CHAPTER 8
DETENTION / RETENTION HYDRAULICS

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Synopsis

Land development activities often alter the hydrologic characteristics of a watershed, which may in turn affect both the timing, velocity, magnitude, and quality of runoff. Stormwater detention/retention to address quantity and quality is required by Nashville Regulations (see Volume 1, Sections 2.4, 2.5, 6.1, 6.6, 6.7) to control and mitigate adverse impacts caused by development. A BMP Treatment Train approach to provide a series of controls is discussed in Volume 4 Section 1. Detention and retention systems are a component in the BMP Treatment Train for a given site.

This chapter provides general design criteria for detention/retention basins as well as procedures for performing preliminary sizing and final reservoir routing calculations. The Storage Indication Method is presented as an acceptable method for detention routing calculations. Land-locked and flood prone drainage areas will require retention/detention storage areas designed to address special conditions to protect public safety from flooding, facility failure, and/or sinkholes. Wet detention facilities with a permanent pool also require special design considerations. This chapter is supported and augmented by Volume 4 Section 6 Permanent Treatment Practices (PTP).

8.1 Design Criteria

In general, detention facilities offer temporary storage accompanied by controlled release of the stored water, while, retention refers to stormwater storage without access to a positive outlet. Some detention and retention facilities may have a permanent pool of water below the outlet elevation; in such cases, water budget calculations are required and should consider average annual, dry season and wet season conditions. Retention facilities shall be implemented with special considerations. These systems require infiltration to recover storage. Additional information about infiltration systems is presented in Volume 4 PTP-14.

The design considerations and criteria for detention/retention facilities should include:

1. Multiple systems and Best Management Practices (BMPs)
2. Release rates for multiple design storms
3. Detention volumes (“live” pool and permanent pool if applicable)
4. Grading, depth, and geometry requirements
5. Outlet structure(s)

Additional selection, design, sizing, and maintenance criteria are presented in Volume 4 Section PTP.

8.1.1 Multiple Systems and Best Management Practices (BMPs)

Detention and retention can be used separately or together in series or parallel with other stormwater BMPs to offer cumulative benefit to stormwater quantity and/or quality. Selecting a series of practices is discussed in Volume 4 Section 1.6, including how to use the BMP Treatment Train to select facility types. Volume 4 PTP-08 discusses multiple systems in additional detail.

8.1.2 Release Rates

Outlet structure release rates should approximate pre-development peak runoff rates for the water quality volume, 2-year through 100-year 24-hour duration storms. The procedures for calculating the live pool for a wet and dry detention pond are presented in Volume 4 PTP-01 and 06, respectively. The release rate for a dry (without permanent pool) detention pond designed to benefit stormwater quality is 24 to 48 hours of the "live" pool volume and 24 to 60 hours for a wet (with permanent pool) detention pond. The live pool contains the water quality volume is dependant upon land use characteristics and average annual runoff capture percentage of at least 85 to 95% or 0.50 to 1.25-inches per acre depending on runoff coefficients. The residence time for a permanent pool in a wet detention pond should be at least two to four weeks.

Design calculations are required to demonstrate that the 2 and 100-year design storms are controlled and that water release rates are designed to facilitate at least a 85 to 95% average annual runoff capture percentage. If these storm events are managed, intermediate storm return periods can be assumed to be adequately managed. V-notch and multistage outlet structures are preferred. V-notch weirs are discussed in Section 8.4.5 and Volume 4 PTP-01 and 06. Multi-stage control structures, such as the one shown in Figure 8-1, are required to provide the proper drawdown time of the water quality volume and control the 2- and 100-year storms.

8.1.3 Detention Volume

Detention volume shall be adequate to attenuate the post-development peak discharge rates to allowable rates determined for Section 8.1.2. Routing calculations shall be consistent with procedures in Section 8.6. Facilities that are to be used as temporary (construction phase) sediment control management practices shall have the excavated detention volume oversized to account for the anticipated amount of sediment to be trapped. If siltation during construction is in excess of sedimentation estimates then the permanent detention volume, design dimensions shall be restored before as-built certification is submitted. Furthermore, as discussed in Volume

4 PTP-02 and 03, the detention volume should be oversized to allow for long-term (5to10 years) sediment storage.

8.1.4 Grading and Depth

The construction of detention/retention facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments should be less than 10 feet in height and should have side slopes no steeper than 3:1 (horizontal to vertical). Embankments may be higher with special considerations presented in Section 8.1.5. Stormwater quality facilities with a littoral zone should be graded at a 6:1 (horizontal to vertical) slope in those areas. The remainder of the grading should be no steeper than 4:1 (horizontal to vertical). Riprap-protected embankments should be no steeper than 2:1. Geo-technical slope stability analysis is recommended for embankments greater than 3 feet in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluations can be found in most soil engineering textbooks, including those by Spangler and Handy (1982) and Sowers and Sowers (1970).

The shape of the facility is important for water quality treatment. The pond should be designed to minimize short-circuiting by including energy dissipaters on inlets that are placed as far away from the outlet structure as possible. The facility should have a shape with at least a 3:1 (preferably up to a 7:1) length to width ratio. If topography or aesthetics require the pond to have an irregular shape then the pond area and volume should be increase to compensate for the dead (very low flow) spaces, but this volume can be considered for stormwater quantity management.

Areas above the normal high water elevation of detention/retention facilities should be sloped at a minimum of 5 percent toward the facilities to allow drainage and to prevent standing water except in areas design control that flow such as landscape swales and biofilters.

The bottom area of dry detention/retention facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2 percent bottom slope is recommended. Concrete lined low flow or pilot channels constructed across the facility bottom from the inlet to the outlet are not preferred. Low flows should be distributed evenly into sheet flow across the bottom of the facility. It should be noted that it is preferred that detention/retention facilities should be designed as off-line structures, where possible, to improve treatment efficiencies. On-line facilities are acceptable depending on pond geometry and specific site characteristics.

The maximum depth of stormwater detention/retention facilities will normally be determined during the permitting process. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of attached weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A maximum depth of 6 to 12 feet is generally reasonable. Aeration may be required in permanent pools deeper than 12 feet to prevent thermal stratification and that could result in anaerobic conditions and odor problems.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements, and required freeboard. Aesthetically pleasing features are also important. A minimum freeboard of 1 foot above the 100-year design storm high water elevation should be provided for impoundment depths of less than 20 feet. No freeboard is required for underground detention systems. Impoundment depths greater than 20 feet are subject to the requirements of the State Dam Safety Act (see Section 8.1.5). Additional guidance is provided in Volume 4 PTP-01 and 06.

8.1.5 Outlet Structure

Outlet structures selected for detention/retention facilities should typically include a principal outlet riser or weir and an emergency overflow and must be able to accomplish the necessary functions of the facility. Outlet structures can take the form of drop inlets or any combination of pipes, weirs, and orifices. The principal outlet is intended to convey the water quality volume and quantity design storm without allowing flow to enter an emergency outlet. Selecting a magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The sizing of a particular outlet structure should be based on results of hydrologic routing calculations (see Section 8.6), consistent with criteria in Sections 8.1.2, 8.1.3, and 8.1.4.

8.1.6 State Dam Safety Program

National responsibility for the promotion and coordination of dam safety lies with the Federal Emergency Management Agency (FEMA). State responsibility for administration of the Tennessee Safe Dams Act of 1973 (T.C.A. 70-2501) in coordination with the provisions of the Federal Dam Safety Act of 1983 (PL 99-662) is assigned to the Department of Health and Environment. Rules and regulations relating to applicable dams are promulgated by this department.

Under these regulations, a dam is an artificial barrier that does or may impound water and that is 20 feet or greater in height or has a maximum storage volume of 30 acre-feet or more. A number of exemptions are allowed from the Safe Dams Act and the applicable state office should be contacted to resolve questions.

Dams are classified as either new or existing, by hazard potential, and by size. Hazard potential categories are listed below:

1. Category 1 dams are located where failure would probably result in any of the following: loss of human life, excessive economic loss due to damage of downstream properties, public hazard, or public inconvenience due to loss of impoundment and/or damage to roads or any public or private utilities.

2. Category 2 dams are located where failure may damage downstream private or public property, but such damage would be relatively minor and within the general financial capabilities of the dam owner. Public hazard or inconvenience due to loss of roads or any public or private utilities would be minor and of short duration. Chances of loss of human life would be possible but remote.

3. Category 3 dams are located where failure may damage uninhabitable structures or land but such damage would probably be confined to the dam owner's property. No loss of human life would be expected.

Size categories are listed in Table 8-1.

Detailed engineering requirements are given in the regulations for new dams. Existing dams constructed of earth embankment must be stable and protected from erosion. For all dams, the 6-hour design storm is specified in Table 8-2. Applicable regulations should be consulted for further details and engineering requirements.

8.2 General Water Quality Procedures

Procedures for designing stormwater quality detention are presented in Volume 4 PTP-01 and 06. However, it should be noted that many stormwater quantity facilities can be designed to account for stormwater quality by modifying the outlet structure, some minor grading and shape changes, and inclusion of a sediment and floatable debris removal forebay or other BMPs. Other information on new or retrofit stormwater quality retention/detention facility design computations and other considerations can be found in Hartigan (1988), Roesner et al. (1998), Hasty, McCormick and Schmidt (1999) . The remained of this chapter will discuss stormwater quantity procedures and computations Volume 4 Sections 1 and 6 (PTP) should be referenced for detail about stormwater quality procedures and computations.

8.3 General Water Quantity Procedures

The following three relationships should be considered when sizing a stormwater detention facility:

1. Inflow hydrographs for a range of design storms (see Chapter 2). This should include the 2, 5, 10, 25, 50, and 100-year events

2. Stage-area-storage curve for the detention basin (see Figure 8-4 for an example)

3. Stage-discharge curve for basin outlet control structure (see Figure 8-5 for an example) to match historic stage-discharge for the site.

A trial and error design procedure is often required, since only the inflow hydrographs are generally known. A general procedure for evaluating these variables is presented below:

1. Compute inflow hydrographs for 2 through 100-year design storms, as required in Volume 1, using procedures from Chapter 2. Both pre- and post-development hydrographs are required for the 2 through 100-year design storms.

