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CHAPTER THIRTY-FIVE

STORAGE FACILITIES

35-1.0 INTRODUCTION

35-1.01 Overview

The traditional design of a storm-drainage system has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As an area urbanizes, this type of design may result in major drainage and flooding problems downstream. The engineering community is now more conscious of the quality of the environment and the impact that uncontrolled increases in runoff can have on our customers. Under favorable conditions, the temporary storage of some of the storm-runoff can decrease downstream flows and often the cost of the downstream conveyance system. This Chapter provides design criteria for a detention or retention storage basin as well as procedures for performing preliminary and final sizing and reservoir-routing calculations.

35-1.02 Safety Considerations

Ponding of water for a significant period of time, at a relatively shallow depth, may introduce an additional risk factor for property damage, personal injury, or loss of life. A storage facility in a location that is easily accessible to the public should be provided with warning signs and fencing adequate to prevent entry onto the site by unauthorized persons. A storage facility located adjacent to a roadway should be provided with an adequate clear zone to minimize the accidental entry of an errant vehicle.

35-1.03 Detention and Retention

An urban stormwater storage facility is referred to as either a detention or retention facility. For this Chapter, these are defined as follows.

1. Detention. A detention facility is that designed to reduce the peak discharge and only detain runoff for a short period of time. Detention storage involves detaining or slowing runoff and then releasing it. A detention basin has a positive outlet that completely empties all runoff between storms. The excavation of a detention facility may sometimes extend below the water table or outlet level where the bottom is sealed by sedimentation.

This is referred to as a detention pond or wet-bottom detention basin. The detention pond also has a positive outlet and releases all temporary storage.

A detention facility may be designed to contain a permanent pool of water. The use of a dry-bottom detention pond is recommended for an INDOT project. Because most of the design procedures are the same for a wet- or a dry-bottom detention facility, the term storage facility will be used in this Chapter to mean either.

2. Retention. A retention facility retains runoff for an indefinite amount of time and has no positive outlet. Runoff is removed only by infiltration through a porous bottom or by evaporation. A retention pond or lake is an example of a retention facility that may be built in a development, and, may enhance the overall project. A retention basin is designed to drain into the groundwater table. This is not addressed herein.

A storage facility is most often small in terms of storage capacity and dam height, and will serve a single outfall from a watershed of a few hectares. A very small facility may be contained in a parking lot or other on-site facility. Although the same principles apply to each storage facility, Section 35-10.0 more-specifically relates to a smaller installation.

If other procedures are needed for the design of a detention or retention facility, these will be specified.

35-1.04 Computer Programs

Routing calculations needed to design a storage facility, although not extremely complex, are time-consuming and repetitive. To assist with these calculations, there are many available reservoir-routing computer programs. If the watershed draining into a storage facility is greater than 0.8 ha, design should be based upon reservoir-routing methods which develop hydrographs for both inflow and outflow. A smaller basin may be analyzed using the storage-indication method or the Rational Method.

35-2.0 USES

35-2.01 Introduction

The use of a storage facility for stormwater management has increased in recent years. Controlling the quantity of stormwater using a storage facility can provide the potential benefits as follows:

1. prevention or reduction of peak runoff rate increases caused by urban development;

2. mitigation of downstream drainage capacity problems;
3. reduction or elimination of the need for downstream outfall improvements; and
4. maintenance of historically low flow rates by controlled discharge from storage.

35-2.02 Objectives

The objectives for managing stormwater quantity by a storage facility are based on limiting peak runoff rates to match either or both of the values as follows:

1. historic rates for specific design conditions (i.e., post-development peak equals pre-development peak for a particular frequency of occurrence); or

2. non-hazardous discharge capacity of the downstream drainage system.

For a watershed without an adequate outfall, the total volume of runoff is critical. A storage facility is used to store the stormwater due to increases in volume and control-discharge rates.

35-3.0 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 35-3A will be used. These symbols were selected because of their wide use in technical publications. The same symbol may be used in existing publications for more than one definition. Where this occurs in this Chapter, the symbol will be defined where it occurs in the text or equations.

35-4.0 DESIGN CRITERIA

35-4.01 General Criteria

Storage may be developed in a depressed area in a parking lot, road embankment, freeway interchange, or a small lake, pond, or depression within an urban development. The utility of a storage facility depends on the amount of storage, its location within the system, and its operational characteristics. An analysis of such a storage facility should consist of comparing the design flow at a point or points downstream of the proposed storage site, with or without storage. Other flows in excess of the design flow that may be expected to pass through the storage facility may be required in the analysis (i.e., 100-year flood). The design criteria for a storage facility should include the following:

1. release rate;

2. storage volume;
3. grading and depth requirements;
4. outlet works; and
5. location.

35-4.02 Release Rate

At a minimum, a storage facility should be designed to detain the 50-year, post-development peak runoff and release it at the 10-year, pre-developed peak runoff rate. If applicable, it should also satisfy the more-restrictive requirements that may be imposed by a local jurisdiction. An emergency overflow capable of accommodating the 100-year discharge may be required in a facility using a dam.

35-4.03 Storage

Routing calculations must be used to demonstrate that the facility-storage volume is adequate to provide the required detention. If sedimentation during construction causes loss of detention volume, design dimensions should be restored before completion of the project. For a detention basin, all detention volume should be drained within the average period between storm events, or 72 h.

35-4.04 Grading and Depth

35-4.04(01) General

The construction of a storage facility requires excavation or placement of an earthen embankment to obtain sufficient storage volume. The embankment should be of less than 2 m height. A vegetated embankment should have side slopes not steeper than 3H:1V. A riprap-protected embankment should not be steeper than 2H:1V. An excavated storage facility should not have an operating design pool depth of greater than 1.5 m. unless specifically approved by the Hydraulics Team.

A minimum freeboard of 0.3 m above the 100-year-storm high-water elevation should be provided.

Other considerations in setting the depth 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 should also be considered in an urban area. Fencing of a basin is addressed in Section 35-14.0.

35-4.04(02) Dry-Bottom Detention

The area above the normal high-water elevation of a storage facility should be sloped toward the facility to allow drainage and to prevent standing water. Finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of a storage facility should be graded toward the outlet to prevent standing-water conditions. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flow and to prevent standing-water conditions.

35-4.04(03) Wet-Bottom Detention

The maximum depth of a permanent storage facility will be determined based on site conditions and design constraints. If the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds should be considered. A depth of 2.0 m is reasonable.

35-4.05 Outlet Works

Outlet works selected for a storage facility include a principal spillway or an emergency overflow and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of a drop inlet, pipe, weir, or orifice. A slotted-riser pipe is discouraged because of clogging problems. A curb opening may be used for parking-lot storage. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet.

An orifice outlet takes the form of a restriction of 300 mm or less placed in a larger pipe. The preferred design for such an outlet consists of placing a smaller pipe on the flowline of a larger pipe. The smaller pipe will be the required size to achieve the desired detention results and is of approximately 300 mm length. Grout is placed around the smaller pipe to fill the area of the larger. This type of construction provides for adequate maintenance and is more durable than a single constrictor plate.

35-5.0 GENERAL PROCEDURE

35-5.01 Data Needs

The data required to complete the storage design and routing calculations is as follows:

1. inflow hydrograph for each selected design storm;

2. stage-storage curve for the proposed storage facility (see Figure 35-5A for an example). For a large storage volume, use hectares/meter, otherwise use cubic meters; and

3. stage-discharge curve for each outlet-control structure (see Figure 35-5B for an example).

Using these data, a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet configurations until the desired outflow hydrograph is achieved. See the example problem in Section 35-8.0.

