CHAPTER 12

MULTIPLE-EVENT DETENTION DESIGN

12.1 INTRODUCTION

Detention design is the conventional solution to stormwater management for typical land development activities. It is probably the least desirable method for handling stormwater runoff, yet it is hard to avoid even for the most cleverly orchestrated designs. Innovative methods should be used to reduce increased runoff generated by landscape change. Design tactics such as disconnected impervious area, reduced impervious area, decentralization of runoff, dispersion of runoff, and minimization of altered landscape are all much more desirable than the uninspired tactic of clear-build-pave followed by capture-store-release.

There are many best management practices (BMPS) that should be incorporated into a design as preliminary remediation efforts. These BMPs include but are not limited to overland flow (vs. piping), vegetated swales, bioretention basins, vegetated infiltration beds, curbless streets, narrower paved

streets, shorter-set back lines (to reduce paved driveway surfaces), and reserved green space. Each BMP by itself can reduce runoff by a small amount. Several BMPs combined in an orchestrated fashion can reduce runoff by a significant amount. The goal is to eliminate as much runoff on-site as possible through creative, on-site management. Once this is done, the remaining increase runoff is then handled by a detention facility, which is most likely considerably smaller in size because of the addition of BMPs to the design. It is possible to reduce all runoff increase by innovative design, but it is very difficult and often requires the selection of a suitable site. However, conceding that stormwater management almost always means storage detention, the goal of any stormwater management plan should be to minimize the volume of storage that must be detained.

12.2 DETENTION VOLUME ESTIMATES

Before any hydrographs can be routed through a detention facility, the facility must be sized to meet the requirements for control. Preliminary estimates of storage can be performed in several ways. Some popular methods are (1) runoff difference, (2) hydrograph subtraction, and (3) NRCS TR-55, Chapter 6, 1986.

12.2.1 Runoff Difference Method

This method is based on the assumption that storage required for stormwater detention is equal to the postdevelopment runoff volume minus the predevelopment runoff volume. This difference represents the increase in total runoff due to development, so this is a logical assumption and it is simple to determine. The universal way to determine storage loss when using hydrograph analysis is to determine the predevelopment hydrograph volume (area under the curve) and the postdevelopment hydrograph volume and look at the difference. If NRCS runoff procedures are being used, then a more direct and simpler method would be to subtract the predevelopment direct runoff from the post development direct runoff. Direct runoff is computed using the curve number method of Equation 8.1 or 8.4. In this case, volume of storage required is computed as

$$V_S = A(Q_D - Q_N) \qquad (12.1)$$

where V_S is the required storage volume in ac-in, A is the site drainage area in acres, Q_D is the runoff depth for the developed condition in inches, and Q_N is the runoff depth for the natural (predevelopment) condition in inches.

Example 12.1 A 62-acre natural watershed located in Virginia will be developed into the Pine Hills residential subdivision. The predevelopment CN

for the site is 68. The postdevelopment *CN* is 76. For a design event equal to 4.2 inches of rainfall, estimate the volume of storage required for detention design.

Solution:

$$Q_N = \frac{\left(4.2 - \frac{200}{68} + 2\right)^2}{\left(4.2 + \frac{800}{68} - 8\right)} = 1.33 \text{ inches}$$

$$Q_D = \frac{\left(4.2 - \frac{200}{76} + 2\right)^2}{\left(4.2 + \frac{800}{76} - 8\right)} = 1.89 \text{ inches}$$

$$V_S = A(Q_D - Q_N) = 62(1.89 - 1.33) = 34.72 \text{ ac-in}$$

This volume is about 2.89 ac-ft, or about 126,034 ft^3. A 1-acre pond with an average pond depth of 3 feet would be a good first estimate, or a 0.5-acre pond with a average pond depth of 6 feet is also a good first guess.

12.2.2 Hydrograph Subtraction

Hydrograph subtraction is similar to the loss of storage method, except it looks at runoff hydrographs a little differently. Instead of subtracting total runoff of the two hydrographs, hydrograph subtraction takes a plot of the predevelopment and postdevelopment hydrographs, and determines the volume difference between the two from time $t = 0$ to the time when the hydrographs first cross on the postdevelopment graph falling limb. Figure 12.1 shows the region that is used to compute the storage estimate. As can be seen in this figure, the volume of storage estimate in this method can be very close to the volume estimated using loss of storage. However, there are times where the falling limb of the predevelopment hydrograph is significantly higher in flows, as compared to the falling limb of the postdevelopment hydrograph. In cases such as these, hydrograph subtraction would most likely give a better estimate of storage required than the loss of storage method.

The procedure for differencing the hydrographs is reasonably simple. Both hydrographs are assumed to have tabulated ordinate data based on a common time step. Each ordinate of the predevelopment hydrograph, beginning with the first, is subtracted from the associated predevelopment hydrograph ordinate until the difference becomes negative. All of the positive differences represent required storage volume. These ordinate differences are summed

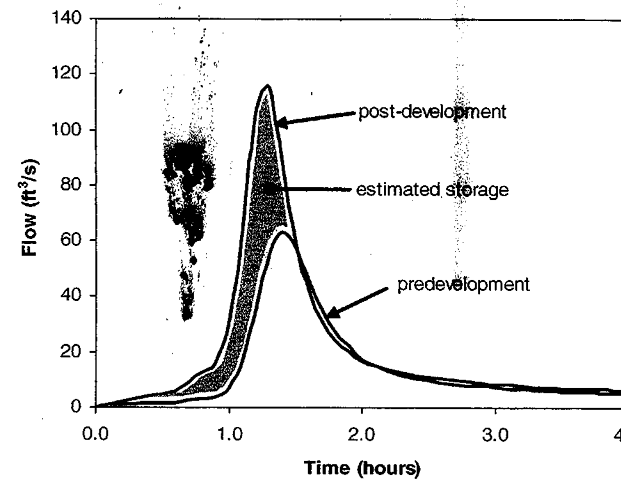

Figure 12.1 Hydrograph subtraction method for detention storage estimate.

and multiplied by the time step to give volume. Units must be consistent. Usually, flow is expressed in ft^3/s, so the time step in the calculation should be expressed in seconds.

Example 12.2 Portions of the 100-year, predevelopment and postdevelopment hydrographs for the Pine Hills subdivision are shown in Table 12.1. An estimate of detention storage for the 100-year event is required.

