SECTION 7 - CULVERTS

7.1.0 GENERAL

The function of a drainage culvert is to pass the 100-year design storm flow without causing excessive backwater or overtopping of the structure and without creating excessive downstream velocities. The designer shall keep energy losses and discharge velocities within allowable limits when selecting a structure that will meet these requirements. The design storm flow shall be determined by the hydrologic methods as set forth in Section 2 of this manual. The system shall accommodate the runoff from ~~a~~ 25-year and 100-year frequency storms meeting the limitations for overflows at bridges and culverts set forth in Section 1.2.4.C and 1.2.4.D. The ~~guidelines~~ rules for manhole spacing described in Section 5.6.0 also apply to culverts. The Federal Highway Administration is a good source of information for the design of culverts and bridges, including the HY-8 software, Hydraulic Design Series publications, and Hydraulic Engineering Circulars. The Army Corps of Engineers' HEC-RAS software is also a good tool for analyzing systems that contain culverts or bridges.

7.2.0 CULVERT HEADWALLS & ENDWALLS

7.2.1 General

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

7.2.2 Conditions at Entrance

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

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

Where,

h_e = Entrance headloss ~~head loss~~, feet
V_2 = Velocity of flow in culvert, ft/sec
V_1 = Velocity of flow approaching culvert, ft/sec
K_e = Entrance loss coefficient as shown in Table 7-1
g = Acceleration due to gravity

7.2.3 Type Of Headwall

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

Table 7-1
Values of Culvert Entrance Loss Coefficients

Type of Entrance	Entrance Coefficient, K_e
Pipe	
Headwall (no wingwalls)	
Grooved edge	0.20
Rounded edge (0.15D radius)	0.15
Rounded edge (0.25D radius)	0.10
Square edge (cut concrete and CMP)	0.40
Headwall with 45° Wingwalls	
Grooved edge	0.20
Square edge	0.35
Headwall with Parallel Wingwalls Spaced 1.25D apart	
Grooved edge	0.30
Square edge	0.40
Beveled edge	0.25
Projecting Entrance (no headwall or wingwalls)	
Grooved edge (RCP)	0.25
Square edge (RCP)	0.50
Sharp edge, thin walls (CMP)	0.90
Sloping Entrance (no headwall or wingwalls)	
Mitered to conform to slope	0.70
Flared-end section	0.50
Box, Reinforced Concrete	
Headwall Parallel to Embankment (no wingwalls)	
Square edge on sides of opening	0.50
Rounded on 3 edges to radius of 1/12 barrel dimension	0.20
Wingwalls at 30° to 75° to barrel axis	
Square edged at crown	0.40
Crown edge rounded to radius of 1/12 barrel dimension	0.20
Wingwalls at 10° to 30° to barrel	
Square edged at crown	0.50
Wingwalls parallel (extension of culvert walls)	
Square edged at crown	0.70
RCP: Reinforced Concrete Pipe CMP: Corrugated Metal Pipe NOTE: The entrance loss coefficients are used to evaluate the culvert or ~~sewer~~ drain capacity operating under outlet control.	

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

A. **Parallel Headwall.**

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

2. Backwater pools are permitted.

B. **Flared Headwall.**

1. Approach velocities are between six (6) and ten (10) feet per second.

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

The wings of flared walls should be located with respect to the direction of the approaching flow, not the culvert axis, as in Figure 7-1 in Appendix ~~E~~ D of this manual.

C. **Warped Headwall.**

1. Approach velocities are between eight (8) and 20 feet per second.

Warped headwalls are effective with aprons to accelerate flow through the culvert.

7.2.4 Debris Fins.

For conditions where more than one (1) box culvert is required, the upstream face of the structure shall incorporate debris deflector fins to prevent debris buildup. For multiple-pipe situations installations of debris fins may be used but are not required.

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

A debris fin is constructed to the height of the culvert. A fin length of one and one-half (1.5) times the height of the box culvert is required. Since the debris fins are subject to the same erosive forces as bridge piers, care must be taken in the design of the footing. A toewall at the upstream end of the debris fin and the apron is recommended. Figure 7-2 in Appendix ~~E~~ D of this manual depicts the conceptual design for debris deflector fins. It should be noted that alternate types of wingwalls can be used other than the parallel shown in Figure 7-2 in Appendix ~~E~~ D of this manual.

7.2.5 Trashracks & Safety End Treatments.

If trashracks or safety end treatments are to be used, appropriate clogging factors must be applied. TxDOT provides some guidance for the selection of clogging factors. "Design of Small Dams" by US Bureau of Reclamation (1987) suggests a trashrack clogging factor of 50%.

7.3.0 CULVERT DISCHARGE VELOCITIES

Placement of a culvert crossing in a channel produces rapid changes in flow regime that can present erosion hazards both upstream and downstream of the culvert location. Design of the culvert should incorporate features that lessen these impacts to the receiving channel to the greatest extent possible. The following concepts should be considered in the design process. These erosion hazards are discussed in greater detail in the Federal Highway Administration (FHWA), Hydraulic Engineering Circular No. 14, "Hydraulic Design of Energy Dissipaters for Culverts and Channels":

- Whenever possible, the culvert axis should match the natural channel alignment upstream and downstream. Matching the channel alignment with the culvert axis prevents a bend in the channel that would be subject to erosion.