35-5.02 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are developed using a topographic map and one of the following formulas, the average-end area, frustum of a pyramid, or prismoidal. A storage basin may be irregular in shape to blend well with the surrounding terrain and to improve aesthetics. Therefore, the average-end-area formula is preferred as the method to be used for a non-geometric area. The double-end-area formula is expressed as follows:

$$V_{1,2} = \frac{d\left(A_1 + A_2\right)}{2} \qquad \text{(Equation 35-5.1)}$$

Where:
- $V_{1,2}$ = storage volume, m^3, between elevations 1 and 2
- A_1 = surface area at elevation 1, m^2
- A_2 = surface area at elevation 2, m^2
- d = change in elevation between points 1 and 2, m

The frustum of a pyramid is expressed as follows:

$$V = \frac{d\left[A_1 + \left(A_1 A_2\right)^{0.5} + A_2\right]}{3} \qquad \text{(Equation 35-5.2)}$$

Where:
- V = volume of frustum of a pyramid, m^3
- d = change in elevation between points 1 and 2, m
- A_1 = surface area at elevation 1, m^2
- A_2 = surface area at elevation 2, m^2

The prismoidal formula for a trapezoidal basin is expressed as follows:

$$V = LWD + \frac{D^2(L+W)}{Z} + \frac{4D^3}{3Z^2} \qquad \text{(Equation 35-5.3)}$$

Where:

V	=	volume of trapezoidal basin, m^3
L	=	length of basin at base, m
W	=	width of basin at base, m
D	=	depth of basin, m
Z	=	side slope factor, ratio of horizontal to vertical

35-5.03 Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A storage facility has two spillways: principal and emergency. The principal spillway is designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir, or other appropriate outlet can be used for the principal spillway or outlet. Tailwater influences and structure losses must be considered in developing discharge curves.

The emergency spillway, when needed, is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed taking into account the potential threat to downstream life and property if the storage facility were to fail. The stage-discharge curve should take into account the discharge characteristics of both the principal and emergency spillways.

35-5.04 Procedure

The procedure for using the above data in the design of a storage facility is described below.

1. Compute inflow hydrographs for runoff from the design storm using the procedure outlined in Chapter Twenty-nine.

2. Perform preliminary calculations to evaluate detention-storage requirements for the hydrographs from Step 1 (see Section 35-7.0 for recommended methods).

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. Ascertain that tailwater effects have been considered.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage-routing equations. If the routed post-development peak discharge from the 50-year design storm exceeds the pre-development 10-year peak discharge, or if the peak stage varies significantly from the estimated peak stage from Step 4, revise the estimated volume and return to Step 3.

6. Where required, consider emergency overflow from runoff due to the 100-year design storm and established freeboard requirements.

7. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream.

This procedure can involve a significant number of reservoir-routing calculations to obtain the desired results.

35-5.05 Computer Procedures

A number of commercial computer software packages exist which automate a number of the steps described above. Although these programs can greatly accelerate the design process, they should be used with caution. The output from these programs should be reviewed considering sound engineering judgment. Except in modeling a drainage area of less than 0.8 ha, the programs must be capable of developing hydrographs for both inflow and outflow. For an area of less than 0.8 ha, the Rational Method is acceptable for generating the inflow hydrographs.

35-6.0 OUTLET HYDRAULICS

35-6.01 Outlets

Sharp-crested-weir flow equations for a no-end contraction, a two-end contraction, and submerged discharge conditions are provided below, followed by equations for a broad-crested weir, V-notch weir, proportional weir, orifice, or a combination of these facilities. If a culvert is used as an outlet works, the procedure described in Chapter Thirty-one should be used to develop stage-discharge data. In analyzing release rates, tailwater influences must be considered to determine the effective head on each outlet. A slotted riser-pipe outlet facility should be avoided.

35-6.02 Sharp-Crested Weir

A sharp-crested weir with no-end contractions is illustrated in Figure 35-6A. The discharge equation for this configuration is as follows (Chow, 1959):

$$Q = LH^{1.5}\left(1.805 + \frac{0.221H}{H_c}\right) \qquad \text{(Equation 35-6.1)}$$

Where:
- Q = discharge, m^3/s
- H = head above weir crest excluding velocity head, m
- H_c = height of weir crest above channel bottom, m
- L = horizontal weir length, m

A sharp-crested weir with two-end contractions is illustrated in Figure 35-6B. The discharge equation for this configuration is as follows (Chow, 1959):

$$Q = (L - 0.2H)H^{1.5}\left(1.805 + \frac{0.221H}{H_c}\right) \qquad \text{(Equation 35-6.2)}$$

Where the variables are the same as for Equation 35-6.4.

Figure 35-6C illustrates a sharp-crested weir and head.

A sharp-crested weir will be affected by submergence if the tailwater rises above the weir-crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is as follows (Brater and King, 1976):

$$Q_S = Q_f\left[1 - \left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \qquad \text{(Equation 35-6.3)}$$

Where:
- Q_S = submergence flow, m^3/s
- Q_f = free flow, m^3/s
- H_1 = upstream head above crest, m
- H_2 = downstream head above crest, m

35-6.03 Broad-Crested Weir

The equation used for the broad-crested weir is as follows (Brater and King, 1976):

$$Q = CLH^{1.5} \quad \text{(Equation 35-6.4)}$$

Where:
- Q = discharge, m^3/s
- C = broad-crested weir coefficient
- L = broad-crested weir length, m
- H = head above weir crest, m

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 yields the maximum C value of 1.704. For sharp corners on the broad-crested weir, a minimum C value of 1.435 should be used. Additional information on C value as a function of weir-crest breadth and head is shown in Figure 35-6D.

35-6.04 V-Notch Weir

The discharge through a V-notch weir can be calculated from the equation as follows (Brater and King, 1976):

$$Q = 1.38 \tan(\varphi/2) H^{2.5} \quad \text{(Equation 35-6.5)}$$

Where:
- Q = discharge, m^3/s
- φ = angle of V-notch, deg
- H = head on apex of notch, m

35-6.05 Proportional Weir

Although more complex to design and construct, a proportional weir may significantly reduce the required storage 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 are as follows (Sandvik, 1985):

$$Q = 2.74 a^{0.5} b \left(H - \frac{a}{3} \right) \quad \text{(Equation 35-6.6)}$$

$$\frac{x}{b} = \left(1 - \frac{1}{3.17} \right) \left[\arctan \left(\frac{y}{a} \right) \right]^{0.5} \quad \text{(Equation 35-6.7)}$$

Where Q = discharge, m^3/s

Dimensions a, b, H, x, and y are shown in Figure 35-6E.

35-6.06 Orifice

A pipe smaller than 300 mm diameter may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions, the formula that applies is as follows:

$$Q = 0.6A(2gH)^{0.5} \qquad \text{(Equation 35-6.8)}$$

Where:
- Q = discharge, m^3/s
- A = cross-section area of pipe, m^2
- g = acceleration due to gravity, 9.81 m/s^2
- D = diameter of pipe, m
- H = head on pipe, from the center of pipe to the water surface, m

Where the tailwater is higher than the center of the opening, the head is calculated as the difference in water-surface elevations.

35-7.0 PRELIMINARY DETENTION CALCULATIONS

35-7.01 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 35-7A.

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

$$V_S = 0.5T_i(Q_i - Q_o) \qquad \text{(Equation 35-7.1)}$$

Where:
- V_S = storage-volume estimate, m^3
- Q_i = peak inflow rate, m^3/s
- Q_o = peak outflow rate, m^3/s
- T_i = duration of basin inflow, s

Consistent units may be used for Equation 35-7.1.

35-7.02 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained through the following regression-equation procedure (Wycoff & Singh, 1986).

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 equation as follows:

$$\frac{V_S}{V_r} = 1.291\left(1 - \frac{Q_o}{Q_i}\right)^{0.753}\left(\frac{t_p}{t_b}\right)^{0.411} \qquad \text{(Equation 35-7.2)}$$

Where:
- V_S = volume of storage, m^3
- V_r = volume of runoff, m^3
- Q_o = outflow peak flow, m^3/s
- Q_i = inflow peak flow, m^3/s
- t_b = time base of the inflow hydrograph, h (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak.)
- t_p = time to peak of the inflow hydrograph, h

35-7.03 Peak-Flow Reduction

A preliminary estimate of the potential peak-flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine the following.

   a. volume of runoff, V_r
   b. peak flow rate of the inflow hydrograph, Q_i
   c. time base of the inflow hydrograph, t_b
   d. time to peak of the inflow hydrograph, t_p
   e. storage volume, V_S

2. Calculate a preliminary estimate of the potential peak-flow reduction for the selected storage volume using the equation as follows (Singh, 1976).

$$\frac{Q_o}{Q_i} = 1 - 0.712\left(\frac{V_S}{V_r}\right)^{1.328}\left(\frac{t_b}{t_p}\right)^{0.546} \qquad \text{(Equation 35-7.3)}$$

Where:
- Q_o = outflow peak flow, m^3/s
- Q_i = inflow peak flow, m^3/s
- V_S = volume of storage, m^3
- V_r = volume of runoff, m^3
- t_b = time base of the inflow hydrograph, h (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5% of the peak.)
- t_p = time to peak of the inflow hydrograph, h

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

35-7.04 Preliminary Basin Dimensions

The following applies.

