Chapter 10 - Detention and Retention

Land development, including road construction, can significantly change runoff characteristics. These activities convert natural pervious areas to impervious areas and alter drainage pathways. Increased imperviousness causes an increased volume of runoff because of reduced infiltration and typically reduce runoff times causing an increase in runoff peaks. In addition, land development generally decreases the natural storage of a watershed by removing trees and vegetation, which reduces the volume of interception storage, and by site grading, which reduces the volume of depression storage.

In urban areas, lined channels, storm drains, and curb and gutter systems often replace natural drainage systems. These constructed systems produce an increase in runoff volume and peak flow, as well as a reduction in the time to peak of the runoff hydrograph. Figure 10.1 illustrates this effect, comparing a predevelopment and post-development runoff hydrograph. As the figure shows, the post-development runoff hydrograph displays a higher and earlier peak runoff. Surface runoff velocities also increase, which can increase surface rill and gully erosion rates. Higher stream velocities may also increase rates of bed-load movement. Designers use detention, retention, or both to mitigate the hydrologic effects of land development and urbanization.

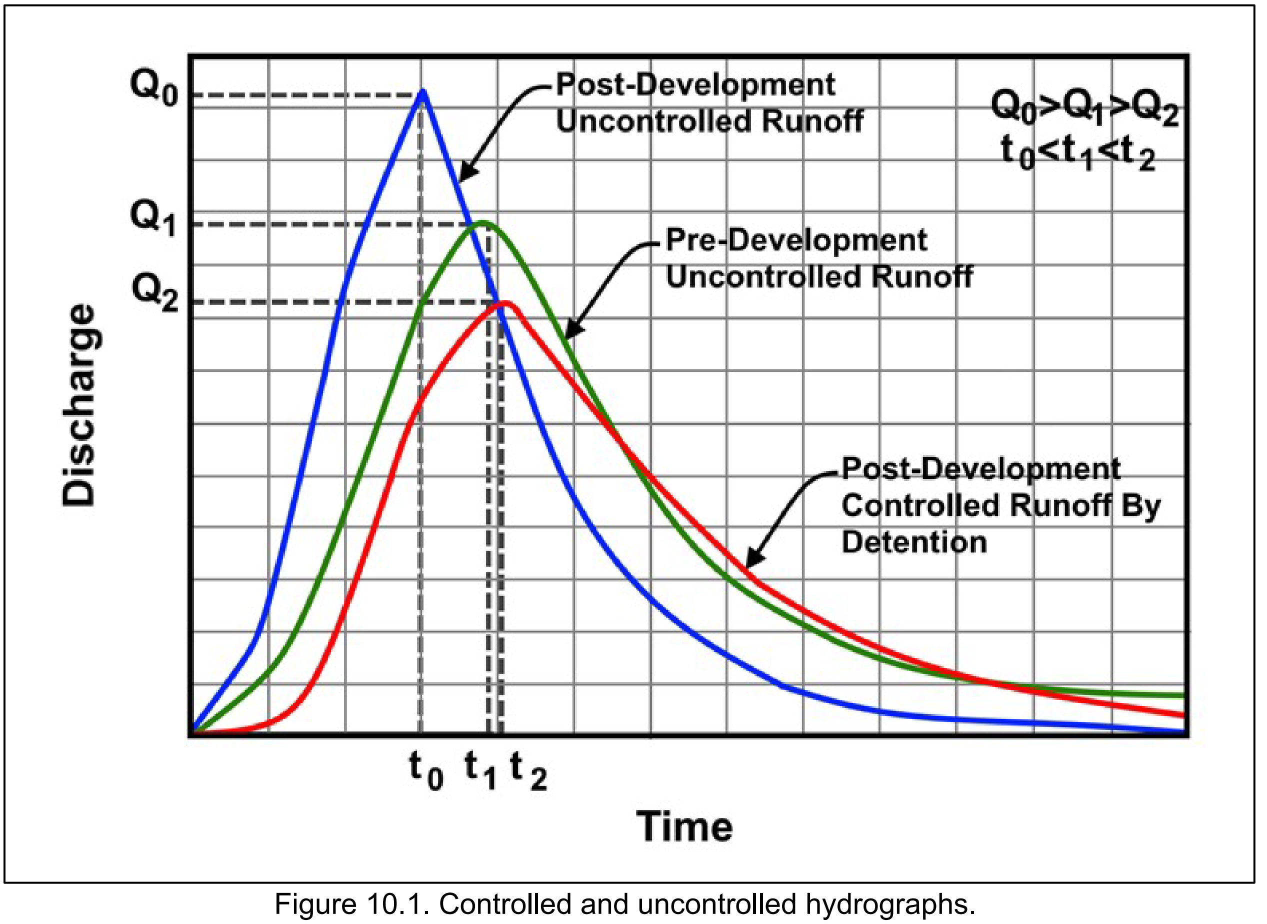

Figure 10.1. Controlled and uncontrolled hydrographs.

10.1 Design Objectives and Challenges

Designers follow applicable design criteria for detention and retention facilities. They also address other important features of these facilities including release timing, safety, and maintenance.

Stormwater Management Ponds

Stormwater quantity and quality control facilities are either detention or retention ponds.

Detention ponds, also known as dry ponds or retarding basins, primarily function to temporarily store a portion of the stormwater runoff volume while providing some water quality benefits.

Retention ponds, also known as wet ponds, have a permanent pool and treat both stormwater runoff quality and quantity.

10.1.1 Design Events

To mitigate the detrimental stormwater runoff effects of land development, designers have several tools that can limit peak flow rates from developed areas to those that occurred prior to development. Designers can use these tools to mitigate one, two, or more design event events, for example the 0.5, 0.1, and 0.01 AEP events. The "post-development controlled runoff by detention" hydrograph in Figure 10.1 illustrates the potential effect of storage in mitigating increased runoff caused by land development.

The storage of stormwater can reduce the frequency and extent of downstream flooding, soil erosion, sedimentation, and water pollution. Designers have also used detention and retention facilities to reduce the costs of large storm drainage systems by reducing the required size for downstream storm drain conveyance systems.

Local government bodies typically establish specific design criteria for peak flow attenuation. Some jurisdictions also require that flow volume be controlled to pre-development levels. Controlling flow volume is only practical when site conditions permit infiltration. To compensate for the increase in flow volume, some jurisdictions require that the peak post-development flow be reduced to below pre-development levels.

Some detention/retention facilities are designed for control of runoff from only a single storm frequency. However, single storm criteria have been found to be rather ineffective since such a design may provide little control of other storms. For example, design for the control of frequent storms (less frequent AEPs) provides little attenuation of lower frequency higher magnitude storm events. Similarly, design for less frequent large storms provides little attenuation for the more frequent smaller storms. Some jurisdictions enforce multiple storm regulatory criteria which dictate that multiple storm frequencies be attenuated in a single design. A common criterion would be to regulate the 0.5, 0.1, and 0.01 AEP events.

10.1.2 Release Timing

The timing of releases from stormwater control facilities can be critical to the proper functioning of overall stormwater systems. As illustrated in Figure 10.1, stormwater quantity control structures reduce the peak flow and increase the duration of the hydrograph at the point of release. However, in some instances this shifting of hydrograph peak times and durations can cause adverse effects downstream.

For example, where the drainage area being controlled is in a downstream portion of a larger watershed, delaying the peak and extending the recession limb of the hydrograph may result in a higher peak on the main channel as illustrated in Figure 10.2. In this example, this can occur if the reduced peak on the controlled tributary watershed is delayed so that it reaches the mainstem at or near the time of the mainstem peak. With multiple detention facilities in a watershed, designers will consider the effects of detention on hydrograph timing downstream to avoid creating flooding problems.

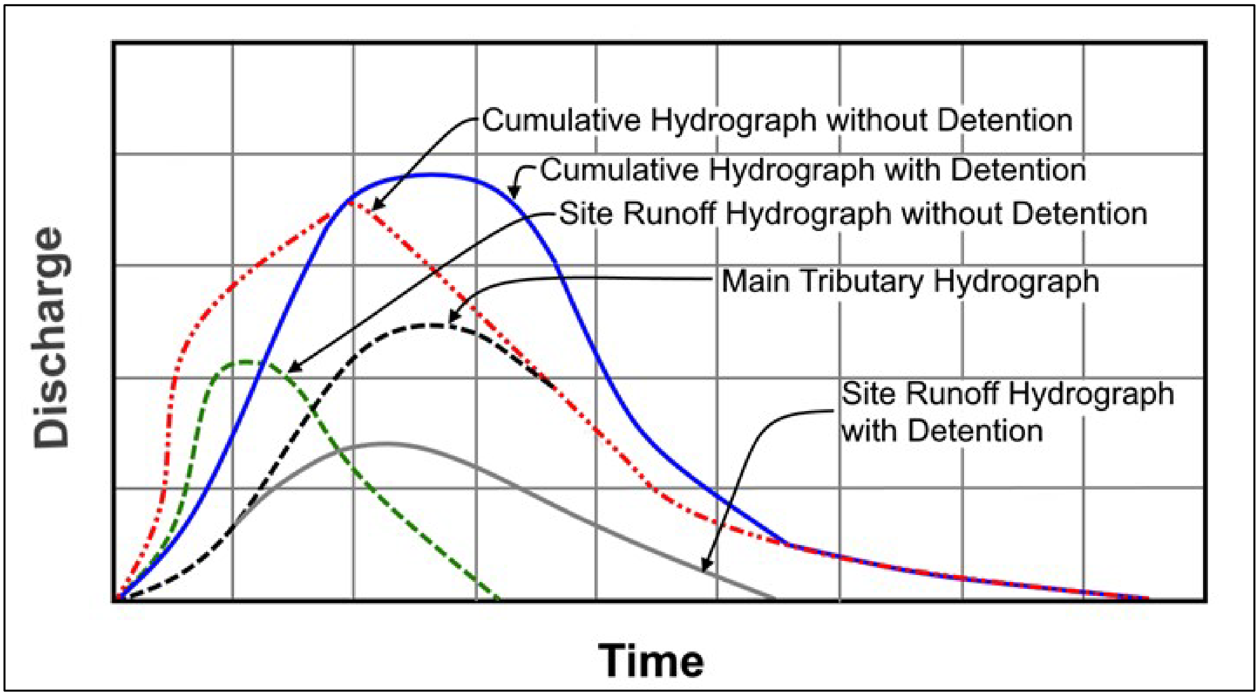

Figure 10.2. Example of cumulative hydrograph with and without detention.

10.1.3 Safety

Detention designers promote safe facilities by preventing public trespass, providing emergency escape aids, and eliminating other hazards. Preventing public trespass recognizes that children and adults may be attracted to the site, regardless of whether the site is intended for their use. For example, the designer may use fences to enclose ponds to limit access. Ideally, designers locate detention basins away from busy streets and intersections.

Designers consider inflow and outflow structures with safety in mind. For example, removable, hydraulically efficient grates and bars may cover the ends of inlet and outlet pipes, particularly if they connect with an underground storm drain system or otherwise present a safety hazard. Safer design of outflow structures would also limit flow velocities at points where people could be drawn into the discharge stream.

Where a detention basin incorporates active recreation areas, designers use mild bottom slopes along the periphery of the basin. Persons who enter a detention pond or basin while stormwater is being discharged may be at risk. The force of the currents may push a person into an outflow structure or may hold a victim under the water where a bottom discharge is used. *Urban Stormwater Management* (APWA 1981) discusses several design precautions intended to improve safety.

10.1.4 Maintenance

Stormwater management facilities depend on proper maintenance to function as intended over time. Depending on the facility type and location, appropriate periodic maintenance may include:

- Scheduled **inspections** may occur for the first few months after construction and on an annual basis thereafter. In addition, inspections during and after major storm events ensure that the inlet and outlet structures continue functioning by either identifying damage or clogging or confirming that none has occurred. They also can identify erosion on embankment side slopes and evidence of soil piping that can degrade the facility.

- **Mowing** at least twice a year discourages woody growth and controls weeds.
- **Sediment, debris, and litter control** at least twice a year maintains functionality and reduces clogging potential. In particular, removing accumulated sediment, debris, and trash around outlet structures prevents clogging of the control device.
- Standing water or soggy conditions within the lower stage of a storage facility can create nuisance conditions such as odors, insects, and weeds. **Nuisance control,** such as providing allowance for positive drainage during design, minimizes these problems.
- Inlet and outlet devices, and standpipe or riser structures deteriorate with time, necessitating **structural repairs and replacement**. The actual life of a structural component will depend on individual site-specific factors, such as soil conditions.

10.2 Storage Facility Types

Designers classify stormwater quantity control facilities as either detention or retention facilities. Detention facilities (dry ponds) store and release or attenuate stormwater runoff by a control structure or other release mechanism. Extended detention usually releases stormwater over a period of days. Retention facilities (wet ponds) maintain a permanent pool and release stormwater via evaporation and infiltration as well as through surface releases using a control structure.

10.2.1 Detention Facilities

Highway and municipal stormwater management plans (SWMPs) most often use the detention concept to limit the peak outflow rate to that which existed from the same watershed before development for a specific range of flood frequencies. Following SWMPs, designers may provide detention storage at one or more locations, either above ground or below ground or both. These locations may exist as impoundments; collection and conveyance facilities; underground tanks; and on-site facilities such as parking lots, pavements, and basins. Detention ponds are the most common type of storage facility used for controlling stormwater runoff peak flows. The majority of these are dry ponds which release all the runoff temporarily detained during a storm.

Designers recommend detention facilities where the facilities will have clear hydrologic, hydraulic, and cost benefits. Additionally, local ordinances may require some detention facilities; in such cases, the governing agency determines appropriate construction which may typically include:

- Design rainfall frequency, intensity, and duration consistent with highway standards and local requirements.
- The outlet structure limits the maximum outflow to allowable release rates. The maximum release rate may be a function of existing or developed runoff rates, downstream channel capacity, potential flooding conditions, and local ordinances.
- Detention facility size, shape, and depth provides sufficient volume to satisfy project storage requirements. Designers determine this by routing the inflow hydrograph through the facility. Section 10.3.1 outlines techniques designers can use to estimate an initial storage volume. Section 10.4 provides a discussion of storage routing techniques.
- An auxiliary outlet that allows overflow potentially resulting from excessive inflow or clogging of the main outlet. This outlet is positioned such that overflows will follow a predetermined route. Preferably, such outflows discharge into open channels, swales, or other approved storage or conveyance features.

- The system releases excess stormwater expeditiously to ensure that the entire storage volume is available for subsequent storms and to minimize hazards. A dry pond, which is a facility with no permanent pool, may need a paved low flow channel to ensure complete removal of water and to aid in nuisance control.
- The facility typically satisfies Federal (see Chapter 2) and State statutes and recognizes local ordinances.
- Provides access for maintenance.
- Avoids being an “attractive nuisance,” which may involve fencing and signage.

Figure 10.3 shows a schematic of the cross-section of a detention basin with a single-stage riser. A pool forms behind the retaining structure. The hydrograph of the post-development flood runoff enters the pool at the upper end of the detention basin. Water can be discharged from the pool through a pipe that passes through or around the detention structure. The size of the pipe can serve to limit the outflow rate, thus forming a permanent pool, with the permanent pool elevation changing only through evaporation and infiltration losses.

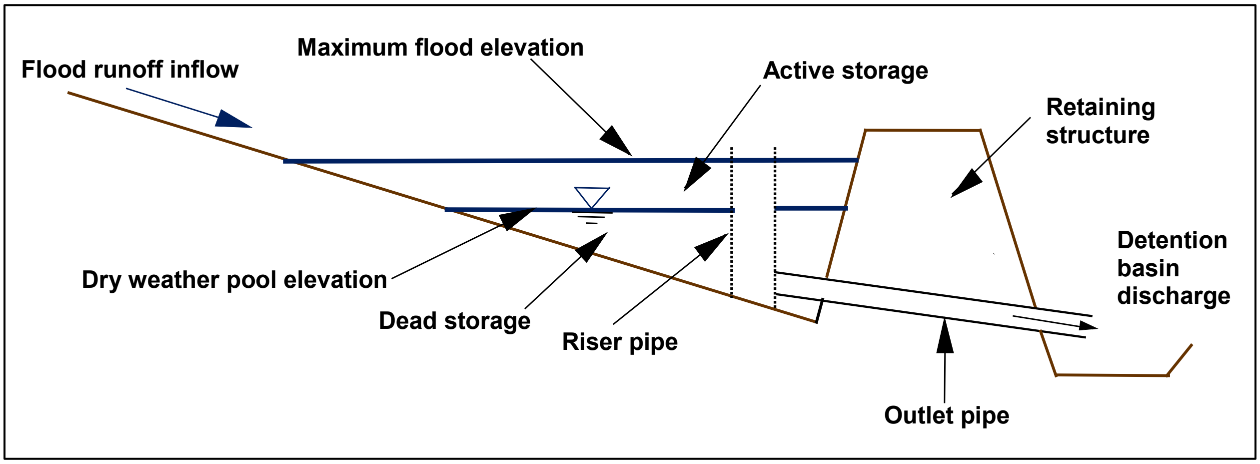

Figure 10.3. Schematic cross-section of a detention basin with a single-stage riser.

Using a permanent pool has several advantages, including water quality control, aesthetic considerations, and wildlife habitat improvement. A permanent pool also increases the total storage volume, involving both a larger retaining structure and a larger commitment of land, both of which increase project cost.

Figure 10.3 does not show several other elements of detention basin design. The designer may determine that the riser inlet should be fitted with both an antivortex device and a trash rack. The antivortex device prevents the formation of a vortex, thus maintaining the hydraulic efficiency of the outlet structure. The trash rack prevents high velocity flows from sucking trash (and people) into the riser.

All detention basins should have an emergency spillway to pass runoff from very large flood events, so the retaining structure is not overtopped and washed out. The elevation of the bottom of the emergency spillway, which will pass high flows around the retaining structure, is above the elevation of the riser outlet, but below the top of the retaining structure. Designers call the zone between the detention or surcharge storage zone.

Various methods exist for the planning and design of stormwater detention facilities. Design involves the simultaneous sizing of the storage volume characteristics and the riser/outlet characteristics. Some stormwater management (SWM) methods (“planning methods”) only apply to estimating the volume of storage that would meet the intent of the SWM policies. Designers use other planning methods to determine the characteristics of the outlet facility. Ultimately, the designer selects the final design using a method that simultaneously estimates the volume of storage and the characteristics of the outlet facility.

The simultaneous solution is important because there is a wide array of feasible solutions for any one site and set of design conditions. When designers separately determine storage volume and outlet facility characteristics, they may produce an ineffective, and possibly incorrect, design. In summary, planning methods provide less accuracy and involve less effort than design methods.

10.2.2 Retention Facilities

In addition to stormwater storage, retention facilities (e.g., wet ponds) may be used for water supply, recreation, pollutant removal, aesthetics, and groundwater recharge. As discussed in Chapter 11, infiltration facilities provide significant water quality benefits, and although groundwater recharge is not a primary goal of highway stormwater management, the use of infiltration basins and swales can provide this secondary benefit in tandem with retention facilities.

Designers typically develop retention facilities to provide the dual functions of stormwater quantity and quality control. These facilities may be provided on the surface or buried. The facility may have a permanent pool, which typically provides pollutant control.

Design criteria for retention facilities are the same as those for detention facilities except they may not include provisions for treating the runoff of lower frequency high magnitude storms. However, designers may apply the following additional criteria:

- Provide sufficient depth and volume below the normal pool level for any desired multiple use activity including pollutant removal.
- Include shoreline protection where erosion from wave action is expected.
- Provide that capability to lower the pool elevation or drain the basin for cleaning, shoreline maintenance, and emergency operations.
- Design dikes or dams with a safety factor commensurate with applicable regulations.
- Consider safety benching below the permanent water line at the toe of steep slopes to guard against accidental drowning.
- Size the emergency spillway and storage volume when closely spaced flood events could prevent complete draining between events.
- Complete detailed engineering geological studies to ensure that the facility will function as planned.

The FHWA (1979) provides additional information on underground detention and retention facilities.

10.3 Detention and Retention Design Information

This chapter introduces preliminary design activities for stormwater detention and retention including estimating the storage volume of the stormwater detention need, developing stage-storage relationships, and developing stage-discharge relationships.

10.3.1 Preliminary Storage Volume

Designers estimate a preliminary estimate of storage to accomplish the hydrograph attenuation goals. The following sections present several methods for determining an initial estimate of storage. Because each method provides preliminary estimates only, the designer may apply several of the methods and use engineering judgment to select an initial storage estimate.

