions or from specified allowable release rate criteria. In addition to the general criteria stated previously, the following MRM criteria must also be followed:

a. Maximum contributing area to the pond is ten (10) acres.
b. All off-site flows must be diverted around the pond.
c. Pond outflows calculated by the MRM may not be used as inflows to another pond (i.e., cascaded ponds cannot be analyzed by the MRM).
d. If the critical storm duration produces a peak flow less than the allowable peak release rate, the storm duration to be used in all subsequent calculation shall be that which produces a peak flow equal to the allowable peak release rate.
e. On-site flows which do not enter a pond are referred to as “bypass” flow. All such flows must be subtracted from the allowable peak flow release rate for the development.
f. Only single-storm events can be analyzed with the MRM.

The MRM is based on the same assumptions as the Rational Method. The most significant assumption is that the period of rainfall intensity averaging is equal to the duration of the storm. This means that the rainfall and corresponding runoff which occurs either before or after that averaging period are not considered in the storage calculations. Comparison of storage volumes calculated by the MRM with volumes calculated by the Hydrograph Method suggests that significant underestimation of required storage volumes may result for areas larger than a few acres. This appears to be a direct result of the assumption just stated. Therefore, a volume adjustment factor is always applied to the storage calculated by this procedure.

The MRM also assumes that the outflow hydrograph can be approximated by either a triangular or trapezoidal shape. This assumption is equivalent to assuming that the effective contributing drainage area increases linearly with time. In other words, there is a linear area-time relationship for the contributing drainage area. If the actual relationship differs significantly from this assumption (e.g., a preponderance of either quickly-arriving or greatly-delayed flows), the pond could be significantly oversized or undersized.

The MRM was originally designed as a graphical procedure and is still widely used and accepted in that format for storage volume calculations. An equation method which duplicates the procedures of the graphical method also is available. Both procedures are described in this section.

1. MRM - Graphical Method

Step 1 - Select the design storm frequency.

Step 2 - Determine the allowable release rate for the pond and plot as a horizontal line, as shown in Figure K-3.

Step 3 - Determine the required hydrologic data for the design drainage area, including the proposed-condition contributing area, runoff coefficient and time of concentration.

Step 4 - Draw a vertical line at the proposed-condition time of concentration, as shown in Figure K-3.

Step 5 - Select storm duration based on five-minute increments added to the proposed-condition time of concentration. A value less than the time of concentration should not be chosen, and any calculated peak flow should be equal to or greater than the allowable release rate. The maximum duration can be determined by combining the Rational Method equation for the proposed conditions with the equation relating rainfall intensity t storm duration and return period, as follows:

$$Q_p = C_p i_D A_p \qquad \text{(Eq. K-4)}$$

where

Q_p = Peak flow rate for proposed conditions and the specific storm duration, cfs

C_p = Runoff coefficient for proposed conditions and the specific storm duration

i_D = Average rainfall intensity for the specific storm duration and return period, in/hr

A_p = Contributing drainage area for proposed conditions, acres

and

$i_D = a/[(t_D+b)^c]$ (Eq. K-5)

where

t_D = Specific storm duration, minutes

and a, b and c are coefficients specified in the TxDOT Hydraulic Manual for Gillespie County as follows:

STORM FREQUENCY	**a**	**b**	**c**
2 - year	49	8.5	0.787
5 - year	60	8.1	0.766
10 - year	71	8.1	0.767
25 - year	82	8.1	0.765
50 - year	94	8.1	0.764
100 - year	104	8.5	0.765

Combining Eq. K-4 and Eq. K-5 produces the following equation for the maximum storm duration:

$t_{Dmax} = (aC_pA_p/Q_p)^{(1/c)} - b$ (Eq. K-6)

Step 6 - For each storm duration selected in Step 5, calculate the peak flow rate with the Rational Method equation and construct the corresponding hydrograph as shown in Figure K-3.

Step 7 - For each storm duration, draw a line connecting the origin with the intersection point of 1) the allowable release rate line and 2) the recession limb of the hydrograph for that storm duration. The indicated storage for a specific storm duration is represented by the difference in volume between the total inflow hydrograph and the outflow hydrograph produced by the line just drawn and the recession limb of the inflow hydrograph. This is depicted by the cross-hatched area in Figure K-3. Tabulate required storages for all storm durations, and identify the largest, which corresponds to the critical storm duration.

Step 8 - Having determined the required storage volume, the engineer should proceed to the actual pond design.

