STORAGE

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1.0 OVERVIEW

This chapter provides guidance for the analysis and design of stormwater *quantity* detention facilities. Detention facilities for the management of stormwater *quality* (i.e., extended detention basins, retention ponds, wetland basins, etc.) are described in Volume 3 of this *Manual*. Detention and retention basins are used for stormwater runoff quantity control to mitigate the effects of urbanization on runoff flood peaks. If there is a need to design a storage facility for both water quality and quantity control purposes, this chapter should be used in conjunction with Volume 3 of this *Manual*.

Topics discussed in this chapter include design storms used for detention, the application of different types of storage facilities, basis for hydrologic and hydraulic design, and various other design considerations. As is the case with major drainageways, the Urban Drainage and Flood Control District (District) strongly encourages the development of multipurpose, attractive detention facilities that are safe, maintainable and viewed as community assets rather than liabilities.

Photograph SO-1—Attractive wet and dry detention facilities in commercial settings have been shown to increase property value.

Photograph SO-2—Dry and extended dry detention facilities can blend into the landscape, especially with the assistance of experienced landscape architects.

2.0 APPLICATION OF DIFFERENT TYPES OF STORAGE

There are two basic approaches to designing storage facilities. When runoff storage facilities are planned on an individual-site basis, they are referred to as “on-site.” Larger facilities that have been identified and sized as a part of some overall regional plan are categorized as “regional” facilities. The regional definition can also be applied to detention facilities that encompass multiple land development projects.

On-site storage facilities usually are designed to control runoff from a specific land development site and are not typically located or designed with the idea of reducing downstream flood peaks along major drainageways for small and/or large storm runoff events. The total volume of runoff detained in the individual on-site facility is quite small, and the detention time for flood control purposes is relatively short. Therefore, unless design (i.e., sizing and flow release) criteria and implementation are applied uniformly throughout the urbanizing or redeveloping watershed, their effectiveness diminishes rapidly along the downstream reaches of waterways. 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 along major drainageways (Glidden 1981; Urbonas and Glidden 1983).

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 larger regional facility(ies) 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 at levels they were design to provide. Prommesberger (1984) inspected 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. He concluded that a lack of adequate maintenance and implementation contributed to a loss of continued function or even presence of these facilities. He also concluded that a lack of institutional structures at the local government level was the major contributor to any of these facilities no longer being in existence or in original operational mode after their initial construction.

Photograph SO-3—On-site storage facility serving town home development (in background) coupled with park.

Facilities designed as part of a watershed planning process, in which the stormwater management needs for the watershed as a whole are developed in a staged regional plan, are called regional facilities. These are often planned and located as part of the District's master planning process. They are typically much larger than on-site facilities. The main disadvantage of the regional facilities is the lack of an institutional structure to fund their implementation early in the development process. Another disadvantage of regional facilities is that they can leave substantial portions of the stream network susceptible to increased flood peaks, and plans must be developed to take this condition into account. In addition, to promote water quality benefits, some form of on-site stormwater management is necessary upstream of the regional facilities. Examples include minimized directly connected impervious areas (MDCIA) that promote flow across vegetated surfaces utilizing "slow-flow" grassed swales and a number of other techniques described in Volume 3 of this *Manual* that reduce stormwater surface runoff volumes.

More economical and hydrologically reliable results can be achieved through stormwater management planning for an entire watershed that incorporates the use of regional facilities. Regional facilities also potentially offer greater opportunities in achieving multi-objective goals such as recreation, wildlife habitat, enhanced property value, open space, and others.

There are several types of stormwater storage facilities, whether they are classified as on-site or regional, namely:

1. Detention—Detention facilities provide temporary storage of stormwater that is released through an outlet that controls flows to pre-set levels. Detention facilities typically flatten and spread the

inflow hydrograph, lowering the peak to the desired (i.e., master plan, pre-development, existing, etc.) flow rate. Often these facilities also incorporate features designed to meet water quality goals.

2. Retention—Retention facilities, as defined this chapter, store stormwater runoff without a positive outlet, or with an outlet that releases water at very slow rates over a prolonged period. These differ in nature and design from “retention ponds” described in Volume 3 of this *Manual* that are used for water quality purposes.

3. Conveyance (Channel) Storage—Conveyance, or channel routing, is an often-neglected form of storage because it is dynamic and requires channel storage routing analysis. Slow-flow and shallow conveyance channels and broad floodplains can markedly retard the build up of flood peaks and alter the time response of the tributaries in a watershed.

4. Infiltration Facilities—Infiltration facilities resemble retention facilities in most respects. They retain stormwater runoff for a prolonged period of time to encourage infiltration into the groundwater. These facilities are difficult to design and implement because so many variables come into play.

5. Other Storage Facilities—Storage can occur at many locations in urban areas for which special considerations typically apply to the use and reliance upon such conditions. Stormwater detention may occur at:

   a) *Random depressions*. Depressions can be filled in during development and cannot be relied upon as permanent.

   b) *Upstream of railroad and highway embankments*. If the designer intends to utilize roadway, railroad, or other embankments for detention storage, some form of ownership of the flood storage pool and some form of control of the outlet must be acquired. An agreement with the roadway, railroad, or other agency that insures the continued existence of the facility over time has to 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. 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.

   c) *Water storage reservoirs.* Colorado State law specifically exempts the reliance of water storage reservoirs for flood control by downstream properties. If the designer or project developers want 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 insures the continued existence of the facility or its detention function over time has to be reached before relying on such reservoirs. In addition, it is necessary to demonstrate that embankment and spillway are safe and stable to insure public safety.

It is beyond the scope of this *Manual* to address these kinds of specialized storage facilities in more detail, but readers are cautioned that the above-mentioned considerations must be taken into account before proposing their use as formal detention facilities.

Detention and retention facilities can be further subdivided into:

1. In-Line Storage—A facility that is located in-line with the drainageway and captures and routes the entire flood hydrograph. A major disadvantage with in-line storage is that it must be large enough to handle the total flood volume of the entire tributary catchment, including off-site runoff, if any.

2. Off-Line Storage—A facility that is located off-line from the drainageway and depends on the diversion of some portion of flood flows out of the waterway into the storage facility. These facilities can be smaller and potentially store water less frequently than in-line facilities.

Irrespective of which type of storage facility is utilized, the designer is encouraged to create an attractive, multipurpose facility that is readily maintainable and safe for the public, under both “dry” (i.e., dry weather) and “wet” (i.e., when runoff is occurring) conditions. Designers are also encouraged to consult with other specialists such as urban planners, landscape architects, and biologists during planning and design.

3.0 HYDROLOGIC AND HYDRAULIC DESIGN BASIS

3.1 Procedures for the Sizing of Storage Volumes

Three procedures for the sizing of detention storage volumes and one for the sizing of retention storage volumes are described in this *Manual*. For detention facilities, two of the procedures may be applied to on-site facilities and facilities serving relatively small tributary areas. For detention facilities serving larger catchments or ones classified as regional, the *Manual* recommends only one design protocol for use within the District, as described below.

3.1.1 Use of Simplified On-Site Detention Sizing Procedures

The three simplified procedures for the sizing of on-site detention volumes described here are “empirical equations” (Section 3.2.2), the modified “Rational Formula-based FAA Method” (Section 3.2.3) and “Full-Spectrum Detention” (Section 3.2.4). The uses of empirical equations by themselves are only applicable for small catchments not exceeding 90 acres. The Rational Formula-based FAA procedure may be applied to tributary catchments up to 160 acres in size, but the District suggests that it is best to limit ther use to tributary areas of 90 acres or less.

The Excess Urban Runoff Control, called *Full Spectrum Detention*, method may be applied to catchments of up to one-square mile in size; however, the simplified approach described in this chapter, including the use of the spreadsheet for this method, is best limited to areas of 160 acres or less.

3.1.2 Use of Hydrograph Routing Detention Sizing Procedure

Whenever the area limits described above in Section 3.1.1. are exceeded (for tributary catchments larger than 90 acres for empirical equations and FAA Method and 160 acres for the *Full Spectrum Detention* method), the District recommends the use of hydrograph flood routing procedures (e.g., using CUHP-generated hydrographs and reservoir routing calculations). In addition, if there are upstream detention facilities in the watershed that catch and route runoff for portions of the upstream tributary area, hydrograph routing methods should be employed.

To be considered as a sub-regional or regional facility by the District, namely part of the major drainageway system, the detention basin has to have a tributary area of 130 acres or more.

If off-site tributary areas contribute runoff to an on-site detention facility, the total tributary area, assuming fully developed off-site land uses, must be included in the sizing of the on-site storage volumes in order to account for the total runoff volume in the watershed.

3.1.3 Water Quality Capture Volume in Sizing Detention Storage

When detention storage volume is sized for a site that also incorporates a water quality capture volume (WQCV) defined in Volume 3 of this *Manual*, check with the local jurisdiction to determine how to account for this volume. Some municipalities within the District will permit partial or full use of the WQCV within the calculated 100-year volume. Others require that the 100-year volume be added to the WQCV. All jurisdictions require the WQCV be added to the 5- or 10-year volume. When clear written local criteria on this matter are absent, the District recommends that no less than 50% of the WQCV be added to the calculated 100-year volume for 100-year volumes obtained using empirical equations and the FAA Method. However, unless the local jurisdiction requires adding all or part of the WQCV to the 100-year volume obtained using the simplified *Full Spectrum Detention* design; District does not recommend adding any part of the WQCV to the 100-year volume. When the analysis is done using hydrograph routing methods, each level of controls needs to be accounted for and the resultant 100-year control volume used in final design.

3.2 Sizing of On-Site Detention Facilities

3.2.1 Maximum Allowable Unit Release Rates for On-Site Facilities

The maximum allowable unit release rates in the Denver area per acre of tributary catchment for on-site detention facilities for various design return periods are listed in Table SO-1. These maximum releases rates will apply for all on-site detention facilities unless other rates are recommended in a District-approved master plan. For regional facilities see Section 3.2.5.

Allowable unit release rates in Table SO-1 for each a soil group in the tributary catchment shall be area-weighted to composite the allowable unit release rate for the total catchment. Multiply this rate by the total tributary catchment's area to obtain the design release rates in cubic feet per second (cfs). Whenever Natural Resources Conservation Service (NRCS) soil surveys are not available, approximate their equivalent types using results of detailed soil investigations at the site.

