Chapter 12
Storage

Contents

Tables

Figures

1.0 Overview

Photograph 12-1. Detention facilities can be designed to integrate the management of both stormwater quality and quantity.

Detention storage facilities manage stormwater quantity by attenuating peak flows during flood events. Depending on the design, they can also enhance stormwater quality by incorporating design components to promote sedimentation, infiltration, and biological uptake. This chapter provides guidance for the analysis and design of storage facilities implemented independently or in combination with stormwater quality facilities. Specific design guidance for stormwater quality facilities (e.g., extended detention basins, wetland basins, sand filters, etc.) are in Volume 3 of the USDCM.

Other topics discussed in this chapter include:

- Regional, sub-regional, and onsite detention facilities,
- Full spectrum detention,
- Basin sizing methodology,
- Outlet structures and safety grates,
- Emergency spillways,
- Landscape considerations,
- Designing for maintenance; and
- Parking lot detention.

Photograph 12-2. Detention facilities can become attractive amenities and have potential to increase property values in commercial and residential settings, especially with the assistance of experienced landscape architects.

UDFCD strongly encourages the development of multipurpose, attractive detention facilities that are safe, maintainable and viewed as community assets rather than liabilities.

2.0 Implementation of Regional, Sub-regional, and On-site Detention Facilities

Colorado law requires detention be provided to control the 100-year peak flow for all new development in the unincorporated portions of all counties, and most incorporated municipalities in Colorado require the same. There are three basic approaches for locating storage facilities in relation to their upstream watersheds. These are:

- Regional Detention
- Subregional Detention
- Onsite Detention

These three approaches are described in the following sections.

2.1 Regional Detention

Regional detention basins serve multiple property owners in watershed areas ranging from about 130 acres to one square mile. Figure 12-1 provides an example configuration for an on-line regional detention approach.

In some cases, regional detention is effective for watershed areas larger than one square mile and for multiple facilities arranged in series; however, due to the complexities associated with how they function within a watershed, these configurations must be modeled and approved in the context of a formal master planning process.

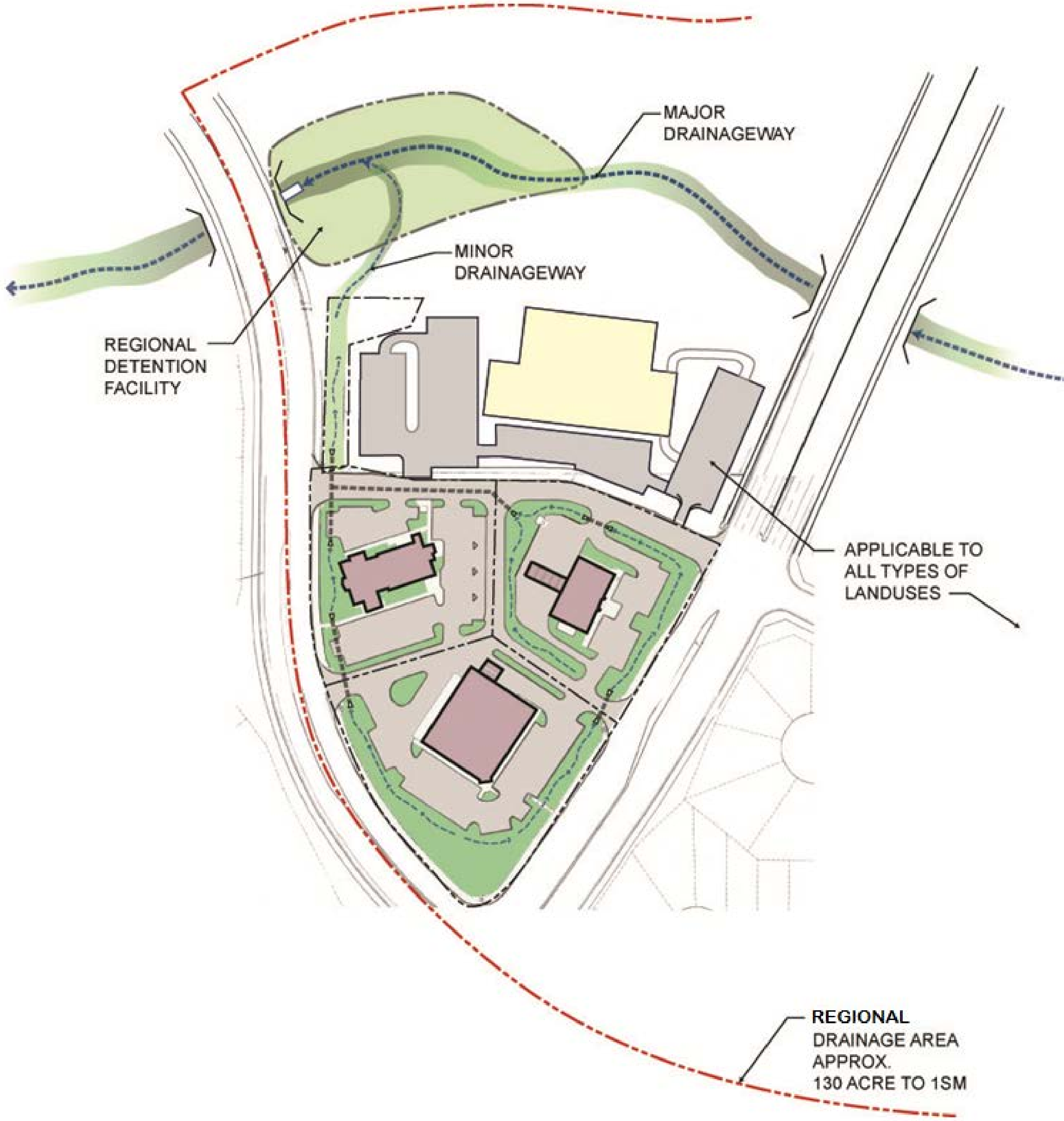


Figure 12-1. Example configuration for regional detention (Source: Arapahoe County)

Regional detention facilities may be constructed by a public entity such as a municipality, special district, or property owner but should always be based on a master plan or a detailed hydrologic model approved by the local jurisdiction that accounts for future development upstream and impacts downstream of the facility.

Compared to on-site facilities, regional detention facilities typically require proportionally less total land area and are more cost effective to construct and maintain. Well-designed regional facilities may also provide more favorable riparian habitat and offer greater opportunities for achieving multi-use objectives, such as combining with park and open space resources and connecting shared use paths.

There are limitations associated with the implementation of on-line regional detention facilities. To avoid excessive accumulation of sediment, it is not recommended that regional detention facilities be constructed on streams experiencing significant upstream bed or bank erosion unless stabilization improvements are constructed ahead of the basin.

When an on-line regional facility is designed to provide water quality, storm water best management practices (BMPs) are still required in the tributary watershed to address water quality and channel stability for the reach upstream of the regional facility. In accordance with MS4 permits and regulations, areas of "New Development and Significant Redevelopment" must be treated with BMPs prior to discharging to a State Water. See Chapter 1 of Volume 3 of the USDCM for additional information when incorporating water quality into a regional facility.

2.2 Subregional Detention

Subregional detention generally refers to facilities that serve multiple landowners or lots and have a total watershed of less than 130 acres. Figure 12-2 illustrates a typical sub-regional detention approach in a commercial area. Most detention facilities located within residential communities are subregional in that they serve multiple lots that are each individually owned. Subregional detention facilities are located off-line from the receiving stream.

Like regional facilities, subregional detention facilities may be constructed by a public entity such as a municipality or special district to serve several landowners in the upstream drainage area, but are more typically designed and constructed by a single developer to serve an area being developed.

Subregional detention offers many of the same benefits as regional facilities in comparison to onsite detention, and is also subject to the same limitations, described in Section 2.1.

Figure 12-2. Example configuration for subregional detention (Source: Arapahoe County)

2.3 Onsite Detention

Onsite detention refers to facilities serving one lot, generally commercial or industrial sites draining areas less than 20 to 30 acres. Figure 12-3 illustrates a typical on-site detention approach.

On-site facilities are usually designed to control runoff from a specific land development site and are not typically located or designed to effectively reduce downstream flood peaks along the receiving stream. The volume of runoff detained in the individual on-site facility is relatively small and, their effectiveness in aggregate has been shown to diminish along the downstream reaches of streams. The application of consistent design and implementation criteria and assurance of their continued maintenance and existence is of paramount importance if large numbers of on-site detention facilities are to be effective in controlling peak flow rates on a watershed scale (Glidden 1981; Urbonas and Glidden 1983).

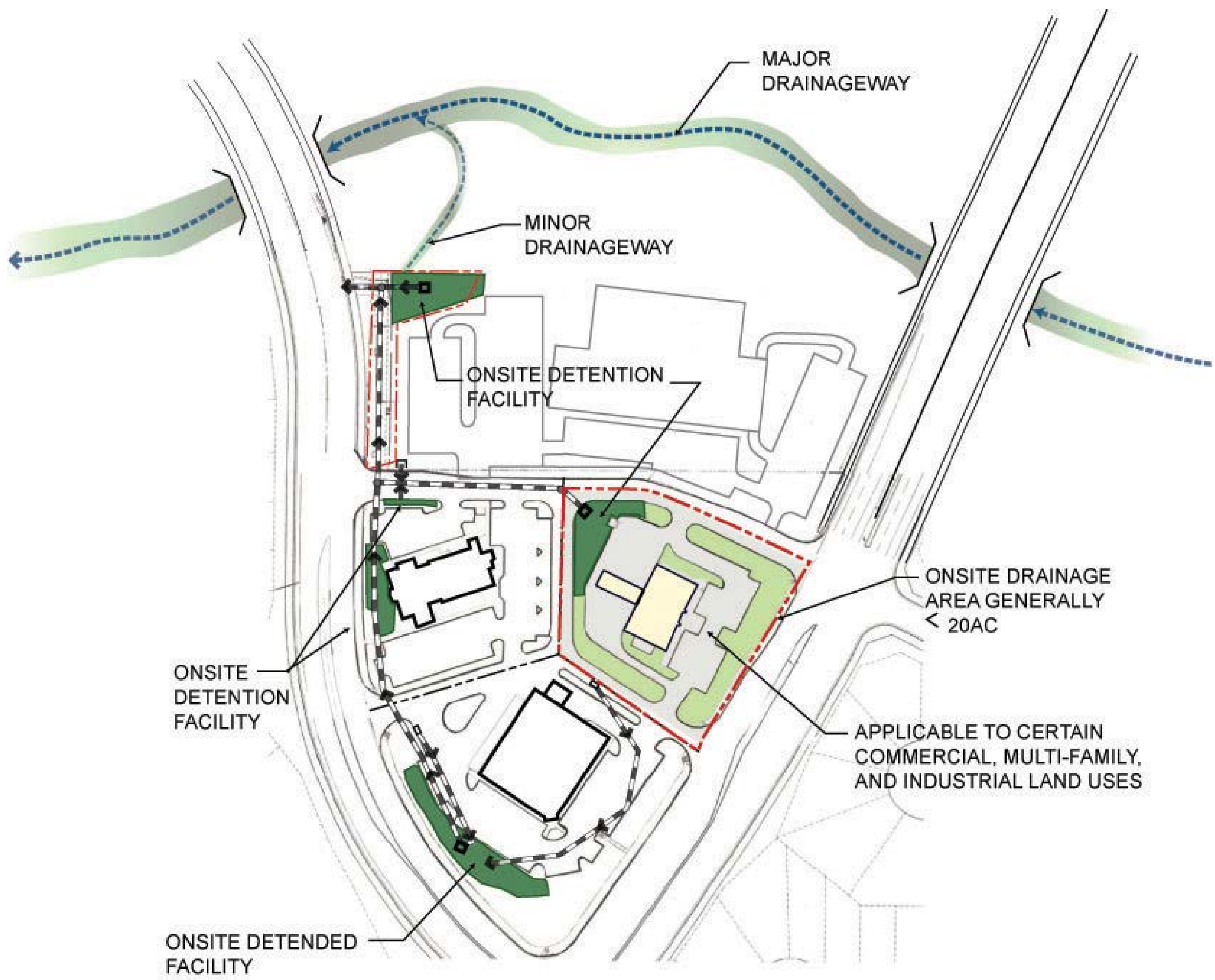


Figure 12-3. Example configuration for on-site detention (Source: Arapahoe County)

The principal advantage of on-site facilities is that developers can be required to build them as a condition of site approval. Major disadvantages include the need for a larger total land area for multiple smaller on-site facilities as compared to a larger regional facility serving the same tributary catchment area. If the individual on-site facilities are not properly maintained, they can become a nuisance to the community and a basis for many complaints to municipal officials. It is also difficult to ensure adequate maintenance and long-term performance. Approximately 100 on-site facilities built, or required by municipalities to be built, as a part of land developments over about a 10-year period were inspected and it was concluded that a lack of adequate maintenance and implementation contributed to a loss of continued function or even presence of these facilities (Prommesberger 1984).

2.4 Detention and UDFCD 100-Year Floodplain Management Policy

In light of the difficulties involved in ensuring the long term effectiveness of on-site detention and privately maintained subregional detention facilities, UDFCD adheres to the following policies when developing hydrology for the delineation and regulation of the 100-year flood hazard zones:

1. Hydrology must be based on fully developed watershed conditions (e.g., imperviousness) as estimated to occur, at a minimum, over the next 50 years.