10.3.1.1 Loss-of-Natural-Storage Method

The loss-of-natural-storage method estimates volumes based on the volume of lost natural storage. This approach is conservative for detention volume estimates, but is more appropriate for estimating retention volumes:

$$Q_s = Q_a - Q_b \tag{10.1}$$

where:

Q_s	=	Storage needed, inch (mm)
Q_a	=	Runoff depth for post-development watershed condition, inch (mm)
Q_b	=	Runoff depth for pre-development watershed condition, inch (mm)

Designers often refer to the variable Q as a volume even though it has the dimension of a depth. It represents the volume of water at the computed depth spread uniformly over the entire watershed. The designer computes the volume of storage, V_s, by multiplying Q_s by:

$$V_S = \alpha A Q_S \tag{10.2}$$

where:

α	=	Unit conversion constant, 3,630 in CU (10 in SI)
A	=	Drainage area, ac (ha)

The designer can compute the runoff depths Q_a and Q_b of equation 10.1 using any one of several methods. For the Natural Resources Conservation Service (NRCS) method, apply the NRCS runoff equation (SCS 1986) using the post-development and pre-development curve numbers (CNs). Using the Rational Method to estimate peak flows, the runoff depths, Q, is:

$$Q = \alpha \left[\frac{q_p t_c}{A} \right] \tag{10.3}$$

where:

q_p	=	peak flow, ft^3/s (m^3/s)
α	=	Unit conversion constant, 0.0165 in CU (6.0 in SI)
A	=	Drainage area, ac (ha)
t_c	=	Time of concentration, min

Solve equation 10.3 for both the pre- and post-development conditions by using the appropriate values of q_p and t_c. Then compute the runoff depth difference by entering the values into equation 10.1, ultimately using it to compute the volume of storage with equation 10.2.

Example 10.1:	Storage estimate for a detention pond.
Objective:	Estimate the storage needed for a detention pond using the loss-of-natural-storage method.

Given: An existing watershed under development with the following characteristics:

A	=	5.7 ac (2.3 ha)
C	=	0.2 and 0.45 (existing and developed conditions, respectively)

t_c	=	18 and 11 min (existing and developed conditions, respectively)
I	=	3.1 in/h (79 mm/h) and 4.0 in/h (102 mm/h), (existing and developed conditions, respectively), using local intensity-duration-frequency (IDF) curves

Step 1. Calculate the peak flows for the existing and developed conditions.

$q_{pb} = (1/\alpha)\, C_b\, i_b\, A = (0.2)(3.1)(5.7) = 3.5\ ft^3/s$

$q_{pa} = (1/\alpha)\, C_a\, i_a\, A = (0.45)(4.0)(5.7) = 10.3\ ft^3/s$

Step 2. Estimate the runoff depths.

Using equation 10.3:

$Q_b = \alpha[(q_{pb}\, t_c)/A] = (1/60.5)[3.5(18)/5.7] = 0.18$ inches

$Q_a = \alpha[(q_{pa}\, t_c)/A] = (1/60.5)[10.3(11)/5.7] = 0.33$ inches

Step 3. Compute the depth and volume of storage.

Using equation 10.1:

$Q_S = Q_a - Q_b = 0.33 - 0.18 = 0.15$ inches

Using equation 10.2:

$V_S = \alpha\, AQ_S = 3630\ (5.7)\ (0.15) = 3{,}100\ ft^3$

Solution: The detention pond needs 3,100 ft^3 (88 m^3) of storage.

10.3.1.2 Actual Inflow/Estimated Release Method

The actual inflow/estimated release method of estimating detention storage volume uses an inflow hydrograph (post-development) and the target peak flow release for the design event. The method estimates the outflow hydrograph by sketching an assumed outflow curve as shown in Figure 10.4. This limits the peak of the estimated outflow hydrograph such that it does not exceed the desired peak outflow from the detention basin. The shaded area between the inflow and the approximated outflow hydrographs represents the estimated storage needed. To determine the necessary storage, measure or mathematically compute the shaded area using a reasonable time period and appropriate hydrograph ordinates.

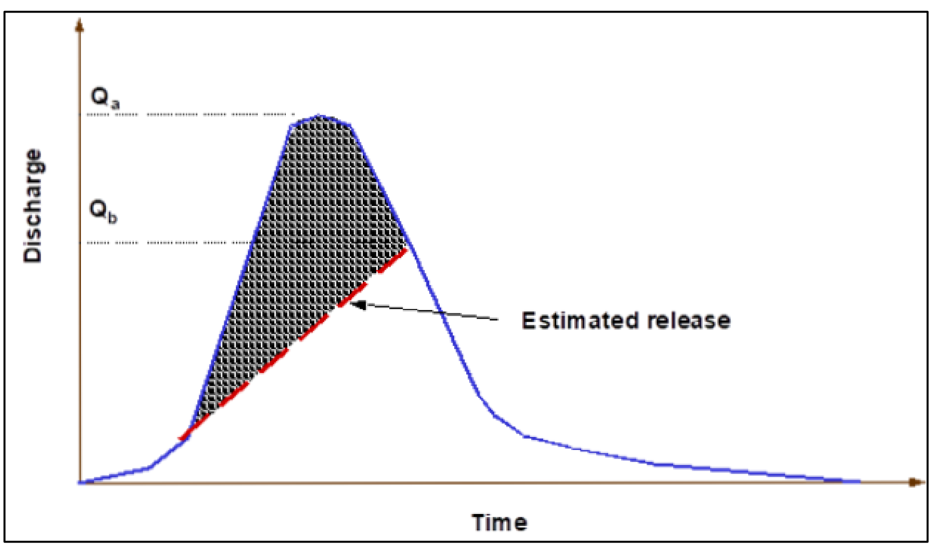

Figure 10.4. Estimating storage using the actual inflow/estimated release of hydrograph method.

10.3.1.3 Rational Method Triangular Hydrograph Method

Designers can obtain a preliminary estimate of the storage volume required for peak flow attenuation from a simplified design procedure that replaces the actual inflow and outflow hydrographs with simplified triangular shapes. This method works best with the Rational Method. Figure 10.5 illustrates the procedure. The area above the outflow hydrograph and inside the inflow hydrograph represents the estimated storage volume:

$$V_S = 0.5\, t_i \,(q_i - q_o) \tag{10.4}$$

where:

V_s	=	Storage volume estimate, ft^3 (m^3)
q_i	=	Peak inflow rate into the basin, ft^3/s (m^3/s)
q_o	=	Peak outflow rate out of the basin, ft^3/s (m^3/s)
t_i	=	Duration of basin inflow, s

The duration of basin inflow equals two times the time of concentration. The triangular hydrograph procedure, originally described by Boyd (1981), compares favorably with more complete design procedures involving reservoir routing.

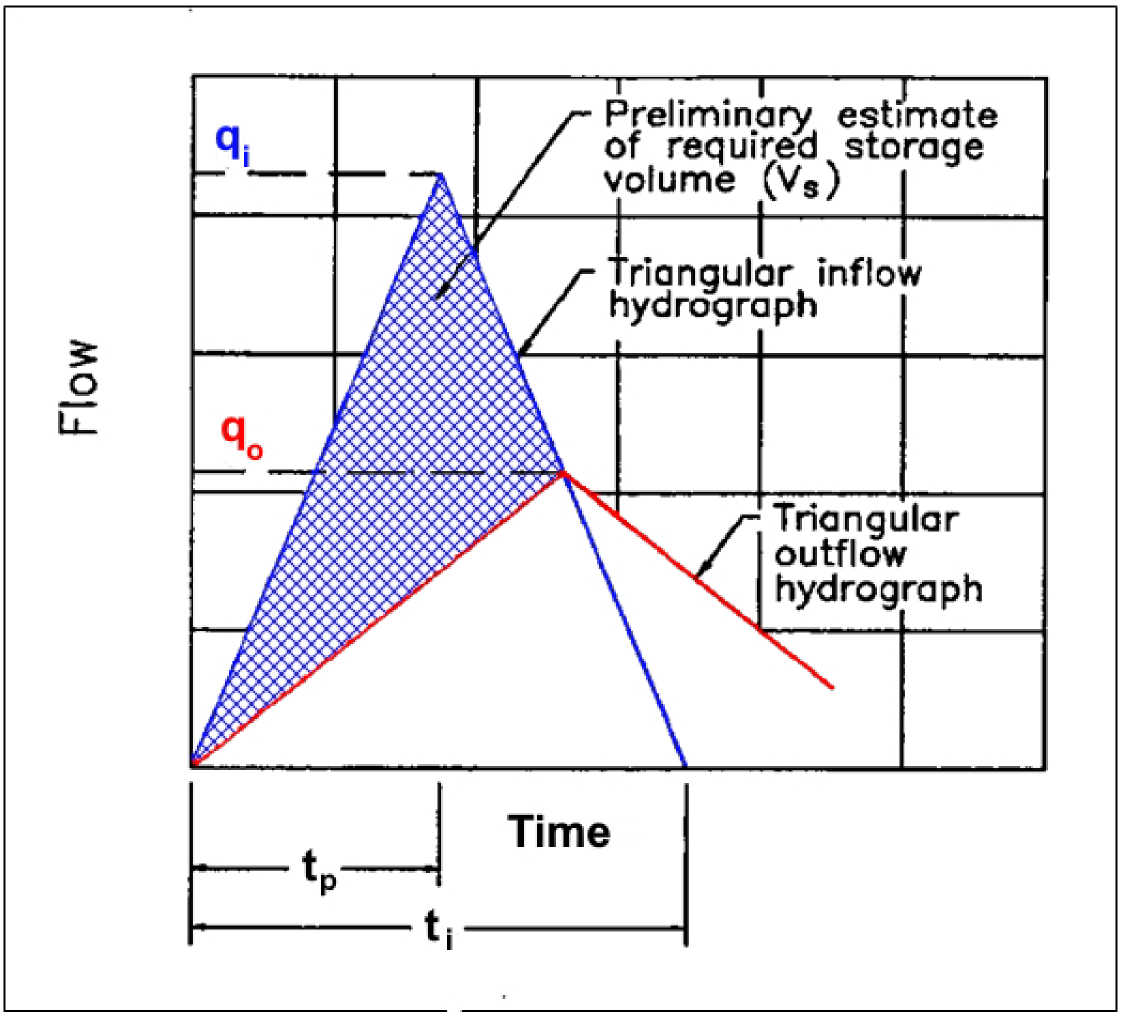

Figure 10.5. Triangular hydrograph method.

10.3.1.4 NRCS TR-55 Procedure

TR-55 (SCS 1986) describes a method for estimating required storage volumes based on peak inflow and outflow rates. The method uses average storage and routing effects observed for a large number of structures. This procedure for estimating storage volume may have errors and, therefore, is only useful for preliminary estimates.

The procedure to estimate required detention storage follows (SCS 1986):

Step 1. Compute discharges.

Determine the peak inflow and outflow discharges q_i and q_o, which correspond to the post- and pre-development flows, respectively.

Step 2. Compute q_o/q_i and R_q.

Calculate the ratio q_o/q_i, which is equal to R_q.

Step 3. Calculate the ratio V_s/V_r for graphical method or compute R_s for arithmetic method.

Calculate R_s:

$$R_S = \frac{Q_S}{Q_a} = C_0 + C_1R_q + C_2R_q^2 + C_3R_q^3 \quad (10.5)$$

The coefficients C_O, C_1, C_2, and C_3 are a function of the NRCS rainfall distribution and are provided in Table 10.1.

Table 10.1. Coefficients for the NRCS detention volume method.

Rainfall Distribution	C_0	C_1	C_2	C_3
I or IA	0.660	-1.76	1.96	-0.730
II or III	0.682	-1.43	1.64	-0.804

Step 4. Determine the storage volume, V_s.

$$V_S = \alpha R_S Q_a A \quad (10.6)$$

where:

α	=	Unit conversion constant, 3,630 in CU (10 in SI)
Q_a	=	Post-development depth of runoff, inches (mm)
A	=	Drainage area, ac (ha)

Example 10.2: Estimation of needed detention pond storage.

Objective: Estimate the needed storage of a detention facility by using the actual inflow hydrograph, Rational Method triangular hydrograph, and NRCS TR-55 methods.

Given: The post-developed (improved conditions) peak flow of 31.1 ft^3/s (0.88 m^3/s) is limited to an outflow rate from the proposed detention facility of 19.4 ft^3/s (0.55 m^3/s). This limiting outflow is a constraint imposed by the downstream receiving water course and is the maximum outflow rate from the drainage area for unimproved conditions. Drainage area, A, equals 43.4 acres.

Step 1. Estimate storage using the actual inflow/estimated release method.

Figure 10.6 illustrates the existing conditions and proposed conditions hydrographs. Assuming the proposed detention facility will produce an outflow hydrograph similar to existing conditions, determine the required detention volume as the area above the existing hydrograph and below the proposed hydrograph. Using the scale on Figure 10.6, 1 inch^2 equals 22,500 ft^3. Measuring the area between the two hydrographs yields an area of 1.5 inches^2, which converts to the following volume:

V_s = (1.5 inches^2) (22,500 ft^3/inch^2) = 33,750 ft^3

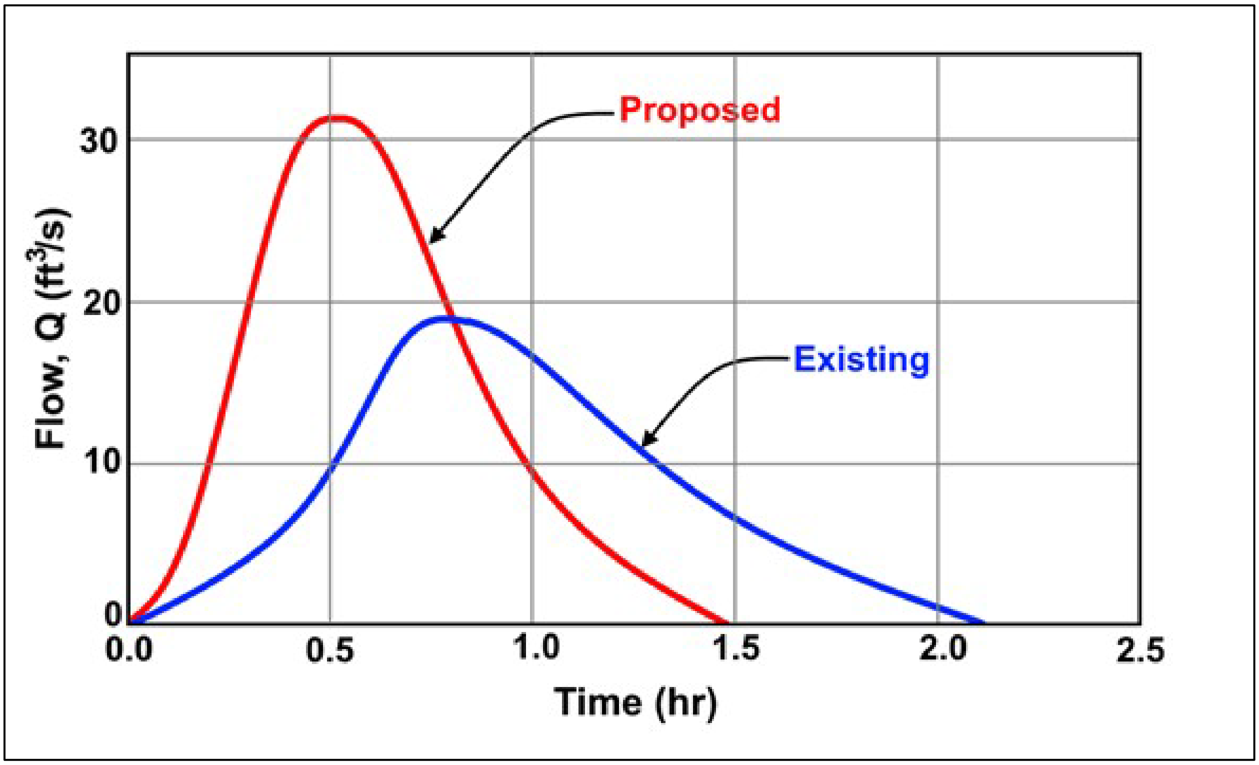

Figure 10.6. Hydrograph for existing and proposed conditions.

Step 2. Calculate storage using the Rational Method Triangular Hydrograph method.

The inflow rate into the detention basin (q_i) is 31.1 ft^3/s. The peak flow rate out of the basin (q_o) is set to be = 19.4 ft^3/s.

Using equation 10.4, compute the initial storage volume as:

$V_s = 0.5\ t_i\ (q_i - q_o) = (0.5)(1.43 \text{ h x } (60 \text{ min/h})(60 \text{ s/min}))(31.1 - 19.4) = 30{,}116 \text{ ft}^3$

Step 3. Determine storage using the NRCS TR-55 Method. Calculate R_q.

As previously noted, the inflow discharge is 31.1 ft^3/s, and the outflow discharge is set to be 19.4 ft^3/s by local ordinance.

The ratio of basin inflow to basin outflow is:

$q_o / q_i = R_q = 19.4 / 31.1 = 0.62$

Step 4. Determine V_s / V_r.

With q_o / q_i = 0.62 and a Type II Storm, calculate R_s using equation 10.5 and the coefficients from Table 10.1. Q_a = 0.43 inches.

$$R_S = Q_s / Q_a = C_0 + C_1R_q + C_2R_q^2 + C_3R_q^3$$

$$= 0.682 + (-1.43)(0.62) + (1.64)(0.62)^2 + (-0.804)(0.62)^3 = 0.23$$

Step 5. Calculate the preliminary estimated storage volume (V_s) using the NRCS TR-55 Method and equation 10.6.

$V_s = \alpha\, R_s\, Q_a\, A$ = (3630) (0.23) (0.43) (43.4) = 15,600 ft^3

Solution: The hydrograph and triangular hydrograph methods result in the most consistent and conservative estimates, whereas the NRCS TR-55 method is less conservative.

10.3.2 Stage-Storage Relationship

A stage-storage relationship links the depth of water and storage volume in the storage facility and depends on the topography at the site of the storage structure. Figure 10.7 illustrates a typical stage-storage curve. Calculate the volume of storage by using simple geometric formulas expressed as a function of storage depth. After estimating the required storage as described in the previous section, determine the configuration of the storage basin to develop the stage-storage curve. The following relationships can be used for computing the volumes at specific depths of geometric shapes commonly used for storage facilities.

10.3.2.1 Rectangular Basins

Figure 10.8 provides a schematic of a rectangular basin, a common shape for underground storage. Compute the volume of a rectangular basin by dividing the volume into rectangular and triangular shapes. If the basin is not on a slope, then the geometry will consist only of rectangular shapes.

Volumes of rectangular and triangular shapes, respectively, are:

$$V = LWD \tag{10.7}$$

$$V = 0.5W\left(\frac{D^2}{S}\right) \tag{10.8}$$

where:

V	=	Volume at a specific depth, ft^3 (m^3)
D	=	Depth of ponding for that shape, ft (m)
W	=	Width of basin at base, ft (m)
L	=	Length of basin at base, ft (m)
S	=	Slope of basin, ft/ft (m/m)

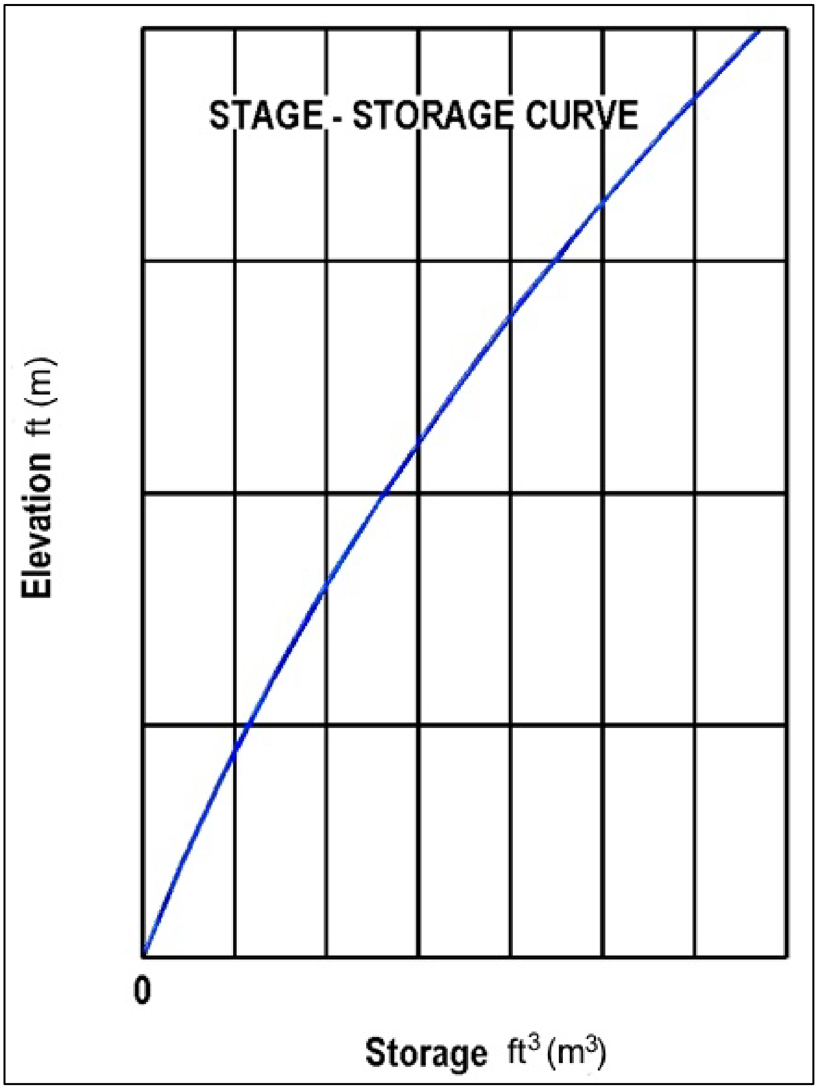

Figure 10.7. Stage-storage curve.