2. MRM - Equation Method

Due to the number of repetitive calculatins and hydrograph which must be produced to determine the critical storm duration by the MRM, the Equation Method was developed. Since the calculated storage volume is directly related to the critical storm duration, differential calculus can be used to determine those values. In order to accomplish this, the peak flow rates for proposed conditions and various storm durations must be expressed as a function of storm duration. As in the Graphical Method, the Rational Method equation and the equation relating rainfall intensity to storm duration and return period (Equation K-5) can be combined to find those values, as follows:

$$Q_p = C_p i_D A_p \qquad \text{(Eq. K-8)}$$
$$= C_p(a/[(t_D+b)^c])\, A_p \qquad \text{(Eq. K-9)}$$

As shown in Figure K-3, the volume calculated for a specific storm duration can be arithmetically calculated as follows:

Inflow Hydrograph Volume

$$V_i = 60(1/2)(Q_p)[(t_D-t_c)+(t_D+t_c)] \qquad \text{(Eq. K-10)}$$

where,

V_i = Inflow Hydrograph volume, ft^3

Q_p = Peak flow rate for proposed conditions and the specific storm duration, cfs

t_D = Time of concentration for the specific storm duration, minutes

t_c = Time of concentration for proposed conditions, minutes

Outflow Hydrograph Volume

$$V_o = 60(1/2)(Q_A)(t_D+t_c) \qquad \text{(Eq. K-11)}$$

where,

V_o = Outflow hydrograph volume, ft^3

Q_A = Allowable peak flow release rate, cfs

t_D = Time of concentration for the specific storm duration, minutes

t_c = Time of concentration for proposed conditions, minutes

Calculated Storage Volume (before adjustment)

$$S_D = V_i - V_o \qquad \text{(Eq. K-12)}$$
$$= 60(1/2)(Q_p)[(t_D-t_c)+(t_D+t_c)] - 60(1/2)(Q_A)(t_D+t_c)$$
$$= 600 Q_p t_D - 30 Q_A(t_D+t_c) \qquad \text{(Eq. K-13)}$$

Combining Equation K-10 and K-11 with Equation K-12 results in an equation for the calculated storage volume as a function of storm duration and the parameters for the proposed condition, as follows:

$$S_D = 60 C_p\{a/[(t_D+b)^c]\}\, A_p\, t_D - 30 Q_A(t_D+t_c) \qquad \text{(Eq. K-14)}$$

where,

S_D = Calculated storage volume, ft^3

Differentiating equation K-14 with respect to t_D and setting the result equal to zero produces an equation in terms of the specified variables, as follows:

$$[(t_D+b)^{c+1}]/[a(t_D+b-ct_D)] = 2\, C_p A_p/Q_A \qquad \text{(Eq. K-15)}$$

Although this equation must be solved iteratively for the critical storm duration, the procedure is simple and converges to a sufficiently accurate answer very quickly. The procedure is as follows:

Step 1 - Pick any initial value for t_D. A nominal value of 30 minutes usually results in satisfactory convergence.

Step 2 - Calculate a value for the left-hand side of the equation, using the current assumed value for t_D.

Step 3 - Divide the right-hand side of the equation by the value calculated in Step 2 to obtain an adjustment factor to apply to t_D.

Step 4 - If the adjustment factor is within 0.01 of unity, multiply the current assumed value for t_D by the adjustment factor from Step 3 to obtain a final value for t_D. This is the critical storm duration value. Go directly to Step 6.

Step 5 - If the adjustment factor is not within 0.01 of unity, multiply the current assumed value for t_D by the adjustment factor from Step 3 to obtain a new assumed value for t_D. Go back to Step 2 and repeat the procedure as often as necessary.

Step 6 - Solve for the calculated storage volume by using Equation K-14 and the final value for t_D from Step 4.

Step 7 - Having determined the required storage volume, the engineer should proceed to the actual pond design.

DETENTION POND MAINTENANCE AND EQUIPMENT ACCESS REQUIREMENTS

1. Silt shall be removed and the pond returned to original lines and grades when standing water conditions occur or the pond storage volume is reduced by more than 10%.

2. To limit erosion, no unvegetated area shall exceed 10 sq. ft in extent.

3. Accumulated paper, trash and debris shall be removed every six (6) months or as necessary to maintain proper operation.

4. Ponds shall be mowed annually between the months of June and September.

5. Corrective maintenance is required any time a pond does not drain completely within 60 hours of cessation of inflow (i.e., no standing water is allowed).

6. Structural integrity of pond embankments shall be maintained at all times.

CULVERT DISCHARGE - VELOCITY DETERMINATIONS

Culvert Discharging On To	Maximum Allowable Velocity (fps)
Sod Earth	6.0
Paved or riprap apron	15.0
Shale	10.0
Rock	15.0

Culverts shall not be designed to discharged into earthen, unprotected (ie: sod or lined) channels. Generally, all culverts shall be designed with a free outfall, and the following head losses shall govern the design of the culvert.

1. Friction Head Loss

$h_f = s_f L$; where,

s_f = Friction slope gradient of the culvert in feet per foot; and

L = Length of culvert in feet

2. Head Loss in Feet Due to Change in Velocity

$h_v = \frac{(V_2^2)}{2g} - \frac{(V_1^2)}{2g}$ where,

V_2 = Velocity in culvert in feet per second;

V_1 = Velocity in channel upstream from culvert in feet per second;

g = Acceleration due to gravity (32.2 feet per second).

3. Head Loss at Upstream Entrance to Culvert Due to Entrance and Change in Section

$h_e = Ce\frac{(V_2^2)}{2g}$ where V_2 is equal to the velocity in the culvert and Ce is the entrance loss coefficient based on the TxDOT Hydraulic Manual.