Solution: The computations are shown in Table 12.1. The fourth column is the difference between the third and second columns. This difference is performed until a negative number is encountered. The positive values are summed and the sum is found to be 271.1 ft^3/s. This sum is used to compute the total volume required:

$$V_S = 271.1 \text{ ft}^3/\text{s} \times 0.1 \text{ hrs} \times 3600 \text{ s/hr} = 97{,}596 \text{ ft}^3$$

12.2.3 NRCS TR-55

NRCS TR-55, 1986, has six chapters, each of which deals with a particular element of urban runoff analysis and management. Chapter 6 presents a method for estimating detention storage requirements based on NRCS 24-hour rainfall distribution types, runoff depths, and peak flows. It is developed through the study of average storage conditions for many structures with

TABLE 12.1 Partial Pre- and Postdevelopment Hydrographs for Example 12.2

Time (hour)	Predevelopment Flow (ft^3/s)	Post-development Flow (ft^3/s)	Flow Difference (ft^3/s)	Cumulative Flow (ft^3/s)
0.0	0.0	0.0	0.0	0.0
0.1	1.0	1.1	0.1	0.1
0.2	1.1	1.9	0.8	0.9
0.3	1.2	3.0	1.8	2.7
0.4	1.4	4.4	3.0	5.7
0.5	1.5	4.9	3.4	9.1
0.6	1.7	5.5	3.8	12.9
0.7	2.4	8.9	6.5	19.5
0.8	3.0	12.3	9.3	28.7
0.9	3.7	15.7	12.0	40.7
1.0	7.4	32.3	24.9	65.6
1.1	17.3	65.9	48.6	114.3
1.2	36.0	107.0	71.0	185.3
1.3	55.5	115.2	59.7	244.9
1.4	63.4	87.7	24.3	269.2
1.5	57.2	59.0	1.8	271.1
1.6	45.6	41.7	−3.8	
1.7	35.0	31.1		
1.8	27.2	24.1		
1.9	22.3	20.4		
2.0	17.5	16.6		

single-stage flow devices, some with orifice control and some with weir control. Estimates should be viewed as preliminary. To begin the procedure, the volume of post-development runoff is computed using Equation 12.2.

$$V_R = 3630\,AQ \qquad (12.2)$$

In this equation, V_R is volume of runoff in ft^3, Q is runoff depth in inches, and A is drainage area in acres. The coefficient 3630 is simply a conversion factor for units. Figure 12.2, or Equations 12.3 or 12.4, is used to determine the ratio of storage volume to runoff volume. The equations are chosen according to NRCS rainfall distribution.

$$\left(\frac{V_S}{V_R}\right)_{I,IA} = 0.660 - 1.76\left(\frac{q_o}{q_i}\right) + 1.96\left(\frac{q_o}{q_i}\right)^2 - 0.730\left(\frac{q_o}{q_i}\right)^3 \qquad (12.3)$$

$$\left(\frac{V_S}{V_R}\right)_{II,III} = 0.682 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^2 - 0.804\left(\frac{q_o}{q_i}\right)^3 \qquad (12.4)$$

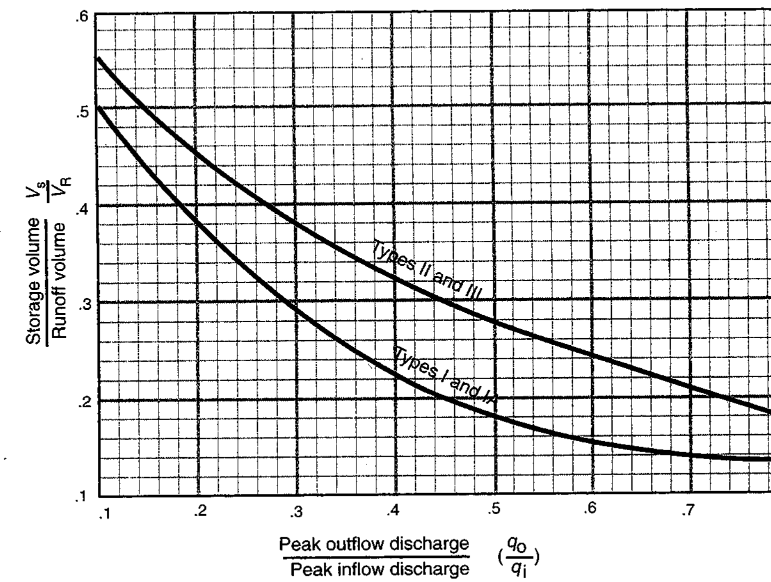

Figure 12.2 Approximate detention basin routing effects for NRCS 24-hour rainfal types I, IA, II, and III (USDA NRCS TR-55, 1986).

where V_S = volume of estimated storage, V_R = volume of runoff, q_o = targe release flow, and q_i = peak basin inflow. Storage required is determine b multiplying the volume ratio by the volume of runoff computed i Equation 12.2.

Example 12.3 For the Pine Hills residential subdivision in Examples 12.1 and 12.2, compute the storage estimate using the NRCS TR-55 method From Table 12.1, the target release rate is 61.4 ft^3/s and the pond inflow i 115.4 ft^3/s.

Solution:

$$V_R = 3630\,AQ = 3630 \times 62 \times 1.89 = 425{,}363\ ft^3$$

$$\frac{q_o}{q_i} = \frac{61.4}{152.7} = 0.550$$

Virginia is located in type II rainfall region, so Equation 12.4 is used.

$$\frac{V_S}{V_R} = 0.682 - 1.43(0.55) + 1.64(0.55)^2 - 0.804(0.55)^3 = 0.258$$

Storage volume is computed as

$$V_S = V_R\left(\frac{V_S}{V_R}\right)_{II} = 425{,}363(0.258) = 109{,}744 \text{ ft}^3$$

12.2.4 Comparison of Methods

The results of the three previous examples provide a comparison of the three storage estimate methods. The results are summarized in Table 12.2. To determine which method provided the best estimate the postdevelopment hydrograph was routed through a single-stage structure with a rectangular orifice to determine the actual storage required (see Section 12.4). In the routing, the peak inflow was reduced to 58.9 ft^3/s, and the storage required was 85,250 ft^3. The errors in estimates range from 15 to 50 percent. The runoff difference method estimated high in this case, and it should be expected to estimate high because it is a simple runoff difference that does not take into account the dynamics of a flood wave moving through a detention pond. Hydrograph subtraction did the best job in this example, and experience has proven this method to be dependable, giving estimates that are usually close to the correct volume. NRCS TR-55 has been accused of giving estimates that are consistently high by 25 to 40 percent. For very small sites of a few acres or less, this accusation may be true. However, the method is a good one, and in many cases provides an estimate that is within a few percent of the final result.