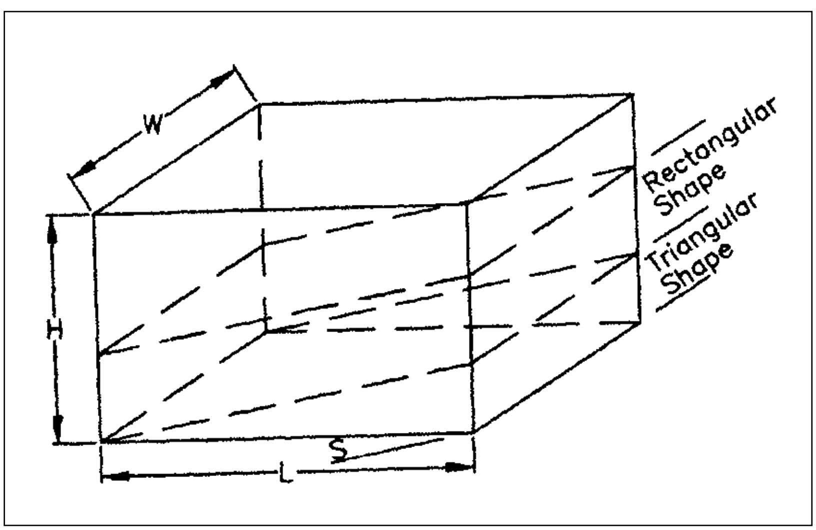

Figure 10.8. Rectangular basin.

A special case, rarely permitted by site, occurs with a horizontal rectangular bottom area and vertical sides. In this case, the storage simply equals the bottom area (i.e., length times width) multiplied by the depth of storage. If the relationship is plotted in a Cartesian axis system with storage as the ordinate and stage as the abscissa, the stage-storage relationship will be a straight line with a slope equal to the surface area of the storage facility.

A variation on the rectangular basin occurs where the bottom of the storage facility is a rectangle (L x W), the longitudinal cross- section is a trapezoid, and the ends are vertical. In this case, the stage-storage relationship is given by:

$$V = \frac{L}{\tan\theta} h^2 + (L\,W)h \tag{10.9}$$

where:

θ	=	Angle of side slopes
h	=	Height above bottom of basin, ft (m)

Graphing equation 10.9 results in a stage-storage relationship with the shape of a second-order polynomial with a zero intercept and a shape that depends on the values of L, W, and θ.

Example 10.3: Storage estimate of rectangular basin.

Objective: Estimate the maximum volume of a rectangular basin.

Given: Consider a rectangular basin:

L	=	656 ft (200 m)
W	=	328 ft (100 m)
Z	=	2

Step 1. Apply equation 10.7, which becomes:

$$S = 2L h^2 + (L\,W)h$$

Step 2. Calculate storage using equation 10.9.

Calculate the storage at a depth of 4.9 ft (1.5 m) as follows:

$S = 1{,}312h^2 + 215{,}200h = 1{,}312\ (4.9)^2 + 215{,}200\ (4.9) = 1{,}086{,}000\ ft^3$

Solution: The maximum volume of the trapezoidal basin is 1,086,000 ft^3 (30,800 m^3).

10.3.2.2 Trapezoidal Basins

Figure 10.9 schematically represents a trapezoidal basin. Calculate the volume of a trapezoidal basin by dividing the volume into components of triangular and rectangular shape and applying equation 10.10. “Z” in this equation equals the ratio of the horizontal to vertical components of the side slope. For example, if the side slope is 1 to 2 (V:H), “Z” equals 2.

$$V = LWD + (L+W)\ ZD^2 + \left(\frac{4}{3}\right) Z^2 D^3 \tag{10.10}$$

where:

V	=	Volume at a specific depth, ft^3 (m^3)
D	=	Depth of ponding or basin, ft (m)

L	=	Length of basin at base, ft (m)
W	=	Width of basin at base, ft (m)
r	=	Ratio of width to length of basin at the base
Z	=	Side slope factor; ratio of horizontal to vertical components of side slope

Estimate the trial dimensions of a basin by rearranging the equation for volume in terms of basin length:

$$L = \frac{\left\{ -ZD(r+1) + \left[(ZD)^2 (r+1)^2 - 5.33(ZD)^2 r + \left(\frac{4rV}{D} \right) \right]^{0.5} \right\}}{2r} \tag{10.11}$$

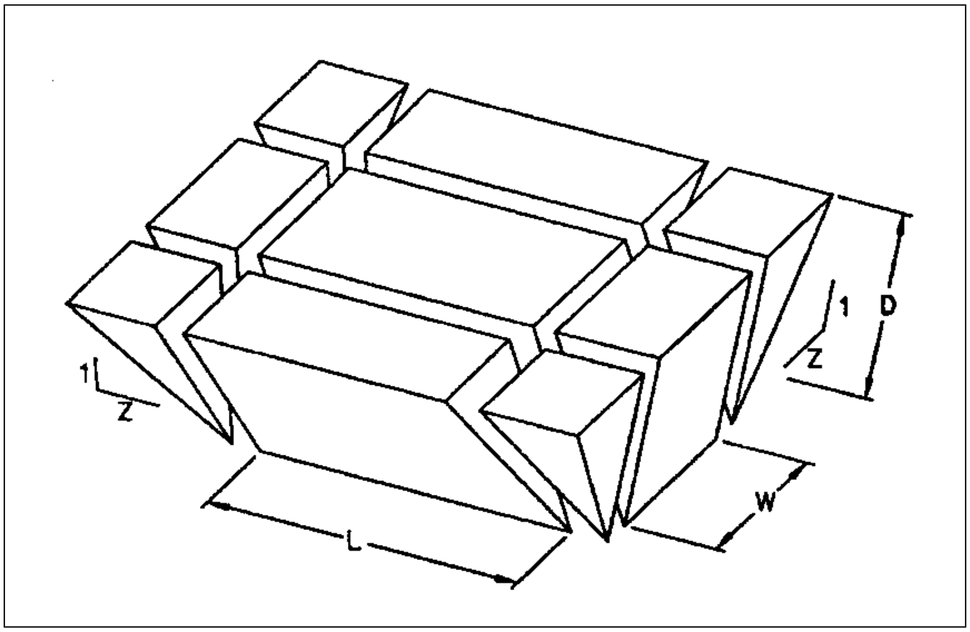

Figure 10.9. Trapezoidal basin.

Example 10.4:	Stage-storage curve of a trapezoidal basin.
Objective:	Find the dimensions of the basin at its base and develop a stage-storage curve for the basin.

Given: Estimated storage volume (V); depth available for storage during a 0.1 AEP event (D); available freeboard (F_B); basin side slopes (Z); and width-to-length ratio of basin (r).

V	=	30,016 ft^3 (850 m^3)
D	=	5.25 ft (1.6 m)
F_B	=	2.0 ft (0.6 m)
Z	=	3 (V:H = 1:3)
r	=	0.5

Develop a stage-storage curve for the basin assuming that the base elevation of the basin equals 32.8 ft (10.0 m), and the crest of the embankment is at 40.0 ft (12.2 m). Determine this crest elevation by adding the 5.25 ft (1.6 m) of available depth plus the 2.0 ft (0.6 m) of freeboard.

Step 1. Establish the basin dimensions.

Substituting the given values in equation 10.11:

$L = \{-Z\ D\ (r + 1) + [(Z\ D)^2\ (r + 1)^2 – 5.33\ (Z\ D)^2\ r + ((4r\ V) / D)]^{0.5}\} / 2r$

$L = \{(-3\ (5.25)\ (0.5 + 1) + [(3\ (5.25))^2\ (0.5+1)^2 – 5.33\ (3\ (5.25))^2\ 0.5+((4\ (0.5)\ 30016)/5.25)]^{0.5}\}/ 2\ (0.5)$

L = 82.82 ft, use L = 85 ft

W = 0.5 L = 42.5 ft, use W = 43 ft

Therefore, select an 85-ft by 43-ft basin.

Step 2. Develop the stage-storage relationship.

By varying the depth (D) in equation 10.10, develop a stage-storage relationship for the trapezoidal basin. Table 10.2 summarizes the results.

Table 10.2. Stage-storage relationship.

Depth (ft)	Stage (ft)	Storage Volume (ft^3)
0	32.81	0
0.66	33.46	2,567
1.31	34.12	5,425
1.97	34.77	8,774
2.62	35.43	12,455
3.28	36.09	16,548
3.94	36.75	21,074
4.59	37.4	26,052
5.25	38.06	31,503
5.91	38.71	37,413
6.56	39.37	43,906

Solution: Table provides the stage-storage curve.

10.3.2.3 Pipes and Conduits

Figure 10.10 provides a definition sketch for a generalized prism in a pipe or other shape of conduit. To provide for sediment transport through the conduit, designers place the conduit on a slope. The prismoidal formula provides an estimate of storage volume based on estimates of cross-sectional areas at three locations:

$$V = \left(\frac{L}{6}\right)(A_1 + 4M + A_2) \qquad (10.12)$$

where:

V	=	Volume of storage, ft^3 (m^3)
L	=	Length of section, ft (m)
A_1	=	Cross-sectional area of flow at base, ft^2 (m^2)
A_2	=	Cross-sectional area of flow at top, ft^2 (m^2)
M	=	Cross-sectional area of flow at midsection, ft^2 (m^2)

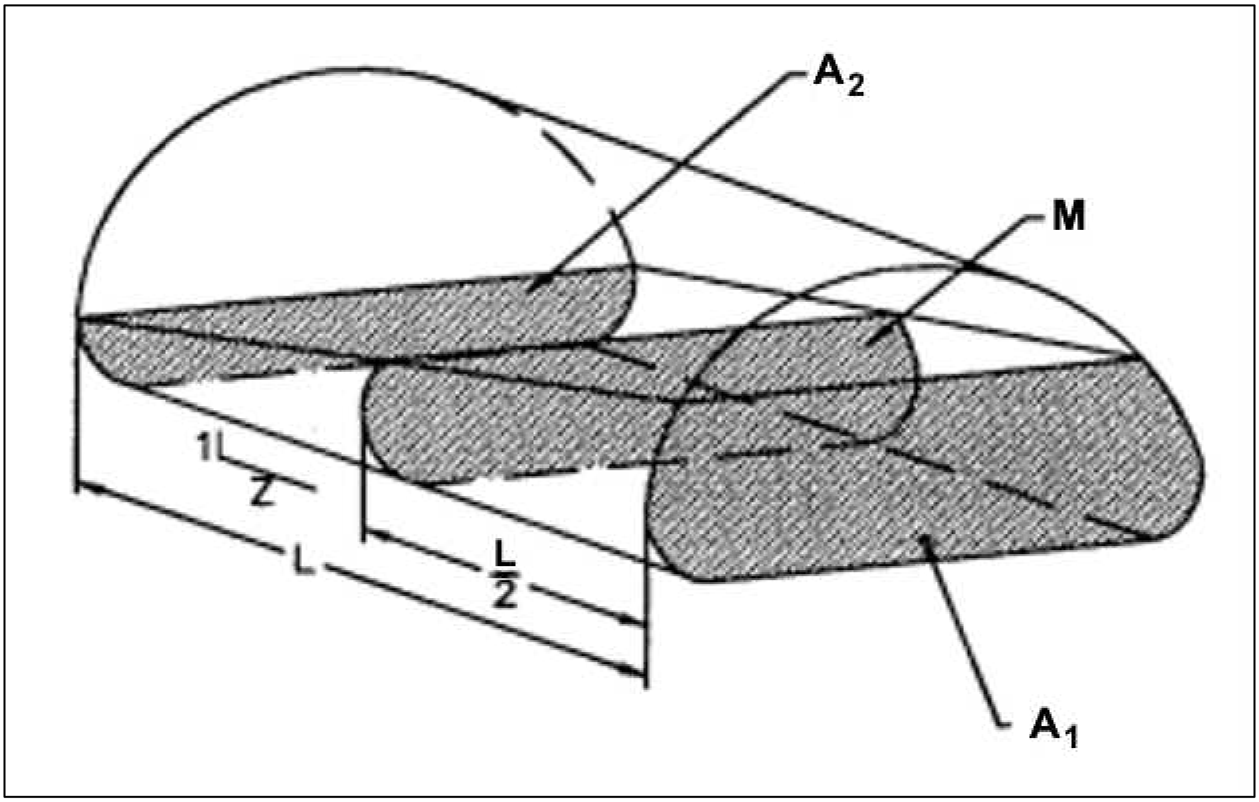

Figure 10.10. Definition sketch for prismoidal formula.

For a circular conduit, as illustrated in Figure 10.11, an exact storage volume results from when d ≤ 2r:

$$V = \frac{H\,(0.667a^3 \pm cB)}{r \pm c} \tag{10.13}$$

$$a = \left[(2r - d)d\right]^{0.5} \tag{10.14}$$

$$c = d - r \tag{10.15}$$

$$\alpha = 2\sin^{-1}\left(\frac{a}{r}\right) \tag{10.16}$$

where:

V	=	Volume of storage, ft^3 (m^3)
B	=	Cross-sectional end area at depth d, ft^2 (m^2)
H	=	Wetted pipe length, ft (m)
r	=	Pipe radius, ft (m)

α	=	Angle as described in Figure 10.11, radians
a, c	=	Distances as described in Figure 10.11, ft (m)
d	=	Flow depth in pipe, ft (m)

To assist in the determination of the cross-sectional area of B, the area of the associated circular segment is:

$$A_s = (\alpha - \sin\alpha)\left(\frac{r^2}{2}\right) \tag{10.17}$$

where:

A_s	=	Segment area, ft^2 (m^2)

Compute the wetted area as follows:

For $d \leq r$; $B = A_s$

For $d > r$; $B = A - A_s$

A is the total pipe area. Alternatively, various texts, such as Brater and King (1976), contain tables and charts for use in determining the depths and areas described in the above equations.

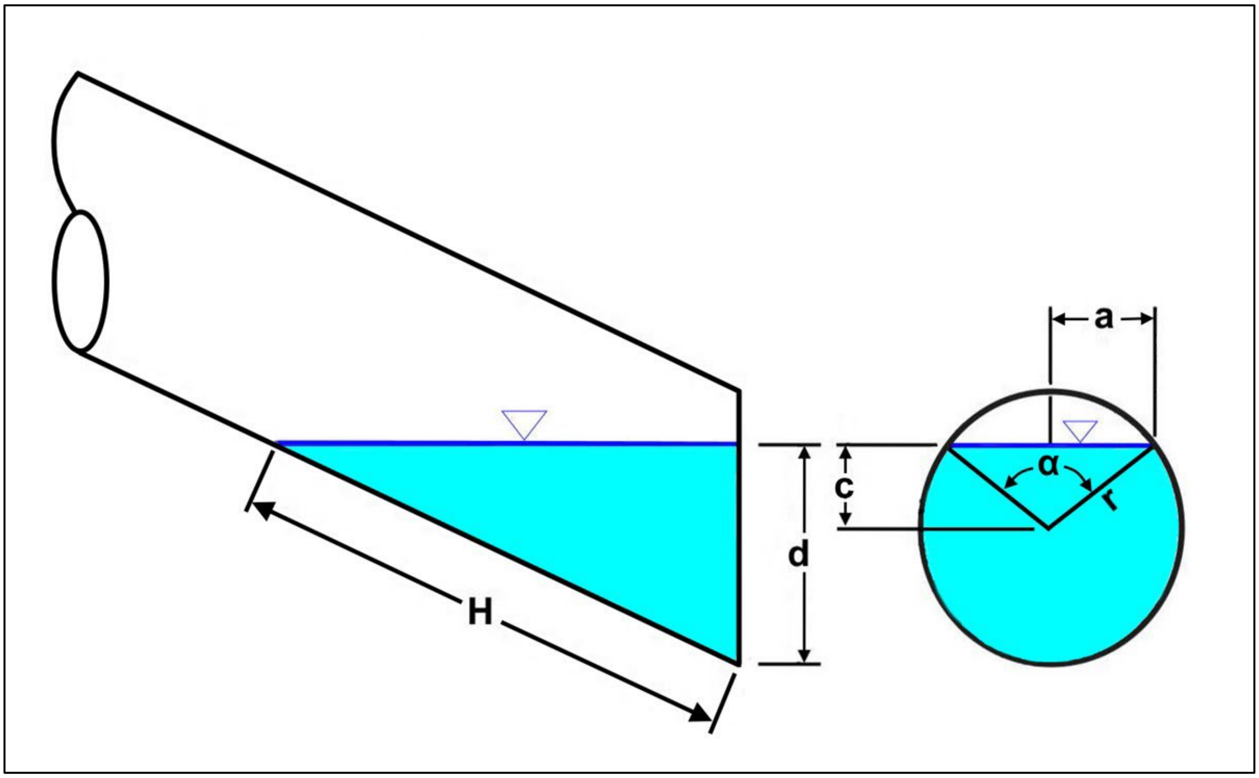

Figure 10.11. Definition sketch for a circular conduit.

Example 10.5: Estimation of pipe storage.

Objective: Develop a stage-storage tabulation between elevations 98 ft (30 m) and 103 ft (31.5 m)

Given: A storm drain pipe having the following properties:

D = 60 inches (1500 mm)
S = 0.01 ft/ft (m/m)
L = 820 ft (250 m)
Invert = 98 ft (30 m)

Step 1. Solve for the volume of storage.

Using equations 10.13 and 10.17:

$V = H [(2/3\ a^3 \pm c\ B) / (r \pm c)]$

$A_s = (\alpha - \sin \alpha) (r^2 / 2)$

Note that: $B = A_s$ for $d \leq r$

$B = A - A_s$ for $d > r$

Step 2. Tabulate storm drain pipe stage-storage curve.

Table 10.3 summarizes the results.

Table 10.3. Storm drain pipe stage-storage relationship.

d (ft)	a (ft)	c (ft)	H (ft)	alpha (rad)	B (ft^2)	V (ft^3)
0.0	0.00	-2.46	0.0	0.000	0.0	0.0
0.7	1.67	-1.80	66.0	1.495	1.5	41.6
1.3	2.18	-1.15	131.0	2.171	4.1	222.8
2.0	2.41	-0.49	197.0	2.739	7.1	588.8
2.6	2.45	0.16	262.0	3.008	10.3	1159.9
3.3	2.32	0.82	328.0	2.462	13.5	1940.0
3.9	1.97	1.48	394.0	1.855	16.3	2917.3
4.6	1.23	2.13	459.0	1.045	18.5	4061.1
4.9	0.00	2.46	492.0	0.000	19.01	4676.9

Solution: Table provides the tabular pipe stage-storage curve.

10.3.2.4 Irregular Basins

Rectangular and trapezoidal basins often oversimplify common detention and retention basins. However, the concepts used to derive the stage-storage relationship for the simple forms also apply to deriving the stage-storage relationship for an actual site that exhibits irregular shapes. The stage-storage relationship is derived as a discrete function (i.e., a set of points) from contours on topographic mapping.

Designers usually develop the storage volume for irregular basins using a topographic map and the average-end area or conic section formulas. The average-end area formula is expressed as:

$$V_{1,2} = \left[\frac{(A_1 + A_2)}{2}\right] d \tag{10.18}$$

where:

$V_{1,2}$	=	Storage volume between elevations 1 and 2, ft^3 (m^3)
A_1	=	Horizontal area at elevation 1, ft^2 (m^2)
A_2	=	Horizontal area at elevation 2, ft^2 (m^2)
d	=	Change in elevation between points 1 and 2, ft (m)

Generally, the average-end area formula approximates irregular basin storage volume well when calculating volumes with small changes in elevation between respective elevations or when the basin width or length is changing but not both.