Table SO-1—Maximum Unit Flow Release Rates (cfs/acre) from On-Site Detention Facilities

Design Return Period (Years)	NRCS Hydrologic Soil Group		
	A	B	C & D
2	0.02	0.03	0.04
5	0.07	0.13	0.17
10	0.13	0.23	0.30
25	0.24	0.41	0.52
50	0.33	0.56	0.68
100	0.50	0.85	1.00

3.2.2 Empirical Equations for the Sizing of On-Site Detention Storage Volumes

Urbonas and Glidden (1983), as part of the District's ongoing hydrologic research, conducted studies that evaluated peak storm runoff flows along major drainageways. The following set of empirical equations provided preliminary estimates of on-site detention facility sizing for areas within the District. They are intended for single return period control and not for use when off-site inflows are present or when multi-stage controls are to be used (e.g., 10- and 100-year peak control). In addition, these equations are not intended to replace detailed hydrologic and flood routing analysis, or even the analysis using the Rational Formula-based FAA method for the sizing of detention storage volumes. The District does not promote the use of these empirical equations. It does not object, however, to their use by local governments who have adopted them or want to adopt them as minimum requirements for the sizing of on-site detention for small catchments within their jurisdiction. If the District has a master plan that contains specific guidance for detention storage or sizing of on-site detention facilities, those guidelines should be followed instead. The empirical equations for NRCS Soil types B, C and D are as follows:

$$V_i = K_i A \qquad \text{(SO-1)}$$

for the 100-year:

$$K_{100} = \frac{(1.78 I - 0.002 I^2 - 3.56)}{900} \qquad \text{(SO-2)}$$

for the 10-year:

$$K_{10} = \frac{(0.95 I - 1.90)}{1{,}000} \qquad \text{(SO-3)}$$

for the 5-year:

$$K_5 = \frac{(0.77 I - 2.65)}{1{,}000} \qquad \text{(SO-4)}$$

For Soil Type A, Equations SO-1 and SO-2 tend to underestimate the needed 100-year detention volume. Instead, Equation SO-5 needs to be used to estimate the 100-year detention volume for Type A Soils (i.e., V_{100A}):

$$V_{100A} = \left(-0.00005501 \cdot I^2 + 0.030148 \cdot I - 0.12\right) \cdot \frac{A}{12} \qquad \text{(SO-5)}$$

in which:

V_i = required volume where subscript i = 100-, 10- or 5-year storm, as appropriate (acre-feet)

K_i = empirical volume coefficient where subscript i = 100-, 10- or 5-year storm, as appropriate

I = fully developed tributary catchment imperviousness (%)

A = tributary catchment area (acres)

Design Example 6.1 shows calculations of allowable release rate and storage requirement using empirical equations.

3.2.3 Rational Formula-Based Modified FAA Procedure

The Rational Formula-based Federal Aviation Administration (FAA) (1966) detention sizing method (sometimes referred to as the "FAA Procedure"), as modified by Guo (1999a), provides a reasonable estimate of storage volume requirements for on-site detention facilities. Again, this method provides sizing for one level of peak control only and not for multi-stage control facilities.

The input required for this Rational Formula-based FAA volume calculation procedure includes:

A = the area of the catchment tributary to the storage facility (acres)

C = the runoff coefficient

Q_{po} = the allowable maximum release rate from the detention facility based on Table SO-1 (cfs)

T_c = the time of concentration for the tributary catchment (see the RUNOFF chapter) (minutes)

P_1 = the 1-hour design rainfall depth (inches) at the site taken from the RAINFALL chapter for the relevant return frequency storms

The calculations are best set up in a tabular (spreadsheet) form with each 5-minute increment in duration being entered in rows and the following variables being entered, or calculated, in each column:

1. Storm Duration Time, T (minutes), up to 180 minutes.

2. Rainfall Intensity, I (inches per hour), calculated using Equation RA-3 from the RAINFALL chapter.

3. Inflow volume, V_i (cubic feet), calculated as the cumulative volume at the given storm duration using the equation:

$$V_i = CIA\left(60T\right) \tag{SO-6}$$

4. Outflow adjustment factor m (Guo 1999a):

$$m = \frac{1}{2}\left(1 + \frac{T_c}{T}\right) \qquad 0.5 \le m \le 1 \text{ and } T \ge T_c \tag{SO-7}$$

5. The calculated average outflow rate, Q_{av} (cfs), over the duration T:

$$Q_{av} = mQ_{po} \tag{SO-8}$$

6. The calculated outflow volume, V_o, (cubic feet), during the given duration and the adjustment factor at that duration calculated using the equation:

$$V_o = Q_{av}\left(60T\right) \qquad \text{(SO-9)}$$

7. The required storage volume, V_S (cubic feet), calculated using the equation:

$$V_s = V_i - V_o \qquad \text{(SO-10)}$$

The value of V_s increases with time, reaches a maximum value, and then starts to decrease. The maximum value of V_s is the required storage volume for the detention facility. Sample calculations using this procedure are presented in Design Example 6.2. The modified *FAA Worksheet* of the UD-Detention Spreadsheet performs these calculations.

3.2.4 Simplified Full-Spectrum Detention Sizing (Excess Urban Runoff Flow Control)

With urbanization, the runoff volume increases. Percentage-wise, this increase is much more noticeable for the smaller storm events than for the very big ones, such as the 100-year storm. Wulliman and Urbonas (2005) suggested a concept they termed *Full Spectrum Detention*. This concept was studied using extensive modeling, including continuous simulations of a calibrated watershed. Based on this modeling the original set of equations was slightly modified to increase the EURV by 10%. The protocol that resulted and that is described below reduced runoff peak flows from urbanized areas to more closely approximate the runoff peaks along major drainageways before urbanization occurred.

This concept captures a volume of runoff defined as the *Excess Urban Runoff Volume*" (EURV) and then releases it over approximately 72-hours. EURV is larger than the Water Quality Capture Volume (WQCV) defined in Volume 3 of this *Manual* and varies with the type of NRCS soil group upon which urbanization occurs. EURV includes within its volume the WQCV, which then makes it unnecessary to deal with it separately when the *Full Spectrum Detention* design is used. *Full Spectrum Detention* Equations SO-11, -12 and -13 may by used to find the EURV depths in watershed inches. They were developed using the hydrologic methods described in this *Manual*.

NRCS Soil Group A: $$EURV_A = 1.1\cdot\left(2.0491\cdot i - 0.1113\right) \qquad \text{(SO-11)}$$

NRCS Soil Group B: $$EURV_B = 1.1\cdot\left(1.2846\cdot i - 0.0461\right) \qquad \text{(SO-12)}$$

NRCS Soil Group C/D: $$EURV_{CD} = 1.1\cdot(1.1381\cdot i - 0.0339) \qquad \text{(SO-13)}$$

in which, $EURV_K$ = Excess Urban Runoff Volume in watershed inches ($K = A, B \text{ or } CD$),

i = Imperviousness ratio (I/100)

By combining the capture and slow release of the EURV with the 100-year control volumes for Soil Types B, C and D recommended by Equations SO-1 and SO-2 or for Soil Type A recommended by Equation SO-5 with the 100-year release rates based on recommendations in Table SO-1, this concept was found to be more effective in controlling peak flow along major drainageways for almost all levels of storms than provided by the simplified equations or the FAA Method, even for relatively large urban catchments.

The EURV is found using volumes obtained for each soil type, which are then area weighted in proportion to the total catchment's area. The watershed inches of EURV are then converted to cubic feet or acre-feet. The total 100-year detention basin volume is found using Equations SO-1 and SO-2 for Type B, C and D soils or Equation SO-5 for Type A soils, which are also area-weighted by soil types and converted to cubic feet or acre feet. The outlet is designed to empty the EURV in approximately 72 hours. Volumes exceeding EURV are controlled by an outlet designed for a composite maximum 100-year release rate based on unit rates recommended in Table SO-1.

Equation 13a was developed to assist in the sizing of the openings of the perforated plate outlet to drain the EURV in 72 hours, provided the outlet follows the standardized design developed originally with the WQCV outlet for an Extended Detention Basin (EDB) described in Volume 3, namely the perforations are spaced vertically on 4" centers. Figure SO-8 depicts the results of this equation in graphical form. The equation and the figure are only applicable for water depths in the basin between one and eight feet and designers should not extrapolate beyond this range. Outlets needing greater or lesser depths than these need to be designed individually using ether EPA SWMM, UD-Detention spreadsheet or other appropriate software. The *Full-Spectrum Worksheet* of the UD-Detention Spreadsheet performs all of these calculations for the standardized designs, including adjustments for imperviousness due to Level 1 and 2 of MDCIA, accounts for the effects of various soil type distributions in the tributary catchment and has a provision for selecting the local government's policy in how the WQCV is treated as part of the 100-year volume, although the District does not recommend adding any portion of the WQCV to the 100-year volume calculated using this spreadsheet.

$$A = \left[\frac{EURV}{0.00528 \cdot H^2 + 0.0655 \cdot H + 0.0492} \right]^{\frac{1}{-0.0018 \cdot H^2 - 0.0068 \cdot H + 1.0015}} \qquad \text{SO-13a}$$

In which, A = open area per row of perforations, in square inches
H = maximum water depth in basin above the bottom of lowest perforation, in feet
$EURV$ = excess urban runoff volume, in acre feet

Whenever possible, it is suggested that circular orifice openings be used, beveled on the downstream side. The goal is to find a commonly available drill-bit size that will match the needed area with as few columns of perforations as possible. To achieve this, the designer should seek a drill bit size that will deliver an area within +5% and -10% of the one calculated using Equation SO-13a or Figure SO-8.

3.2.5 Excess Urban Runoff Flow Control at Regional Facilities

The simplified full-spectrum detention concept described above is appropriate for volume and outlet sizing of detention facilities serving on-site watersheds of up to 160 acres. For full-spectrum basins serving larger watersheds, the EURV portion of the basin still needs to be sized using Equations SO-10 through SO-12 and the outlet designed to empty this volume in approximately 72-hours. The 100-year peak flow control volume above the EURV has to be sized, and its outlet designed, using full hydrograph routing protocols. The hydrograph routing option is also available for smaller sub-watersheds as well. Regardless of which 100-year sizing and outlet design option is used for regional facilities, the maximum 100-year release rates cannot exceed the release rates based on unit discharges recommended in Table SO-1 or pre-developed peak 100-year flow rates for the tributary watershed, whichever are less, or those recommended in a District accepted master plan.