1. Plot the control-structure location on a contour map.

2. Select a desired depth of ponding for the design storm.

3. Divide the estimated storage volume needed by the desired depth to obtain the surface area required of the reservoir.

4. Based on site conditions and contours, estimate the geometric shapes required to provide the estimated reservoir surface area.

35-8.0 ROUTING CALCULATIONS

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing).

1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown in Figures 35-8A and 35-8B, respectively.

2. Select a routing time period, Δt, to provide at least five points on the rising limb of the inflow hydrograph ($\Delta t < T_c/5$).

3. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of $SK(O_1/2)\Delta t$ versus stage. An example tabulation of storage-characteristics curve data is shown in Figure 35-8C.

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, $S_1 - (O_1/2)\Delta t$, can be determined from the appropriate storage-characteristics curve (example shown in Figure 35-8D).

5. Determine the value of $S_2 + (O_2/2)\Delta t$ from the equation as follows:

$$S_2 + \frac{O_2 \Delta t}{2} = \left(S_1 - \frac{O_1 \Delta t}{2} \right) + \frac{(I_1 + I_2)\Delta t}{2} \qquad \text{(Equation 35-8.1)}$$

Where:

S_2	=	storage volume at time 2, m^3
O_2	=	outflow rate at time 2, m^3/s
Δt	=	routing time period, s
S_1	=	storage volume at time 1, m^3
O_1	=	outflow rate at time 1, m^3/s
I_1	=	inflow rate at time 1, m^3/s
I_2	=	inflow rate at time 2, m^3/s

Other consistent units are equally appropriate.

6. Enter the storage-characteristics curve at the calculated value of $S_2 + (O_2/2)\Delta t$ 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.

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.

35-9.0 EXAMPLE PROBLEM

35-9.01 Example

This example demonstrates the application of the methodology provided herein for the design of a detention-storage facility. Example inflow hydrographs and associated peak discharges for both pre- and post-development conditions are assumed to have been developed using hydrologic methods from Chapter Twenty-nine.

35-9.02 Design Discharge and Hydrographs

A storage facility is to be designed for runoff from both the 10-year and 50-year design storms. INDOT requires that the 50-year post-development peak discharge attains or does not exceed the 10-year pre-development peak discharge. Example peak discharges from the 10-year and 50-year design storm events are as follows:

1. pre-development 10-year peak discharge = 5.66 m^3/s; and
2. post-development 50-year peak discharge = 7.08 m^3/s.

Because the post-development 50-year peak discharge must not exceed the pre-development 10-year peak discharge, the allowable outflow discharge cannot exceed 5.66 m^3/s.

Example runoff hydrographs are shown in Figure 35-9A. Inflow durations from the post-development hydrographs are approximately 1.2 and 1.25 h, respectively, for runoff from the 10-year and 50-year storms.

35-9.03 Preliminary Volume Calculations

Preliminary estimates of required storage volume are obtained using the simplified method outlined in Section 35-7.0. The required storage volume, V_S, to contain the difference between $Q_{50\ (POST)}$ and $Q_{10\ (PRE)}$ is computed using Equation 35-7.1.

$$V_S = 0.5T_i(Q_i - Q_o) = 0.5T_i\ (Q_{50\ (POST)} - Q_{10\ (PRE)})$$
$$\mathrm{V_S} = 0.5\ (1.25)\ (3600)\ (7.08 - 5.66) = 3195\ \mathrm{m}^3$$

35-9.04 Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from the 50-year design storm is provided in Figure

35-9B. The storage-discharge relationship is developed by requiring the preliminary storage volume estimates of runoff for the 50-year design storm to be provided once the corresponding allowable peak discharge occurred. Discharge values are computed by solving the broad-crested-weir equation for head, *H*, assuming a constant discharge coefficient of 1.71, a weir length of 1.22 m and no tailwater submergence. The capacity of the storage-relief structure is assumed to be negligible.

Storage routing is conducted for runoff from the 50-year design storm to confirm the preliminary storage-volume estimates and to establish design-water surface elevations. Routing results using the Stage-Discharge-Storage data shown in Figure 35-9B and the Storage Characteristics Curves shown in Figures 35-8A and 35-8B, and 0.1-h time steps are shown in Figure 35-9C for runoff from the 50-year design storm. The preliminary design provides adequate peak discharge attenuation.

For the routing calculations, Equation 35-8.1 should be used.

The value in Column 6 should equal that in Column 3 plus that in Column 5.

Because the routed peak discharge is lower than the maximum allowable peak discharges for the 50-year design storm event, the weir length can be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

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

35-10.0 DRY-BOTTOM DETENTION BASIN

35-10.01 Introduction

A dry-bottom detention basin is a depressed area that stores runoff during wet weather and is dry the rest of the time. It is popular because of its comparatively low cost; few design limitations; and ability to serve a large or small watershed.

35-10.02 Design

The following applies.

1. Quantity. The pond should be designed to provide the required detention. It should be able to safely pass a 100-year storm. It should be designed using the procedures described in Sections 35-5.4 and 35-5.5. A 100-year storm should be routed through the facility to ensure that the embankment will not be damaged or fail during the passage of the storm. To improve the efficiency of the outlet, it may be necessary to include an antivortex device.

2. Outlet. The outlet for a dry basin can be designed in a variety of configurations. However, INDOT discourages the use of a riser pipe. A larger flow is accommodated by an emergency spillway.

35-10.03 Other Considerations

The sideslopes of the pond should not be steeper than 3H:1V to facilitate maintenance activities. The floor of the pond should be sloped at 2% toward the outlet to prevent ponding. The maximum operating pool depth should not exceed 2.0 m without approval of the INDOT Hydraulics Team.

Routine maintenance activities include an annual inspection, preferably during wet weather, and mowing, as needed.

35-11.0 WET-BOTTOM DETENTION BASIN

35-11.01 Introduction

A wet-bottom detention basin is similar to a dry-bottom detention basin in that it detains stormwater. However, it is different in that it retains a permanent pool during dry weather. A wet-bottom detention basin is more expensive than a dry-bottom detention basin.

35-11.02 Design

The following applies.

1. Quantity. A wet-bottom detention basin should provide the required detention and be able to safely pass a 100-year storm.

2. Outlet. The outlet can be designed in a variety of configurations. However, INDOT discourages the use of a riser pipe.

35-11.03 Other Considerations

The sideslopes of the pond should not be steeper than 3H:1V both above and below water for both safety and maintenance. Normal pool depth should not exceed 1.5 m. The maximum operating-pool depth should not exceed 2.5 m.

Routine maintenance includes annual inspections, preferably during wet weather, and mowing as needed.

35-11.04 Illustration

See Figure 35-11A for an illustration of a wet-bottom detention basin.

35-12.0 LANDLOCKED RETENTION

A watershed area that drains to a central depression without a positive outlet is typical of an area including karst topography. It can be evaluated using a mass-flow-routing procedure to estimate flood elevations. Although the procedure is straightforward, the evaluation of basin outflow is a complex hydrogeologic phenomenon that requires quality field measurements and a thorough understanding of local conditions. Because the outflow rate for a flooded condition is difficult to calculate, field measurements are desirable. See the AASHTO *Model Drainage Manual* for more-detailed information.

35-13.0 CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

To ensure acceptable performance and function, a storage facility that requires extensive maintenance is discouraged. The maintenance problems that are typical of an urban detention facility are as follows:

1. weed growth;
2. grass and vegetation maintenance;
3. bank deterioration;
4. standing water or soggy surfaces;
5. mosquito control;
6. blockage of outlet structures;
7. litter accumulation; and
8. maintenance of fences and perimeter plantings.

A proper design should focus on the elimination or reduction of maintenance requirements by addressing the potential for problems as follows.

1. Both weed growth and grass maintenance may be addressed by constructing sideslopes that can be maintained using available power-driven equipment, such as a tractor mower.

