CHAPTER 5. DETENTION DESIGN

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1.0 INTRODUCTION

1.1 Purpose of the Chapter

The purpose of this chapter is to provide guidance for designing facilities to detain stormwater runoff from new developments and redevelopments. The intent of the detention facilities is to protect connected channels and adjoining property from adverse impacts caused by increased peak flow rates and runoff volumes that can result if stormwater control measures are not implemented.

1.2 Impact of Urbanization on the Quantity of Stormwater Runoff

Urbanization results in increased levels of imperviousness which frequently causes increased peak flow rates and increased runoff volumes from developed sites. Historically, the traditional approach for stormwater management was to move runoff away from structures and transportation systems as quickly and efficiently as possible. However, this approach resulted in impacts such as:

- Flooding of downstream properties;
- Widening and instability of downstream channels; and
- Habitat damage and ecosystem disruption, resulting in streambed and bank erosion which leads to the associated sediment and pollutant transport.

These types of adverse impacts will occur unless measures are taken to reduce the runoff and control the rate of discharge from newly developed sites.

2.0 APPLICABILITY

The stormwater detention requirements outlined in this chapter apply to all new developments. Waivers may be granted by the City for the following cases:

- Redevelopments of commercial, industrial, or multi-family home developments that are less than 1 acre and are not part of a larger common plan of development and
- Single-family residential developments of less than one acre that are not part of a larger common plan of development.

For sites that are smaller than 1 acre, or for sites that are being redeveloped, the City may allow the property owner to pay a fee in-lieu-of implementing the detention measures described in this chapter.

3.0 STORMWATER DETENTION DESIGN OBJECTIVES

The primary objectives of the City's stormwater detention requirements are described below:

- **Post-project peak flow rates must match pre-project conditions** - Onsite detention facilities must be designed so that peak flow rates for post-project conditions are limited to pre-project levels. To maintain peak flow rates at pre-development levels, a multi-frequency outlet design approach is required. The designer must demonstrate that the 1-, 2-, 10-, 25-, 50- and 100-year post-development peak flow rates are limited to the corresponding pre-development flow rates. If the detention facility is also being used to provide water quality treatment, then the calculated WQCV for the facility (see Chapter 10 – *Water Quality*) must be added to the 100-year storage volume of the facility.
- **Low-flow orifice** - Detention basins designs must include a low-flow orifice designed to discharge at the 1-year peak flow rate. The low-flow orifice must be a minimum of 2 inches in diameter to reduce the potential for plugging.
- **Spillways must be designed to convey 100-year runoff** - Overflow spillways for detention facilities must permit the passage of the runoff from the 100-year event, based on fully urbanized conditions for the entire tributary watershed with no upstream detention. A freeboard of 1 foot must be provided for the 100-year event design flows. If downstream safety considerations warrant, it may be necessary to size a spillway for greater than a 100-year event.

These criteria for peak flow attenuation apply for onsite facilities unless other rates are recommended in a City-approved master plan. As a result of these requirements, three conditions must be examined for determination of attenuation requirements for onsite facilities:

- Pre-project conditions
- Post-project conditions
- Fully urbanized conditions for the entire tributary watershed with no upstream detention.

3.1 Other Important Considerations for Detention Facility Selection and Design

In addition to the design considerations above, the following factors shall be considered when selecting and designing a detention facility for a site:

- ***Public Safety*** – Detention facilities shall be evaluated in terms of public safety and the risks or liabilities that occur during implementation. Public safety is always one of the most important design considerations. Wet detention ponds must have side slopes that are no steeper than 4:1 (H:V) and must incorporate fencing and/or a safety bench into the design.
- ***Public Acceptability*** - The detention facility shall consider the expected response from the public, particularly neighboring residential properties, if any.

- ***Agency Acceptability*** – Selection of a detention facility for a site shall consider the expected response of agencies that will oversee the facility and their relationship to regulatory requirements.
- ***Mosquito Control*** – A specific component of public health and safety related to the design of detention facilities is the issue of mosquito control. The potential for mosquito breeding and the spread of mosquito-borne illnesses in detention facilities must be addressed. In general, the biggest concern is the creation of areas with shallow stagnant water and low dissolved oxygen that creates prime mosquito habitat. Studies indicate that pools of deep water (≥ 5 feet) and pools with residence times less than 72 hours are less likely to breed mosquitoes. Therefore, dry detention basins must have outlets designed to drain in 24 to 48 hours. Careful design and proper management and maintenance of systems can effectively control mosquito breeding.
- ***Reliability/Maintenance/Sustainability*** – The detention facility shall be effective over an extended time and be able to be properly operated and maintained over time. This may involve requiring subdivision covenants and designating individuals responsible for the operation and maintenance of detention facilities.

4.0 TYPES OF DETENTION FACILITIES

4.1 Private versus Common Detention Facilities

There are two basic approaches to designing storage facilities, which vary depending on the type of development:

- **Onsite or private facilities:** Detention facilities that are planned on an individual site basis.
- **Common or regional facilities:** Detention facilities that are planned to serve multiple lots, a subdivision, or larger area.

Depending on the type of development, requirements for detention basins may vary, as described below:

- **Subdivision:** These are developments that involve the subdivision of property. One or more detention basins may be required depending on the natural drainage patterns of the development.
- **Single Lot Commercial:** Generally, these are developments on lots that are not part of a residential subdivision. Basins shall be designed for full development of the lot based on zoning unless land use restrictions dictate less land is available for development.
- **Multiple Properties:** Multiple properties or developments may be served by a regional basin that is not within the boundary of the development.

4.2 Type of Detention Facilities

Generally, the type of detention is determined by the required design objectives as well as the appearance and function desired by the developer. Detention basins fall into one of the following three design categories:

- **Dry detention basin -** Designed for several different frequency rainfalls for flood control only. Dry basins drain over 1 to 2 days. The outlet is typically composed of orifices and/or weirs.
- **Extended detention basin -** Designed for pollutant removal and potentially for flood control. Extended detention basins drain over an extended period of time, typically 1 to 3 days. The outlet is typically composed of a filtered control as well as orifices and/or weirs.
- **Wet basin –** A wet basin contains a permanent pool of water and is designed for pollutant removal, flood control, and often aesthetics. Wet basins may be designed to drain down to the permanent pool level over a short or long period of time.

Unplanned (or non-engineered) storage may also be present in features such as sinkholes and the upstream side of railroad and highway embankments. When planning a development along a major waterway, such non-engineered storage should be accounted for when calculating existing flow rates but generally should not be accounted for when calculating ultimate future peak flow rates.

