# Chapter 5 – Detention

## 5.1 Overview and Policies

Detention is a practice in which storage is provided to temporarily impound stormwater runoff during an event and to release runoff in a controlled manner. A detention volume is created because inflows to the detention facility exceed the controlled outflow rate, and runoff accumulates in the designated detention area until inflow rates decrease and the outflow empties the storage volume in a controlled manner. The typical purpose of a detention facility is to manage runoff to control the effects of peak runoff rates on downstream conveyances and property. For new development, it is common practice to require detention so that peak flow rates for the proposed development do not exceed pre-development, or existing, peak flow rates for design events. In the context of redevelopment or infill development in urban areas, the goal of detention often is to ensure that runoff from the development site does not exceed the capacity of the existing conveyance system or exacerbate existing flooding problems.

Much of the development in Aspen falls into the categories of redevelopment or infill development. Historically development in the commercial core and surrounding areas did not incorporate flood control detention for the minor and major events. Given Aspen's proximity to the Roaring Fork River and the large undeveloped drainage area in the Roaring Fork River watershed above Aspen, detention is not a practice that provides major flood control benefits for the City of Aspen. In addition, channel scour, which may be more pronounced in the absence of detention, has not been reported as a chronic problem in the vicinity of Aspen. However detention is an important practice for runoff from the City and offsite drainage areas draining through the City to be conveyed to the Roaring Fork River without causing significant damage to properties in town.

The City of Aspen's stormwater management goal is to mimic Aspen's natural hydrology to the maximum extent practicable. By requiring treatment of the water quality capture volume, discussed in **Chapter 8 – Water Quality**, it, in effect, provides infiltration or detention for many very small events (on the order of a 6-month to 1-yr event), approximately 80% of the storms in Aspen. For the remaining 20% of storms, the larger events, a small portion of the runoff will be detained in the water quality treatment areas. The remaining runoff must be conveyed to the river safely and in a manner that doesn't increase the flooding potential of downstream properties.

Given these observations, the City's detention policy is as follows, by area:

1. **Commercial core/downtown and sub-urban areas draining to the stormwater system depicted in Figure 1.1 –** These drainage areas are highly impervious and are served by the City of Aspen stormwater infrastructure. In general, the minor, 10-year, event in this area is handled by storm sewers and the major, 100-year, event is accommodated by a combination of storm sewers and street flow. Additionally, detention (above the WQCV) appears to have minimal effects on the system capacity in this area due to the large basins located upland of the City which are not being detained and contribute a large portion of flow to the system. Because the City's system generally has the capacity to accommodate the existing flow volumes and rates and because detention has minimal effects on system capacity, detention is not required for sites that discharge directly to the City's system in this area (see Figure 1.1). For sites that cannot discharge directly to the City's system, detention to the historic rate is required or a project may have the option to pay a fee-in-lieu of providing the required detention (see Section 2.12.140 of the Municipal Code).

2. **Sub-urban area not analyzed or not served by public storm sewer –** These are areas outside of the downtown area and drain to unanalyzed portions of the City's stormwater system, the Roaring Fork River or its tributaries via local drainage systems. Detention to the historic peak flow rates for the 5- and 100-year events is required for new development and redevelopment activities that disturb or add more than 1,000 square feet of impervious area. If the disturbed or

added impervious area is less than 25% of the total site then only the new area must be so treated. If the disturbed or added impervious area is more than 25% of the total site then the site shall be treated as a new development and the total site shall meet detention requirements to the historic peak flow rates. A project may have the option to pay a fee-in-lieu of providing the required detention (see Section 2.12.140 of the Municipal Code).

3. **Private regional drainage systems** – These are areas, such as subdivisions or Metro districts, in which the detention requirements for the entire planned development have already been met. For these areas, detention (above the WQCV) is only required for runoff not planned/designed for in the accepted/approved drainage plan of the subdivision/district. In cases of increased runoff, detention to the pre-developed rate will be required for the design storm used in the original design.

In instances where the City is aware of existing flooding problems and/or undersized conveyances, the City reserves the right to require minor and/or major event detention for any of the areas described above. Over-detention (i.e. providing detention that decreases developed peak flow rates to less than pre-development conditions) may be required in instances where there are downstream system capacity limitations, although this situation will require site specific evaluation by a registered professional engineer.

<u>The above policies relate to detention only, not requirements for the WQCV</u>. See **Chapter 8 – Water Quality** for WQCV requirements that apply to all development and redevelopment in the City.

If there is a need to design detention facility for both flood mitigation and water quality enhancement, this chapter should be used in conjunction with **Chapter 8 – Water Quality** in this Manual. Following the guidelines in this Manual, multipurpose, attractive detention facilities should be developed that are safe, maintainable and viewed as community assets rather than liabilities.

This chapter provides guidance to design stormwater flood control detention facilities. Topics discussed in this chapter include:

- Evaluation of detention requirements,
- Types of detention facilities,
- Design considerations,
- Determination of allowable release,
- Design procedures,
- Methods for calculating detention volume requirements,
- Initial sizing for detention basin design, and
- Preliminary design and final refinement.

## 5.2 Evaluation of Detention Requirements

For detention requirements for a development or redevelopment site in Aspen, there are several conditions that generally must be evaluated for the minor and major events, including the following:

1. **Historic** — Historic conditions are defined as the conditions on a site prior to any development activities (i.e. before construction of any *existing or proposed* buildings or infrastructure). Since development first began in Aspen with mining in the late 1800's, there has been significant impervious area added in the City, up to more than a 90% level in the commercial core, with very little flood control detention. Given the nature and density of the development that currently exists in the City in many areas, detention of runoff to historic conditions is not always practical. Therefore, the policy for the City of Aspen is that redevelopment and add-on projects provide detention to match existing conditions. Historic conditions primarily apply to new developments in the City rather than redevelopment project, although it may also serve as a "benchmark" for redevelopment projects that use over-detention to address existing downstream flooding problems.

This "do no more harm than formerly" approach is a pragmatic approach to detention, realizing many of the space-constraints in the City, but it does have limitations on its effectiveness. In some portions of the City storm sewer and/or street capacity are not sufficient to prevent flooding under existing conditions and flooding problems currently exist. The "do no more harm" approach is not expected to alleviate existing flooding in these areas, but it is not expected to make it worse. For areas that have undersized storm sewers, improvements for routine flooding problems are expected to result from City plans to upgrade all storm sewers in the commercial core to 10-year capacity. The City, at its discretion, may require detention beyond that dictated by existing conditions, including over-detention, to address pressing flooding problems within the City. Site specific analysis by a registered engineer will be necessary in these conditions.

2. **Proposed Conditions**— Proposed conditions include both existing and new imperviousness for a project. In all cases evaluation of existing and proposed peak runoff rates shall include off-site drainage areas that drain through the project site. For evaluating proposed conditions, all off-site areas shall be considered fully developed without detention, unless dedicated approved detention facilities are in plan that can reasonably be relied on. It may not be necessary for a development with an on-site detention facility to provide detention for off-site flows, but detention facilities must be designed to safely convey off-site flows while providing the required detention for on-site flows. For most small sites, the Rational Method will be applied to determine existing and proposed flow rates for the minor and major events. The proposed condition time of concentration shall not be greater than the existing condition time of concentration, unless detailed calculations are submitted to justify the proposed increase.

## 5.3 Types of Detention Basins

The main objective of stormwater detention is to mitigate increased storm runoff volume and reduce the peak flow rates. Detention basins provide temporary storage of stormwater that is released through an outlet at a pre-set release rate. There are two primary types of detention basins that are commonly used:

- Dry detention basins—Dry detention basins are facilities that store water primarily during runoff events. As inflow rates increase the detention basin fills and impounds water as it is released via an outlet at a controlled rate. As inflows subside, the detention basin continues to release the stored water over an extended period of time, eventually draining completely. Dry detention ponds commonly have grassed bottoms, and when designed with appropriate side slopes and layouts may provide multiple functions when dried out including open space, wildlife habitat and in some cases recreational areas. Dry detention ponds may also provide water quality functions - when the water quality capture volume is captured and infiltrated the volume can be "nested" in the minor and major events volumes.