2. No detention basin will be recognized in the development of hydrology unless:

    a. It serves a watershed that is at least 130 acres or otherwise provides substantial flood reduction, and

    b. It is owned (or controlled by legal document) and maintenance is either performed or required by a public agency, and

    c. The public agency has committed to ensure that the detention facility continues to operate in perpetuity as designed.

These policies are for the definition and administration of the 100-year floodplain and floodway zones and the design of facilities along major drainageways. The intent is not to discourage communities from using subregional or onsite detention discussed above. Subregional and onsite detention can be very beneficial for stormwater quality and quantity management, reducing the sizes of local storm drains and other conveyances, and providing a liability shield (defense) when needing to address the issue of keeping stormwater-related damages to downstream properties from increasing as upstream lands are developed.

3.0 Full Spectrum Detention as the Recommended Approach

The design guidance provided in this chapter is based on an approach termed "full spectrum detention." The intent of full spectrum detention is to reduce the flooding and stream degradation impacts associated with urban development by controlling peak flows in the stream for a range of events.

3.1 Background

Roofs, streets, parking lots, sidewalks, and other impervious surfaces increase peak flows, frequency of runoff and total volume of stormwater surface runoff when compared to pre-development conditions.

This increase is most pronounced for the smaller, more frequent storms and can result in stream degradation and water quality impacts as well as flooding during the large events.

Criteria for stormwater detention design have evolved from a focus on the minor and major events to an approach shown to better control peak flows for a wide range of events. In the interest of stream stability, specific focus should be placed on frequent events. Incorporating a slow release for the water quality capture volume (WQCV) helps to address very frequent urban runoff events; however, it is also important to extend the volume of water attenuated to capture the range of flows that transport the most bed load in the receiving stream. This range of flows depends on reach-specific characteristics but is typically between the annual event and the 5-year peak flow rate. Runoff events in this range can produce profound geomorphic changes in ephemeral, intermittent and perennial streams resulting in severe erosion, loss of riparian habitat, and water quality degradation.

Furthermore, outflow hydrographs from traditional detention facilities tend to be "flat-topped" and broad, maintaining flows near the maximum release rates for relatively long periods of time. This allows hydrographs released from multiple independent basins to overlap and add to each other to a greater degree than they would have during pre-development conditions. Thus, traditional detention practices can result in an increase in *watershed-wide* discharges even if individual detention facilities each would control peak discharges to pre-developed conditions.

Full spectrum detention is designed to address these two limitations of traditional detention. First, it is focused on controlling peak discharges over the full spectrum of runoff events from small, frequent storms up to the 100-year flood. Second, full spectrum detention facilities produce outflow hydrographs that, other than a small release rate of the excess urban runoff volume (EURV), replicates the shape of pre-development hydrographs. Full spectrum detention modeling shows reduction of urban runoff peaks to levels similar to pre-development conditions over an entire watershed, even with multiple independent detention facilities.

3.2 Excess Urban Runoff Volume

The lower portion of volume in a full spectrum detention facility is designed to capture and slowly release the excess urban runoff volume (EURV). The EURV is the difference between the developed condition runoff volume and the pre-development volume. Based on the hydrologic methods used within the UDFCD region, the EURV appears to be relatively consistent at any given level of imperviousness for the range of storms (generally beyond the 2-year event) that produce runoff. This is illustrated in Figure 12-4. The full spectrum detention concept is to reduce runoff for all the frequent storms (smaller than approximately the 2-year event) to as close to zero as possible and less than the threshold value for erosion in most streams. When this is done, the remaining runoff from a site approximates the runoff volume for pre-development conditions.

The EURV is typically two to three times the water quality capture volume (WQCV) and the release rates are generally comparable. Therefore, designing for EURV typically results in a design that also meets the recommended drain time for treatment of the WQCV.

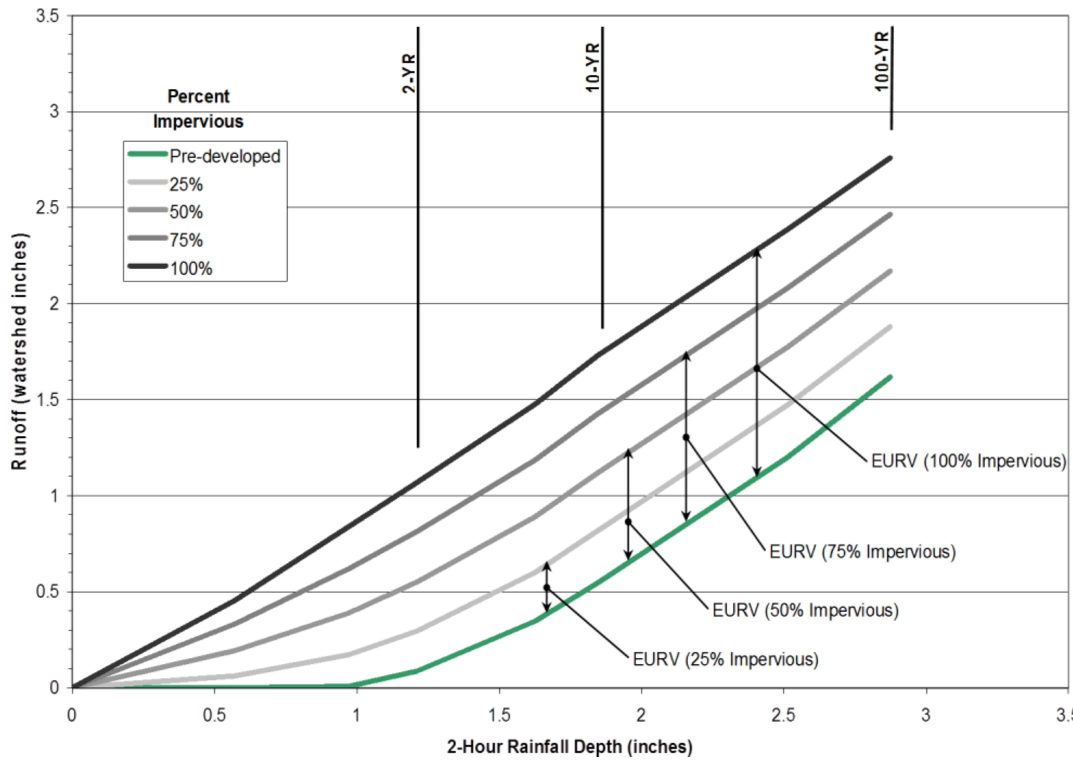


Figure 12-4. EURV is relatively constant for runoff producing storms

The upper portion of volume in a full spectrum detention facility is designed to reduce the developed condition 100-year peak discharge down to 90 percent of the pre-development 100-year peak flow rate from the tributary sub-watershed. Through modeling, it has been found that releasing 90 percent of the 100-year event peak discharge at each full spectrum detention basin within a watershed results in flows in the receiving stream that are near pre-development. Figure 12-5 illustrates the effectiveness of full spectrum detention in comparison to traditional practices for a test watershed made up of fifty 100-acre subwatersheds each modeled with a detention basin (Wulliman and Urbonas, 2005).

Benefits of Implementing Full Spectrum Detention on a Watershed Level

- A properly designed full spectrum detention facility can reduce urban peak discharges to levels similar to pre-development conditions for the full spectrum of runoff events from small, frequent storms up to the 100-year event. This reduces the stresses imposed by urban runoff on streams so degradation will occur at reduced rates compared to conventional detention practices.
- With the capture and slow release of the EURV mitigating to some degree the additional runoff impacts associated with development, the remaining volume that is released from a full spectrum facility approximates the runoff from the upstream area for pre-development conditions. This allows regional full spectrum detention to effectively control peak discharges within a watershed even when multiple independent facilities are used.
- The design of full spectrum detention is relatively simple, and certainly no more complex than traditional detention practices.
- Required 100-year storage volumes for full spectrum detention facilities are generally similar to traditional flood control and water quality detention practices.

Because of these benefits, UDFCD recommends the use of full spectrum detention over typical detention criteria associated with stormwater quantity.

3.3 Compatibility of Full Spectrum Detention with Minor and Major Event Detention

The EURV and 100-year detention volumes are similar in magnitude to 10-year/100-year detention facilities volumes. The main difference is that the EURV described in Section 2.2 is drained at a much slower rate than the 10-year detention volume would be based on past criteria provided in this manual.

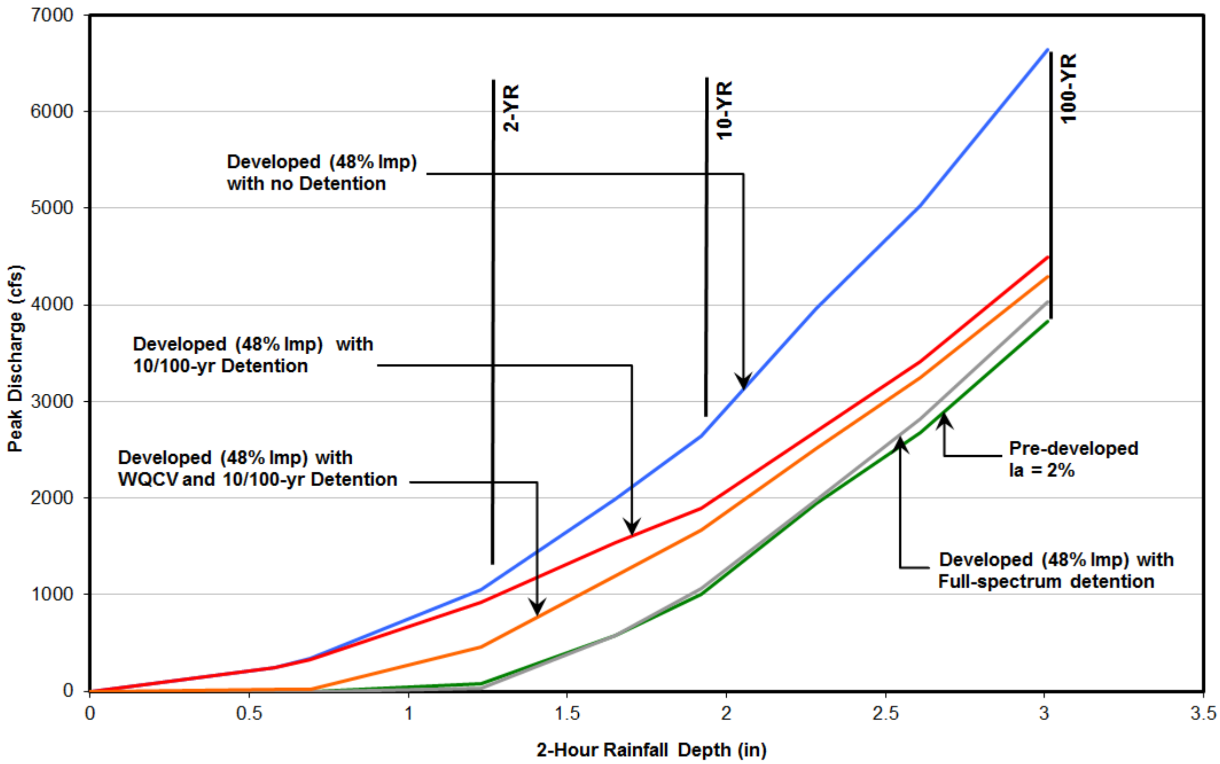


Figure 12-5. Comparison of full spectrum detention and conventional practices for a sample watershed consisting of fifty 100-acre subwatersheds

Where existing master plans recommend detention facilities designed to address minor and major events, UDFCD generally intends that these will be implemented as full spectrum facilities; however, the final determination of detention policy should be made by each jurisdiction.

There may be opportunities to convert existing 10-year/100-year detention facilities into full spectrum facilities by reducing the capacity of the 10-year control orifice to a EURV release rate, and ensuring that the debris grate for the EURV orifices and the 100-year outlet and emergency spillway for the facility are adequate.

3.4 Water Quality Capture Volume and Full Spectrum Detention

This section provides criteria for incorporating five types of WQCV treatment best management practices (BMPs) into full spectrum detention basins. Volume 3 of the USDCM further describes these BMPs. They are:

- Extended detention basins,
- Retention ponds,
- Constructed wetland ponds,
- Sand filters, and
- Rain gardens (bioretention)

The 100-year full spectrum detention volume described in this chapter is consistently expressed as the total detention volume including EURV; also, EURV consistently includes the water quality volume. Therefore, the WQCV and the EURV are both inclusive of the 100-year full spectrum detention volume and UDFCD does not recommend adding any part of the WQCV to either the EURV or the 100-year volumes calculated based on Section 3.0.

Figure 12-6 illustrates an extended detention basin combined with full spectrum detention. In the figure, Zone 1 represents the water quality portion of the facility. Zone 2 represents the difference between the EURV and Zone 1. Zone 3 represents the difference between the 100-year volume and the EURV. The design volume, drain time, and release rate of each zone of an extended detention basin combined with full spectrum detention is shown in Table 12-1.