5.0 HYDROLOGIC AND HYDRAULIC DESIGN

5.1 Detention Volume Design Methods

Two design methods that are acceptable for use in detention design are summarized in Table DET-1 below. The appropriate method is dependent on the detention volume required and the impervious area added by the development. When determining which method is acceptable, the calculated volume takes precedence over the impervious area added.

Table DET-1: Acceptable Detention Design Methods

Detention Design Method	**Acceptable Volume** (cubic feet)	**Approximate Acceptable Amount of Added Impervious Area**
Simplified (Modified FAA) Method (Section 5.1.1)	<20,000 ft^3	30 acres
Hydrograph Methods (Section 5.1.2)	Any size	> 30 acres

5.1.1 Modified FAA Rational-Based Method - For Detention Volume Less than 20,000 ft^3

For onsite detention volumes of less than approximately 20,000 ft^3 (this typically corresponds to developments with less than approximately 5 acres of residential development or less than 2.5 acres of commercial development), an acceptable simplified method of detention design is the Rational Method-based FAA Method (1966), as modified by Guo (1999a). This method can be used for: 1) multiple design events for a site to determine storage requirements for various return intervals, or 2) initial sizing of detention storage volumes whenever a detailed hydrograph routing design method is used.

The inputs required for the Modified FAA volume calculation procedure include:

> A = Area of the catchment tributary to the storage facility (acres)
>
> C = Runoff coefficient (unitless)
>
> Q_{po} = Allowable maximum peak outflow rate from the detention facility based on pre-project conditions (cfs)
>
> t_c = Time of concentration for the tributary catchment (see Chapter 4 – *Determination of Stormwater Runoff*) (minutes)
>
> i = Rainfall intensity corresponding to t_c for relevant return frequency storms (as determined from the intensity-duration-frequency table in Chapter 4 – *Determination of Stormwater Runoff*) (in/hr)

As shown by example in Section 7.1, the calculations are best set up in a tabular (spreadsheet) form (see Table DET-3). Each time increment (typically 5 minutes) is entered in rows, and the following variables are entered or calculated in each column:

1. **Storm Duration Time -** (t) (minutes), up to 180 minutes. For longer durations, a hydrograph-based method is required.
2. **Rainfall Intensity** – (i) (inches per hour), based on the intensity-duration-frequency table (Table RO-5) in Chapter 4 – *Determination of Stormwater Runoff*.
3. **Inflow volume** – (V_i) (ft^3), calculated as the cumulative volume at the given storm duration using the equation:

$$V_i = CiA(60t) \qquad \textbf{(Equation DET-1)}$$

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

$$m = \frac{1}{2}\left(1 + \frac{t_c}{t}\right) \qquad 0.5 \le m \le 1 \text{ and } t \ge t_c \qquad \textbf{(Equation DET-2)}$$

2. **Calculated average outflow rate** – (Q_{av}) (cfs), over the duration t:

$$Q_{av} = mQ_{po} \qquad \textbf{(Equation DET-3)}$$

3. **Calculated outflow volume** – (V_o) (ft^3), during the given duration and the adjustment factor at that duration calculated using the equation:

$$V_o = Q_{av}(60t) \qquad \textbf{(Equation DET-4)}$$

4. **Required storage volume** – (V_s) (ft^3), calculated using the equation:

$$V_s = V_i - V_o \qquad \textbf{(Equation DET-5)}$$

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.

Notes regarding the Rational Formula-Based Modified FAA Method

1. The Rational Formula Based Modified FAA Method may be used to find an ***initial*** storage volume for any size watershed. This technique for initial detention sizing yields best results when the tributary watershed area is less than 300 acres, but can be applied to larger watersheds, although the final design volumes may need to be adjusted significantly.
2. If the Modified FAA Method is used and it is determined that the required storage volume is greater than 20,000 ft^3, then a hydrograph method should be used to determine the basin storage requirements (see Section 5.1.2 for hydrograph methods).
3. Because the FAA Method calculates the required detention volume only, methods described in Section 5.2 must be used to design the outlet works.

5.1.2 Hydrograph Methods - For Detention Volume Greater than 20,000 ft^3

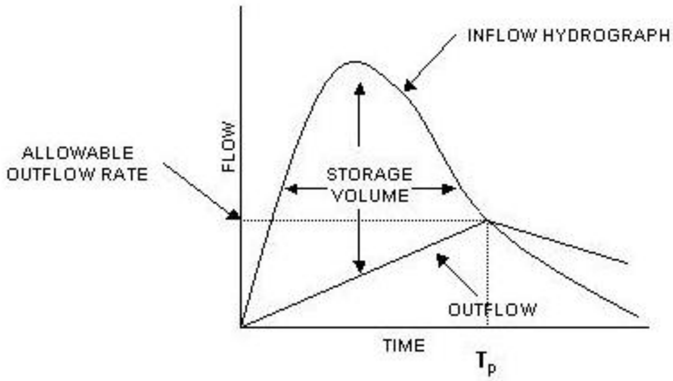
For detention volumes greater than 20,000 ft^3 (typically 5 acres or more of residential development or 2.5 acres or more of commercial development) the designer must use the hydrograph sizing procedures described in this section.

5.1.2.1 Hydrograph Volumetric Method – for Estimating Detention Volume

To make an initial estimate of the required storage volume for a detention facility of more than 20,000 ft^3, the Hydrograph Volumetric Method can be used to measure the difference between the inflow hydrograph and the proposed outflow hydrograph (i.e., the desired maximum release rates for the facility). This technique assumes that the required detention volume is equal to the difference in volume between the inflow hydrograph and the simplified outflow hydrograph. This is represented by the area between those 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 DET-1).

Generally, the inflow hydrograph is obtained from a hydrograph method using the Huff distribution presented in Chapter 4 – *Determination of Stormwater Runoff*. 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.

Figure DET-1: Hydrograph Volumetric Method of Detention Volume Sizing



The volume can be calculated by setting up tabular calculations, as shown by example in Table DET-4 (see Section 7.2). Descriptions of the variables in the table columns include:

1. **Time -** (T) (minutes), from 0 to T_p in uniform increments. Time increments (Δt) of 5 minutes are typically used. T_p is the time (in minutes) where the descending limb of the inflow hydrograph is equal to the allowable release rate.

2. **Inflow rate** - (Q_i) (cfs), to the detention basin corresponding to the time T. The inflow rate can be obtained using the SCS Unit Hydrograph Method with the Huff distribution presented in Chapter 4 – *Determination of Stormwater Runoff*.

3. **Outflow rate** – (Q_o) (cfs), calculated as:

$$Q_o = \frac{T}{T_p} Q_{po}$$ **(Equation DET-6)**

In which:

Q_{po} = the peak outflow rate. The allowable peak outflow rate is determined from City criteria or a City-approved master plan.