- Bio-retention or sand filter facilities that provide the WQCV can be increased in volume to provide detention for minor and major events. When these two BMPs are utilized the storage may be nested. That is, the WQCV is included within (not in addition to) the volumes captured in the minor or major events.

- Wet detention basins—Wet detention basins, sometimes referred to as retention ponds, differ from dry detention basins in that they include permanent pools that persist during inter-event dry periods. Strictly speaking, "retention" refers to ponds that do not have an outlet but instead hold runoff that is either ultimately evaporated or infiltrated. These types of ponds are not common in Colorado and can carry water rights issues that should be investigated prior to incorporating a wet detention basin into the design. More typically, a wet detention pond (as opposed to a strict "retention" pond) provides detention storage volumes for the minor and major events (and sometimes the WQCV) above the elevation of the permanent pool and releases the surcharge volume via an outlet that regulates flows. Wet detention ponds can be aesthetic amenities, providing a water feature, but there are additional design considerations relative to dry detention ponds. Adequate baseflow to sustain the permanent pool, water rights considerations, management of potential algae, dry weather circulation and mixing, and additional safety considerations are some of the factors that must be considered.

There are a number of other distinctions of types of detention basins that are useful to understand:

- On-site, sub-regional, regional—*On-site* detention ponds typically provide treatment for a single lot or development. They are generally small and privately operated and maintained. They are generally more costly per unit volume of runoff detained than sub-regional or regional facilities because of economies of scale and they typically do not provide multiple functions in addition to peak attenuation due to their small size. *Sub-regional* detention facilities refer to ponds that provide peak attenuation for multiple parcels and are typically larger than on-site facilities. An example of a sub-regional detention facility would be a detention pond in a neighborhood park that provides detention for surrounding residences and businesses. *Regional* detention facilities generally provide treatment for drainage areas in excess of 130 acres (a regionally accepted threshold for differentiating between "local" and "regional drainage"). Regional facilities offer the greatest potential for multiple uses in addition to stormwater detention given their large size. Regional facilities are generally publicly operated and maintained. The Jennie Adair constructed wetlands are an example of a regional water quality facility in Aspen.

- On-line/in-stream, offline/off-stream—On-line or in-stream detention facilities are typically situated along drainageways and provide detention for all upstream contributing areas, on-site and off-site. As a result most on-line facilities are sub-regional or regional facilities. For on-line facilities, it is often necessary to provide on-site water quality treatment to protect the reach of the stream that runs from the site of the individual developments to the regional facility. In addition, given that on-line systems are typically coincident with waterways, wetland permitting is often necessary for construction of such facilities. Off-line or off-stream detention facilities may be adjacent to major drainageways, but usually provide treatment for only a single sub-watershed contributing runoff to the drainageway. Off-line facilities are commonly on-site or sub-regional facilities.

- Above-ground, below-ground—Far and away, the most common types of detention facilities in Colorado and across the country are above-ground facilities. Above-ground facilities have many advantages over underground facilities including easier inspection and maintenance; potential for incorporating multiple functions, including aesthetics, open space, recreation, etc.; ability to drain via gravity versus pumped outfall; lower costs; and other factors. Underground detention is a practice that is disallowed in many municipalities because of historic problems with operation and maintenance and a general "out-of-sight out-of-mind" mentality. Some municipalities that allow underground water quality facilities choose to disallow underground flood control detention—because if an underground water quality facility fails to function, there is potential for impairment of water quality; but if an underground flood control facility fails to function, people may experience flooding and significant damage. Given extremely high real estate values in Aspen, and the need to provide flood control detention for public health, safety and welfare, underground detention is a practice that may be considered by the City and allowed on a case-by-case basis. **It is not a desirable practice, and should be used only as a last resort when the applicant is able to demonstrate that above-ground options are infeasible.** For any underground detention applications, the applicant will be required to develop a rigorous inspection, operations and maintenance plan that must be approved by the City during the design phase.

## 5.4 Design Considerations

Design of a detention system involves an integration of functional integrity, land value, aesthetics, recreation, and safety that merges the system into the urban setting. From the engineering perspective, the design of a stormwater detention basin shall take the following factors into consideration.

### 5.4.1 Location

In an urban area there are many potential multi-use areas where detention potentially can be incorporated including parking lots, parks, sport fields, roadside ditches/culvert crossings, and naturally depressed areas on individual lots. In addition to engineering considerations, selection of a basin site depends on land ownership, cost, public safety, and maintenance. It is important to apply the concept of multiple uses so that detention facilities can provide open space, landscape amenities, habitat and other functions. Coordination with a landscape architect and other related professionals during the design of a detention facility can make the difference between a soggy, unattractive "hole in the ground" and a multi-functional feature that is an amenity to a development.

### 5.4.2 Basic Basin Layout

The basic elements for a detention basin (**Figure 5.1**) include: (1) inlet work at the entrance to collect runoff flows, (2) a *forebay* for sediment settlement, (3) *trickle channels* to pass low flows (not required for wet ponds or wetland basins), (4) a *storage basin* to detain flood water, (5) an *outlet structure* to control the release of water, and (6) a maintenance system.

At the basin entrance, energy dissipaters shall be designed for erosion protection. Inflows first fill up the forebay and then overtop the level spreaders along the top of the forebay and flow into the storage basin. Low flows will be released into the trickle channel that runs through the bottom of the basin to the micro pool in front of the outfall structure. In general, the capacity of a trickle channel is 1.0 to 3.0 % of the 100-year peak discharge, and it shall pass 1/3 to 1/2 of the 2-year event. The longitudinal slope for the trickle channel should be between 0.4 and 1.0 % to assure adequate slope for conveyance of low flows on the mild side and to avoid supercritical flow and/or erosive velocities of the high side. The trickle channel drains into the micro pool for stormwater quality control, or directly releases low flows through the outfall concrete vault. The micropool should be sized in accordance with **Chapter 8 – Water Quality**.

The basin length to width ratio should be between 2:1 (L:W) and 3:1 so that the flood flows can sufficiently expand and diffuse into the water body to enhance the sedimentation process. If this range of length to width ratios is difficult to obtain for a proposed pond, the designer may consider using baffles or berms within the pond to adjust the effective length to width ratio.

Slopes on embankments must be designed to maintain bank stability—for reasons of public safety, maintenance, slope stability and others, slopes on earthen embankments shall not be steeper than 4H:1V and on riprap embankments shall not be steeper than 3H:1V. The geometry of the basin shall be designed for multiple flood events. As shown in **Figure 5.1**, the lower storage volume in a basin is shaped by the 10-year event. The volume from the 10-year water level to the weir crest shall provide additional storage to accommodate the 100-year flood event. The freeboard height is from the weir crest up to the brim-full level of the basin and shall be a minimum of 1.0 foot.