The conic section formula approximates irregular basin storage volume more accurately when both length and width of a basin are changing as shown in Figure 10.12. The conic approximation is:

$$V = d\left[\frac{A_1 + (A_1 A_2)^{0.5} + A_2}{3}\right] \tag{10.19}$$

where:

V	=	Volume of frustum of a pyramid, ft^3 (m^3)
A_1	=	Surface area at elevation 1, ft^2 (m^2)
A_2	=	Surface area at elevation 2, ft^2 (m^2)
d	=	Change in elevation between points 1 and 2, ft (m)

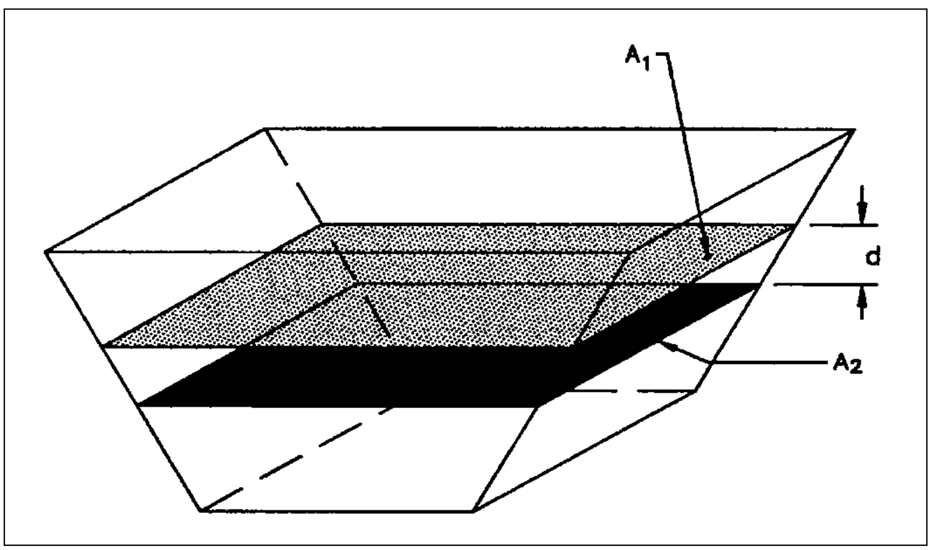

Figure 10.12. Definition sketch for irregular basins.

Example 10.6: Storage estimate of basin for an irregular basin.

Objective: Estimate the maximum volume using topography and average area method.

Given: Topographic map for site shown in Figure 10.13.

Step 1. Using average area method, develop a stage-storage curve.

The area bounded by each 1-ft contour line is estimated and the average area within adjacent contours is computed. Using equation 10.9 compute the stage-storage relationship which is summarized in Table 10.4 and plotted in Figure 10.14.

Solution: Based on the contours of the site, the maximum storage is 61,985 ft^3 (1760 m^3).

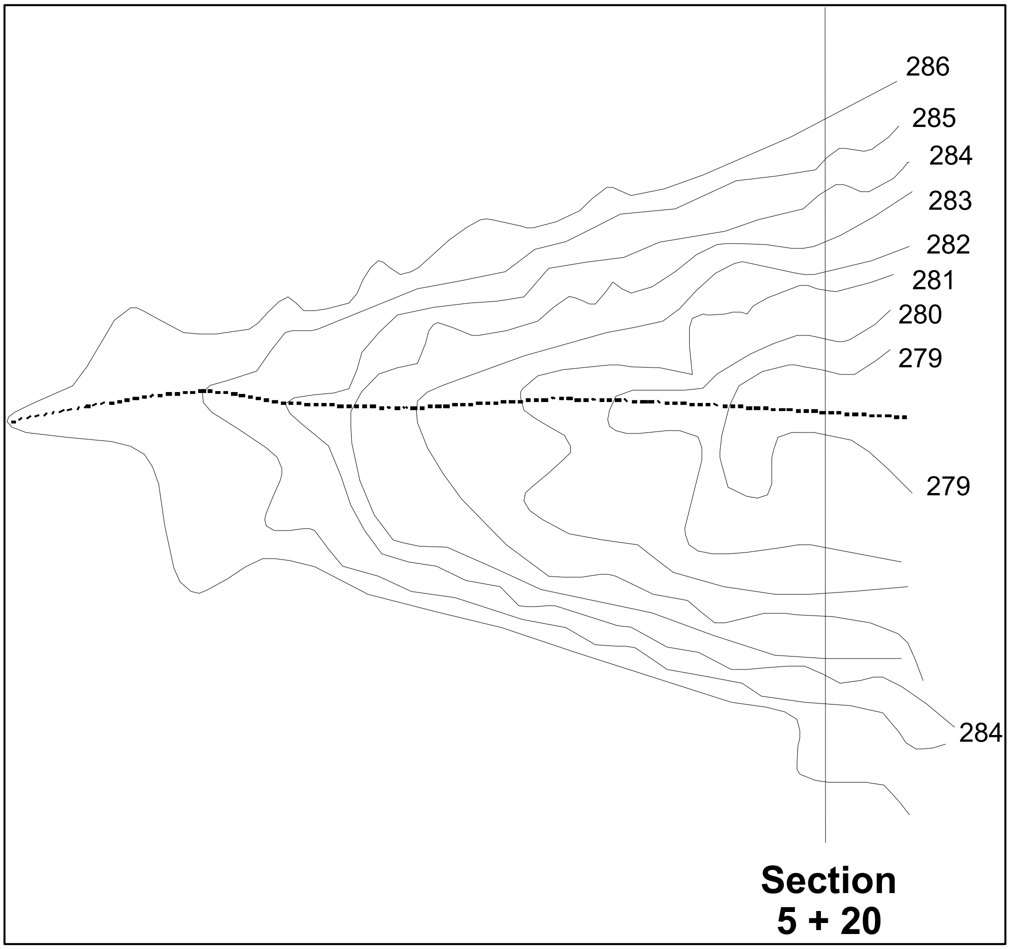

Figure 10.13. Topographic map for deriving stage-storage relationship at site of structure (section 5+20).

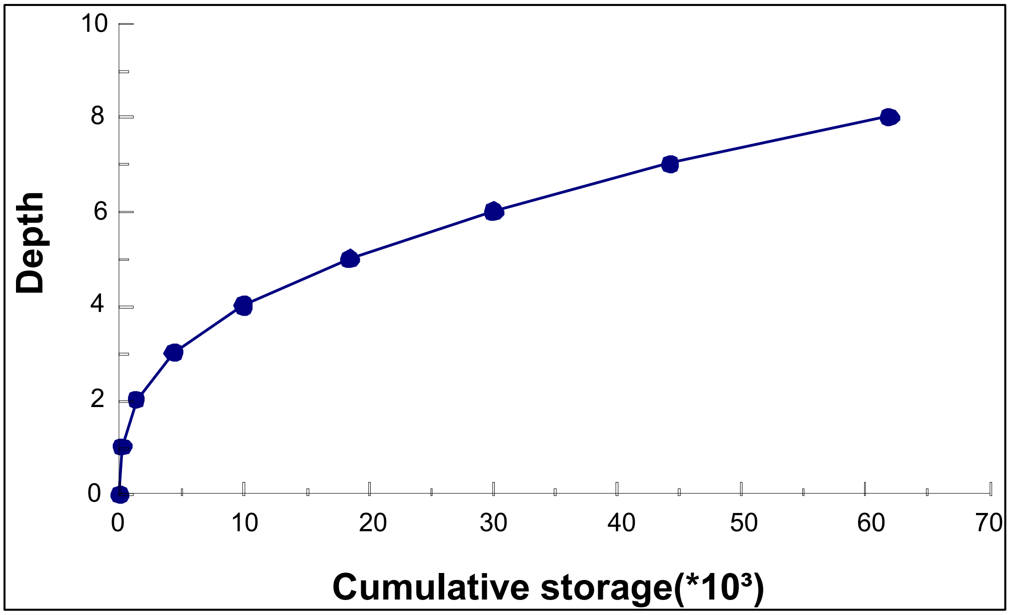

Figure 10.14. Stage-storage relationship.

Table 10.4. Derivation of stage-storage relationship for example irregular basin.

Contour Elevation (ft)	Total Area within Contour Elevation (ft^2)	Average Area (ft^2)	Contour Interval (ft)	Depth (ft)	Change in Storage (ft^2)	Cumulative Storage (ft^3)
278	0			0		0
		245	1		245	
279	490			1		245
		1,115	1		1,115	
280	1,740			2		1,360
		3,015	1		3,015	
281	4,290			3		4,375
		5,585	1		5,585	
282	6,880			4		9,960
		8,600	1		8,600	
283	10,320			5		18,560
		11,575	1		11,575	
284	12,830			6		30,135
		14,165	1		14,165	
285	15,500			7		44,300
		17,685	1		17,685	
286	19,870			8		61,985

Many storage facilities include a permanent pool. In such cases, the elevation of the weir or the bottom of the orifice is set above the elevation of the bottom of the pond. Storage below the outlet elevation is called dead storage. Storage above the outlet elevation is called active storage. Total storage equals the sum of the active and dead storages.

Example 10.7:	Dead and active storage.
Objective:	Determine dead and active storages.

Given: A storage facility with dead and active storage with the following characteristics.

Δh = 0.25 ft (0.076 m)
A_b = 2,000 ft^2 (186 m^2) (bottom surface)

Step 1. Establish depth increment.

Use a topographic map of the detention facility site to measure areas. Use a depth increment of 0.25 ft for computation.

Step 2. Tabulate stage-active storage relationship.

Table 10.5 can be used to illustrate the development of stage-active storage relationship. Column 2 gives the areas and column 3 gives the average areas. The incremental volumes equal the product of the change in depth, 0.25 ft, and the average area. The total storage at a given depth is the sum of the incremental storages up to and including that depth.

Table 10.5. Computation of stage-active storage relationship for example basin.

(1) Depth h (ft)	(2) Surface Area, A (ft^2)	(3) Average Area, A (ft^2)	(4) Incremental Volume ΔV (ft^3)	(5) Total Volume V (ft^3)	(6) Dead Storage V_d (ft^3)	(7) Active Storage V_a (ft^3)
0	2,000	-	-	0	0	0
-	-	2,100	525.0	-	-	-
0.25	2,200	-	-	525.0	525.0	0
-	-	2,250	562.5	-	-	-
0.50	2,300	-	-	1087.5	1087.5	0
-	-	2,400	600.0	-	-	-
0.75	2,500	-	-	1687.5	1087.5	600.0
-	-	2,650	662.5	-	-	-
1.00	2,800	-	-	2350.0	1087.5	1262.5
-	-	2,900	725.0	-	-	-
1.25	3,000	-	-	3075.0	1087.5	1987.5
-	-	3,050	762.5	-	-	-
1.50	3,100	-	-	3837.5	1087.5	2750.0

Solution:	If the outlet facility has a minimum elevation of 0.5 ft (0.15 m), all storage below this elevation is dead storage. The active storage is 0.0 at an elevation of 0.5 ft (0.15 m). The active storage above 0.5 ft (0.15 m) equals the difference between the total storage and the dead storage. The stage (column 1) versus active storage (column 7) would be used when designing a storage facility.

10.3.3 Stage-Discharge Relationship (Performance Curve)

A stage-discharge (performance) curve describes the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has both a principal and an emergency outlet. The principal outlet usually conveys the design flood without allowing flow to enter the emergency outlet for flooding events up to the design flood for the principal outlet. The principal outlet structure typically consists of combinations of pipe culverts, weirs, orifices, or other appropriate hydraulic controls selected to satisfy single or multiple applicable hydrologic criteria. Designers develop composite stage-discharge curves by estimating the cumulative effects of the discharge rating relationships for each component of the outlet structure.

10.3.3.1 Discharge Pipes

Detention facilities typically have discharge pipes as part of the outlet structure and may also have a riser. The design of these pipes can be for either single or multistage discharges. A single step discharge system consists of a single culvert entrance system and is not designed to carry emergency flows. A multi-stage outlet structure adds a riser at the inlet end of the pipe. The design of an inlet structure allows the design discharge to pass through a weir or orifice in the lower levels of the structure and the emergency flows to pass over the top of the structure or through an emergency spillway. The design of the pipe allows it to carry the full range of flows from a drainage area.

For outlets without a riser, the facility design resembles a simple culvert. HDS-5 (FHWA 2012a) outlines appropriate design procedures. For multistage control structures, design of the inlet control structure considers both the design flow and the emergency flows. Designers develop a stage-discharge curve for the full range of flows the structure would experience. Typically, the design flows are orifice flow through whatever shape the designer has chosen while typically the higher flows are weir flow over the top of the control structure. Designers can use the equations in Section 10.3.3.2 for orifices and the equations in Section 10.3.3.3 for weirs. Designers select the pipe to carry all flows considered in the design of the control structure.

In designing a multistage structure, the designer first develops peak flows that must be passed through the facility. Next, the designer selects a pipe that passes the peak flow within the allowable headwater and develops a performance curve for the pipe. Third, the designer develops a stage-discharge curve for the inlet control structure, recognizing that the headwater for the discharge pipe will be the tailwater that needs to be considered in designing the inlet structure. Last, the designer uses the stage-discharge curve in the basin routing procedure.

Example 10.8:	Sizing detention basin outlet pipe.
Objective:	Find the size pipe needed to carry the maximum allowable flow rate from the detention basin.

Given: The maximum head on pipe (H_{max}), inlet invert elevation, length (L), slope (S), roughness (n), and square edge entrance coefficient (K_e), for a corrugated steel discharge pipe as shown in Figure 10.3. The discharge pipe outfall is free (not submerged).

H_{max}	=	2.3 ft (0.75 m)
Invert	=	32.8 ft (10.0 m)
L	=	164 ft (50 m)
S	=	0.04 ft/ft (m/m)
n	=	0.024

K_e = 0.5

Step 1. Evaluate the pipe for inlet and outlet control.

Using the same discharges from example 10.2, the maximum predeveloped discharge from the watershed is 19.4 ft^3/s (0.55 m^3/s). Since the discharge pipe can function under inlet or outlet control, the pipe size will be evaluated for both conditions. The larger pipe size will be selected for the final design.

Step 2. Size the pipe for inlet control.

Using HDS-5 (FHWA 2012a) yields the relationship between head on the pipe and the resulting discharge for inlet control. The analysis shows the pipe diameter that will carry the flow under the specified conditions is 30 inches.

Step 3. Size the pipe for outlet control.

Using HDS-5 (FHWA 2012a) yields the relationship between head on the pipe and discharge for barrel (outlet) control. The analysis shows the pipe diameter that will carry the flow under the specified conditions is 27 inches.

Solution: For the design, select pipe diameter equal to 30 inches (750 mm).

10.3.3.2 Orifices

Figure 10.15 shows a schematic of a tank with a hole of area in its bottom. Assuming all losses can be neglected, write Bernoulli's equation between a point on the surface of the pool (point 1) and a point in the cross-section of the orifice (point 2):

$$\frac{P_1}{\gamma}+\frac{V_1^2}{2g}+z_1=\frac{P_2}{\gamma}+\frac{V_2^2}{2g}+z_2 \tag{10.20}$$

where:

P	=	Pressure, lb/ft^2 (N/m^2 or Pa)
V	=	Velocity, ft/s (m/s)
γ	=	Specific weight, lb/ft^3 (N/m^3)
z	=	Height above datum, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

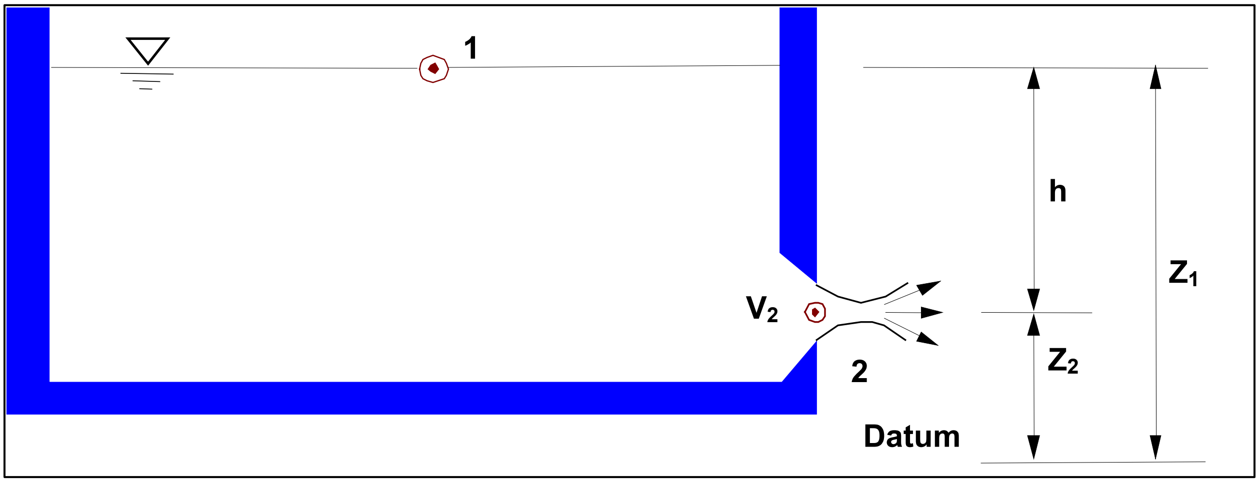

Figure 10.15. Schematic diagram of flow through an orifice.

This can be simplified by assuming: 1) the pressure at both points is atmospheric, therefore $p_1 = p_2$; 2) the surface area of the pool A_1 is very large relative to the area of the orifice A_2, so from the continuity equation V_1 is essentially zero; and 3) $z_1 - z_2 = h$. Equation 10.20 becomes:

$$h = \frac{V_2^2}{2g} \tag{10.21}$$

Solving for V_2 and substituting it into the continuity equation yields:

$$Q = AV = A\sqrt{2gh} \tag{10.22}$$

Equation 10.22 assumes ideal conditions with zero energy losses and atmospheric pressure across the opening of the orifice. It is actually atmospheric at a point below the orifice, where the cross-sectional area of the discharging water is slightly smaller than the area of the orifice. To account for these conditions, the orifice flow equation introduces a discharge coefficient:

$$Q = C_o A_o \sqrt{(2gh_o)} \tag{10.23}$$

where:

Q	=	Orifice flow rate, ft^3/s (m^3/s)
C_o	=	Discharge coefficient
A_o	=	Area of orifice, ft^2 (m^2)
h_o	=	Effective head on the orifice measured from the centroid of the opening, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2)

Values of C_o range from 0.5 to 1.0, with a value of 0.6 commonly used, for square-edged, uniform orifice entrance conditions. For ragged edged orifices, such as those resulting from the use of an acetylene torch to cut orifice openings in corrugated pipe, use a value of 0.4.

If the orifice is not horizontal, the depth, h, is usually measured from the center of area of the orifice.

If the orifice discharges as a free outfall, i.e., unsubmerged, then measure the effective head from the centerline of the orifice to the upstream water surface elevation. For a submerged orifice discharge, the effective head equals the difference in elevation of the upstream and downstream water surfaces. Figure 10.16 shows this latter condition of a submerged discharge.

Orifice or Culvert

Pipes smaller than 1 ft (0.3 m) in diameter may be analyzed as a submerged orifice when h_o/D is greater than 1.5. Headwater and tailwater effects influence pipes greater than 1 ft (0.3 m) in diameter, which function as discharge pipes, not just as an orifice; analysis accordingly incorporates this design consideration.

Designers use orifice plates on riser structures to control outflow from a detention pond. As shown in Figure 10.17, orifice plates consist of multiple openings. Compute flow through multiple orifices by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, multiplying the discharge for a single orifice by the number of openings yields the total flow.

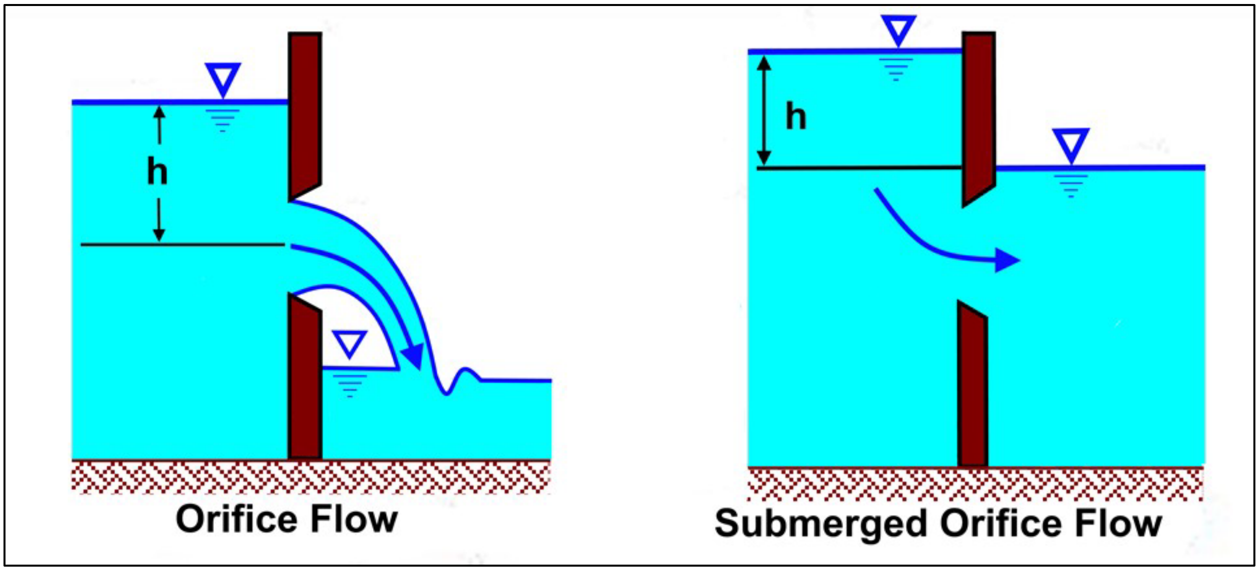

Figure 10.16. Definition sketch for orifice flow submergence.