- Depressed entrances should be avoided. If capacity or depth of cover forces the use of a depressed entrance, the upstream apron should be designed to prevent progressive degradation of the upstream channel. Additionally, the potential for deposition should be considered.

- The local conditions within the channel reach should be evaluated to determine if there is degradation present. In a degrading channel, headcuts can migrate upstream and compromise the integrity of the culvert. If channel degradation is anticipated, the design should accommodate for future erosion.

- At a culvert outlet, exit velocities should be minimized to the greatest extent practical. Channel erodibility and local scour potential should be evaluated and taken into account. Due to the dynamic nature of alluvial channels, a flexible armoring system is preferred in streams subject to erosion. The City of Austin Standard 508S-20 (Stormdrain Outfall Protection, Culvert Under Roadway/Inline) may be used for protection of the receiving channel from erosive forces. Where degradation is expected, the volume of riprap should be increased to account for loss of base level downstream of the culvert outlet. The rock riprap used in this standard detail must be designed by the engineer for the specific hydraulic conditions present based on ECM 1.4.6(D). Where energy dissipation is needed at a culvert outlet, a rock riprap basin is preferred over rigid structures (see HEC-14 for design guidelines).

- If multiple boxes or culverts are necessary, different flowline elevations for each structure should be evaluated. The culverts in the center of the channel should be lower to match with the existing natural channel with the outlying culverts raised up to more closely coincide with the natural channel “terrace” elevation. This will help to minimize sedimentation in the outlying culverts, help preserve the integrity of the channel system, and reduce maintenance costs. A culvert may be depressed below the channel flowline, such that a natural channel bottom is maintained, as long as n-values and cross-sectional area are adjusted appropriately when modeling conveyance.

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

~~**Velocity**~~	~~Outlet Protection~~
~~Below six (6) ft/sec~~	~~Riprap protection. (Four (4) inch minimum thickness) or alternate approved material.~~
~~Above six (6) ft/sec~~	~~Structurally reinforced apron, six (6) inch minimum thickness with toe wall.~~

Riprap armoring is preferred; however, if site conditions necessitate a concrete apron, the ~~The~~ minimum apron length which provides transition from a culvert outlet to an open channel shall be calculated from the following equation:

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

Where,

L = Apron length, feet
V = culvert discharge velocity, ft/sec
D = height of box culvert or diameter of pipe culvert, feet

7.4.0 SELECTION OF CULVERT SIZE AND FLOW CLASSIFICATION

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

7.4.1 Culvert Hydraulics

A. **Inlet Control Condition.**

Inlet control for culverts may occur in two (2) ways.

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

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

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

B. **Outlet Control Condition.**

There are three (3) types of outlet control culvert flow conditions:

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

2. The headwater submerges the culvert opening, the culvert outlet is not submerged by the tailwater (Condition B or C, Figure 7-3 in Appendix ~~E~~ D of this manual).

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

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

Please refer to the publications of the Federal Highway Administration for design calculations.

~~Bernoulli's Equation is:~~

$$\cancel{d_1 + V_1^2/2g + LS_0 = TW + h_e + h_f + h_v \qquad \text{(Eq. 7-3)}}$$

~~The sum of the first two (2) terms on the left-hand side of Equation 7-3 is equal to the headwater (HW). That is:~~

~~$HW = d_1 + V_1^2/2g$ (Eq. 7-4)~~

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

~~$HW + LS_0 - TW = h_e + h_f + h_v$ (Eq. 7-5)~~

~~From Figure 7-4 (in Appendix E of this manual),~~

~~$HW + LS_0 = H_L + TW$~~

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

~~$H_L = HW + LS_0 - TW$ (Eq. 7-6)~~

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

~~$H_L = h_e + h_f + h_v$ (Eq. 7-7)~~

~~in which $h_v = V^2/2g$. (Eq. 7-8)~~

~~For inlet losses, the governing equation is Equation 7-1:~~

~~$h_e = K_e (V_2^2 - V_1^2)/2g$~~

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

~~$h_e = K_e V^2/2g$ (Eq. 7-9)~~

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

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

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

~~Where n = Manning's coefficient~~
~~L = Length of culvert, feet~~
~~R = Hydraulic radius, feet~~
~~V = Velocity of flow in the culvert, ft/sec~~

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

~~$H = (K_e + 1 + 29n^2L/R^{1.33})V^2/2g$ (Eq. 7-11)~~

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

C. **Depths of Tailwater and Headwater.**

In culverts flowing with outlet control, tailwater is an important factor in computing both the headwater

depth and the hydraulic capacity of a culvert. Thus, in many culvert designs, it becomes necessary to determine tailwater depth in the outlet channel.

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

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

Please refer to the publications of the Federal Highway Administration for ~~design calculations~~ examples of required designs.