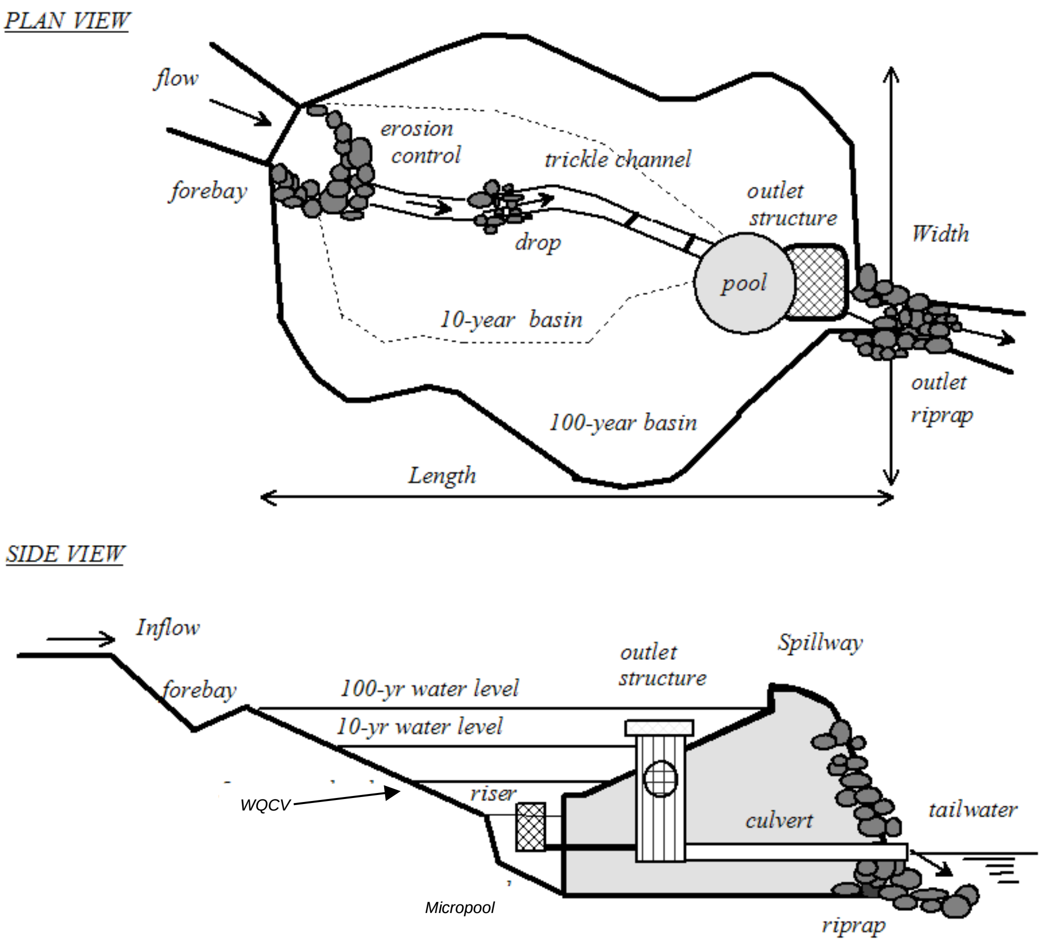


**Figure 5.1 General Layout of a Detention Basin for Multiple Storm Events including WQCV**

(Note: for sub-urban areas not served by public storm sewer, 5-year event is used for minor event rather than 10-year event as shown)

### 5.4.3 Inlet and Outlet Works

The inlet and outlet of a detention basin shall be protected from erosion and deposition of sediments. A level spreader is often used at the transition from the forebay to the main body of the pond to disperse the concentrated inflow into uniform sheet flow into the basin. The level spreader may consist of level concrete curbing designed to overflow uniformly form the forebay to the pond or a level concrete ribbon at the edge of the forebay with a gravel filled trench on the upgradient side (low flows initially percolate into the gravel trench—as the gravel trench fills, excess flows spill out uniformly across the concrete ribbon.

Additional information on level spreaders is provided in **Chapter 8—Water Quality**. Riprap rundowns are generally not recommended for pond inflows because of past problems with erosion of rundowns. A forebay, equipped with a level spreader is a preferable alternative.

The outlet works includes a concrete drop box that has orifices and weirs to collect flows into the box, and an outfall conduit to discharge the allowable release. Orifice and weir coefficients must be selected according to operational requirements. A trash rack is critically important with regard to public safety. The surface area of a trash rack should be no less than four (4) times the orifice opening area.

Detention ponds that have an outlet pipe terminating in the gutter of a street, such as through a chase section, present potential ponding and icing problems in the gutter, and create hazards to the traveling public during periods in which the pond is emptying rapidly. Therefore, detention ponds shall be designed to outlet into a storm sewer, drainageway, or other designated drainage system that is reasonably available, as determined by the City. It must be shown that the storm sewer, drainageway, or other designated drainage system where the pond outlets have the capacity to convey the detention pond flows.

The City may allow an outlet to discharge into the gutter in cases where a storm sewer or other drainage system is not reasonably available when the minor storm (5- or 10-year) peak flow for the tributary area is less than 3.5-cubic feet per second, and it must be shown that the street has adequate capacity to convey the excess runoff within the allowable limits. A transition from the outlet pipe to a curb chase will normally be required and the chase section shall be designed to convey the discharge at a low velocity. The location of the outlet shall be designed to minimize potential problems or conflicts with other improvements, and shall be angled toward the downstream slope of the gutter to direct flows downstream instead of perpendicularly into the street. Discharge into the gutter will not be allowed on local streets.

### 5.4.4 Liners

A storage facility may require an impermeable clay or synthetic liner for a number of reasons. Stormwater detention and retention facilities have the potential to raise the groundwater level in the vicinity of the basin. If the basin is close to structures or other facilities that could be damaged by raising the groundwater level, consideration should be given to lining the basin with an impermeable liner. A liner should be considered in the following circumstances:

1. For detention facilities within ten feet of a building foundation. Even for facilities that are located more than ten feet from a building foundation, a liner may be considered if the facility is upgradient of the foundation, if expansive soils are known to exist, or if there are site specific concerns with the potential for the foundation to be exposed to moisture seeping from the pond. Drainage designers are urged to confer with geotechnical engineers regarding the potential implications of infiltrating stormwater on building foundations.

2. For detention facilities with underlying average seasonal high groundwater within 3 feet of the planned pond bottom or where groundwater levels are determined to be within 3 feet or a nearby building foundation.

3. Areas of existing groundwater or soil contamination.

An impermeable liner may also be warranted in a retention basin where the designer seeks to limit seepage from a permanent pond. Alternatively, there are situations where the designer may seek to encourage seepage of stormwater into the ground. In this situation, a layer of permeable material may be warranted.

### 5.4.5 Groundwater Impacts and Baseflows

When the detention basin operates as a dry pond, it is necessary to assure that the average time interval between two adjacent storm events allows the basin to dry up. On the contrary, if the detention basin is designed to be a wet pond, care must be taken to assess infiltration to and exfiltration from the local groundwater table. It is necessary to carefully evaluate the water budget between the groundwater and surface water, and associated hydrologic losses to ensure that the pond will remain wet.

An important component of the hydrologic budget for a pond in an urban setting can be baseflows generated from irrigation return flows, snowmelt throughout the spring and early summer and other urban water uses. For detention facilities such as wet ponds and constructed wetland basins, baseflows should be characterized on a seasonal basis and factored into the hydrologic budget for the facility.

### 5.4.6 Tailwater Effects

The performance of an outlet is controlled by the headwater in the basin and the tailwater in the downstream water receiving system (such as the water surface elevation in the storm sewer, stream or lake). It is important to assess the downstream tailwater conditions when estimating the release capacity from the basin under design.

### 5.4.7 Maintenance Access

All-weather, stable access to the bottom, inflow, forebay, and outlet works areas shall be provided for maintenance vehicles. Maximum grades should be 10 percent, and a solid driving surface of gravel, rock, concrete, or gravel-stabilized turf should be provided.