12.3 MULTIPLE-STAGE OUTLET FLOW ANALYSIS

Storage detention for land development activities typically requires the management of several runoff events based on frequency of occurrence. Land-development regulations typically call for three to five controlled events,

TABLE 12.2 Comparison of Results of Examples 12.1 through 12.3 for the Pine Hills Subdivision

Method	Storage Estimate	% Error
Runoff difference	126,034	+47.8
Hydrograph subtraction	97,596	+14.5
NRCS TR-55	109,744	+28.7
Modified-Puls routing	85,250	—

chosen from the standard list of the 1-, 2-, 5-, 10-, 25-, 50-, and 100-year return periods. In practical terms, the 1-10-100 criterion is probably more than adequate. If a detention structure can handle these three events, it is probably doing a pretty good job on the remaining four. Nevertheless, the designer must create a combination of outlet points in the detention pond outlet structure to meet the requirements of a particular regulation. Multiple outlets in a single structure is the common approach to this design problem. The process is presented in Section 12.4.

Before design can be performed, it is important to understand the process of analyzing a multiple outlet structure. In general, the hydraulics of culvert flow, orifice flow, and weir flow are adequate to analyze a structure. There are situations where energy methods are required to analyze flow, and the computations then become more complicated.

The fundamental equations used are the orifice and weir equations, Equations 3.26 and 4.19.

$$q_o = C_o A\sqrt{2gH_o} \qquad (3.26)$$

$$q_w = CLH_w^{3/2} \qquad (4.19)$$

Specific flow relations for each opening in the outlet structure are created in terms of water surface elevation, instead of water depth, above the opening. Flows are tracked in reference to water surface elevation, and all flows created by a particular water surface for individual openings are summed to get the total flow through the structure. The analysis is illustrated by example.

Example 12.4 Figure 12.3 shows a sketch of a three-stage outlet structure for a small detention pond. It consists of a 2-feet-by-4-feet concrete riser box with three inflow holes (stages 1, 2, and 3) and one outflow hole (outfall culvert). The culvert does not control flow through the structure, nor does is cause backwater to occur in the riser box. The specific geometry of each opening is summarized in Table 12.3. Create an outflow versus elevation (E-O curve) for this structure, for the elevation range of 592.00 feet to 597.00 feet, using an elevation step of 0.20 feet.

Solution: Since the outfall culvert does not control flow, the analysis begins with the first stage.

Stage 1. The rectangular orifice will initially act like a weir until the pond depth exceeds the vertical dimension. After water depth covers the entire opening, the weir flow stops and the orifice flow occurs. Therefore, this stage requires the development of two equations.

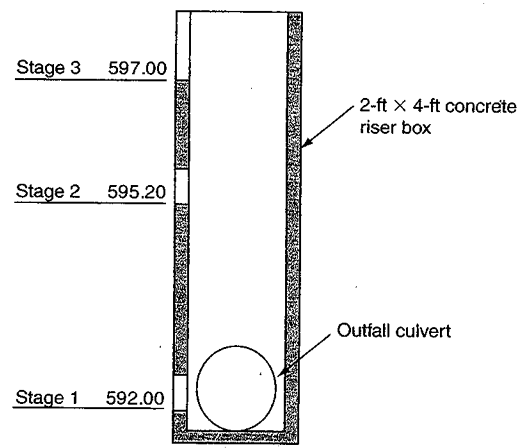

Figure 12.3 Schematic drawing of the three-stage outlet structure of Example 12.4.

1. For water surface elevations (WSEL) 592.00 ft to 592.55 ft: The orifice acts like a weir, where L = 0.55 ft, headwater depth of the weir is expressed in terms of $WSEL$ as $H_w = WSEL - 592.00$ ft.

$$q_{w,1} = CLH_w^{3/2} = 3.1(0.55)(WSEL - 592.00)^{1.5}$$
$$= 1.705(WSEL - 592.00)^{1.5}$$

2. For WSEL 592.55 to 597.00 ft: Orifice flow occurs where $A = 0.28$ ft², $C = 0.60$, and $H_o = WSEL - 592.28$ ft.

$$q_o = C_o A\sqrt{2gH_o} = 0.6(0.28)(64.4)^{0.5}(WSEL - 592.28)^{0.5}$$
$$= 1.444(WSEL - 592.28)^{0.5}$$

Flows are computed for the valid range and are shown in Table 12.4. $WSEL$ = 592.55 feet is the breakpoint between the two flow types. Breakpoints usually give different answers, and the disparity is probably caused by uncertainty in the flow coefficients. The lower value of the two is accepted as the correct flow rate, simply as an arbitrary choice. An average value may be equally valid.

TABLE 12.3 Geometric Characteristics of the Outlet Structure Analyzed in Example 12.4

Stage	Geometric Shape	Flow Coefficient	Dimensions	Invert Elevation (ft)
1	rectangular orifice	0.6	0.55 ft × 0.55 ft	592.00
2	rectangular orifice	0.6	0.53 ft × 0.53 ft	595.20
3	constricted rectangular weir	3.1	2.25 ft wide	597.00

TABLE 12.4 Summary of Flow Calculations of Stage 1 of Example 12.4

$WSEL$ (ft)	$H_{w,1}$ (ft)	$q_{w,1}$ (ft³/s)	$H_{o,1}$ (ft)	$q_{o,1}$ (ft³/s)
592.00	0	0		
592.20	0.20	0.15		
592.40	0.40	0.43		
592.55	0.55	0.70	0.28	0.75
592.60			0.33	0.82
592.80			0.53	1.05
593.00			0.73	1.23
...			...	...
597.40			5.13	3.27
597.60			5.33	3.33
597.80			5.53	3.39
598.00			5.73	3.45

Stage 2. Again, this rectangular orifice acts first like a weir, and then as an orifice.