2. Bank deterioration can be controlled with protective lining or by limiting bank slopes.

3. Standing water or soggy surfaces may be eliminated by sloping the basin bottom toward the outlet, or constructing a low-flow pilot channel across the basin bottom from the inlet to the outlet.

4. Once the problems listed above are addressed, mosquito control will not be a major problem.

5. An outlet structure should be selected to minimize the possibility of blockage. Small pipes tend to block easily and should be avoided).

6. The facility should be located for easy access where the maintenance associated with litter and damage to fences or perimeter plantings can be conducted regularly.

35-14.0 PROTECTIVE TREATMENT

Safety considerations include reducing the chance of drowning by fencing the basin, reducing the maximum depth, or including ledges or mild slopes to prevent a person from falling in and facilitate his or her escape from the basin. Protective treatment may be required to prevent entry to a facility that poses a hazard to children, and, to a lesser extent, all persons. Fences and signs will be required for a detention area where the following conditions exist.

1. Rapid stage increases make escape difficult.
2. Water depth either exceeds 1 m for more than 24 h or the area is permanently wet.
3. Sideslopes equal or exceed 1.5H:1V.

Where a storage facility is located near a roadway, the road should be provided with an adequate clear zone. The maximum operating-pool depth will be limited to 2.0 m unless approved by the Hydraulics Team.

35-15.0 REFERENCES

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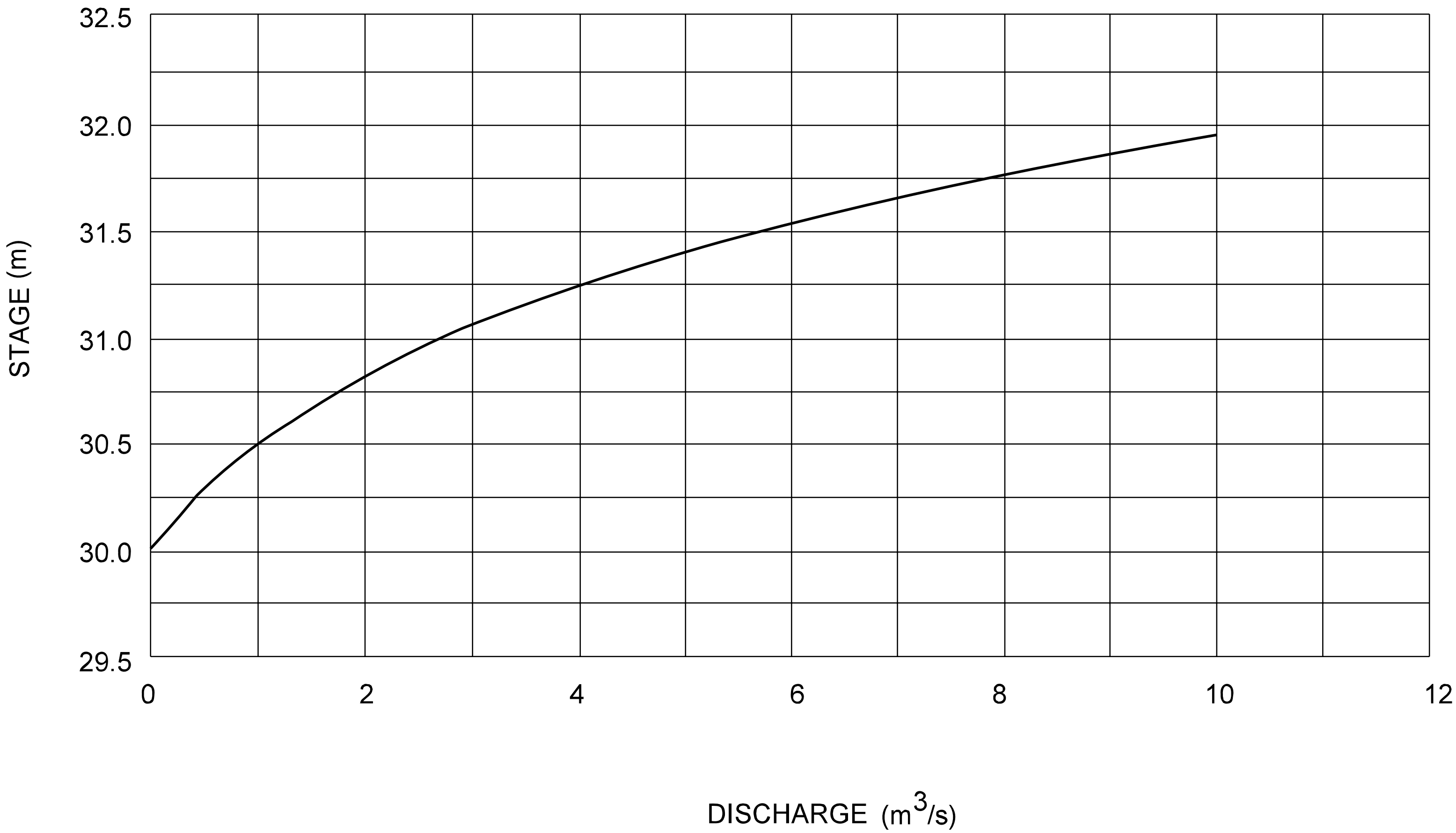
Symbol	Definition	Unit
A	Cross-sectional or surface area	m^2
C	Weir coefficient	(none)
d	Change in elevation	m
D	Depth of basin; Diameter of pipe	m
f	Infiltration rate	mm/h
g	Acceleration due to gravity	m/s^2
H	Head on structure	m
H_c	Height of weir crest above channel bottom	m
I	Inflow rate for storage computations	m^3/s
L	Length	m
O	Outflow rate	mm/h
Q	Flow rate	mm/h
S	Storage volume	ha-m
t	Routing time period	s
t_b	Time based on hydrograph	h
T_i	Duration of basin inflow	h
t_p	Time to peak	h
V_S	Storage volume	m^3
W	Width of basin	m
z	Sideslope factor	(none)