### 5.4.8 Retaining Walls

The use of retaining walls within detention basins is generally discouraged due to the potential increase in long-term maintenance costs and concerns regarding the safety of the general public and maintenance personnel. If retaining walls are proposed, footings shall be located above the WQCV. Wall heights not exceeding 30-inches are preferred, and walls shall not be used on more than 50-percent of the pond circumference. If terracing of retaining walls is proposed, adequate horizontal separation shall be provided between adjacent walls. The horizontal separation shall ensure that each wall is loaded by the adjacent soil, based on conservative assumptions regarding the angle of repose. Separation shall consider the proposed anchoring system and equipment and space that would be needed to repair the wall in the event of a failure. The failure and repair of any wall shall not impact or affect loading on adjacent walls. In no case shall the separation be less than 2 times the adjacent wall height, such that a plane extended through the bottom of adjacent walls shall not be steeper than 2 (horizontal) to 1 (vertical). The maximum ground slope between adjacent walls shall be 4-percent.

Walls shall not be used where live loading or additional surcharge from maintenance equipment or vehicle traffic could occur. The horizontal distance between the top of a retaining wall and any adjacent sidewalk, roadway, or structure shall be at least three times the height of the wall and may not be used for parking or as a driveway.

### 5.4.9 Other Considerations

Operation of a stormwater detention system may also involve many institutional issues, including the infrastructure needed to ensure proper planning, design, construction, operation, and maintenance. A monitoring or regulatory mechanism is required to ensure that the approved design is constructed, operational integrity is implemented, and maintenance is regularly provided.

Other considerations include public safety, facility access, landscaping, and aesthetics. Locating detention basins in areas reserved to meet site landscaping requirements is generally encouraged. Incorporating detention into landscaped areas generally creates detention facilities which are easy to inspect, are relatively easy to maintain, and can enhance the overall aesthetics of a site.

## 5.5 Design Procedure

The City of Aspen recommends that a detention basin be designed with two levels of release controls (minor [5- or 10-year] and major [100-year] events), in addition to the water quality capture volume (WQCV) control (if WQCV is being provided in detention basin). Although the detention basin is shaped to accommodate the minor and major storms, the operation of a detention basin needs to accommodate events of all kinds, including events that are greater than the 100-year event. **Figure 5.2** outlines the design steps beginning with basin site selection. During the initial design stage, little information is available. Therefore, it is suggested that the basin geometry be approximated by a triangular, rectangular or circular shape, and the basin operation be approximated by the inlet control capacity determined by weir and orifice hydraulics only. Of course, when the project moves to the final design, the preliminary design will be refined with more information. For instance, the tailwater and backwater effects must be considered to refine the basin characteristic curves, and the basin performance must be evaluated using hydrologic routing techniques. The below procedure will be an iterative process until all design criteria and safety concerns are satisfied.

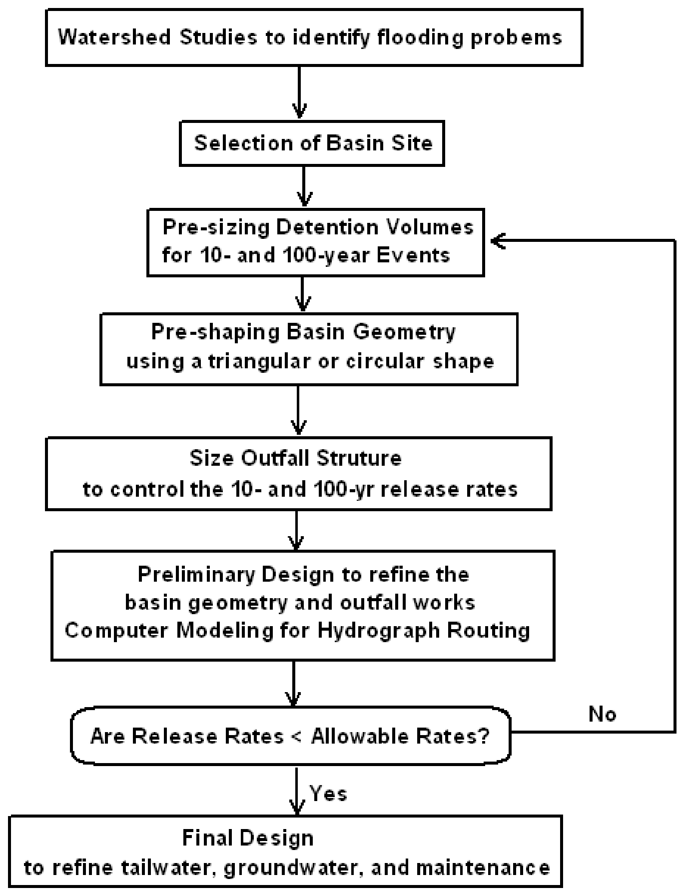


**Figure 5.2 Flow Chart for Detention Basin Design**

## 5.6 Initial Design Detention Volumes

### 5.6.1 Rational Volume Method for Detention Volume (Watershed <90 acres)

The Federal Aviation Administration (FAA) procedure was modified to provide a reasonable estimate of the required storage volume for small on-site detention facilities (FAA 1996, Guo 1999a). This method is a rational-formula based approach that is only applicable to small urban watersheds less than 90 acres. In this approach, the design rainfall duration is unknown. The engineer shall investigate the required detention volumes for a range of rainfall durations, starting from the time of concentration, $T_c$, until the volume is maximized. The computational steps are as follows:

1. Select a design event, 10 or 100-year.

2. Select rainfall duration, $T_d$, ≥ $T_c$. For this method, multiple event durations are chosen until the storage volume is maximized. The design duration should be chosen to be greater than or equal to the time of concentration.

3. Calculate design rainfall intensity using the rainfall Intensity-Duration-Frequency (IDF) curve (**Chapter 2 - Rainfall**) or formula:

$$I = \frac{29 P_1}{(10 + T_d)^{0.789}}$$

**(Equation 5-1)**

Where $I$ = rainfall intensity in inch/hour, $T_d$ = duration in minutes, and $P_1$ = one-hour rainfall depth in inches for the design frequency (see **Chapter 2 – Rainfall**).

4. Calculation of inflow volume:

$$V_i = \frac{1}{720} C\ I\ T_d\ A$$

**(Equation 5-2)**

Where $V_i$ = inflow volume in $ft^3$, C = runoff coefficient, and A = tributary area in $ft^2$.

5. Calculation of outflow volume, $V_o$:

$$V_o = 30(1 + \frac{T_c}{T_d}) Q_a T_d$$

**(Equation 5-3)**

Where V = outflow volume in $ft^3$, $T_c$ = time of concentration in min, $T_d$ = rainfall duration in min, and $Q_a$ = allowable release rate (peak flow or runoff from the design storm, **Equation 3-1**) in cfs.

6. Volume difference, $V_d$, is calculated as:

$V_d = V_i - V_o$ for the selected rainfall duration. **(Equation 5-4)**

Repeat Steps 2 thorough 6 for another rainfall duration until the volume difference is maximized. The design detention volume, S, is determined as the maximum storage volume determined from evaluating multiple rainfall durations. This method is sometimes referred to as the “bow string” method because the shape of the curve of storage versus event duration is shaped like a bow—the bow arcs like a parabola with the required storage volume increasing initially as the duration of rainfall increases beyond the time of concentration, eventually reaching a maximum value and then decreasing again as the duration increases beyond the critical design duration. This is illustrated in the example below in **Section 5.6.2.**

It is noted that the design rainfall duration for detention volume sizing is always longer than the time of concentration of the watershed.