1. For WSEL 595.20 to 595.73 feet: The orifice acts like a weir, where $C = 3.1$, $L = 0.53$ feet and $H_w = WSEL - 595.20$ ft.

$$q_{w,2} = 1.643(WSEL - 592.00)^{1.5}$$

2. For WSEL 595.73 to 597.00 feet: Orifice flow occurs where $C = 0.6$, $A = 0.53$ ft², and H_w = WSEL − 595.46, substituting into the orifice equation, gives

$$q_{o,2} = 1.348(WSEL - 595.46)^{0.5}$$

Using these two equations, the appropriate values of WSEL are used to compute flow. Results are in Table 12.5.

Stage 3. This is a simple rectangular weir with $C = 3.1$, $L = 2.25$ ft, and $H_w = WSEL - 597.00$ ft.

1. For WSEL 597.00 to 598.00 ft, the weir equation gives

$$q_{w,3} = 6.975(WSEL - 597.00)^{1.5}$$

TABLE 12.5 Summary of Flow Computations for Stage 2 of Example 12.4

WSEL (ft)	$H_{w,2}$ (ft)	$q_{w,2}$ (ft^3/s)	$H_{o,2}$ (ft)	$q_{o,2}$ (ft^3/s)
595.20	0	0		
595.40	0.20	0.15		
595.60	0.40	0.42		
595.73	0.53	0.63	0.27	0.70
595.80			0.34	0.78
596.00			0.54	0.98
596.20			0.74	1.15
. . .			. . .	. . .
597.40			1.94	1.87
597.60			2.14	1.97
597.80			2.34	2.06
598.00			2.54	2.14

Using this equation, the appropriate values of WSEL are used to compute flow. Results are in Table 12.6.

The results of the analysis of the three stages are merged into one table, and all flows associated with a particular *WSEL* are added to get the total flow through the outlet structure. The final outflow rating table is shown in Table 12.7 and the E O curve is plotted in Figure 12.4. The figure shows the distinct break points where one flow equation ends and the next begins. Weir curves are concave upward and orifice equations are concave downward.

This is the basic analysis process for establishing a rating curve for a multiple-stage outlet structure. In practice, the simplicity of the structure, in terms of hydraulic calculation, is not always possible. As backwater effects in the riser box affect the flow, more complex analysis methods are required. To perform these complex calculations on the desktop or with even with an electronic spreadsheet is difficult. Most analyses in practice are done with the aid of commercial software, specifically designed for stormwater detention design.

TABLE 12.6 Summary of flow computations for Stage 3 of Example 12.4

WSEL (ft)	$H_{w,3}$ (ft)	$q_{w,3}$ (ft^3/s)
597.00	0.00	0.00
597.20	0.20	0.62
597.40	0.40	1.76
597.60	0.60	3.24
597.80	0.80	4.99
598.00	1.00	6.98

TABLE 12.7 Outlet Rating Table Summary Calculations for Example 12.1

WSEL (ft)	Stage 1 $q_{w,1}$ (ft^3/s)	Stage 1 $q_{o,1}$ (ft^3/s)	Stage 2 $q_{w,2}$ (ft^3/s)	Stage 2 $q_{o,2}$ (ft^3/s)	Stage 3 $q_{w,3}$ (ft^3/s)	Total Discharge (ft^3/s)
592.00	0.00					0.00
592.20	0.15					0.15
592.40	0.43					0.43
592.55	0.70					0.70
592.60		0.82				0.82
592.80		1.05				1.05
593.00		1.23				1.23
593.20		1.39				1.39
593.40		1.53				1.53
593.60		1.66				1.66
593.80		1.78				1.78
594.00		1.89				1.89
594.20		2.00				2.00
594.40		2.10				2.10
594.60		2.20				2.20
594.80		2.29				2.29
595.00		2.38				2.38
595.20		2.47	0.00			2.47
595.40		2.55	0.15			2.70
595.60		2.63	0.42			3.05
595.73		2.68	0.63			3.31
595.80		2.71		0.78		3.49
596.00		2.78		0.98		3.76
596.20		2.86		1.15		4.01
596.40		2.93		1.30		4.23
596.60		3.00		1.43		4.43
596.80		3.07		1.56		4.63
597.00		3.14		1.67	0.00	4.81
597.20		3.20		1.77	0.62	5.59
597.40		3.27		1.87	1.76	6.90
597.60		3.33		1.97	3.24	8.54
597.80		3.39		2.06	4.99	10.44
598.00		3.45		2.14	6.98	12.57

12.4 STORAGE AND OUTLET DESIGN PROCEDURE

Regulating flow for several frequency events requires the design of a multiple-opening outlet structure for the detention facility. The typical structure is a riser box attached to the upstream end of a discharge culvert laid underneath the detention pond embankment. Outlet openings are placed at different elevations along the riser box to allow the correct amount of flow to be released to the outfall culvert for each design event. The procedure for designing a

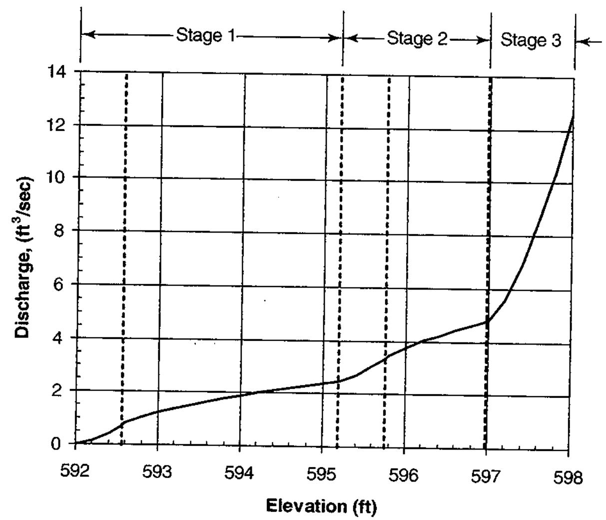

Figure 12.4 Elevation-outflow curve for the outlet structure of Example 12.4. Dashed lines show locations where a stage changes from weir to orifice flow, or where a new stage begins to add flow.

multiple-stage stormwater detention facility is a multistep process that is summarized as follows:

1. *Runoff and hydrographs*
   1.1 For the predevelopment condition of the site, perform hydrograph analyses for each controlled event. See Chapter 9.
   1.2 For the post-development condition, perform hydrograph analyses for each controlled event. See Chapter 9.
   1.3 Establish target release outflows for each event. Some local and regional regulations require target release flows to be a percentage of the pre-development peak flow.
2. *Preliminary storage estimate*
   2.1 Estimate storage required for each event. See Section 12.2.
   2.2 Make a preliminary design of detention pond shape and size according to the maximum storage required and site grading plan.
   2.3 Create an elevation-storage relationship for the detention pond.
   2.4 Determine the approximate water surface elevation in the pond for the storage requirement of each event, based upon the detention storage estimates of step 2.1.