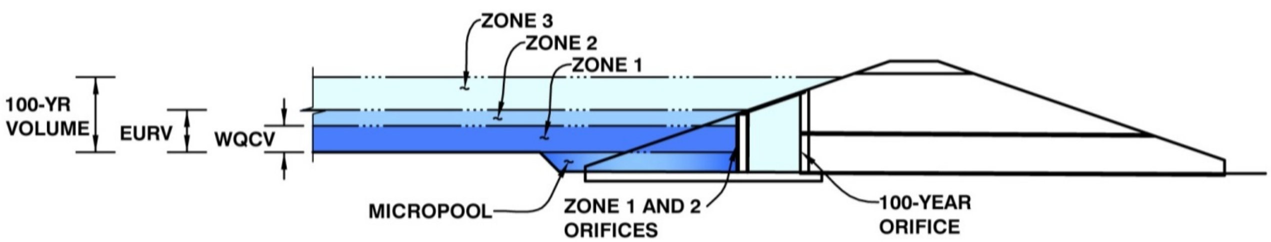


Figure 12-6. Extended detention basin combined with full spectrum detention

Table 12-1. Extended detention basin combined with full spectrum detention

Zone	Volume	Drain Time of Zone, hrs	Maximum Release Rate
1	40-hr WQCV	40	Based on drain time
2	EURV minus (40-hr WQCV))	12 to 32[1]	Based on drain time
3	100-yr minus EURV	Based on release rate	0.9(predevelopment Q_{100})

[1]Colorado law requires 97% of the 5-year event to drain within 72 hours.

Because each of the five WQCV treatment BMPs has slightly different sizing criteria and release rate criteria, as described in Volume 3 of the USDCM, the design of full spectrum detention facilities also varies based on type of WQCV BMP. The design of a retention pond combined with full spectrum detention is shown in Figure 12-7 and in Table 12-2.

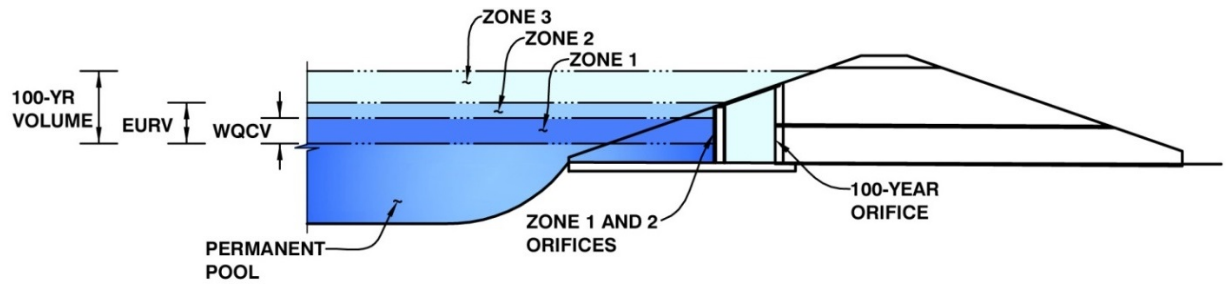


Figure 12-7. Retention pond combined with full spectrum detention

Table 12-2. Retention pond combined with full spectrum detention

Zone	Volume	Drain Time of Zone, hrs	Maximum Release Rate
1	12-hr WQCV	12	Based on drain time
2	EURV minus 12-hr WQCV	12 to 60[1]	Based on drain time
3	100-yr minus EURV	Based on release rate	0.9(predevelopment Q_{100})

[1]Colorado law requires 97% of the 5-year event to drain within 72 hours.

The design of a constructed wetland pond combined with full spectrum detention is shown in Figure 12-8 and in Table 12-3.

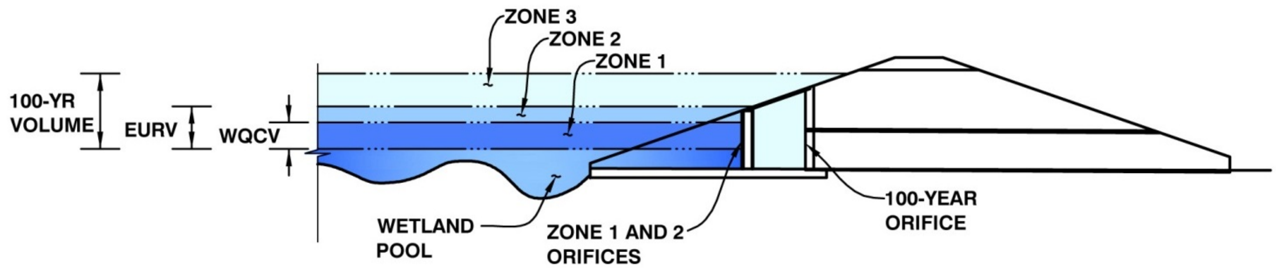


Figure 12-8. Constructed wetland pond combined with full Sspectrum detention

Table 12-3. Constructed wetland pond combined with full spectrum detention

Zone	Volume	Drain Time of Zone, hrs	Maximum Release Rate
1	24-hr WQCV	24	Based on drain time
2	EURV minus 24-hr WQCV	12 to 48[1]	Based on drain time
3	100-yr minus EURV	Based on release rate	0.9(predevelopment Q_{100})

[1]Colorado law requires 97% of the 5-year event to drain within 72 hours.

The design of a sand filter combined with full spectrum detention is shown in Figure 12-9 and in Table 12-4. Although the water quality event is released through the filter media, it is recommended that an orifice be provided to drain Zone 2 (the balance of the EURV) and a grated inlet or spillway be used to control the release of Zone 3 (the balance of the 100-year volume). This configuration reduces the amount of Zone 2 and 3 runoff flowing through the filter media.

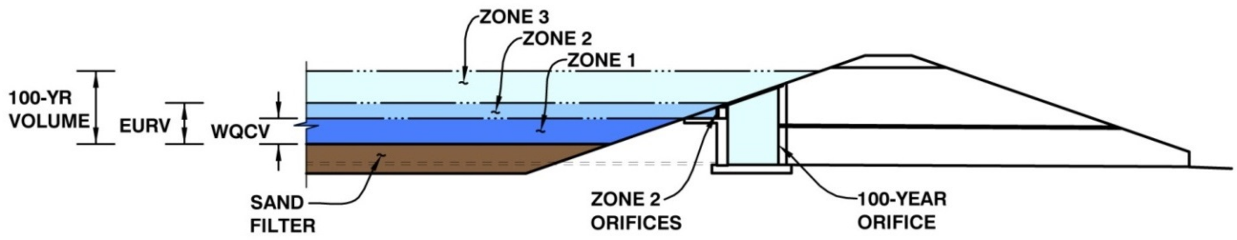


Figure 12-9. Sand filter combined with full spectrum detention

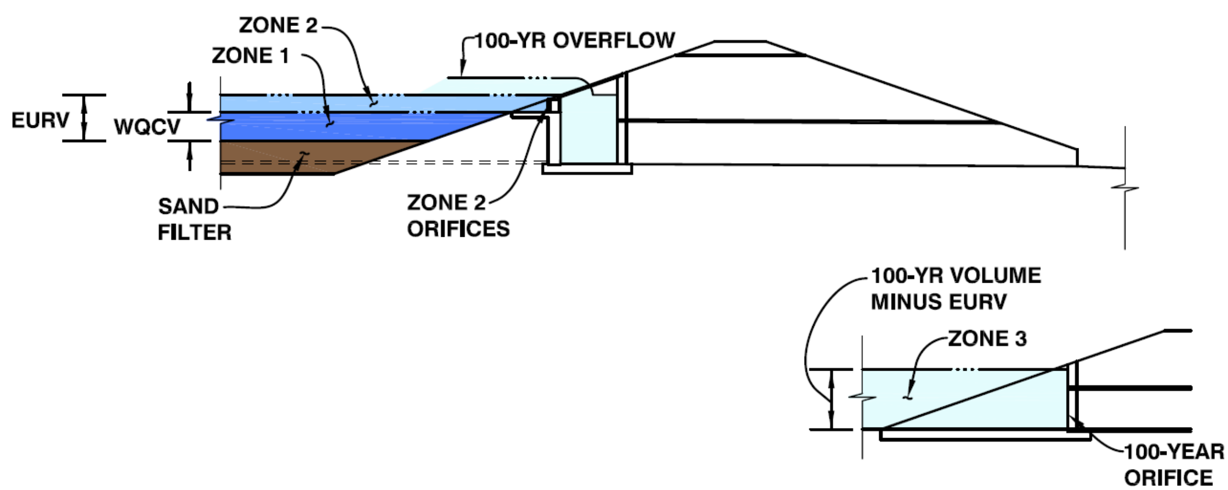


Figure 12-10. Sand filter and zone 2 combined with downstream zone 3 basin

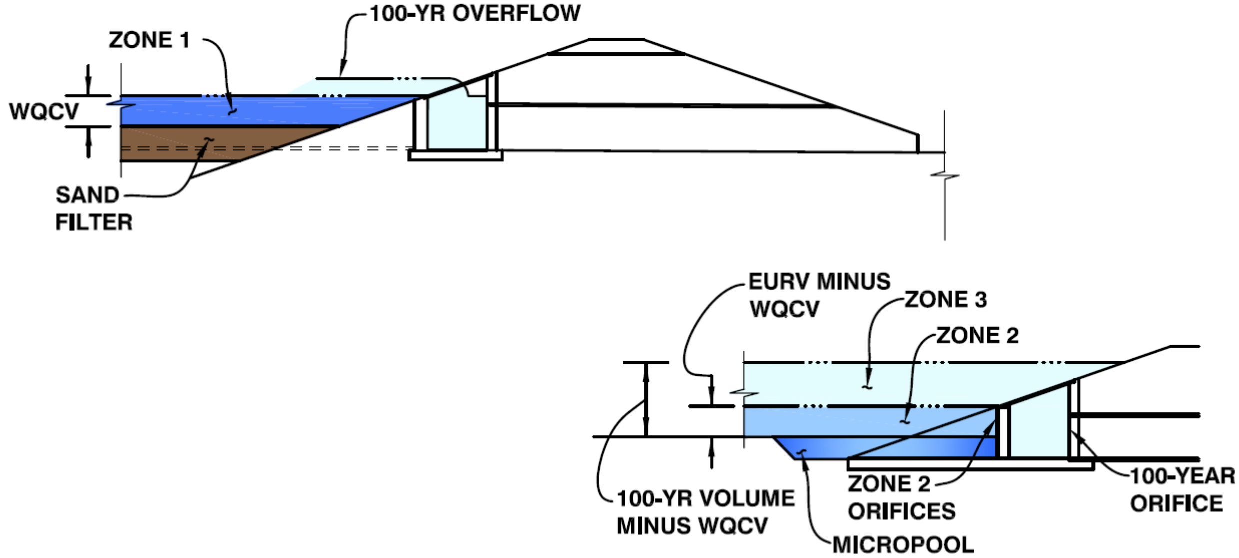


Figure 12-11. Stand-alone sand filter with downstream zone 2/zone 3 basin

The design of a bioretention facility combined with full spectrum detention is shown in Figure 12-12 and in Table 12-4. As in a sand filter, it is recommended that an orifice plate be provided to drain Zone 2 (the balance of the EURV) and a grated inlet or spillway be used to control the release of Zone 3 (the balance of the 100-year volume). Because these facilities are often implemented in compact areas and in multiple installations such as in parking medians and small landscaped areas, and because maintaining vegetation is critical to the facility, it is recommended to separate these facilities from Zone 3 or from both Zone 2 and 3. Configurations of separate facilities are shown in Figures 12-13 and 12-14. In these cases, the volume, drain time, and release rate of the zones are still determined based on Table 12-4.

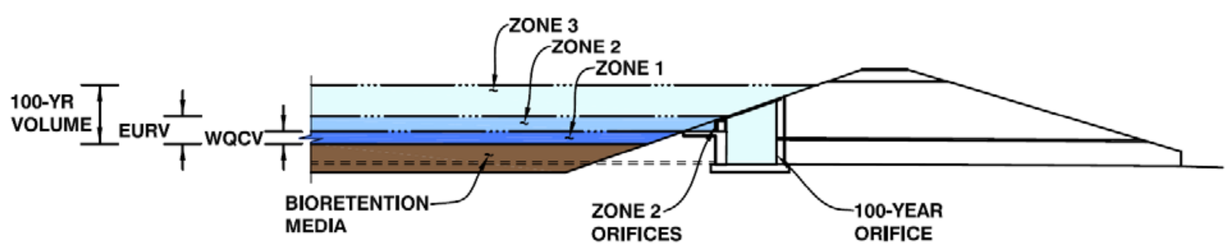


Figure 12-12. Bioretention combined with full spectrum detention (not ideal for vegetation)