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

~~$HW = H + h_0 - LS_0$ (Eq. 7-12)~~

~~Where,~~

~~HW = vertical distance from flow line at the entrance to the pool surface, feet~~
~~H = head loss, feet (use appropriate nomograph)~~
~~h_0 = vertical distance from flow line at the outlet to the hydraulic grade line, feet~~
~~(In this case h_0 equals TW, measured in feet above the flow line.)~~
~~S_0 = slope of barrel, ft/ft~~
~~L = culvert length, feet~~

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

~~$h_0 = (d_c + D)/2$ or TW, (Eq. 7-13)~~

~~Where,~~

~~d_c = critical depth in feet (d_c cannot exceed D)~~
~~D = height of culvert opening in feet whichever value is greater.~~

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

A series of nomographs for various culvert materials and shapes have been developed by the Federal Highway Administration and the various pipe manufacturers. The nomographs ~~presented herein~~ include those for inlet control conditions**,** ~~(Figures 7-5 to 7-10 in Appendix E of this manual) and~~ outlet control conditions ~~(Figures 7-11 to 7-17 in Appendix E of this manual). The~~ and the critical depth for pipes of different shapes ~~are shown in Figures 7-18 to 7-22 in Appendix E of this manual~~.

7.4.2 Design Procedures

The Federal Highway Administration has published guidance for the hydraulic design of culverts. Their guidance includes design methodology, check lists, design charts, and tables which ~~can~~ shall be used to perform culvert designs that meet the City's performance criteria.

~~The State Highway Department's THYSYS program can be used for culvert design in addition to help~~

~~calculate the culvert size and related computations. Design procedures are as follows:~~

~~A. Step 1: List design data.~~

~~1. Design discharge Q, cfs~~

~~2. Approximate length L of culvert, feet~~

~~3. Slope of culvert, ft/ft~~

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

~~5. Allowable flow velocities in natural stream~~

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

~~B. Step 2: Determine the first trial size culvert.~~

~~Since the procedure given is one of trial and error, the initial trial size can be determined by one of the following ways:~~

~~1. Make an arbitrary selection.~~

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

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

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

~~1. Assuming INLET CONTROL~~

~~- Using the trial size from Step 2, find the headwater depth HW by use of the appropriate inlet control nomograph (Figures 7-5 to 7-10 in Appendix E of this manual). HW in this case is found by multiplying HW/D obtained from the nomograph by the height of the culvert (D). Tailwater (TW) conditions are neglected in this determination.~~

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

~~2. Assuming OUTLET CONTROL~~

~~- Determine the depth of tailwater (TW), in feet, for the design flood condition at the outlet.~~

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

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

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

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

satisfactory as determined under Step 3-1.

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

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

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

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

7.4.3 Instructions For Using Nomographs

A. **Inlet-Control Nomographs (Figures 7-5 to 7-10 in Appendix E of this manual).**

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

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

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

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

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

a. Compute HW/D for given conditions.

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

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

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

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

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

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

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

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

B. **Outlet-Control Nomographs (Figures 7-11 to 7-17 in Appendix E of this manual).**

Outlet control nomographs can be used to solve Equation 7-11 for head H when the culvert barrel flows

~~full for its entire length. They are also used to determine H for some part-full flow conditions with outlet control. These nomographs do not give a complete solution for HW, since they give only H in the equation $HW = H + h_0 - LS_0$.~~

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

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

~~b. Begin nomograph solution by locating starting point on length scale. To locate the proper starting point on the length scales, follow three (3) steps:~~

~~Step 1: If the n value of the nomograph corresponds to that of the culvert being used, select the length curve for the proper K_e and locate the starting point at the given culvert length. If a K_e curve is not shown for the selected K_e, see Step 2 below. If the n value for the culvert selected differs from that of the nomograph, see Step 3 below.~~

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

~~Step 3: For a different roughness coefficient n_1 than that of the chart n, use the length scales shown with an adjusted length L_1, calculated by the following equation:~~

~~$L_1 = L(n_1/n)^2$ (Eq. 7-14)~~

~~c. Using a straightedge, connect point on length scale to size of culvert barrel and mark the point of crossing on the "turning line." See Instruction 2 below for size considerations for rectangular box culvert.~~

~~d. Pivot the straightedge on this point on the turning line and connect given discharge rate. Read head in feet on the head (H) scale. For values beyond the limit of the chart scales, find H by solving Equation 7-13.~~

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

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

~~b. Connect proper point (see instruction 1) on length scale to barrel area and mark point on turning line.~~

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

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

7.4.4 Example 7-1

The following example problem utilizes computation Table 7-2 for a culvert rating curve calculation.

Given:	Culvert size	= 48 inches RCP
	Length ~~length~~ L	= 110 feet
	n value	= 0.012
	Inlet elevation	= 720.0 feet
	Outlet ~~outlet~~ elevation	= 718.8 feet
	Slope ~~slope~~ S_0	= 0.010

Entrance condition (square edge), K_e = 0.50
Maximum elevation for embankment = 732.0 ft.

Find: Culvert rating curve

Table 7-2 is used to make ~~take~~ the computations for the culvert design.

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

Step 2. Based on the inlet control conditions, the ratio of HW/D is used to find the flows (Q) which are put in Column 4. In this example, the FHWA nomograph for a circular pipe under inlet control conditions ~~Figure 7-5 (B) in Appendix E of this manual~~ is utilized.

Step 3. For outlet control conditions, the flow rate Q in ~~column~~ Column 4 is used to determine the headloss ~~head loss~~ (H) in Column 5. In this example, the FHWA nomograph for a circular RCP under outlet control conditions ~~Figure 7-12 in Appendix E of this manual~~ is utilized.