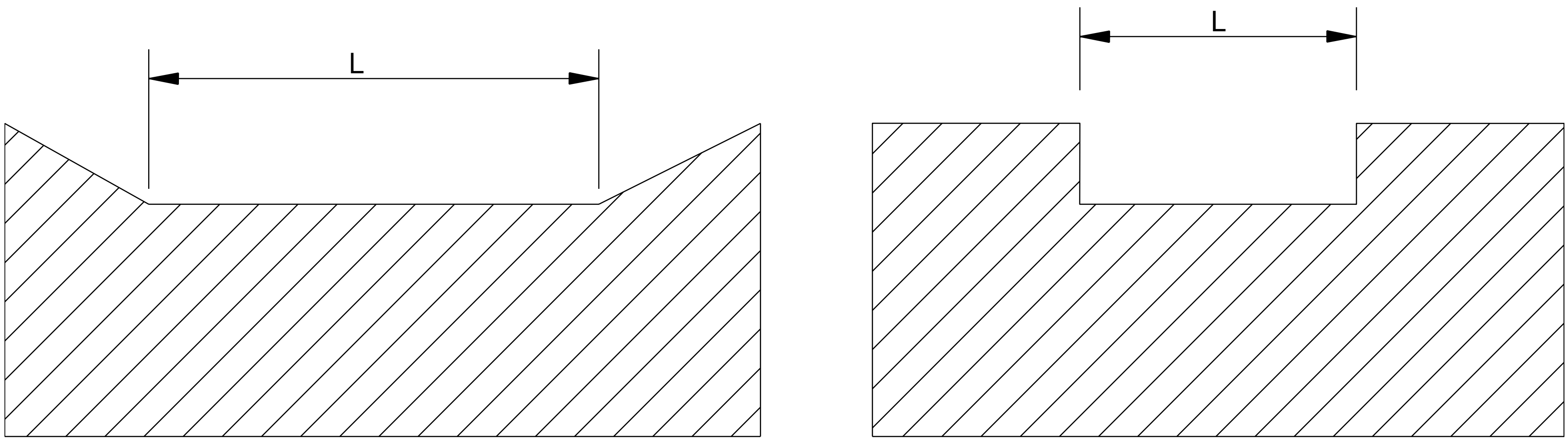
SYMBOLS, DEFINITIONS, AND UNITS

Figure 35-3A



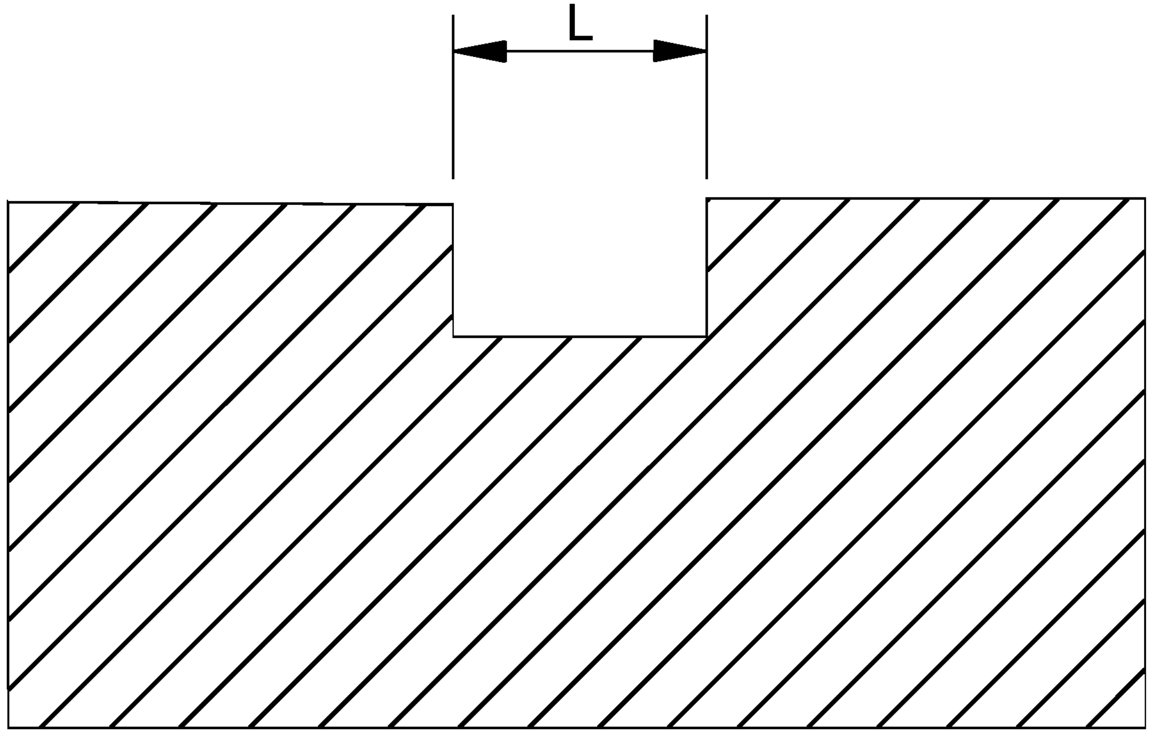
EXAMPLE STAGE-DISCHARGE CURVE

Figure 35-5B



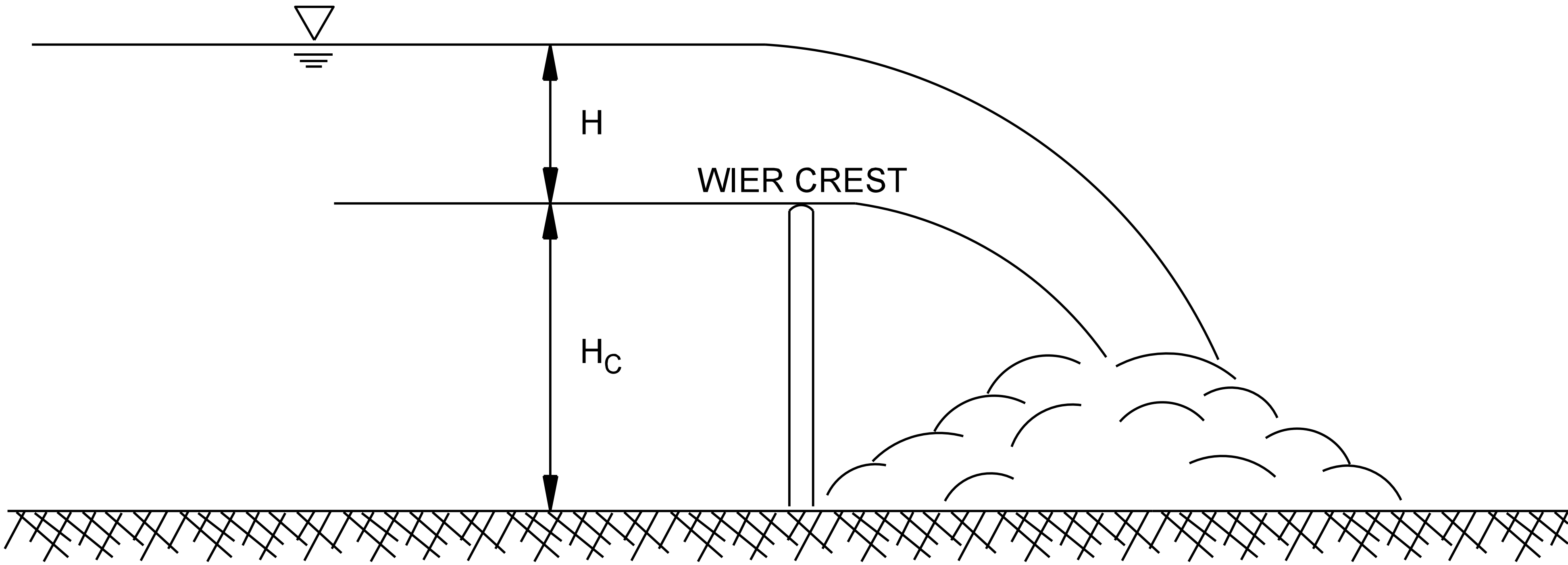
SHARP-CRESTED WEIR
(No-End Contractions)

Figure 35-6A



SECTION PLAT SHEET
(Two-End Contractions)