2. Perform preliminary calculations to evaluate detention storage requirements (see Section 8.5) for the hydrographs from Step 1. If detention/retention requirements are satisfied for the 2 and 100-year design storms, intermediate storms are assumed to be controlled.

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used.

4. Size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the Storage Indication Method (see Section 8.6.1). If the routed post-development peak discharges from the 2 through 100-year design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

6. Evaluate the downstream effects of detention outflow to ensure that the recession limb of the outflow hydrograph does not cause downstream flooding problems. The potential significance of downstream effects from detention can be evaluated by comparing the recession limbs of the pre-development and routed post-development hydrographs. When the maximum difference in discharge rates and the hydrograph time base both increase by more than 20 percent for the routed post-development hydrograph, then watershed modeling or information from a watershed master plan is required to show that downstream impacts can be controlled.

7. Evaluate the control structure outlet velocity and provide stabilization if velocity is greater than 3 ft/s for any storm event.

Since this procedure can involve a significant number of reservoir routing calculations, a computer method is useful for conducting final routing computations (See Chapter 12). Other information on retention/detention basin design computations can be found in articles by Mason and Rhomberg (1982, 1983), McKinnon (1984), Mein (1980), Rossmiller (1982), and Sandvik (1985).

8.4 Outlet Hydraulics

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlet works, procedures presented in Chapter 5 should be used to develop stage-discharge data. Slotted riser pipe outlet facilities should be avoided.

8.4.1 Sharp-Crested Weirs—No End Contractions

A sharp-crested weir with no end contractions is illustrated in Part A of Figure 8-5. The discharge equation for this configuration (Chow, 1959) is expressed as:

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right) L H^{1.5} \qquad (8\text{-}1)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 8-5, Part C)

H_c = Height of weir crest above channel bottom, in feet (see Figure 8-5, Part C)

L = Horizontal weir length, in feet

8.4.2 Sharp-Crested Weirs—Two End Contractions

A sharp-crested weir with two end contractions is illustrated in Part B of Figure 8-5. The discharge equation for this configuration (Chow, 1959) is expressed as:

$$Q = \left(3.27 + 0.4\frac{H}{H_c}\right)(L - 0.2H) H^{1.5} \qquad (8\text{-}2)$$

where:

Q = Discharge, in cfs

H = Head above the weir crest excluding velocity head, in feet (see Figure 8-5, Part C)

H_c = Height of weir crest above channel bottom, in feet (see Figure 8-5, Part C)

L = Horizontal weir length, in feet

8.4.3 Sharp-Crested Weirs—Submerged Discharge

The effect of submergence on a sharp-crested weir should be considered when applying Equations 8-1 and 8-2. When the tailwater rises above the weir crest elevation, the discharge over the weir will be reduced. To account for this submergence effect, the free discharge obtained by Equations 8-1 or 8-2 should be modified using the following equation (Brater and King, 1976):

$$Q_s = Q_f \left[1 - \left(\frac{H_2}{H_1} \right)^{1.5} \right]^{0.385} \qquad (8\text{-}3)$$

where:

Q_s = Submergence flow, in cfs

Q_f = Free flow, in cfs

H_1 = Upstream head above crest, in feet

H_2 = Downstream head above crest, in feet

8.4.4 Broad-Crested Weirs

The general form of the broad-crested weir equation (Brater and King, 1976) is expressed as:

$$Q = C\ L\ H^{1.5} \qquad (8\text{-}4)$$

where:

Q = Discharge, in cfs

C = Broad-crested weir coefficient

L = Broad-crested weir length, in feet

H = Head above weir crest, in feet

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087 for a broad-crested weir. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Additional information on C values as a function of weir crest breadth and head, from Brater and King (1976), is included in Table 8-3.

8.4.5 V-Notch Weirs

The discharge through a v-notch weir can be evaluated using the equation (Merritt, et al, 1995):

$$Q = C_1 \tan\left(\frac{q}{2}\right) H^{2.5} \qquad (8\text{-}5)$$

where:

C_1 = discharge coefficient (See Figure 8-2)

Q = Discharge, in cfs

θ = Angle of v-notch, in degrees

H = Head on vortex of notch, in feet

8.4.6 Proportional Weirs

Although more complex to design and construct, a proportional weir may reduce the required detention/retention volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs from Sandvik (1985) are as follows:

$$Q = 4.97\ a^{1/2}\ b(H - a/3) \qquad (8\text{-}6)$$

$$x/b = 1 - \frac{1}{p}\left(\arctan\sqrt{y/a}\right) \qquad (8\text{-}7)$$

where Q is the weir discharge, in cfs, and the dimensions a, b, H, x, and y are shown in Figure 8-6.

8.4.7 Orifices

The discharge through an orifice can be evaluated using the equation:

$$Q = CA\,(2gH)^{0.5} \qquad (8\text{-}8)$$

where:

Q = Discharge, in cfs

C = Orifice coefficient (a value of 0.6 is usually appropriate; see Table 8-3 if additional information is desired)

A = Area of orifice, in square feet

g = Acceleration due to gravity, 32.174 feet/second^2

H = Head above orifice centroid, in feet

8.5 Preliminary Detention Calculations

8.5.1 Storage Volume

A preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 8-7.

The required storage volume may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = 0.5\, T_i\, (Q_i - Q_o) \qquad (8\text{-}9)$$

where:

V_s = Storage volume estimate

Q_i = Peak inflow rate

Q_o = Peak outflow rate

T_i = Duration of basin inflow

Any consistent units may be used for Equation 8-9.

An alternative preliminary estimate of the detention volume required for a specified peak flow reduction also can be obtained by the following regression equation procedure:

1. Determine input data, including the allowable peak outflow rate, Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

2. Calculate a preliminary estimate of the ratio V_s/V_r using the input data from Step 1 and the following equation (Wycoff & Singh, 1976):

$$\frac{V_s}{V_r} = \frac{1.291\left(1 - \frac{Q_o}{Q_i}\right)^{0.753}}{\left(\frac{t_b}{t_p}\right)^{0.411}} \qquad (8\text{-}10)$$

   where:

   V_s = Volume of storage, in inches

   V_r = Volume of runoff, in inches

   Q_o = Outflow peak flow, in cfs

   Q_i = Inflow peak flow, in cfs

   t_b = Time base of the inflow hydrograph, in hours, determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak

   t_p = Time to peak of the inflow hydrograph, in hours

3. Multiply the ratio V_s/V_r from Step 2 by the volume of runoff in the inflow hydrograph to obtain the estimated storage volume required to keep from exceeding the allowable peak outflow rate for each design storm event.

8.5.2 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected detention volume can be obtained by the following regression equation procedure:

1. Determine input data, including the volume of runoff, V_r, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, the time to peak of the inflow hydrograph, t_p, and the storage volume, V_s.

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Wycoff & Singh, 1976) :

$$\frac{Q_o}{Q_i} = 1 - 0.712\left(\frac{V_s}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546} \qquad (8\text{-}11)$$

   where:

   Q_o = Outflow peak flow, in cfs

   Q_i = Inflow peak flow, in cfs

   V_s = Volume of storage, in inches

   V_r = Volume of runoff, in inches

   t_b = Time base of the inflow hydrograph, in hours, determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak

   t_p = Time to peak of the inflow hydrograph, in hours

3. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated in Step 2 to obtain the estimated peak outflow rate, Q_o, for the selected storage volume.

8.6 Routing Calculations

The Storage Indication Method is recommended for reservoir routing calculations for detention facility final design.

8.6.1 Storage Indication Method

The following procedure is used to perform a reservoir routing by the Storage Indication Method:

1. Develop an inflow hydrograph, a stage-discharge curve, and a stage-storage curve for the proposed detention facilities. Example stage-storage and stage-discharge curves are presented in Figures 8-3 and 8-4, respectively.

2. Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph.

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $S \pm O\Delta t/2$ versus stage. An example tabulation of storage characteristics curve data is shown in Table 8-4 and Figure 8-8.

4. For a given time interval, I_1 and I_2 are known. Given the depth of storage or stage, H_1, at the beginning of that time interval, $S_1 - O_1\Delta t/2$ can be determined from the appropriate storage characteristics curve (e.g., Figure 8-8).

5. Determine the value of $S_2 + O_2At/2$ from the following relationship:

$$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t \right] + \left[\frac{I_1 + I_2}{2}\Delta t \right] \qquad (8\text{-}12)$$

where:

S_2 = Storage volume at time 2, in cubic feet

O_2 = Outflow rate at time 2, in cfs

Δt = Routing time period, in seconds

S_1 = Storage volume at time 1, in cubic feet

O_1 = Outflow rate at time 1, in cfs

I_1 = Inflow rate at time 1, in cfs

I_2 = Inflow rate at time 2, in cfs

Other consistent units are equally appropriate, as demonstrated in the examples below.

6. Enter the appropriate storage characteristics curve (e.g., Figure 8-8) at the value of $S_2 + O_2\Delta t/2$ determined in Step 5 and read off a new depth of water, H_2.

7. Determine the value of O_2, which corresponds to a stage of H_2 determined in Step 6, using the stage-discharge curve (e.g., Figure 8-4).

8. Repeat Steps 1 through 7 by setting new values of I_1, O_1, S_1, and H_1 equal to the previous I_2, O_2, S_2, and H_2, and using a new I_2 value. This process is continued until the entire inflow hydrograph has been routed through the storage basin.

8.6.2 Example Problems

Example 8-1. Routing Using the Storage Indication Method

An example application of the Storage Indication Method using data presented in Figures 8-3, 8-4, and 8-8 is presented in Table 8-5. The inflow hydrograph is given in columns 1 and 2 of Table 8-5. The objective is to find the outflow using the Storage Indication Method. A step-by-step discussion of the calculations summarized in Table 8-5 is presented below.

1. Using the data tabulated in Column 2 of Table 8-5, calculate:

$$\frac{(I_1 + I_2)\Delta t}{2}$$

   and tabulate these values in Column 3 of Table 8-5. For routing calculations in this example, inflows and outflows are converted to units of acre-feet per minute. The final outflow is reported in cfs.

2. Given that $S_1 - O_1\Delta t/2 = 0.05$ acre-foot for $H_1 = 0$ foot, find $S_2 + O_2\Delta t/2$ by adding 0.05 + 0.01 (Column 5 value plus Column 3 value) and tabulate 0.06 acre-foot in Column 6 of Table 8-5.