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

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

Step 6. The critical depth (dc) is found from the FHWA nomographs for critical depth ~~Figures 7-18 to 7-22 in Appendix E of this manual~~, and then used to compile Column 8.

Step 7. The headwater depth (HW) now can be computed from the following equation: HW = H + ho - LSo

Where,

HW	= vertical distance from flow line at the entrance to the pool surface, feet
H	= headloss, feet (use appropriate nomograph)
ho	= vertical distance from flow line at the outlet to the hydraulic grade line, feet
So	= slope of barrel, ft/ft
L	= culvert length, feet

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

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

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

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

7.5.0 HYDRAULIC CONSIDERATIONS IN BRIDGE DESIGN

7.5.1 General

Section 1.2.4.~~C~~ ~~and Section 1.2.4.D~~ of this manual ~~the City of Austin's Drainage Policy~~ states the City's criteria ~~position~~ concerning storm water overtopping bridge structures. ~~The current policy for overtopping of~~

~~residential streets is a maximum of 12 inches for the 100 year frequency storm, and for any street other than residential, the allowable maximum is six (6) inches for the 100 year frequency storm.~~

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

A. Channel transitions into and out of the bridge opening.

B. Overall length and height of bridge.

C. Cross-sectional opening of bridge.

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

E. Bridge alignment relative to general flow of main channel i.e., "skewed" crossing.

F. Number of crossings or bridge openings.

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

H. Design flows for bridge opening to pass.

I. Any freeboard requirements for channel design.

7.5.2 Types Of Flow For Bridge Design

Three (3) types of flow caused by bridge construction on a flood plain are shown in Figure 7-~~23~~ 5 in Appendix ~~E~~ D of this manual. The three flow types are described below:

A. **Type I Flow.**

Referring to Figure 7-~~23A~~ 5A in Appendix ~~E~~ D of this manual, it can be observed that normal water surface is above critical depth at all points. This has been labeled Type I, or subcritical flow, the type usually encountered in practice. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between cross-sections 1 and 4.

B. **Type IIA Flow.**

There are at least two (2) variations of Type II flow which will be described here as Types IIA and IIB. For Type IIA flow, Figure 7-~~23B~~ 5B in Appendix ~~E~~ D of this manual, normal water surface in the unconstricted channel again remains above critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at cross-section 4).

C. **Type IIB Flow.**

The water surface for Type IIB flow, Figure 7-~~23C~~ 5C in Appendix ~~E~~ D of this manual, starts out above both normal water surface and critical depth upstream, passes through critical depth in the constriction and then returns to normal. The return to normal depth can be rather abrupt as in Figure 7-~~23C~~ 5C in Appendix ~~E~~ D of this manual, taking place in the form of a poor hydraulic jump, since normal water surface in the stream is above critical depth.

D. **Type III Flow.**

In Type III Flow, Figure 7-~~23D~~ 5D in Appendix ~~E~~ D of this manual, the normal water surface is below critical depth at all points and the flow throughout is supercritical. This is an unusual case requiring a steep gradient but such conditions do exist, particularly in mountainous regions. Theoretically, backwater should not occur for this type, since the flow throughout is supercritical. It is more than likely that an undulation of

the water surface will occur in the vicinity of the constriction, as indicated on Figure 7-~~23D~~ 5D in Appendix ~~E~~ D of this manual.

A more thorough and complete discussion of these parameters and preliminary design procedures are presented in Chapters 1 and 11 of Hydraulics of Bridge Waterways by U.S. Department of Transportation Federal Highway Administration, Second Edition, September, 1973.

7.5.3 Modeling Hydraulic Conditions

The most commonly used backwater program for modeling hydraulic conditions at existing or proposed bridge crossings is the U.S. Army Corps of Engineers HEC-RAS.

Table 7-2 Calculation Table for Culvert Design

Click to view Table 7-2

Example Table 7-2 Calculation Table for Culvert Design

Click to view Example Table 7-2

SECTION 8 - STORMWATER MANAGEMENT

8.1.0 GENERAL

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

The basic concept of SWM for peak rates of runoff is to provide for a temporary storage of stormwater runoff. Runoff is then released 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 8-1 in Appendix ~~E~~ D of this manual represents a storm runoff event without SWM, while the dashed line hydrograph depicts the same event with SWM. The peak flow of the undetained hydrograph could exceed the capacity of the downstream conveyance system and thereby cause surcharging and flooding problems. With the introduction of the SWM facility, the solid lined hydrograph is spread over a longer time period and its peak is reduced. The area between the two curves to the left of their intersection represents the volume of runoff~~,~~ temporarily stored or detained in the SWM facility.

The City of Austin approaches the control of excess flows ~~to comply with the Drainage Policy Section of this manual~~ through the application of both on-site ~~and~~ /off-site and regional SWM detention facilities ~~regional SWM~~. Essentially, the distinction between the two approaches is that on-site or off-site is generally limited to site specific criteria, while regional incorporates a basin-wide hydrologic analysis.