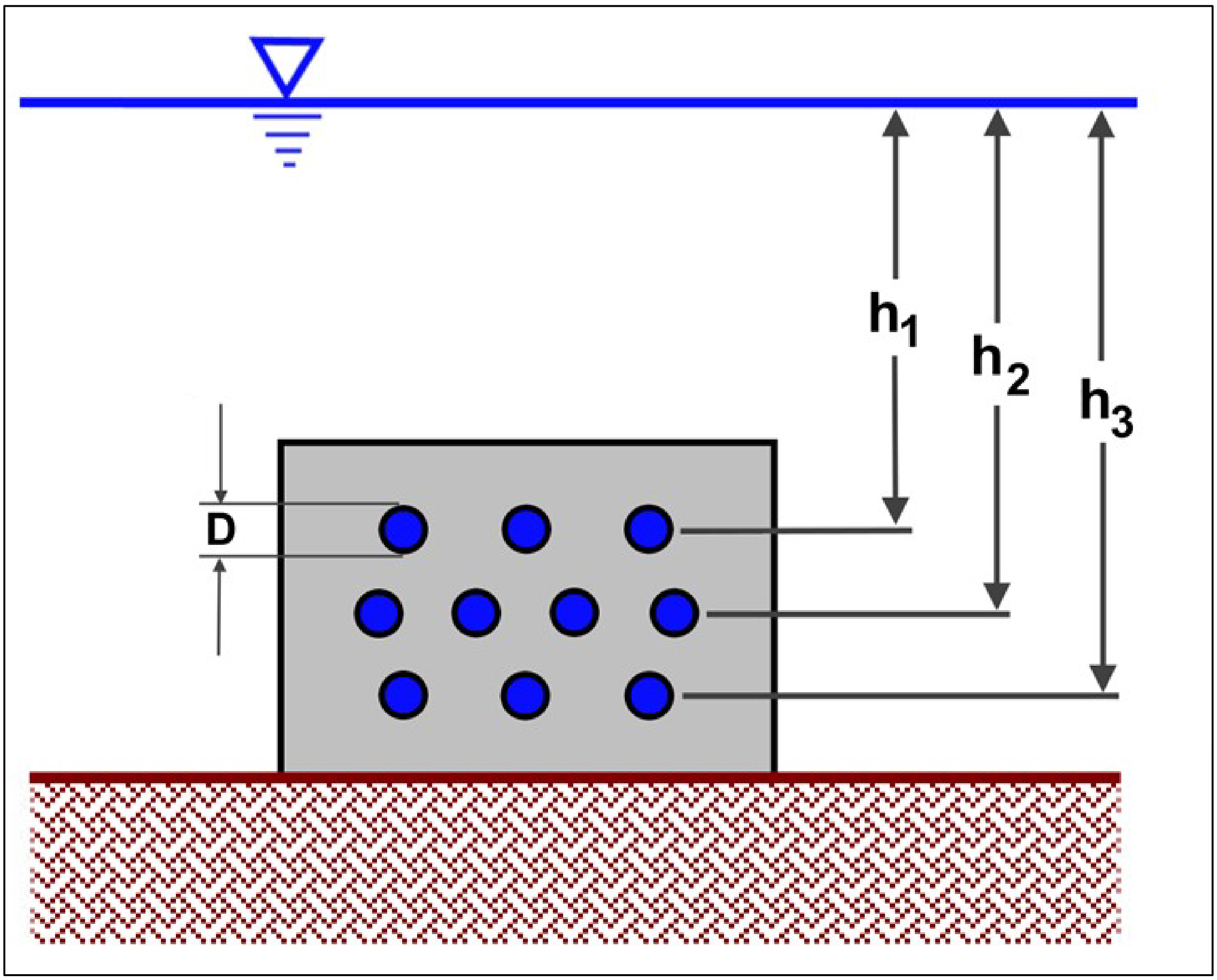

Figure 10.17. Orifice plate with multiple openings.

Example 10.9:	Orifice flow through an orifice plate.
Objective:	Estimate total discharge through an orifice plate with multiple openings.

Given: Given the orifice plate in Figure 10.17 with a free discharge and:

D = 1.0 inch or 0.0833 ft (25 mm)
h_1 = 3.61 ft (1.1 m)
h_2 = 3.94 ft (1.2 m)
h_3 = 4.26 ft (1.3 m)

Step 1. Use a modification of equation 10.23 to calculate the flows for each row of orifices.

$Q_1 = (0.6)\,[(3)(\pi{*}0.0833^2 / 4)]\,[2{*}32.2{*}3.61]^{0.5} = 0.15\ ft^3/s$

$Q_2 = (0.6)\,[(4)(\pi{*}0.0833^2 / 4)]\,[2{*}32.2{*}3.94]^{0.5} = 0.21$

$Q_3 = (0.6)\,[(3)(\pi{*}0.0833^2 / 4)]\,[2{*}32.2{*}4.26]^{0.5} = 0.16$

Step 2. Sum the orifice flows for each row of equal heads to arrive at the total flow.

$Q_{total} = Q_1 + Q_2 + Q_3 = 0.52\ ft^3/s$

Solution: The flow through the orifice plate equals 0.52 ft^3/s (0.015 m^3/s).

Example 10.10:	Orifice stage-discharge rating curve.
Objective:	Develop the stage-discharge rating curve between 32.80 ft (10 m) and 39.37 ft (12.0 m).

Given: Given the circular orifice in Figure 10.16(a) with:

D = 0.49 ft (0.15 m)
invert = 32.80 ft (10.0 m)
C_o = 0.60

Using equation 10.23 with D = 0.49 ft yields the following relationship between the effective head on the orifice (h_o) and the resulting discharge:

h_o = Depth - D/2

For D = 3.3 ft, $h_o = 3.3 - (0.49)/2 = 3.06$

$Q = C_o A_o [(2g(h_o)]^{0.5} = (0.6)(\pi{*}0.49^2 / 4)\,[2{*}32.2{*}3.06]^{0.5} = 1.59\ ft^3/s$

Table 10.6 summarizes the stage-discharge curve.

Solution: The table summarizes the stage-discharge tabulation for the orifice.

Table 10.6. Stage-discharge orifice flow example.

Depth (ft)	Stage (ft)	Discharge (ft^3/s)
0	32.8	0
0.7	33.5	0.61
1.3	34.1	0.93
2	34.8	1.2
2.6	35.4	1.39
3.3	36.1	1.59
3.9	36.7	1.74
4.6	37.4	1.89
5.2	38.1	2.04
5.9	38.7	2.16
6.6	39.4	2.29

10.3.3.3 Weirs

The following sections provide relationships for sharp-crested, broad-crested, V-notch, and proportional weirs.

10.3.3.3.1 Sharp-Crested Weirs

Consider the cross-section shown in Figure 10.18. Point 1 is located upstream of the weir at a distance where the weir does not influence the flow characteristics. Point 2 is at the weir. The following analysis assumes: (1) ideal flow, (2) frictionless flow, (3) critical flow conditions at the obstruction, and (4) the obstruction has a unit width perpendicular to the direction of flow.

For the critical flow conditions, the following equations describe the hydraulics at the obstruction:

$$F_r = 1 = \frac{V_c}{(gy_c)^{0.5}} \tag{10.24}$$

$$y_c = \left(\frac{q_u^2}{g}\right)^{\frac{1}{3}} \tag{10.25}$$

$$E_c = y_c + \frac{V_c^2}{2g} = \frac{3}{2}y_c \tag{10.26}$$

where:

F_r	=	Froude number
V_c	=	Critical velocity, ft/s (m/s)
y_c	=	Critical depth, ft (m/s)
q_u	=	Discharge rate per unit width, ft^2/s (m^2/s)
E_c	=	Minimum specific energy, ft (m)

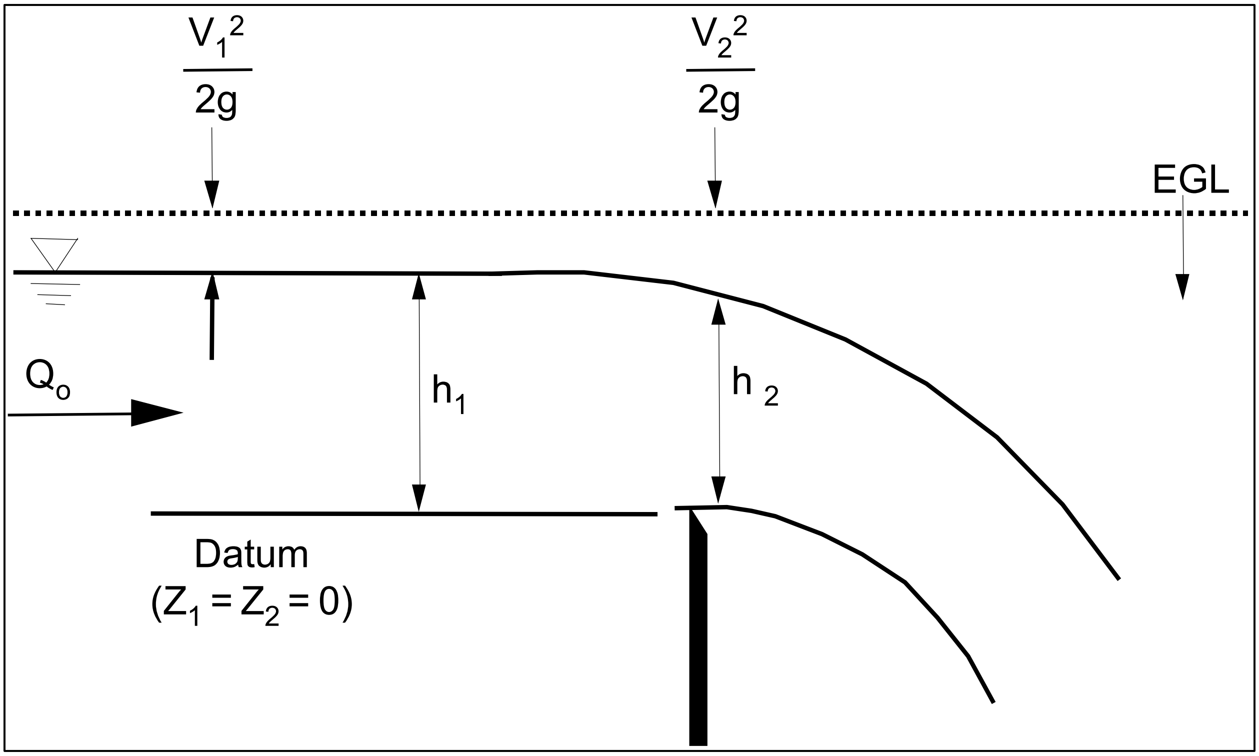

Figure 10.18. Schematic diagram of flow over a sharp-crested weir

Assuming hydrostatic pressure at sections 1 and 2, then $P_i/\gamma = h_i$. Thus, Bernoulli's equation is:

$$h_1 + \frac{V_1^2}{2g} + z_1 = h_2 + \frac{V_2^2}{2g} + z_2 \tag{10.27}$$

By setting the datum at the top of the weir, $z_1 = z_2 = 0$, assuming that the velocity head at section 1 is much smaller than the velocity head at section 2, and recalling that the flow passes through critical depth as it passes over the weir, then $V_2 = V_C$ and $h_2 = y_C$, reducing equation 10.27 to:

$$h_1 = \frac{V_c^2}{2g} + y_c \tag{10.28}$$

The velocity head is $V_c^2/2g = y_c/2$. Defining $h = h_1$, then:

$$h = \frac{y_c}{2} + y_c = \frac{3y_c}{2} \text{ or } y_c = \frac{2}{3}h \tag{10.29}$$

Solving equation 10.25 for q_u, it then follows that:

$$q_u = \left(gy_c^3\right)^{0.5} = \left[g\left(\frac{2}{3}h\right)^{0.5}\right] = \left(\frac{8g}{27}\right)^{0.5} h^{\frac{3}{2}} = \left(\frac{2}{\sqrt{27}}\right)\sqrt{2g}h^{\frac{3}{2}} \tag{10.30}$$

Letting Q = Lq_u, the general weir equation is:

$$Q = \left(\frac{2}{\sqrt{27}}\right)\sqrt{2g}\,h^{1.5}\,L \qquad (10.31)$$

where:

Q	=	Discharge over a horizontal weir, ft^3/s (m^3/s)
h	=	Head (depth) of approach flow above the weir, ft (m)
L	=	Weir length, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s^2 (9.81 m/s^2).

Weir Coefficient

It is significant to note that many presentations of the weir equation embed the 2g into the coefficient, C_w, making it a dimensioned rather than dimensionless quantity. By keeping the gravity term in the equation, C_w becomes a property of a specific weir type and not dependent on the system of units.

Equation 10.31 represents ideal flow over a weir. Because actual weirs perform less efficiently, engineers modify the equation by the addition of a weir coefficient, C_w The value of C_w depends on the type of weir, head, weir height, and other factors. It also includes the initial quotient in equation 10.31.

$$Q = C_w\sqrt{2g}\,h^{1.5}\,L \qquad (10.32)$$

For the sharp-crested weir of this derivation, values of C_w can range from 0.27 to 0.38. The range reflects the variation in losses from alternative weir/flow configurations. Losses depend on the depth of flow over and approaching the weir, weir length, weir thickness, and weir height. Even laboratory studies have difficulty accurately estimating C_w. Designers can use a value of 0.37 for sharp-crested rectangular weirs where more information is not available.

Equation 10.32 provides the discharge relationship for sharp-crested weirs with no end contractions (illustrated in Figure 10.19). Designers typically treat flow over the top edge of a riser pipe as flow over a sharp-crested weir with no end constrictions.

Equation 10.33 provides the discharge equation for sharp-crested weirs with end contractions (illustrated in Figure 10.19). As indicated above, the value of the coefficient C_w varies with the ratio h/h_c (see Figure 10.20 for definition of terms). For values of the ratio h/h_c less than 0.3, designers often use a constant C_w of 0.415.

$$Q = C_w\sqrt{2g}\left(L - 0.2\,h\right)h^{1.5} \qquad (10.33)$$

Submergence affects sharp-crested weirs when the tailwater rises above the weir crest elevation, as shown in Figure 10.20. These effects reduce discharge over the weir. The discharge equation for a submerged sharp-crested weir is (Brater and King 1976):

$$Q_s = Q\left(1 - \left(\frac{h_2}{h_1}\right)^{1.5}\right)^{0.385} \qquad (10.34)$$

where:

Q_s	=	Submerged flow, ft^3/s (m^3/s)
Q	=	Unsubmerged weir flow, ft^3/s (m^3/s)

h_1 = Upstream head above crest, ft (m)
h_2 = Downstream head above crest, ft (m)

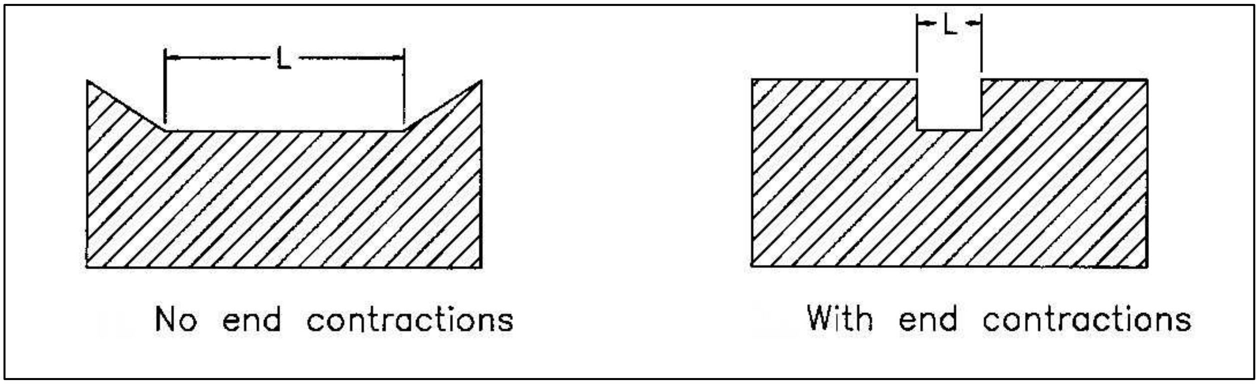

Figure 10.19. Sharp-crested weir contractions.

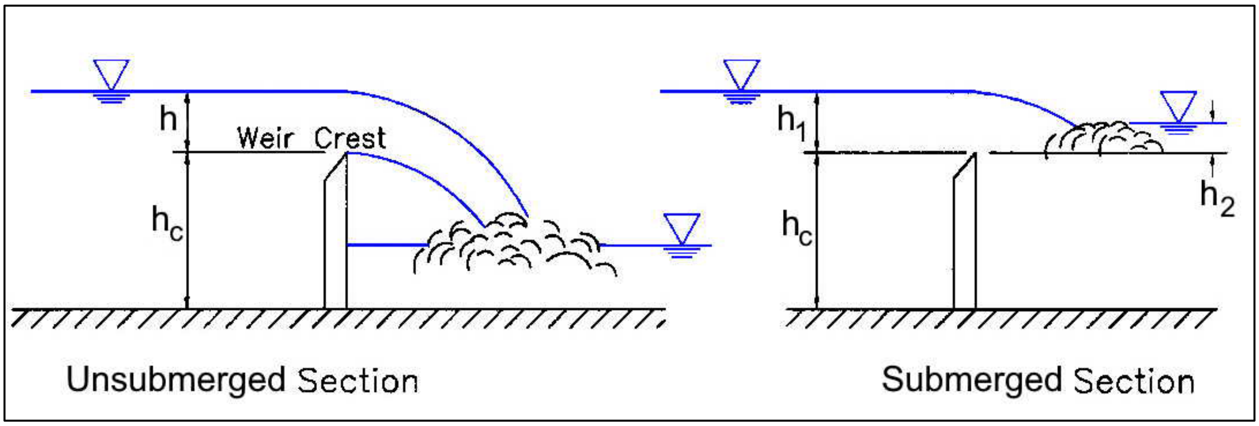

Figure 10.20. Sharp-crested weir submergence.

10.3.3.3.2 Broad-Crested Weir

Equation 10.32 also applies to a broad-crested weir. 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 weir coefficient value of 0.41. For sharp corners on the broad-crested weir, use a minimum value of 0.29. Table 10.7 provides additional information on C_w values as a function of weir crest breadth and head.

10.3.3.3.3 V-Notch Weir

Figure 10.21 illustrates discharge through a v-notch weir. Calculate discharge from (Brater and King 1976):

$$Q = C_w \sqrt{2g}\left[\tan\left(\frac{\theta}{2}\right)\right]h^{2.5} \qquad (10.35)$$

where:

Q = Discharge, ft^3/s (m^3/s)
Θ = Angle of v-notch, degrees
h = Head on apex of v-notch, ft (m)
C_w = Weir coefficient for a v-notch weir (typically equal to 0.31)

Table 10.7. Broad-crested weir coefficient values (adapted from Brater and King 1976).

Head * (ft)	Breadth of Weir Crest (ft)										
	0.5	0.75	1	1.5	2	2.5	3	4	5	10	15
0.2	0.35	0.34	0.34	0.33	0.32	0.31	0.30	0.30	0.29	0.31	0.33
0.4	0.36	0.35	0.34	0.33	0.33	0.32	0.32	0.32	0.31	0.32	0.34
0.6	0.38	0.36	0.34	0.33	0.33	0.32	0.33	0.34	0.34	0.34	0.34
0.8	0.41	0.38	0.36	0.33	0.32	0.32	0.33	0.33	0.33	0.34	0.33
1.0	0.41	0.39	0.37	0.34	0.33	0.33	0.33	0.33	0.33	0.33	0.33
1.2	0.41	0.40	0.38	0.36	0.34	0.33	0.33	0.33	0.33	0.34	0.33
1.4	0.41	0.41	0.40	0.36	0.35	0.33	0.33	0.33	0.33	0.33	0.33
1.6	0.41	0.41	0.41	0.38	0.36	0.34	0.33	0.33	0.33	0.33	0.33
1.8	0.41	0.41	0.41	0.38	0.36	0.34	0.33	0.33	0.33	0.33	0.33
2.0	0.41	0.41	0.41	0.38	0.36	0.34	0.34	0.33	0.33	0.33	0.33
2.5	0.41	0.41	0.41	0.41	0.38	0.36	0.35	0.34	0.33	0.33	0.33
3.0	0.41	0.41	0.41	0.41	0.40	0.38	0.36	0.34	0.33	0.33	0.33
3.5	0.41	0.41	0.41	0.41	0.41	0.40	0.37	0.34	0.33	0.33	0.33
4.0	0.41	0.41	0.41	0.41	0.41	0.41	0.38	0.35	0.34	0.33	0.33
4.5	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.36	0.34	0.33	0.33
5.0	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.38	0.35	0.33	0.33
5.5	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.41	0.36	0.33	0.33

*Measured a distance at least 2.5 times critical depth upstream of the weir.