3. Enter the $S + O\Delta t/2$ storage characteristics curve in Figure 8-8 and read the stage at the value of 0.06 acre-foot. This value is found to be 100.10 feet and is tabulated as stage H_2 in Column 7 of Table 8-5.

4. Using the stage of 100.10 feet found in Step 4, enter the stage-discharge curve (Figure 8-4) and find the discharge corresponding to that stage. In this case, O is approximately 1 cfs and is tabulated in Column 8 of Table 8-5.

5. Assign the value of H_2 to H_1, find a new value of $S_1 - O_1\Delta t/2$ from Figure 8-8, and repeat the calculations for Steps 2, 3, and 4. Continue repeating these calculations until the entire inflow hydrograph has been routed through the storage basin.

6. The Storage Indication Method calculations give a peak outflow of 220 cfs. The inflow hydrograph has a peak rate of 360 cfs, so a reduction of approximately 40 percent is calculated.

Example 8-2. Multi-Design Storm Quantity Management Detention/Retention Calculations

This example demonstrates the application of the methodology presented in this chapter for typical detention/retention facility design. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using appropriate hydrologic methods from Chapter 2.

Part 1. Design Discharge and Hydrographs

As specified in Section 8.1.1, the detention/retention design should consider both the 2- and 10-year design storm events. Example peak discharges from these storm events are as follows:

Pre-development 2-year peak discharge	150 cfs
Pre-development 10-year peak discharge	200 cfs
Post-development 2-year peak discharge	190 cfs
Post-development 10-year peak discharge	250 cfs

Since the post-development peak discharge must not exceed the pre-development peak discharge (Section 8.1.1), the allowable design discharges are 150 and 200 cfs for the 2- and 10-year storms, respectively.

Example runoff hydrographs are shown in Table 8-6. Inflow durations of the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for the 2- and 10-year storms.

Part 2. Preliminary Detention Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlined in Section 8.5.1. For the 2- and 10-year storms, the required storage volumes, V_s, are computed as follows:

2-year storm:

$$V_s = (0.5)(3,600)(1.2)(190\text{-}150)/43,560$$

$$V_s = 1.98 \text{ acre-feet}$$

10-year storm:

$$V_s = (0.5)(3{,}600)(1.25)(250\text{-}200)/43{,}560$$

$$V_s = 2.58 \text{ acre-feet}$$

Part 3. Facilities Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a retention/detention facility that should provide adequate peak flow attenuation for both the 2- and 10-year design storms are presented in Table 8-7. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates for both the 2-and 10-year design storms to be provided when the corresponding allowable peak discharges occurred. Storage values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 feet, and no tailwater submergence. The capacity of storage relief structures (see Section 8.7) was assumed to be negligible.

Reservoir routing was conducted using the Storage Indication Method (see Section 8.6.1) for both the 2- and 10-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Storage characteristics required for routing are presented in Table 8-7 and Figure 8-9.

Routing results using 0.1-hour time steps are shown in Tables 8-8 and 8-9 for the 2- and 10-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 2- and 10-year design storms.

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, the 100-year design storm should be routed through the facilities to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability and maintenance, grading to prevent standing water, and provisions for public safety.

Part 4. Downstream Effects

An estimate of the potential effects (increased peak flow rate and recession time) of detention/retention on downstream facilities (see Section 8.3, Step 6) may be obtained by comparing hydrograph recession limbs from the pre-development and routed post-development runoff hydrographs. Example comparisons are shown for the 2- and 10-year design storms in Figures 8-10 and 8-11, respectively.

8.6.3 NRCS TR-55 Graphical Method

The NRCS (formerly SCS) has a graphical method for estimating the peak flow reduction capability of detention ponds. The information in the 1975 version of TR-55 has been extensively revised (USDA, NRCS, 1986) and is not duplicated in this manual, since it is only recommended for preliminary calculations.

8.7 Land-Locked Retention

Watershed areas that drain to a central depression with no positive outlet are typical of karst topography and can be evaluated using a mass flow routing procedure to estimate flood elevations. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires good field measurements and a thorough understanding of local conditions. Since outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

The steps presented below for the mass routing procedure are illustrated by the example in Figure 8-12:

1. Obtain cumulative rainfall data for the 100-year frequency, 10-day duration design event from Figure 8-13.

2. Calculate the cumulative inflow to the land-locked retention basin using the rainfall data from Step 1 and an appropriate runoff procedure from Chapter 2. Plot the mass inflow to the retention basin (see Figure 8-12 for an example).

3. Develop the basin outflow from appropriate field measurements of hydraulic conductivity, taking into consideration worst-case water table conditions. Hydraulic conductivity should be established using in situ test methods, then compared to observed performance characteristics of the site. Plot the mass outflow as a straight line with a slope corresponding to worst-case saturated hydraulic conductivity estimated a professional engineer from geotechnical field tests outflow in inches/hour (see Figure 8-12 for an example).

4. Draw a line tangent to the mass inflow curve from Step 2, which has a slope parallel to the mass outflow line from Step 3.

5. Locate the point of tangency between the mass inflow curve of Step 2 and the tangent line drawn for Step 4. The distance from this point of tangency and the mass outflow line represents the maximum storage required for the design runoff.

6. Determine the flood elevation associated with the maximum storage volume determined in Step 5. Use this flood elevation to evaluate flood protection requirements of the project. The zero volume elevation should be established as the normal wet season water surface or water table elevation or the pit bottom, whichever is highest. This should be determined from a geotechnical test and analysis.

7. If the project area discharges into a stormwater system tributary to the land-locked depression, detention storage facilities are required to comply with the pre-development discharge requirements for the project (see Section 8.1).

Unless the retention basin is designed to have a permanent pool, including water budget calculations and provisions for preventing anaerobic conditions, relief structures should be provided to prevent standing water conditions. Depths greater than 12 feet should be equipped with an aerator to prevent thermal stratification.

8.8 Permanent Pool Facilities

MWS encourages the use of permanent pool detention/retention facilities designed for stormwater quality benefit. It also recognizes that wet detention ponds are preferable over dry detention ponds because of the added sediment storage flexibility provided by the permanent pool easing some maintenance activities (namely sediment removal is required less frequently). Provisions for safe slopes, safety benches (grading), access restriction to dangerous areas (fencing), weed control, mosquito control shelf, and aeration for prevention of anaerobic conditions should be considered. MWS may reject facility designs with the potential for becoming nuisances or health hazards.

8.8.1 Water Budget Calculations

Water budget calculations are required (see Volume 1, Section 6.8) for all permanent pool facilities and should consider performance for average annual and wet season conditions. The water budget should consider all significant inflows and outflows including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation, groundwater inflow, and outflow.

Average annual runoff may be computed using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Infiltration and exfiltration should be based on site-specific soils testing data. Evaporation may be approximated using the mean monthly pan evaporation or free water surface evaporation data presented in Table 8-10.

8.8.2 Example Problem

<u>Example 8-3. Water Budget Calculations</u>

A shallow basin with an average surface area of 3 acres and a bottom area of 2 acres is planned for construction at the outlet of a 100-acre watershed. The watershed is estimated to have a post-development runoff coefficient of 0.4. Site-specific soils testing indicates that the average infiltration rate is about 0.1 inch per hour. Determine for average annual conditions if the facility will function as a permanent pool.

1. From NOAA rainfall records, the average annual rainfall is about 50 inches.

2. From Table 8-10, the mean annual evaporation is 35 inches.

3. The average annual runoff is estimated as:

   RO = (0.3) (50 inches) (100 acres).
   RO = 1,500 acre-inches

4. The average annual evaporation is estimated as:

   EVAP = (35 inches) (3 acres)
   EVAP = 105 acre-inches

5. The average annual infiltration is estimated as:

$$INFIL = \left(\frac{0.1 in}{hr}\right)\left(\frac{24\ hr}{day}\right)\left(\frac{365\ days}{yr}\right)(2\ acres)$$

   INFIL = 1,752 acre-inches

6. Neglecting basin outflow and assuming no change in storage, the runoff (or inflow) less evaporation and infiltration losses is

   NET BUDGET = 1,500 - 105 - 1752
   NET BUDGET = -357 acre-inches

   and the proposed facility will not function as a permanent pool.

7. Revise pool design as follows;

   Average surface area, 2 acres

Bottom area, 1 acre

8. Recompute the evaporation and infiltration (Steps 4 and 5):

EVAP = (35 inches) (2 acres)

EVAP = 70 acre-inches

$$INFIL = \left(\frac{0.1\ in}{hr}\right)\left(\frac{24\ hr}{day}\right)\left(\frac{365\ days}{yr}\right)(1\ acre)$$

INFIL = 876 acre-inches

9. The revised runoff less evaporation and infiltration losses is

NET BUDGET = 1,500 - 70 - 876
NET BUDGET = 554 acre-inches

The revised facility is assumed to function as a permanent pool. To evaluate actual performance conditions, continuous simulation procedures should be applied.

8.9 Construction and Maintenance Considerations

An important step in the design process is identifying whether special provisions are warranted to properly construct or maintain proposed facilities (see Volume 1 Section 6.8). To assure acceptable performance and function, MWS discourages the design of stormwater detention/ retention facilities that may require excessive maintenance. The following maintenance activities should be considered:

1. Weed growth
2. Grass maintenance
3. Sediment removal
4. Slope deterioration
5. Mosquito control

Proper design may eliminate or reduce maintenance requirements by addressing the potential for problems to develop. Both weed growth and grass maintenance may be addressed by constructing side slopes that can be maintained using available power-driven equipment, such as tractor mowers. Sediment removal may be facilitated by constructing forebays or baffle boxes at the inlets to contain sediment for easy removal. Bank deterioration can be controlled with protective soil bioengineering techniques or lining or by limiting bank slopes. Mosquito control will not be a major problem if the permanent pool is designed with a 12-inch shelf at the edge.

8.10 Access Management

Access management may be required to prevent entry to facilities that present a hazard to children and, to a lesser extent, all persons. Fences may be required for detention areas where one or more of the following conditions exist:

1. Rapid stage changes (greater than 2-feet over 30 minutes) would make escape practically impossible for small children.

