## A. Introduction

The methods described in Chapter 3 - Section 9 Detention Storage Design are used to estimate the volume of the detention storage. The second step in the design of the detention basin is determining the physical characteristics of the outlet structure. The outflow from a detention basin depends on the type and the size of the outlet structure. A relationship between the stage and the discharge can be determined from the hydraulics of these structures. Primary outlets provide the critical function of the regulation of flow for structural stormwater controls. The outlet may be configured as a single outlet, or can be configured with multiple outlet devices to provide multi-stage outlet control. For a single-stage system, the outlet can be designed as a simple pipe or culvert. For multi-stage control structures, the inlet is designed considering a range of design flows. All detention basins are designed with a secondary outlet, sometimes called an emergency or auxiliary spillway. The secondary spillway is provided to convey the release of the maximum runoff discharge for the 100-year storm event. The secondary spillway is most often a weir-type control, separate from the primary outlet structure and configured as part of the detention basin embankment.

A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately, or are combined to discharge at a single location. This section provides an overview of outlet structure hydraulics and design for stormwater detention facilities. The designer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

## B. Outlet structure types

The most common types of outlets can be categorized into three groups: orifice-type, weir-type, and riser-pipe structures. There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design:

- Orifices
- Perforated risers
- Pipes/culverts
- Sharp-crested weirs
- Broad-crested weirs
- V-notch weirs
- Proportional weirs
- Combination outlets

Each of these outlet types has a different design purpose and application:

- Water quality and channel protection flows are normally handled with smaller, more protected outlet structures, such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.
- Larger flows, such as overbank protection and extreme flood flows, are typically handled through a riser with different-sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

1. **Orifices.** An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice. For a single orifice, as illustrated in Figure C3-S12-1, the orifice discharge can be determined using the standard orifice equation below.

**Equation C3-S12-1**

$$Q - CA_0(2gh)^{0.5}$$

Where:
Q = the orifice flow discharge (cfs)
$C_d$ = dimensionless coefficient of discharge
$A_0$ = cross-sectional area of orifice or pipe ($ft^2$)
g = acceleration due to gravity (32.2 $ft/sec^2$)
$D_o$ = diameter of orifice or pipe (ft)
h = effective head on the orifice, from the center of orifice to the water surface

**Equation C3-S12-1a**
For circular orifices:

$$A_0 = \frac{\pi {D_o}^2}{4}$$

**Equation C3-S12-1b**
For rectangular orifices:

$$A_0 = bD$$

Where:
b and D represent the side lengths of the rectangular opening

Typical values for $C_d$ are 0.6 for square edge uniform entrance conditions, and 0.4 for ragged edge orifices (FHWA, 1996).

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure C3-S12-1.

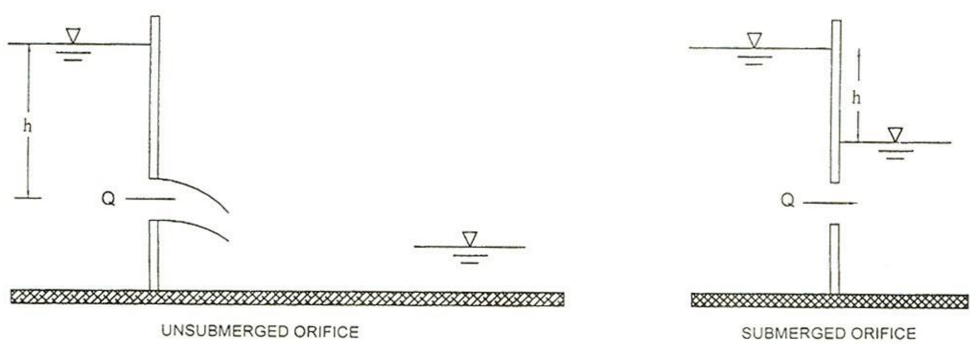


**Figure C3-S12-1: Orifice outlets**

2. **Weir-type outlets.** Rectangular broad-crested weirs, overflow spillways and sharp-crested weirs are included in this group. The discharge over these structures (Figure C3-S12-2) is determined using the general form of the equation (Brater and King, 1976).

**Equation C3-S12-2**

$$Q = C_w L (2gh)^{3/2}$$

Where:
$C_w$ = dimensionless weir discharge coefficient
L = effective weir length, ft
H = water depth above the crest

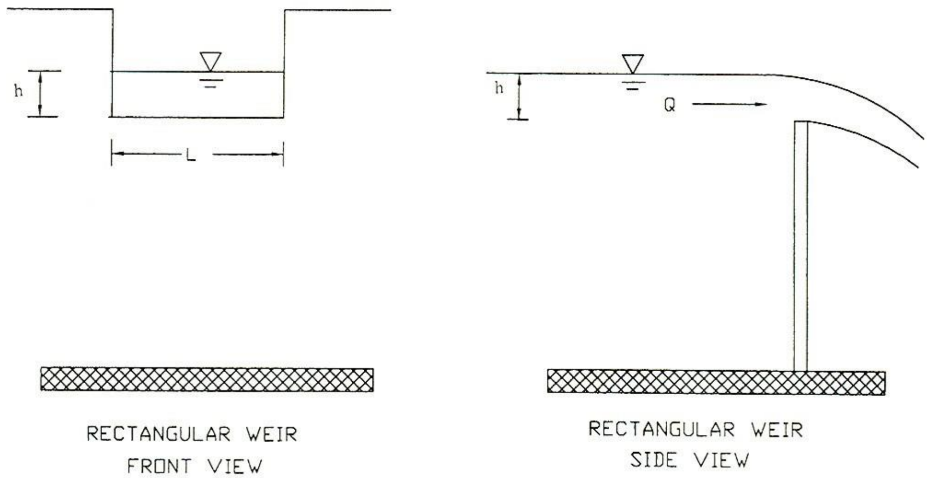



**Figure C3-S12-2: Weir outlet**

3. **Broad-crested weirs.** A weir in the form of a relatively long raised channel control crest section is a broad-crested weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow will occur when upstream head above the crest is between the limits of about $^1/_{20}$ and ½ the crest length in the direction of flow (USBR, 1997). If the upstream edge of a broad-crested weir is so rounded as to prevent contraction, and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table C3-S12-1.

**Table C3-S12-1: Broad-crested weir coefficient ($C_w$) values**

| Head (h)[1] (feet) | Weir Crest Breadth (b) (feet) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | **0.50** | **0.75** | **1.00** | **1.50** | **2.00** | **2.50** | **3.00** | **4.00** | **5.00** | **10.00** | **15.00** |
| 0.2 | 2.80 | 2.75 | 2.69 | 2.62 | 2.54 | 2.48 | 2.44 | 2.38 | 2.34 | 2.49 | 2.68 |
| 0.4 | 2.92 | 2.80 | 2.72 | 2.64 | 2.61 | 2.60 | 2.58 | 2.54 | 2.50 | 2.56 | 2.70 |
| 0.6 | 3.08 | 2.89 | 2.75 | 2.64 | 2.61 | 2.60 | 2.68 | 2.69 | 2.70 | 2.70 | 2.70 |
| 0.8 | 3.30 | 3.04 | 2.85 | 2.68 | 2.60 | 2.60 | 2.67 | 2.68 | 2.68 | 2.69 | 2.64 |
| 1.0 | 3.32 | 3.14 | 2.98 | 2.75 | 2.66 | 2.64 | 2.65 | 2.67 | 2.68 | 2.68 | 2.63 |
| 1.2 | 3.32 | 3.20 | 3.08 | 2.86 | 2.70 | 2.65 | 2.64 | 2.67 | 2.66 | 2.69 | 2.64 |
| 1.4 | 3.32 | 3.26 | 3.20 | 2.92 | 2.77 | 2.68 | 2.64 | 2.64 | 2.65 | 2.67 | 2.64 |
| 1.6 | 3.32 | 3.29 | 3.28 | 3.07 | 2.89 | 2.75 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| 1.8 | 3.32 | 3.32 | 3.31 | 3.07 | 2.88 | 2.74 | 2.68 | 2.66 | 2.65 | 2.64 | 2.63 |
| 2.0 | 3.32 | 3.31 | 3.30 | 3.03 | 2.85 | 2.76 | 2.27 | 2.68 | 2.65 | 2.64 | 2.63 |
| 2.5 | 3.32 | 3.32 | 3.31 | 3.28 | 3.07 | 2.89 | 2.81 | 2.72 | 2.67 | 2.64 | 2.63 |
| 3.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.20 | 3.05 | 2.92 | 2.73 | 2.66 | 2.64 | 2.63 |
| 3.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.19 | 2.97 | 2.76 | 2.68 | 2.64 | 2.63 |
| 4.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.70 | 2.64 | 2.63 |
| 4.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.74 | 2.64 | 2.63 |
| 5.0 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.07 | 2.79 | 2.64 | 2.63 |
| 5.5 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 3.32 | 2.88 | 2.64 | 2.63 |

[1]Measured at least 2.5h upstream of the weir

4. **V-notch weirs.** The discharge through a V-notch weir (Figure C3-S12-3) can be calculated from the following equation (Brater and King, 1976).

**Equation C3-S12-3**

$$Q = 2.5\tan\left(\frac{\theta}{2}\right)h^{2.5}$$

Where:
Q = discharge (cfs)
θ = angle of V-notch (degrees)
H = head on apex of notch (ft)

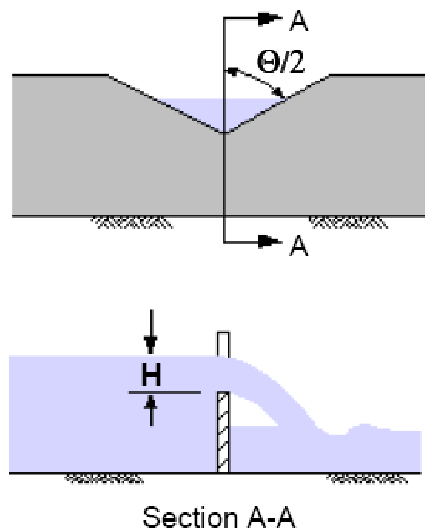



**Figure C3-S12-3: V-notch weir**
For a 60° V-notch weir:
$$\boldsymbol{Q - 4.33h^{2.5}}$$
For a 90° V-notch weir:
$$\boldsymbol{Q = 2.5h^{2.5}}$$

5. **Cipoletti (trapezoidal) weir.** The Cipoletti (or trapezoidal) weir has side slopes in the vertical to horizontal ratio of 4:1. Cipoletti weirs are considered fully contracted, and are installed as described below. The discharge coefficient for Cipoletti weirs is 3.367 (in English units), and it does not depend on L or P as for the rectangular weir. The discharge coefficient formulation is simpler than for rectangular weirs, but the accuracy is somewhat decreased - about ±5% (USBR, 1997). The Cipoletti weir equation is shown below for Q in cfs ($ft^3/s$), and head and length in feet units (USBR, 1997).