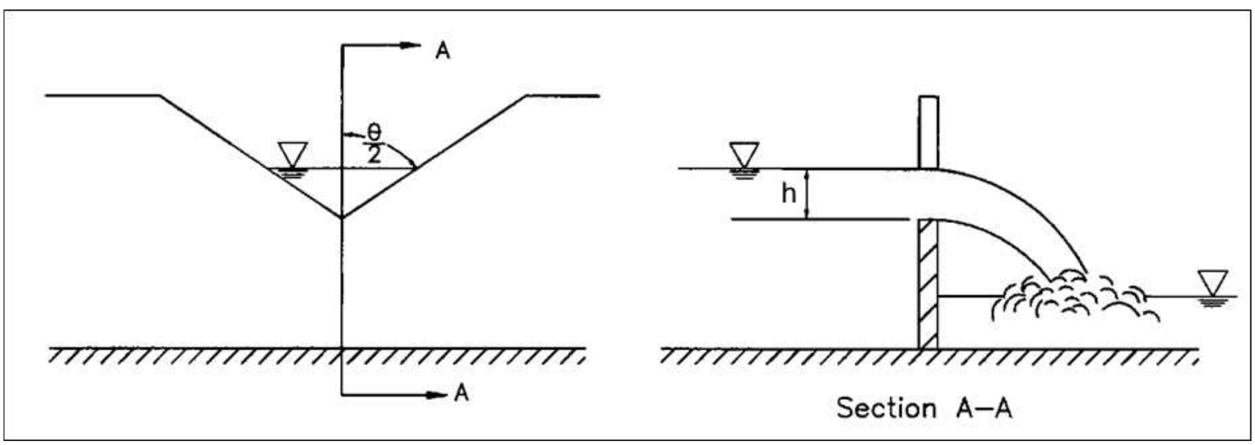

Figure 10.21. V-notch weir.

10.3.3.3.4 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 differs from other control devices by having a linear head-discharge relationship. This relationship is achieved by allowing the discharge area to vary nonlinearly with head with a configuration shown in Figure 10.22. The following equation describes the relations between the dimensions of a proportional weir (Sandvik 1985):

$$\frac{x}{b} = 1 - (0.315)\left[\arctan\left(\frac{y}{a}\right)^{0.5}\right] \qquad (10.36)$$

Discharge for a proportional weir is (Sandvik 1985):

$$Q = C_w \sqrt{2g}\, a^{0.5}\, b\left(h - \frac{a}{3}\right) \qquad (10.37)$$

where:

Q	=	Discharge, m^3/s (ft^3/s)
h	=	Head above horizontal sill, ft (m)
C_w	=	Weir coefficient for a proportional weir (typically equal to 0.62)

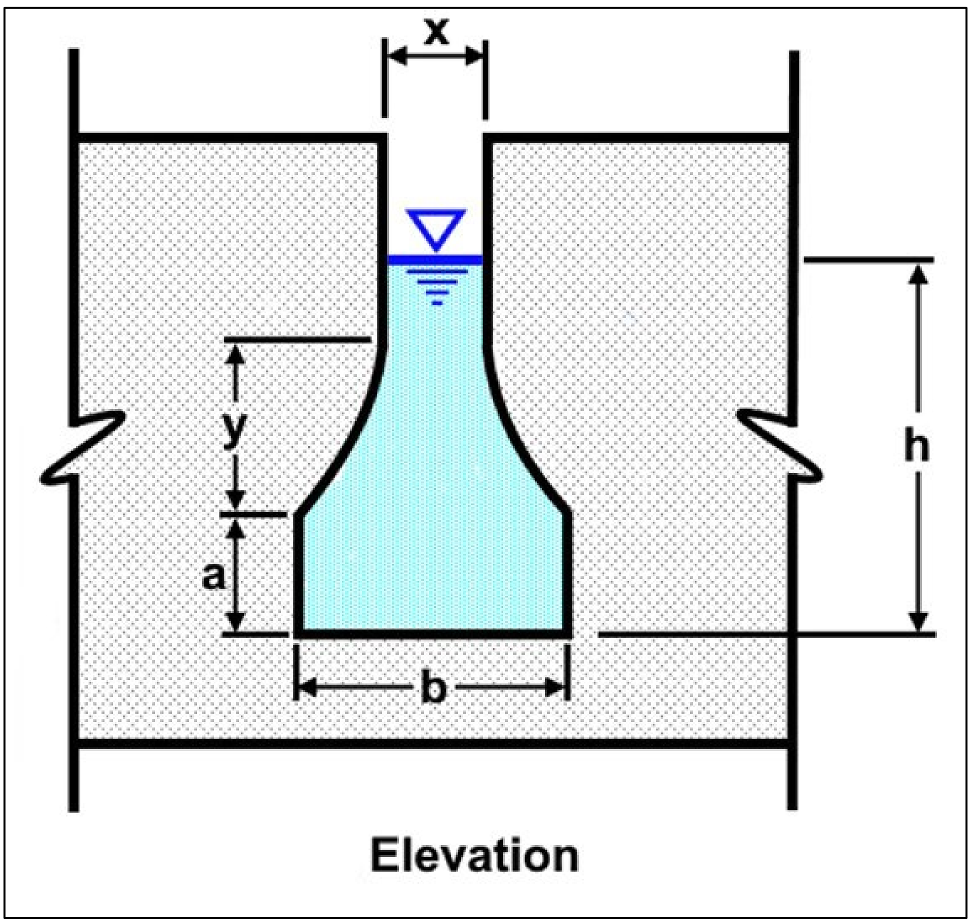

Figure 10.22. Proportional weir.

Example 10.11: Detention pond riser hydraulics.

Objective: Establish a stage-discharge rating curve for a riser pipe for water surface elevations in the pond between 32.8 ft (10.0 m) and 39.4 ft (12.0 m).

Given: The diameter (D), crest elevation (EL), and weir height (H_c) for a riser pipe as shown in Figure 10.23 with the following characteristics:

D = 1.74 ft (0.53 m)

EL = 35.4 ft (10.8 m)

H_c = 2.6 ft (0.8 m)

Since the riser pipe functions as both a weir and an orifice (depending on stage), develop the rating by comparing the stage-discharge produced by both weir and orifice flow.

Step 1. Establish relationship between orifice flow and head.

Using equation 10.23 for orifices with D = 1.74 ft yields the following relationship between the effective head on the orifice (H_o) and the resulting discharge:

$Q = C_o A_o [(2g(h_o)]^{0.5} = (0.6)(\pi * 1.74^2 / 4) [2*32.2*h_o]^{0.5} = (0.6)(\pi\ 1.74^2 /4)(64.4 * h_o)^{0.5}$

$= 11.44\ h_o^{0.5}$

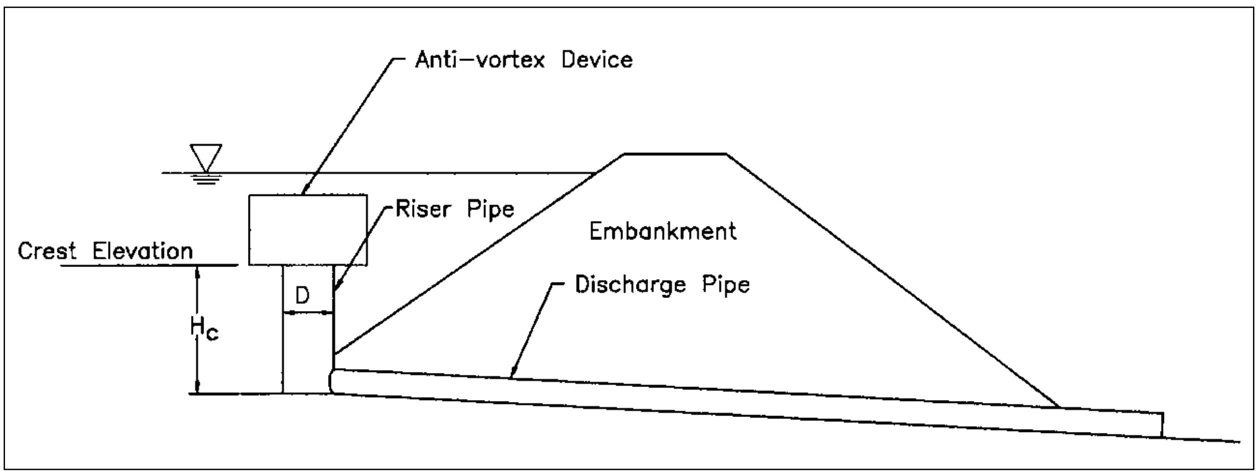

Figure 10.23. Riser pipe configuration for example.

Step 2. Establish relationship between weir flow and head.

Using the dimensionless equation 10.32 for sharp-crested weirs with C_W = 0.41 (h/h_c assumed less than 0.3), and L = pipe circumference = 5.5 ft yields the following relationship between the effective head on the riser (h) and the resulting discharge:

$Q = C_W (2g)^{0.5} L h^{1.5} = (0.41) (64.4)^{0.5} (5.5) h^{1.5} = 18.32 h^{1.5}$

Step 3. Tabulate stage-discharge relationship.

Using the relationships established in steps 1 and 2, develop the stage-discharge curve, shown in Table 10.8.

Table 10.8. Stage-discharge relationship.

Stage (ft)	Effective Head (ft)	Orifice Flow (ft^3/s)	Weir Flow (ft^3/s)
32.8	0	0	0
35.4	0	0	0
35.8	0.33	6.6	3.5 *
36.1	0.66	9.2 *	9.8
36.7	1.31	13.1 *	27.5
37.4	1.97	15.9 *	50.6
38.1	2.63	18.7 *	77.7
38.7	3.28	20.8 *	108.8
39.4	3.94	22.6 *	143.2

*Designates controlling flow.

Solution: The flow condition (orifice or weir) producing the lowest discharge for a given stage determines the controlling relationship. As illustrated in Table 10.8, at a stage of 35.76 ft (10.9 m) weir flow controls the discharge through the riser. However, at and above a stage of 36.09 ft (11.0 m), orifice flow controls the discharge through the riser.

10.3.3.4 Auxiliary Spillway

An auxiliary (emergency) spillway provides a controlled overflow relief for storm flows exceeding the design discharge for the storage facility. A broad-crested overflow weir, as shown in Figure 10.24, represents a suitable emergency spillway for highway detention facilities. The transverse cross-section of the weir cut is typically trapezoidal in shape for ease of construction. The embankment height is typically at an elevation 1 ft to 2 ft above the maximum design storage elevation. It is preferable to have a freeboard of 2 ft minimum. However, for very small impoundments, less than 1- to 2-acre surface area an absolute minimum of 1-foot of freeboard may be acceptable (UDFCD 2001). The USDA-NRCS Engineering Field Handbook (2021) provides design information for auxiliary spillways.

10.3.3.5 Composite Stage-Discharge Relationships

Presented earlier in this chapter, Figure 10.3 shows a schematic of a basin with an outlet structure consisting of a riser and a pipe outlet that together form the outlet structure controlling the discharge from the basin. The riser may include one or more weir or orifice openings. The designer evaluates the outlet structure to develop a composite stage-discharge curve. In addition to determining the diameter of the pipe outlet, the designer also establishes invert elevations of the pipe. For a basin with a permanent pool, both the volume of dead storage and the corresponding elevation of the permanent pool are set.

In the sizing of outlet structures, the designer determines both the required volume of storage and the physical characteristics of the structure including the outlet pipe dimensions; the riser dimensions; orifice and weir dimensions; and the elevations of the pipe outlet, weirs, and orifices. A single-stage riser provides for a single control feature suitable for satisfying a single hydrologic criterion. A multiple-stage riser provides for multiple hydrologic criteria.

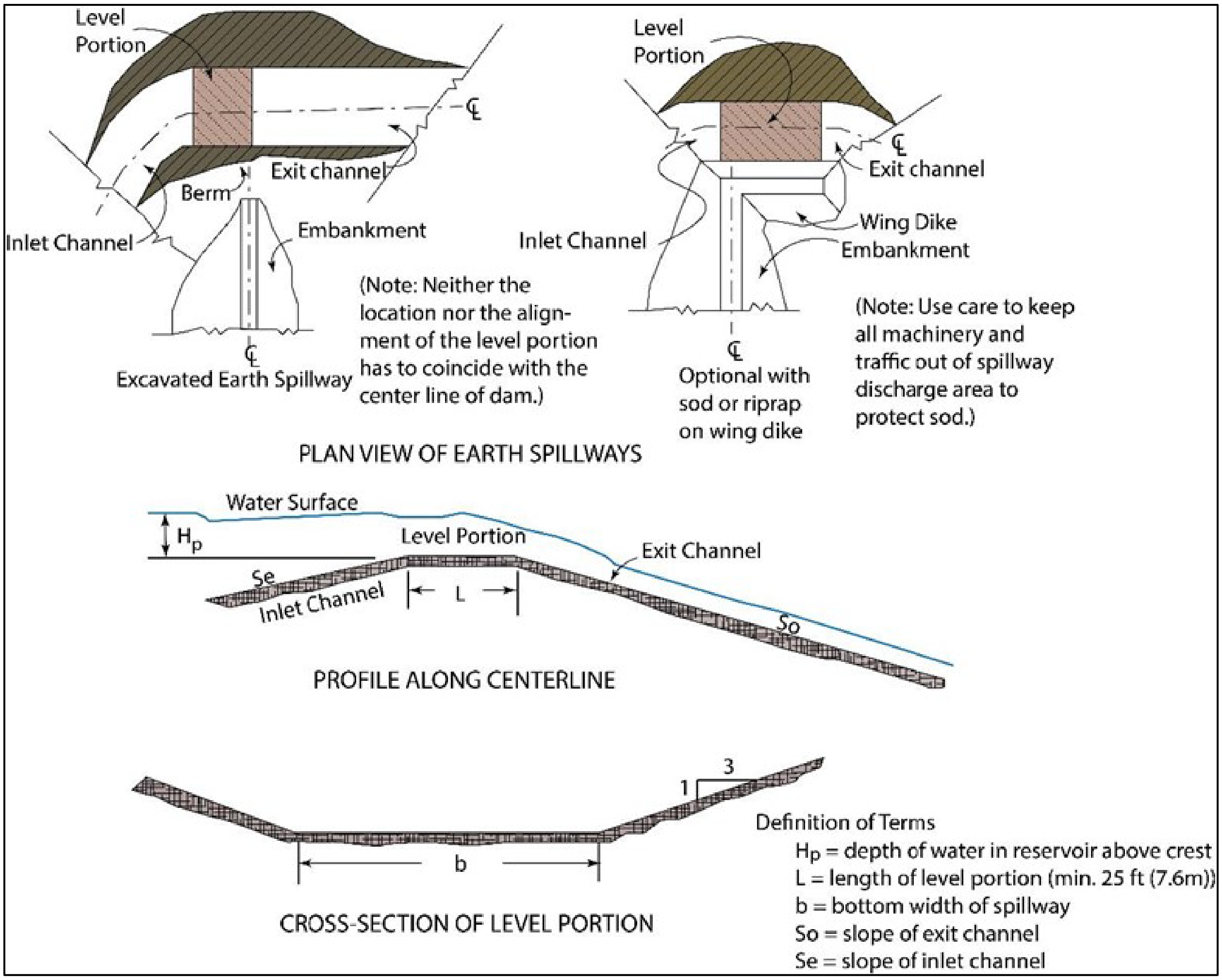

Figure 10.24. Profile and cross-section of excavated earth spillway. Source: USDA-NRCS (2021).

Estimating the characteristics of an outlet structure uses the following inputs:

- Watershed characteristics, including area, pre- and post-development times of concentration, and pre- and post-development curve numbers (assuming NRCS CN procedures are used for abstractions).
- Rainfall depth(s) for the design storm(s).
- Characteristics of the riser and outlet pipe structure, including pipe roughness (n), length, and an initial estimate of the diameter.
- Elevation information, including stage vs. storage values, the wet pond elevation, if applicable, and the elevation of the centerline of the pipe.
- Hydrologic and hydraulic models, including a model for estimating peak flows and runoff depths, a model for estimating the volume of storage as a function of pre- and post-development peak flows, and a model for estimating weir and orifice coefficients, as appropriate.

10.3.3.5.1 Single-Stage Risers

Single-stage risers, as shown in Figure 10.25, provide runoff control for cases where practice specifies one exceedance frequency. The following procedure estimates both the riser characteristics and the volume of storage (adapted from Woodward 1983).

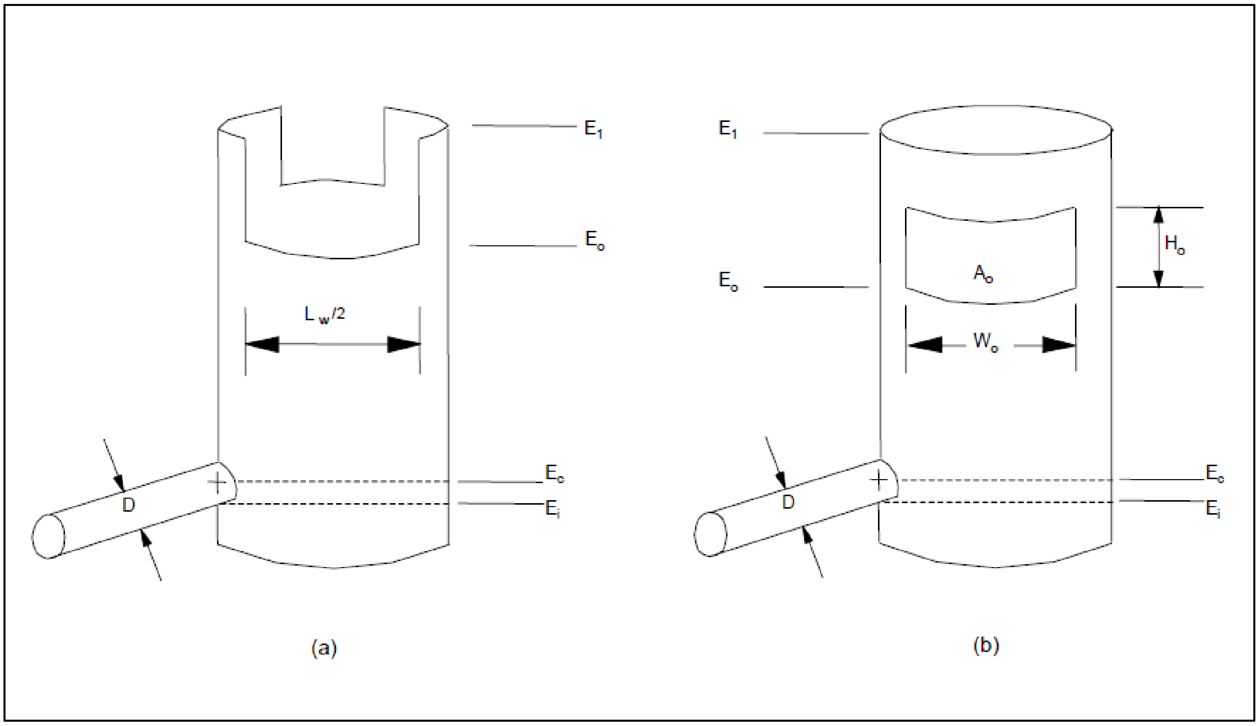

Figure 10.25. Single-stage riser characteristics for (a) weir flow and (b) orifice flow.

Step 1. Design the culvert barrel.

Design the culvert barrel using the procedures of HDS-5 (FHWA 2012a) and set the culvert invert elevation. To simplify the riser design, the designer selects a culvert so that the maximum culvert headwater, which is based on the maximum outlet design discharge, is no higher than the invert of the orifice opening (or weir). If this is not the case, this condition "submerges" the opening forcing the designer into an iterative procedure to establish the stage-discharge relationship.

Step 2. Estimate the preliminary storage volume.

Estimate the preliminary storage volume, V_s, using one of the techniques described in Section 10.3.1.

Step 3. Estimate the dead storage.

Using the elevation E_o of either the weir or the bottom of the orifice, obtain the volume of dead storage V_d from the elevation-storage curve.

Step 4. Estimate the total storage needed.

Compute the total storage:

$$V_t = V_d + V_s \tag{10.38}$$

Step 5. Obtain highest water surface elevation.

Enter the elevation-storage curve with Vt to obtain the water surface elevation, E_l.