2. Water depths either exceed 2.5 feet for more than 24 hours or are permanently wet and have side slopes steeper than 4:1 (horizontal:vertical). This is for sites where it is impracticable to grade a mosquito bench, safety bench, and 6:1 (horizontal:vertical) littoral zone slope.

3. Side slopes equal or exceed 2:1 (horizontal to vertical).

Table 8-1
Size Categories For Dams In Tennessee

Category	Storage (acre-ft)	Height (ft)
Small	30 to <1,000	20 to <41
Intermediate	1,000 to 50,000	41 to 100
Large	>50,000	>100

Table 8-2
Minimum Freeboard Design Storms For Dams In Tennessee

Hazard Potential Category	Size	Freeboard Design Storm (6-hour)
Category 3 (Low)	Small	100 yr
	Intermediate	α PMP[a]
	Large	½ PMP
Category 2 (Significant)	Small	α PMP
	Intermediate	½ PMP
	Large	PMP
Category 1 (High)	Small	½ PMP
	Intermediate	PMP
	Large	PMP

[a]Probable maximum precipitation, defined as the precipitation resulting from a storm containing the most critical probable conditions.

Reference: Tennessee Department of Health and Environment (1973, 1987).

Table 8-3
Broad-Crested Weir Coefficient C Values
As A Function Of Weir Crest Breadth And Head

Measured Head, H[a]	Weir Crest Breadth (ft)										
(ft)	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

[a]Measured at least 2.5H upstream of the weir.

Reference: Brater and King (1976).

Table 8-4
Example Tabulation Of Storage Characteristics Curves

Stage	Storage[a]	Discharge[b]		$S - \frac{0}{2}\Delta t^{c}$	$S + \frac{0}{2}\Delta t^{c}$
(ft above NGVD)	(acre-ft)	**(cfs)**	(acre-ft/hr)[d]	(acre-ft)	(acre-ft)
100	0.05	0	0	0.05	0.05
101	0.3	15	1.24	0.20	0.40
102	0.8	35	2.89	0.56	1.04
103	1.6	63	5.21	1.17	2.03
104	2.8	95	7.85	2.15	3.45
105	4.4	143	11.82	3.41	5.39
106	6.6	200	16.53	5.22	7.98
107	10.0	275	22.73	8.11	11.89

[a]Obtained from Figure 8-3.

[b]Obtained from Figure 8-4.

[c]Δt = 10 min = 0.167 hour.

[d]1 cfs = 0.0826 acre-ft/hr.

Table 8-5
Storage Indication Method—Example 8-1 Calculations

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Time (min)	Inflow (cfs)	$\frac{(I_1 + I_2)\Delta t}{2}$ (acre-ft)	H_1 (ft)	$S_1 - \frac{0_1}{2}\Delta t$ (acre-ft)	$S_2 - \frac{0_2}{2}\Delta t$ (acre-ft)	H_2 (ft)	Outflow O (cfs)
0	0						
10	2	0.01	0.0	0.05	0.06	100.10	1
20	27	0.20	100.10	0.06	0.26	101.10	16
30	130	1.08	101.10	0.21	1.29	102.20	41
40	300	2.96	102.20	0.61	3.57	104.10	100
50	360	4.55	104.10	2.20	6.75	105.60	175
60	289	4.47	105.60	4.40	8.87	106.25	217
70	194	3.33	106.25	5.80	9.13	106.30	220
80	133	2.25	106.30	5.90	8.15	106.05	205
90	91	1.54	106.05	5.30	6.84	105.65	177
100	61	1.05	105.65	4.50	5.55	105.10	147
110	37	0.67	105.10	3.60	4.27	104.50	116
120	20	0.39	104.50	2.70	3.09	103.80	87
130	11	0.21	103.80	1.90	2.11	103.05	64
140	5	0.11	103.05	1.18	1.29	102.25	43
150	1	0.04	102.25	0.63	0.67	101.40	22
160	0	0.0	101.40	0.35	0.35	100.70	10

Note: $S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \textit{Equation } 8-12$

(column 6) = (column 5) + (column 3)

Table 8-6
Example 8-2 Runoff Hydrographs

	Pre-Development Runoff		Post-Development Runoff	
Time (Hours)	2-Year (cfs)	10-Year (cfs)	2-Year (cfs)	10-Year (cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190	250
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1
1.3	7	12	0	0

Table 8-7
Example 8-2 Stage-Discharge-Storage Data

Stage (ft)	Q (cfs)	S (acre-ft)	$S_1 - \frac{0}{2}\Delta t$ (acre-ft)	$S_1 - \frac{0}{2}\Delta t$ (acre-ft)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52
2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.91
4.0	100	1.40	1.81	0.99
4.3	110	1.52	1.97	1.06
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60
6.2	190	2.46	3.25	1.68
6.4	200	2.58	3.41	1.76
6.6	210	2.71	3.57	1.84
6.8	220	2.83	3.74	1.92
7.0	230	2.95	3.90	2.00
7.2	240	3.08	4.07	2.09
7.4	250	3.21	4.24	2.017

Notes:
1. Broad-crested weir length = 4 feet.
2. Discharge coefficient = 3.1 (assumed constant).

Table 8-8
Example 8-2 Reservoir Routing For The 2-Year Storm

Time	Inflow	$\frac{(I_1 + I_2)\Delta t}{2}$	H_1	$S_1 - \frac{O_1}{2}\Delta t$	$S_2 + \frac{O_2}{2}\Delta t$	H_2	Outflow O
(hours)	(cfs)	(acre-ft)	(ft)	(acre-ft)	(acre-ft)	(ft)	(cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	38	0.16	0.0	0.00	0.16	0.43	3
0.2	125	0.67	0.43	0.10	0.77	2.03	36
0.3	190	1.30	2.03	0.50	1.80	4.00	99
0.4	125	1.30	4.00	0.99	2.29	4.80	130
0.5	70	0.81	4.80	1.21	2.02	4.40	114
0.6	39	0.45	4.40	1.12	1.57	3.60	85
0.7	22	0.25	3.60	0.87	1.12	2.70	55
0.8	12	0.14	2.70	0.65	0.79	2.08	37
0.9	7	0.08	2.08	0.50	0.58	1.70	27
1.0	4	0.05	1.70	0.42	0.47	1.30	18
1.1	2	0.02	1.30	0.32	0.34	1.00	12
1.2	0	0.01	1.00	0.25	0.26	0.70	7
1.3	0	0.00	0.70	0.15	0.15	0.40	3

Note: $S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right]$ *Equation* 8 − 12

(column 6) = (column 5) + (column 3)

Allowable outflow = 150 cfs (see Table 8-6).

Table 8-9
Example 8-2 Reservoir Routing For The 10-Year Storm

Time (hours)	Inflow (cfs)	$\frac{(I_1 + I_2)\Delta t}{2}$ (acre-ft)	H_1 (ft)	$S_1 - \frac{O_1}{2}\Delta t$ (acre-ft)	$S_2 + \frac{O_2}{2}\Delta t$ (acre-ft)	H_2 (ft)	Outflow O (cfs)
0.0	0	0.00	0.00	0.00	0.00	0.00	0
0.1	50	0.21	0.21	0.00	0.21	0.40	3
0.2	178	0.94	0.40	0.08	1.02	2.50	49
0.3	250	1.77	2.50	0.60	2.37	4.90	134
0.4	165	1.71	4.90	1.26	2.97	5.80	173
0.5	90	1.05	5.80	1.30	2.35	4.95	137
0.6	50	0.58	4.95	1.25	1.83	4.10	103
0.7	29	0.33	4.10	1.00	1.33	3.10	68
0.8	16	0.19	3.10	0.75	0.94	2.40	46
0.9	9	0.10	2.40	0.59	0.69	1.90	32
1.0	5	0.06	1.90	0.44	0.50	1.40	21
1.1	3	0.03	1.40	0.33	0.36	1.20	16
1.2	1	0.02	1.20	0.28	0.30	0.90	11
1.3	0	0.00	0.90	0.22	0.22	0.60	6

Note: $S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \textit{Equation } 8-12$

(column 6) = (column 5) + (column 3)

Allowable outflow = 200 cfs (see Table 8-6).

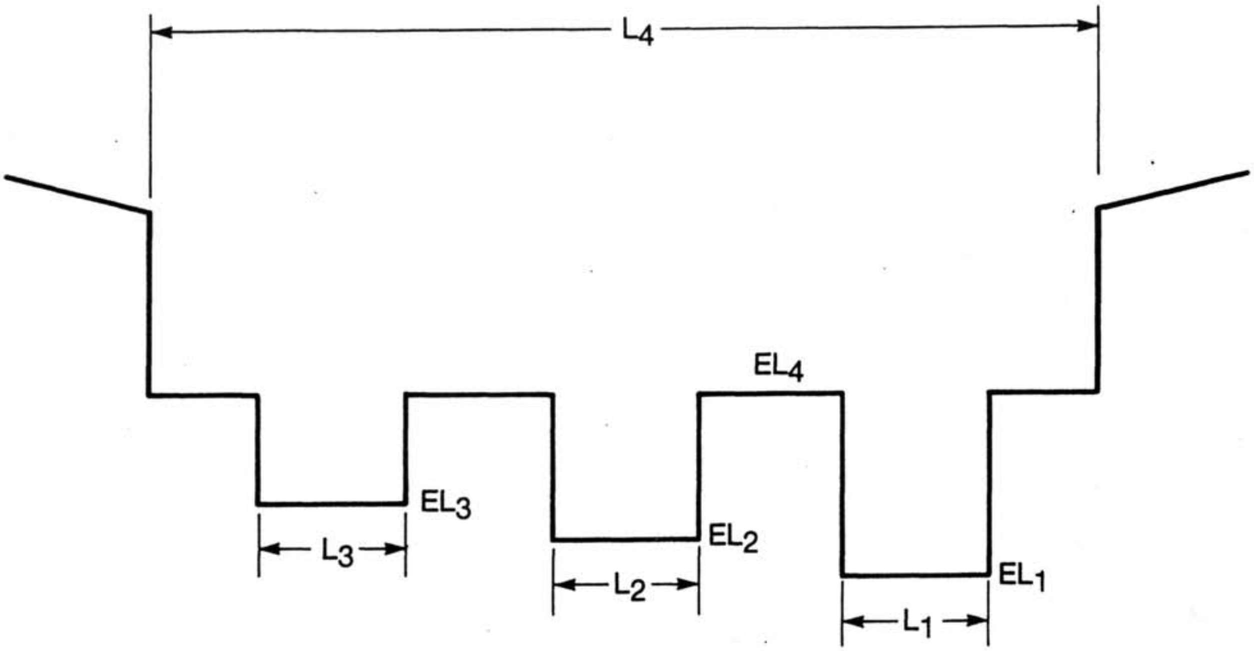
Table 8-10
Mean Monthly Evaporation Data

Month	Pan Evaporation[a] (inches)	Free Water Surface Evaporation[b] (inches)
January	1.10	0.85
February	1.69	1.30
March	3.10	2.39
April	4.79	3.69
May	5.47	4.21
June	6.28	4.84
July	6.38	4.91
August	5.70	4.39
September	4.24	3.26
October	3.24	2.49
November	1.92	1.48
December	1.60	1.23
Mean Annual	45.51	35.04

[a]Based on recorded data from 1977 through 1986 at Cheatham Lock and Dam Station.