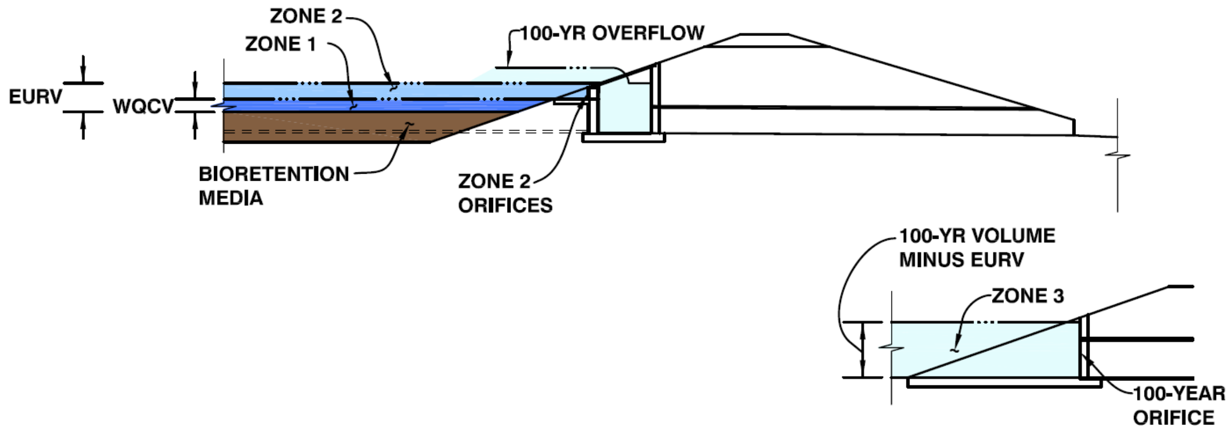


Figure 12-13. Bioretention and zone 2 combined with downstream zone 3 basin

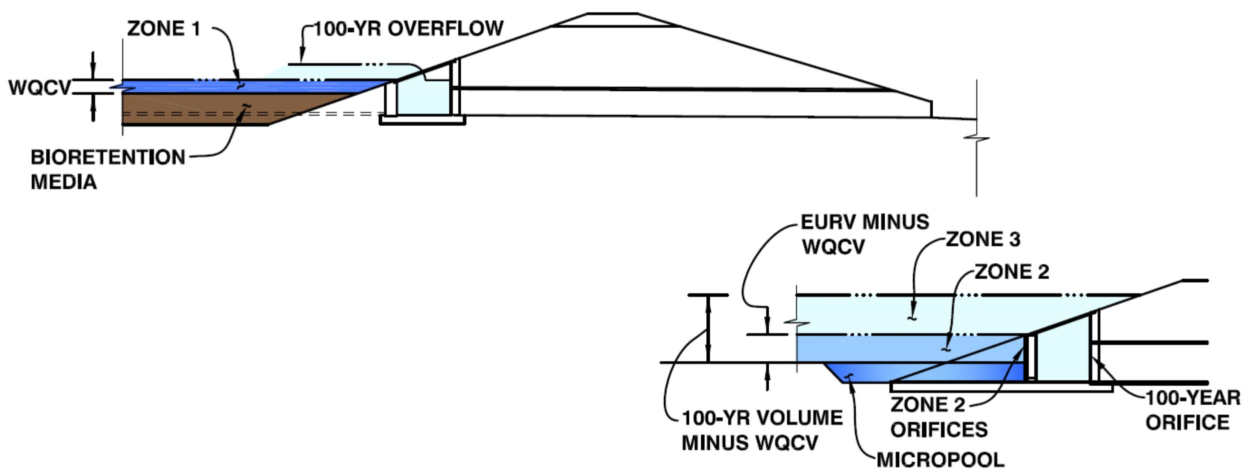


Figure 12-14. Stand-alone bioretention with downstream zone 2/zone 3 basin

Table 12-4. Sand filter or bioretention facility combined with full spectrum detention

Zone	Volume	Drain Time of Zone, hrs	Maximum Release Rate
1	12-hr WQCV	12	Based on drain time
2	EURV minus 12-hr WQCV	12 to 32[1]	Based on drain time
3	100-yr minus EURV	Based on release rate	0.9(predevelopment Q_{100})

[1]Colorado law requires 97% of the 5-year event to drain within 72 hours.

4.0 Sizing of Full Spectrum Detention Storage Volumes

Three methods for sizing full spectrum detention storage volumes are described in the USDCM, as follows:

1. Simplified Equation
2. UD-Detention workbook
3. Hydrograph routing using CUHP and SWMM

The recommended range of application for the methods based on upstream watershed area is shown in Table 12-5. Full spectrum detention facilities may be sized using any of the methods shown in the table for the ranges of watershed area; however, the UD-Detention workbook more accurately represents input variables than the simplified equation and the hydrograph approach provides the most accurate approach. UDFCD recommends the hydrograph routing approach when evaluating multiple full spectrum detention facilities arranged in parallel or series in a watershed. The three sizing methods are described in the following sections.

Table 12-5. Applicability of full spectrum sizing methods based on watershed area

Watershed Properties	Sizing Method		
	Simplified Equations	UD-Detention[1]	CUHP/SWMM Hydrograph Routing
Less than 10 acre	X	X	
10 to 50 acres		X	X
50 to 130 acres		X	X
130 acres to 1 mile^2		X	X
Greater than 1 mile^2		X	X
Multiple detention facilities in parallel or series			X

[1]See Section 4.2 for additional discussion on the use of UD-Detention for the preliminary design and final design of a full spectrum facility.

4.1 Simplified Equations

Simplified equations are provided in this section for determining full spectrum detention design volumes and 100-year release rates. Once these values are determined, a full spectrum detention facility may be designed according to the technical guidance described in Section 5.0.

4.1.1 Full Spectrum Detention Volume

Three different volumes are associated with the design of a full spectrum detention facility, as illustrated in Section 3.4. These are:

1. **WQCV** (Zone 1)
2. **EURV** (Zone 1 plus Zone 2)
3. **100-year volume** (sum of Zones 1, 2, and 3)

Within the ranges identified in Table 12-5, these volumes may be determined using simplified equations, as described below.

WQCV. The water quality capture volume for each of the five types of water quality facilities shown in Section 3.4 can be calculated based on the procedures described in Volume 3 of the USDCM.

EURV. Use equations 12-1, 2 and 3 to find EURV in watershed inches for specific soil types.

$$\mathrm{EURV_A} = 1.68i^{1.28} \qquad \text{Equation 12-1}$$

$$\mathrm{EURV_B} = 1.36i^{1.08} \qquad \text{Equation 12-2}$$

$$\mathrm{EURV_{C/D}} = 1.20i^{1.08} \qquad \text{Equation 12-3}$$

Where:

$\mathrm{EURV_K}$ = Excess urban runoff volume in watershed inches (*K* indicates NRCS soils type),
i = Imperviousness ratio (a decimal less than or equal to 1)

The Technical Memorandum entitled *Determination of the EURV for Full Spectrum Detention Design*, dated December 22, 2016 documents the derivation of these equations. This is available at www.udfcd.org. Apply the equations above for each of the soil types found in the watershed and then calculate a weighted average value based on the relative area proportion of each soil type. Convert the EURV in watershed inches to a volume multiplying it by the watershed area.

Whenever NRCS soil surveys are not available for a catchment area, soils investigations are recommended to estimate equivalent soil type.

100-Year Volume. A simplified equation can be used to determine the required 100-year full spectrum detention volume for tributary areas less than 10 acres. This volume includes the EURV (and the EURV includes the WQCV). UDFCD does not recommend adding additional volume above that provided in Equation 12-4. The derivation of this equation is documented in a Technical Memorandum entitled *Estimation of Runoff and Storage Volumes for Use with Full Spectrum Detention*, dated January 5, 2017 (available at www.udfcd.org). If a more detailed analysis is desired, see Table 12-5. The 100-year volume in watershed inches is converted to cubic feet or acre-feet by multiplying by watershed area and converting units.

$$V_{100} = P_1 \left[\begin{array}{l} 0.806i^{1.225} + 0.109i^{0.225})A\% + (0.412i^{1.371} + 0.371i^{0.371})B\% \\ + (0.341i^{1.389} + 0.398i^{0.389})CD\% \end{array} \right] \qquad \text{Equation 12-4}$$

Where:

V_{100} = detention volume in watershed inches

P_1 = one-hour rainfall depth (inches)

i = imperviousness ratio (a decimal less than or equal to 1)

A%, B%, and CD% = indicates percentage of each NRCS soils type (expressed as a decimal)

4.1.2 100-year Release Rates

The maximum allowable 100-year release rate for a full spectrum detention facility is equal to 90 percent of the predevelopment discharge for the upstream watershed. Modeling has shown that using this release rate for multiple full spectrum detention basins within a watershed is effective in controlling future development peak discharges in the receiving stream to levels below predevelopment conditions for the 2, 5, 10, 25, 50, and 100-year events.

The predevelopment 100-year unit discharge for specific soil types per acre of tributary catchment varies based on the watershed slope and the watershed shape (described as the ratio of the flow length squared to the watershed area). Use Equation 12-5 with coefficients provided in Tables 12-6, 12-7, and 12-8 to calculate the peak unit flow rate based on an assumed predevelopment imperviousness of 2%. When using this equation, UDFCD recommends a sloped value no less than 0.01 and no greater than 0.04 and a shape value no less than one and no greater than six. Multiply the 100-year peak unit flow rate by 0.9 to determine the allowable 100-year release from a watershed.

See the Technical Memorandum entitled *UDFCD Predeveloped Peak Unit Flowrates,* dated December 21, 2016 for documentation of the following equation and tables. This is available at www.udfcd.org.

$$q = P_1 C_1 S^{C_2} \left(\frac{L^2}{A} \right)^{C_3}$$ Equation 12-5

Where:

q = peak unit flow rate (cfs/acre)

P = one-hour precipitation depth (in) from NOAA Atlas 14

S = watershed flow path slope (ft/ft)

L = watershed flow path length (ft)

A = area of tributary (ft^2)

C_1, C_2, C_3 = coefficients dependent on event frequency (see Tables12-6, 12-7, and 12-8)

Table 12-6. Coefficients for NRCS hydrologic soil group A

Return Period →		2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
Leading Coeff.	C_1	0.0014	0.0104	0.0208	0.0478	0.2652	0.5622	0.9318
Slope Exp.	C_2	0.1684	0.2065	0.2070	0.2491	0.2056	0.2021	0.1853
Shape Exp.	C_3	-0.3533	-0.4430	-0.4453	-0.4406	-0.4385	-0.4286	-0.3933

Table 12-7. Coefficients for NRCS hydrologic soil group B

Return Period →		2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
Leading Coeff.	C_1	0.0285	0.0377	0.3509	0.8566	1.0437	1.2088	1.4061
Slope Exp.	C_2	0.1911	0.1855	0.2069	0.1761	0.1743	0.1677	0.1640
Shape Exp.	C_3	-0.4045	-0.3950	-0.4446	-0.3729	-0.3696	-0.3542	-0.3470

Table 12-8. Coefficients for NRCS hydrologic soil group C

Return Period →		2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
Leading Coeff.	C_1	0.0338	0.2418	0.5375	0.9920	1.1614	1.3053	1.4949
Slope Exp.	C_2	0.1869	0.2005	0.1901	0.1720	0.1715	0.1651	0.1623
Shape Exp.	C_3	-0.3946	-0.4280	-0.4055	-0.3641	-0.3637	-0.3490	-0.3438

When multiple soil types exist in the watershed, use the table values for each soil type and calculate a weighted average value relative to the area proportion of each soil type. Use Equation 12-6 to calculate the allowable discharge from the basin.

$$Q = 0.9aq \qquad \text{Equation 12-6}$$

Where:

Q = Allowable 100-year release rate (cfs)

a = Area of watershed (acres)

q = weighted average unit release rate based on relative proportions of watershed soil types (cfs/acre)

Unless otherwise recommended in an approved master plan, the maximum releases rates described in this section are for all full spectrum detention facilities.

4.1.3 Predevelopment Peak Discharges for Various Return Periods

The intent of the UDFCD full spectrum detention policy is to manage developed condition peak flows to levels similar to predevelopment conditions for a full range of return periods in areas serviced by full spectrum detention facilities. To gain a sense for the magnitude of predevelopment peak flow rates for various return rates, see the Technical Memorandum entitled *UDFCD Predeveloped Peak Unit Flowrates,* (MacKenzie and Rapp, 2016). This is available at www.udfcd.org.

4.2 UD-Detention Workbook

Beyond the simplified equation described in Section 4.1, an Excel-based workbook is available to size full spectrum detention basins for the range of watershed sizes identified in Table 12-5. UD-Detention is available at www.udfcd.org. This workbook uses the Modified Puls reservoir routing method to evaluate performance of the facility based on tributary watershed parameters and variables associated with the basin/pond geometry and outlet configuration. It compares calculated release rates to predevelopment discharges for the 2, 5, 10, 25, 50, and 100-year events. UD-Detention allows analysis of any retention pond or detention basin including extended detention, bioretention, sand filters, basins that may or may not be full spectrum, basins that only include one or two controlled zones, or basins having unusual outlet structures.

Section 8.0 of this chapter includes an example problem using each of the workbooks.

4.2.1 Hydrograph Routing using CUHP and SWMM

Hydrograph routing using CUHP and SWMM is a third option for sizing and designing full spectrum detention facilities, based on the watershed properties identified in Table 12-5. This is the only method that is able to assess the performance of multiple detention facilities arranged in parallel or in series in a watershed. Hydrograph routing using SWMM is similar to the evaluation mode of UD-Detention in that the user needs to input stage-area and stage-discharge information based on a preliminary design and iterations may be necessary to arrive at a final basin and outlet structure configuration that reduces developed condition peak flows to levels equal to or below predevelopment conditions.

The reservoir routing capabilities in SWMM determine a detention basin's outflow characteristics given the stage-discharge relationship for a reservoir outlet link and the stage- area relationship for the reservoir storage node of the model. The stage- area relationship is determined by finding the water surface areas of the basin at different depths or elevations, which are then used by the model to calculate the incremental volumes used as the stage rises and falls. The basin layout and outlet structure are modified as needed after each model run to adjust the corresponding stage-area and stage-discharge data pairs, until the outflow from the basin meets the specified flow limit. No description of the theory of reservoir routing is provided in the USDCM, as the subject is well described in many hydrology reference books (Viessman and Lewis 1996; Guo 1999b).

For full spectrum basins evaluated using hydrograph routing, the EURV portion of the basin still needs to be sized using Equations 12-1 through 12-3 in Section 4.1 and the outlet designed to empty this volume as described in Section 3.4. The 100-year peak flow control volume above the EURV (Zone 3) must be determined, and its outlet designed using full hydrograph routing protocols. The maximum allowable 100-year release rate should not exceed 90 percent of the approved predevelopment release rate determined through CUHP/SWMM modeling of the upstream watershed (this may vary slightly from the predevelopment discharge values presented in Section 4.1.2), or maximum flow rates recommended in an accepted master plan.