3.2.6 Multi-Level Control

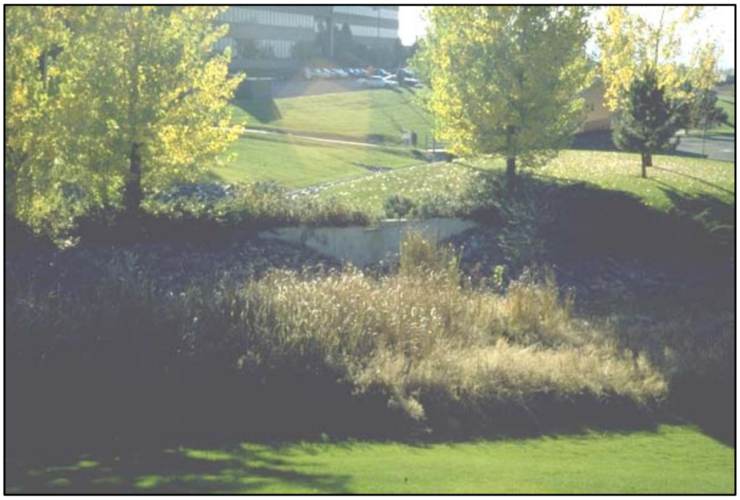
The District recommends that no more than two levels of controls, in addition to the WQCV controls, be used for on-site detention facilities. These levels can be the 10- or 100-year storm, in combination with the 2-, 5- or the 10-year storm, as appropriate. More levels of control may appear to provide increased protection, but the added complexity of design and the questionable accuracy of results rarely justifies it. As an alternative to this three-level control recommended above, one can chose the two-level control offered by Sections 3.2.4 and 3.2.5 above to achieve broader levels of peak runoff control and possibly less expensive outlet design.

3.2.7 On-Site Detention and UDFCD 100-year Floodplain Management Policy

While UDFCD has confidence in the ability of many on-site detention basins to control peak flow rates to predevelopment level for small urban catchments, this is not the case for larger watersheds. The complexities of predicting where each on-site detention basin is going to be installed as areas urbanize, how each is going to be designed and built, and then applying the detention routing technology on an evolving and diffuse system of control facilities is beyond anyone's ability to assess or predict. In addition, the UDFCD has no ability or power to insure that all on-site detention facilities will continue to be maintained and their function will not deteriorate over time. In fact, evidence suggests to the contrary (Prommersberger, 1984) that many on-site detention facilities do not receive needed maintenance and do not provide the original design function over time. Prommersberger (1984) found that many, in fact, have never been built as designed. In response to these complexities of implementation and future maintenance uncertaities, the UDFCD adheres to the following policies when developing hydrology for the delineation and regulation of the 100-year flood hazard zones within its boundaries:

1. Hydrology has to be based on fully developed watershed condition as estimated to occur, at a minimum, over the next 50 years.
2. No on-site detention basin will be recognized in the development of hydrology unless:

a. It serves a watershed that is larger than 130-acres, and
b. It provides a regional function, and
c. It is owned and maintained by a public agency, and
d. The public agency has committed itself to maintain the detention facility so that it continues to operate in perpetuity as designed and built.



Photograph SO-4—This on-site dry detention facility (note short concrete dam) promotes pollutant removal in smaller runoff events.

These policies are for the definition and administration of the 100-year floodplain and floodway zones and the design of facilities along major drainageways. They are not intended to discourage communities from using on-site detention, including the EURV control (i.e., Full-Spectrum Detention) discussed above. On-site detention can be very beneficial for stormwater quality and quantity management, reducing the sizes of local storm sewers and other conveyances, and providing a liability shield (defense) when needing to address the issue of keeping stormwater-related damages from increasing to downstream properties as lands are developed. However, unless detention is regional in nature with a government having property rights to operate and maintain it in perpetuity, and is designed in accordance with an approved master plan, it will not be considered eligible for District's maintenance assistance program (see Chapter 5 for maintenance eligibility discussion). Furthermore, Colorado law requires detention be provided to control the 100-year peak flow for all new development in the unincorporated portions of all counties.

3.3 Design Storms for Sizing Storage Volumes

Typically, more than one design storm usually is controlled when designing detention or retention facilities. Water quality storage and release is based on the recommendations in Volume 3 of this *Manual*. For drainage and flood control design, the 2-, 5-, 10-, 25- and 100-year design storms are often considered and used, as required by local municipality. Sizing may sometimes be driven by downstream conveyance system capacities and public safety concerns in addition to standard local detention sizing requirements. Sizing of emergency spillways may also require the use of design storms larger than the 100-year storm. What follows is a thumbnail description of the factors to consider for each.

Photograph SO-5—Multipurpose detention facilities are strongly encouraged, as they often become community focal points.

3.3.1 Water Quality Capture Volume

This was discussed in detail under Sections 3.1.3 and 3.2.4 for facilities that include quantity and quality storage, and the reader is referred to them. The specific recommendations for the sizing of the WQCV are given in Volume 3 of this *Manual*.

3.3.2 Drainage and Flood Control

Sizing of storage facilities and outlet works for flood control purposes is generally based on whether the facility is on-site or regional. For an individual development sites, local municipalities will dictate which design storms need to be addressed. On a watershed level, full system master planning studies are needed to identify the appropriate release rates for various design storms. Whenever a District-approved master plan recommends detention sites and release rates, or on-site detention/retention storage and release rates, this sizing and rates should be used in final design of detention/retention facilities. Other considerations that have to be taken into account include downstream system stability, the drainageway's capacity to convey discharges from the detention/retention facility in combination with the downstream runoff contributions, potential for flood damages to downstream properties, and other factors that may be specific to each situation.

3.3.3 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), 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. Generally, embankments should be fortified against and/or have spillways that, at a minimum, are capable of conveying the total not-routed peak 100-year storm discharge from a fully developed total tributary catchment, including all off-site areas, if any. Detailed analysis, however, of downstream hazards should be performed and may indicate that the embankment protection and/or spillway design needs to be for events much larger than the 100-year design storm.

3.3.4 Retention Facilities

A retention facility (a basin with a zero release rate or a very slow release rate) is used when there is no available formal downstream drainageway, or one that is grossly inadequate. When designing a retention facility, the hydrologic basis of design is difficult to describe because of the stochastic nature of rainfall events. Thus, sizing 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. For this reason, retention basins are not recommended as a permanent solution for drainage problems. They have been used in some instances as temporary measures until a formal system is developed downstream. When used, they can become a major nuisance to the community duo to problems that may include mosquito breeding, safety concerns, odors, etc.

When a retention basin is proposed as a temporary solution, the District recommends that it be sized to capture, as a minimum, the runoff equal to 1.5 times the 24-hour, 100-year storm plus 1-foot of freeboard. The facility also has to be situated and designed so that when it overtops, no human-occupied or critical structures (e.g., electrical vaults, homes, etc.) will be flooded, and no catastrophic failure at the facility (e.g., loss of dam embankment) will occur. It is also recommended that retention facilities be as shallow as possible to encourage infiltration and other losses of the captured urban runoff. When a trickle outflow can be accepted downstream or a small conduit can be built, provided and sized it in accordance with the locally approved release rates, preferably capable of emptying the full volume in 14 days or less.

3.4 Reservoir Routing of Storm Hydrographs for Sizing of Storage Volumes

The reservoir routing procedure for the sizing of detention storage volumes is more complex and time consuming than the use of empirical equations, FAA procedure or the simplified *Full Spectrum Detention* protocol. Its use requires the designer to develop an inflow hydrograph for the facility. This is generally accomplished using CUHP and UDSWM computer models as described in the RUNOFF chapter of this *Manual*. The hydrograph routing sizing method is an iterative procedure that follows the steps detailed below (Guo 1999b).

1. Select Location: The detention facility's location should be based upon criteria developed for the specific project. Regional storage facilities are normally placed where they provide the greatest overall benefit. Multi-use objectives such as the use of the detention facility as a park or for open space, preserving or providing wetlands and/or wildlife habitat, or others uses and community

needs influence the location, geometry, and nature of these facilities.

Photograph SO-6—Public safety is an important design consideration for detention facilities, including the potential need for safety/debris racks on outfall structures, as shown in this dry pond.

2. Determine Hydrology: Determine the inflow hydrograph to the storage basin and the allowable peak discharge from the basin for the design storm events. The hydrograph may be available in published district outfall system planning or a major drainageway master plan report. The allowable peak discharge is limited by the local criteria or by the requirements spelled out in a District-approved master plan.

3. Initial Storage Volume Sizing: It is recommended that the initial size of the detention storage volume be estimated using the modified FAA method described in Section 3.2.3, the Full Spectrum Detention protocols in Section 3.2.4 or the hydrograph volumetric method detailed in Section 3.4.1.

4. Initial Shaping of the Facility: The initial shape of the facility should be based upon site constraints and other goals for its use discussed under item 1, above. This initial shaping is needed to develop a stage-storage-discharge relationship for the facility. The design spreadsheets provided on the CD version of this *Manual* are useful for initial sizing.

5. Outlet Works Preliminary Design: The initial design of the outlet works entails balancing the initial geometry of the facility against the allowable release rates and available volumes for each stage of hydrologic control. This step requires the sizing of outlet elements such as a perforated plate for controlling the releases of the WQCV, orifices, weirs, outlet pipe, spillways, etc.

6. Preliminary Design: A preliminary design of the overall detention storage facility should be completed using the results of steps 3, 4 and 5, above. The preliminary design phase is an

iterative procedure where the size and shape of the basin and the outlet works are checked using a reservoir routing procedure and then modified as needed to meet the design goals. The modified design is then checked again using the reservoir routing and further modified if needed. Though termed "preliminary design," the storage volume and nature and sizes of the outlet works are essentially in final form after completing this stage of the design. They may be modified, if necessary, during the final design phase.

7. Final Design: The final design phase of the storage facility is completed after the hydraulic design has been finalized. This phase includes structural design of the outlet structure, embankment design, site grading, a vegetation plan, accounting for public safety, spillway sizing and assessment of dam safety issues, etc.

Photograph SO-7—This retention pond has an embankment with upstream and downstream gentle sideslopes, which promotes dam safety and multipurpose use.

3.4.1 Initial Sizing

The intent of initial sizing of the facility is only to determine a starting point for the reservoir routing procedure that will be used to prepare the preliminary design for the facility. The initial sizing methods are not adequate for final design of the facility. Two methods for initial sizing are discussed below.

The Rational Formula-based modified FAA method may be used to find an initial storage volume for any size catchment. This technique for initial sizing yields best results when the tributary catchment area is less than 320 acres. The designer needs to understand that the design volumes may need to be adjusted significantly regardless of the tributary area once full hydrograph routing is performed.

It is also possible to use the inflow hydrograph, along with desired maximum release rates, to make an initial estimate of the required storage volume. This technique assumes that the required detention volume is equal to the difference in volume between the inflow hydrograph and the simplified outflow hydrograph. It is represented by the area between these two hydrographs from the beginning of a runoff event until the time that the allowable release occurs on the recession limb of the inflow hydrograph (Guo 1999b) (see Figure SO-1). The inflow hydrograph is generally obtained using CUHP/SWMM computer model computations. The outflow hydrograph can be approximated using a straight line between zero at the start of the runoff to a point where the allowable discharge is on the descending limb of the inflow hydrograph, T_p. The volume are calculated by setting up tabular calculations with the following columns:

1. The time T (in minutes) from 0 to T_p in 5-minute increments. T_p is the time (in minutes) where the descending limb of the inflow hydrograph is equal to the allowable release rate.