[b]Pan coefficient is 0.77.

Reference: U.S. Department of Commerce, NOAA (1977-1988).



TYPICAL ELEVATION

For Water Surface Elevation From	To	Discharge Is Computed By
EL_1	EL_2	$Q_1 = C_1L_1 (EL_{WS} - EL_1)^{3/2}$
EL_2	EL_3	$Q_2 = C_2L_2 (EL_{WS} - EL_2)^{3/2} + Q_1$
EL_3	EL_4	$Q_3 = C_3L_3 (EL_{WS} - EL_3)^{3/2} + Q_2$
EL_4	—	$Q_4 = C4 (L4 - L_3 - L_2 - L_1) (EL_{WS} - EL_4)^{3/2} + Q_3$

where:

L_i = Length of weir opening i, in feet

EL_i = Invert elevation of weir opening i, in feet

Q_i = Cumulative discharge above weir opening i, in cfs

EL_{WS} = Elevation of the water surface, in feet

C_i = Weir discharge coefficient

Figure 8-1
Example Multi-Stage Control Structure

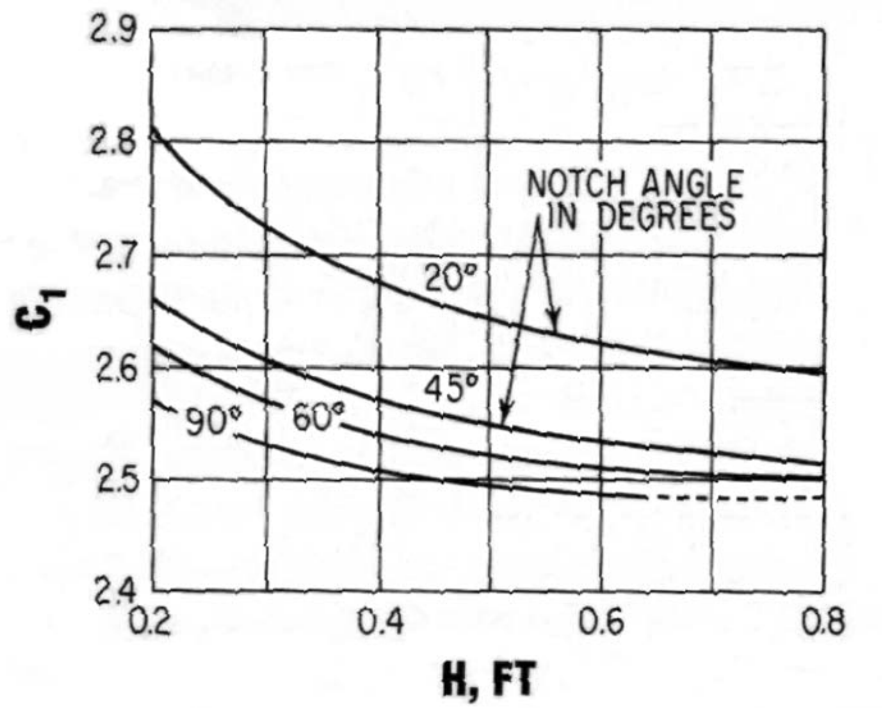


Figure 8-2
Sharp-Crested "V" Notch Weir Discharge Coefficients

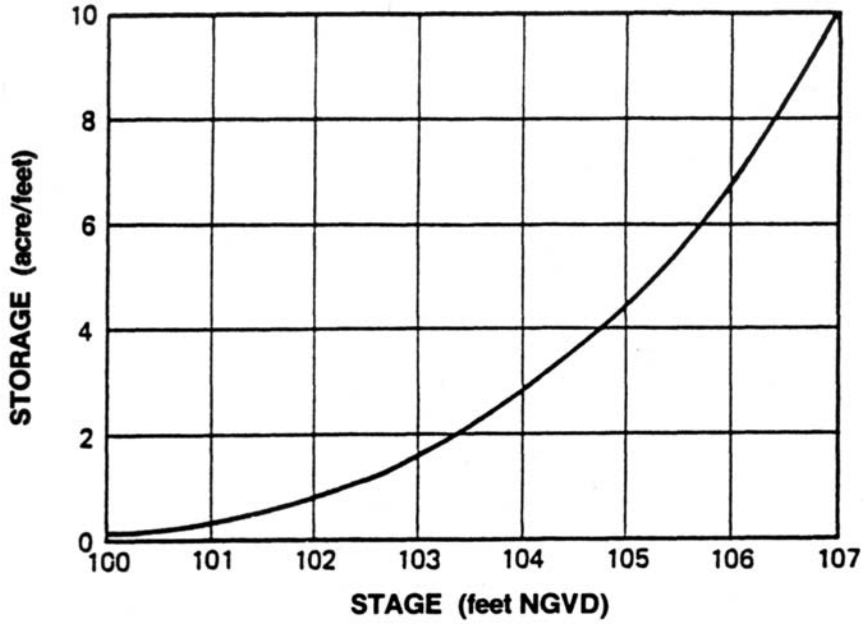


Figure 8-3