8.2.0 REGIONAL STORMWATER MANAGEMENT PROGRAM

8.2.1 General

The Regional Stormwater Management Program (RSMP) provides for the planning, design and construction of public regional drainage improvements, using fees paid by the owners of those developments. The RSMP is administered by the Watershed Protection Department. The RSMP uses a watershed-wide approach to analyze potential flooding problems, identify appropriate mitigation measures, and select site locations and design criteria for regional drainage improvements. These improvements may include ~~detention and retention~~ regional detention ponds, ~~waterway enlargement and channelization~~, channel modifications, ~~and~~ improved conveyance structures, and voluntary floodplain buyouts. The RSMP is established in watersheds ~~in and around~~ inside and outside of the City that are currently developing and have potential for flooding problems as undeveloped land is converted to impervious cover. In these watersheds, the RSMP allows developers to participate in the program ~~(~~in lieu of constructing on-site detention facilities if the proposed development will produce no ~~identifiable~~ additional adverse flooding impact to other nearby and downstream properties due to increased runoff. Existing limitations for RSMP participation include: the lack of conveyance or flooding problems in the downstream conveyance system, such as existing buildings in and near flood prone areas; undersized storm drain systems and substandard roadway crossings in the public right-of-way; and flood-prone tributaries and creeks. ~~of constructing on-site controls) if the resulting use of regional drainage improvements will produce no identifiable adverse impact to other properties due to increased runoff from the proposed development. An ongoing long-term goal of the Watershed Engineering Division is to develop master drainage plans for all watersheds in and around the City. These plans will~~

~~include flood plain analyses, identification of existing and potential flooding problems, benefit/cost analyses of potential solutions, and identification of future participants in the RSMP.~~

The fees charged for participation in the RSMP are non-refundable and are based upon the size of the development, the proposed land use and the development intensity. The fees are deposited in a dedicated fund and are allocated for the watershed in which each development is located. For additional information on the RSMP, please refer to the RSMP link on the Watershed Protection Department Programs page on the City of Austin's website, www.austintexas.gov. ~~Appendix D.~~

~~The benefits afforded by the RSMP include the following:~~

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

~~B. Adequate maintenance is more likely due to the City's vested interest and responsibility in the RSMP.~~

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

~~D. The expanded land area required for regional ponds lends itself to other uses (e.g., parks, nature areas, organized sports, etc.).~~

8.2.2 Participation Guidelines

A. **General.** The following ~~guidelines~~ rules are provided for those developments that desire to participate in the RSMP. Reference should be made to Table 8-1 ~~Section 1.4.0~~ for a listing of the watersheds in which participation is available.

Table 8-1 lists the Austin-area watershed codes named after the primary watercourse of the watershed and indicates those watershed basins which are presently a part of the RSMP. However, this does not preclude a regional application in any watershed. Watershed boundary delineations are maintained by the City. Use of any other delineation must be approved by the City and reflected on official City GIS maps, as designated by the Director of the Watershed Protection Department, prior to use.

To determine the exact service area boundaries and regional pond locations the engineer should contact the ~~Watershed Engineering Division (WED) of the~~ Watershed Protection ~~and Development Review~~ Department. Developers who choose to provide on-site SWM should refer to Section 8.3.0 for design criteria. Participation may be granted upon determination of the applicant's ability to satisfy the requirements set forth below. It should be understood however, that this policy cannot cover all situations and that final judgment of eligibility shall be made by ~~WED~~ the Watershed Protection Department.

~~It is suggested that each RSMP applicant shall submit a completed request form and a SWM concept plan to WED at the time of preliminary plan submittal or site plan submittal.~~

~~The request form and a check list for the concept plan are contained in Appendix D. In addition, a drainage report may be required. The contents of the drainage report will be specified by Watershed Engineering Division on a case-by-case basis. In addition to these general requirements, each development will be placed in one or more of the "Types" listed below and shall meet each of those requirements.~~

Table 8-1 Watersheds Eligible for RSMP Participation

CODE	WATERSHED NAME	DISCHARGES INTO
BAR	Barton Creek	Lady Bird Lake

BER	Bear Creek	Onion Creek
BUL	Bull Creek	Lake Austin
CAR	Carson Creek	Colorado River
CTM	Cottonmouth Creek	Onion Creek
CCE	Country Club East	Colorado River
CCW	Country Club West	Colorado River
DKR	Decker Creek	Gilleland Creek
DRE	Dry Creek East	Colorado River
EBO	East Bouldin Creek	Lady Bird Lake
ELM	Elm Creek	Colorado River/Gilleland Creek
HRS	Harris Branch	Gilleland Creek
LKC	Lake Creek	Brushy Creek
LWA	Little Walnut Creek	Walnut Creek
LBR	Little Bear Creek	Bear Creek
NFD	North Fork Dry Creek	Dry Creek (East)
ONI	Onion Creek	Colorado River
RAT	Rattan Creek	Lake Creek
RIN	Rinard Creek	Onion Creek
SHL	Shoal Creek	Lady Bird Lake
SLA	Slaughter Creek	Onion Creek
SBG	South Boggy Creek	Onion Creek
SFD	South Fork Dry Creek	Dry Creek (East)
WLN	Walnut Creek	Colorado River
WBL	West Bull Creek	Bull Creek
WMS	Williamson Creek	Onion Creek

B. ~~Categories for~~ Participation Requirements.

It is required that each RSMP applicant shall submit a completed request form and engineering submittal to the Watershed Protection Department at the time of preliminary plan submittal or site plan submittal. To view the request form and a check list for the engineering submittal, please refer to the RSMP program link in the Watershed Protection page of the City of Austin's website at www.austintexas.gov.