2. The inflow rate Q_i (cfs) to the detention basin corresponding to the time T. The inflow rate is an input value that is generally obtained from a CUHP/SWMM hydrologic analysis.

3. The outflow rate Q_o (cfs) calculated as:

$$Q_O = \frac{T}{T_p} Q_{po} \tag{SO-14}$$

   in which, Q_{po} is the peak outflow rate, where allowable peak outflow rate is determined from a District master plan, local ordinance, or other considerations described in Section 3.3.2.

4. The incremental storage volume = (column 2 – column 3) · 300 seconds.

5. The total storage volume calculated as the sum of the values in column 4.

Design Example 6.3 illustrates this procedure.

The **Hydrograph Worksheet** of the UD-Detention Spreadsheet performs these computations.

3.4.2 Initial Shaping

The initial shaping of the storage basin provides a starting point for defining the stage-storage relationship. The stage-storage relationship has to be refined during preliminary design phase of the project. The initial shaping is easiest when regular geometry (such as a triangle, rectangle, or elliptical) is used for approximation. The detention volume needed for any specific design storm is combined with site constraints (e.g., size or depth limitations, number of control stages, etc.) and the simplified formulas describing the basin geometry in order to develop an initial depth, length, and width for the basin. Design spreadsheets can be used to assist in preliminary shaping of the basin. This does not mean that the District encourages the use of storage facilities with uniform geometric properties. To the contrary, the District encourages designers to collaborate with landscape architects to develop storage facilities that are visually attractive, fit into the fabric of the landscape, and enhance the overall character of an area. However, using regular geometries can speed up initial sizing of a non-uniformly shaped facility.

3.4.3 Outlet Works Design

Outlet works are structures that control the release rates from storage facilities. Figure SO-2 illustrates three concepts for detention basin outlets. Two are from Volume 3 of the *Manual* that provides for a three-level flow control including the control of the Water Quality Capture Volume (WQCV). The other is for a two-level control designed for release of the *Excess Urban Runoff Volume* (EURV) over 72-hours and control of the 100-year peak flow to a specified maximum rate. Both include an orifice plate for release of the WQCV or the EURV. The first concept also provides for the 2- to 10-year (or other return period) storm controls through drop boxes and orifices at the bottom of the boxes. The other provides and orifice at the bottom of one drop box to control the 100-year (or other return period) release rate. The weir length of the drop box is best oversized after reducing its length by the trash rack bars so as not to become the primary control when the trash rack has some clogging. The goal is to have the orifice at the bottom of the box and in front of the outlet pipe exercise the desired flow control at the maximum stage in the basin.

The hydraulic capacity of the various components of the outlet works (orifices, weirs, pipes) can be determined using standard hydraulic equations. The discharge pipe of the outlet works functions as a culvert. See the CULVERTS chapter of the *Manual* for guidance regarding the calculation of the hydraulic capacity of outlet pipes. The following discussion regarding weirs and orifices is adapted from *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22 (Brown, Stein, and Warner 1996). 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.

Design aids for the design of basins and outlet works are provided on several of the worksheets in the UD-Detention Spreadsheet available for downloading from the District's web site.

3.4.3.1 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, as illustrated in Figure SO-3a, orifice flow can be determined using Equation SO-15.

$$Q = C_o A_o (2gH_o)^{0.5} \qquad \text{(SO-15)}$$

in which:

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

C_o = discharge coefficient (0.40 – 0.65)

A_o = area of orifice (ft^2)

H_o = effective head on each orifice opening (ft)

g = gravitational acceleration (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.

For square-edged, uniform orifice entrance conditions, a discharge coefficient of 0.61 should be used. 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.

3.4.3.2 Weirs

Relationships for sharp-crested, broad-crested, V-notch, and proportional weirs are provided below:

Sharp-Crested Weirs: Typical sharp-crested weirs are illustrated in Figures SO-4a through SO-4d. Equation SO-16 provides the discharge relationship for sharp-crested weirs with no end contractions (illustrated in Figure SO-4a):

$$Q = C_{scw} L H^{1.5} \qquad \text{(SO-16)}$$

in which:

Q = discharge (cfs)

L = horizontal weir length (ft)

H = head above weir crest excluding velocity head (ft)

H_c = height of weir crest above the approach channel bottom (ft)

C_{scw} = 3.27 + 0.4 (H/H_c)

The value of the coefficient C_{SCW} varies with the ratio H/H_c (see Figure SO-4c for definition of terms). When the ratio H/H_c less than 0.3, a constant C_{SCW} of 3.33 is often used.

Equation SO-17 provides the discharge equation for sharp-crested weirs with end contractions (illustrated in Figure SO-4b). As stated above, the value of the coefficient C_{SCW} varies with the ratio H/H_c and becomes a constant 3.33 when H/H_c is less than 0.3.

$$Q = C_{scw}(L - 0.2H)H^{1.5} \qquad \text{(SO-17)}$$

Another form of sharp crested weir is the *Cipoletti* weir. It is a trapezoidal weir with sides slope at 1-horizontal to 4-vertical. The equation for this weir is:

$$Q = 3.367 \cdot L \cdot H^{1.5} \tag{SO-17a}$$

Sharp-crested weirs will be affected by submergence when the tailwater rises above the weir crest elevation, as shown in Figure SO-4d. The result will be that the discharge over the weir will be reduced.

The discharge equation for a submerged sharp-crested weir is:

$$Q_s = Q_r \left[1 - \left(\frac{H_2}{H_1} \right)^{1.5} \right]^{0.385} \tag{SO-18}$$

in which:

Q_s = submerged flow (cfs)

Q_r = un-submerged weir flow from Equation SO-15 or SO-16 (cfs)

H_1 = upstream head above crest (ft)

H_2 = downstream head above crest (ft)

Flow over the top edge of the riser pipe can initially be treated, until the throat of the pipe takes over the hydraulic control, as flow over a sharp-crested weir with no end constrictions. Equation SO-17 should be used for this case.

Broad-Crested Weir: The equation typically used for a broad-crested weir is:

$$Q = C_{BCW} L H^{1.5} \tag{SO-18}$$

in which:

Q = discharge (cfs)

C_{BCW} = broad-crested weir coefficient (This ranges from 2.38 to 3.32 as per Brater and King (1976). A value of 3.0 is often used in practice.)

L = broad-crested weir length (ft)

H = head above weir crest (ft)

V-Notch Weir: The discharge through a V-notch or triangular weir is shown in Figure SO-5 and can be calculated from the following equation:

$$Q = C_t \tan\left(\frac{\theta}{2} \right) H^{2.5} \tag{SO-20}$$

in which:

C_t = Coefficient for Triangular Weir taken from the table below

Q = discharge (cfs)

θ = angle of V-notch in degrees

H = head above the apex of V-notch (ft)

Depth H in feet	Coefficient C_t for V-Notch Angle θ			
	20°	45°	60°	90°
0.2	2.81	2.66	2.62	2.57
0.4	2.68	2.57	2.53	2.51
0.6	2.62	2.53	2.51	2.49
0.8	2.60	2.52	2.50	2.48

3.4.4 Preliminary Design

The preliminary design stage consists of refining the design of the basin (size, shape and elevation) and outlet structure (type, size, configuration). At this time, the basin's bottom may be sloped as needed to provide drainage to the outlet and/or trickle channel to prevent the bottom from becoming boggy and habitat for mosquito breeding. Preliminary design is an iterative process that determines the detention basin's outflow characteristics given the stage-storage-discharge parameters of the basin and the inflow hydrograph(s). The stage-storage-discharge characteristics are modified as needed after each model run until the outflow from the basin meets the specified flow limit. No description of the theory of reservoir routing is provided in this *Manual*. The subject is well described in many hydrology reference books (Viessman and Lewis 1996; Guo 1999b).

Reservoir modeling can be carried out in a number of different ways. The EPA SWMM model provides for reservoir routing. The modeler provides a stage-discharge relationship for a reservoir outlet junction and the stage-surface area relationship for the storage junction of the model or the detention facility. The stage-surface 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. If the storage facility is modeled as part of a larger system being addressed through a master planning effort, the SWMM model must be used. For the design of individual detention sites that goes into greater detail than used in watershed master planning model, the District's UD POND software provides a reliable and relatively easy tool to facilitate detention basin design.

3.4.5 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, trash rack design, 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.

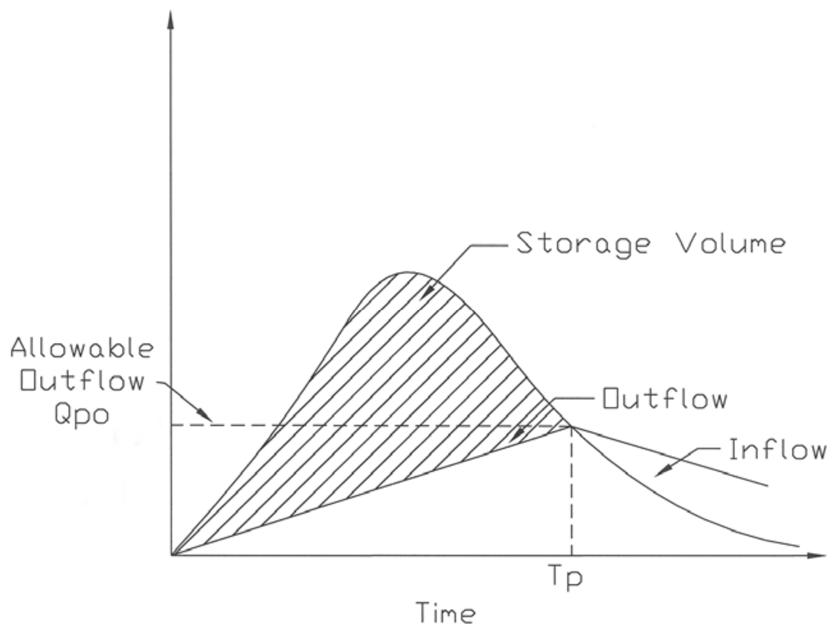
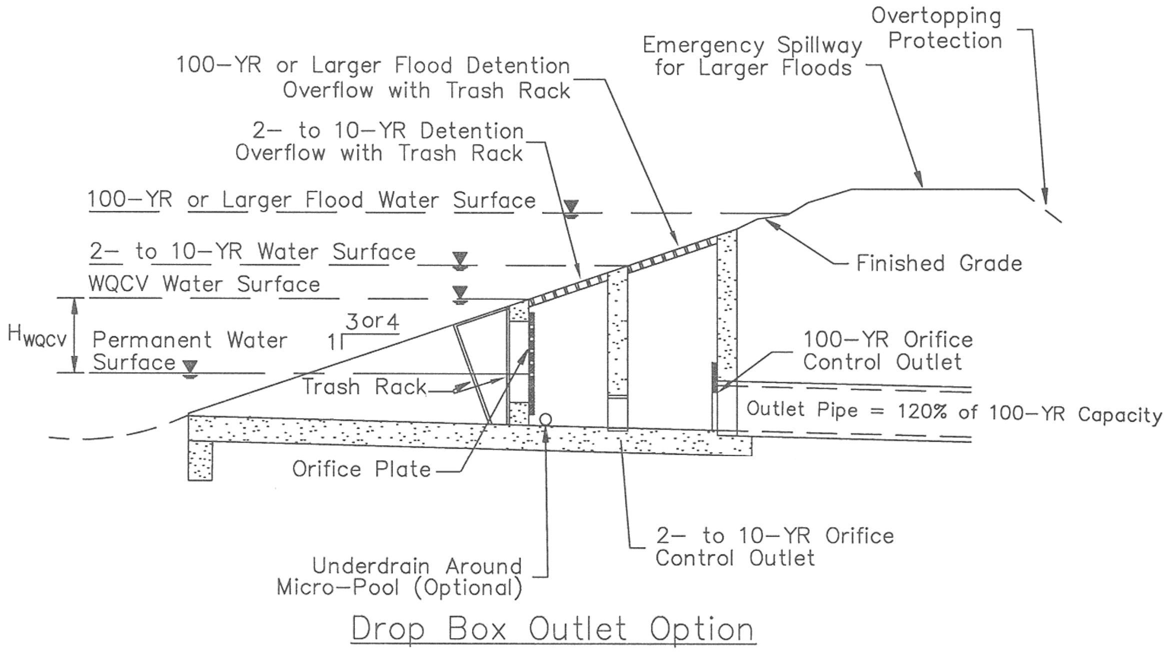