Example Stage-Storage Curve

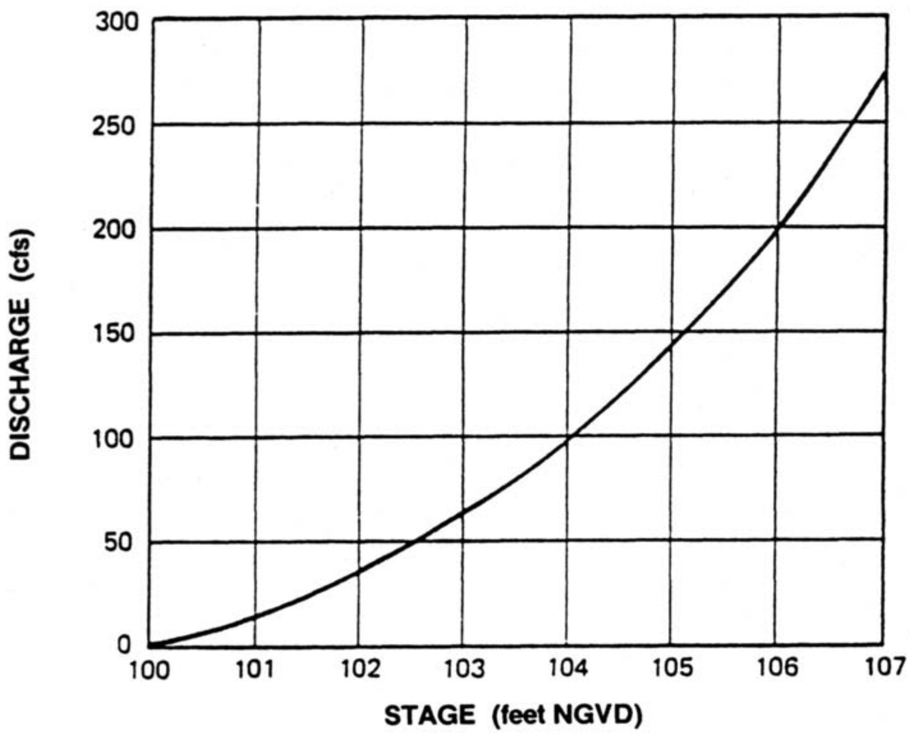


Figure 8-4
Example Stage-Discharge Curve

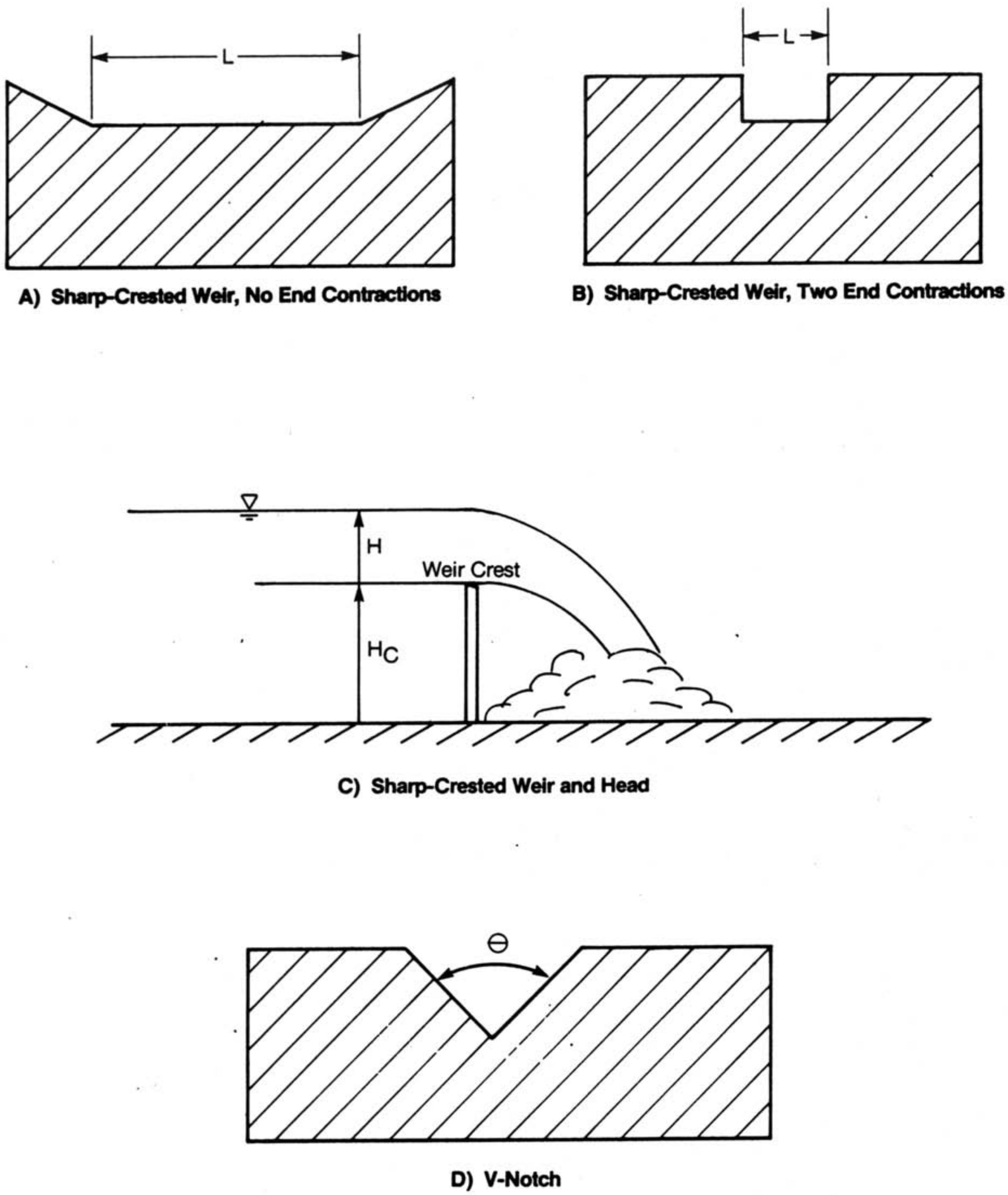
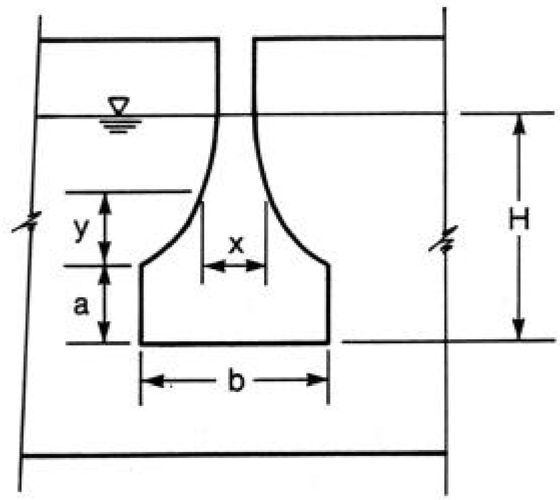
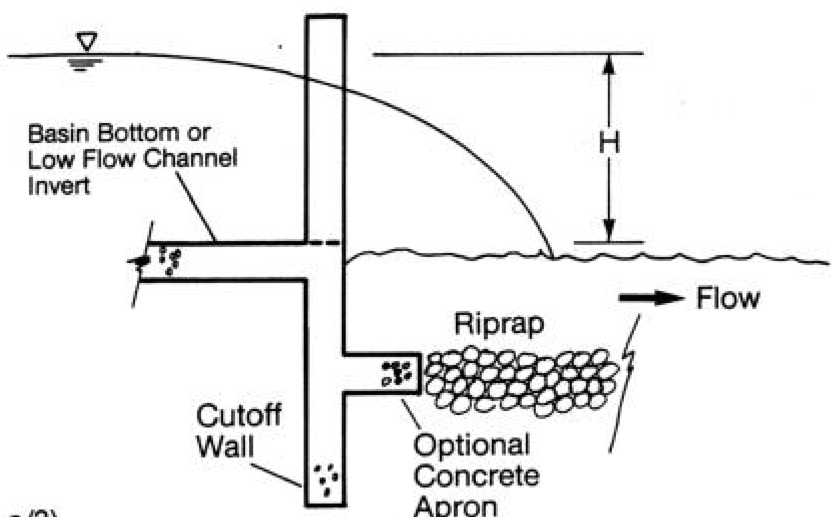


Figure 8-5
Illustrations of Weir Flow Control Structures



ELEVATION



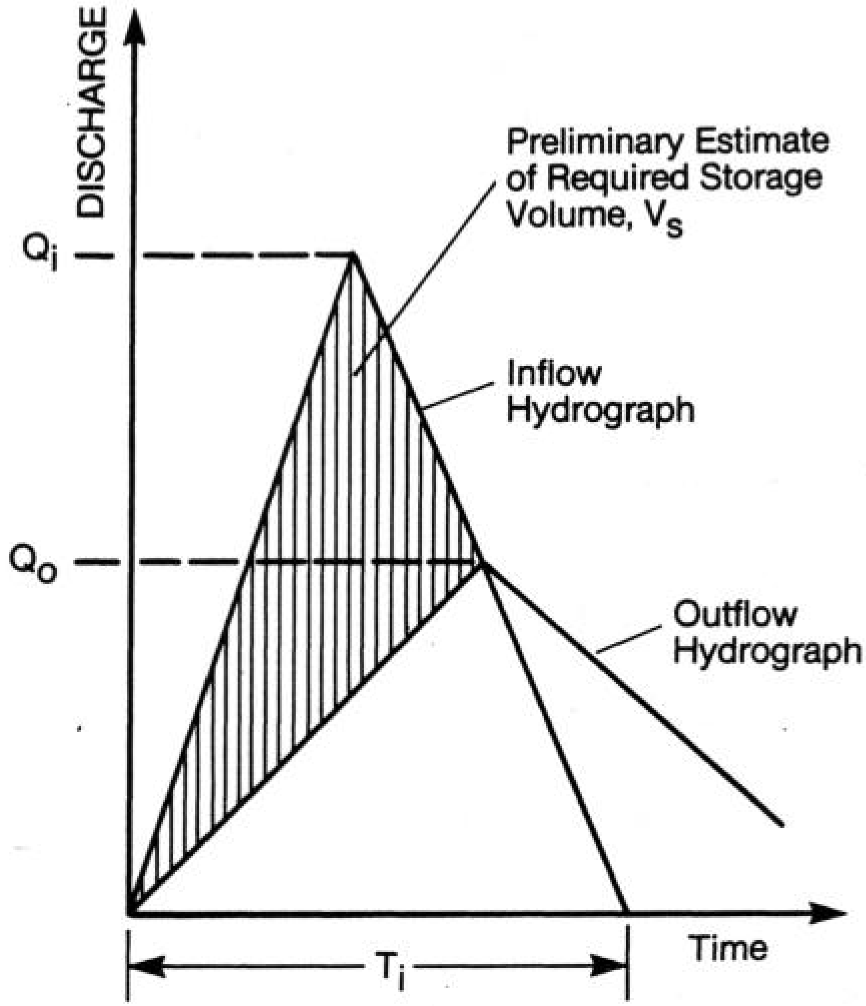
$$Q = 4.97\, a^{1/2} b\, (H - a/3)$$

$$x/b = 1 - 1/\pi\, (\arctan \sqrt{y/a})$$

SECTION

Reference: Sandvik (1985).

Figure 8-6
Dimensions Used for Design of a Proportional Weir



$$V_s = 0.5\, T_i\, (Q_i - Q_o)$$

where:

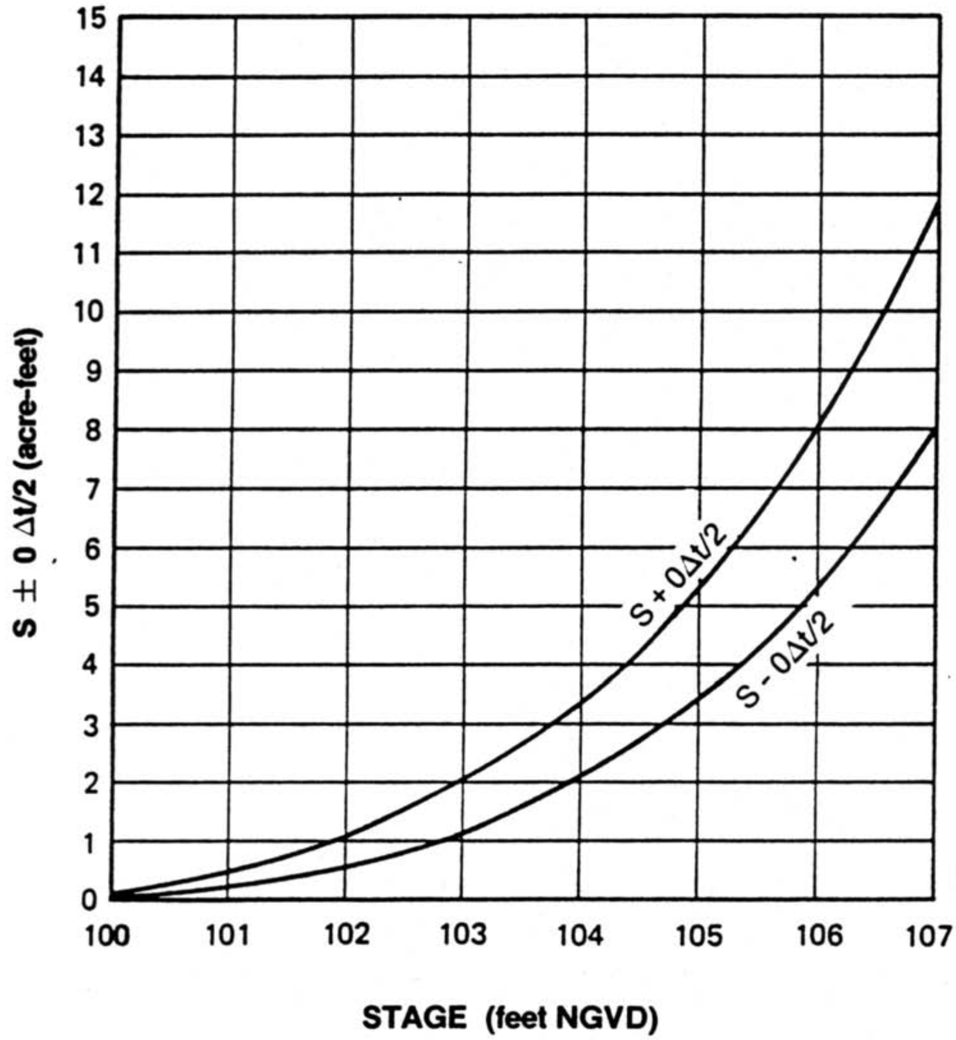
V_s = Storage volume estimate

Q_i = Peak inflow rate

Q_o = Peak outflow rate

T_i = Duration of basin inflow

Figure 8-7
Triangular Shaped Hydrographs for Preliminary Estimate
of Required Storage Volume



Note: Data presented in Table 8-4.