**Equation C3-S12-4**

$$Q = 3.367Lh^{3/2}$$

Where:
Q = discharge (cfs)
L = weir length (ft)
h = depth of water above crest (ft)

Note that L is measured along the bottom of the weir crest (not along the water surface). Weir side slopes should have a vertical to horizontal ratio of 4:1. Head (h) should be measured at a distance of at least 4h upstream of the weir.

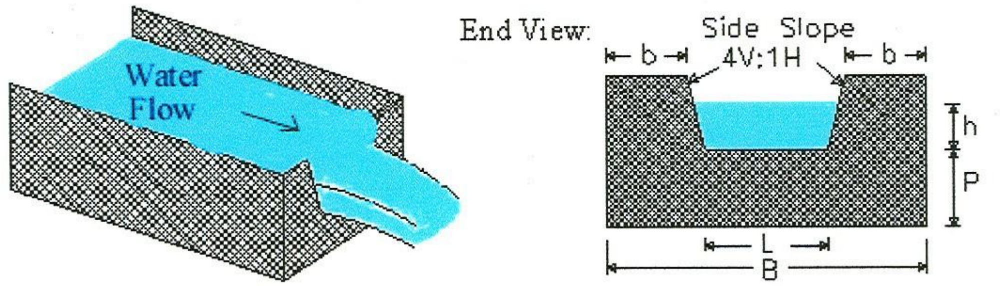



**Figure C3-S12-4: Cipoletti (trapezoidal) weir**

It doesn't matter how thick the weir is, except where water flows through the weir. The weir should be between 0.03 and 0.08 inches thick in the opening. If the bulk of the weir is thicker than 0.08 inch, the downstream edge of the opening can be chambered at an angle greater than 45° (60° is recommended) to achieve the desired thickness of the edges. Water surface downstream of the weir should be at least 0.2 feet below the weir crest (i.e. below the bottom of the opening).

Measured head (h) should be greater than 0.2 feet, but less than L/3. P is measured from the bottom of the upstream channel, and should be greater than $2h_{max}$, where $h_{max}$ is the maximum expected head. b is measured from the sides of the channel and also should be greater than $2h_{max}$.

6. **Pipes and culverts.** Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse- slope underwater pipe is often used for water quality or channel protection outlets. Pipes smaller than 12 inches in diameter may be analyzed as a submerged orifice as long as H/D is greater than 1.5. Note: For low-flow conditions, when the flow reaches and begins to overflow the pipe, weir flow controls the hydraulics. The flow will transition to orifice flow as the stage increases above the top of the opening.

   Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Chapter 14, or by using Equation C3-S12-5 (NRCS, 1984). The following equation is a general pipe flow equation that is derived through the use of the Bernoulli and continuity principles.

**Equation C3-S12-5**

$$Q = A_p \left[\frac{2gh}{1 + k_m + k_p L}\right]^{0.5}$$

Where:
Q = discharge (cfs)
$A_p$ = pipe cross sectional area (ft2)
g = acceleration of gravity (ft/s$^2$)
H = elevation head differential (ft)
$k_m$ = coefficient of minor losses (use 1.0)
$k_p$ = pipe friction coefficient (Manning's n and pipe diameter, D)

$$5087n^2/D^{4/2}$$

L = pipe length (ft)

7. **Proportional weirs.** Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary non-linearly with head. A typical proportional weir is shown in Figure C3-S12-5. Design equations for proportional

weirs are (Sandvik, 1985):

**Equation C3-S12-6**

$$Q = 4.97a^{0.5}b\left(H - \frac{a}{3}\right)$$

**Equation C3-S12-7**

$$\frac{x}{b} = 1 - \left(\frac{1}{3.17}\right)\left(arc\tan\left(\frac{y}{a}\right)^{0.5}\right)$$

Where:
Q = discharge (cfs)
Dimensions a, b, H, x, and y are shown in Figure C3-S12-5.

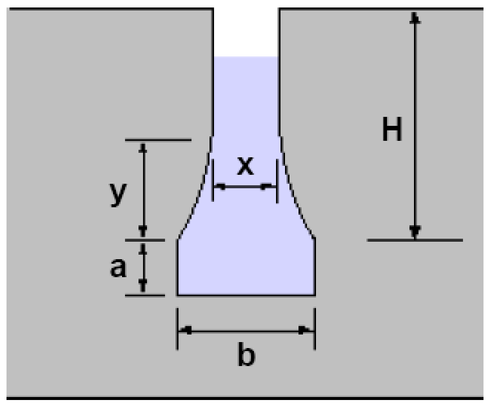



**Figure C3-S12-5: Proportional weir dimensions**

8. **Stand pipes and inlet boxes.** Stand pipes and inlet boxes have intake openings that are parallel to the water surface, as shown in Figure C3-S12-6. The structure is called a stand-pipe if it has a circular cross section and an inlet box from a rectangular cross section. Both surface openings discharge into a barrel sized large enough to prevent surcharge.

   Stand pipes and inlet boxes operate as weirs when the head over the structure is low (Equation C3-S12-2). The crest length, L, is calculated as:

   **Equation C3-S12-8**

   $$L = \pi D$$

   and

   **Equation C3-S12-9**

   $$L = 2B + 2D$$

   The equations above are respectively for stand pipes and inlet boxes where D = pipe diameter (ft) and B and D are the side lengths of the inlet box. It is important to note that the $C_w$ coefficient for this type of structure will have a different value from rectangular weirs and need special attention to detail. At higher heads, the stand pipe and inlet box will function as an orifice (Equation C3-S12-1a and Equation C3-S12-1b) will apply. The ranges over which the weir and orifice equation apply are not well established. The change from weir to orifice behavior occurs gradually over a transition depth. Typical practice is to use a transition head, $h_T$ defined as

   **Equation C3-S12-10**

   $$h_T = \frac{C_0 A_0}{C_w L}$$

and use the weir equation for $h < h_T$ and the orifice equation for $h > h_T$.

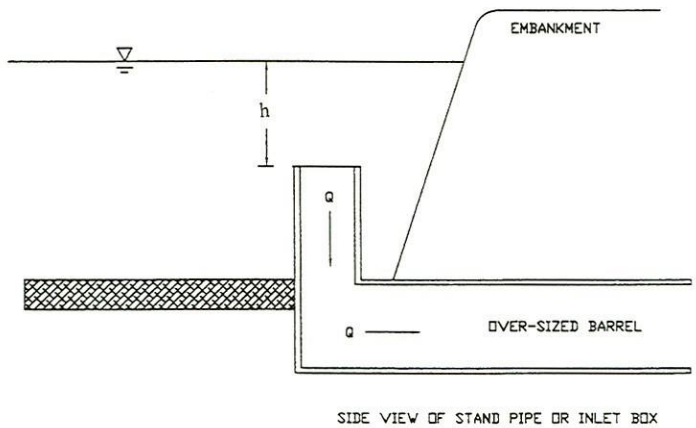



**Figure C3-S12-6: Riser style outlet**

9. **Perforated risers.** A special kind of orifice flow is a perforated riser, as illustrated in Figure C3-S12-7. The riser used in these systems is a vertical pipe perforated with equally-spaced round holes. Water enters the riser from the detention basin through the holes and flows into the outlet structure conduit. An orifice plate is installed at the bottom of the riser to restrict the flow if needed. In the perforated riser, an orifice plate at the bottom of the riser or in the outlet pipe just downstream from the elbow at the bottom of the riser, controls the flow. It is important that the perforations in the riser convey more flow than the orifice plate so as not to become the control. A formula was developed by McEnroe (1988) in reference to Figure C3-S12-7 that defined the intake characteristics of a perforated riser without a bottom orifice plate and is expressed as:

**Equation C3-S12-11**

$$Q = C_s\left(\frac{2A_s}{3h_s}\right)(2gh)^{3/2}$$

Where:
$C_s$ = dimensionless discharge coefficient of the side holes
$A_s$ = total area of the side holes, ft$^2$
$h_s$ = length of the perforated segment of the riser pipe, ft