Step 6. Size the outlet opening.

For an orifice, determine its characteristics:

- Assume an orifice height, H_o.
- Compute the area of the orifice opening assuming unsubmerged flow, A:

$$A_o = \left(\frac{1}{C_d\sqrt{2g}}\right)\frac{Q_{pb}}{\left(E_1 - E_o - \frac{H_o}{2}\right)^{0.5}} \quad (10.39)$$

where:

A_o	=	Orifice opening area, ft² (m²)
Q_{pb}	=	Discharge through the orifice (pre-development), ft³/s (m³/s)
E_1	=	Water surface elevation upstream of the orifice, ft (m)
E_o	=	Elevation of the bottom of the orifice, ft (m)
g	=	Gravitational acceleration, 32.2 ft/s² (9.81 m/s²)

$H_o/2$ adjusts for head being measured from the center of the orifice.

For a rectangular orifice, compute the width of the orifice opening W_o:

$$W_o = \frac{A_o}{H_o} \quad (10.40)$$

For a weir, determine the weir length for unsubmerged flow:

$$L_w = \left(\frac{1}{\sqrt{2g}C_w}\right)\frac{Q_{pb}}{(E_1 - E_0)^{1.5}} \quad (10.41)$$

Step 7. Verify performance with storage routing.

Verify the design performance using storage routing. (The approximate procedures for estimating storage volume do not account for performance of the outlet structure throughout the passage of the inflow hydrograph and may result in an under- or over-design.)

Example 10.12: Single-stage riser design.

Objective: Reduce the post-development 0.5 AEP peak flow to the pre-development peak flow.

Given: A single-stage riser is being designed for a pond. It must reduce post-development 0.5 AEP peak to the pre-development peak flow. The outlet structure will consist of an outlet pipe culvert and a riser with a rectangular orifice. The pond will include permanent pool elevation for water quality purposes.

A	=	11.2 ac (4.53 ha)
$Q_{2,post}$	=	17.7 ft³/s (0.5 m³/s)
$Q_{2,pre}$	=	2.8 ft³/s (0.08 m³/s)
Invert	=	96.8 ft (29.5 m) (pond bottom)

EL_{pool}	=	98.8 ft (30.1 m) (dead storage)

Step 1. Select culvert size, slope, and material.

First, select and locate a culvert barrel so that the headwater at the culvert entrance under the design discharge does not reach the invert of the orifice forcing it to operate under submerged conditions. Use the procedures of HDS-5 (FHWA 2012a) and consider the site constraints on culvert slope and invert location.

D	=	36 inches (910 mm)
L	=	34.4 ft (10.5 m)
n	=	0.012 (reinforced concrete pipe)

If the selected size is not available, specify the next available larger size. Place the entrance invert at an elevation of 96.1 ft (29.3 m). The designer selects the culvert inlet so that the culvert headwater, i.e., the depth of water inside the riser, does not rise above the lowest riser opening for the design conditions, where possible. The designer also adjusts the culvert invert to be compatible with site conditions.

Step 2. Develop Stage-storage curve.

Section 10.3.2 gives details on constructing a stage-storage relationship. For this example, Table 10.9 gives the stage-storage relationship at the site of the detention structure. Based on the permanent pool elevation, interpolate the dead storage from the stage-storage curve to be 13,400 ft^3.

Step 3. Estimate total storage.

Compute the active storage using one of the methods presented in Section 10.3.1, estimating it at 23,700 ft^3. The total storage equals the sum of the active and dead storages, 37,100 ft^3.

Step 4. Find the maximum stage.

Find the elevation corresponding to the total storage by interpolating the stage-storage curve, which is 101.4 ft.

Step 5. Compute orifice flow.

The orifice invert was established at an elevation of 98.8 ft to create the permanent pool. Assuming an initial orifice height of 0.5 ft, compute the area of the orifice in the riser with equation 10.39 to create an outflow of 2.8 ft^3/s at the assumed stage in the pond:

$$A_o = \{1 / [0.6\ (2*32.2)^{0.5}]\}\ \{2.8 / [101.4 - 98.8 - (0.5 / 2)]^{0.5}\} = 0.379\ ft^2$$

$$W_o = A_o / D = 0.379 / 0.5 = 0.76\ ft$$

Solution:	One possible orifice is 0.5 ft by 0.76 ft. If the calculated dimensions are not practical construction sizes, conduct an iterative trial process with practical dimensions resulting in the same performance until finding suitable dimensions. It is not usually appropriate to select the next larger available dimensions because this will allow excessive discharge through the orifice. Using storage routing, check the design before finalizing. The diameter of the riser (if it is also circular) usually equals 2 to 3 times the diameter of the outlet culvert.

Table 10.9. Stage-storage curve.

Elevation (ft)	Volume (ft^3)
96.8	0
97.0	1,390
97.5	4,410
98.0	7,620
98.5	11,240
99.0	15,540
99.5	19,850
100.0	24,440
100.5	29,280
101.0	34,120
101.5	39,940
102.0	46,170
102.5	52,860
103.0	60,120
103.5	68,420
104.0	77,990
104.5	88,030

10.3.3.5.2 Multiple-Stage Risers

Designers use multi-stage risers where two or more exceedance frequencies apply. Figure 10.26 depicts a two-stage riser, which is similar to the single-stage riser except that it includes either two weirs or a weir and an orifice. For the weir/orifice combination, the orifice controls the more frequent event, and the weir controls the larger event. Designers refer to the runoff from the smaller and larger events as the low-stage and high-stage events, respectively. Recognizing that the two events will not occur simultaneously, designers control the high-stage event using both the low-stage weir or orifice and the high-stage weir.

The procedure for a multi-stage riser follows the same general steps as for the single-stage riser, except the designer determines both the high-stage weir and low-stage outlet characteristics. The procedure uses the same inputs for sizing a two-stage riser as for a single-stage riser but relies on computing many of the values for both the low-stage and the one or more high-stage events.

Designers can size a two-stage riser for the cases where the low-stage outlet is either a weir or an orifice and the high-stage outlet is a weir using the following steps:

Step 1. Design culvert barrel.

Design the culvert barrel using the same process as for the single-stage riser. For the multi-stage riser, the maximum design discharge corresponds to the highest-stage event.

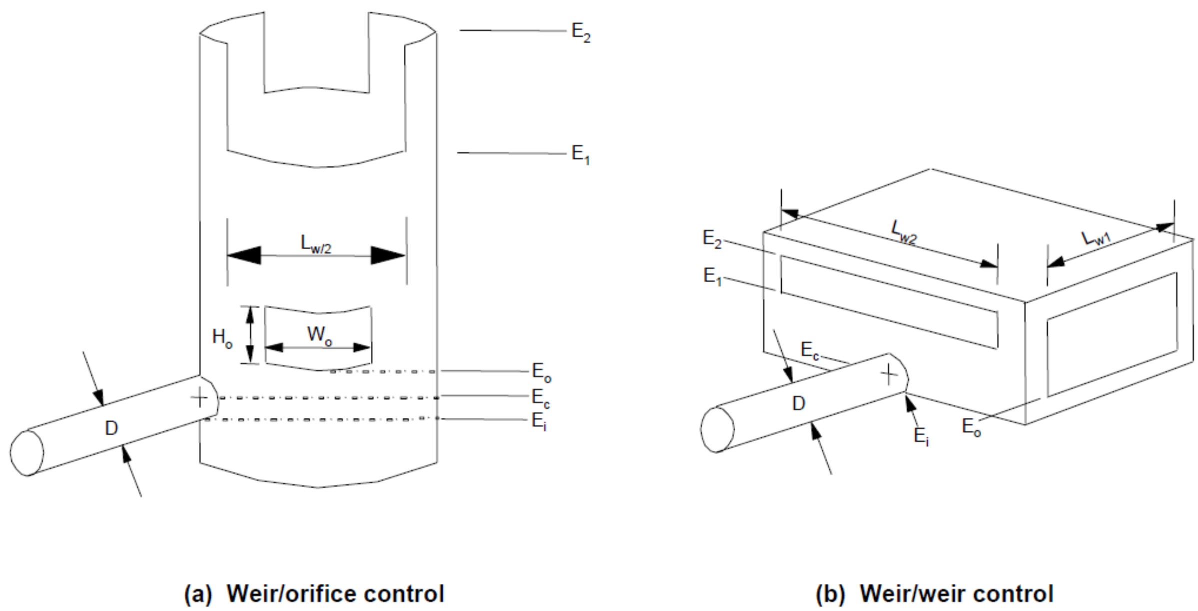

Figure 10.26. Two-stage riser.

Step 2. Design the low-stage opening.

Design the low-stage opening as described for a single-stage riser. If the low-stage opening is a weir of the configuration shown in Figure 10.26(b), it operates independently up to an elevation equal to the invert of the high-stage weir.

Step 3. Estimate the high-stage storage volume.

Estimate the appropriate high-stage storage volume, V_{s2} (Section 10.3.1).

Step 4. Compute the high-stage storage.

Compute the total high-stage storage:

$$V_{t2} = V_d + V_{s2} \tag{10.42}$$

Step 5. Obtain the water surface elevation.

Enter the elevation-storage curve with V_{t2} to obtain the water surface elevation, E_2.

Step 6. Estimate discharge through the low-stage opening.

Estimate the discharge through the low-stage opening during the high-stage event.

If the low-stage outlet is an orifice, the discharge is:

$$q_{o2} = C_{d1}\sqrt{2g}\,A_{o1}\left(E_2 - E_0 - \frac{h_o}{2}\right)^{0.5} \tag{10.43}$$

If the low-stage outlet is a weir, the discharge is:

$$q_{o2} = C_{w1}\sqrt{2g}\,L_{w1}(E_2 - E_0)^{1.5}$$

Step 7. Set high-stage invert.

Set the invert of the high-stage weir equal to E_1 (maximum elevation during the low-stage event).

Step 8. Compute high-stage weir length.

Compute the high-stage weir length:

$$L_{w2} = \left(\frac{1}{\sqrt{2g}\,C_w}\right)\frac{q_{pb2} - q_{o2}}{\left(E_2 - E_1\right)^{1.5}} \quad (10.44)$$

Step 9. Analyze additional high-stage openings.

If designing additional high-stage openings, repeat steps 3 through 8 accounting for all other openings that are active during the event.

Step 10. Verify with routing.

Verify the design performance using storage routing to confirm that the outlet structure and storage volume perform as intended.

Table 10.10 summarizes a composite stage-discharge relationship for an outlet control device consisting of a low flow orifice and a riser pipe connected to an outflow pipe. The structure also includes an emergency spillway. Table 10.6, Table 10.8, and emergency spillway computations summarize the individually calculated components. Using the methods in Section 10.3.3.4, the spillway passes 0, 39.55, and 55.8 ft^3/s at elevations of 38.1, 38.7, and 39.4 ft, respectively. The total discharge equals the sum of individual components at each stage.

Figure 10.27 illustrates the same information. Initially, the low flow orifice controls the discharge. At an elevation of 35.4 ft the water surface in the storage facility reaches the top of the riser pipe and begins to flow into the riser. The flow at this point equals a combination of the flows through the orifice and the riser. Orifice flow through the riser controls the riser discharge above a stage of 36.1 ft. At an elevation of 38.0 ft, flow begins to pass over the emergency spillway. Beyond this point, the total discharge from the facility equals the sum of the flows through the low flow orifice, the riser pipe, and the emergency spillway. Additionally, the designer ensures that the outlet pipe from the detention basin is large enough to carry the total flows from the low orifice and the riser section so that the outlet pipe does not limit the flow from the basin.

Table 10.10. Composite stage-discharge tabulation.

Stage (ft)	Low Flow Orifice (ft^3/s)	Riser Orifice Flow (ft^3/s)	Emergency Spillway (ft^3/s)	Total Discharge (ft^3/s)
32.8	0	0	0	0.00
33.5	0.61	0.00	0.00	0.61
34.1	0.93	0.00	0.00	0.93
34.8	1.20	0.00	0.00	1.20
35.4	1.39	0.00	0.00	1.39
36.1	1.59	9.18	0.00	10.77
36.7	1.74	13.07	0.00	14.81
37.4	1.89	15.89	0.00	17.78
38.1	2.04	18.72	0.00	20.76
38.7	2.16	20.84	39.55	62.55
39.4	2.29	22.60	55.80	80.69

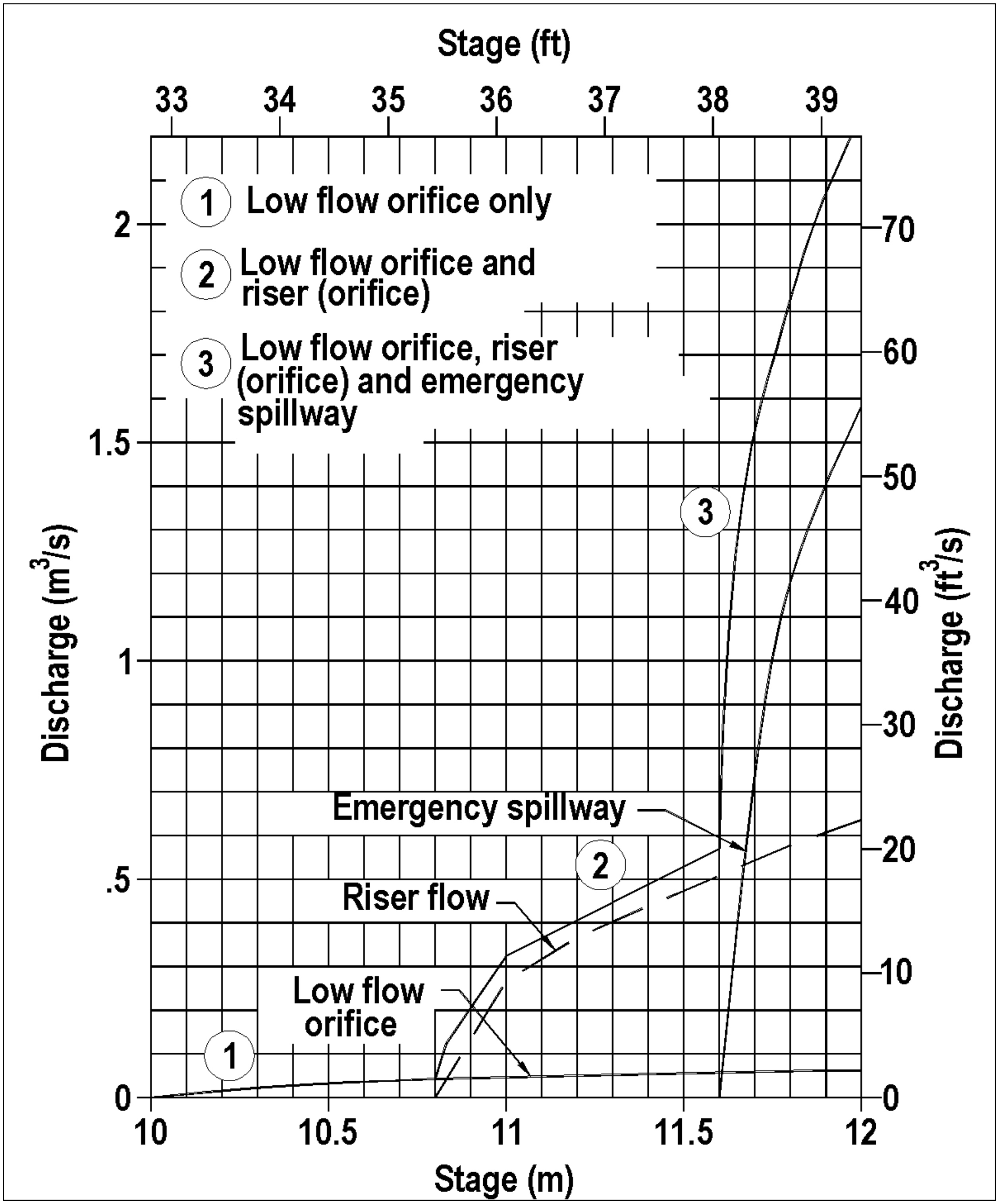

Figure 10.27. Typical composite stage-discharge relationship.

10.4 Water Budgets for Wet Ponds

Designers prepare water budgets for permanent pool facilities to confirm that the pool will be present under the conditions intended by the designer. At a minimum, these typically include average rainfall conditions but may also include alternative wet or dry conditions. A water budget considers all significant inflows and outflows, including, but not limited to, rainfall, runoff, infiltration, exfiltration, evaporation, and outflow.

10.4.1 Water Budget Components

Designers can compute average annual runoff using a weighted runoff coefficient for the tributary drainage area multiplied by the average annual rainfall volume. Base infiltration and exfiltration on site-specific soils testing data. Approximate evaporation using the mean monthly pan evaporation or free water surface evaporation data for the area of interest.

Analyzing infiltration involves knowledge of soil permeabilities and hydrogeologic conditions in the vicinity of the basin. Although county soils reports often publish infiltration rates, designers may secure field measurements when site-specific estimates for these parameters are important for the water budget. This is particularly important in karst areas where the hydrogeologic phenomenon controlling infiltration rates may be complex.

HDS-2 (FHWA 2002) provides a detailed discussion on water budgets, specifically as they relate to wetland applications for roadway projects.

Example 10.13: Water budget of a retention pond.

Objective: For average annual conditions, determine if the facility will function as a wet pond with a permanent pool.

Given: A shallow basin with characteristics including average surface area (A_s), bottom area (A_B), watershed area (A), post-development runoff coefficient (C_D), average infiltration rate for soils (F_a), average annual rainfall from rainfall records (P_a), and mean annual evaporation (E_a).

A_s = 3 ac (1.21 ha)
A_B = 2 ac (0.81 ha)
A = 100 ac (40.5 ha)
C_D = 0.3
F_a = 0.1 in/h (2.5 mm/h)
P_a = 50 inches (127 cm)
E_a = 35 inches (89 cm)

Step 1. Compute average annual runoff.

Runoff = C Q_D A (modification of equation 4.1)

Runoff = (0.3) (4.17 ft) (100 ac) (43,560 ft^2/ac) = 5,445,000 ft^3

Step 2. Estimate the average annual evaporation.

Evaporation = E_a A_s = (2.92 ft) (3 ac) (43,560 ft^2/ac) = 381,150 ft^3

Step 3. Estimate the average annual infiltration.

Infiltration = (F_a) (time) (A_B) = (0.01 in/h) (24 h/day) (365 days/yr) (2.0 ac) (43,560 ft^2/ac)

= 6,359,760 ft^3

Step 4. Estimate the runoff (or inflow) less evaporation and infiltration losses.

Assuming no basin outflow and no change in storage, the runoff (or inflow) less evaporation and infiltration losses is:

Net Budget = 5,445,000 – 381,150 – 6,359,760 = -1,295,910 ft^3

Since the average annual losses exceed the average annual rainfall, the proposed facility will not function as a wet pond with a permanent pool. If the facility needs to function with a permanent pool, accomplish that by reducing the pool size.

Step 5. Revise the pool surface area.

Pool surface and bottom areas equal 2.0 ac and 1.0 ac, respectively.

Step 6. Recompute the evaporation and infiltration.

Evaporation = (2.92 inches) (2.0 ac) (43560 ft^2/ac) = 254,100 ft^3

Infiltration = (0.01) (24) (365) (1.0) (43560/12) = 3,179,880 ft^3

Step 7. Estimate revised runoff less evaporation and infiltration losses.

Net Budget = 5,495,000 – 254,100 – 3,179,880 = 2,011,020 ft^3

Solution: The revised facility appears able to function as a wet pond with a permanent pool. However, these calculations use average precipitation, evaporation, and losses. During years of low rainfall, the pool may not persist.

10.4.2 Water Budget for Landlocked Storage

Designers can evaluate watershed areas which drain to central depressions with no positive (gravity-driven) outlet using a mass flow routing procedure to estimate flood elevations. Typical examples include retention basins in karst topography or other areas having high infiltration rates. Although this procedure is fairly straightforward, the evaluation of basin outflow is a complex hydrologic phenomenon that depends on good field measurements and a thorough understanding of local conditions. Since outflow rates for flooded conditions are difficult to calculate, field measurements are desirable.

Figure 10.28 illustrates a mass routing procedure for the analysis of landlocked retention areas. The step-by-step procedure is:

Step 1. Obtain cumulative rainfall data for the design storm.

If no local criteria are available, AASHTO (2014) suggests a 0.01 AEP, 10-day storm.

Step 2. Calculate the cumulative inflow.

Using the rainfall data from step 1 and an appropriate runoff hydrograph method (see Chapter 4), estimate the cumulative inflow.

Step 3. Develop the basin outflow.

From field measurements of hydraulic conductivity or infiltration, considering worst-case water table conditions, estimate the basin outflow. Establish hydraulic conductivity/infiltration using in situ test methods. Then, plot the mass outflow with a slope corresponding to the worst-case outflow.

Step 4. Draw tangent line.