3. *Preliminary outfall culvert*
   Size a culvert to carry the largest controlled event through the outlet structure and out of the pond, keeping the headwater elevation less than or equal to the pond bottom, if possible.
4. *Preliminary outlet structure geometry*
   4.1 Begin the design by addressing the first design event.
   4.2 For the design event, size an outlet opening in the riser box based on the expected water surface elevation in the pond (Step 2.4) and the target release outflow (Step 1.3).
   4.3 Create an elevation-outflow relationship for the outlet structure, taking into account all hydraulic elements of the design stage, riser box, outfall culvert, and any previously sized outlet openings.
   4.4 Route the post-development hydrograph through the pond.
   4.5 Verify that the target release rate has not been violated. If it has, repeat steps 4.2 through 4.5.
   4.6 Establish storage required to handle the previously routed event hydrograph, and note the maximum water surface elevation in the pond during the routing.
   4.7 Using the maximum water surface elevation. Determine the flow that the next stage must handle, considering the additional flow capacity of all previously created openings in the riser box.
   4.8 Size the opening for the next stage and place the invert at, or slightly above, the water surface elevation determined in step 4.6.
   4.9 Repeat steps 4.3 through 4.8 until all controlled events are handled correctly by the geometry of the outlet structure.
   4.10 If the storage capacity of the pond is found to be insufficient during the design of any of the outlet stages, the geometry of the pond must be adjusted and the design returns to step 2.2.
5. *Emergency Spillway*
   5.1 Determine the maximum flow that must be handled by the emergency spillway.
   5.2 Find a location for the construction of the spillway that will not put the pond embankment at risk if it begins to erode.
   5.3 Size the emergency spillway.

Detention design with multiple outlets requires this trial-and-error solution in several steps. The above process just described minimizes the number of trials required to design the structure.

12.5 DESIGN EXAMPLE

The best way to understand the step-by-step process of multiple event detention design is to go through a design example. The following example is completed with the aid of the Virginia Tech/Penn State Urban Hydrology Model (VTPSUHM) (Seybert et. al, 2005), a computer program written at Penn State University and Virginia Tech specifically for the solution of common hydraulic and hydrologic procedures used in stormwater management design. In this example, VTPSUHM is used to compute average curve numbers, time of concentrations, runoff hydrographs, and estimated detention volumes. It is then used to size the multiple outlets in the detention basin riser box, create the outlet structure rating curve, and route the postdevelopment hydrographs through the detention basin.

Example 12.5 Aron Meadows is a proposed residential subdivision located in southeastern Pennsylvania. A multiple-event detention facility must be designed to manage the increased runoff. The following data and design criteria are provided.

Watershed

- Drainage area is 13.6 acres.
- Soils are hydrologic soil group C for the entire site.
- Predevelopment conditions:
  t_c = 34 minutes, CN = 71 (meadow and some trees)
- Postdevelopment conditions:
  t_c = 19 minutes, CN = 80 (1/2 acre residential lots)

Design Criteria

- For the 1-, 10-, and 100-year events, reduce the postdevelopment peak flows to 100 percent of the predevelopment peak flows.
- Design rainfall depths for the 24-hour storm are 2.40, 4.56, and 7.44 inches for the 1-, 10-, and 100-year events, respectively.
- The NRCS tabular method must be used for the runoff modeling.
- Detention-pond side slopes must be 3H:1V maximum, with 5H:1V preferred for ease of maintenance.
- Pond depth cannot exceed 8 feet of water.
- Pond freeboard must be 1 foot minimum.
- Pond embankment must have a minimum top width of 10 feet.
- The outlet structure must be a concrete riser box with a concrete pipe used for the outflow culvert.
- The outfall culvert slope is 0.010 ft/ft.
- Tailwater on the outfall culvert will be zero because it discharges to a trapezoidal swale that eventually leads to a receiving stream.

Solution: The remainder of this chapter covers the solution to Example 12.5.

12.5.1 Runoff and Hydrographs

With the data given, the design begins with creating predevelopment and postdevelopment hydrographs. The NRCS tabular method in VTPSUHM is used to create six hydrographs. The resulting graphs are shown in Figure 12.5 and 12.6. The peak flow data and total runoff depths for the post development condition are summarized in Table 12.8.

12.5.2 Preliminary Storage Estimates

With this hydrograph data, an estimate of detention storage is made for eac event. The NRCS TR-55 method is used because it is simple, quick, an reasonably good. For the 1-year event, the ratio of flows is 0.32. Using Figur 12.2, the volume ratio is 0.37 and the estimated storage volume is

$$V_R = \left(\frac{V_S}{V_R}\right)3630\,AQ = (0.37)3630(13.6)(0.82) = 14{,}978\ \text{ft}^3$$

Similar calculations are made for the 10- and 100-year events, and the result are shown in Table 12.9.

Required storage of the pond is approximately 61,000 ft^3. The geometr of the pond depends on site topography. For the purposes of this exampl it is assumed that a relatively flat site with mild slopes and approximatel 0.5 acres of surface area has been reserved for the detention basin

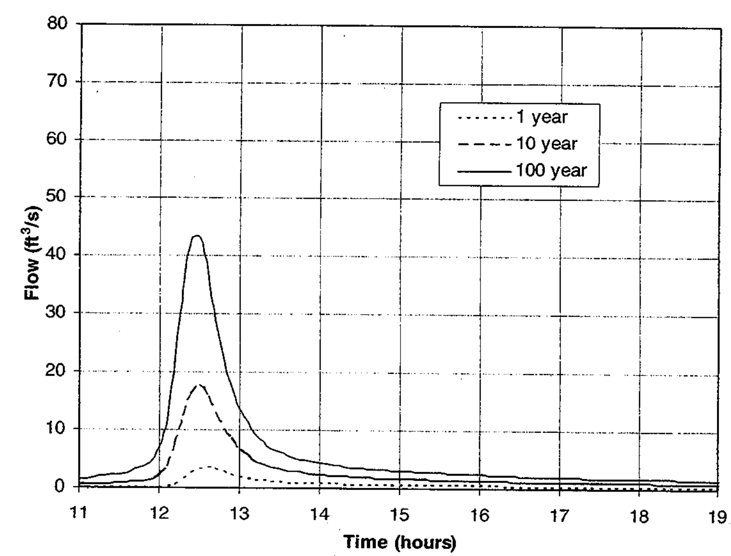

Figure 12.5 Predevelopment hydrographs for Aron Meadows subdivision.