Figure SO-1—Hydrograph Volumetric Method for Initial Basin Pre-Sizing



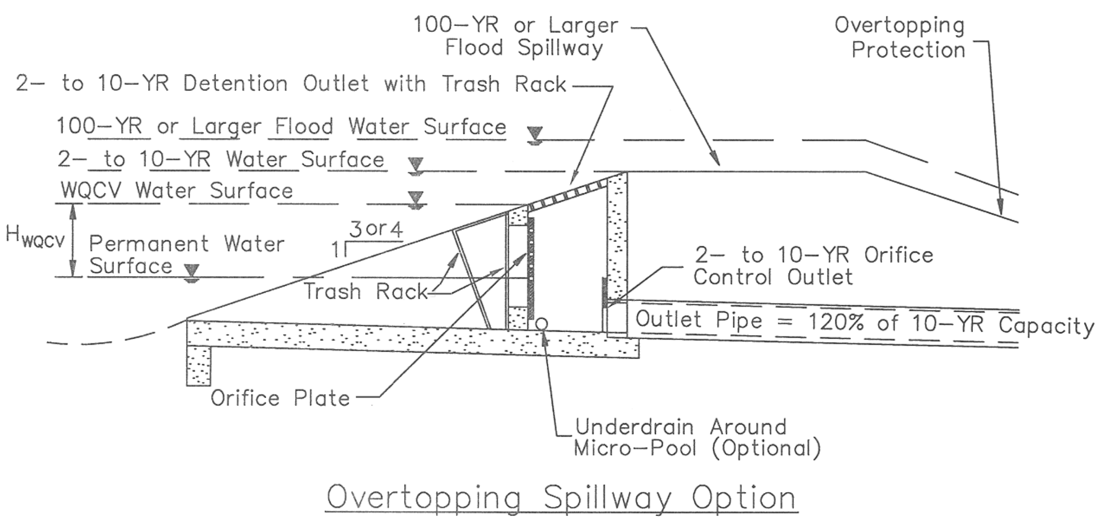
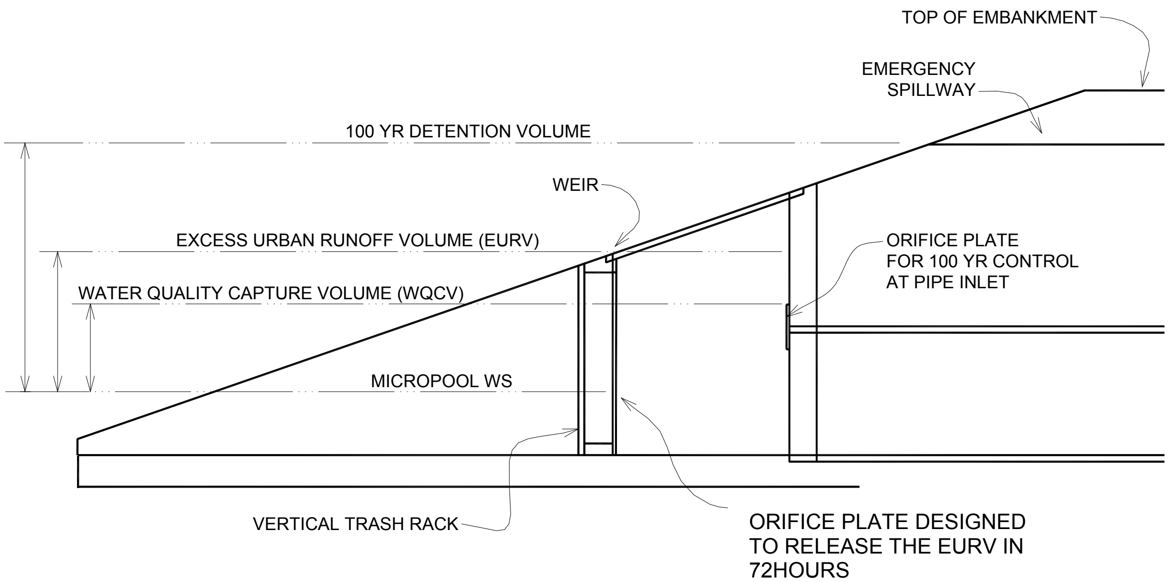


Figure SO-2—Typical Outlet Structure Profiles
(Sheet 1 of 2: Three-Level Peak Flow Control Case)



EXAMPLE OUTLET STRUCTURE

Figure SO-2—Typical Outlet Structure Profiles
(Sheet 2 of 2: Full Spectrum Detention Two-Level Flow Control Case)

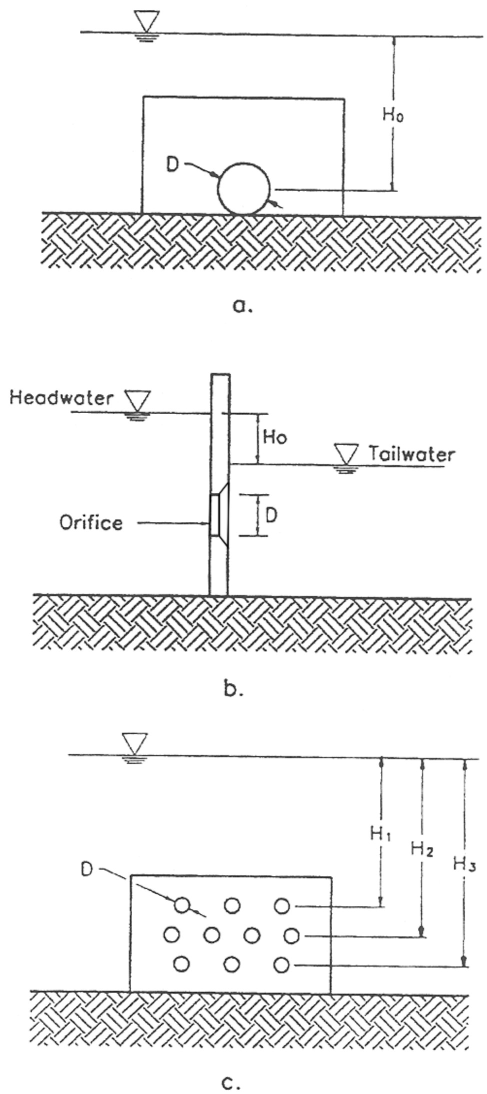


Figure SO-3—Illustration Defining Hydraulic Head for Flow through Orifice(s)

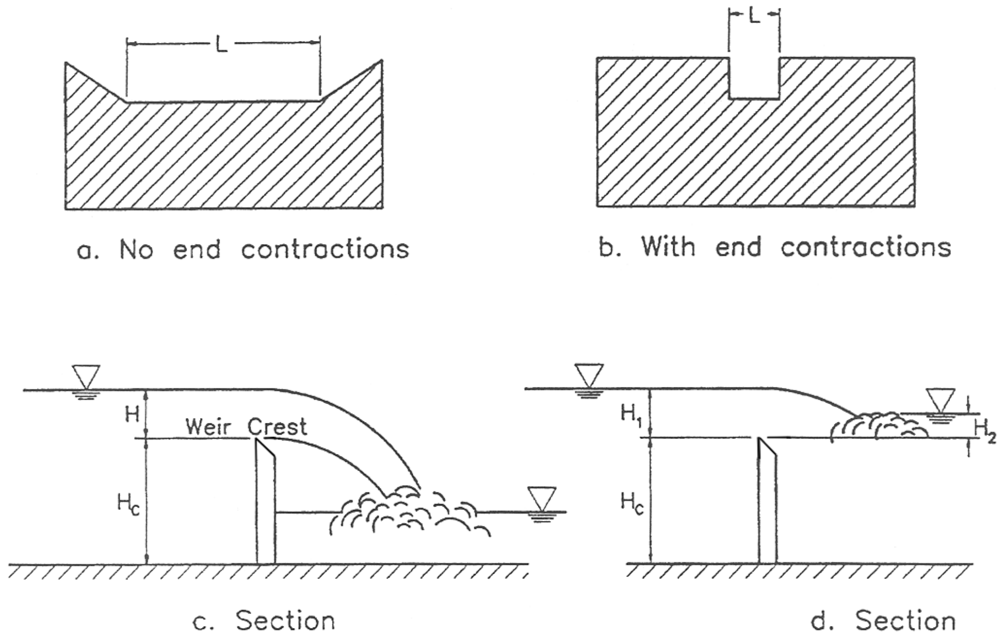


Figure SO-4—Sharp-Crested Weirs

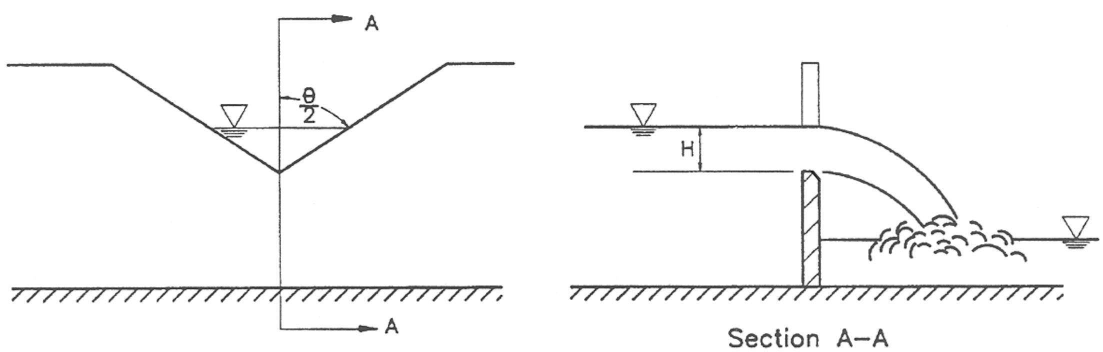


Figure SO-5—V-Notch Weir

4.0 FINAL DESIGN CONSIDERATIONS

Final design of a storage facility should recognize the kinds of considerations described in this section. It is beyond the scope of this *Manual* 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). The District urges all designers to review and adhere to the guidance in such references because the failure of even small embankments can have serious consequences for the public and the municipalities downstream of the embankment. General guidelines for the final design phase of detention or retention facilities follows.