### 5.6.2 Design Example for Rational Volume Method

A commercial development has a total tributary area of 2 acres. Under the post-development imperviousness of 80%, the runoff coefficient for the 100-year event is 0.86 and the time of concentration for the project site is calculated to be 12 minutes. The 100-year 1-hour precipitation depth is 1.69 inches. The allowable release rate is already determined to be 5.0 cfs for the 100-year event. For rainfall duration of 15 minutes, the required detention volume is calculated as:

$$I_d = \frac{29 \times P_1}{(10 + T_d)^{0.79}} = \frac{29 \times 1.69}{(10 + 15)^{0.79}} = 3.87 \text{ inch/hour}$$

$$V_i = \frac{1}{720} C\ I\ T_d\ A = \frac{1}{720} \times 0.86 \times 3.87 \times 15 \times (2 \times 43560) = 6035 \text{ cubic feet}$$

$$V_o = 30(1 + \frac{T_c}{T_d}) Q_a T_d = 30(1 + \frac{12}{15}) \times 5.0 \times 15 = 4050 \text{ cubic ft}$$

$$V_d = V_i - V_o = 6035 - 4050 = 1985 \text{ cubic ft}$$

Repeat this above procedure as shown in **Table 5.1**. The 100-year detention volume for this example is found to be 2180 cubic feet under rainfall duration of 21 minutes. It is imperative to continue these calculations with increasing durations until the storage volume is maximized and begins to decrease. See **Figure 5.3** for a graphical depiction of the "bow string."

**Table 5.1 Detention Volume by Rational Volume Method**

| **Inputs** | | | | |
|---|---|---|---|---|
| Area = | | 2.0 | acres | |
| tc = | | 12 | min | |
| P1 = | | 1.69 | in | |
| Release = | | 5.0 | cfs | |
| C = | | 0.86 | | |
| | | | | |
| **Calculations** | | | | |
| Duration | Rainfall Intensity | Inflow Volume | Outflow Volume | Storage Volume |
| (min) | (in/hr) | ($ft^3$) | ($ft^3$) | ($ft^3$) |
| 15 | 3.87 | 6035 | 4050 | 1985 |
| 18 | 3.54 | 6623 | 4500 | 2123 |
| 21 | 3.26 | 7130 | 4950 | 2180 |
| 24 | 3.03 | 7576 | 5400 | 2176 |
| 27 | 2.84 | 7973 | 5850 | 2123 |

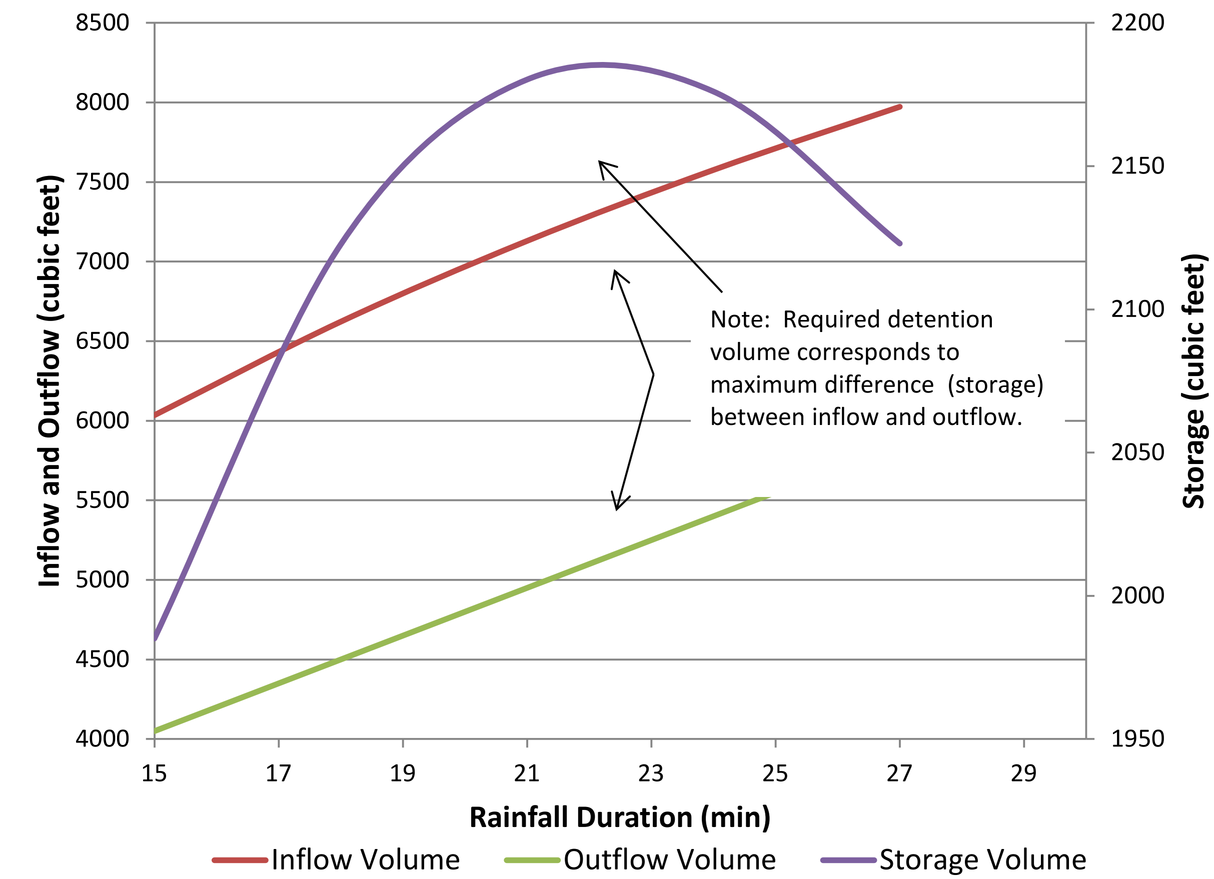


**Figure 5.3 Detention Volume Example "Bow String"**

### 5.6.3 Hydrograph Method for Detention Volume

The detention volume can be calculated using the hydrograph method as the difference between the inflow and outflow hydrographs. This method is applicable to all sizes of watershed as long as the inflow hydrograph is readily available. The inflow hydrograph to a detention basin is often generated from the developed watershed condition using CUHP or SWMM computer programs. As illustrated in **Figure 5.4**, the detention volume is the volume difference between the inflow and outflow hydrographs from the beginning of the event to the time when the allowable release occurs on the recession hydrograph. At the planning stage, the outlet hydraulics have not yet been developed. For convenience, the outflow hydrograph is approximated by a triangular shape with its peak flow equal to the allowable release (Malcom 1982, Guo 1999b). As shown in **Figure 5.4**, the outflow rate, O(t), at time t on the linear rising limb is estimated as:

$$O(t) = \frac{Q_a}{T_P} t \quad \text{for } 0 \le t \le T_P \qquad \textbf{(Equation 5-5)}$$

in which,

$O(t)$ = linear outflow rate in cfs,
$Q_a$ = allowable release in cfs,
$T_p$ = time to peak in minutes on outflow hydrograph, and
$t$ = elapsed time in minutes.

If the calculated outflow rate is greater than the inflow rate, the outflow rate should be set equal to the inflow rate.

The detention volume is the sum of the volume difference between the inflow hydrograph and the rising limb of the outflow hydrograph as:

$$S = \sum_{t=0}^{t=T_P} [I(t) - O(t)] \Delta t \times 60 \qquad \textbf{(Equation 5-6)}$$

in which

$S$ = design detention volume in $ft^3$,
$I(t)$ = inflow rate in cfs at time $t$, and
$\Delta t$ = time increment such as five (5) or ten (10) minutes.