5.0 Design Considerations

The design of a detention facility entails detailed hydraulic, structural, geotechnical, and civil design. This includes a detailed site grading plan, embankment design, spillway design, hydraulic and structural design of the outlet works, safety grate design, maintenance access, consideration of sedimentation and erosion potential within and downstream of the facility, liner design (if needed), etc. Collaboration between geotechnical engineers, structural engineers, hydrologic and hydraulic engineers, land planners, landscape architects, biologists, and/or other disciplines is encouraged during the preliminary and final design phases.

It is beyond the scope of the USDCM to provide detailed dam design guidance. There are many excellent references in this regard, such as *Design of Small Dams* (U.S. Bureau of Reclamation 1987). UDFCD urges all designers to review and adhere to the guidance in such references as failure of even small embankments can have serious consequences for the public and the municipalities downstream of the embankment.

As discussed in Section 3.4, full spectrum detention facilities are configured together with one of five types of water quality basins described in Volume 3 of the USDCM. The design of the water quality portion of the facility, illustrated as Zone 1 in Section 3.4, is described in detail in Volume 3. The following guidelines for the design of full spectrum detention facilities apply to Zones 2 and 3 as shown in Figures 12-6 through 12-14.

5.1 General Layout and Grading

Storage facility geometry and layout are often best developed in concert with land planners and landscape architect. Whenever desirable and feasible, multiple uses of a basin should be considered, such as creation of riparian and wetland vegetation, wildlife habitat, paths, and other passive or active recreation opportunities. If multiple uses are being contemplated, it is recommended that the inundation of passive recreational areas be limited to one or two occurrences a year and of active recreation areas to once every two years. Generally, the area within Zone 1 and Zone 2 is not well suited for active recreation facilities such as ballparks, playing fields, and picnic areas, but may be suitable for passive recreation such as wildlife habitat and some hiking trails. It is desirable to shape the water quality portion of the facility (Zones 1 and 2) with a gradual expansion from the inlet and a gradual contraction toward the outlet, thereby minimizing short-circuiting.

Maintenance is also an important consideration with respect to layout and grading. Consider how lower areas of the basin, such as the forebay and micropool will be accessed, and with what equipment.

5.2 Storage Volume

Provide the total 100-year storage volume determined using one of the three methods described in Section 4, along with additional basin storage and depth necessary to contain emergency flows and provide freeboard as described in Section 5.3.

5.3 Embankments

Embankment should be designed to not catastrophically fail during the 100-year and larger storms that the facility may encounter. The following criteria apply in many situations (ASCE and WEF 1992):

- **Side Slopes:** For ease of maintenance, the side slopes of the embankment should not be steeper than 3(H):1(V), with 4(H):1(V) preferred. The embankment's side slopes should have fully vegetated coverage, with no trees or shrubs above the basin floor. Soil-riprap protection (or equivalent) may be necessary to protect it from wave action on the upstream face, especially in retention ponds.

- **Settlement and Compaction:** The design height of the embankment should be increased by roughly 5 percent to account for settlement. All earth fill should be free from unsuitable materials and all organic materials such as grass, turf, brush, roots, and other organic material subject to decomposition. The fill material in all earth dams and embankments should be compacted to at least 95 percent of the maximum density based on the Modified Proctor method of ASTM D698 testing.

- **Freeboard:** The elevation of the top of the embankment should be a minimum of 1 foot above the water surface elevation when the emergency spillway is conveying the maximum design or emergency flow. When the embankment is designed to withstand overtopping of the undetained peak flow without failure, freeboard requirements may be reduced or waived.

Anti-Seepage may also be required. This topic is covered in detail in FEMA's *Technical Manual: Conduits through Embankment Dams* (2005) and NRCS's *National Design Construction and Soil Mechanics Center Technical Note – Filter Diagrams for CO-1 Structures* (2003). Construction of a filter diaphragm will be adequate in most scenarios covered in this chapter.

If the storage facility is determined to be "jurisdictional" per the criteria of Colorado Division of Water Resources (DWR), also known as the Office of the State Engineer, the embankment shall be designed, constructed and maintained to meet DWR's most-current criteria for jurisdictional structures. The design for an embankment of a stormwater detention or retention storage facility should be based upon a site-specific engineering evaluation.

5.4 Emergency Spillways

Provide an open channel emergency spillway to convey flows that exceed the primary outlet capacity or when the outlet structure becomes blocked with debris. When the storage facility falls under the jurisdiction of the DWR, design this bill way based on the storm prescribed by the DWR (DWR 2007). 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 (e.g., avoidance of a critical facility). Generally, embankments should be fortified against and/or have spillways that, at a minimum, are capable of conveying the 100-year peak runoff from the fully developed tributary area (prior to routing flows though the detention basin). However, detailed analysis and determination of downstream hazards (such as critical facilities) should be performed and may indicate that the embankment protection and/or spillway design needs to be designed for events larger than the 100-year design storm.

An emergency spillway also controls the location and direction of the overflow. Clearly depict the emergency spillway and the path of the emergency overflow downstream of the spillway and embankment on the construction plans and do not allow structures (such as utility boxes) to be placed in the path of the emergency spillway or overflow.

Soil riprap is the most common method for providing embankment protection on a spillway. Although not preferred, baffle chute spillways may also be considered on a case by case basis. Further discussion regarding these two types of embankment protection is provided below.

5.4.1 Soil Riprap Spillway

Soil riprap embankment protection should be sized based on methodologies developed specifically for overtopping embankments. Two such methods have been documented (U.S. Nuclear Regulatory Commission, 1988 and Robinson et al., 1998). See these publications for a complete description of sizing methodology and application information. Figure 12-21 illustrates typical rock sizing for small (under 10-feet high) embankments based on these procedures that may be used during preliminary design to get an approximate idea of rock size. Final design should be based on the more complete procedures documented in the referenced publications. The thickness should be based on the criteria identified in the *Open Channels* chapter for steep channels. For spillway design, it is critical that the soil riprap has an adequate percentage of well-graded rock.

The invert of the emergency spillway is set at or above the 100-year water surface elevation (based on local jurisdiction criteria). A concrete wall is recommended at the emergency spillway crest extending at least to the bottom of the soil riprap located immediately downstream. The top of the crest wall at the sides should extend to the top of the embankment, at least one foot above the spillway elevation.

5.4.2 Baffle Chute Spillway

The USBR has developed design standards for a reinforced concrete chute with baffle blocks on the sloping face of a spillway, commonly referred to as baffled chute drop spillway. The primary reference that is recommended for the design of these structures is *Design of Small Dams* (1987). In addition, *Design of Small Canal Structures* (Aisenbrey, et al. 1978) and *Hydraulic Design of Stilling Basins and Energy Dissipators* (Peterka 1984) may provide useful information for the design of baffle chute spillways.

Photograph 12-3. Baffle chute drop after several decades of service.

The hydraulic concept behind baffle chute spillways involves flow repeatedly encountering obstructions (baffle blocks) that are of a nominal height equivalent to critical depth. The excess energy is dissipated through the drop by the momentum loss associated with reorientation of the flow. *Design of Small Dams* provides guidelines for sizing and spacing the blocks. Designing for proper approach velocities is critical to structure performance. One advantage of this type of spillway is that it does not require any specific tailwater depth. However, the designer does need to consider local flow and scour patterns in the transition back to the channel.

For safety reasons and considerations of appearance, a baffle chute spillway is not recommended for use as a grade control structure in a stream. Caution is advised when using a baffle chute spillway in a high debris area because the baffles can become clogged, resulting in overflow, low energy dissipation, and direct impingement of the erosive stream jet downstream.

A step by step procedure for the design of a baffle chute drop spillway is provided in *Design of Small Dams*. Typical design elements consist of upstream transition walls, a rectangular approach chute, a sloping apron (generally equal to the downstream slope of the basin embankment) that has multiple rows of baffle blocks and downstream transition walls. The toe of the chute extends below grade and is backfilled with loose riprap to prevent undermining of the structure by eddy currents or minor degradation downstream. The structure is effective even with low tailwater; however, greater tailwater reduces scour at the toe. The structure lends itself to a variety of soils and foundation conditions.

The steps involved in the construction of a baffle chute spillway are typical of the construction of any reinforced concrete structure, and include subgrade preparation, formwork, setting reinforcing steel, placing, finishing and curing concrete, and structure backfilling. Baffle chutes generally provide consistent, dependable hydraulic performance and are relatively straightforward to construct. Potential construction challenges include foundation integrity, water control, and managing the multiple phases of formwork, reinforcing, and concrete placement and finishing.

5.5 Outlet Structure

Outlet structures control release rates from storage facilities and should be sized and structurally designed to release flows at the specified rates without structural or hydraulic failure. Sizing guidance is provided earlier in this chapter with additional guidance in Volume 3 of the USDCM.

The most common design consists of a configuration that releases the WQCV (Zone 1) and the balance of the EURV (Zone 2) through an orifice plate (typically a steel plate containing a vertical column of small, equally-spaced orifices. The 100-year volume above the EURV (Zone 3) is then controlled by an orifice at the bottom of the outlet vault structure, or drop box, after spilling over the crest of the drop box. The crest of the drop box acts as a weir and its length, as well as the size of the drop box opening, needs to be oversized to account for flow area reduction by the safety grate bars and blockage by debris. Figure OS-1 in Volume 3 of the USDCM provides guidance for determining initial minimum trash rack sizes for an outlet structure. Values from this figure account for clogging and metalwork losses through the safety grate. In addition to using Figure OS-1, also ensure that the velocity through the grate unhindered by debris blockage does not exceed 2 feet per second.

Drop box Design Considerations

Considerations for the cost and appearance of the structure can limit the size of the drop box. However, it is important to consider maintenance access and ensure that neither the crest of the box nor the safety grate (even when partially clogged) is limiting flow to the 100-year orifice.

Safety considerations (pinning by impingement velocity through the grate) may also dictate a larger structure. Use Figure OS-1 in Volume 3 of the USDCM to size the grate while separately ensuring that velocity does not exceed 2 feet per second through the safety grate in its unclogged condition.

Additionally, UDFCD recommends providing a rail in any location where a drop exceeds 3-feet.

Design procedures for analyzing drop box hydraulics and accounting for debris blockage are described in Sections 5.5.1 through 5.5.4. Additional discussion regarding safety grates and debris blockage can be found in Section 5.6.

The hydraulic capacity of the various components of the outlet works (orifices, weirs, pipes) can be determined using the UD-Detention Excel workbook, or other standard hydraulic equations. 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. The following sections describe procedures to generate a rating curve for four example types of 100-year drop box outlet structures. See Volume 3 of the USDCM for sizing the water quality orifices and incorporating water quality features into the outlet structure.

5.5.1 Flush Safety Grate

A flush grate drop box is a grate, either bar or close mesh, that is flush with the top of the box opening. The box opening may be horizontal or constructed with the slope of the embankment (as shown in Figure 12-15).

Evaluate the top of the outlet box for both weir (A) and orifice (B) flow at increasing water depths. The lesser of the two calculated flow values will indicate which controls for a given depth. Detailed discussion regarding weir and orifice hydraulics are in Section 5.14. Apply the net weir length and orifice open area, considering blockage by grating and potential debris, as discussed in Section 5.6. UDFCD contracted with Bureau of Recreation (USBR) to construct a physical model to refine weir/orifice calculations for a sloping drop box. Equations within the UD-Detention workbook equations are based on the USBR physical model. Documented at www.udfcd.org.

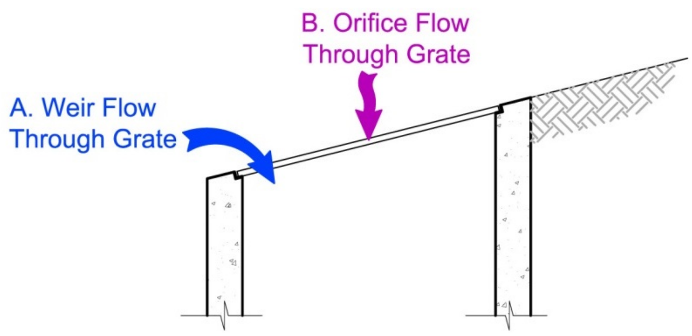


Figure 12-15. Flush grate (sloping drop box shown)

5.5.2 Raised Grate with Multiple Vertical Openings

A raised grate with multiple vertical openings offers improved flow capacity and resistance to debris blockage. It has vertical openings (open bar or close mesh) on two to four sides. See Figure 12-16 for a graphical representation of this grate configuration.

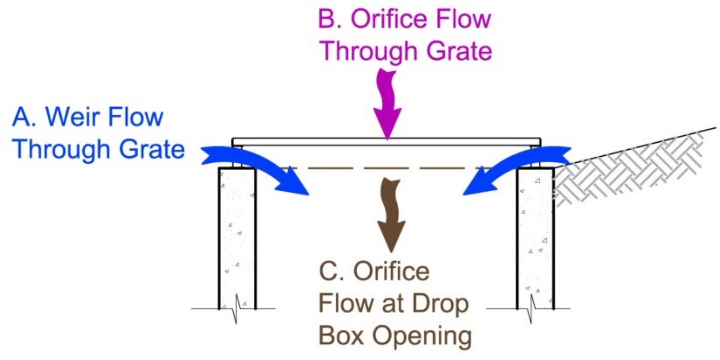


Figure 12-16. Grate with vertical openings (horizontal drop box shown)

This outlet must be evaluated for the two separate flow conditions (listed below and shown in Figure 12-16) to determine which controls at each incremental depth:

- **A. Weir Flow:** Calculate weir flow using the drop box interior perimeter reduced for the vertical grate supports and a 10% perimeter reduction for clogging.
- **B. Orifice Flow:** Calculate orifice flow using the smaller of the interior drop box area or the total grate area reduced for metalwork and debris clogging.