4.1 Storage Volume

The determination of storage volume for quantity control is described earlier in this chapter. If the storage facility includes a WQCV, the appropriate flood storage volume should be provided, one that is in addition to the WQCV, as discussed under Sections 3.1.3 and 3.2.4. Determination of the WQCV is described in Volume 3 of the *Manual*. In the case of on-site detention, if the Excess Urban Runoff Volume is to be provided (i.e., Full Spectrum Detention) in conjunction with the 100-year volume obtained using empirical equations, no additional volume for WQCV needs to be provided within the 100-year basin. When using the Full Spectrum Detention concept with regional detention, the flood control volume has to be calculated using full hydrograph routing procedures.

4.2 Potential for Multiple Uses

Whenever desirable and feasible, incorporate water quality detention into a larger flood control facility. Also, when feasible, provide for other urban uses such as active or passive recreation and wildlife habitat. If multiple uses are being contemplated, use the multiple-stage detention basin to limit inundation of passive recreational areas to one or two occurrences a year and active recreation areas to once every two years. Generally, the area within the WQCV is not well suited for active or passive recreation facilities such as ballparks, playing fields, picnic areas, wildlife habitat, or hiking trails. These are best located above the water quality storage level.

4.3 Geometry of Storage Facilities

The geometry of a storage facility depends on specific site conditions such as adjoining land uses, topography, geology, preserving/creating wildlife habitat, volume requirements, etc. Several key features should be incorporated in all storage facilities located within the District (see Figure SO-6). These include (a) 4:1 or flatter side slopes of all banks, (b) low-flow or trickle-flow channel unless a permanent pool takes its place, (c) forebay, (d) pond bottom sloped at least 1.0 percent to drain toward the low-flow or trickle-flow channel or the outlet, (e) micro pool at the outlet for Extended Detention Basins in Volume 3 of this *Manual*, and (f) emergency spillway or fortification of the embankment to prevent catastrophic failure when overtopped.

It is desirable to shape the water quality portion of the facility with a gradual expansion from the inlet and a gradual contraction toward the outlet, thereby minimizing short-circuiting. Storage facility geometry and layout are best developed in concert with a land planner/landscape architect.

4.4 Embankments and Cut Slopes

If the storage facility is "jurisdictional," namely, subject to regulation by the Colorado State Engineer's Office (SEO), the embankment shall be designed, constructed and maintained to meet SEO 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. In general, the 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):

1. Side Slopes—For ease of maintenance, the side slopes of the embankment should not be steeper than 3H:1V, with 4H:1V preferred. The embankment's side slopes should be well vegetated, and soil-riprap protection (or the equivalent) may be necessary to protect it from wave action on the upstream face, especially in retention ponds.

2. Freeboard—The elevation of the top of the embankment shall 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 survive its overtopping without failure, freeboard requirements may be waived. When relevant, all SEO dam safety criteria must be carefully considered when determining the freeboard capacity of an impoundment.

3. Settlement—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.

4. Emergency Spillway—An emergency spillway will often be needed to convey flows that exceed the primary outlet capacity, unless the embankment is designed to convey overtopped flows without failure (e.g., buried soil cement, grouted boulders, concrete walls with splash pads, etc.).

4.5 Linings

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, consider lining the basin with an impermeable liner. An impermeable liner may also be warranted in a retention pond where the designer seeks to limit seepage from a permanent pool.

Alternatively, there are situations where the designer may seek to encourage infiltration of stormwater into the ground. In this situation, a layer of permeable material may be warranted.

4.6 Inlets

Inlets to the detention facility should incorporate energy dissipation to limit erosion. They should be designed in accordance with drop structure or impact stilling basin criteria in the HYDRAULIC STRUCTURES chapter of this *Manual*, or using other approved energy dissipation structures. In addition, incorporate forebays or sediment traps at all inflow points to detention facilities to deposit coarse sediment being delivered by stormwater to the facility. These forebays will need regular maintenance to lessen the sediment being transported and deposited on the storage basin's bottom.

4.7 Outlet Works

Outlet works should be sized and structurally designed to release at the specified flow rates without structural or hydraulic failure. The design guidance for outlet works used for water quality purposes is included in Volume 3 of the *Manual* and for full-spectrum detention earlier in this chapter.

4.8 Trash Racks

Provide trash racks of sufficient size that do not interfere with the hydraulic capacity of the outlet. See Figure SO-7 for minimum trash rack sizes.

4.9 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, whether native or non-native grasses are desired, and the other potential uses (park, open space, etc.) of the area. A planting plan should be developed for new facilities to meet their intended use and setting in the urban landscape. Generally, trees and shrubs are not recommended on dams or fill embankments (see the REVEGETATION chapter). However, use of trees on the sides of detention basins will not interfere with their flood control operation of increase maintenance need significantly. Also, sparse planting of tree on bottoms of larger regional detention 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. At the same time use of shrubs on the banks and bottom, while not affecting the flood routing, can increase maintenance significantly by providing traps for debris that are difficult to clean and obstructions for the mowing of grasses.

4.10 Operation and Maintenance

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

1. Use of flat side slopes along the banks and the installation of landscaping that will discourage

entry (thick, thorny shrubs) along the periphery near the outlets and steeper embankment sections are advisable. Also, use of safety railings at vertical or very steep structural faces is needed to public safety. 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 impoundment by the owner or agency responsible for maintenance.

3. Bank slopes, bank protection needs, and vegetation types are important design considerations for site aesthetics and maintainability.

4. Permanent ponds should have provisions for complete drainage for sediment removal or other maintenance. 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.

5. For facilities designed for multipurpose use, 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 tile drainage system if active recreation is contemplated.

6. Adequate dissolved oxygen supply in ponds (to minimize odors and other nuisances) can be maintained by artificial aeration. Use of fertilizer and EPA approved pesticides and herbicides adjacent to the permanent pool pond and within the detention basin should be controlled.

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 apt to be high.

9. Underground tanks or conduits designed for detention should be sized and designed to permit pumping or multiple entrance points to remove accumulated sediment and trash.

10. All detention facilities should be designed with sufficient depth to allow accumulation of sediment for several years prior to its removal.

11. Permanent pools should be of sufficient depth to discourage excessive aquatic vegetation on the bottom of the basin, unless specifically provided for water quality purposes.

12. Often designers use trash racks and/or fences to minimize hazards. These may become trap debris, impede flows, hinder maintenance, and, ironically, fail to prevent access to the outlet. On the other hand, desirable conditions can be achieved through careful design and positioning of the structure, as well as through landscaping that will discourage access (e.g., positioning the outlet away from the embankment when the permanent pool is present, etc.). Creative designs, integrated with innovative landscaping, can be safe and can also enhance the appearance of the outlet and pond. Such designs often are less expensive initially.

13. To reduce maintenance and avoid operational problems, outlet structures should be designed with no moving parts (i.e., use only pipes, orifices, and weirs). Manually and/or electrically operated gates should be avoided. 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. One way of doing this is to use a larger outlet pipe and to construct orifice(s) in the headwall to reduce outflow rates. Outlets should be robustly designed to lessen the chances of damage from debris or vandalism. The use of thin steel plates as sharp-crested weirs is best avoided because of potential accidents, especially with children. Trash/safety racks must protect all outlets.

14. Clean out all forebays and sediment traps on a regular basis or when routine inspection shows them to be ¼ to ½ full.

See Volume 3 of this *Manual* for additional recommendations regarding operation and maintenance of water quality related facilities, some of which also apply to detention facilities designed to meet other objectives.

Photograph SO-8—Maintenance considerations must be carefully accounted for during design, with sediment accumulation a particular concern.

4.11 Access

All weather stable access to the bottom, inflow, forebay, and outlet works areas shall be provided for maintenance equipment. Maximum grades should be no steeper than 10 percent, and a solid driving surface of gravel, rock, concrete, or gravel-stabilized turf should be provided.

4.12 Geotechnical Considerations

The designer must take into full account the geotechnical conditions of the site. These considerations may include issues related to 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.

4.13 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 (a) whether the facility will be located on a jurisdictional wetland, (b) whether the facility is to be located on a waterway that is regulated by the U.S. Army Corps of Engineers as a “Waters of the U.S.”, and (c) whether there are threatened and endangered species or habitat in the area.

There are also non-regulatory environmental issues that should be taken into account. Detention facilities can become breeding grounds for mosquitoes unless they are properly designed, constructed and maintained. Area residents may view riparian habitat destruction necessary for construction of the facility objectionably. Considerations of this kind must be carefully taken into account and early discussions with relevant federal, state and local regulators are recommended.

In addition, under Colorado Water Law, storage impoundments can be subject to regulation from a water rights perspective by the SEO. For larger facilities, particularly those with permanent pools, the designer is encouraged to check with the SEO or a qualified water rights attorney to determine which water rights regulations apply.