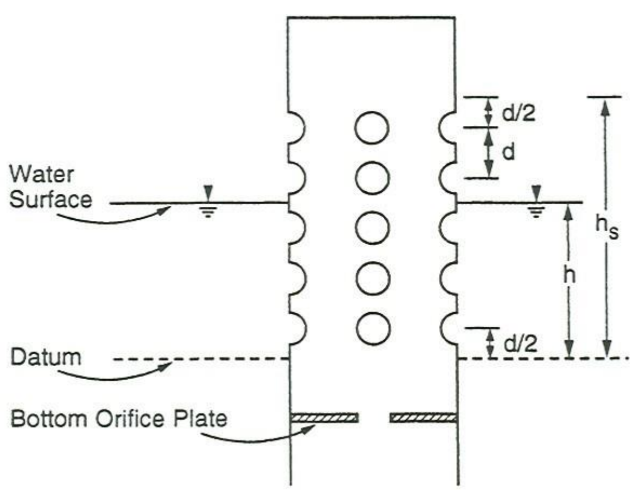
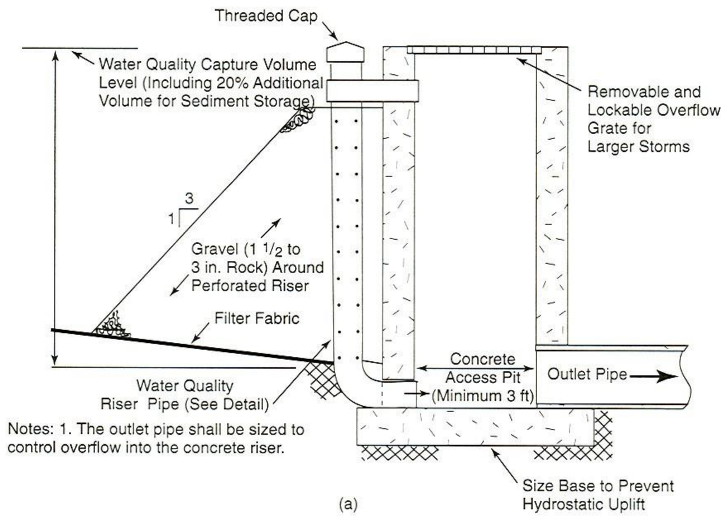
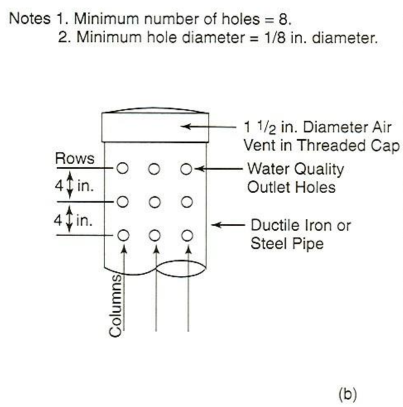



**Figure C3-S12-7: Definition schematic for perforated riser intake**

Both h and $h_s$ are measured from the same datum located at a distance d/2 below the centroid of the last row of side holes, where d = vertical spacing between centerlines of horizontal rows of holes of the side holes. The accepted value of the coefficient $C_s$ is 0.611 (McEnroe et al, 1988). Equation C3-S12-11 is only valid when $h < h_s$.

Equation C3-S12-11 will represent the stage-discharge relationship of the detention basin if the capacity of the outlet conduit is greater than the intake capacity of the perforated riser. This condition can be satisfied by designing the outlet conduit to flow partially full at the maximum outflow rate. Note that the form of Equation C3-S12-11 is the same as Equation C3-S12-2 (broad-crested weir equation), so a perforated riser could be classified for modeling purposes as a weir-type outlet. An example of a perforated riser water quality outlet with gravel pack protection is illustrated in Figure C3-S12-8.




| Maximum Number of Perforated Columns | | | | |
|---|---|---|---|---|
| Riser Diameter, in. | Hole Diameter, in. | | | |
| | 1/4 in. | 1/2 in. | 3/4 in. | 1 in. |
| 4 | 8 | 8 | ± | ± |
| 6 | 12 | 12 | 9 | ± |
| 8 | 16 | 16 | 12 | 8 |
| 10 | 20 | 20 | 14 | 10 |
| 12 | 24 | 24 | 18 | 12 |

| Hole Diameter, in. | Area of Hole, sq in. |
|---|---|
| 1/8 | 0.013 |
| 1/4 | 0.049 |
| 3/8 | 0.110 |
| 1/2 | 0.196 |
| 5/8 | 0.307 |
| 3/4 | 0.442 |
| 7/8 | 0.601 |
| 1 | 0.785 |

**Figure C3-S12-8: Perforated riser outlet: (a) outlet works with riser barrel and gravel pack for inlet debris protection and (b) water quality riser pipe detail**
Source: UFCD, 2005

10. **Combination outlets.** Combinations of orifices, weirs, and pipes can be used to provide multi- stage outlet control for different control volumes within a storage facility (i.e., water quality volume, channel protection volume, overbank flood protection volume, and/or extreme flood protection volume). There are generally two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure C3-S12-9 shows an example of an outlet structure designed for multiple levels of control, including the lower level control for the wet ED pond. The orifice plate outlet devices in Figure C3-S12-9 are sized to provide the total area needed to drain the ED volume in the specified time (usually 40 hours at brim-full capacity. A table for determining the spacing and total area of the orifices is provided in Table C3-S12-2 (UFCD, 2005).

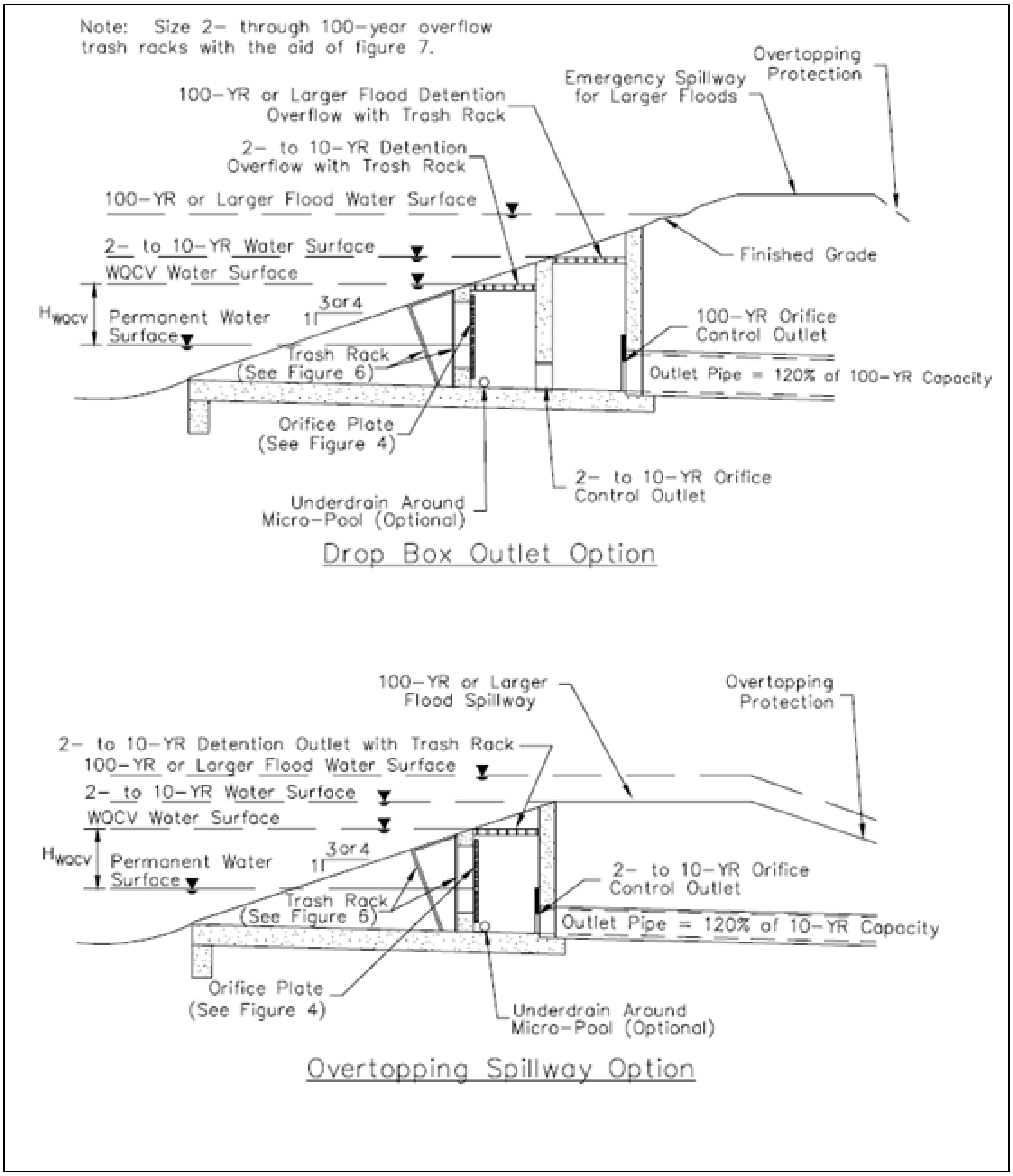



**Figure C3-S12-9: Combination outlet design for wet extended detention pond**
Source: UFCD, 2005

**Table C3-S12-2: Orifice plate sizing details**

**Circular Perforation Sizing:** This table may be used to size perforation in a vertical plate of riser pipe

| Hole Dia. (in)* | Hole Dia. (in) | Min Sc (in) | Area per Row (sq in) | | |
|---|---|---|---|---|---|
| | | | n = 1 | n = 2 | n = 3 |
| ¼ | 0.250 | 1 | 0.05 | 0.10 | 0.15 |
| 5/16 | 0.313 | 2 | 0.08 | 0.16 | 0.24 |
| 3/8 | 0.373 | 2 | 0.11 | 0.22 | 0.33 |
| 7/16 | 0.438 | 2 | 0.15 | 0.30 | 0.45 |
| ½ | 0.500 | 2 | 0.20 | 0.40 | 0.60 |
| 9/16 | 0.563 | 3 | 0.25 | 0.50 | 0.75 |
| 5/8 | 0.625 | 3 | 0.31 | 0.62 | 0.93 |
| 11/16 | 0.688 | 3 | 0.37 | 0.74 | 1.11 |
| ¾ | 0.750 | 3 | 0.44 | 0.88 | 1.32 |
| 13/16 | 0.813 | 3 | 0.52 | 1.04 | 1.56 |
| 7/8 | 0.875 | 3 | 0.60 | 1.20 | 1.80 |
| 15/16 | 0.938 | 3 | 0.69 | 1.38 | 2.07 |
| 1 | 1.000 | 4 | 0.79 | 1.58 | 2.37 |
| 1 1/16 | 1.063 | 4 | 0.89 | 1.78 | 2.67 |
| 1 1/8 | 1.125 | 4 | 0.99 | 1.98 | 2.97 |
| 1 3/16 | 1.188 | 4 | 1.11 | 2.22 | 3.33 |
| 1 ¼ | 1.250 | 4 | 1.23 | 2.46 | 3.69 |
| 1 5/16 | 1.313 | 4 | 1.35 | 2.70 | 4.05 |
| 1 3/8 | 1.375 | 4 | 1.48 | 2.96 | 4.44 |
| 1 7/16 | 1.438 | 4 | 1.62 | 3.24 | 4.86 |
| 1 ½ | 1.500 | 4 | 1.77 | 3.54 | 5.31 |
| 1 9/16 | 1.563 | 4 | 1.92 | 3.84 | 5.76 |
| 1 5/8 | 1.625 | 4 | 2.07 | 4.14 | 6.21 |
| 1 11/16 | 1.688 | 4 | 2.24 | 4.48 | 6.72 |
| 1 ¾ | 1.750 | 4 | 2.41 | 4.82 | 7.23 |
| 1 13/16 | 1.813 | 4 | 2.58 | 5.16 | 7.74 |
| 1 7/8 | 1.875 | 4 | 2.76 | 5.52 | 8.28 |
| 1 15/16 | 1.938 | 4 | 2.95 | 5.90 | 8.85 |
| 2 | 2.000 | 4 | 3.14 | 6.28 | 9.42 |

n = Number of columns of perforations

| | | | |
|---|---|---|---|
| Minimum steel plate thickness | ¼” | 5/16” | 3/8” |

*Designer may interfere to the nearest 32nd inch to better match the needed area if desired.