Figure 35-6B



SHARP-CRESTED WEIR AND HEAD

Figure 35-6C

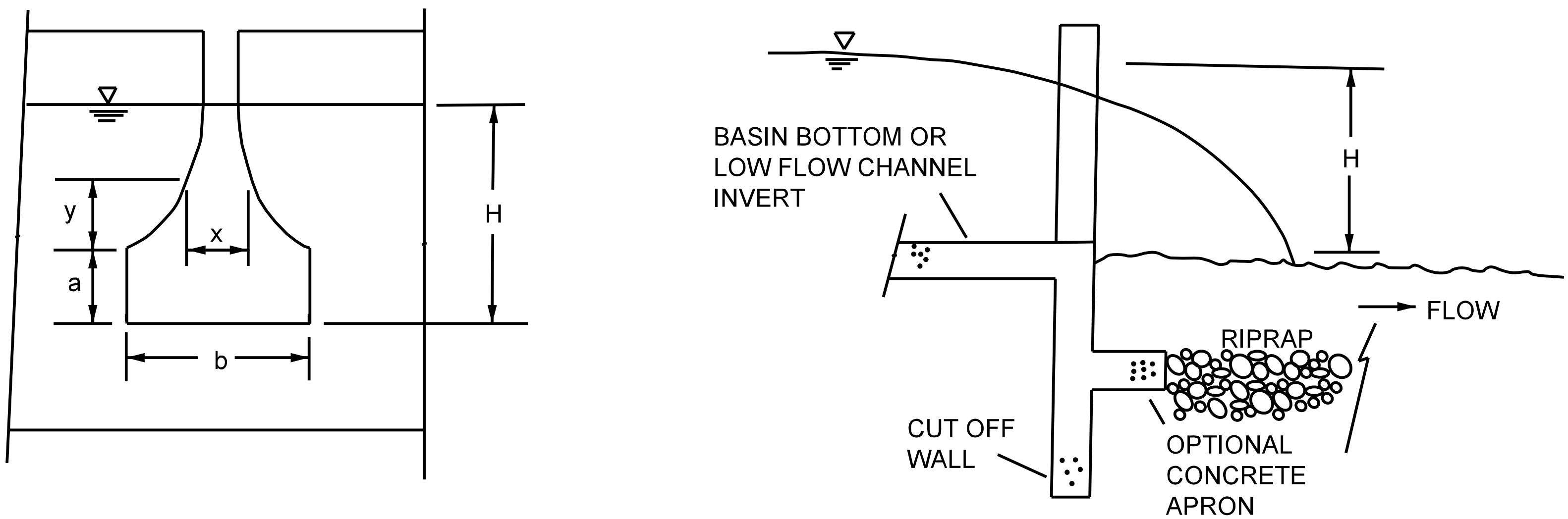
Measured Head, H *	Breadth of Crest of Weir (m)										
(m)	0.15	0.23	0.30	0.46	0.61	0.76	0.91	1.22	1.52	3.05	4.57
0.06	1.55	1.52	1.49	1.45	1.40	1.37	1.3	1.31	1.29	1.37	1.48
0.12	1.61	1.55	1.50	1.46	1.44	1.44	1.42	1.40	1.38	1.41	1.49
0.18	1.70	1.60	1.52	1.46	1.44	1.44	1.48	1.49	1.49	1.49	1.49
0.24	1.82	1.68	1.57	1.48	1.44	1.44	1.47	1.48	1.48	1.49	1.46
0.30	1.83	1.73	1.65	1.52	1.47	1.46	1.46	1.47	1.48	1.48	1.45
0.37	1.83	1.77	1.70	1.58	1.49	1.46	1.46	1.47	1.47	1.49	1.46
0.43	1.83	1.80	1.77	1.61	1.53	1.48	1.46	1.46	1.46	1.47	1.46
0.49	1.83	1.82	1.81	1.69	1.60	1.52	1.48	1.47	1.46	1.46	1.45
0.55	1.83	1.83	1.83	1.69	1.59	1.51	1.48	1.47	1.46	1.46	1.45
0.61	1.83	1.83	1.82	1.67	1.57	1.52	1.25	1.48	1.46	1.46	1.45
0.76	1.83	1.83	1.83	1.81	1.69	1.60	1.55	1.50	1.47	1.46	1.45
0.91	1.83	1.83	1.83	1.83	1.77	1.68	1.61	1.51	1.47	1.46	1.45
1.07	1.83	1.83	1.83	1.83	1.83	1.76	1.64	1.52	1.48	1.46	1.45
1.22	1.83	1.83	1.83	1.83	1.83	1.83	1.69	1.54	1.49	1.46	1.45
1.37	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.59	1.51	1.46	1.45
1.52	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.69	1.54	1.46	1.45
1.68	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.59	1.46	1.45

* *Measured at least 2.5*H *upstream of the weir.*

Reference: Brater and King (1976).

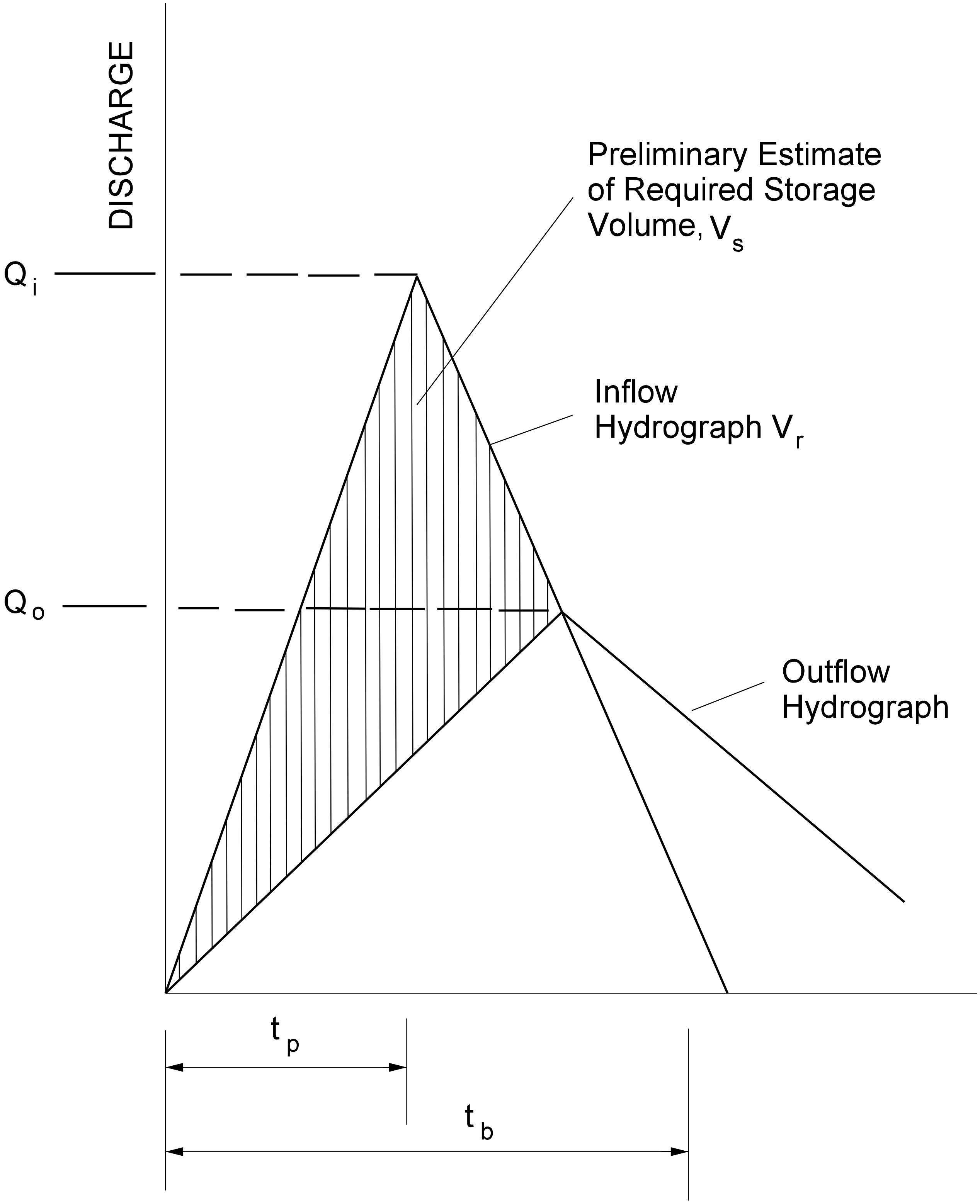
BROAD-CRESTED-WEIR COEFFICIENT, *C*, AS A FUNCTION OF WEIR-CREST BREADTH AND HEAD WEIR-CREST BREADTH