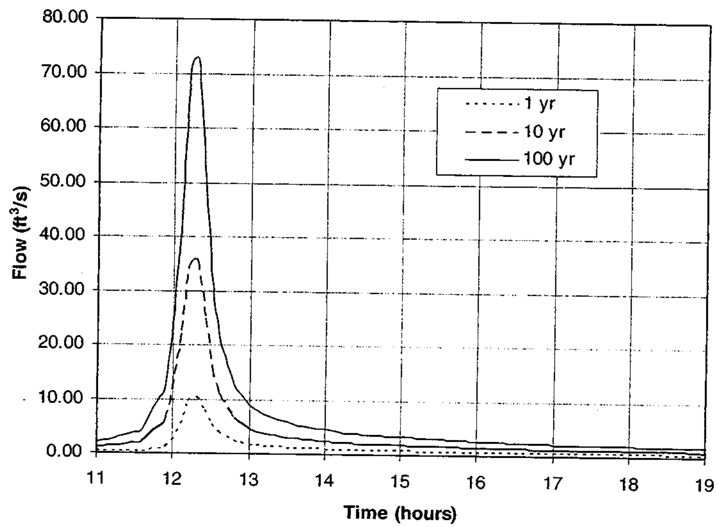

Figure 12.6 Postdevelopment hydrographs for Aron Meadows subdivision.

With 21,700 ft^2 of surface area and 5:1 side slopes, a pond 6 feet deep with 1 foot of freeboard is a reasonable guess.

By trial and error, a rectangular basin with 5:1 side slopes all around is investigated for possible dimensions. An arbitrary elevation datum of 100.00 ft is set for the pond bottom. A basin that is approximately 80 feet by 90 feet on the bottom has the elevation-storage characteristics shown in Table 12.10. Sizing calculations were performed with a spreadsheet using the average end-area method. For irregular sites, CAD drawings are typically used to create this information. In this example, for simplicity, slope requirements of the basin bottom are not considered in the volume calculations. However, bottom slopes should be taken into account if they are deemed necessary.

The elevation-storage curve is plotted in Figure 12.7 and will be used to select invert elevations in the riser box design. Notice that the elevation-storage data shows a storage of 61,500 ft^3 at elevation 105 feet. This basin

TABLE 12.8 Peak Flow Data for the Six Hydrographs of Aron Meadows

Return Period (yr)	1	10	100
q-pre (ft^3/s)	3.4	17.5	43.2
q-post (ft^3/s)	10.5	35.5	72.6
Q-post (in)	0.82	2.51	5.10

TABLE 12.9 Storage Estimates for the Three Design Events of Aron Meadows

Return period (yr)	1	10	100
Flow ratio q_o/q_i	0.32	0.49	0.60
V_S/V_R	0.37	0.28	0.24
V_R	14,980	34,700	60,430

provides excess capacity for the routing design. This is always a good approach. It is much easier to reduce the original basin size than it is to increase the size once routing has shown it to be inadequate.

12.5.3 Preliminary Outlet Structure Geometry

The outlet structure design begins with a concrete riser box. The final dimensions will be determined once the outlets and outfall culvert are sized. The design is sequential, beginning with the outfall culvert, stage 1, stage 2, and finally stage 3.

Preliminary Outfall Culvert Size. The outfall culvert must carry the 100-year target release flow of 43.2 ft^3/s. In addition, the backwater level in the riser box should be kept below the invert of the first stage. This condition will guarantee that all openings in the riser box will discharge freely to the atmosphere for all design events. This condition, although desirable, is not often possible. In order for this to happen, the crown of the culvert exiting

TABLE 12.10 Elevation-Storage Data for the Preliminary Detention Pond

Elev (ft)	Side Dim. (ft × ft)	Area (ft^2)	Volume (ft^3)	Total Vol (ft^3)
100	80 × 90	7,200		0
			8,100	
101	90 × 100	9,000		8,100
			10,000	
102	100 × 110	11,000		18,100
			12,100	
103	110 × 120	13,200		30,200
			14,400	
104	120 × 130	15,600		44,600
			16,900	
105	130 × 140	18,200		61,500
			19,600	
106	140 × 150	21,000		81,100
			22,500	
107	150 × 160	24,000		103,600

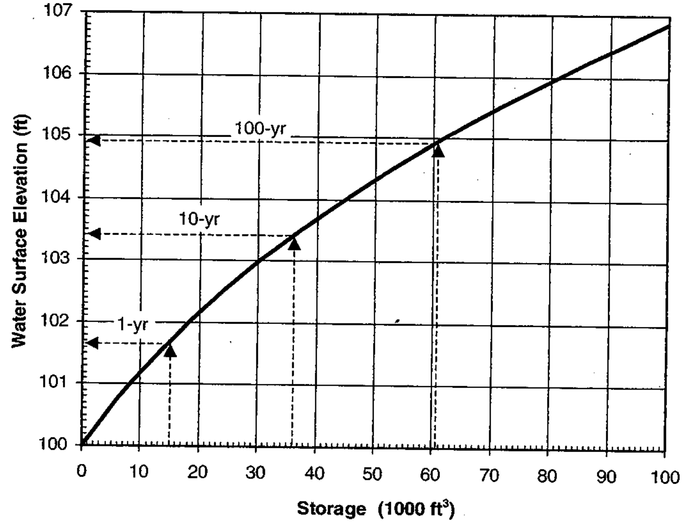

Figure 12.7 Elevation-Storage curve for the Aron Meadows detention pond.

the riser box most likely must be below the invert of the first stage. Many times there is not enough elevation to allow this, so a compromise is usually made, where backwater is allowed to cover the lower stage, but kept from affecting the flow of the upper stages. As a first guess, the invert of the outfall culvert is set to 98.00 feet.