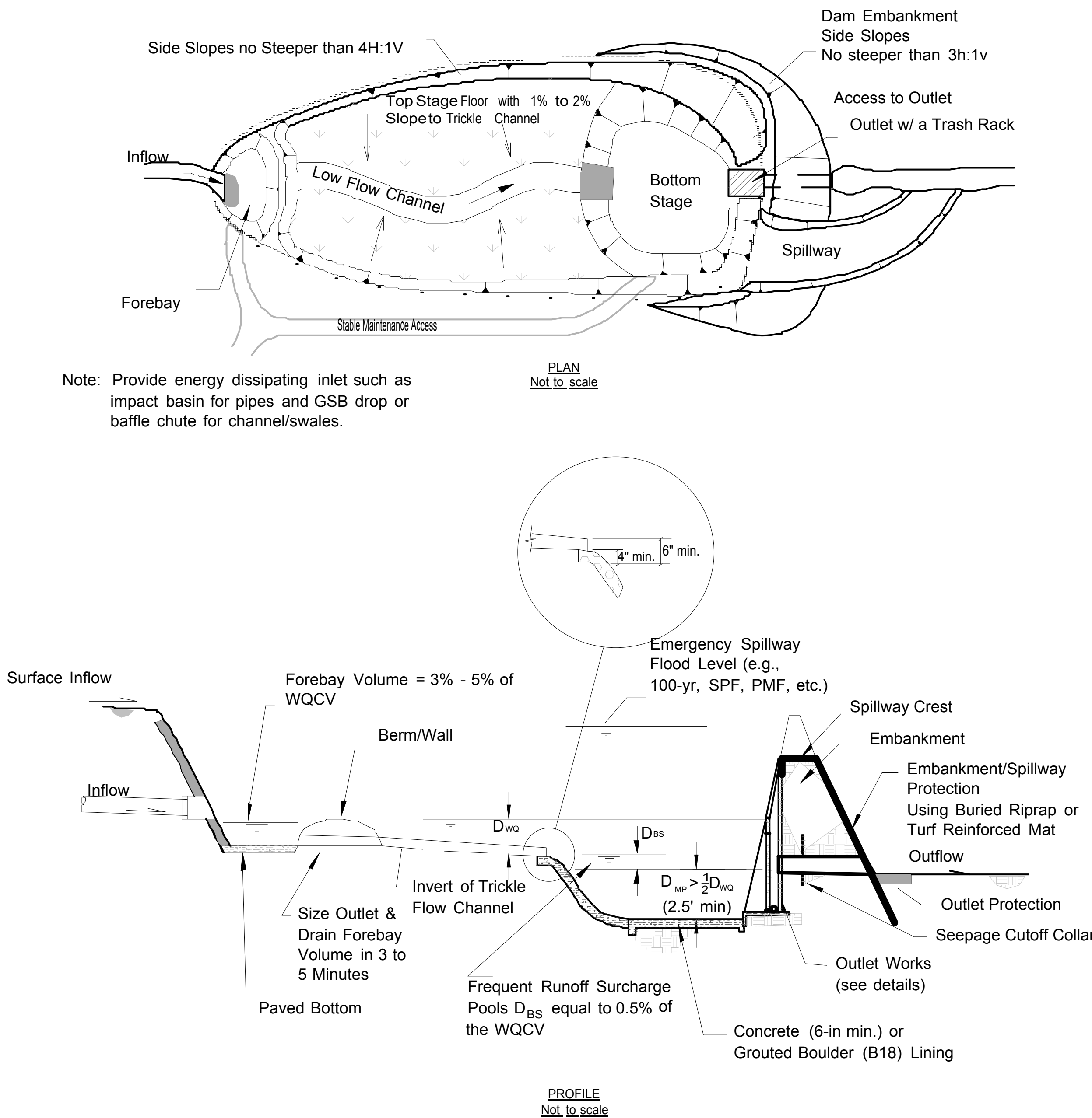


Figure SO-6—Plan and Profile of an Extended Detention Basin in a Flood Control Detention Basin

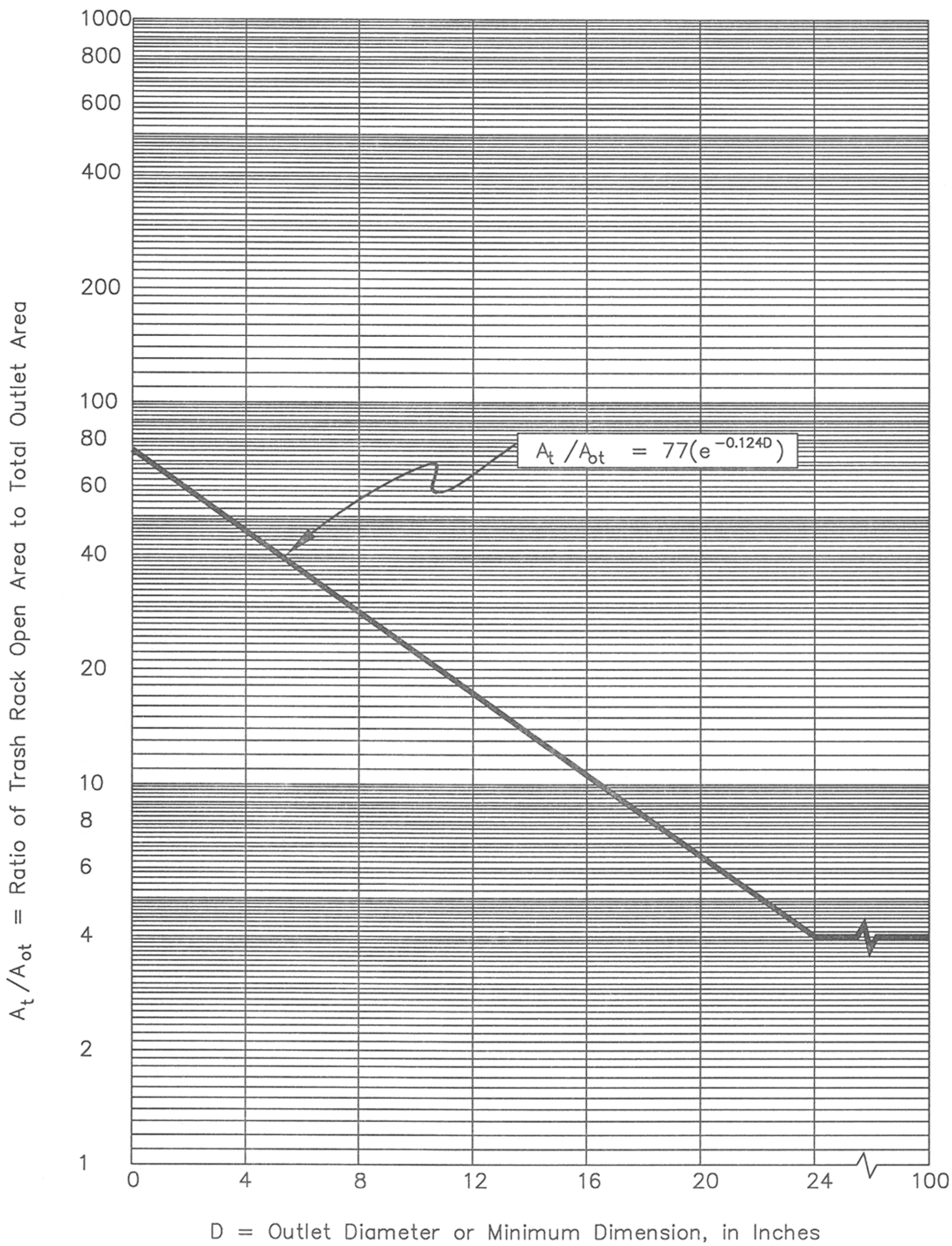


Figure SO-7—Minimum Trash Rack Open Area—Extended Range

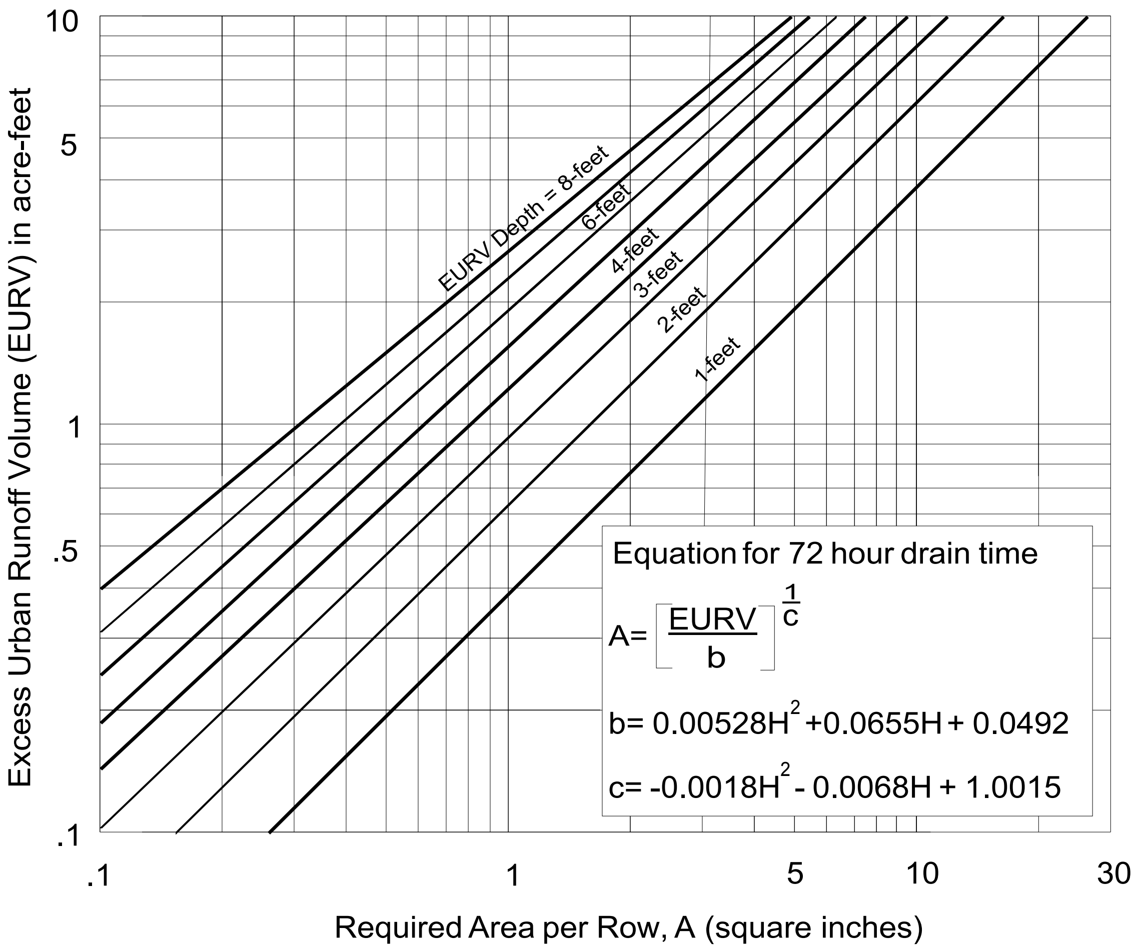


Figure SO-8—Outlet Sizing for EURV Control with 72-hour Drain Time for On-Site Detention

5.0 DISTRICT MAINTENANCE ELIGIBILITY FOR DETENTION FACILITIES

The District has a program to assist local jurisdictions in the on-going maintenance of major drainage facilities including detention facilities. These guidelines change over time as budgets, priorities and needs of the community change. Check the District's Web site (www.udfcd.org) for the most-current maintenance eligibility requirements. Maintenance Eligibility Guidelines as of June 2001 are provided on the CD version of this *Manual*.