4. **Incremental Storage Volume** - (V_s) (acre-feet), calculated as:

$$V_s = (Q_i - Q_o) \cdot \Delta t \cdot 60 \text{ seconds}$$ **(Equation DET-7)**

5. **Total cumulative storage volume –** (acre-feet), calculated as the sum of the incremental storage volumes:

$$V_s total = \sum V_s \text{ incremental}$$ **(Equation DET-8)**

5.1.2.2 Modified Puls Method – For Design of Detention Facilities

To design detention facilities larger than 20,000 ft^3, the Modified Puls method is recommended for reservoir routing for detention facility design. This reservoir routing method calculates an outflow hydrograph for a detention facility based on a given inflow hydrograph and the storage-outflow characteristics of a facility. This method is typically implemented using computer programs such as HEC-HMS, TR-20 or proprietary software packages. Model input is typically a storage-outflow relationship for the detention facility. This section provides background on the Modified Puls method. The description is adapted from *Fundamentals of Hydraulic Engineering Systems* (Hwang and Houghtalen 1996). An example of the Modified Puls method is included with the other examples at the end of this Section.

The mathematical basis of Modified Puls routing is the continuity equation (conservation of mass with constant density). Simply stated, the change in storage is equal to inflow minus outflow. In differential format, the equation can be expressed as:

$$\frac{dS}{dt} = I - O$$ **(Equation DET-9)**

Where:

dS/dt = rate of change of storage with respect to time

I = instantaneous inflow

O = instantaneous outflow

If average rates of inflow and outflow are used, an acceptable solution can be obtained over a discrete time step (Δt) using:

$$\frac{\Delta S}{\Delta t} = \bar{I} - \overline{O}$$ **(Equation DET-10)**

Where ΔS is the storage change over the time step. By assuming linearity of flow across the time step, the storage equation may be expressed as:

$$\Delta S = \left[\frac{(I_i + I_j)}{2} - \frac{(O_i + O_j)}{2} \right] \cdot \Delta t$$ **(Equation DET-11)**

Where the subscripts *i* and *j* designate inflow and outflow at the beginning and end of the time step, respectively.

The storage relationship in Equation DET-11 has two unknowns. Because the inflow hydrograph must be defined prior to performing the routing calculations (using the SCS Unit Hydrograph Method with the Huff rainfall distribution), inflow values (I_i and I_j) are known. Likewise, the time increment (Δt) is chosen, and outflow at the beginning of the time step (O_i) was solved in the previous time step calculations (or specified as an initial value). That leaves the storage increment (ΔS) and the outflow at the end of the time step (O_j) as unknowns. Because both storage and outflow (for uncontrolled outlet devices) are related to the depth of water in the detention facility, they are related to one another. This relationship is employed to compute the solution.

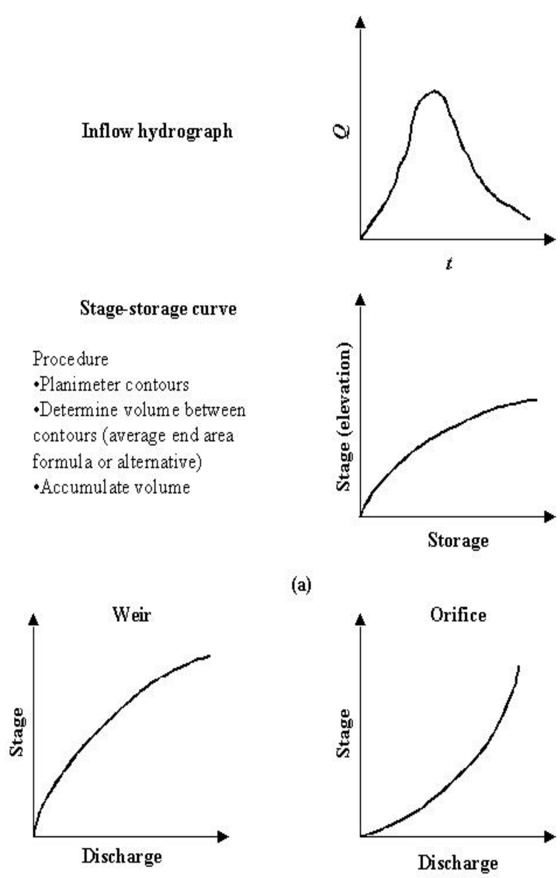
The data requirements to perform Modified Puls reservoir routing include:

1. An inflow hydrograph (determined using the SCS Unit Hydrograph Method as described in Chapter 4 – *Determination of Stormwater Runoff*).
2. A storage versus outflow relationship for the detention facility (see Section 5.2 for outlet works calculations). The stage-storage and stage-outflow relationships may be used to generate the storage-outflow relationship.

Figure DET-2 displays these data requirements graphically. The procedure for obtaining the stage (elevation) versus storage curve is described in the figure. Also, the two basic types of outlet devices (weirs and orifices) are noted with typical stage-discharge relationships.

Figure DET-2: Data Requirements for Storage Routing

(Source: UDFCD USDCM, adapted from Hwang and Houghtalen, 1996)



The Modified Puls routing method reformulates Equation DET-11, as shown by Equation DET-12:

$$(I_i + I_j) + \left[\frac{2S_i}{\Delta t} - O_i\right] = \left[\frac{2S_j}{\Delta t} + O_j\right] \qquad \textbf{(Equation DET-12)}$$

Where (S_j-S_i) equals the change in storage (ΔS). The advantage of this expression is that all of the known values are on the left side and all of the unknowns are grouped on the right.

The solution procedure for Modified Puls routing is as follows:

1. Determine the appropriate inflow hydrograph for the detention facility (see Chapter 4 – *Determination of Stormwater Runoff*).
2. Select a routing interval (Δt). Linearity of inflows and outflows over the time step is assumed.
3. Determine stage-storage relationship for the detention facility.
4. Determine stage-discharge relationship for the outlet device(s) selected (see Section 5.2 for calculations regarding stage-discharge relationship for outlet works).
5. Establish the storage-outflow relationship by setting up a table with the following headings (note that headings b through e correspond with variables in Equation DET-12:
    a. Elevation
    b. Outflow (O)
    c. Storage (S)
    d. 2S/Δt
    e. 2S/Δt+O
4. Plot the (2S/Δt+O) versus *O* relationship.
5. Perform routing using a table with the following headings:
    a. Time
    b. Inflow at time step i (I_i)
    c. Inflow at time step j (I_j)
    d. *2S/Δt*-O
    e. 2S/*Δt+O*
    f. Outflow

For an example application of the Modified Puls method, see Section 7.3.