Figure 8-8
Example Storage Characteristics Curves

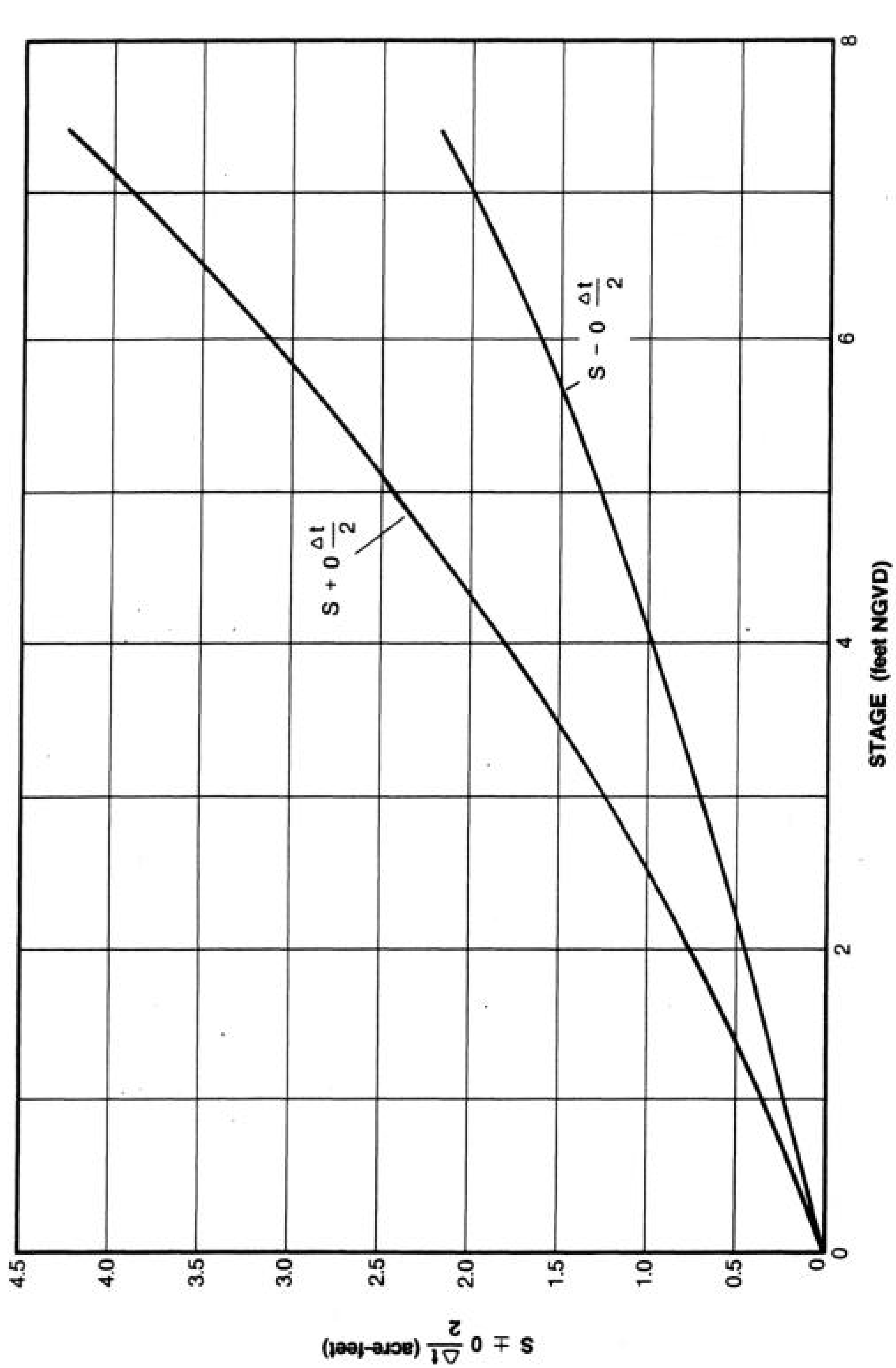


Figure 8-9
Example 8-2 Storage Characteristics Curves

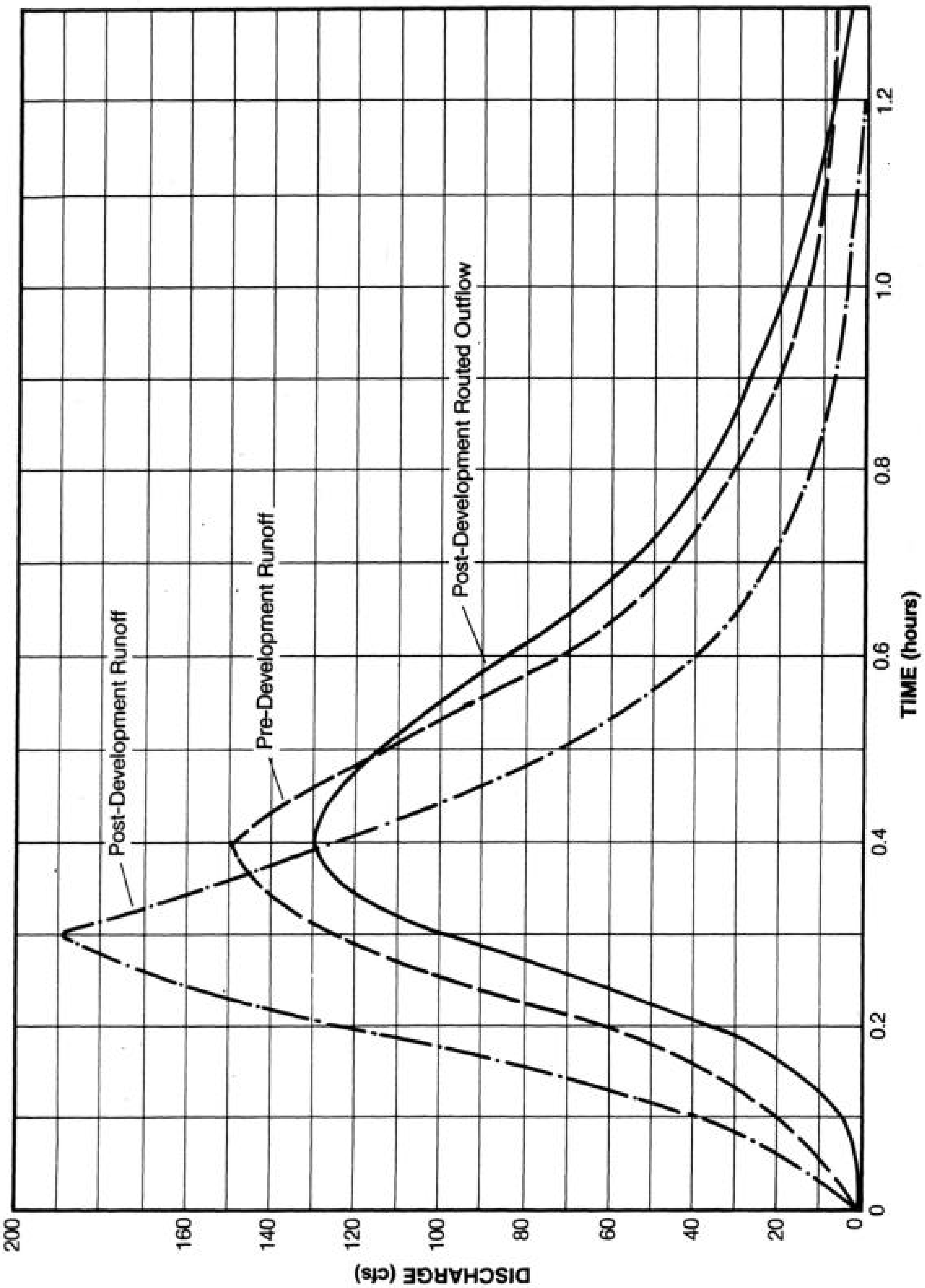


Figure 8-10
Example 8-2 Pre- and Post-Development Runoff Hydrographs
and Routed Outflow Hydrograph for the 2-Year Design Storm