**Rectangular Perforation Sizing:** Use only one rectangular column whenever two 2-in diameter circular perforations cannot provide needed outlet area. (Rectangular Height = 2”, Rectangular Width = Required, Area per Row / 2”)

| Rectangular Hole Width (in) | Min Steel Thickness (in) |
|---|---|
| 5 | ¼ |
| 6 | ¼ |
| 7 | 5/32 |
| 8 | 5/16 |
| 9 | 11/32 |
| 10 | 3/8 |
| >10 | ½ |

Source: UFCD, 2005

## C. Extended detention (water quality and channel protection) outlet design

Extended detention orifice sizing is required in design applications that provide extended detention for downstream channel protection or the ED portion of the water quality volume. In both cases, an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both WQv extended detention and Cpv control (wet ED pond, micropool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices - one for the water quality control outlet and one for the channel protection drawdown.

The outlet hydraulics for peak control design (overbank flood protection and extreme flood protection) is usually straightforward in that an outlet is selected that will limit the peak flow to some pre-determined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality treatment or downstream channel protection, however, the storage volume is detained and released over a specified amount of time (e.g., 24 hours). The release period is a brim drawdown time, beginning at the time of peak storage of the water quality volume until the entire calculated volume drains out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods. The WQv is determined by the procedures included in Chapter 3 - Section 6 Small Storm Hydrology with the water quality capture volume (WQCV) being the preferred method for extended detention basins (ASCE/WEF method). A minimum drawdown time (detention time) of 24 hours should be used. Water quality performance will improve as the brim-full detention time approaches 40 hours.

The following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999.

- Use the maximum hydraulic head associated with the storage volume and maximum flow, and calculate the orifice size needed to achieve the required drawdown time; and route the volume through the basin to verify the actual storage volume used and the drawdown time.
- Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time. These two procedures are outlined in the examples below, and can be used to size an extended detention orifice for water quality and/or channel protection.

1. **Method 1: maximum hydraulic head with routing.** A wet ED pond sized for the required water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice. Given the following information, calculate the required orifice size for water quality design.

   Given: water quality volume (WQv) = 0.76 ac-ft = 33,106 $ft^3$ Maximum hydraulic head ($H_{max}$) = 5.0 ft (from stage vs. storage data)

a. **Step 1:** Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the water quality volume (or channel protection volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

$$Q_{avg} = \frac{33{,}106ft^3}{(24hr)\left(\frac{3{,}600s}{hr}\right)} = 0.38cfs$$

$$Q_{max} = 2 \times Q_{avg} = 2 \times .038 = 0.76cfs$$

b. **Step 2:** Determine the required orifice diameter by using the orifice equation (Equation C3-S12-1) and $Q_{max}$ and $H_{max}$:

$$Q = CA(2gH)^{0.5}$$

or:

$$A = \frac{Q}{C(2gH)^{0.5}}$$

$$A = \frac{0.76}{0.6[(2)(32.2)(5.0)]^{0.5}} = 0.71ft^3$$

Determine pipe diameter from

$$A = \frac{3.14d^2}{4}$$

then

$$d = \left(\frac{4A}{3.14}\right)^{0.5}$$

$$D = \left[\frac{4(0.071)}{3.14}\right]^{0.5} = 0.30ft = 3.61in$$

Use a 3.6-inch diameter water quality orifice.

Routing the water quality volume of 0.76 ac-ft through the 3.6-inch water quality orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours, or if the actual time achieved will provide adequate pollutant removal.

2. **Method 2: average hydraulic head and average discharge.** Using the data from the previous example, use Method 2 to calculate the size of the outlet orifice.

Given: water quality volume (WQv) = 0.76 ac-ft = 33,106 ft3 Average hydraulic head ($h_{avg}$) = 2.5 ft (from stage vs. storage data)

a. **Step 1:** Determine the average release rate to release the water quality volume over a 24-hour time period.

$$Q = \frac{33{,}106ft^3}{(24hr)\left(\frac{3{,}600s}{hr}\right)} = 0.38cfs$$

b. **Step 2:** Determine the required orifice diameter by using the orifice equation and the average head on the orifice:

$$Q = CA(2gH)^{0.5}$$

or

$$A = \frac{Q}{C(2gH)^{0.5}}$$

$$A = 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05ft^3$$

Determine pipe diameter from

$$A = \frac{3.14D^2}{4}$$

then

$$d = \left(\frac{4A}{3.14}\right)^{0.5}$$

$$D = \left[\frac{4(0.05)}{3.14}\right]^{0.5} = 0.252ft = 3.3in$$

Use a 3-inch diameter water quality orifice.

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

## D. Sizing of single-stage risers

For the sizing of risers, it is necessary to determine both the required volume of storage and the physical characteristics of the riser. Initial estimates of the required storage volume for WQV and Cpv are discussed in Chapter 5, section 6. Estimates of the required storage for peak flow control ($Q_p$) are presented in Chapter 3 - Section 9 Detention Storage Design. The physical characteristics of the outlet structure include the outlet pipe diameter, the riser diameter, either the length of the weir or the area of the orifice, and the elevation characteristics of the riser. Single-stage risers with weir flow and orifice flow are illustrated in Figure C3-S12-10 and Figure C3-S12-11, respectively. The equations used to define the relationship between the discharge (Q) and the depth in feet (h) above the weir or orifice were presented above.

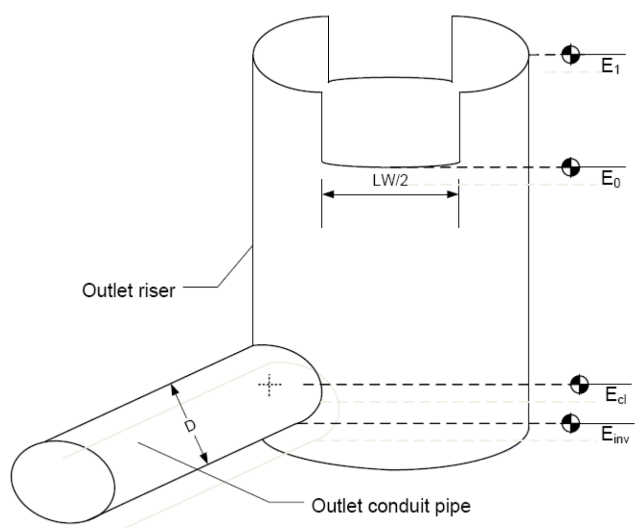



**Figure C3-S12-10: Single-stage riser with weir flow**

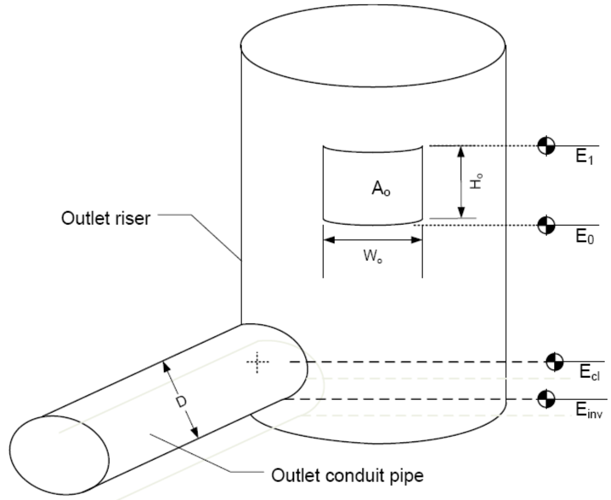



**Figure C3-S12-11: Single stage riser with orifice flow**

The flow conditions for different water surface conditions can also alter the flow condition (orifice vs. weir) for the outlet structure, as illustrated in Figure C3-S12-12 and Figure C3-S12-13. In Figure C3-S12-12, the flow condition changes from a weir to an orifice as the water surface elevation rises. The flow conditions in the rider barrel and the outlet pipe can also impact the hydraulic performance of the structure.