5.2 Outlet Works Design

To maintain peak flow rates at pre-development levels, a multi-frequency outlet design approach is required. The designer must demonstrate that the 1-, 2-, 10-, 25-, 50- and 100-year post-development peak flow rates are limited to the corresponding pre-development flow rates. The outlet design must be compatible with the calculated volume and volume design for each design event to ensure peak discharges do not exceed pre-development rates for each design event. For example, for the water surface elevation corresponding to the volume calculated for the 10-year event, the outlet should be designed to discharge no greater than the 10-year pre-development peak flow rate. If the facility is also providing water quality treatment, then the detention volume and outlet design must also incorporate the WQCV (See Chapter 10 – *Water Quality*).

The hydraulic capacity of the various components of the outlet works (i.e., pipes, orifices, weirs) can be determined using standard hydraulic equations described below. (Note: Because the discharge pipe of an outlet works functions as a culvert, the reader is directed to Chapter 8 – *Culvert & Bridge Hydraulic Design*, for guidance regarding the calculation of the hydraulic capacity of outlet pipes).

To create a rating curve for an entire outlet, a composite total outlet rating curve can be developed based on the rating curves developed for each of the components of the outlet and then selecting the most restrictive element that controls the release at a given stage.

5.2.1 Orifices

Single or multiple orifices may be used in a detention facility and are commonly used as a low-flow control. The hydraulics of each can be superimposed to develop the outlet rating curve. The basic orifice equation is:

$$Q = C_o A_o (2gH_o)^{0.5} \qquad \textbf{(Equation DET-13)}$$

Where:

Q = orifice discharge flow rate (cfs)

C_o = discharge coefficient (use 0.61 for square-edged, uniform opening, ranging down to 0.4 for ragged edge orifice)

A_o = area of orifice (ft^2)

H_o = effective head on the orifice (ft)

g = gravitational acceleration (32.2 ft/s^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 orifice discharge is submerged, then the effective head is the difference in elevation of the upstream and downstream water surfaces.

5.2.2 Weirs

Several different types of weirs may be used, including:

- Rectangular sharp-crested weirs
- Broad-crested weirs
- Broad-crested slot and v-notch weirs

The methods for calculating the discharge from these types of weirs are described below:

Rectangular Sharp-Crested Weirs: A sharp-crested weir is defined as a weir with a wall thickness of 6 inches or less. The basic equation for a rectangular sharp-crested weir is:

$$Q = CL_{eff}H^{3/2}$$ **(Equation DET-14)**

Where:

Q = Weir discharge (cfs)

L_{eff} =effective horizontal weir length (ft) (as calculated in Equation DET-15 to account for contractions)

$$L_{eff} = L_{total} - 0.1 \cdot N \cdot H$$ **(Equation DET-15)**

Where (for L_{eff}):

L_{total} = the total weir length (ft)

N = number of contracted sides*

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

*N=0 corresponds to the case of a suppressed rectangular weir, for which the channel width is equal to the weir opening length, and N=2 corresponds to the case of a contracted rectangular weir, where both sides of the weir are some distance inward away from the channel edge, narrowing (contracting) the channel width.

C = weir coefficient (as calculated in Equation DET-16 or DET-17)

The weir coefficient is a function of the head above the weir crest, H, and the height of the weir crest above the pond or channel bottom, H_c. For ratios of H/H_c up to approximately 10, the following equation should be applied to determine C (Debo and Reese 2003):

$$C = 3.237 + 0.428 \cdot \frac{H}{H_c} + 0.0175 \cdot H$$ **(Equation DET-16)**

For ratios of H/H_c greater than 15, the weir coefficient is found using:

$$C = 5.68(1 + \frac{H_c}{H})^{1.5}$$ **(Equation DET-17)**

For ratios of H/H_c between 10 and 15, the designer should interpolate between Equations DET-16 and DET-17.

Broad-Crested Weirs: The equation for a broad-crested weir is:

$$Q = CLH^{3/2}$$ **(Equation DET-18)**

Where:

Q = Weir discharge (cfs)

C = Broad-crested weir coefficient (from Table DET-2)

L = Broad-crested weir length (ft) (For weirs with tapered sides, it is acceptable to use a length equal to the average of the upper and lower weir lengths.)

H = Head above weir crest (ft)

Table DET-2: Broad-Crested Weir Coefficients

Head Above Weir (ft)	C 6-inch thick wall crest	C 8-inch thick wall crest	C 12-inch thick wall crest	C 10-foot thick wall crest
0.2	2.80	2.75	2.69	2.49
0.4	2.92	2.80	2.72	2.56
0.6	3.08	2.89	2.75	2.70
0.8	3.30	3.04	2.85	2.69
1.0	3.32	3.14	2.98	2.68
1.2	3.32	3.20	3.08	2.69
1.4	3.32	3.26	3.20	2.67
1.6	3.32	3.29	3.28	2.64
1.8	3.32	3.32	3.31	2.64
2.0	3.32	3.31	3.30	2.64
2.2	3.32	3.32	3.31	2.64
2.5	3.32	3.32	3.32	2.64
3.0	3.32	3.32	3.32	2.64
3.5	3.32	3.32	3.32	2.64
4.0	3.32	3.32	3.32	2.64

Source: Brater and King, 1976.

Broad-Crested Slot and V-Notch Weirs: Capacity of broad-crested slot and V-notch weirs shall be determined by the following equation:

$$Q = 0.86H + (3.65W + 5.82z)H^{1/2}$$ **(Equation DET-19)**

(Source: J. Wilson, University of Missouri-Rolla)

In which:

Q = discharge (cfs)

H = upstream head (ponded depth above the slot invert) (ft) (maximum of 6 ft)

W = slot invert width perpendicular to flow (ft) ($0.333<W<2.0$)

z = slope of slot sides expressed in terms of H: V ($0<z<0.6$)

6.0 OTHER DESIGN CONSIDERATIONS

6.1 Potential for Multiple Uses

When designing a detention facility, multi-purpose uses, such as active or passive recreation and wildlife habitat, are encouraged in addition to providing the required storage volume. Facilities used for recreation should be designed to inundate no more frequently than every two years.

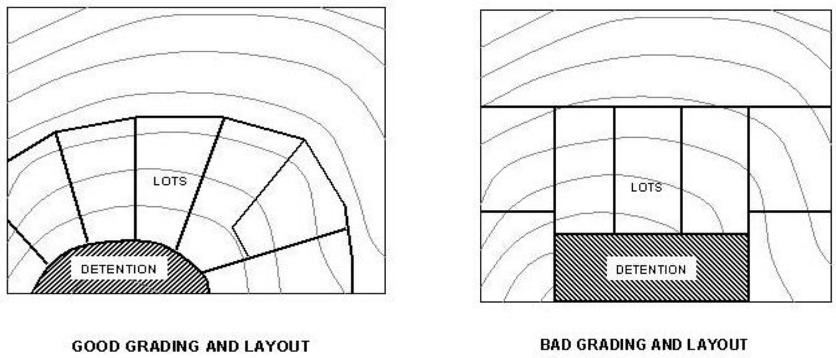
6.2 Detention Basin Location

Detention basins preferably should be located at the natural low point of the site and must discharge to the natural drainage location to minimize downstream impacts.