In order to participate in the program the applicant must satisfy all of the following conditions:

~~1. Type I - All developments that are contiguous with the main branch of a channel (as defined by the WED) and whose post-developed flows discharge directly into that are classified as Type I. In watersheds where the RSMP is available, participation may be allowed under one or more of the following conditions:~~

~~a. The Development is in an area of the watershed where the WED has determined that participation would not have adverse impacts on other properties.~~

~~b. The existing regional pond has available excess (unused or non-dedicated) capacity.~~

~~The City shall make every effort to afford participation to all applicants classified as Type I. (However, participation may be denied for reasons outside the scope or authority of the RSMP.)~~

~~2. Type II - All other developments which discharge into an intervening drainage system (i.e., storm sewer, tributary channel, etc.) are classified as Type II. Participation may be allowed if one or more of the conditions for Type I participation and both of the following conditions are met:~~

1. The intervening drainage system from the site to the tributary or main branch of the downstream mapped floodplain must have the capacity to provide for the fully developed 100-year storm from the entire upstream drainage area. If the downstream systems are undersized or downstream flooding conditions exist, RSMP participation may be approved if it can be verified there will be no ~~identifiable~~ additional adverse flooding impact to downstream properties for storm events up to and including the 100-year storm. ~~provisions may be made for upgrading them to required capacity.~~

2. The submitted engineering analysis must include a certified statement by ~~the design engineer~~ a licensed engineer in the State of Texas that no additional adverse flooding impacts to other property will occur as a result of the proposed improvements.

~~b.~~

3. An easement for unconditional conveyance of the fully-developed 100-year flood event from the site to the main branch or tributary of the watershed must be either in place, or acquired before participation is allowed.

C. Special Conditions. In addition to the specific criteria given above, the engineer should note the following conditions which could arise:

1. Should a regional detention facility or the intervening public drainage system be committed to its maximum capacity, an applicant may (at the City's discretion), increase the capacity of the regional facility or drainage system through approved modifications. The funding of any such modifications will be the responsibility of the applicant, and may ~~shall~~ take the place of the required ~~prescribed~~ participation fee:~~.~~

   a. if the cost of the improvements are equal to or greater than the required fee; and

   b. the improvements provide a public benefit.

2. If the subject tract desires to participate but intends to develop prior to construction of the regional facility or conveyance improvements, provisions must be made by the applicant for temporary on-site detention.

3. Existing on-site ponds may be removed if the development is approved as a participant in the RSMP and the Watershed Protection Department ~~WED~~ reviews and approves such removal.

D. Participation Fees. Participation fees will be calculated at the time of SWM concept plan submittal. To view the ~~The~~ fee schedule and the present fees for participation, please refer to the RSMP link on the Watershed Protection Department Programs page on the City of Austin's website, www.austintexas.gov. ~~is posted on the RSMP section of the Watershed Protection website.~~ Any increase will be posted at least 30 days prior to enactment. ~~The present fees for participation are listed in Appendix D.~~ The participation fees shall apply to all areas, except dedicated greenbelts, common areas, permanent retention facilities, and

areas undevelopable in accordance with City of Austin Ordinances.

Participation fees will be used by the City to fund the design and construction of regional drainage facilities for the management of stormwater peak rates of runoff ~~and water quality~~.

After a development is accepted for participation, fees shall be paid in accordance with the following:

1. Single-Family and Duplex Subdivisions.

a. For single-family subdivisions which do require the construction of streets or drainage facilities, a letter of credit must be posted with the Watershed Protection ~~and Development Review~~ Department in an amount equal to the total participation fee prior to final plat approval. This letter of credit must be replaced by cash prior to construction plan approval.

b. For single-family subdivisions which do not require the construction of streets, payment ~~(cash or cashier check only)~~ must be made prior to final plat approval.

2. Commercial and Multi-Family Site Development. For commercial and multi-family site development (including ~~includes~~ triplexes, fourplexes, apartments and condominiums), payment ~~(cash or cashier check only)~~ must be made prior to issuance of a development permit.

3. Multi-Family, Commercial and Industrial Subdivisions. For multi-family, commercial and industrial subdivisions, payment ~~(cash or cashier check only)~~ shall be made prior to final plat approval for the rights-of-way. In addition, the applicant shall assign, by plat note, the responsibility for payment of the participation fee by the individual lot developer prior to their development permit approval.

Upon payment of fees, the agreement available via the RSMP link on the Watershed Protection Department Programs page on the City of Austin's website, www.austintexas.gov ~~shown in Appendix D~~ shall be signed and act as a binding agreement between the developer and the City.