There are some common features for which the District's policy is unlikely to change over time. One is the requirement that the facility be owned by or be under control of a public body. "Public body" is defined as a local government (city or county), special district (such as a park district), or a metropolitan district that has a service plan that includes the maintenance and operation of drainage facilities. The public body has to have a reliable funding source to fund maintenance. Legal maintenance access to the detention facility must be made available to the District by the local jurisdiction in accordance with any of the following criteria:

1. The facility is owned by a public body that has accepted primary maintenance responsibility for it.
2. The ownership of the facility is by a private entity (such as a homeowners association owning common areas), but a body has accepted primary maintenance responsibility and has a maintenance access easement(s) that allows it to perform maintenance if the owner does not. Easements crossing individual lots are not acceptable.

6.0 DESIGN EXAMPLES

6.1 Example—Empirical Equations Sizing of a Detention Basin

Determine the required detention volume and allowable release rate for the 10-year and 100-year storm events for a 15-acre site that is in a jurisdiction that has adopted the empirical detention requirements and release rates shown in Section 3.2.1. The NRCS soil survey shows the site has hydrologic soil group B soils. The site will have a developed percentage imperviousness of 45%.

Examination of the District-approved master plan for the area indicates that the current empirical detention requirements for the area may be used. Examination of topographic mapping for the area indicates that no up-gradient off-site flows will traverse the site.

Determine the allowable release rates from Table SO-1:

10-year release rate = 0.23 · 15 acres = 3.45 cfs

100-year release rate = 0.85 · 15 acres = 12.75 cfs

Determine the 10-year required storage volume from Equations SO-1 and SO-3:

Using Equation SO-3, $K_{10} = \{(0.95 \cdot 45) - 1.9\}/1000 = 0.041$

Using Equation SO-1, $V_{10} = 0.041 \cdot 15$ acres = 0.61 acre-feet

The detention required for the 10-year storm is 0.61 acre-feet

Determine the 100-year required storage volume from Equations SO-1 and SO-2:

Using Equation SO-2, $K_{100} = \{(1.78 \cdot 45) - (0.002 \cdot 45^2) - 3.56\}/900 = 0.081$

Using Equation SO-1, $V_{100} = 0.081 \cdot 15$ acres = 1.21 acre-feet

The detention required for the 100-year storm is 1.21 acre-feet

6.2 Example—Rational Method Analysis

Use the FAA method to determine the required detention volume for the 100-year storm event for a 15-acre site that will have a developed percentage imperviousness of 45%. The NRCS soil survey shows the site has hydrologic soil group B soils. The allowable release rate from the basin has to be limited to the unit values in Table SO-1. The time of concentration has been calculated at 12 minutes. The 100-year, 1-hour point precipitation is 2.6 inches.

A runoff coefficient, C, of 0.51 is determined using Table RO-5 of the RUNOFF chapter (the 45% row and

100-year storm column of the type B soils table equals 0.51). The calculations are shown in spreadsheet form UD-Detention workbook in Table SO-2.

Table SO-2—FAA Method Calculations
(From UD-Detention Workbook)

Determination of Detention Volume Using Modified FAA Method

Rainfall durations must be entered in an increasing order.

Rainfall Duration minutes (input)	Rainfall Intensity inch/hr (output)	Inflow Volume cubic feet (output)	Adjustment Factor (output)	Average Outflow cfs (output)	Outflow Volume cubic feet (output)	Storage Volume cubic feet (output)
0.00	12.02	0				
5.00	8.72	20,021	1.00	12.75	3,825	16,196
10.00	6.95	31,902	1.00	12.75	7,650	24,252
15.00	5.83	40,119	0.90	11.48	10,328	29,791
20.00	5.05	46,316	0.80	10.20	12,240	34,076
25.00	4.47	51,257	0.74	9.44	14,153	37,105
30.00	4.02	55,351	0.70	8.93	16,065	39,286
35.00	3.66	58,838	0.67	8.56	17,978	40,861
40.00	3.37	61,873	0.65	8.29	19,890	41,983
45.00	3.13	64,559	0.63	8.08	21,803	42,757
50.00	2.92	66,967	0.62	7.91	23,715	43,252
55.00	2.74	69,150	0.61	7.77	25,628	43,522
60.00	2.58	71,147	0.60	7.65	27,540	43,607
65.00	2.45	72,987	0.59	7.55	29,453	43,535
70.00	2.32	74,694	0.59	7.47	31,365	43,329
75.00	2.22	76,287	0.58	7.40	33,278	43,010
80.00	2.12	77,780	0.58	7.33	35,190	42,590
85.00	2.03	79,186	0.57	7.28	37,103	42,083
90.00	1.95	80,514	0.57	7.23	39,015	41,499
95.00	1.88	81,774	0.56	7.18	40,928	40,846
100.00	1.81	82,972	0.56	7.14	42,840	40,132
105.00	1.75	84,114	0.56	7.10	44,753	39,362
110.00	1.69	85,206	0.55	7.07	46,665	38,541
115.00	1.63	86,252	0.55	7.04	48,578	37,675
120.00	1.58	87,256	0.55	7.01	50,490	36,766
125.00	1.54	88,222	0.55	6.99	52,403	35,820
130.00	1.49	89,152	0.55	6.96	54,315	34,837
135.00	1.45	90,050	0.54	6.94	56,228	33,823
140.00	1.41	90,917	0.54	6.92	58,140	32,777
145.00	1.38	91,757	0.54	6.90	60,053	31,704
150.00	1.34	92,569	0.54	6.89	61,965	30,604
155.00	1.31	93,358	0.54	6.87	63,878	29,480
160.00	1.28	94,123	0.54	6.85	65,790	28,333
165.00	1.25	94,867	0.54	6.84	67,703	27,164
170.00	1.23	95,590	0.54	6.83	69,615	25,975
175.00	1.20	96,295	0.53	6.81	71,528	24,767
180.00	1.17	96,981	0.53	6.80	73,440	23,541

Stormwater Detention Volume (Cubic Feet) = 43,607

The required storage volume is 43,607 cubic feet (approx. 1.0 acre-foot). This compares to 1.21 acre-feet calculated for the same catchment in Design Example 6.1 using the empirical equations.

6.3 Example—Hydrograph Procedure Preliminary Sizing

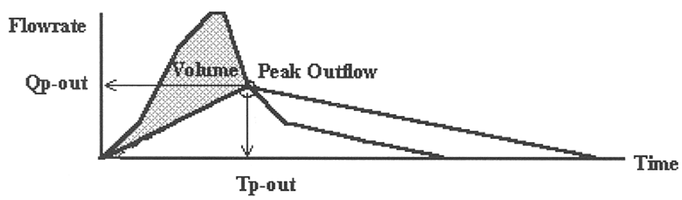
Use the hydrograph method to determine the preliminary size of a detention basin that will detain the 100-year peak flow to historic conditions for a catchment that has the following characteristics: area = 90 acres, length of catchment = 0.53 miles, length to centroid = 0.30 miles, impervious area = 67%, catchment slope = 0.0178 ft/ft, pervious retention = 0.35 inches, impervious retention = 0.05 inches, type B soils. 100-yr, 1-hour rainfall depth = 2.6 inches. The peak outflow is to be limited to the allowable unit release rates shown in Table SO-1.

The calculations are set up in tabular form similar to one illustrated in Table SO-3 that was taken from the UD-Detention workbook. The inflow hydrograph is calculated using the CUHP model and entered into the second column of the calculations. The preliminary sizing for the detention basin indicates a storage volume of 11.2 acre-feet.

Table SO-3—Detention Volume Estimate Using a Hydrograph
(From UD-Detention Workbook)

DETENTION VOLUME ESTIMATE USING A HYDROGRAPH

(For Catchments > 90 and < 160 acres)



a. Enter the inflow storm hydrograph with a time interval as:
time interval on hydrograph dt = 5.0 minutes

Time minutes (input)	Inflow hydrograph cfs (input)	**Outflow** Rising Hy cfs (output)	Incremental Volume acre-ft (output)	**Storage** Volume acre-ft (output)
0.0	0.00	0.00	0.00	0.00
5.0	0.00	4.78	0.00	0.00
10.0	3.00	9.56	0.00	0.00
15.0	16.00	14.34	0.01	0.01
20.0	41.00	19.13	0.15	0.16
25.0	84.00	23.91	0.41	0.58
30.0	179.00	28.69	1.04	1.61
35.0	312.00	33.47	1.92	3.53
40.0	333.00	38.25	2.03	5.56
45.0	289.00	43.03	1.69	7.25
50.0	240.00	47.81	1.32	8.58
55.0	196.00	52.59	0.99	9.56
60.0	160.00	57.38	0.71	10.27
65.0	135.00	62.16	0.50	10.77
70.0	115.00	66.94	0.33	11.10
75.0	88.00	71.72	0.11	11.22
80.0	64.00	76.50	0.00	11.22
85.0	47.00	----	----	----
90.0	36.00	----	----	----
95.0	29.00	----	----	----
100.0	25.00	----	----	----
105.0	23.00	----	----	----
110.0	21.00	----	----	----
115.0	20.00	----	----	----
120.0	20.00	----	----	----

b. You must provide the design information as:
Max. Allowable Peak Outflow Qp-out = 76.50 cfs
Time to Peak Outflow Tp-out= 80.00 minutes
(Qp-out,Tp-out) is a point on the recession of the inflow hydrograph.

C. Detention Storage Volume = 11.22 acre-ft

NOTE: THIS IS A FIRST APPROXIMATION ONLY

7.0 CHECKLIST

Criterion/Requirement	☐
If facility falls under State Engineer’s jurisdiction, it must it meet all of State Engineer’s requirements?	
Side slopes must be 4:1 or flatter.	
Embankment (dam fill) slopes must be 3:1 or flatter (4:1 or flatter preferred).	
Trickle channels are not required for retention ponds (“wet” ponds) and wetland basins, but the District will provide only limited maintenance assistance of these areas.	
The longitudinal slope for trickle channels shall be at least 0.4% for concrete bottoms and at least 1% for other bottoms.	
The pond bottom cross slope toward trickle channel or outlet shall be at least 1%.	
Maintenance access ramps to the pond bottom have at least 8 feet wide stabilized surface and have a 10%, or longitudinal flatter slope and turning radii that permit large maintenance equipment access.	
Provide an emergency spillway or embankment protection for flows that exceed primary outlet capacity.	
Provide a minimum 1-foot freeboard before embankment overtops.	
Outlet structures meter out the discharges as required by local municipality’s criteria.	
Trash racks provided that do not interfere with the hydraulic capacity of the outlet.	
Tributary inflow points to the ponds have adequate energy dissipation and/or protection to prevent erosion.	
Designs consider the safety of the public.	
Pre-sedimentation forebay provided.	
WQCV is increased by 20% to account for sediment accumulation.	
Geotechnical considerations (embankment stability, geologic hazard, seepage) are taken into account and documented.	
Vegetation takes into account frequency and duration of inundation.	

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