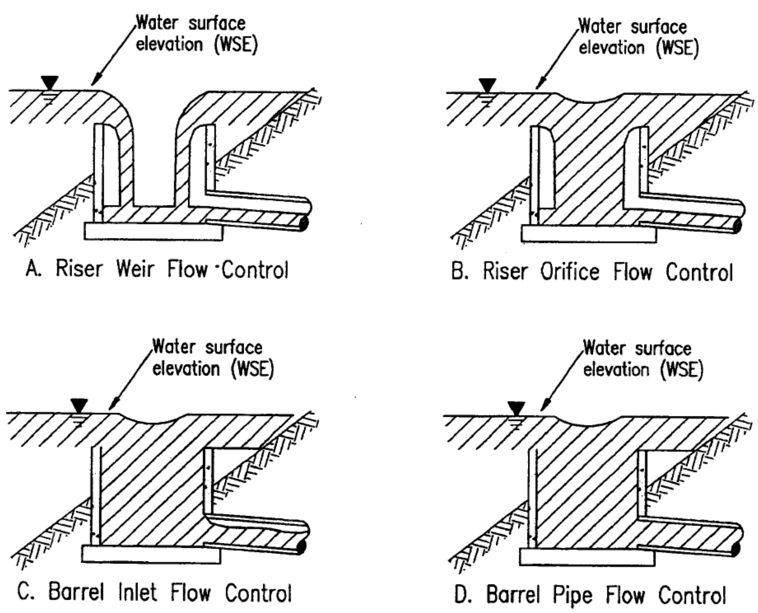



**Figure C3-S12-12: Riser flow diagrams**
Source: VDCR, 1999

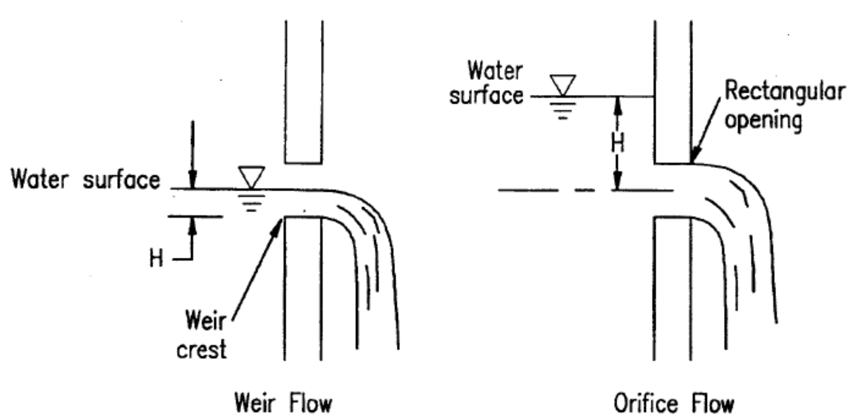



**Figure C3-S12-13: Weir and orifice flow**
Source: VDCR, 1999

The required input for sizing a single-stage riser includes the following:

- The pre- and post-development runoff volume ($Q_b$ and $Q_a$) in inches
- The peak discharges for the pre-and post-development conditions ($q_b$ and $q_a$) in cfs
- The length and roughness of the outlet pipe (Manning's n)
- The drainage area (A) acres
- The elevation of the bottom of either the orifice or weir.
- The stage-storage relationship for the proposed site (Chapter 3 - Section 10 Channel and Storage (Reservoir) Routing)

The general procedure for sizing the riser involves the following steps:

- Step 1: Estimate the volume of storage required using method presented in Chapter 3 - Section 9 Detention Storage Design.
- Step 2: Estimate the required depth of storage from the stage-storage curve for the site (the procedure is described in Chapter 3 - Section 10 Channel and Storage (Reservoir) Routing).
- Step 3: Determine the diameter of the outlet pipe.
- Step 4: Size the orifice or weir.

**Procedure for sizing the riser**

In the steps outlined below, the required volume of storage will also be determined. The following steps can be used and summarized in Table C3-S12-4 (Outlet Design Worksheet):

1. For the design storm frequency and using the 24-hour rainfall and pre- and post-development CN's, determine:
   a. Predevelopment runoff depth, $Q_b$, using WinTR-55.
   b. Post-development runoff depth, $Q_a$, using WinTR-55.
2. From the WinTR-55 analysis:
   a. Determine the predevelopment peak discharge, $q_{pb}$.
   b. Determine the post-development peak discharge, $q_{pa}$.
3. Compute the discharge ratio: $R_q = q_{pb}/q_{pa}$
4. Using $R_q$ from Step 3 and the $V_s/V_r$ curve (Figure C3-S12-14), find the required storage volume, $V_s$:

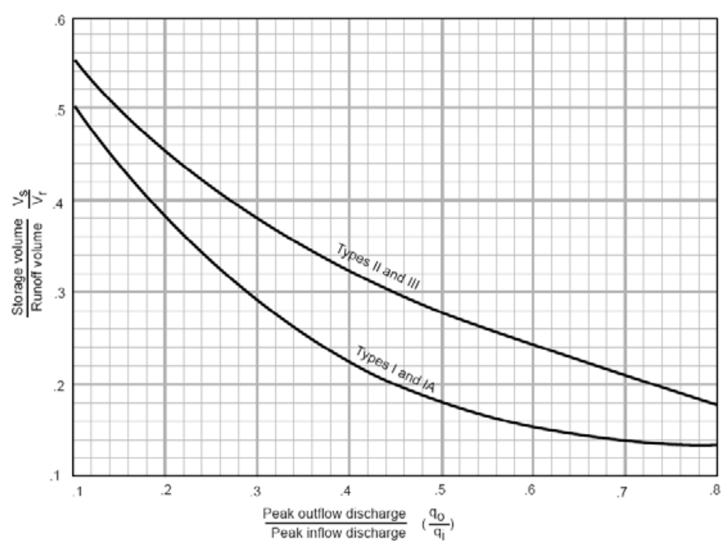



**Figure C3-S12-14: Approximate detention basin routing**

Source: NRCS TR-55, 1986

5. Compute the volume of storage in inches:
   a. $V_s = V_r R_q$ where $V_r = Q_a$
   b. Convert $V_s$ to acre-ft (multiply by A/12 where A is in acres)
6. Using the elevation $E_0$, obtain the volume of dead storage, $V_d$ from the elevation-storage curve. See example

curve in Figure C3-S12-15 and procedure in Chapter 3 - Section 10 Channel and Storage (Reservoir) Routing.

7. Compute the total storage in acre-feet: $V_T = V_d + V_s$
8. From the elevation-storage curve, use the value of $V_T$ to determine the water surface elevation, $E_1$.
9. Size the outlet pipe:
   a. Obtain the friction head loss coefficient $K_p$ from Table C3-S12-3.
   b. Using the product $LK_p$, obtain C* from Figure C3-S12-15 where L is length of the conduit pipe.

**Table C3-S12-3: $K_p$ values for RCP (n=0.013) and CMP (n= 0.024)***

| Pipe Diameter (in) | RCP | CMP |
|---|---|---|
| 12 | 0.03129 | 0.10665 |
| 18 | 0.01822 | 0.06211 |
| 24 | 0.01242 | 0.04233 |
| 30 | 0.01061 | 0.03617 |
| 27 | 0.00922 | 0.03143 |
| 36 | 0.00723 | 0.02465 |
| 42 | 0.00589 | 0.02007 |
| 48 | 0.00493 | 0.01680 |
| 54 | 0.00421 | 0.01436 |
| 60 | 0.00366 | 0.01247 |
| 72 | 0.00287 | 0.00978 |

*For other values of n and D, $K_p$ can be calculated by

$$K_p = 5087 n^2 D^{-4.3}$$

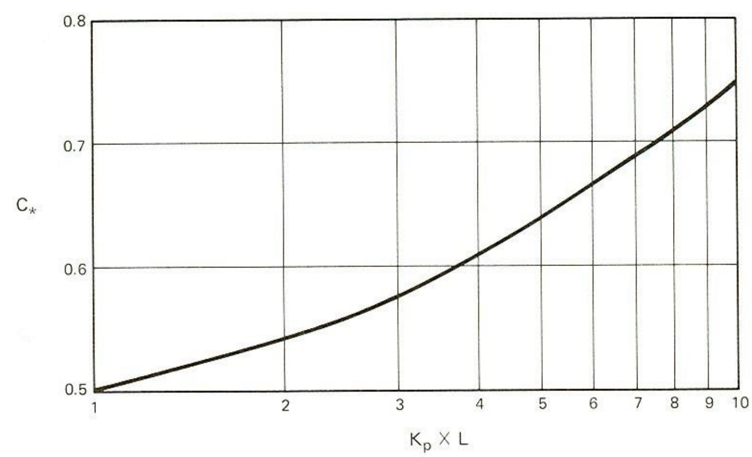



**Figure C3-S12-15**: Conduit diameter coefficient C*
Source: Chow

10. If the outlet is an orifice determine the characteristics of the orifice:
    a. Set the orifice width, $W_o$; as a starting value use 0.75D
    b. Compute the required area, $A_o$: $A_o = 0.2283q_{pb}/(E_1-E_0)^{0.5}$
    c. Compute the height: $H_o = A_o/W_o$
11. If the outlet is a weir, determine the characteristics of the weir length: $L_w = q_{pb}/3.1(E1-E0)^{1.5}$
12. Determine the conduit invert elevation (ft) at the face of the riser: $E_{inv} = E_c - D/2$

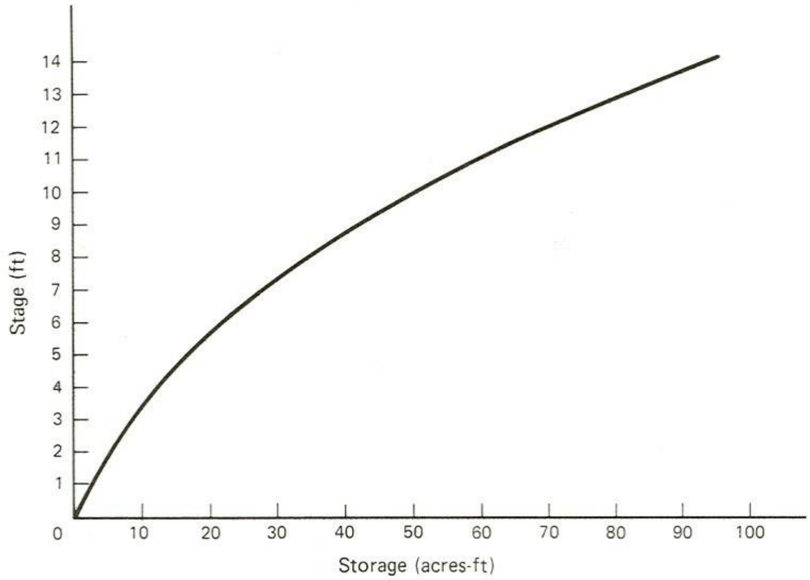



**Figure C3-S12-16: Example elevation (stage)-storage curve**

Once the physical characteristics of the outlet structure are determined, the stage-discharge relationship can be determined for a range of values of h. This stage-discharge relationship can then be used along with the stage-storage curve to complete a storage routing to confirm the final design. In the case of two-stage riser, as the stage increases a composite stage-discharge relationship will govern.