The first stage in the outlet structure will be placed at elevation 100.00 feet, making this a dry pond design. The volume estimate for storage controlled by the first stage was determined to be 15,000 ft^3. Using this information with the E S curve of Figure 12.7 reveals that the approximate water surface elevation (WSEL) in the pond necessary to control the first event will be about 101.7 feet. The second stage invert will be near this elevation. Therefore, to keep backwater effects away from the second stage, the headwater on the outfall culvert should be kept to 3.7 feet or less. It is assumed that the culvert is acting under inlet control simply because this is a common condition for riser-box outfall culverts. With this information and assumptions, a trial HW/D ratio of 1 is used to estimate the culvert size. Figure 11.8 is the FHWA HDS-5 chart for circular concrete pipe under inlet control. If inlet control is assumed, and a square edge entrance condition is used with $HW/D = 1$ and $q = 43$ ft^3/s, Figure 11.8 indicates that the required pipe diameter is about 40 inches. The next closest commercially available diameter is 42 inches. If a 42-inch culvert is used, then Figure 11.8 shows that the HW/D will be close to 0.88. This means that the headwater needed to send 43 ft^3/s of flow through a 42-inch (3.5-foot) outfall culvert is about

0.88 × 3.5 ft = 3.08 ft, which would make the WSEL in the pond 101.08 ft. This is acceptable since it is below the expected water surface controlled by the first stage (101.7 ft), so a 42-inch culvert with square-edge entrance condition is used to start the design. Figure 12.8 shows the configuration of the riser box at this point in the design.

The length of the culvert must be about 80 to 85 feet (5 to 6 feet deep pond with 1 foot of freeboard, 5H:1V embankment slopes, and 10 feet across the top of the embankment).

Note: At this point, it is not known if the culvert acts as inlet control or outlet (friction) control. However, we do know that inlet control is typically more constricting than outlet control. If the culvert is friction control, the design is most likely still functional. If we desire, we can return to the outfall culvert design and try to reduce this diameter after the riser is designed.

Stage 1. 1-year Event. The invert of the first stage is 100.00 feet. The target release rate is 3.4 ft^3/s, and the estimated required storage is 14,980 ft^3. As mentioned earlier, the estimated storage is used with the E-S curve to estimate the maximum WSEL in the pond while controlling the first event. The WSEL is estimated to be 101.7 feet. This indicates that 1.7 feet of water will be available in the pond to drive flow through the first stage orifice.

A rectangular orifice is chosen as the outlet geometry and the flow coefficient C_o is assumed to be 0.6. The size of the opening is estimated using the orifice equation, Equation 3.26, solving for area.

$$A = \frac{q_o}{C_o\sqrt{2gH_o}} = \frac{3.4}{0.60[2(32.2)(1.7)]^{0.5}} = 0.54 \text{ ft}^2$$

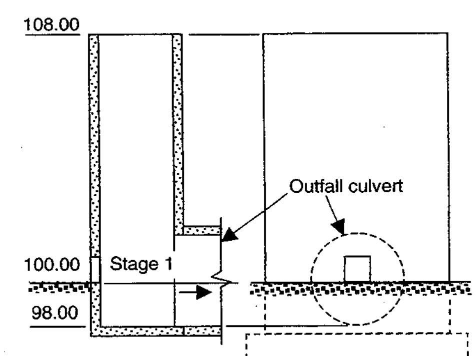

Figure 12.8 Riser box configuration for the outfall culvert and first stage in Example 12.5.

With this area, a rectangular orifice of 0.75 feet wide and 0.72 feet high provides the estimated area. However, note that H_o is defined with respect to the center of the orifice, and H_o used in this solution is to the orifice bottom. So, a better estimate of H_o might be about $1.7 - 0.72/2 = 1.34$ feet. With this value A becomes 0.61 ft². An opening of 0.80 feet wide by 0.75 feet high is a better estimate.

The opening first acts like a constricted weir ($C_w = 3.1$) during the stages 100.00 feet to 100.75 feet, and then like an orifice ($C_o = 0.6$) during stages 100.75 feet and up. H_w is measured from the weir invert and H_o is measured at the orifice centerline. These parameters are expressed in terms of WSEL as

$$H_w = WSEL - 100.00$$

$$H_o = WSEL - 100.375$$

The weir and orifice equations (Equations 4.19 and 3.26) become

$$q_w = CLH_w^{3/2} = 3.1(0.80)H_w^{3/2} = 2.48(WSEL - 100.00)^{3/2}$$

$$q_o = C_oA\sqrt{2gH_o} = 0.6(0.61)\sqrt{2gH_o} = 2.93(WSEL - 100.375)^{1/2}$$

These equations are used to create the elevation-outflow curve for the first stage. WSELs at even intervals—starting at 100.00 feet and ending at 108.00 feet—are used to compute flow through the opening. The WSEL of 108.00 feet is the maximum expected elevation in the pond exceeded by 2 feet, just to be conservative. During the calculations, it is typically assumed that backwater does not affect the flow of the first stage. However, to be accurate, flow depths in the outfall culvert should be checked at every computation interval to determine if backwater exists on the orifice opening. If it does, then the orifice equation as given in Equation 3.26 is not valid. Some form of the energy equation would be used instead. In this example, VTPSUHM is used to compute this rating curve, and the program checks for submergence of the orifice. If submergence occurs, VTPSUHM makes an appropriate calculation to take submergence into account. The rating curve result for the first stage is shown in Figure 12.9.

The figure shows the characteristic shape of the weir equation followed by the orifice equation, with the transition point at 100.75 feet. VTPSUHM also verified that the outfall culvert is flowing as inlet control during this portion of the rating curve. This elevation outflow curve, along with the elevation-storage curve Figure 12.7, is used to route the 1-year postdevelopment hydrograph through the detention facility. The process for basin routing is presented in Chapter 9.