5.5.3 Raised Safety Grate with Vertical Opening

A grate with one vertical opening may also offer improved flow capacity and resistance to debris blockage. Figure 12-17 provides a graphical representation of this grate configuration.

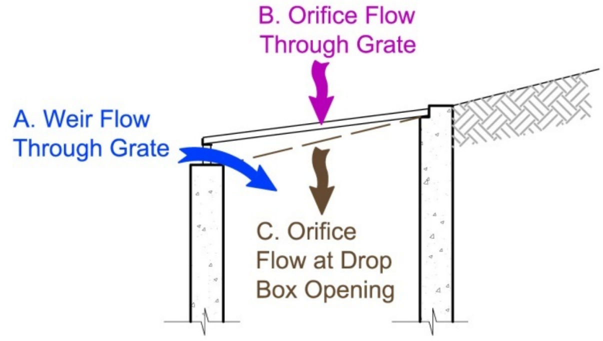


Figure 12-17. Grate with vertical opening (sloping drop box shown)

This outlet must be evaluated for the two separate flow conditions (listed below and shown in Figure 12-17) to determine which controls at each incremental depth:

- **A. Weir Flow:** Calculate weir flow using the average of the net perimeter length (i.e., reduced for metalwork and clogging) calculated from the condition shown in Figure 12-15 and Figure 12-16.

- **B. Orifice Flow:** Calculate orifice flow using the smaller of the interior drop box area or the total grate area reduced for metalwork and debris clogging. To account for clogging, the vertical grate net opening area should be reduced by 10%, while the horizontal grate net opening should be reduced by 50%. To simplify orifice calculations at various stages for a vertical or sloping grate, use the UD-Detention workbook.

5.5.4 Raised Grate with Offset Vertical Openings

A grate with offset vertical openings is a bar or close mesh grate that is elevated and extends beyond the sidewalls of the concrete outlet structure. This results in a vertical and horizontal gap between the grate and the walls of the drop box on all four sides of the structure and provides grate area below floating debris similar to a micropool design (See Volume 3 of the USDCM). Figure 12-18 shows a horizontal grate configuration. The grate could also be sloped.

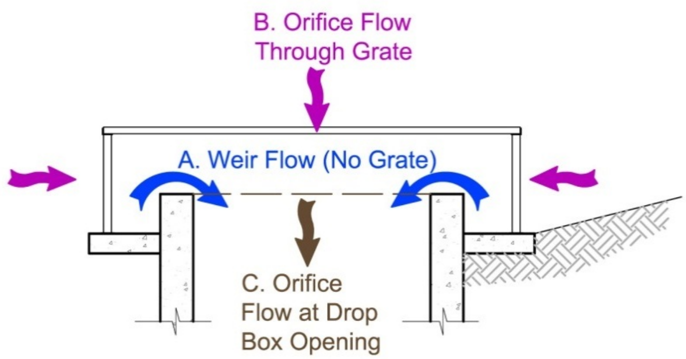


Figure 12-18. Grate with offset vertical panels (horizontal drop box shown)

This outlet must be evaluated for three separate flow conditions (listed below and shown in Figure 12-18) to determine which controls at each incremental depth:

- **A. Weir Flow:** Calculate weir flow over the walls of the drop box using the smaller of the unclogged drop box perimeter or the grate perimeter reduced for metalwork and 10% debris clogging.

- **B. Orifice Flow:** Calculate orifice flow using the smaller of the interior drop box area or the total grate area reduced for metalwork and debris clogging.

5.5.5 Outlet Pipe Hydraulics

Once the hydraulics of the top of a drop box are evaluated using the procedures discussed in Sections 5.5.1 through 5.5.4, the capacity of the outlet pipe and its orifice plate flow restrictor must be determined for increments of increasing water depth. The discharge pipe of the outlet works should be evaluated to ensure it is not under outlet control as a culvert at the 100-year (or design) discharge, and the orifice plate covering the opening of this pipe in the bottom of the drop box should be evaluated to ensure it limits flow to the required release rate. See the *Culverts* chapter for guidance regarding the calculation of the hydraulic capacity of outlet pipes. The UD-Culvert workbook can be used to determine the controlling condition of the culvert downstream of the orifice flow restrictor plate, while the UD-Detention workbook was designed to simplify these tasks.

The stage-discharge relationship of the outlet pipe and orifice is then compared to the controlling stage-discharge relationship for the top of the drop box plus flow through the water quality/EURV orifices may also be added. The ultimate control of the outlet is the smaller value of the flow through the top of the drop box plus water quality/EURV orifices, and the flow through the outlet pipe orifice over the range of stage. The design goal is that the outlet pipe orifice controls flow for the 100-year event, and the grate controls for more frequent return periods.

Determining the final hydraulics of the outlet structure becomes an iterative process. A final stage discharge curve is determined by completing the steps outlined above. This stage-discharge curve and the basin geometry are then input into the UD-Detention workbook or a SWMM model to evaluate hydrograph routing and the associated maximum stage, storage volume, and release rate. Often times it will be necessary to adjust the dimensions of the outlet box or the restrictor plate and orifice area of the outlet pipe to achieve the desired outflow from the basin. The goal is to have the 100-year orifice at the bottom of the box in front of the outlet pipe control the 100-year release rate at the maximum stage, not the hydraulic condition at the top of the outlet box. A final check on the overall safety of the outlet should be made to ensure that the velocity of flow through the grate open area reduced for metalwork but not for clogging does not exceed 2 ft/s.
.

5.6 Trash Racks and Debris Blockage

Trash racks should always be installed as part of an outlet structure to reduce safety concerns. Consider maintenance of the structure and potential access by the public when selecting the type of trash rack. For example, a close mesh grate will be more appropriate in high pedestrian traffic but will require more frequent maintenance as it will catch smaller debris. Trash racks of sufficient size should always be provided on an outlet structure so that they do not interfere with the hydraulic capacity of the outlet. See figure OS-1 in Chapter 4 of Volume 3 of this manual for the minimum open area based on the outlet size.

Typically, outlet structure safety grates consist of either a bar grate, a close mesh grate or an open grate as shown in Figure 12-19 below. Close mesh and bar grates can be used for horizontal, sloping or vertical surfaces. Open grates are typically only used along vertical openings, as shown in Figures 12-16 through 12-18 of Section 5.5. Figure 12-19 provides typical dimensions for the three aforementioned grates. The open area of the grate is typically provided by the manufacturer for prefabricated grates. Alternatively, this can be calculated. It is always appropriate to apply a debris blockage reduction. This is typically 50%. In some cases, it may be appropriate to increase or decrease this value based upon the potential for debris at a specific site. Considerations should include land cover and the type of grate at a minimum.

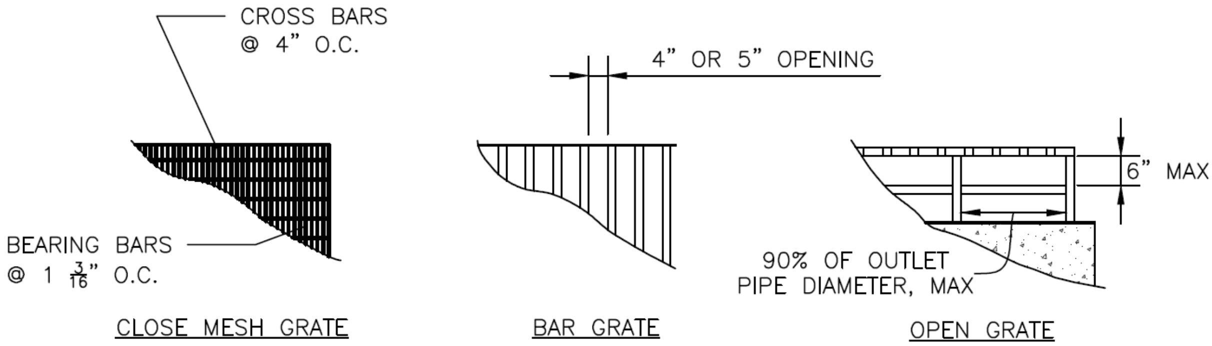


Figure 12-19. Typical grate configurations for outlet structures

5.7 Inlets

Inlets should provide energy dissipation to limit erosion. They should be designed in accordance with drop structure or pipe outlet criteria in the *Hydraulic Structures* chapter of the USDCM, or using other energy dissipation structures as appropriate. Additionally, forebays or sediment traps are recommended to provide a location to remove coarse sediment from the system prior to it being deposited in the vegetated area of the basin. Forebays need regular monitoring and maintenance.

5.8 Vegetation

The type of grass used in vegetating a newly constructed storage facility is a function of the frequency and duration of inundation of the area, soil types, and the other potential uses (park, open space, etc.) of the area. UDFCD recommends use of native grasses to reduce frequency and cost of maintenance and help maintain infiltration rates. See the *Revegtation* chapter for detailed information on establishing vegetation, including soil testing and amendments, seed mixes, and plantings. A planting plan should be developed for new facilities to meet their intended use and setting in the urban landscape. Trees and shrubs are not recommended on dams or fill embankments. However, use of trees immediately outside of detention basins will not interfere with their flood control operation or increase maintenance needs significantly. Also, sparse planting of trees basins may also be acceptable as long as they are not located near inlets and outlet or on the emergency spillway(s) and will not interfere significantly with maintenance or create clogging problems with the water quality screen. On the other hand, use of shrubs on the banks and bottom, while not affecting the flood routing, can increase maintenance significantly by providing traps for a source of debris and obstructing maintenance procedures. Because storage facilities are frequently wet, they are ideal nurseries for invasive and undesirable plants such as Siberian Elms, Russian Olives, Tamarisk, etc. This unplanned vegetation should be removed annually.

5.9 Retaining Walls

The use of retaining walls within detention basins is generally discouraged due to the potential increase in long-term maintenance access and costs as well as concerns regarding the safety of the general public and maintenance personnel. Where walls are used, limit the length of the retaining walls to no more than 50 percent of the basin perimeter. Also, consider potential fall hazards associated with pedestrians, cyclists, and vehicles in determining the appropriate treatment between a sidewalk, path, or roadway and the top of the wall. Considerations include distance from the public to the wall, curvature of the path or roadway, single or terraced walls, surrounding land use, and volume of traffic. Potential solutions include dense vegetation, seat walls, perimeter fencing, safety railing and guardrail. In some cases walls less than 2 feet will warrant a hard vertical barrier; in other cases a 3-foot wall may be the point at which this barrier is appropriate. Check requirements of the local jurisdiction. UDFCD recommends providing a hard vertical barrier in any location where walls exceed 3-feet.

Adequate horizontal separation between terracing walls should be provided to ensure that each wall is loaded by the adjacent soil, based on conservative assumptions regarding the angle of repose. When determining the separation between walls, consider the proposed anchoring system and the required equipment/space needed to repair the wall in the event of a failure. Ensure that failure and repair of any wall does not impact loading on adjacent walls. Separation between adjacent walls should be at least twice the adjacent wall height, such that a plane extended through the bottom of adjacent walls would not be steeper than a 2(H):1(V) slope. Slope of finished grade between walls should not exceed 4 percent. Wall designs exceeding these criteria or exceeding a height of 30 inches should only be performed by a Professional Engineer and should include a structural analysis for the design, evaluating the various loading conditions that the wall may encounter. Also consider a drain system behind the wall to ensure that hydrostatic pressures are equalized as the water level changes in the basin.

5.10 Access

All weather stable maintenance access shall be provided to elements requiring periodic maintenance. Guidance for equipment access to water quality components is discussed in Volume 3 of the USDCM. This guidance may also be relevant for flood control (only) facilities.

5.11 Geotechnical Considerations

The designer must take into full account the geotechnical conditions of the site. These considerations include issues related to ground water elevation, embankment stability, geologic hazards, seepage, and other site-specific issues.

It may be necessary to confer with a qualified geotechnical engineer during both design and construction, especially for the larger detention and retention storage facilities.

5.12 Linings

Sometimes an impermeable clay or synthetic liner is necessary. Stormwater detention and retention facilities have the potential to raise the groundwater level in the vicinity of the basin. Where there is concern for damage to adjacent structures due to rising ground water, consider lining the basin with an impermeable liner. An impermeable liner may also be warranted for a retention pond where the designer seeks to limit seepage from the permanent pool. Note that if left uncovered, synthetic lining on side slopes creates a serious impediment to egress and a potential drowning hazard. See the Retention Pond Fact Sheet in Volume 3 of the USDCM for guidance and benefits associated with the constructing a safety wetland bench.

5.13 Environmental Permitting and Other Considerations

The designer must take into account environmental considerations surrounding the facility and the site during its selection, design and construction. These can include regulatory issues such as:

- If construction will create disturbance or otherwise modify a jurisdictional wetland,
- If the facility is to be located on a waterway that is regulated by the U.S. Army Corps of Engineers as a “Water of the U.S.”, and

- If there are threatened and endangered species or habitat in the area.