6.3 Detention Basin Grading

Detention basin grading shall conform to the natural topography of the site to the maximum extent practical. Developments should be laid out around the existing waterways and proposed detention basin (see Figure DET-3). Layouts conforming to existing topography often reduce construction costs, land disturbance and maintenance costs, and increase aesthetic quality. Existing slopes should be used to the maximum extent practical. If slopes are modified, the maximum unrestrained allowable slope is 3H:1V. Exceptions to these criteria must be justified through engineering and/or economic studies. Significant modifications to existing topography may require geologic impact studies and geotechnical analysis, particularly where shallow bedrock is believed to be present.

Figure DET-3: Examples of Good and Bad Location, Grading and Lot Layout for Detention
Source: UDFCD USDCM



6.4 Geometry of Storage Facilities

The geometry of a detention facility depends on specific site conditions such as adjoining land uses, topography, geology, existing natural features, volume requirements, etc. The following criteria apply to the geometry of detention facilities:

- **Pond side slopes** - Pond side slopes of 3:1 (H:V) are the maximum permissible. Slopes between 5H:1V and 10H:1V are encouraged. If slopes steeper than 3H:1V are desired, the engineer must demonstrate why 3H:1V slopes are not feasible and provide an explanation regarding how the steeper slopes will be maintained and how safety concerns will be addressed.
- **Pond bottom slopes** - Pond bottom slopes must be a minimum of 1 percent to ensure drainage.
- **Pond shape** - The water quality portion of a facility (if present) should be shaped with a gradual expansion from the inlet and a gradual contraction toward the outlet, thereby minimizing short-circuiting. The minimum length:width ratio should be 2:1. Storage facility geometry and layout are best developed with input from a land planner/landscape architect.
- **Low-flow channel** - A 5-foot wide concrete low-flow channel shall be provided where the 2-year design flow exceeds 5 cfs. However, for water quality basins or wetlands, concrete low-flow channels may not be desirable. Alternative materials, as described below, should be discussed with and approved by City staff.
- **Outlet & Materials** - Hard improvements such as concrete, asphalt, and metal must be used to control the 1-year design flow, except for wetlands or water quality basins where a hard bottom is not desirable. In such cases, a mixture of soil and rock planted with appropriate vegetation may be used for the low flow channel. Between the 1- and 10-year design flows, hard armor/grass composites may be considered, provided that velocities are low enough to ensure stability. Sod, turf reinforcement mat or other composite designs may be used above the 10-year water surface,

provided that they are appropriate for design velocities. Sod is acceptable for velocities less than 4 fps. Turf reinforcement mat or other composite materials are acceptable for velocities less than 8 fps. For velocities of 8 fps or more, a manufactured hard lining, riprap, or other suitable armor material is necessary (see Chapter 7 – *Open Channel Flow Design*).

6.5 Embankments and Cut Slopes

If the detention storage structure is a jurisdictional facility, meaning it is subject to regulation by the Arkansas Natural Resources Commission (ARNC), the embankment shall be designed, constructed, and maintained to meet most current ARNC criteria for jurisdictional structures. The design for an embankment of a storm water detention or retention storage facility should be based upon a site-specific engineering evaluation. The embankment should be designed to prevent catastrophic failure during the 100-year and larger storms as well as other natural disasters such as wildfires, tornados, and hurricanes with established design guidelines, standards, or other requirements. The following criteria frequently apply (ASCE and WEF 1992):

- **Side Slopes:** For ease of maintenance, side slopes of the embankment should not be steeper than 3H:1V. The embankment's side slopes should be well vegetated, and riprap protection (or the equivalent) may be necessary to protect it from wave action on the upstream face, especially in retention ponds.
- **Emergency Spillway:** An emergency spillway will usually be needed to convey flows that exceed the capacity of the primary outlet and/or impoundment.
- **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 relevant, all Arkansas Natural Resources Commission dam safety criteria must be carefully considered when determining the freeboard capacity of an impoundment.
- **Settlement:** The design height of the embankment should be increased by roughly 5% 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 material subject to decomposition. The fill material in all earth dams and embankments should be compacted to at least 95% of the maximum density obtained from compaction tests performed by the Modified Proctor method in ASTM D698.

6.6 Linings

Detention facilities may require an impermeable clay or synthetic liner for a number of reasons. Storm water detention and retention facilities have the potential to raise the groundwater level in the vicinity of the basin. If the basin is close to structures or other facilities that could be damaged by raising the groundwater level, consideration should be given to lining the pond. An impermeable liner may also be warranted in a retention basin where the designer seeks to limit seepage from a permanent pond. Alternatively, there are situations where the designer may seek to encourage seepage of storm water into the ground. In this situation, a layer of permeable material may be warranted.

6.7 Inlets and Forebays

Inlets to the detention facility should incorporate energy dissipation or outlet protection designed to handle discharge velocities into the pond to limit erosion and should be designed in accordance with criteria in Chapter 7 – *Open Channel Flow Design*.

A forebay, while optional, should be considered when the design volume exceeds 20,000 ft^3 or a large sediment, trash, or debris load is anticipated due to upstream land use. A forebay provides an opportunity for larger particles to settle out in the inlet area, which has a solid surface bottom to facilitate mechanical sediment removal. The forebay volume for the extended dry detention basin should be between 3 and 5 percent of the design volume. Forebays will need regular maintenance to reduce the sediment being transported and deposited on the storage basin's bottom.

6.8 Outlet Works

Outlet works should be sized and structurally designed to release at the specified flow rates without structural or hydraulic failure. Design guidance for outlet works used for water quality purposes is included in Chapter 10 – *Water Quality*.

6.9 Trash Racks

Trash racks should be sized to not interfere with the hydraulic capacity of the outlet and must be designed in a manner that is protective of public health, safety and welfare. See Chapter 10 – *Water Quality* for minimum trash rack sizes.

6.10 Vegetation

The type of vegetation specified for a newly constructed storage facility is a function of several factors, including:

- The frequency and duration of inundation of the area
- Soil types

- The desire for native versus non-native vegetation
- Other potential uses of the area (e.g., park, open space, etc.)

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.

6.11 Public Safety Concerns

For retention ponds (i.e., a pond that typically has a permanent pool), the pond must either have a safety bench or be surrounded by fencing a minimum of 48 inches in height.