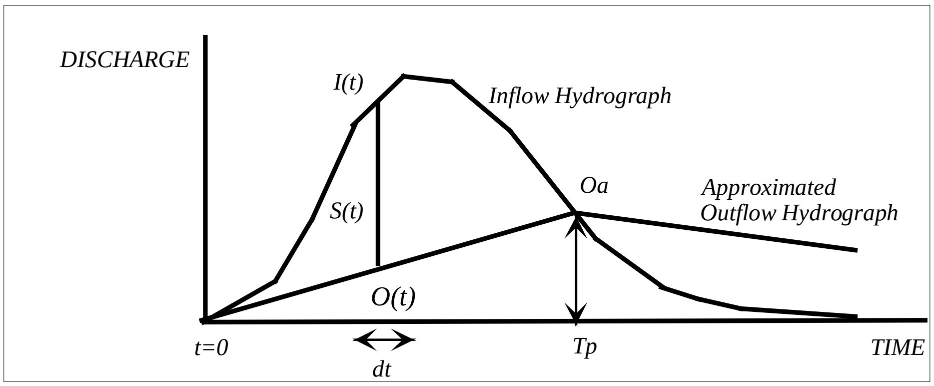


**Figure 5.4 Detention Volume by Hydrograph Method**

### 5.6.4 Example for Hydrograph Method for Detention Volume

As shown in **Table 5.2**, the 100-year inflow hydrograph for the 2-acre commercial development was generated by the CUHP05 computer model. The 100-year peak flow is calculated to be 7.9 cfs. The 100-year allowable release from the watershed is set to be 5 cfs at $t$ = 30 minutes. Under the assumption of a linear rising outflow hydrograph, the linear outflow can be calculated as follows:

$$O(t) = \frac{5}{30} t$$

Where the calculated outflow is greater than the inflow listed in **Table 5.2**, the outflow is assumed equal to the inflow.

Using five (5) minutes as the time increment, the accumulative storage volume, $S(t)$ at time $t$, is computed as:

$$S(t) = \sum_{t=0}^{t=30} [I(t) - \frac{5}{30} t] \times (5 \times 60)$$

Incremental volume is calculated as the inflow minus the outflow rate times the time increment (300 seconds) for each time increment.

As shown in **Table 5.2**, the linear outflow column and the cumulative volume column provide the paired volume and outflow values, as the approximate basin's storage-outflow curve. At $T_p$= 30, the design detention volume is found to be 2130 cubic ft which is similar to the Rational Volume Method in **Section 5.6.2**.

**Table 5.2 Preliminary Storage Volume-Outflow Curve by Hydrograph Method**

| **Input** | | | | |
|---|---|---|---|---|
| Inflow hydrograph is given from SWMM | | | | |
| Maximum outflow rate | | 5 | cfs | |
| Time of maximum outflow | | 30 | minutes | |
| **Calculations** | | | | |
| Time | 100-year Inflow I(t) | Linear Outflow O(t) | Incremental Volume | Cumulative Volume S(t) |
| (min) | (cfs) | (cfs) | ($ft^3$) | ($ft^3$) |
| 0 | 0.0 | 0.00 | 0 | 0 |
| 5 | 0.0 | 0.00 | 0 | 0 |
| 10 | 0.6 | 0.60 | 0 | 0 |
| 15 | 2.1 | 2.10 | 0 | 0 |
| 20 | 5.0 | 3.33 | 500 | 500 |
| 25 | 7.9 | 4.17 | 1120 | 1620 |
| 30 | 6.7 | 5.00 | 510 | 2130 |
| 35 | 4.5 | 4.50 | 0 | 2130 |

## 5.7 Initial Shaping of a Storage Basin

The initial shaping of a storage basin provides a starting point for defining the stage-storage relationship. The stage-storage relationship will be refined during the preliminary and final design phases of the project. The initial shaping can be approximated using a regular geometry such as a triangular basin. For instance, the base area of a triangular basin in **Figure 5.5** is calculated by the base width, *B*, and length, *L*, as:

$$A_1 = 0.5BL$$ **(Equation 5-7)**

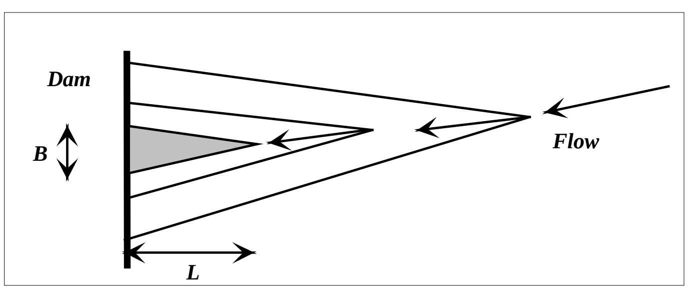


**Figure 5.5 Pre-shaping for a Triangular Basin**

Assuming that the width and length of the triangular cross section increase uniformly with depth, the top cross area is:

$$A_2 = \frac{1}{2}(B + 2zh)(L + 2zh)$$ **(Equation 5-8)**

Where

$A_1$ = lower base area,
$A_2$= upper area,
$h$ = vertical spacing between two sections, and
$z$ = average side slope between these two adjacent layers.

The volume, $V$, between these two triangular layers is approximated as:

$$V \cong \frac{1}{2}(A_1 + A_2)h$$ **(Equation 5-9)**

The required side slope, no steeper than 4H:1V, between these two adjacent layers can be incorporated into the volume calculation. Using the initial sizing and shaping procedure, the approximate *stage-storage* and *stage-contour area* can then be established. Upon completion of the initial design, the engineer can begin to work on the topography at the basin site to refine the basin geometry. The City encourages designers to collaborate with landscape architects to develop storage facilities that are attractive visually, fit into the fabric of the landscape, and enhance the overall character of the neighboring area.

## 5.8 Initial Outlet Works

Outlet works are the structures that control the depth of water and release rates from storage facilities. The outlet system for a basin must be designed with a full understanding of the downstream tailwater effects. The performance of the outfall culverts must be evaluated for a range of headwater depths at the entrance, and tailwater depths at the exit. **Figures 5.6 and 5.7** show typical designs that may be used. These designs incorporate the water quality capture volume (WQCV), and include a perforated (or orifice) plate to release the WQCV. The minor storm release is controlled by the size of the orifice at the bottom of the drop box. The trash rack at the top of the drop box shall have a surface area at least four (4) times the orifice opening area. The outfall culvert at the bottom of a second drop box should be sized to convey 120% of the intended design flow so it does not act as a constriction.  A minimum of 1-foot of freeboard should be provided above the design water surface elevation.