8.2.3 Watershed ~~Master Plans~~ Floodplain Models

The City of Austin maintains hydrologic and hydraulic floodplain management models for most of the watersheds within the City of Austin's Full Purpose, Limited Purpose and Extra Territorial Jurisdictions. Where applicable, these City of Austin regulatory models should be used as base models for the impact analyses and drainage design associated with development. Users of these models should be aware that the floodplain models provided have been developed on a watershed-wide basis and may therefore not be applicable without modification on a site-by-site basis. A licensed engineer in the State of Texas ~~Texas Licensed Professional Engineer~~ must certify any results based on these models or modified versions thereof that are submitted to the City as part of the land development review and permit approval process. The City also maintains copies of the FEMA regulatory models. However, since the City has obtained these models directly through the consultants who developed them, there may be changes which have been approved by FEMA that are not incorporated into the models that the City has on file. Only FEMA can provide the official regulatory models used for flood insurance purposes. All models maintained by the City may be obtained, free of charge, through the FloodPro application at the following location: www.austintexas.gov/floodpro. Model requests, comments, concerns and questions also may be sent to the City's Floodplain Office via email at FloodPro@austintexas.gov.

~~General - A watershed master plan is a tool for planning regional SWM activities throughout a particular watershed. The watershed master plan includes the most current hydrologic and hydraulic analyses and floodplain mapping. The master plan may also include information which documents existing and anticipated flood hazards and information concerning existing or proposed regional SWM improvements. The master plans are located at the Watershed Engineering Division of the Watershed Protection and Development Review Department and should be consulted when regional SWM or drainage projects are being planned.~~

8.3.0 STORMWATER MANAGEMENT PONDS

8.3.1 General

Stormwater Management (SWM) ponds may be of two basic types: On-site/off-site and regional. In general, on-site or off-site ponds are those which are located off-channel and provide stormwater management for a particular project or development. Regional ponds are designed to provide stormwater management in conjunction with other improvements on a watershed-wide basis. SWM ponds may be further classified as retention or detention ponds and may incorporate water quality best management practices (BMPs) as defined in the Environmental Criteria Manual ~~such as sedimentation, infiltration, or filtration~~. The performance and safety criteria in this section apply to all ponds which provide management of peak rates of stormwater runoff regardless of type.

8.3.2 Performance Criteria for ~~on-Site~~ SWM Ponds

A. Detention ponds shall be designed to reduce post-development peak rates 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. For the post-development hydrologic design of the SWM pond, any off-site areas which drain to the pond shall be assumed to remain in the existing condition. If off-site flows are conveyed through the SWM pond, the SWM pond outlet structure must be designed to safely pass 100-year fully developed off-site flows in accordance with the Safety Criteria set forth in Section 8.3.3.

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

~~ON-SITE SWM POND CLASS~~	~~DRAINAGE AREA~~
~~Small~~	
~~Large~~	~~25-64 acres~~

B. For design purposes, any pond with a drainage area larger than 64 acres shall be classified as a regional pond. Performance criteria for regional ponds shall be reviewed and approved by the Watershed Protection Department on a project-by-project basis. The determination shall be based on a preliminary engineering study prepared by a licensed engineer in the State of Texas~~the engineer~~.

C. Maximum retention or “draw-down” time for flood detention ponds shall not exceed 24 hours from the time of peak storage to the time of complete emptying of the pond, as determined by hydrograph routing or other calculations acceptable to the City. This requirement does not apply to facilities in which retention or “draw-down” time is required to be greater than 24 hours. Only the portion of the volume within a water quality control available after 24 hours of drawdown time may be used or credited towards detention requirements.

~~B. On-site SWM ponds shall be designed to reduce post-development peak rates of discharge to existing pre-development peak rates of discharge for the 2, 10, 25 and 100 year storm events at each point of discharge from the project or development site. 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.~~

~~8.3.3 Performance Criteria For Regional SWM Ponds~~

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

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

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

~~B. Performance criteria for regional ponds shall be determined by the Watershed Engineering Division on~~

~~a project-by-project basis. The determination shall be based on a preliminary engineering study prepared by the engineer and on the Watershed Master Plan for the watershed.~~

8.3.~~4~~ 3 Safety Criteria For SWM Ponds

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 SWM pond.

A. The spillway, outfall, embankment, and appurtenant structures shall be designed to safely pass the design storm hydrograph with the freeboard shown in the table below. All contributing on-site drainage areas, ~~including on-site~~ and off-site areas which are routed through the SWM pond, shall be assumed to be fully developed in order to properly size the spillway, outfall, embankment and appurtenant structures. Any orifice with a dimension smaller than or equal to 12 inches shall be assumed to be fully blocked. For all spillways (especially enclosed conduits), the ability to adequately convey the design flows must take into account any submergence of the outlet, any existing or potential obstructions in the system and the capacity of the downstream system. ~~For these reasons, enclosed conduit spillways connecting directly to other enclosed conduit systems are discouraged. If used, they must be justified by a rigorous analysis of all enclosed conduit systems connected to the spillway.~~

DETENTION POND CLASS		DESIGN STORM EVENT	FREEBOARD ON TOP OF ENBANKMENT, FT
On-site/Off-site	Small (DA < 25 ac)	100-year	0
	Large (25 ≤ DA < 64 ac)	100-year	1.0
Regional	DA ≥ 64 ac	100-year	2.0

~~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 by a dam-break analysis based on the principles outlined in Chapter 299 of the Texas Administrative Code. The dam-break analysis shall be submitted to the Watershed Engineering Division for approval.~~

B. Any hydraulic structure designed to impound storm water that has a height greater than or equal to six (6) feet at any point along the perimeter of the ~~stormwater management (~~SWM~~)~~ pond is a dam and must be designed to safely pass 75 percent of the probable maximum flood (PMF) as evidenced by certification using the statement provided in DCM 8.3.~~4~~3.B.3. by an engineer licensed in the State of Texas. The certification statement may be divided into the four disciplines of hydrology, hydraulics, structural and geotechnical and independently certified.