There are also non-regulatory environmental issues that should be considered. UDFCD recommends early discussions with relevant federal, state and local regulators on these issues. Issues may include the following:

- Potential for encountering contaminated soils during excavation,
- Proper implementation of design elements to mitigate mosquito breeding (i.e., a micropool)
- Concern from area residents regarding the disturbance of existing riparian habitat that may be required for construction of the basin, and
- Colorado water rights issues related to large permanent pools or retention ponds.

5.14 Orifice and Weir Hydraulics

The following discussion regarding weirs and orifices is adapted from *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22, Third Edition (Brown et al., 2009).

5.14.1 Orifices

Multiple orifices may be used in a detention facility, and the hydraulics of each can be superimposed to develop the outlet-rating curve. For a single orifice or a group of orifices, orifice flow can be determined using Equation 12-7.

$$Q = C_o A_o (2gH_o)^{0.5}$$ Equation 12-7

Where:

Q = the orifice flow rate through a given orifice (cfs)

C_o = discharge coefficient (0.60 recommended for square-edge orifices)

A_o = area of orifice (ft^2)

H_o = effective head on each orifice opening (ft)

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

If the orifice discharges as a free outfall, the effective head is measured from the centroid of the orifice to the upstream water surface elevation. If the downstream jet of the orifice is submerged, then the effective head is the difference in elevation between the upstream and downstream water surfaces.

5.14.2 Weirs

Flow over a horizontal spillway or drop box crest can be calculated using the following equation for a horizontal broad-crested weir. See Figure 12-7 for a graphical representation of weir flow.

<u>Horizontal Broad-Crested Weir:</u> The equation typically used for a broad-crested weir is:

$$Q = C_{BCW} L H^{1.5}$$ Equation 12-8

Where:

Q = discharge (cfs)

C_{BCW} = broad-crested weir coefficient (This ranges from 2.6 to 3.0. A value of 3.0 is often used in practice.) See Hydraulic Engineering Circular No. 22 for additional information.

L = broad-crested weir length (ft)

H = head above weir crest (ft)

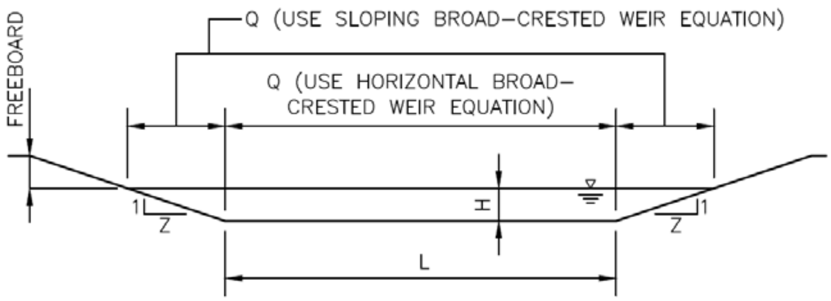


Figure 12-20. Sloping broad-crest weir

Sloping Broad-Crested Weir: Figure 12-20 shows an example of a sloping broad-crested weir. The equation to calculate the flow over the sloping portion of the weir is as follows:

$$Q = \left(\frac{2}{5}\right) C_{BCW} Z H^{2.5}$$ Equation 12-9

Where:

Q = discharge (cfs)

C_{BCW} = broad-crested weir coefficient (This ranges from 2.6 to 3.0. A value of 3.0 is often used in practice.) See Hydraulic Engineering Circular No. 22 for additional information.

Z = side slope (horizontal: vertical)

H = head above weir crest (ft)

Note that in order to calculate the total flow over the weir depicted in Figure 12-20, the results from Equation 12-8 must be added to two times the results from Equation 12-9.

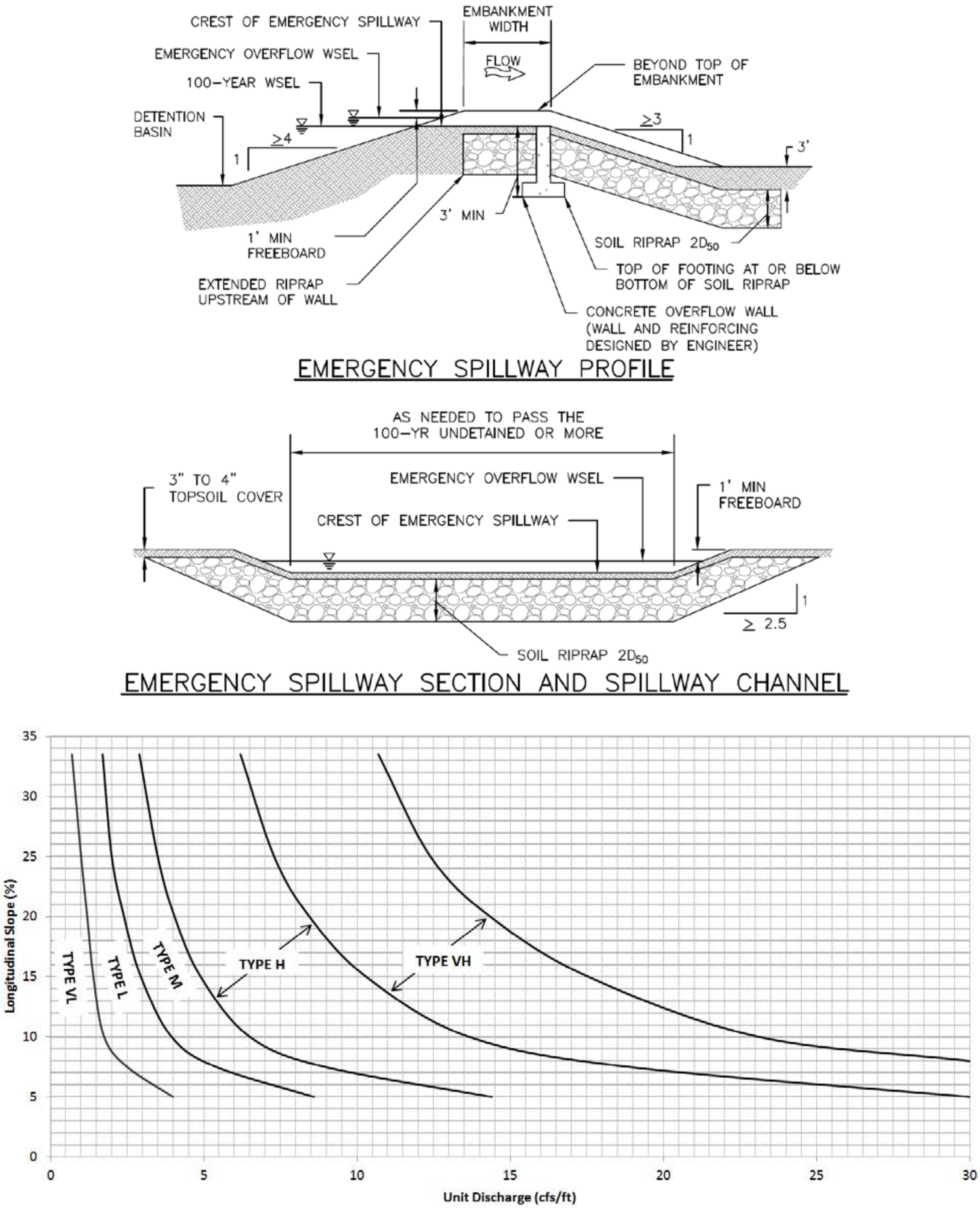


Figure 12-21. Embankment protection details and rock sizing chart (adapted from Arapahoe County)

6.0 Additional Configurations of Detention Facilities

In addition to regional, sub-regional, and onsite full spectrum detention facilities described in Section 2, there are a number of specialized types and configurations for storage that require special considerations.

6.1 Water Storage Reservoirs

Colorado State law specifically exempts the reliance of water storage reservoirs for flood control by downstream properties. If a project developer or local jurisdiction wants to utilize them for detention storage, some form of ownership of the flood storage pool and outlet function must be acquired from the reservoir owner. An agreement with the reservoir owner that ensures the continued existence of the facility or its detention function over time must be reached before relying on such reservoirs. It is also necessary to demonstrate that the embankment and spillway are safe and stable to ensure public safety.

6.2 Upstream of Railroad and Highway Embankments

Storage behind road, railroad, and other embankments can also be lost due to site grading and fill changes and/or the installation of larger culverts or bridges. If the designer intends to utilize roadway, railroad, or other embankments for detention storage, some form of ownership of the flood storage pool and control of the outlet must be acquired. An agreement with the roadway, railroad, or other agency that ensures the continued full flood protection benefit of the facility over time must be reached before relying on the facility. In addition, it is necessary to demonstrate that 1) roadway, railroad or other embankment stability will not be compromised, 2) embankment overtopping during larger storms will not impact upstream or downstream properties, and 3) the storage facility will remain in place as a detention facility in perpetuity.

6.3 Side-Channel Detention Basins

Also referred to as offline detention, this type of storage facility is located immediately adjacent to a stream and depends on a diversion of some portion of flood flows out of the waterway into the detention basin, typically over a side-channel spillway. These facilities can be used to “shave the peak” off of a flood hydrograph and can potentially be smaller and store water less frequently than on-line facilities. These facilities do not include WQCV or EURV and therefore address only flood peak reduction. They generally have limited application, but may be one of the storage alternatives considered during watershed master planning studies.

6.4 Parking Lot Detention

Parking lot islands or adjacent landscape areas can be desirable locations to provide WQCV or even EURV; however, it is recommended that the maximum water surface for WQCV or EURV be kept below the elevation of the pavement surface.

It is more problematic to provide100-year detention within parking lots given the inconvenience imposed by ponding water in areas of vehicle and pedestrians use. If 100-year parking lot detention is allowed by local jurisdictions, depth limitations and signage requirements should be considered carefully.

6.5 Underground Detention

Because of the problems associated with placing detention “out of sight”, the difficulty and hazardous nature of access for maintenance, seepage concerns, and uncertain design life for vessels subject to corrosion, underground detention is not recommended by UDFCD. Some local jurisdictions may allow underground 100-year detention in limited high-density urban developments; in those cases, careful consideration must be given to requirements to ensure ongoing inspection, maintenance, and functionality.

6.6 Blue Roofs

A blue roof is a rooftop designed to provide detention. Rooftop detention was removed from this manual as part of a previous update because conventional systems could be easily manipulated by maintenance personal that viewed standing water on a roof as problematic and would make adjustments to the outlet resulting in loss of detention. Depending on the design, blue roofs can be successful in providing storage and slow release of the WQCV or larger events. To ensure long-term maintenance, the design should both appear as an intentional part of the roof design and should not be easily bypassed.

Photograph 12-4. This blue roof system utilizes trays. The design appears as an intentional feature to the lay person. Additionally, the design is such that it cannot be easily manipulated. Photo courtesy of Geosyntec.

6.7 Retention Facilities

Retention facilities (basins with a zero release rate or a very slow release rate) have been used in some instances as temporary measures when there is no formal downstream drainage system, or one that is grossly inadequate, until an adequate system is developed. However, these facilities are problematic on a number of levels. Sizing these facilities for a given set of assumptions does not ensure that another scenario produced by nature (e.g., a series of small storms that add up to large volumes over a week or two) will not overwhelm the intended design. In addition, water rights concerns and problems associated with standing water make these facilities undesirable. For these reasons, retention basins are recommended by UDFCD only as a choice of last resort.

After taking into consideration the concerns summarized above, if a retention facility is to be designed and constructed then UDFCD recommends the following design parameters. The retention facility should be sized to capture, as a minimum, 2.0 times the 24 hour, 100-year storm plus 1 foot of freeboard.

7.0 Designing for Safety, Operation, and Maintenance

Maintenance considerations during design include the following (adapted from ASCE and WEF 1992).

1. Use of mild side slopes (e.g., no steeper than 4(H):1(V)) along the banks and installation of landscaping that will discourage entry along the periphery near the outlets and steeper embankment

sections are advisable. Also, use of safety railings at vertical or very steep structural faces. If the impoundment is situated at a lower grade than and adjacent to a highway, installation of a guardrail is in order. Providing features to discourage public access to the inlet and outlet areas of the facility should be considered.

2. The facility should be accessible to maintenance equipment for removal of silt and debris and for repair of damages that may occur over time. Easements and/or rights-of-way are required to allow access to the facility by the owner or agency responsible for maintenance.

3. Permanent ponds should have provisions for complete drainage for sediment removal (or other means of sediment removal). The frequency of sediment removal will vary among facilities, depending on the original volume set aside for sediment, the rate of accumulation, rate of growth of vegetation, drainage area erosion control measures, and the desired aesthetic appearance of the pond.

4. For multiuse facilities, especially those intended for active recreation, the play area might need special consideration during design to minimize the frequency and periods of inundation and wet conditions. It may be advisable to provide an underground drainage system if active recreation is contemplated.

5. Adequate dissolved oxygen supply in ponds (to minimize odors and other nuisances) may be able to be maintained by artificial aeration.

6. Use of fertilizer, pesticides and herbicides adjacent to the permanent pool pond and within the detention basin should be avoided (this includes EPA-approved pesticides and herbicides).

7. Secondary uses that would be incompatible with sediment deposits should not be planned unless a high level of maintenance will be provided.

8. French drains or the equivalent are almost impossible to maintain, and should be used with discretion where sediment loads are expected to be high.

9. Detention facilities should be designed with sufficient depth to allow accumulation of sediment based on a sustainable frequency of maintenance.