**Table C3-S12-4: Worksheet for single-stage and two-stage riser outlet riser design**

| Watershed Characteristics | | | | |
|---|---|---|---|---|
| | **Units** | **Pre-** | **Post-** | **Comments** |
| A | acres | | | |
| CN | - | | | 2C-5 & WinTR-55 |
| $t_c$ | hr | | | 2C-3 & WinTR-55 |
| P | in | | | **Error! Reference source not found.** |
| $I_a/P$ | | | | |
| $I_a/P$ | | | | |

| Outlet Facility Characteristics | | |
|---|---|---|
| **n** | | **Comment** |
| L | ft | |
| D | ft | Initial estimate |
| $K_p$ | | |
| C* | | |
| $E_0$ | ft | |
| $E_c$ | ft | |

| Step | Parameter | Units | Low Stage | High Stage | Comments |
|---|---|---|---|---|---|
| 1 | $Q_b$<br>$Q_a$ | inches<br>inches | | | From WinTR-55 |
| 2 | $q_{pb}$<br>$q_{pa}$ | ft³/sec<br>ft³/sec | | | From WinTR-55<br>or Rational method |
| 3 | $R_q$ | | | | $R_q = \frac{q_{pb}}{q_{ba}}$ |
| 4 | $V_r$ | inches | | | $V_r = Q_a$ (From WinTR-55)<br>$Q_a$ =volume of post-dev runoff |
| 5 | $V_s$ | inches<br>acre-ft | | | $V_s = V_r R_q$<br>$V_s = V_s(A/12)$ |
| 6 | $V_d$ | acre-ft | | | From elevation-storage curve |
| 7 | $V_T$ | acre-ft | | | $V_T = V_d + V_s$ |
| 8 | E | ft | | | From elevation-storage curve |
| 9 | D | ft | | | $D = C \times {q_{pb}}^{0.5}(E_1 - E_c)^{-0.25}$ |
| 10 | $W_o$<br>$A_o$<br>$H_o$<br>$q_{o2}$ | ft<br>ft<br>ft<br>ft³/sec | | | Begin with 0.75D<br>$A_o = 0.2283 q_{pb}/(E_1 - E_0)^{0.5}$<br>$H_0 = A_o/W_o$<br>$q_{02} = 4.82 A_0 (E_2 - E_1)^{0.5}$ |
| 11 | $L_{w1}$<br>$q_{o2}$ | ft<br>ft³/sec | | | $L_{w1} = q_{pb1}/[3.1(E_1 - E_0)^{1.5}]$<br>$q_{o2} = 3.1 L_{w1}(E_2 - E_1)^{1.5}$ |
| 12 | $L_{w2}$ | ft | | | $L_{w2} = (q_{pb2} - q_{o2})/[3.1(E_2 - E_1)^{1.5}]$ |
| 13 | $E_{inv}$ | ft | | | $E_{inv} = E_{cl} - 0.5D$ |

## E. Design of multiple-stage outlets

Multiple-stage outlets are used when control of flow rates for two or more design requirements are needed. Multiple outlet configurations are used when control of extended detention for WQv and/or Cpv are required in addition to peak flow control for overbank flood protection ($Q_p$) and the extreme flood ($Q_f$). Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. The goal for multiple-stage outlets is to determine the most economical and hydraulically efficient design. A number of iterative storage routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The final stage-discharge table or rating curve will be a composite of the different outlets that are used for different elevations within the multi-stage riser. The structure of a two-stage outlet riser is similar to a single-stage outlet, except that it includes a weir and orifice or two weirs (Figure C3-S12-17). For the weir-orifice structure, the orifice is used to control the more frequent event (WQv or Cpv), and the larger event (>5-year) is controlled using the weir. Runoff from the smaller and larger events is often referred to as the low-stage and high-stage events, respectively. Since the two events will not occur at the same time, both the low-stage weir/orifice and the high-stage weir will function to control the high-stage event.

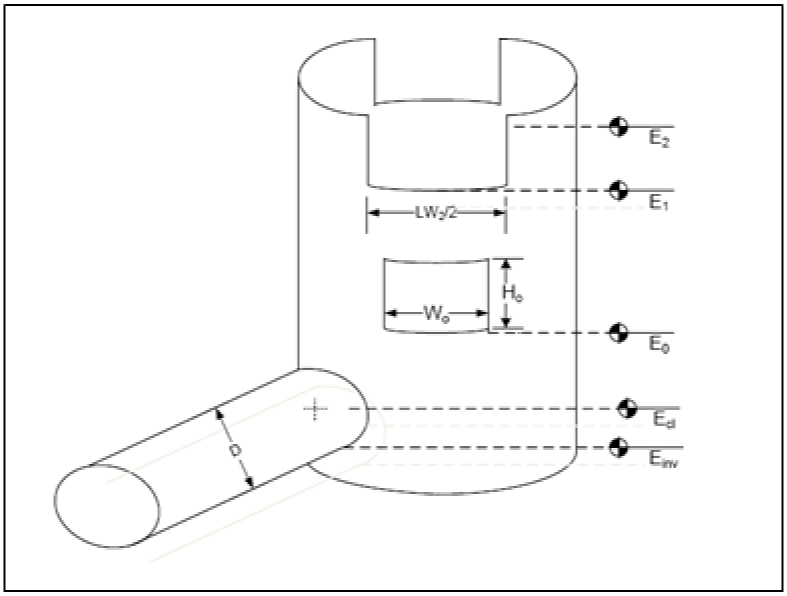



**Figure C3-S12-17: Schematic of two-stage outlet riser**

A two-stage configuration with an orifice-weir can provide adequate control since the orifice does not need to pass large flow rates. However, an orifice will not pass larger flow rates at low heads, so for basins situated on sites with lower relief, an orifice would not be an efficient design because the opening becomes larger as the amount of available head decreases. On flatter sites, the designer should consider evaluating a weir for the high-frequency event as well. For a given head, a weir will be more efficient in conveying the outlet discharge. Since sites with mild slopes will not be capable of high heads, a weir configuration would be better suited for controlling both the low-frequency and high-frequency events. An alternative to the two-stage riser with an orifice and a weir is an outlet configuration with two risers, each serving as a weir. The weir crests are set at different elevations with one riser controlling the high-frequency event (WQv or Cpv) and the other riser controlling the low-frequency event (peak discharge control). The two risers can be connected to a single pipe-outlet for discharge downstream.

**Procedure for sizing the riser**
The sizing for a two-stage riser requires some additional considerations from the single-stage riser procedure above. The procedure follows the same general steps, but both a high-stage weir and low-stage orifice/weir must be determined. The input is basically the same, but must be determined and checked for both the high- and low-level design events (design storm frequencies). For example, there will be a design storm event associated with each of the low-stage and high-stage outlets. For the low-stage outlet, the controlling volume could be the WQv or the Cpv and the extended detention time would be used to determine the size of the required opening as presented earlier in this section.

The basic consideration for two-stage riser sizing is that there are two individual flow rate and storage volumes for each riser, and there are different design storm conditions for each.

For an outlet configured with two openings, a composite stage-discharge curve is developed to cover the range of different flow conditions as the water level rises. An example is illustrated in Figure C3-S12-18.

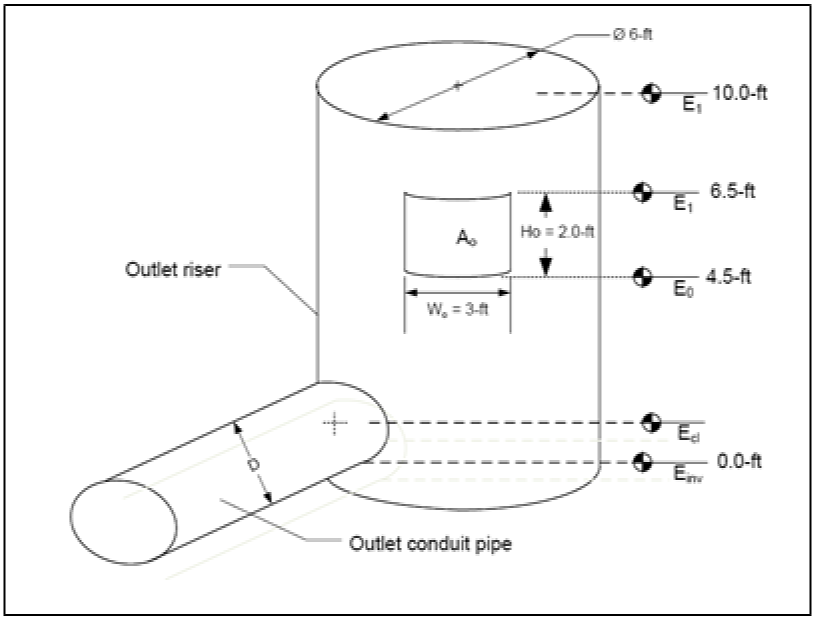



**Figure C3-S12-18: Example two-stage outlet riser**

The riser has an inside diameter of 6 feet, and the top of the riser will function as a broad-crested weir of length L = 6.0π ft = 18.84 feet. The orifice has an area of 6 ft2, and the stage-discharge relationship can be described as follows:

$$Q = 0$$

$$h \leq 4.5ft$$

$$Q = C_w L(2gh)^{3/2} = (3.1)(3)[2g(h - 4.5)]^{3/2} = 9.3[2g(h - 4.5)]^{3/2}$$

$$4.5ft < h \leq 6.5ft$$

$$Q = CA_o(2gh)^{0.5} = (0.6)(3)(2)[2g(h - 4.5)]^{0.5} = 3.6[2g(h - 4.5)]^{0.5}$$

$$6.5ft < h \leq 10ft$$

$$Q = 18.0\pi(h - 10)^{1.5} + 3.6[2g(h - 4.5)]^{0.5}$$

$$10ft < h$$

## F. Extended detention outlet protection

Small low-flow orifices such as those used for extended detention applications can easily clog, preventing the structural control from meeting its design purpose(s), and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

- The use of a reverse slope pipe attached to a riser for a stormwater pond or wetland with a permanent pool (see Figure C3-S12-19). The inlet is submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe, and to avoid discharging warmer water at the surface of the pond.
- The use of a hooded outlet for a stormwater pond or wetland with a permanent pool (see Figure C3-S12-20 and Figure C3-S12-21).
- Internal orifice protection through the use of an over-perforated vertical stand pipe with ½-inch orifices or slots that are protected by wire cloth and a stone filtering jacket (see Figure C3-S12-20 and Figure C3-S12-22).
- Internal orifice protection through the use of an adjustable gate valves can to achieve an equivalent orifice diameter.