For detention ponds (i.e., a pond that is generally dry), and especially if children are apt to play in the vicinity of the impoundment, use of relatively flat side slopes along the banks is advisable. In addition, installation of landscaping that will discourage entry, such as thick, thorny shrubs, is suggested for locations along the periphery, near the inlets and outlets, and at steeper embankment sections.

The use of thin steel plates as sharp-crested weirs should be avoided because of potential accidents, especially with children. If steel plate weir's or orifice plates are desired, the designer must demonstrate how exposed edges will be protected.

If the impoundment is situated adjacent to and at a lower grade than a street or highway, installation of a guardrail between the roadway and the pond is required. Guard Rail shall comply with all standard street and highway specifications.

6.12 Operations and Maintenance

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

1. **Inspection/Maintenance access -** The facility should be accessible for personnel to inspect and for maintenance equipment to remove silt and debris and for repair of damages that may occur over time. An access easement and/or right-of-way is required to allow access to the impoundment by the owner or agency responsible for maintenance. The access should have a maximum grade of 10 percent and have a solid driving surface of gravel, rock, concrete, or turf on a stabilized bed designed to support vehicle loads.
2. **Sediment removal considerations** - Permanent ponds should have provisions for complete drainage for sediment removal or other maintenance. The frequency of sediment removal will vary. It is dependent 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. Sediment should be removed when its depth accumulates to 6 inches, or 5%, whichever is less. A depth gauge is required at the outlet to facilitate determining when

sediment removal is necessary as well as the pond depth. Also, appearance may dictate more frequent cleaning. Detention facilities shall be designed with sufficient depth to allow accumulation of sediment for several years prior to its removal. A general guideline is to oversize the storage capacity of a detention facility by 20 percent of the WQCV (see Chapter 10 – *Water Quality*) to allow for sediment storage.

3. **Sediment concerns** - Secondary uses that are incompatible with sediment deposits should not be planned unless a high level of maintenance will be provided. French drains or the equivalent are almost impossible to maintain and should be used with discretion where sediment loads are apt to be high.
4. **Dissolved oxygen concentrations in pond** - Adequate dissolved oxygen supply in wet ponds (to minimize odors and other nuisances) shall be maintained by artificial aeration. Use of fertilizer and pesticides adjacent to the permanent pool pond should be carefully controlled.
5. **Underground tank maintenance** - Underground tanks or conduits designed for detention should be sized and designed to permit pumping. Multiple entrance points shall be provided to remove accumulated sediment and trash.
6. **Permanent pool depth** - Permanent pools shall have a minimum depth of 6 feet to discourage excessive aquatic vegetation on the bottom of the basin, unless the vegetation is specifically provided for water quality purposes.
7. **Aesthetics and landscaping** - Trash racks and/or fences are often used to minimize hazards. These may become eyesores, 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. Creative designs, integrated with innovative landscaping, can be safe and can also enhance the appearance of the outlet and pond. In addition, bank slopes, bank protection, and vegetation types are important design considerations for site aesthetics and maintainability.
8. **Avoid moving parts** - To reduce maintenance and avoid operational problems, outlet structures should be designed with no moving parts (i.e., use only pipes, orifices, and weirs).
9. **Outlet openings** - To reduce maintenance, outlets should be designed with openings as large as possible, be compatible with the depth-outflow relationships desired, and be designed with water quality, safety, and aesthetic objectives in mind.
10. **Resistant to vandalism** - Outlets should be robustly designed to lessen the chances of damage from debris or vandalism.
11. **Maintenance of forebays and sediment traps** - Clean out all forebays and sediment traps on a regular basis or when routine inspection shows them to be ¾ full.

See Chapter 10 – *Water Quality*, 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.

6.13 Access

All-weather, stable access to the bottom, inflow, forebay, and outlet works areas shall be provided for inspection personnel and maintenance vehicles. Maximum grades should be 10%, and a solid driving surface of gravel, rock, concrete, or turf on a stabilized bed designed to support vehicle loads.

6.14 Geotechnical Considerations

The designer must account for 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 larger detention and retention storage facilities.

6.15 Environmental Permitting and Other Considerations

The designer must account for environmental considerations surrounding the facility and the site during its selection, design and construction. These can include regulatory questions such as: 1) Will the facility be located in a jurisdictional wetland? 2) Will the facility be located on a waterway regulated by the USACE as a “Water of the U.S.” and/or 3) Are there threatened and endangered species or habitat in the area? See Chapter 1 – *Stormwater Submittal Requirements* for more information on regulatory and permitting requirements.

Other non-regulatory environmental issues should also be taken into account. Detention facilities can become breeding grounds for mosquitoes unless they are properly designed, constructed and maintained. Area residents may object to facilities that impact riparian habitat or wetlands. Considerations of this kind must be carefully accounted for, and early discussions with relevant federal, state and local regulators are recommended.

7.0 EXAMPLES

7.1 Rational Formula-Based Modified FAA Procedure Example

Use the Rational Formula-Based Modified FAA Procedure (described in Section 5.1.1) to determine the required detention volume for the 100-year storm event for a 40-acre watershed, based on single-family land use. The watershed has a 100-year runoff coefficient of 0.56 and a time of concentration of 25 minutes. The post-development 100-year, undetained peak flow rate from the watershed is 157 cfs. The pre-project 100-year peak flow rate for the site is 90 cfs.

Given the information above, the following variables are known:

A = 40 acres

C = 0.56

Q_{po} = 90 cfs

t_c = 25 minutes

Following the methodology outlined in Section 5.1.1, Table DET-3 can be created to determine the required detention volume.

Table DET-3: Rational Formula-Based Modified FAA Procedure *Example*

Rainfall Duration (min)	Rainfall Intensity (in/hr)	Inflow Volume (ft^3)	Outflow Adjustment Factor	Calculated Average Outflow (cfs)	Calculated Outflow Volume (ft^3)	Required Storage Volume (ft^3)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	-----	-----	-----	-----	-----	-----
5	11.76	79,027	1.00	90.0	27,000	52,027
10	10.32	138,701	1.00	90.0	54,000	84,701
15	8.84	178,214	1.00	90.0	81,000	97,214
20	7.91	212,621	1.00	90.0	108,000	104,621
25	7.2	241,920	1.00	90.0	135,000	106,920
30	6.4	258,048	0.92	82.5	148,500	109,548
35	5.8	272,832	0.86	77.1	162,000	110,832
40	5.32	286,003	0.81	73.1	175,500	110,503
45	4.95	299,376	0.78	70.0	189,000	110,376
50	4.58	307,776	0.75	67.5	202,500	105,276
55	4.26	314,899	0.73	65.4	216,000	98,899
60	4.03	324,979	0.71	63.8	229,500	95,479
65	3.78	330,221	0.69	62.3	243,000	87,221
70	3.6	338,688	0.68	61.1	256,500	82,188
75	3.47	349,776	0.67	60.0	270,000	79,776
80	3.35	360,192	0.66	59.1	283,500	76,692
85	3.23	368,995	0.65	58.2	297,000	71,995
90	3.11	376,186	0.64	57.5	310,500	65,686
95	2.98	380,486	0.63	56.8	324,000	56,486
100	2.86	384,384	0.63	56.2	337,500	46,884
105	2.74	386,669	0.62	55.7	351,000	35,669
110	2.62	387,341	0.61	55.2	364,500	22,841
115	2.49	384,854	0.61	54.8	378,000	6,854
120	2.37	382,234	0.60	54.4	391,500	0