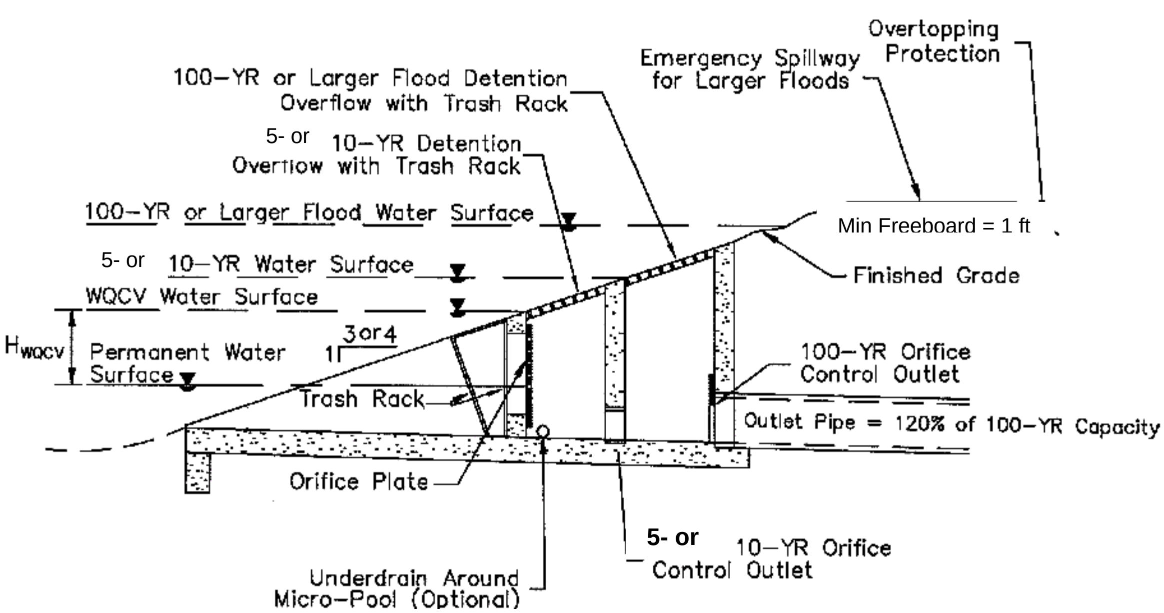

**Figure 5.6 Drop Box Outlet with Conduit to Release the 100-year Flow (5- or 10-year Minor Event)**

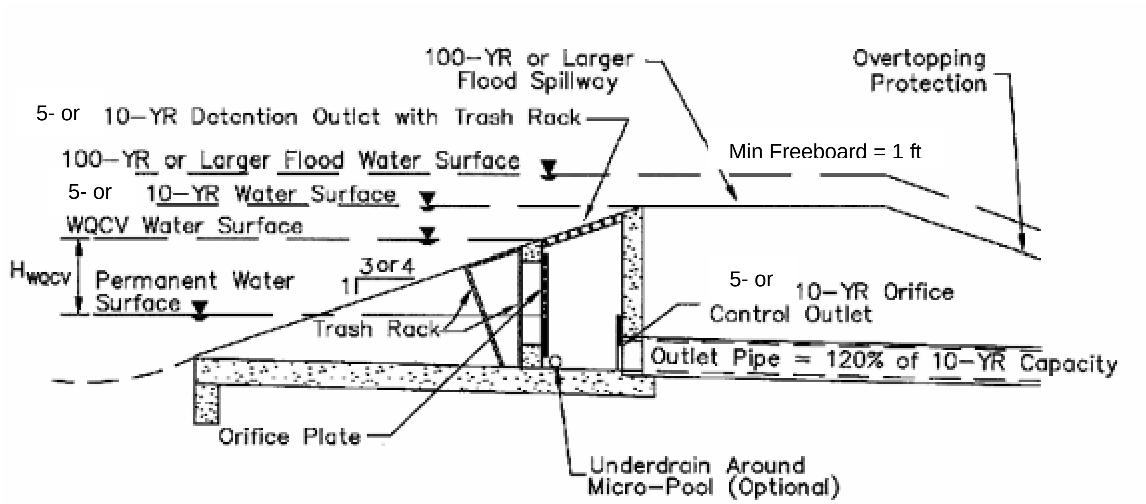


**Figure 5.7 Drop Box Outlet with Overtopping Spillway to Release 100-year Flow (5- or 10-year Minor Event)**

The hydraulic capacities of the various components of the outlet works (orifices, weirs, pipes) can be determined using standard hydraulic equations (Brown et. al. in 1996, King and Brater in 1976). The discharge pipe of the outlet works functions as a culvert. A rating curve for the entire outlet can be developed by combining the rating curves developed for each of the components of the outlet and then selecting the most restrictive element that controls a given stage for determining the composite total outlet rating curve (Stahre and Urbonas, 1990).

Note that when detention storage volumes are very small, an outlet similar to what is illustrated in **Figures 5.6 and 5.7** may not be practical or necessary. For very small detention volumes, the designer should refer to the guidance provided for bioretention facilities in **Chapter 8 – Water Quality**.

In addition to the outlet designs depicted above, there are a number of variations that may be successfully applied in Aspen. The designer is encouraged to review other criteria manuals including the UDFCD Urban Storm Drainage Criteria Manual and design details from Douglas County and Arapahoe County, Colorado that provide details of self-contained micropools for small sites that may be applicable in Aspen.

### 5.8.1 Orifices

Multiple orifices may be used to collect water flow into the drop box. The flow capacity of each orifice can be superimposed to develop the total flow. For a single orifice as illustrated in **Figure 5.8**, orifice flow can be determined as:

$$Q_o = C_o A_o \sqrt{2gH_o} \qquad \textbf{(Equation 5-10)}$$

in which:

$Q_o$ = orifice flow rate (cfs),
$C_o$ = discharge coefficient,
$A_o$ = opening area of orifice (ft$^2$),
$H_o$ = effective headwater depth (ft), and

$g$ = gravitational acceleration (32.2 ft/sec$^2$).

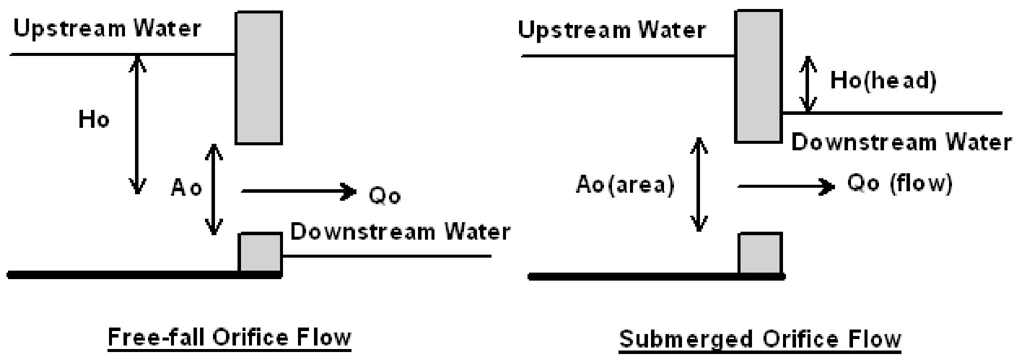


**Figure 5.8 Orifice Flow**

If the orifice discharges as a free-fall outfall, the effective head is measured from the upstream water surface elevation to the centroid of the orifice opening. If the orifice is submerged, the effective head is the difference in elevation between the upstream and downstream water surfaces. A discharge coefficient of 0.6 is recommended 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, a value of 0.4 should be used (ASCE and WEF, 1992). Volume 3 of the Denver Urban Storm Drainage Criteria Manual provides additional information on orifice sizing, spacing of multiple orifices, acceptable materials and calculations. The designer should refer to information on Extended Dry Detention Basins in Volume 3 for additional information and guidance.

### 5.8.2 Weirs

Typical sharp-crested and broad-crested weirs are illustrated in **Figure 5.9**. The general formula for weir flow is described as:

$$Q_w = C_w (L_w - 0.1NH_w) H_w^{1.5}$$ **(Equation 5-11)**

$$C_w = 3.27 + 0.4 \frac{H_w}{H_c}$$ **(Equation 5-12)**

in which

$Q_w$ = weir flow (cfs),
$Cw$ = weir coefficient,
$L_w$ = horizontal weir length (ft),
$N$ = number of end contraction,
$H_c$ = height of weir crest (ft), and
$H_w$ = headwater depth above the weir crest (ft).

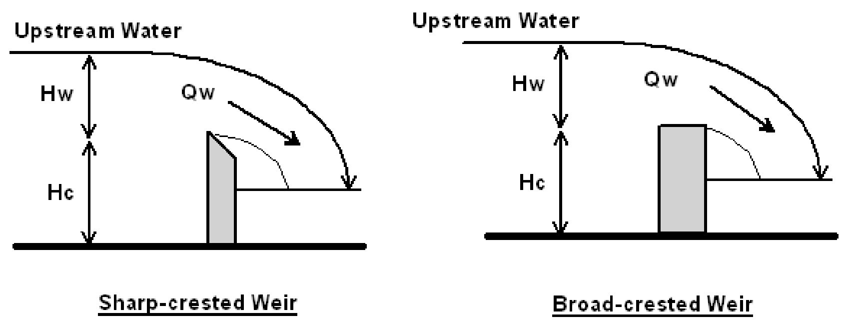


**Figure 5.9 Sharp and Broad-crested Weirs**

In practice, *N* = 0, 1, or 2, depending on the weir geometry. Without details of the weir layout, it is acceptable to assume that *N* = 0. **Equation 5-11** is also applicable to broad-crested orifices when *N* = 0 and $C_w$ = 3.0. The discharge through a V-notch weir is shown in **Figure 5.10**. The stage-flow relationship for a V-notch weir is a function of the central angle and headwater depth as:

$$Q_V = 1.38\tan(\frac{\theta}{2})H_v^{2.5}$$ **(Equation 5-13)**

in which

$Q_v$ = discharge (cfs),
ɵ□= angle of V-notch in degrees, *and*
$H_v$ = headwater depth above the apex of the V-notch (ft).