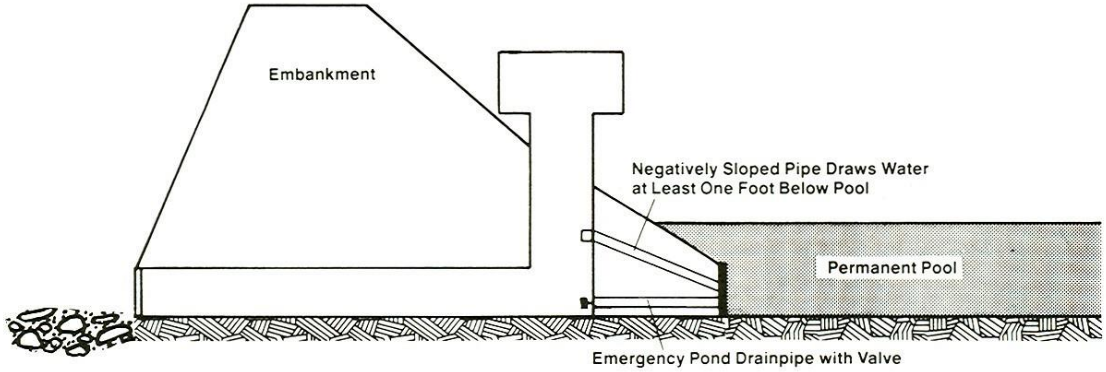



**Figure C3-S12-19: Reverse slope pipe outlet**
Source: Schueler, 1987

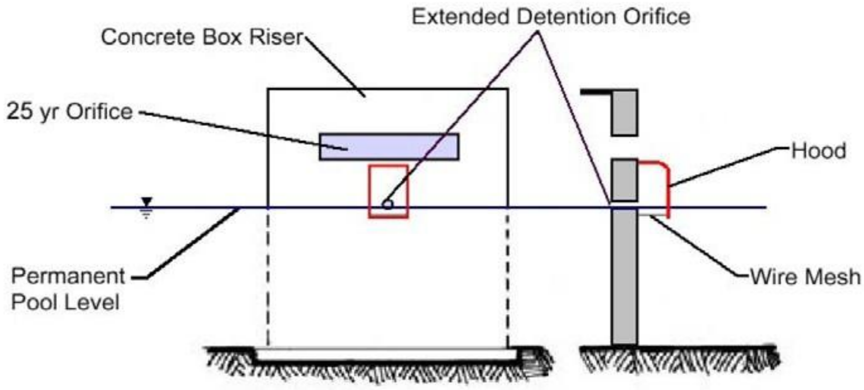



**Figure C3-S12-20: Schematic of hooded outlet**
Source: Schueler, 1987

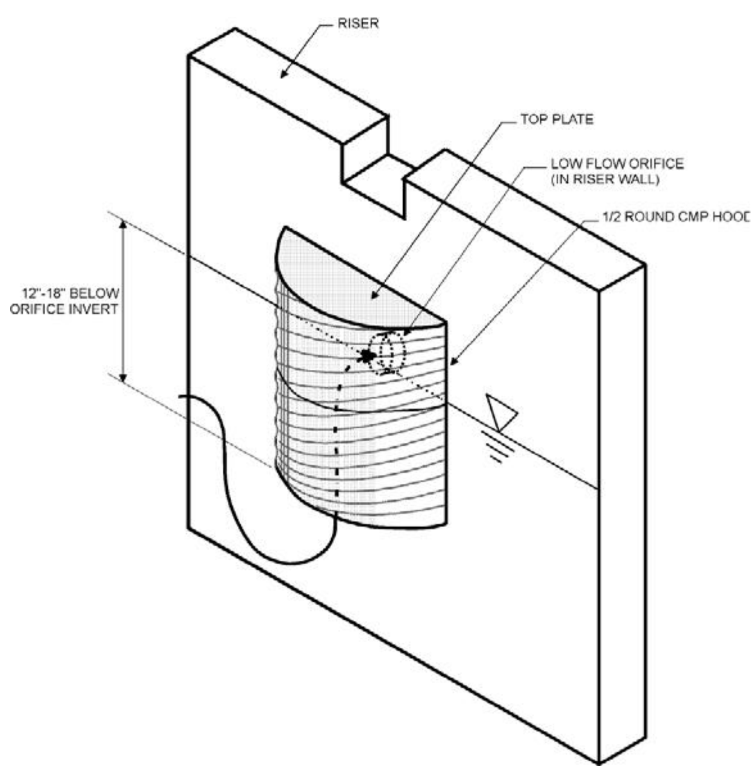



**Figure C3-S12-21: Hooded outlet**
Source: Virginia DCR, 1999

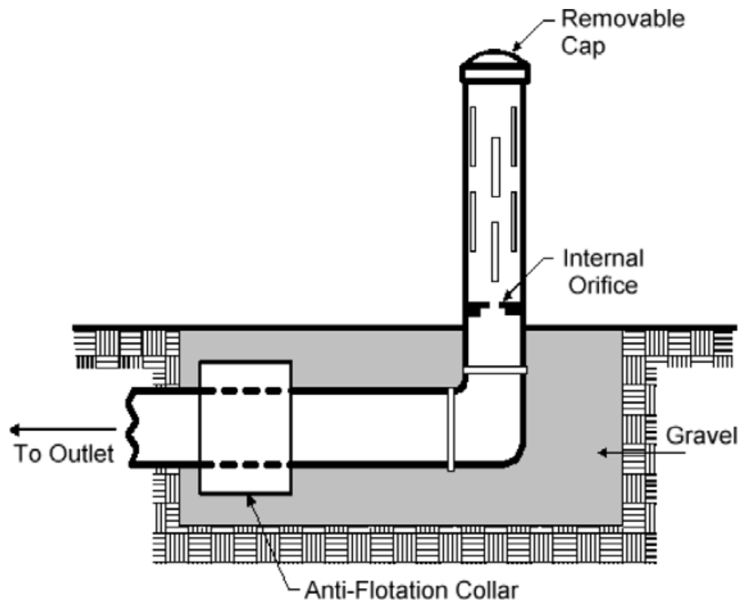



**Figure C3-S12-22: Slotted pipe riser**
Source: Schueler, 1987

## G. Trash racks and safety grates

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances, trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure.
- Capturing debris in such a way that relatively easy removal is possible.
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas.
- Providing a safety system that prevents anyone from being drawn into the outlet, and allows them to climb to safety.

The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety, and size of outlet. Well-designed trash racks can also have an aesthetically-pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in Figure C3-S12-23. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.

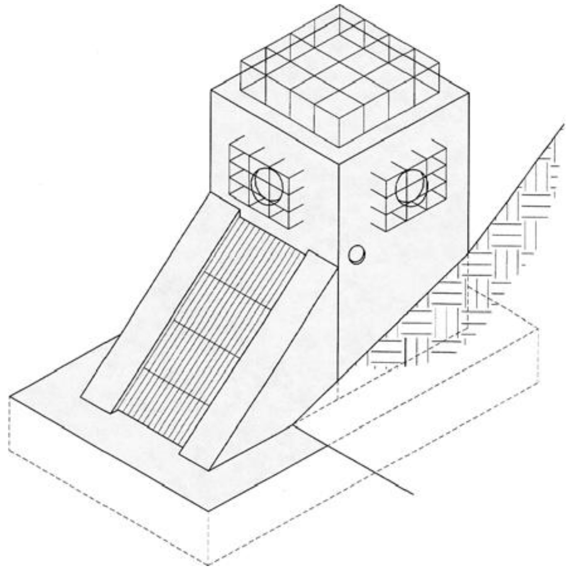


**Figure C3-S12-23: Example outlet structure with several types of trash racks**
Source: VDCR, 1999

The trash racks must have a combined total open area such that partial plugging will not adversely restrict flows through the outlet works. While a universal guideline does not exist for stormwater outlets, a common rule-of-thumb is to provide a trash rack open area at least 10 times larger than the control outlet orifice (ASCE, 1992). The surface area of all trash racks should be maximized, and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required. To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete, it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3:1 to 5:1 to allow trash to slide up the rack with flow pressure and rising water level - the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in various industry literature. Figure C3-S12-24 gives opening estimates based on outlet diameter (UDFCD, 2005). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive, or if a person becomes pinned to the rack. Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1992). Racks can be hinged on top to allow for easy opening and cleaning. The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore, head losses through the grate should be calculated.

A number of empirical loss equations exist, though many have difficult to estimate variables. Two are provided below to allow for comparison.

ASCE/WEF (1992) provides the following equation (based on German experiments) for losses through bar screens. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40-50% is chosen as a working assumption.

**Equation C3-S12-12**

$$H_g = K_{g1}\left(\frac{w}{x}\right)^{\frac{4}{3}}\left(\frac{{V_u}^2}{2g}\right)sin\theta_g$$

Where:
$H_g$ = head loss through grate (ft)
$K_{g1}$ = bar shape factor:
2.42 - sharp-edged rectangular
1.83 - rectangular bars with semicircular upstream faces
1.79 - circular bars
1.67 - rectangular bars with semicircular up- and downstream faces
w = maximum cross-sectional bar width facing the flow (in)
x = minimum clear spacing between bars (in)
$V_u$ = approach velocity (ft/s)
$\theta_g$ = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks, but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

**Equation C3-S12-13**

$$H_g = {K_g}^2{V_u}^2/2g$$

Where:
${K_g}^2$ is defined from a series of fit curves as:

sharp edged rectangular (length/thickness = 10)

$${K_g}^2 = 0.00158 - 0.03217A_r + 7.1789{A_r}^2$$

sharp edged rectangular (length/thickness = 5)

$${K_g}^2 = -0.00731 + 0.069453A_r + 7.0856{A_r}^2$$

round edged rectangular (length/thickness = 10.9)

$${K_g}^2 = -0.00101 + 0.02520A_r + 6.0000{A_r}^2$$

circular cross section

$${K_g}^2 = 0.00866 + 0.13589A_r + 6.0357{A_r}^2$$

and $A_r$ is the ratio of the area of the bars to the area of the grate section.