Notes:
Column (1) Storm duration (t) in 5-minute increments (typical)
Column (2) Intensity for storm duration (t) from intensity-duration-frequency table in Chapter 4 – *Determination of Stormwater Runoff*. Note: some values are from linear interpolation of tabular data.
Column (3) =C*Col (2)*A*60*Col (1) = 0.56*Col(2)*40*60*Col (1) [Equation DET-1]
Column (4) = 0.5*(1+[tc/Col (1)]) = 0.5*(1+[25/Col (1)]) [Equation DET-2]
Column (5) = Col (4)*Qpo = Col (4)*90 [Equation DET-3]
Column (6) = Col (5)*60*Col (1) [Equation DET-4]
Column (7) = Col (3) – Col (6) [Equation DET-5]
Shaded cell in Column 7 denotes maximum required detention volume using the Modified FAA Procedure.

The required detention volume is determined from the maximum storage volume (see column 7 in Table DET-3). For this example, the required detention volume is 110,832 ft^3 or 2.5 acre-feet (see shaded cell in Table DET-3). Because this volume exceeds the 20,000-ft^3 threshold for applicability of the FAA method for final detention sizing, this should be treated as an initial estimate, and a hydrograph-based method should be used to determine detention storage requirements.

7.2 Hydrograph Volumetric Method Example

Use the Hydrograph Volumetric method (described in Section 5.1.2.1) to determine the preliminary detention volume required, given an inflow hydrograph for a 20-acre commercial site (calculated according to guidelines in Chapter 4 – *Determination of Stormwater Runoff*) and a maximum allowable release rate of 30 cfs.

The tabular format for use with the inflow hydrograph method is shown in Table DET-4 below. The time and flow ordinates of the inflow hydrograph are entered in columns 1 and 2. Based on the inflow hydrograph, the allowable release rate of 30 cfs is matched on the falling limb at a time between 102 and 108 minutes, so 108 minutes is used as an estimate for T_p.

Table DET-4: Simplified Detention Volume Calculation *Example*

Time (min)	Inflow Hydrograph (cfs)	Outflow Rising Hydrograph (cfs)	Incremental Storage Volume (ac-ft)	Cumulative Storage Volume (ac-ft)
(1)	(2)	(3)	(4)	(5)
0	0	0	0.00	0.00
6	0	2	0.00	0.00
12	5	3	0.02	0.02
18	41	5	0.30	0.31
24	97	7	0.75	1.06
30	128	8	0.99	2.05
36	130	10	0.99	3.05
42	122	12	0.91	3.95
48	107	13	0.78	4.73
54	91	15	0.63	5.36
60	77	17	0.50	5.86
66	66	18	0.40	6.26
72	56	20	0.30	6.56
78	45	22	0.19	6.75
84	37	23	0.12	6.87
90	33	25	0.07	6.94
96	31	27	0.04	6.98
102	30	28	0.02	7.00
108	30	30	0.00	7.00
114	28			

Columns (1) & (2) Input from SCS Unit Hydrograph analysis with Huff distribution
Column (3) = (T/Tp)*Qpo = (Col(1)/108)*30 [Equation DET-6]
Column (4) = ((Col (2) – Col (3))*60*6)/43560. (includes unit conversion). Note: if Col (2) – Col (3) < 0, then Col (4) = 0.
Column (5) = (Col (5) Row (i-1)) + (Col (4) Row (i))

7.3 Modified Puls Method - Reservoir Routing Example

Use the Modified Puls Method (described in Section 5.1.2.2) to determine the outflow hydrograph for a proposed detention facility. Given the inflow hydrograph from the example in 7.2 for a 20-acre commercial site, a detention basin with the stage-storage relationship in Table DET-5 is proposed.

Table DET-5: Stage-Storage Relationship for Detention Facility

Stage (Elevation above Mean Sea Level)	Storage (acre feet)
1320	0
1321	0.5
1322	1.5
1323	4.0
1324	7.0
1325	10.0

The stage-outflow relationship for the detention facility outlet structure (determined from hydraulic analysis) is summarized in Table DET-6.

Table DET-6: Stage-Outflow Relationship for Detention Facility

Stage (Elevation above Mean Sea Level)	Outflow (cfs)
1320	0
1321	5
1322	10
1323	20
1324	30
1325	40

The following steps are used to determine the outflow hydrograph for this proposed facility:

1. **Determine the inflow hydrograph -** The inflow hydrograph should be developed following guidance in Chapter 4 – *Determination of Stormwater Runoff*.

2. **Select a routing interval (Δt) -** A rule of thumb for selecting the routing interval is to divide the rising limb of the hydrograph into ten increments. Since it takes about 40 minutes for the hydrograph to peak, use a routing interval of 4 minutes.

3. **Storage-outflow relationship** - Establish the storage-outflow relationship as shown in Table DET-7:

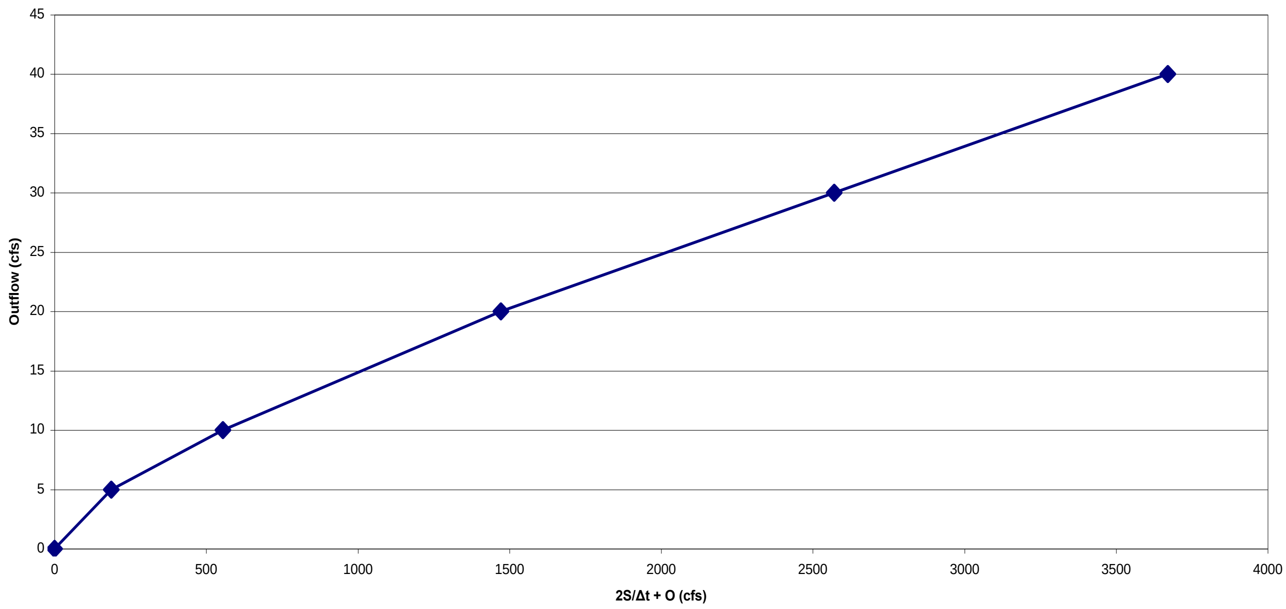
Table DET-7: Storage-Outflow Relationship for Detention Facility

Stage (Elevation above Mean Sea Level)	Outflow (*O*) (cfs)	Storage (*S*) (acre-feet)	*2S/Δt* (cfs)	*2S/Δt + O* (cfs)
(1)	(2)	(3)	(4)	(5)
1320	0	0.0	0	0
1321	5	0.5	182	187
1322	10	1.5	545	555
1323	20	4.0	1452	1472
1324	30	7.0	2541	2571
1325	40	10.0	3630	3670

Columns (1) and (2) from Table DET-5
Columns (1) and (3) from Table DET-6
Column (4) = 2S/Δt*(unit conversion) = 2*Col (3)/(4 min * 60 sec/min)* (43560 ft^2/acre)
Column (5) = Col (4) + Col (2)

4. **Plot the (*2S/Δt*) + *O* versus *O* relationship** - Plot values from Table DET-7. This relationship is shown in Figure DET-4.

Figure DET-4: 2S/Δt +O versus O for Reservoir Routing *Example*



5. **Perform the Modified-Puls routing using a table:**

   An example of the Modified-Puls routing method is shown in Table DET-8. Table heading descriptions are provided following the table.

Table DET-8: Modified Puls Routing Table

Time (min)	Inflow (I_i) (cfs)	Inflow (I_j) (cfs)	*2S/Δt – O* (cfs)	*2S/Δt + O* (cfs)	Outflow (*O*) (cfs)
(1)	(2)	(3)	(4)	(5)	(6)
0	0.00	0.01	0	--	0
4	0.01	0.59	0.01	0.01	0.0006
8	0.59	5.40	0.59	0.62	0.02
12	5.40	25.61	6.23	6.58	0.18
16	25.61	60.13	35.24	37.23	1.00
20	60.13	97.40	114.48	120.97	3.24
24	97.40	121.10	259.69	272.01	6.16
28	121.10	130.28	460.26	478.19	8.96
32	130.28	130.03	688.22	711.64	11.71
36	130.03	124.85	919.94	948.53	14.29
40	124.85	117.18	1141.29	1174.81	16.76
44	117.18	107.44	1345.25	1383.32	19.03
48	107.44	96.71	1528.09	1569.87	20.89
52	96.71	86.37	1687.50	1732.24	22.37
56	86.37	77.29	1823.33	1870.58	23.63
60	77.29	69.90	1937.62	1986.99	24.69
64	69.90	63.07	2033.65	2084.81	25.58
68	63.07	56.02	2113.98	2166.62	26.32
72	56.02	48.75	2179.22	2233.07	26.93
76	48.75	42.31	2229.21	2283.99	27.39
80	42.31	37.42	2264.82	2320.26	27.72
84	37.42	34.42	2288.67	2344.55	27.94
88	34.42	32.54	2304.35	2360.52	28.08
92	32.54	31.38	2314.95	2371.31	28.18
96	31.38	30.72	2322.37	2378.87	28.25
100	30.72	30.30	2327.86	2384.46	28.30
104	30.30	29.96	2332.19	2388.88	28.34
108	29.96	29.24	2335.70	2392.46	28.38
112	29.24	26.98	2338.11	2394.90	28.40
116	26.98	24.08	2337.55	2394.33	28.39
120	24.08	21.58	2331.93	2388.61	28.34
124	21.58	19.40	2321.11	2377.59	28.24
128	19.40	16.20	2305.90	2362.09	28.10
132	16.20	11.82	2285.67	2341.49	27.91
136	11.82	7.66	2258.37	2313.69	27.66
140	7.66	4.56	2223.20	2277.86	27.33
144	4.56	2.83	…	…	…

For Table DET-8, columns 1-3 are known inputs into the table. The remaining columns are unknown (blank) when the routing process begins. The objective is to complete the last column, which represents the outflow hydrograph. Inputs and calculations for each column include:

- **Column 1** (time) and **Column 2** (inflow) provide the design inflow hydrograph (obtained using methods described in Chapter 4 – *Determination of Stormwater Runoff*).
- **Column 3** is the value from column 2 moved earlier in time (up the table) one time increment.

- **Column 4**: To initiate the routing process with little or no inflow, assume the initial value is 0. The next value of $2S_j/\Delta t - O_j$ confirms this assumption. Subsequent values of $(2S/\Delta t) - O$ are calculated by doubling the outflow values in column 6 and subtracting them from $(2S/\Delta t) + O$.

- **Column 5**: The values in column 5 are calculated by applying the continuity equation (storage relationship) in Equation DET-20:

$$(I_i + I_j) + \left[\frac{2S_i}{\Delta t} - O_i\right] = \left[\frac{2S_j}{\Delta t} + O_j\right] \qquad \textbf{(Equation DET-20)}$$

for the first time increment (4 minutes), this is: $(0 + 0.01) + [0] = [0.01]$,

- **Column 6**: The first value of outflow is assumed to be equal to inflow. Subsequent values are obtained from the $(2S/\Delta t) + O$ versus O relationship in Figure DET-4 and Table DET-8. Linear interpolation can be used to determine O values for a given $(2S/\Delta t) + O$ using Table DET-8 for values that cannot be easily read from Figure DET-4 (for the first row of Column 6, see Step 2 above).

8.0 REFERENCES

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