1. The height of the hydraulic structure (dam) is measured from the top of the structure to the downstream intersection of the structure and the natural or excavated ground, whichever is lower.

2. The PMF is computed by using the probable maximum precipitation (PMP) values as described in Section 2.6 of the Drainage Criteria Manual.

3. Dam Safety Certification Statement:

I [name of professional engineer] Texas license number [number] certify that the design of the dam in this set of plans can safely pass 75 percent of the Probable Maximum Flood based on the hydrologic, hydraulic, structural and geotechnical analysis using standard accepted engineering practices.

4. SWM ~~Stormwater Management~~ ponds that are considered dams as defined in this section of the ~~DCM~~ Drainage Criteria Manual may not be designed or constructed with any trees or other woody vegetation on the dam structure or within 20-feet of the upstream or downstream toe of the dam. This 20-foot clear zone must be called out on the site plan and for City maintained facilities must be part of the drainage easement dedicated for the dam facility. The toe of the dam is the junction of the constructed dam structure with the natural ground.

5. SWM ~~Stormwater Management~~ ponds that are considered dams as defined in this section of the ~~DCM~~ Drainage Criteria Manual may not have permanent irrigation systems installed on the dam.

6. SWM ~~Stormwater Management~~ ponds that are considered dams as defined in this section of the ~~DCM~~ Drainage Criteria Manual must be vegetated with grasses that do not exceed 12 inches in height and can be mowed as frequently as weekly. Examples include Bermuda grass and buffalo grass.

7. SWM ~~Stormwater Management~~ ponds that are considered dams as defined in this section of the ~~DCM~~ Drainage Criteria Manual shall provide a fixed vertical marker on or near the emergency spillway indicating the water surface elevation relative to the top of the main embankment. The markings should be in half-foot increments, viewable from the furthest point of access, and must be retroreflective as defined by the Texas Manual on ~~uniform~~ Uniform Traffic Control Devices (TMUTCD).

C. All SWM ponds shall be designed using a hydrograph routing methodology. The appropriate City of Austin rainfall distribution shall be used to determine all runoff hydrographs.

D. 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 ~~11~~-15	8
15-20	10
20-25	12
25-35	15

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

This amount shall in no case be less than 5% of the total fill height. All earthen embankments shall be compacted to 95% of maximum density in accordance with COA standard specifications.

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

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

H. When designing ponds in series (i.e., when the discharge of one ~~(1)~~ becomes the inflow of another), a licensed engineer in the State of Texas ~~the engineer~~ must submit a hydrologic analysis which demonstrates the system's adequacy. This analysis must incorporate the construction of hydrographs for all inflow and outflow components.

~~I. No outlet structures from stormwater management facilities, groundwater collection, detention, filtration~~

~~and/or sedimentation ponds, parking detention or other improvements discharging concentrated flows concentrating structures shall be designed to discharge concentrated flow directly onto arterial or collector streets. For local streets, no concentrated discharge from sites larger than 0.25 acres is permitted. All concentrated 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 of the outfall, fiscal security shall be posted for the extension of 300 feet of storm drain; and 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. Concentrated discharge across a sidewalk area will not be allowed. A channel section can be used under the sidewalk area, provided it is covered by a method approved by the Public Works Department and the outlet device utilizes sheet flow methods.~~

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

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

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

8.3.~~5~~ 4 Outlet Structure Design

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

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

A. **Rectangular Weir Flow Equation** (See Figure 8-2 in Appendix ~~E~~ D of this manual)

$$Q = CLH^{3/2} \qquad \text{(Eq. 8-1)}$$

Where

Q = Weir discharge, cubic feet per second
C = Weir Coefficient
L = horizontal length, feet
H = Head on weir, feet

B. **V - notch Weir Flow Equation** (See Figure 8-2 in Appendix ~~E~~ D of this manual)

$$Q = C_v \tan (0/2)H^{2.5} \qquad \text{(Eq. 8-2)}$$

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

C. **Orifice flow equation** (See Figure 8-2 in Appendix ~~E~~ D of this manual)

$$Q = C_oA(2gH)^{0.5} \qquad \text{(Eq. 8-3)}$$

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 ~~WED~~ the Watershed Protection Department prior to use.

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

8.4.0 DETENTION POND STORAGE DETERMINATION

A flow routing analysis using detailed hydrographs must be applied for all detention pond designs. The Soil Conservation Service hydrologic methods (~~available in~~ TR-20~~, HEC-1~~) and the Hydrologic Engineering Center (HEC) hydrologic methods (HEC-HMS) may be used. The engineer may use other methods but must have their acceptability approved by the Director.

8.5.0 DETENTION BASIN MAINTENANCE AND EQUIPMENT ACCESS REQUIREMENTS

Refer to Section 1.6.3.C of the Environmental Criteria Manual.