A trapezoidal weir can be formed by a rectangular weir and two half-notch weirs. The flow through a trapezoidal weir is estimated as the sum of these two individual weirs.

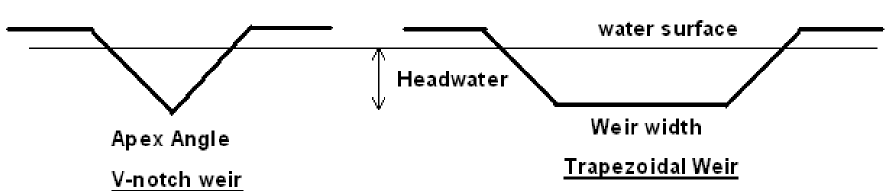


**Figure 5.10 V-notch Weir and Trapezoidal Weir**

### 5.8.3 Culverts

Water enters the concrete drop box in **Figure 5.11** from the orifices and weirs. The outfall pipes are usually short enough, 200 to 300 feet, to act like a culvert. Outflow culverts should have a minimum diameter of 15 inches. The operation of the culvert is affected by the tailwater depth. When the culvert is under a full flow condition, the energy balance between Sections 1 and 2 is written as:

$$H = Y + LS_o = (K_e + K_x + K_b + K_n)\frac{V_c^2}{2g} + \frac{V_c^2}{2g}$$ **(Equation 5-14)**

$$K_n = 184.1\frac{N^2 L}{D^{\frac{4}{3}}}$$ for a circular pipe **(Equation 5-15)**

$$K_n = 29.0\frac{N^2 L}{R^{\frac{4}{3}}}$$ for a non-circular pipe **(Equation 5-16)**

in which

- $H$ = water surface elevation at the entrance in ft,
- $Y$ = headwater depth in ft at the entrance,
- $L$ = length of the pipe in ft,
- $S_o$ = pipe slope in ft/ft,
- $D$ = diameter or height of barrel,
- $K_e$ = entrance loss coefficient (0.2 to 0.5),
- $K_x$ = exit loss coefficient ( 0.5 to 1.0),
- $K_b$ = bend loss coefficient as shown in **Table 5.3,**
- $K_n$ = friction coefficient,
- $V_c$ = flow velocity,
- $N$ = Manning's roughness coefficient (0.015 for concrete pipe),
- $d$ = tailwater depth, and
- $R$ = hydraulic radius.

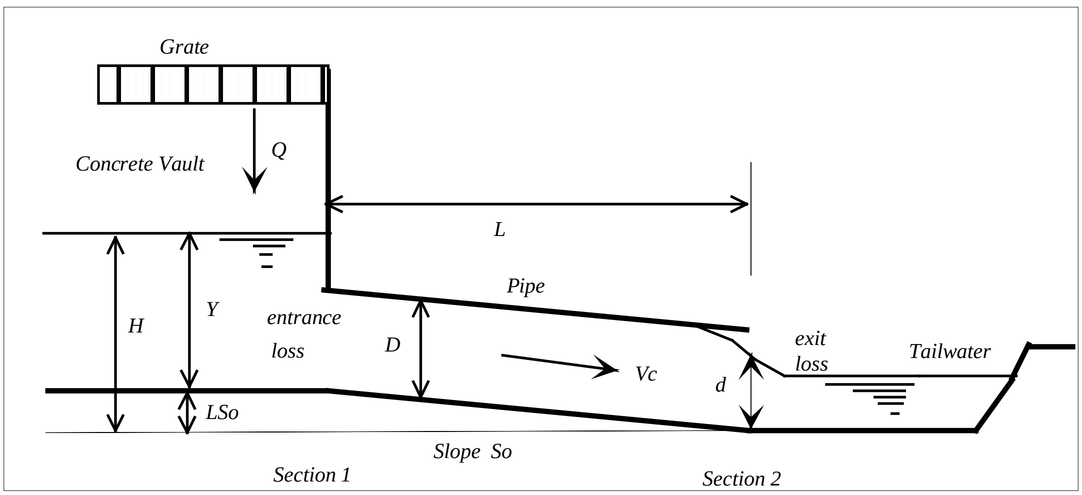


**Figure 5.11 Outlet Culvert Hydraulics**

If the tailwater depth is not known, it is suggested that the tailwater depth be estimated by the average of the culvert diameter and critical depth, $d_c$, as:

$$d = \frac{D + d_c}{2}$$ **(Equation 5-17)**

Let $K$ be the sum of all the loss coefficients as:

$$K = K_e + K_x + K_b + K_n$$ **(Equation 5-18)**

The release capacity of the pipe, $Q_P$, is

$$Q_P = V_c A_c = A_c\sqrt{\frac{1}{K+1}}\sqrt{2g\left(Y + LS_o - d\right)}$$ **(Equation 5-19)**

in which $A_c$ = pipe cross sectional area. **Equation 5-19** is similar to the orifice equation except that the orifice coefficient is calculated using the sum of all loss coefficients.

**Table 5.3 Bend Loss Coefficients**

| Bend Angle in Degrees | Bend Loss Coefficient |
|---|---|
| | 0.050 |
| 22.500 | 0.100 |
| 45.000 | 0.400 |
| 60.000 | 0.640 |
| 90.000 | 1.320 |

### 5.8.4 Spillway Sizing

The overflow spillway of a storage facility should be designed to pass flows in excess of the design flow of the outlet works. When the storage facility falls under the jurisdiction of the Colorado State Engineer's Office (SEO), such as the Dam Safety requirements, the spillway's design storm is prescribed by the SEO (SEO 1988). If the storage facility is not a jurisdictional structure, the size of the spillway design storm should be based upon the risk and consequences of a facility failure.

## 5.9 Preliminary Design

The preliminary design stage consists of refining the design of the basin (size, shape and elevation) and outlet structure (type, size, geometry). During preliminary design, it is necessary to address such subjects as public safety, appearance, water quality, access, maintenance, multipurpose benefits and other concerns of this kind. This is an iterative procedure that can use a variety of reservoir routing schemes to determine the detention basin's performance under the given stage-storage-discharge characteristics. The stage-storage-discharge characteristics are modified as needed until the outflow from the basin meets the specified allowable flow rates. No description of the theory of reservoir routing is provided in this Manual. This subject is described well in many hydrology reference books (e.g., Viessman and Lewis 1996; Guo 1999b). The computer design tool, UD POND, is available at www.UDFCD.org. It provides a reliable and relatively easy tool to facilitate detention basin design.

## 5.10 Final Design

The final design of the storage facility entails detailed hydraulic, structural, geotechnical, and civil design. This includes detailed grading of the site, embankment design, spillway design, outlet works hydraulic and structural design and detailing, trash rack design, consideration of sedimentation and erosion potential within and downstream of the facility, liner design (if needed), etc. For applications where pumping will be required (underground detention for example), the designer may need to collaborate with a mechanical or electrical engineer. Collaboration among geotechnical engineers, structural engineers, hydrologic and hydraulic engineers, land planners, landscape architects, biologists, and/or other disciplines is encouraged during the final design, so that attractive and safe multipurpose facilities are constructed that become community assets rather than eyesores.

## 5.11 References

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