Figure 35-6D



PROPORTIONAL WEIR DIMENSIONS

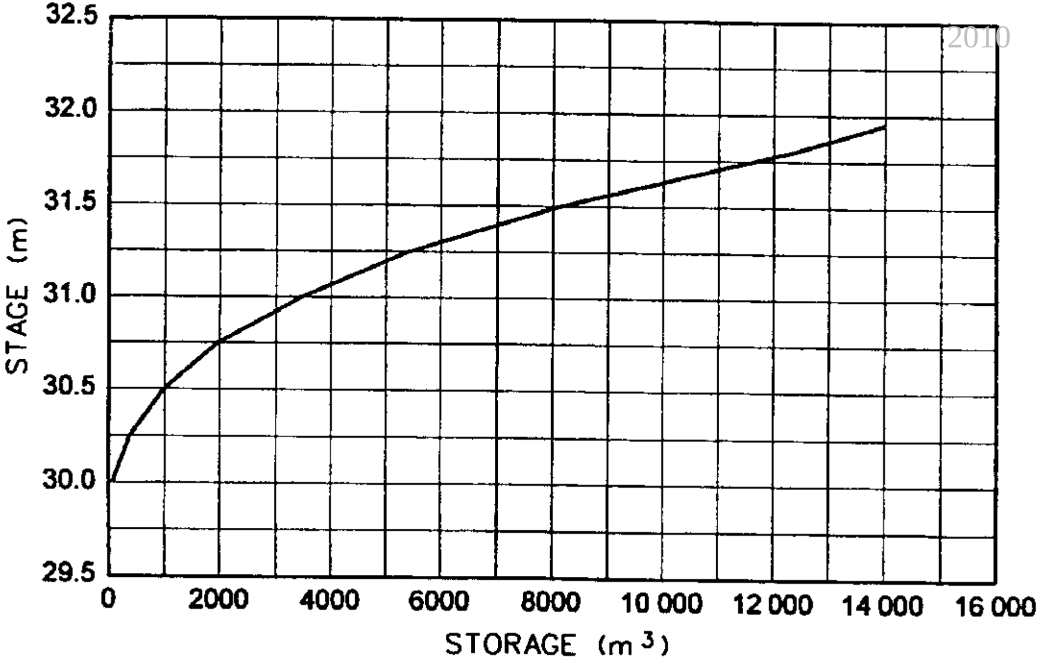
Figure 35-6E



TRIANGULAR SHAPED HYDROGRAPHS

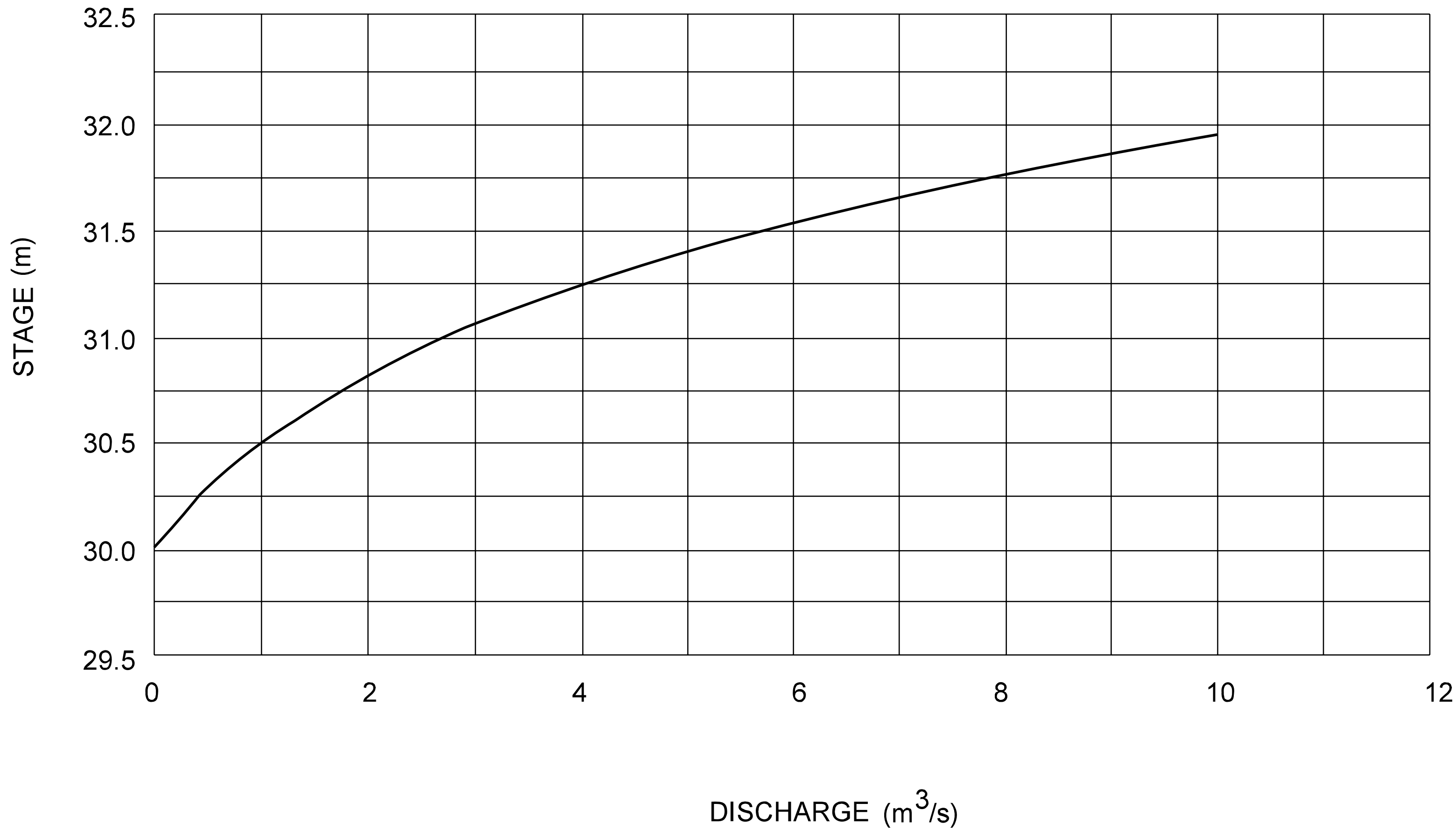
(For Preliminary Estimate Of Required Storage Volume)

Figure 35-7A



EXAMPLE STAGE-STORAGE CURVE

Figure 35-8A



EXAMPLE STAGE-DISCHARGE CURVE

Figure 35-8B

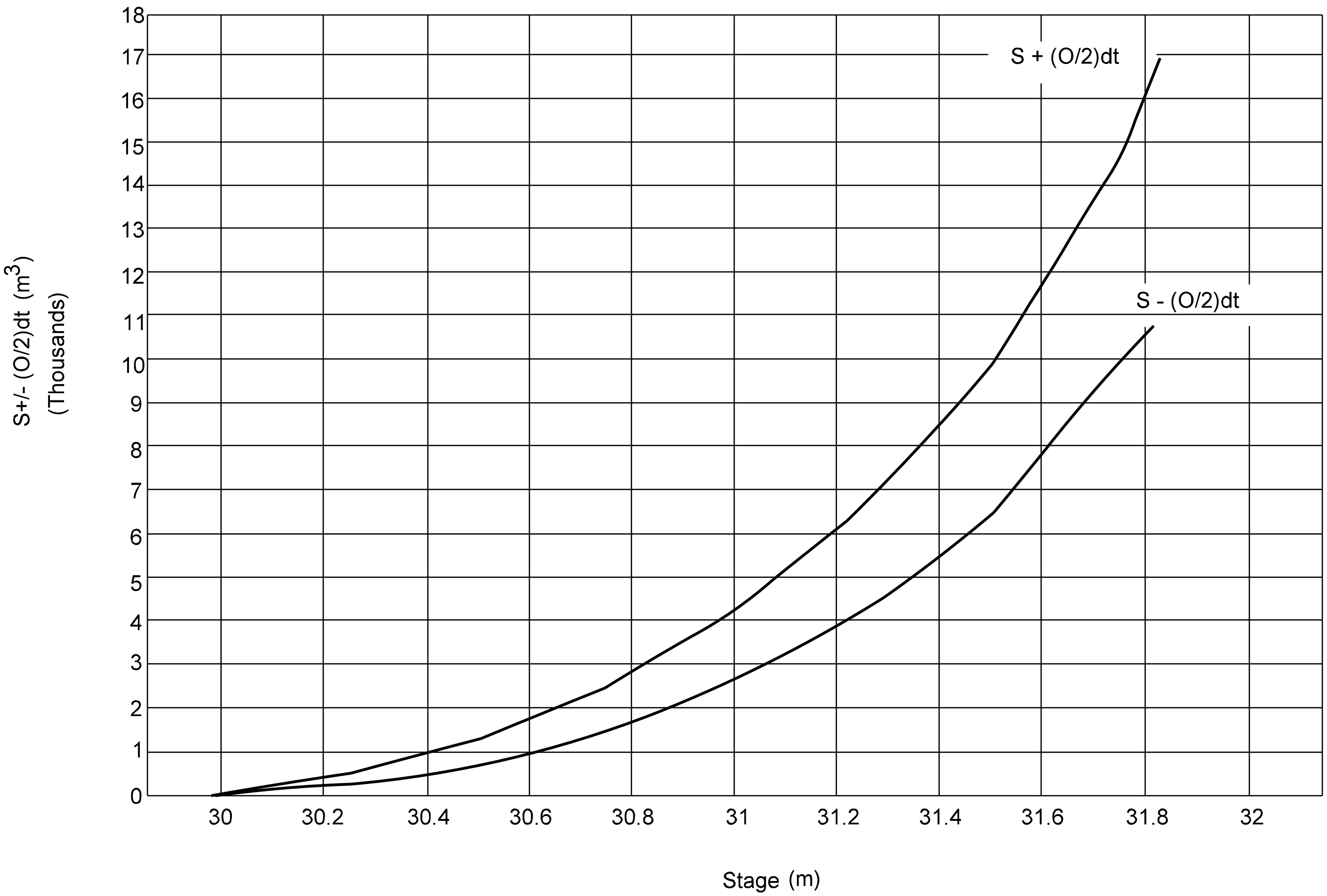
(1) Stage (m)	(2) Storage [1] (m^3)	(3) Discharge [2] (m^3/s)	(4) $S - O\Delta t/2$ (m^3)	(5) $S + O\Delta t/2$ (m^3)
30.00	62	0	62	62
30.25	370	0.42	244	496
30.50	987	0.99	686	1288
30.75	1974	1.78	1439	2509
31.00	3454	2.69	2645	4263
31.25	5427	4.05	4210	6644
31.50	8141	5.66	6440	9842
31.75	12 335	7.79	9990	14 680

[1] Obtained from the Stage-Storage Curve.
[2] Obtained from the Stage-Discharge Curve.