10. Often designers use fences to minimize hazards. These may trap debris, impede flows, hinder maintenance, and, ironically, fail to prevent access to the outlet. However, desirable conditions can be achieved through careful design and positioning of the structure, as well as through landscaping that will discourage access. Creative designs, integrated with innovative landscaping, can be safe and can also enhance the appearance of the outlet and basin. When developing the landscape plan also consider landscape maintenance requirements.

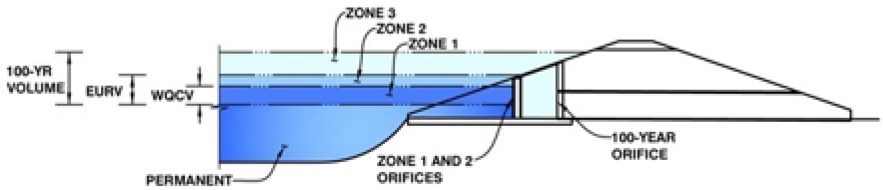
11. To reduce maintenance and avoid operational problems, outlet structures should be designed with no unmonitored moving parts (i.e., use only pipes, orifices, and weirs). Manually and/or electrically operated gates should be avoided unless equipped with remote monitoring and an emergency operation plan. To reduce maintenance, outlets should be designed with openings as large as possible, compatible with the depth-discharge relationships desired and with water quality, safety, and aesthetic objectives in mind. For the 100-year discharge, use a larger outlet pipe and install a restrictor plate (orifice) to reduce outflow rates. Outlets should be robustly designed to lessen the chances of damage from debris or vandalism.

See Volume 3 of the USDCM for additional recommendations regarding operation and maintenance of water quality related facilities.

8.0 Design Examples

8.1 Example - Design of a Full Spectrum Detention Sand Filter Basin using UD-Detention

Determine the required full spectrum detention volume and approximate area for a sand filter basin to receive runoff from 20 acres in Denver. The site is 75% impervious and has NRCS hydrologic soil group C/D. The watershed slope is 0.5% and the length of the watershed is 1300 feet.



Example Zone Configuration (Retention Pond)

Required Volume Calculation

Sand Filter (SF)	SF			
Watershed Area =	20.00	acres		
Watershed Length =	1,300	ft		
Watershed Slope =	0.005	ft/ft		
Watershed Imperviousness =	75.00%	percent		
Percentage Hydrologic Soil Group A =	0.0%	percent		
Percentage Hydrologic Soil Group B =	0.0%	percent		
Percentage Hydrologic Soil Groups C/D =	100.0%	percent		
Desired WQCV Drain Time =	12.0	hours		
Location for 1-hr Rainfall Depths =	Denver - Capitol Building			
Water Quality Capture Volume (WQCV) =	0.399	acre-feet	Optional User Override	
Excess Urban Runoff Volume (EURV) =	1.466	acre-feet	1-hr Precipitation	
2-yr Runoff Volume (P1 = 0.83 in.) =	0.977	acre-feet		inches
5-yr Runoff Volume (P1 = 1.09 in.) =	1.406	acre-feet		inches
10-yr Runoff Volume (P1 = 1.33 in.) =	1.776	acre-feet		inches
25-yr Runoff Volume (P1 = 1.69 in.) =	2.434	acre-feet		inches
50-yr Runoff Volume (P1 = 1.99 in.) =	2.945	acre-feet		inches
100-yr Runoff Volume (P1 = 2.31 in.) =	3.557	acre-feet		inches
500-yr Runoff Volume (P1 = 3.14 in.) =	5.040	acre-feet		inches
Approximate 2-yr Detention Volume =	0.917	acre-feet		
Approximate 5-yr Detention Volume =	1.324	acre-feet		
Approximate 10-yr Detention Volume =	1.591	acre-feet		
Approximate 25-yr Detention Volume =	1.878	acre-feet		
Approximate 50-yr Detention Volume =	2.015	acre-feet		
Approximate 100-yr Detention Volume =	2.231	acre-feet		

Enter the watershed parameters into the blue user input cells in the *Basin* tab. A drop down box allows the user to indicate the location. Alternatively, the user may enter their own 1- hour precipitation values. The worksheet calculates the runoff and detention volumes and populates the remaining cells, as shown above.

Stage-Storage Calculation

Zone 1 Volume (WQCV)	0.399	acre-feet
Zone 2 Volume (EURV - Zone 1)	1.066	acre-feet
Zone 3 Volume (100-year - Zones 1 & 2)	0.765	acre-feet
Total Detention Basin Volume =	2.231	acre-feet
Initial Surcharge Volume (ISV) =	N/A	ft^3
Initial Surcharge Depth (ISD) =	N/A	ft
Total Available Detention Depth (H_{total}) =	4.00	ft
Depth of Trickle Channel (H_{TC}) =	N/A	ft
Slope of Trickle Channel (S_{TC}) =	N/A	ft/ft
Slopes of Main Basin Sides (S_{main}) =	4	H:V
Basin Length-to-Width Ratio ($R_{L/W}$) =	2	
Initial Surcharge Area (A_{ISV}) =	0	ft^2
Surcharge Volume Length (L_{ISV}) =	0.0	ft
Surcharge Volume Width (W_{ISV}) =	0.0	ft
Depth of Basin Floor (H_{FLOOR}) =	0.00	ft
Length of Basin Floor (L_{FLOOR}) =	196.3	ft
Width of Basin Floor (W_{FLOOR}) =	98.1	ft
Area of Basin Floor (A_{FLOOR}) =	19,258	ft^2
Volume of Basin Floor (V_{FLOOR}) =	0	ft^3
Depth of Main Basin (H_{MAIN}) =	4.00	ft
Length of Main Basin (L_{MAIN}) =	228.3	ft
Width of Main Basin (W_{MAIN}) =	130.1	ft
Area of Main Basin (A_{MAIN}) =	29,702	ft^2
Volume of Main Basin (V_{MAIN}) =	97,168	ft^3
Calculated Total Basin Volume (V_{total}) =	**2.231**	acre-feet

Once the user defines each zone, the available depth for detention, basin side slopes, and length to width ratio (shown above), the workbook calculates the approximate basin geometry and volume and populates stage storage values based on this geometry and approximate routed volume.

Next, use the *Outlet Structure* tab to design outlet control for each zone of the detention basin. The workbook allows for several different outlet configurations. Filtration BMPs (i.e., sand filters and rain gardens) release the WQCV (Zone 1 for this example) through an underdrain. Zone 2 (EURV-WQCV for this example), will be drained through a circular orifice located immediately above the WQCV water surface elevation. Zone 3 (100-yr – EURV) will be released when water overtops the outlet structure (weir) and is restricted at the entrance to the outlet pipe. This example uses a restrictor plate. Selection of the outlet configuration is located at the top of the *Outlet* tab (see the screenshot below).

Clear Input Parameters (Including Tables)

	Stage (ft)	ne Volume (ac-	Outlet Type	
Zone 1 (WQCV)	0.86	0.399	Filtration Media	Filtration Media with Underdrain
Zone 2 (EURV)	2.81	1.066	Circular Orifice	Vertical Orifice (Circular)
Zone 3 (100-year)	4.00	0.765	Weir&Pipe (Restrict)	Weir and Pipe (w/ Restrictor Plate)
		2.231	Total	

To size the Zone 1 outlet, enter a value for "underdrain orifice invert depth" (depth from the top of the sand bed to the invert of the underdrain at the outlet). Press the "Calculate Underdrain Orifice Diameter to match WQCV Drain Time" button. The underdrain parameters are also calculated and shown below.

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

Underdrain Orifice Invert Depth =	2.00	ft (distance below the filtration media surface)
Underdrain Orifice Diameter =	3.23	inches

Calculate Underdrain Orifice Diameter to match WQCV Drain Time

(Blue cells in the next section are marked "N/A" because the user did not select this as an outlet type. Skip this section.)

Zone 2 will outlet from the basin through a circular orifice (see screenshot below). This orifice should be located immediately above the WQCV. This zone extends up to the EURV water surface elevation. The workbook pulls both of these values from the *Basin* tab. Note the stage\storage description in the first column of the table in the *Basin* tab. Press the "Size Vertical Orifice to drain (EURV – WQCV) only" and enter a value for the time to drain this volume. For this example, we specify 24 hours. The user can come back to this section any time and modify the drain time. This is typically done to meet desired drain times for various return periods.

User Input: Vertical Orifice (Circular or Rectangular)

	Zone 2 Circular	Not Selected	
Invert of Vertical Orifice =	0.86	N/A	ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice =	2.81	N/A	ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter =	0.38	N/A	inches

Calculated Parameters for Vertical Orifice

	Zone 2 Circular	Not Selected	
Vertical Orifice Area =	0.00	N/A	ft^2
Vertical Orifice Centroid =	0.02	N/A	feet

Size Vertical Orifice to drain (EURV - WQCV) only

Use the next section of the *Outlet Structure* tab to size the overflow weir and restrictor plate. Again, the appropriate overflow weir height populates automatically from the basin tab. This is the elevation of the EURV surface elevation in the basin. Fill in approximate values for the drop box. This example uses a square inside dimension of 4 feet for the drop box and a flat top.

Enter the depth of the invert of the outlet pipe along with reasonable values for the diameter and restrictor plate height. The workbook will resize these as needed to match a release of 90% of the predevelopment 100-year peak runoff rate per USDCM criteria. Press the "Size Outlet Pipe to match 90% of the Predevelopment 100-year Peak Runoff Rate" button. The workbook will adjust the size of the outlet pipe diameter, the height of the restrictor plate, and sizes an emergency spillway. See the screenshot below.

User Input: Overflow Weir (Dropbox) and Grate (Flat or Sloped)

	Zone 3 Weir	Not Selected	
Overflow Weir Front Edge Height, Ho =	2.81	N/A	ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length =	4.00	N/A	feet
Overflow Weir Slope =	0.00	N/A	H:V (enter zero for flat grate)
Horiz. Length of Weir Sides =	4.00	N/A	feet
Overflow Grate Open Area % =	70%	N/A	%, grate open area/total area
Debris Clogging % =	50%	N/A	%

Calculated Parameters for Overflow Weir

	Zone 3 Weir	Not Selected	
Height of Grate Upper Edge, H_t =	2.81	N/A	feet
Over Flow Weir Slope Length =	4.00	N/A	feet
Grate Open Area / 100-yr Orifice Area =	6.48	N/A	should be ≥ 4
Overflow Grate Open Area w/o Debris =	11.20	N/A	ft^2
Overflow Grate Open Area w/ Debris =	5.60	N/A	ft^2

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

	Zone 3 Restrictor	Not Selected	
Depth to Invert of Outlet Pipe =	3.00	N/A	ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter =	18.00	N/A	inches
Restrictor Plate Height Above Pipe Invert =	17.00		inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate

	Zone 3 Restrictor	Not Selected	
Outlet Orifice Area =	1.73	N/A	ft^2
Outlet Orifice Centroid =	0.73	N/A	feet
Half-Central Angle of Restrictor Plate on Pipe =	2.67	N/A	radians

Size Outlet Plate to match 90% of Predevelopment 100-year Peak Runoff Rate

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage=	3.80	ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length =	16.00	feet
Spillway End Slopes =	4.00	H:V
Freeboard above Max Water Surface =	1.00	feet

Size Emergency Spillway to pass Developed 100-yr Peak Runoff Rate

Calculated Parameters for Spillway

Spillway Design Flow Depth=	0.88	feet
Stage at Top of Freeboard =	5.68	feet
Basin Area at Top of Freeboard =	0.80	acres

The workbook provides output related to the drain time for each storm frequency, the ratio of peak outflow to predevelopment flow, and other pertinent information.

Routed Hydrograph Results

Design Storm Return Period =	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
One-Hour Rainfall Depth (in) =	0.53	1.07	0.83	1.09	1.33	1.69	1.99	2.31	3.14
Calculated Runoff Volume (acre-ft) =	0.399	1.466	0.977	1.406	1.776	2.434	2.945	3.557	5.040
OPTIONAL Override Runoff Volume (acre-ft) =									
Inflow Hydrograph Volume (acre-ft) =	0.398	1.465	0.976	1.406	1.774	2.433	2.945	3.556	5.030
Predevelopment Unit Peak Flow, q (cfs/acre) =	0.00	0.00	0.01	0.08	0.23	0.60	0.82	1.12	1.77
Predevelopment Peak Q (cfs) =	0.0	0.0	0.2	1.6	4.6	11.9	16.5	22.4	35.4
Peak Inflow Q (cfs) =	5.4	19.7	13.2	18.9	23.8	32.4	39.1	47.1	66.2
Peak Outflow Q (cfs) =	0.4	0.6	0.5	0.6	2.5	11.5	18.1	20.4	39.1
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.4	0.5	1.0	1.1	0.9	1.1
Structure Controlling Flow =	Filtration Media	Vertical Orifice 1	Vertical Orifice 1	Vertical Orifice 1	Overflow Grate 1	Overflow Grate 1	Overflow Grate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	0.2	1.0	1.6	1.8	1.8
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	12	35	25	34	38	38	37	37	36
Time to Drain 99% of Inflow Volume (hours) =	12	36	26	35	39	39	39	39	39
Maximum Ponding Depth (ft) =	0.69	2.61	1.78	2.51	2.95	3.27	3.45	3.74	4.29
Area at Maximum Ponding Depth (acres) =	0.48	0.59	0.54	0.59	0.61	0.63	0.65	0.66	0.70
Maximum Volume Stored (acre-ft) =	0.318	1.347	0.870	1.288	1.552	1.752	1.861	2.057	2.426

The maximum ponded area for this design example is 0.66 acres, while the maximum volume stored is 2.06 ac-ft.

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