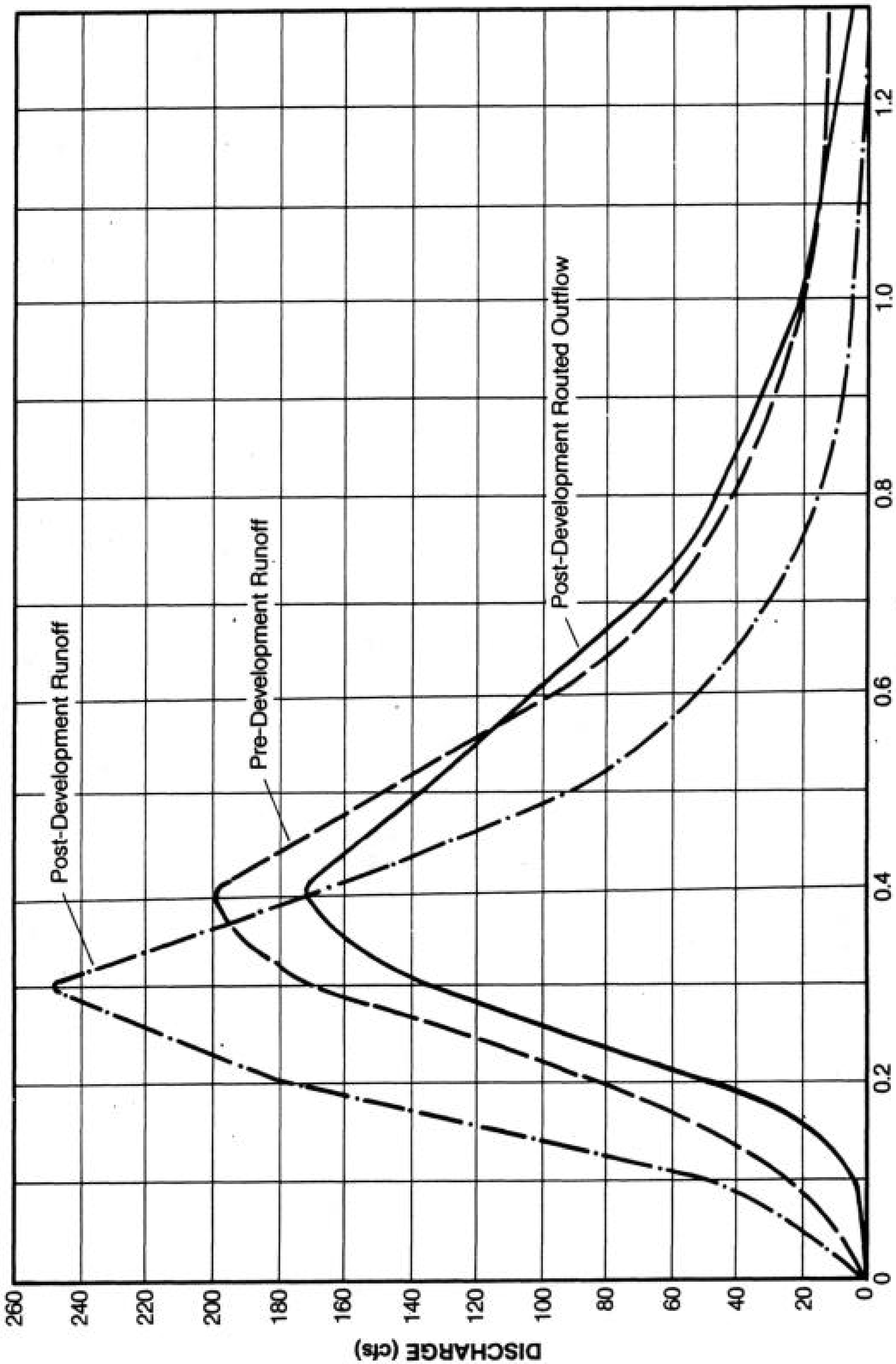


Figure 8-11
Example 8-2 Pre- and Post-Development Runoff Hydrographs
and Routed Outflow Hydrograph for the 10-Year Design Storm

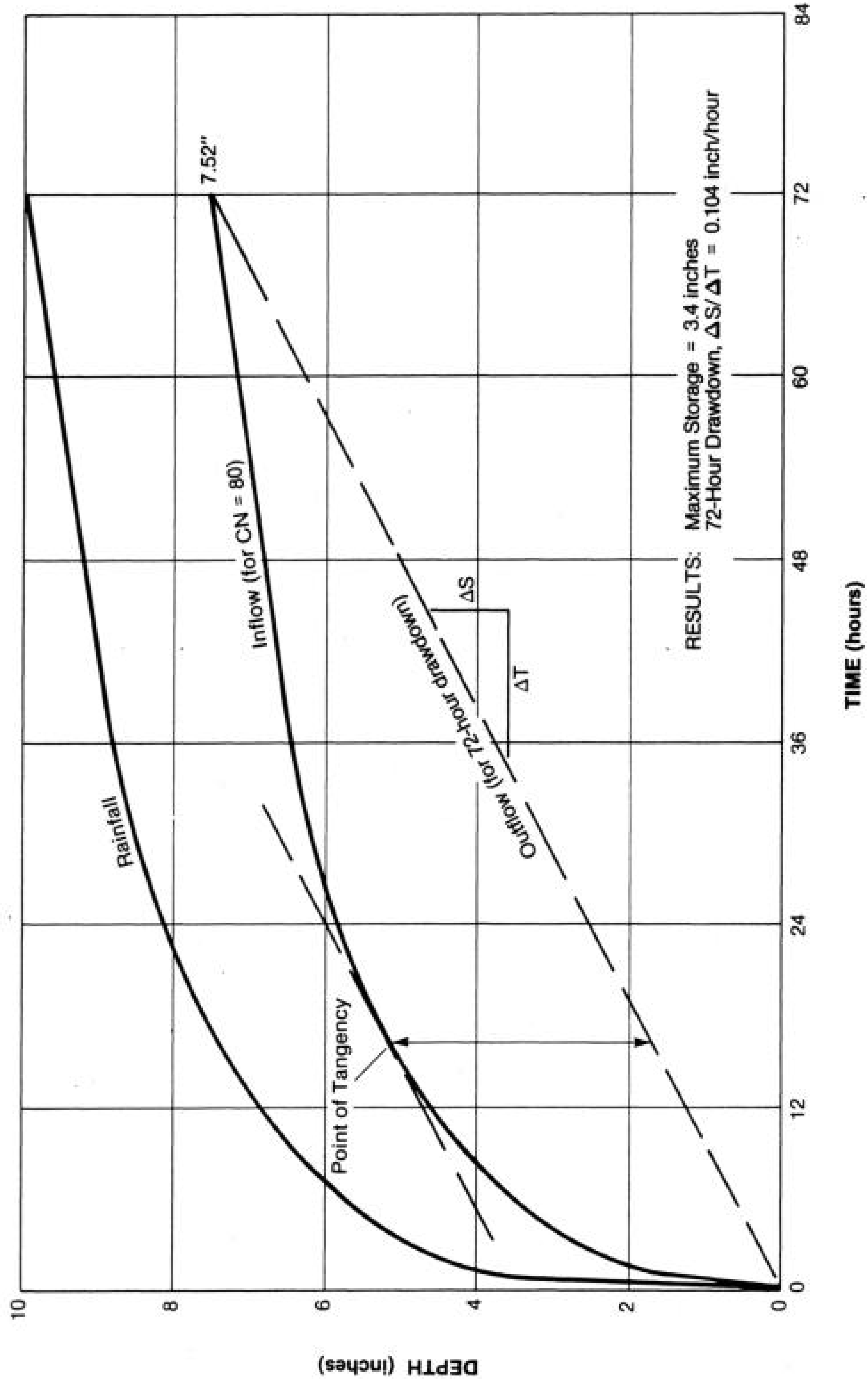


Figure 8-12
Mass Routing Example for Land-Locked Retention Areas

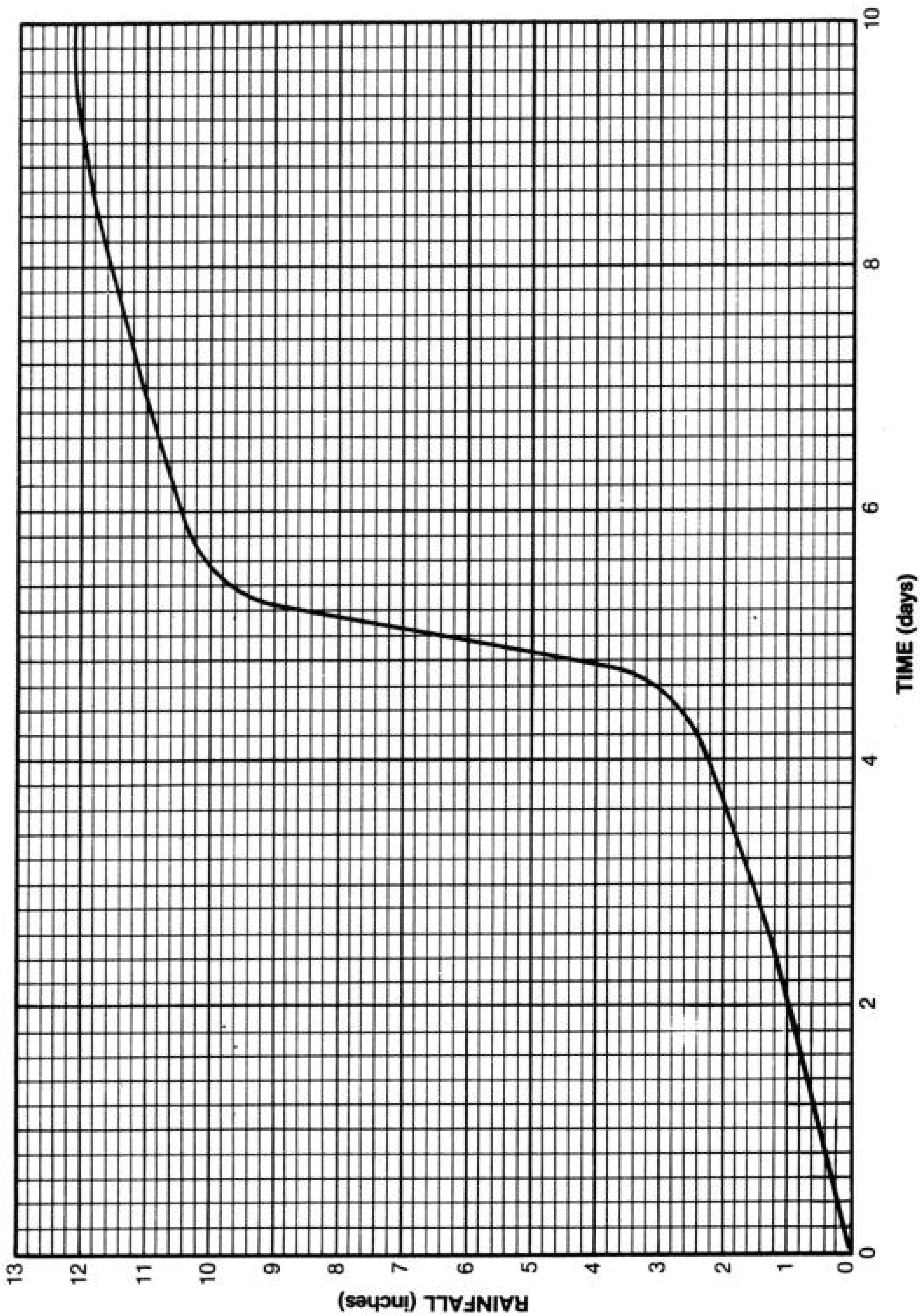


Figure 8-13
Cumulative Rainfall Data for the 100-Year,
10-Day Design Storm