Note: t = *10 min = 0.167 h*

STORAGE CHARACTERISTICS

Figure 35-8C



STORAGE CHARACTERISTIC CURVE

Figure 35-8D

Time (h)	Pre-Development Runoff, 10-Year Storm (m^3/s)	Post-Development Runoff, 50-Year Storm (m^3/s)
0.0	0	0
0.1	0.68	1.42
0.2	2.29	5.04
0.3	4.81	5.66
0.4	5.66	4.67
0.5	4.25	2.55
0.6	2.69	1.42
0.7	1.73	0.82
0.8	1.13	0.45
0.9	0.79	0.25
1.0	0.51	0.14
1.1	0.42	0.08
1.2	0.37	0.03

EXAMPLE RUNOFF HYDROGRAPHS

Figure 35-9A

(1) Stage (m)	(2) Q (m^3)	(3) S (m^3/s)	(4) $S_1 - O\Delta t/2$ (m^3)	(5) $S_1 + O\Delta t/2$ (m^3)
0	0	0	0	0
0.27	0.3	320	266	1093
0.46	0.6	520	403	2360
0.57	0.9	690	528	3266
0.65	1.1	850	652	3993
0.77	1.4	1000	748	5077
0.87	1.7	1150	844	6162
0.97	2.0	1295	935	7247
1.07	2.3	1445	1031	8332
1.17	2.6	1580	1103	9595
1.25	2.9	1725	1203	10 500
1.38	3.4	2010	1398	12 310
1.47	3.7	2160	1494	13 395
1.54	4.0	2310	1590	14 480
1.61	4.2	2441	1676	15 384
1.69	4.6	2590	1762	16 648
1.77	4.9	2740	1858	17 733
1.81	5.1	2885	1967	18 458
1.95	5.7	3195	2176	20 485
2.07	6.2	3490	2374	22 439
2.13	6.5	3640	2470	23 524
2.25	7.0	3960	2700	25 335

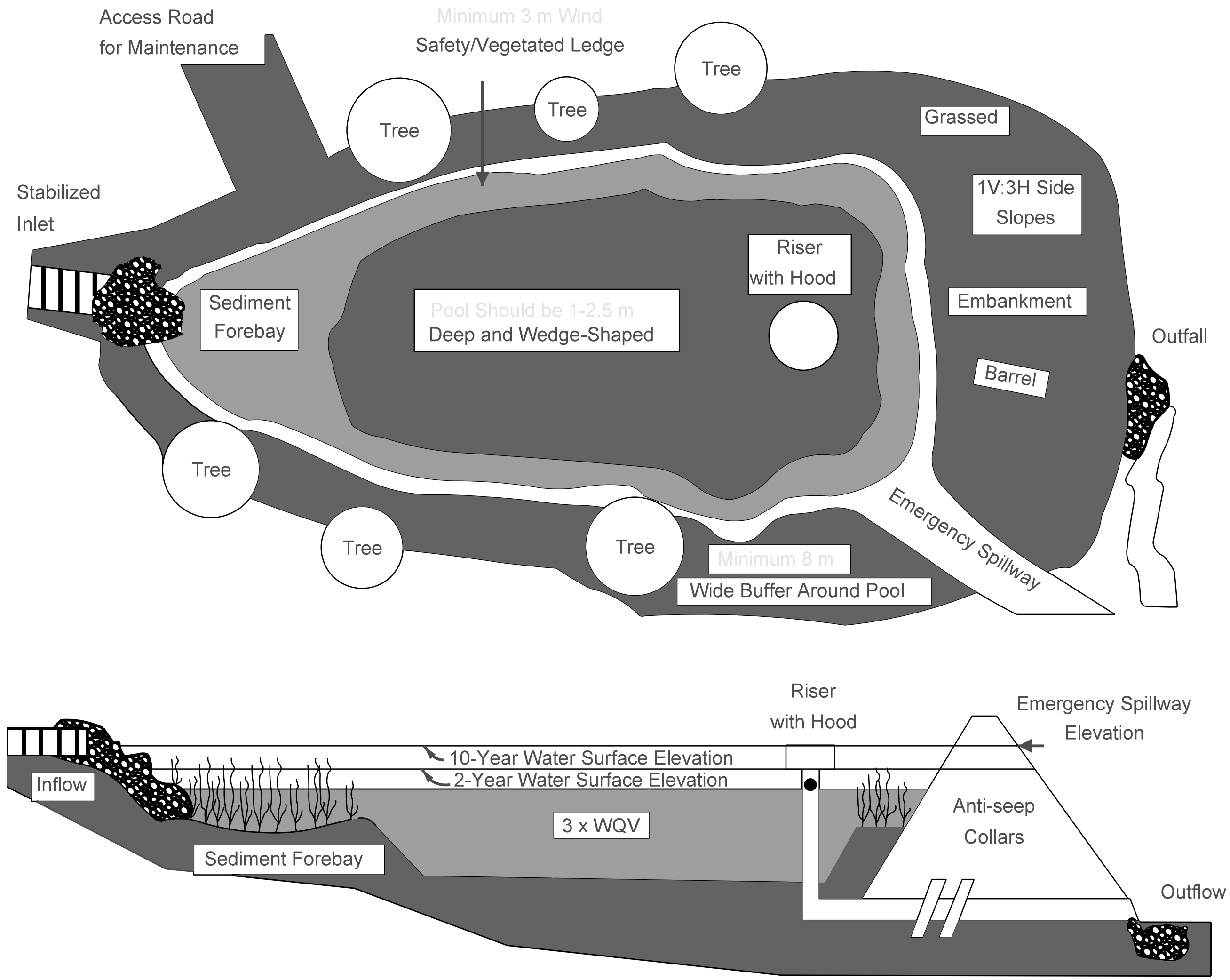
STAGE-DISCHARGE STORAGE DATA

Figure 35-9B

(1) Time (h)	(2) Inflow (m^3/s)	(3) $(I_1 + I_2)/2$ (m^3)	(4) H_1 (m)	(5) $S_1 - O_1\Delta t/2$ (m^3)	(6) $S_2 + O_2\Delta t/2$ (3) + (5), (m^3)	(7) H_2 (m)	(8) Outflow (m^3/s)
0	0	0	0	0	0	0	0
0.1	1.4	256	0	0	256	0.12	0.9
0.2	5.0	1163	0.12	99	1262	0.76	1.4
0.3	7.1	2182	0.76	740	2922	1.49	3.8
0.4	4.7	2115	1.49	1554	3669	1.77	4.9
0.5	2.6	1300	1.77	1604	2903	1.51	3.9
0.6	1.4	715	1.51	1542	2257	1.25	2.9
0.7	0.8	403	1.25	1234	1637	0.94	1.9
0.8	0.5	229	0.94	925	1154	0.73	1.3
0.9	0.3	126	0.73	728	854	0.58	0.9
1.0	0.1	70	0.58	543	613	0.43	0.6
1.1	0.1	40	0.43	407	447	0.37	0.5
1.2	0	20	0.37	345	365	0.27	0.3
1.3	0	0	0.27	271	271	0.18	0.2

STORAGE ROUTING FOR 50-YEAR STORM

Figure 35-9C



WET POND
(After Schueler, 1987

Figure 35-11A