Draw a line tangent to the mass inflow curve from step 2 having a slope parallel to the mass outflow line from step 3.

Step 5. Locate the point of tangency.

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 multiplied by the drainage area represents the maximum storage required for the design runoff.

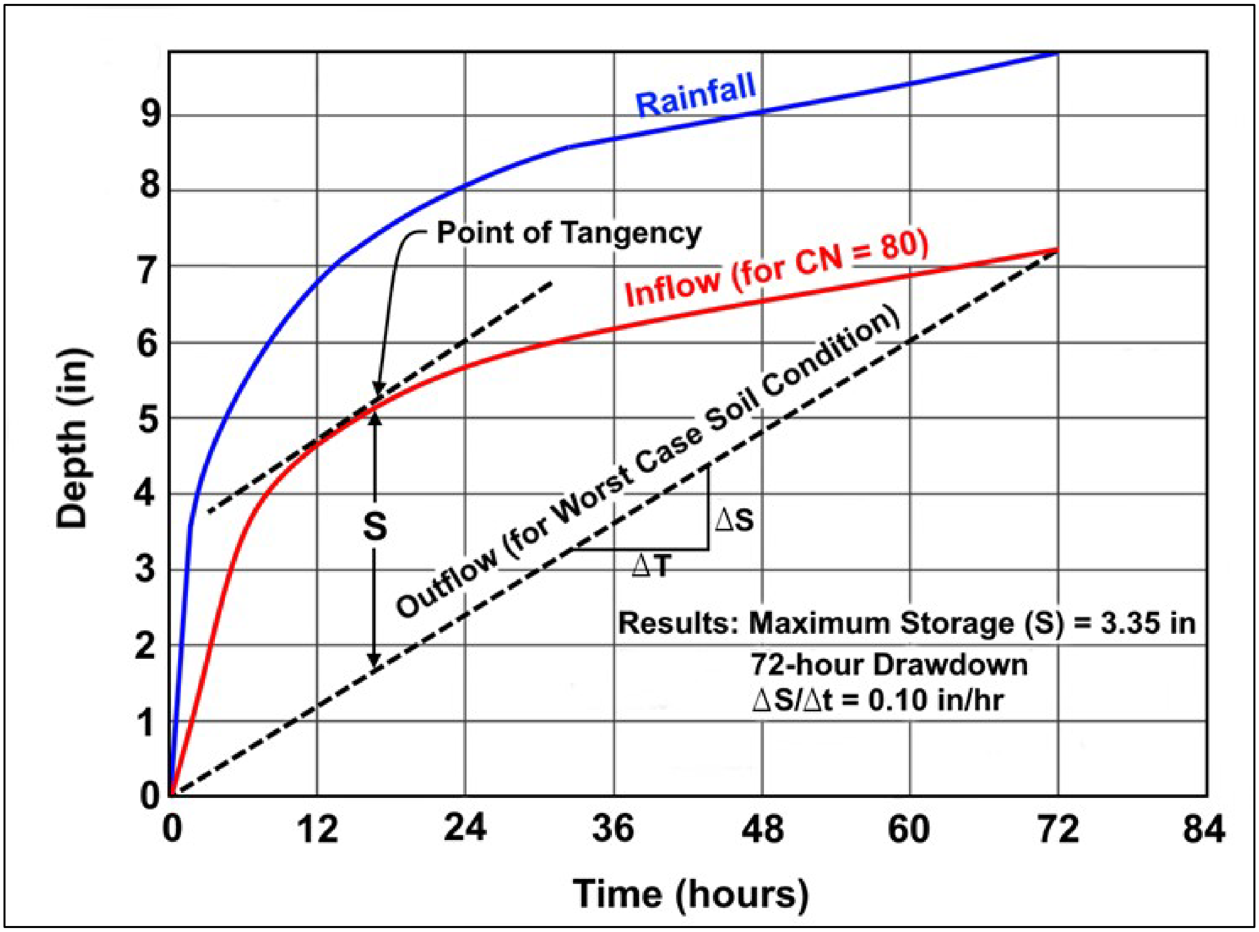

Figure 10.28. Mass routing procedure.

Step 6. Determine the flood elevation.

Estimate 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. Establish the zero-volume elevation as the normal wet season water surface or water table elevation or the pit bottom, whichever is highest.

If runoff from a project area discharges into a drainage system tributary to the landlocked depression, the pre-development discharge requirements for the project may include detention storage facilities.

10.5 Storage Routing

Most commonly, designers route the inflow hydrograph through a detention pond using the storage-Indication or modified Puls method. This method derives from the continuity equation which states that the inflow minus the outflow equals the change in storage. By taking the average of two closely spaced inflows and two closely spaced outflows, the discretized continuity equation is expressed by:

$$\frac{\Delta S}{\Delta t} = \frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} \tag{10.45}$$

where:

ΔS	=	Change in storage, ft^3 (m^3)
Δt	=	Time interval, min
I	=	Inflow, ft^3 (m^3)
O	=	Outflow, ft^3 (m^3)

Subscripts 1 and 2 refer to the beginning and end of the time interval, respectively. Figure 10.29 illustrates the routing process graphically with the inflow hydrograph as the input to the routing and the outflow hydrograph as the result of the routing. HDS-2 (FHWA 2002) provides a more detailed description of storage routing.

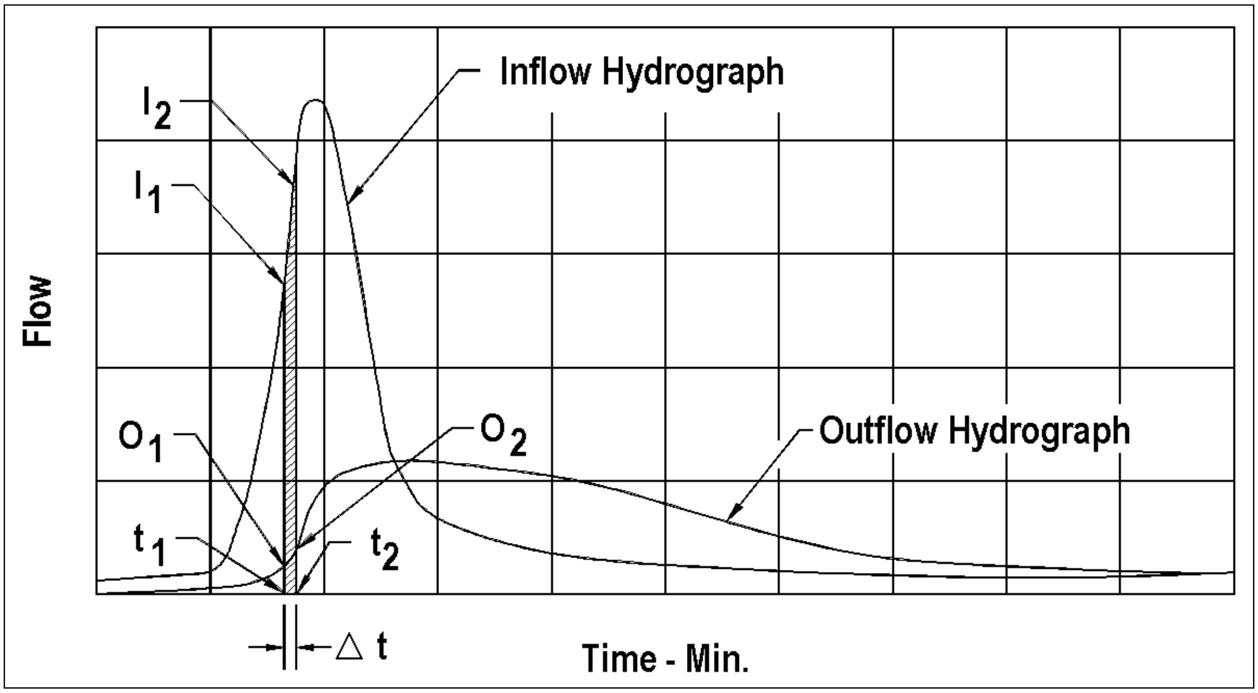

Figure 10.29. Inflow and routed outflow hydrographs.

10.6 Detention Design Procedure

Detention basin design has the hydrologic goals of identifying both the outlet structure characteristics (discharge pipe, weirs, and orifices) and the storage geometry that will limit the computed post-development peak flow out of the detention basin so that it does not exceed the target discharge (commonly the peak flow under pre-development conditions). Both the target discharge and the outlet structure characteristics influence the stage-storage-discharge relationship. Because of this interdependence, detention design uses an iterative procedure. The design procedure begins with some assumptions about the outlet structure characteristics. Then the designer applies the storage routing procedure to determine whether the design meets the target discharge requirement. If the routed discharge exceeds the target discharge, then the designer adjusts the design to reduce the discharge until the target is met. Conversely, if the routed discharge is significantly below the target discharge, the designer may also adjust the design to reduce the needed storage and potentially reduce the cost of the facility.

Four inputs support the design:

- Initial conditions. Establish initial conditions, including setting the time interval Δt, the storm time at which computations end, and the initial outflow O_1 and storage S_1.
- Inflow hydrograph. Compute the design storm inflow hydrograph and the target maximum discharge, q_o, for the design storm. The design storm inflow hydrograph is usually the output from the convolution of a rainfall-excess hyetograph and a unit hydrograph, with the post-development conditions used to compute the rainfall excess. The target discharge is usually the peak flow of the pre-development hydrograph. See Chapter 4.
- Stage-storage relationship. Obtain topographic information and compute the stage-storage relationship. See Section 10.3.2.
- Stage-discharge relationship. Set the riser characteristics (i.e., number of stages, type of outlet, and values of the discharge coefficients). See Section 10.3.3.

The designer routes the inflow hydrograph through the current basin design using the routing procedure introduced in Section 10.5 based on the initial conditions, stage-storage relationship, and stage-discharge relationship. After routing, the designer compares its peak flow of the outflow hydrograph to the target discharge q_o. If it is greater than q_o, then the capacity of the assumed outlet configuration is too large. Thus, the designer will decrease the weir lengths or orifice areas. If the peak flow of the outflow is less than the target discharge q_o, the assumed outlet configuration meets the target, but may be overdesigned. The designer can increase the weir lengths or orifice areas. Any adjustment to the riser structure involves recomputing the stage-discharge relationship followed by rerouting the inflow hydrograph. When the peak outflow approximately equals the target discharge, the assumed outlet facility is a reasonable design.

The designer estimates required storage by the largest value of storage associated with the outflow hydrograph. The designer uses the maximum storage as input to the stage-storage curve to estimate the depth of storage. The designer evaluates the computed design for safety and cost.

Example 10.14: Detention basin design process.

Objective: Design a detention basin storage and outlet structure.

Given: A watershed described as:

A = 38 acres (15.8 hectares)
AEP = 0.1

Step 1. Determine the hydrologic goals for the detention basin.

Using an appropriate hydrologic method, the pre-development and post-development peaks are:

$q_{pb} = 50\ ft^3/s$

$q_{pb} = 131\ ft^3/s$

Development within the watershed increased the 0.1 AEP discharge by 162 percent. The detention basin design goal is to reduce the peak flow from 131 ft^3/s to 50 ft^3/s.

Step 2. Establish inflow hydrograph.

Develop an input hydrograph for design. Figure 10.30 displays the inflow hydrograph with a peak of 131 ft^3/s. Use the ordinates on a 0.1-hour increment for a 2-hour period of the 24-hour storm; discharges for the remainder of the 24-hour storm duration are either zero or very small.

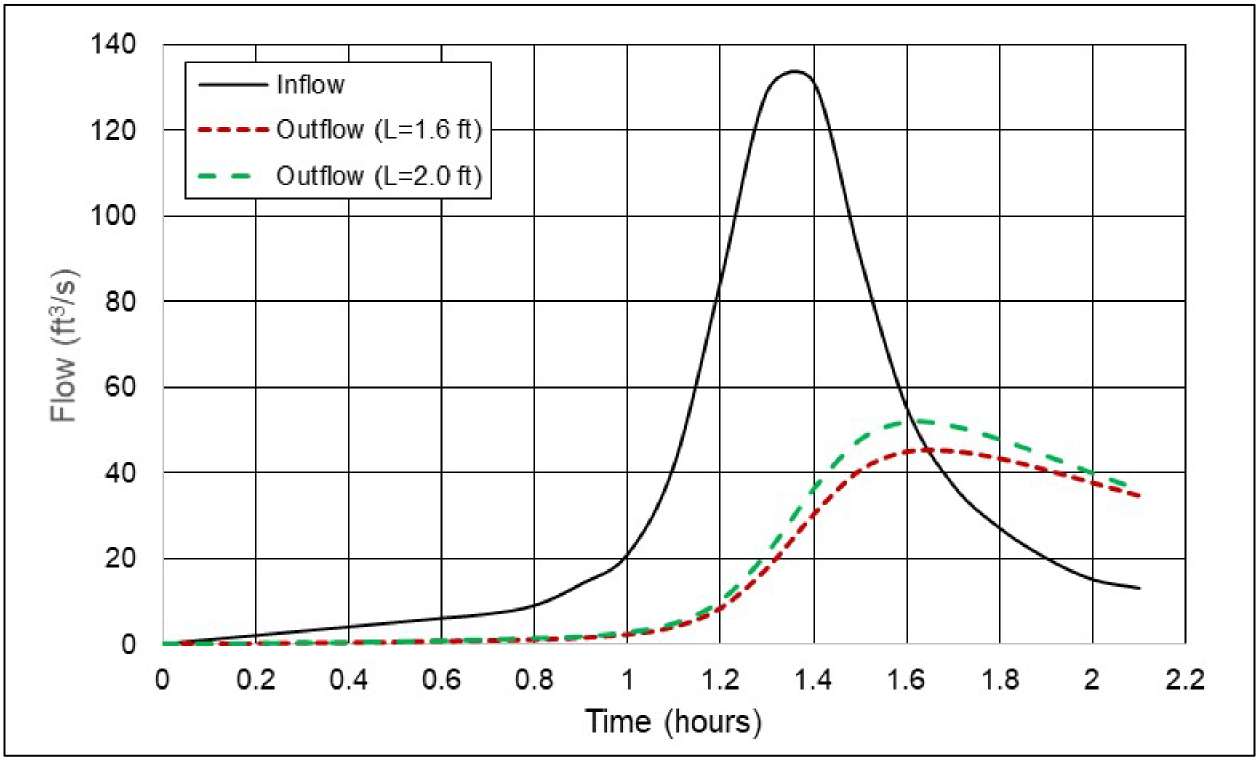

Figure 10.30. Inflow and routed outflow hydrographs for detention basin example.

Step 3. Develop stage-storage curve.

Compute the stage-storage relationship from topographic information. Table 10.11 shows the estimated surface areas at the site from the topography at an increment of 0.5 ft. Multiply the average area (column 3) and the incremental depth of 0.5 ft to yield the incremental storage (column 4). Accumulate the incremental storage values to compute a cumulative storage at each elevation.

Step 4. Develop stage-discharge curve.

The designer selects a one-stage riser with a weir. Make an initial estimate of the weir length using the weir equation, with an assumed depth of 4.9 ft and the target outflow discharge of 50 ft^3/s from step 1:

$$L_w = \left(\frac{1}{\sqrt{2g}\,C_w}\right)\frac{q_o}{h^{1.5}} = \left(\frac{1}{3.09(0.94)}\right)\frac{50}{(4.9)^{1.5}} = 1.6 \text{ ft}$$

Use an initial weir length of 1.6 ft. Figure 10.31 summaries the resulting stage-discharge curve.

Step 5. Perform routing to estimate peak outflow and maximum storage.

Route the inflow hydrograph with the storage-indication method introduced in Section 10.5. Figure 10.30 gives the results when the weir length is 1.6 ft. The peak outflow of 45 ft^3/s occurs at approximately 1.6 hours.

From Figure 10.31, a discharge of 45 ft^3/s occurs when the stage in the pond is approximately 4.5 ft. From Table 10.11, the storage at this stage is approximately 160,000 ft^3.

Table 10.11. Derivation of stage-storage relationship.

(1) Depth, h (ft)	(2) Surface Area, A (ft^2)	(3) Average Area (ft^2)	(4) Incremental Storage (ft^3)	(5) Storage Volume, V_s (ft^3)
0.00	33,500	-	-	0
-	-	33,650	16,825	-
0.50	33,800	-	-	16,825
-	-	34,050	17,025	-
1.00	34,300	-	-	33,850
-	-	34,450	17,225	-
1.50	34,600	-	-	51,075
-	-	34,950	17,475	-
2.00	35,300	-	-	68,550
-	-	35,500	17,750	-
2.50	35,700	-	-	86,300
-	-	35,950	17,975	-
3.00	36,200	-	-	104,275
-	-	36,500	18,250	-
3.50	36,800	-	-	122,525
-	-	37,150	18,575	-
4.00	37,500	-	-	141,100
-	-	37,750	18,875	-
4.50	38,000	-	-	159,975
-	-	38,400	19,200	-
5.00	38,800	-	-	179,175

Step 6. Confirm compliance with design criteria.

Since the computed peak with L = 1.6 ft is less than the allowable peak (50 ft^3/s), reduce the required storage volume by allowing a higher peak flow to leave the basin.

To accomplish this, increase the weir length to 2.0 ft and repeat the computations. Figure 10.31 summarizes the stage-discharge curve for the new weir length. Figure 10.30 shows the routed the inflow hydrograph with a resulting peak flow of 52 ft^3/s. This exceeds the allowable peak of 50 ft^3/s.

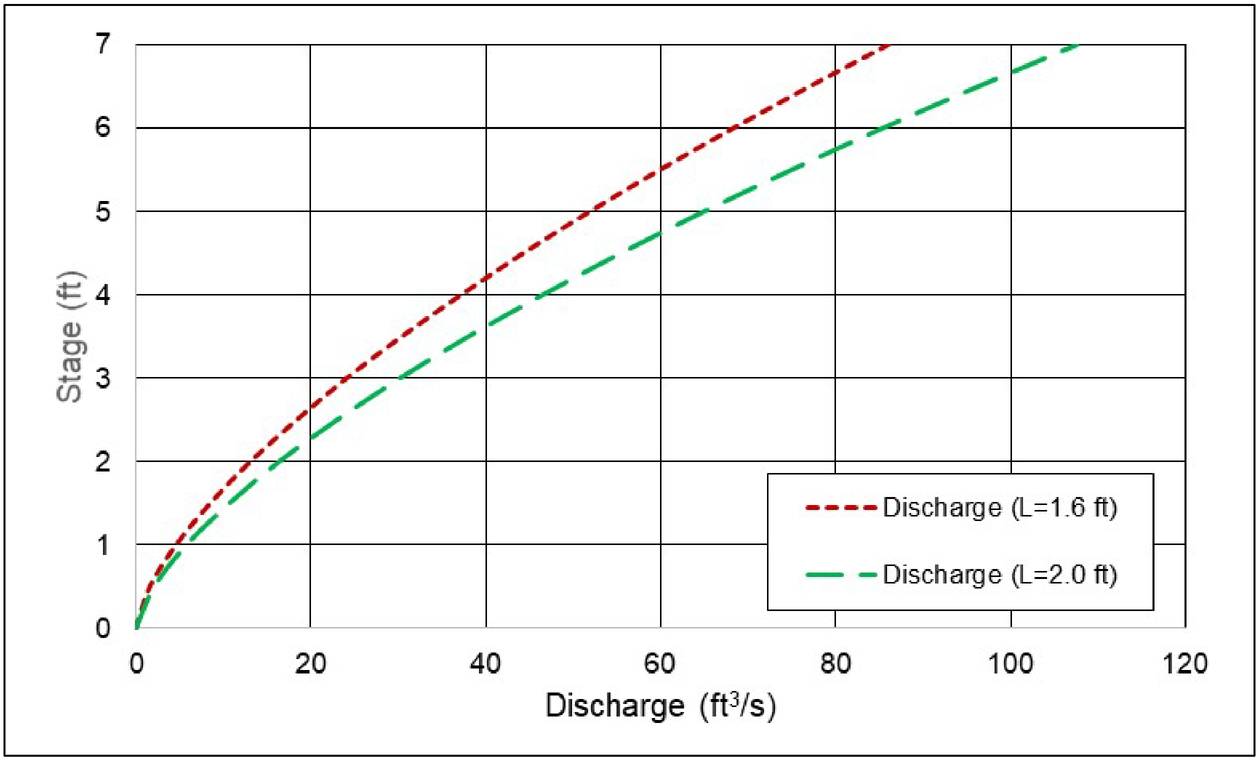

Figure 10.31. Stage-discharge curves for detention basin example with weir lengths of 1.6 and 2.0 ft.

Solution:	It is appropriate to evaluate whether an additional iteration on weir length is worthwhile, especially in the context of available constructed weir lengths. The 1.6 ft (0.5 m) length meets the design requirement; the 2.0 ft (0.61 m) length does not. If an intermediate length is constructible, further analysis may be warranted. If not, select the 1.6 ft (0.5 m) length.