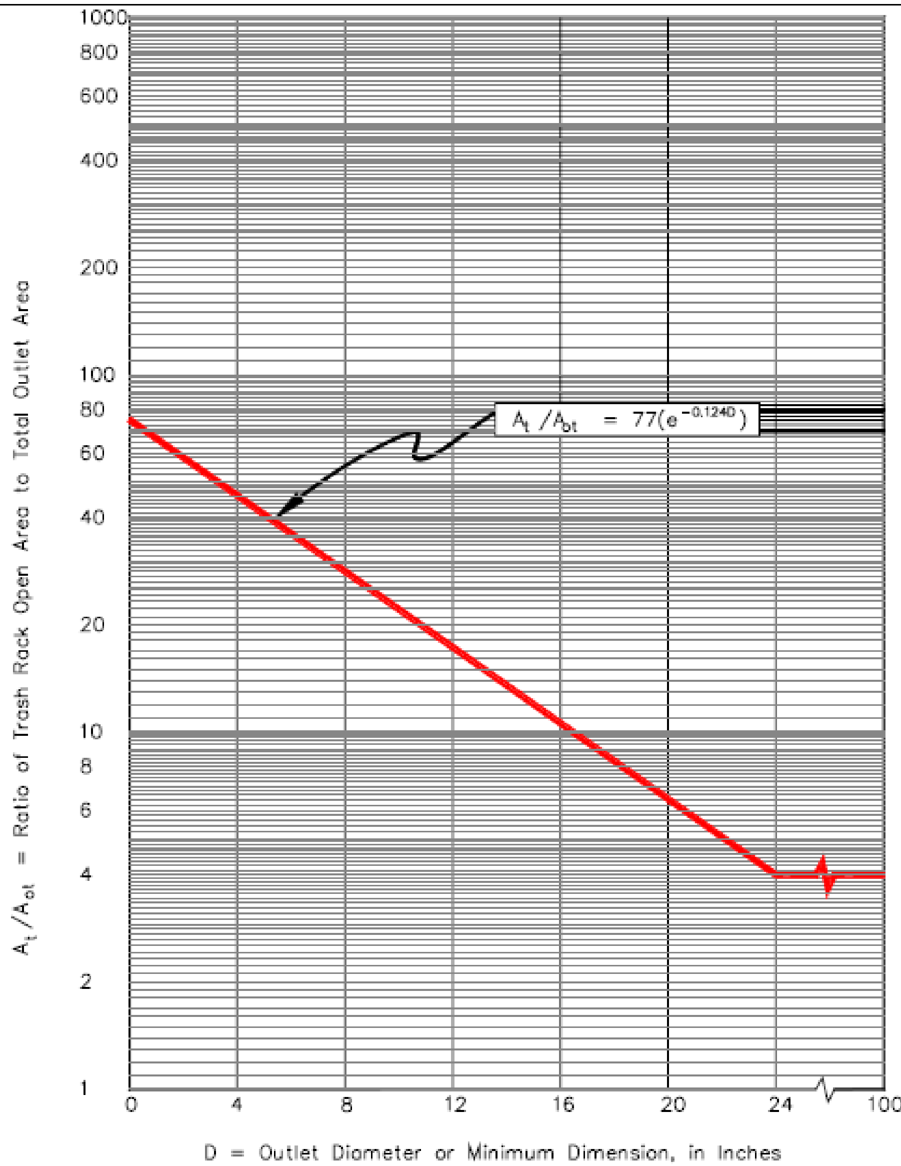



**Figure C3-S12-24: Minimum trash rack open area**
Source: UFCD, 2005

## H. Secondary outlets

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure C3-S12-25 shows an example of a secondary spillway. In many cases, onsite stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. For these smaller detention basins, the standard design approach is to size the spillway to convey the 100-year flood discharge, or design the embankment to withstand overtopping without failure. The State of Iowa regulations for small earthen embankment dams are contained in Iowa DNR Technical Bulletin 16 (1990). Most onsite detention basins will be classified as moderate-hazard structures if located with residential and or commercial infrastructure, and buildings in the downstream corridor. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir. By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor. An earthen embankment structure is classified a major structure, according to the following hazard classes and criteria (Iowa DNR Tech Bulletin #16):

- **High**. All structures.
- **Moderate.** Permanent storage >100 acre-feet; permanent + temporary storage >250 acre-feet at top-of-dam elevation.
- **Low.** Height-storage product >30,000 (based on emergency spillway crest elevation).

For moderate-hazard and low-hazard dams classified as major structures, the freeboard design flood is determined as:

- 0.5 x PMF (probable maximum flood)
- Flood hydrograph produced by multiplying the ordinate of the PMF hydrograph by 0.5
- 6-hour duration storms

For low-hazard dams not classified as major structures the freeboard design flood is determined as:

- Product of emergency spillway crest height in feet (measured from channel elevation at centerline of dam) and the total storage volume (acre-ft) at the emergency spillway crest elevation is between 3,000 and 30,000

$$Rainfall = P100 + 0.12(PMP - P100)$$

- 6-hour storm duration
- For dams without emergency spillways, storage volume and effective height determined by measuring from channel bottom to top of dam at the centerline.
- Rainfall = P50 and 24-hour duration event

**Emergency spillway design**

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (Figure C3-S12-25). The emergency spillway is proportioned to pass flows in excess of the design flood (typically the 100-year flood or greater) without allowing excessive velocities, and without overtopping of the embankment. Flow in the emergency spillway is open channel flow (see Chapter 15 for more information). Normally, it is assumed that critical depth occurs at the control section. NRCS manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper than 3:1 horizontal to vertical.

General design criteria for secondary spillways are:

- Should only operate at floods greater than the principal spillway design flood
- Flow velocities should be non-erosive: ≈5 fps
- Construct on undisturbed soil
- Use ramp spillway on constructed fill
- Use smooth horizontal and vertical transitions and alignments
- Place outlet a safe distance from the downstream toe of the structure
- Energy dissipation at the outlet

Design criteria for earthen secondary spillways are:

- Minimum bottom width: 10 feet
- For major structures, minimum depth is 3 feet
- For non-major structures, minimum depth is 2 feet
- Profile through the emergency spillway should be horizontal for at least 30 feet through the crest control section
- Exit channel slopes: >1% and <10%; maintain critical depth control at the crest

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.

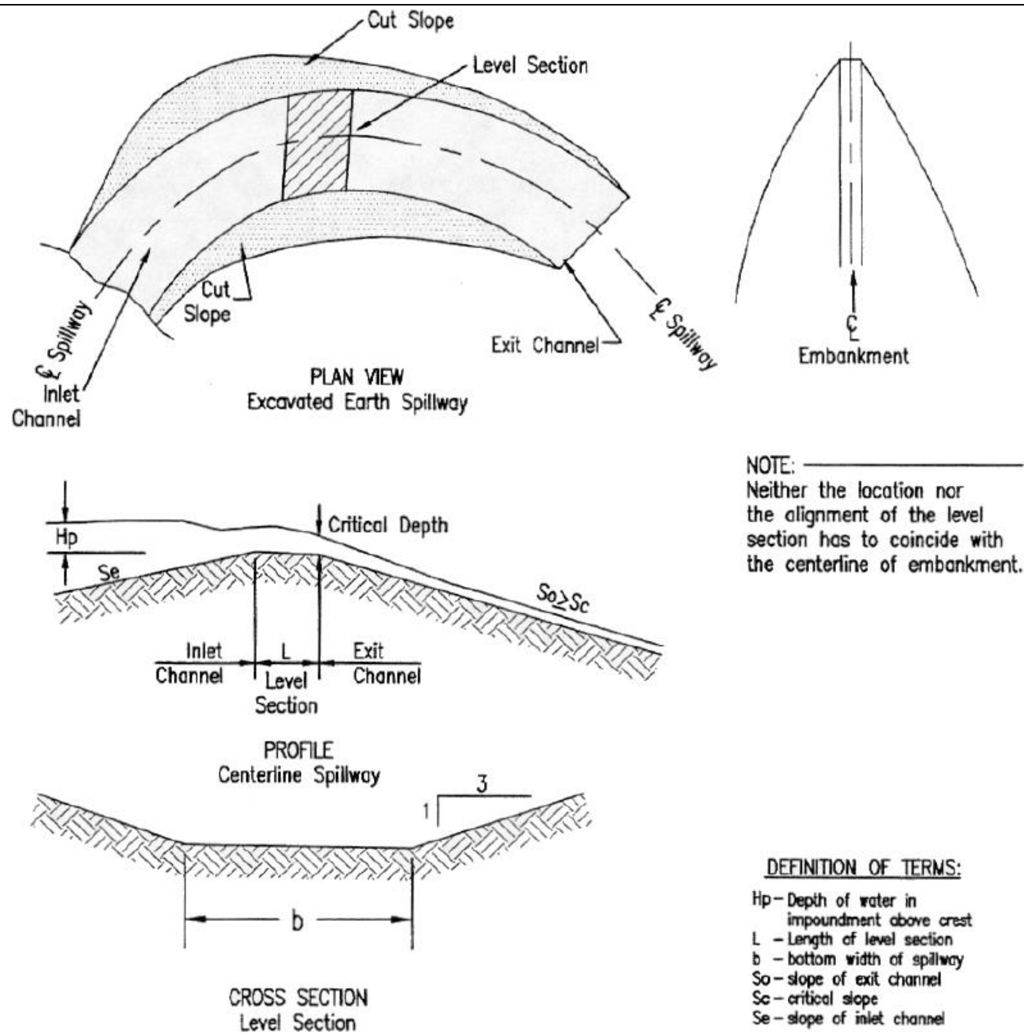



**Figure C3-S12-25: Secondary spillway plan and profile**
Source: VDCR, 1999