VTPSUHM is used to do the calculations, and from the tabulated output (not shown) the routing results show that the peak flow out of the basin is 3.04 ft³/s at 12.7 hours. This outflow meets the target release criteria, but

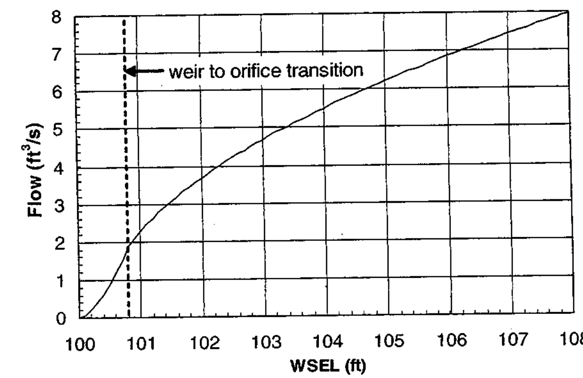

Figure 12.9 First-stage rating curve for the first estimate of required opening size i Example 12.5.

could be increased. So a second trial is performed, increasing flow area b about 15 percent. The first-stage geometry is adjusted to $A = 0.69$ ft², wit opening size of 0.833 by 0.833 feet. A new rating curve is generated, and th 1-yr postdevelopment hydrograph is routed again. This time the peak outflo is 3.36 ft³/s at 12.7 hours. This is very close to the target release, and th opening size is accepted as good. The inflow and outflow hydrographs f the routing are shown in Figure 12.10.

VTPSUHM provides information on storage and WSEL as part of th routing output, and the first portion is shown in Table 12.11. At the pea basin outflow of 3.36 ft³/s (second one in the table), the maximum WSEL

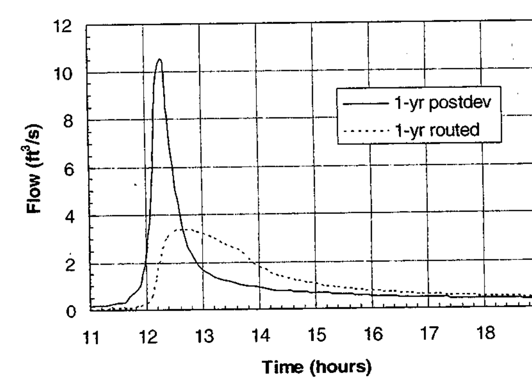

Figure 12.10 Summary hydrograph results of the routing for the first stage in E ample 12.5.

TABLE 12.11 VTPSUHM Output for the Routing of the 1-yr Hydrograph through the Detention Pond

Event Time (hrs)	Pond Inflow (ft^3/s)	Storage Used (ac-ft)	Water Surface Elevation (ft)	Basin Outflow (ft^3/s)
11.0	0.16	0.0000	100.00	0.00
11.1	0.18	0.0014	100.01	0.01
11.2	0.20	0.0029	100.02	0.01
11.3	0.22	0.0045	100.02	0.02
11.4	0.26	0.0063	100.03	0.03
11.5	0.29	0.0083	100.04	0.04
11.6	0.32	0.0104	100.06	0.05
11.7	0.55	0.0136	100.07	0.06
11.8	0.78	0.0185	100.10	0.08
11.9	1.00	0.0250	100.13	0.13
12.0	2.33	0.0372	100.20	0.23
12.1	5.49	0.0663	100.36	0.56
12.2	9.67	0.1209	100.65	1.37
12.3	10.51	0.1880	101.01	2.57
12.4	8.14	0.2419	101.24	3.04
12.5	5.51	0.2723	101.38	3.27
12.6	3.95	0.2840	101.43	3.36
12.7	3.00	0.2849	101.43	3.36
12.8	2.37	0.2795	101.41	3.32
12.9	2.02	0.2704	101.37	3.26
13.0	1.68	0.2591	101.32	3.17

Remaining VTPSUHM output is truncated from this table.

the pond is 101.43 feet and the maximum pond storage is 0.2849 ac-ft, which is 12,410 ft^3. This compares favorably to the original estimate of 14,980 ft^3.

The summary design of stage 1 is:

Square orifice: 0.833 ft by 0.833 ft, with invert at 100.00 ft

$$q_{w,1} = 2.58(\text{WSEL} - 100.00)^{3/2} \tag{12.5}$$

$$q_{o,1} = 3.34(\text{WSEL} - 100.417)^{1/2} \tag{12.6}$$

$$q_{peak} = 3.36 \text{ ft}^3/\text{s},\ S = 12{,}410 \text{ ft},\ \text{WSEL} = 101.43 \text{ ft}$$

Stage 2. 10-year Event. The WSEL of the first stage is used to set the invert of the second stage. Elevation 101.45 feet is chosen. The target release

rate is 17.5 ft^3/s, and the estimated required storage is 35,930 ft^3. Estimated storage is used with the E S curve to estimate the maximum WSEL in the pond while controlling the second event. Figure 12.7 indicates that this WSEL is about 103.4 ft. This suggests that about 1.95 ft (103.4 − 101.45) of head is available to drive flow through the second stage and 2.98 ft (103.4 − 100.417) of head is available to drive flow through the first stage. The design flow for the second stage is the target release flow for the 10-year event minus the flow passing through the first stage. The first stage flow is estimated using Equation 12.6 as

$$q_{o,1} = 3.34(2.98)^{1/2} = 5.77 \text{ ft}^3/\text{s}$$

The design flow for the second stage becomes 17.5 − 5.77 = 11.7 ft^3/s Like the first stage, rectangular orifice geometry is chosen for design. Th required area is computed as in the first stage, and consideration for the half height of the orifice is subtracted from the available head. A guess at th opening dimensions is 1 foot in height and the remainder in width. So th available head is better estimated as 1.95 − 0.5 = 1.45 foot, and the are estimate becomes

$$A_2 = \frac{q_o}{C_o\sqrt{gH_o}} = \frac{11.7}{0.60[2(32.2)(1.45)]^{0.5}} = 2.02 \text{ ft}^2$$

Therefore, a rectangular orifice that is 1 foot high and 2 feet wide, wi invert at 101.45, is the first guess at the second-stage opening size.

A new outlet structure rating curve must be generated. The final ratir curve for the first stage is updated with the addition of this new opening. Th outflow calculations are restarted at elevation 101.45 feet, and four equatio will apply: Equations 12.5 and 12.6, which apply to the first stage, and t orifice and weir equations for the second stage. These four equations becon

$$q_{w,1} = 2.58(WSEL - 100.00)^{3/2} \quad (12.$$

$$q_{o,1} = 3.34(WSEL - 100.417)^{1/2} \quad (12.$$

$$q_{w,2} = 6.20(WSEL - 101.45)^{3/2} \quad (12.$$

$$q_{o,2} = 9.63(WSEL - 101.95)^{1/2} \quad (12$$

These equations are used as appropriate to compute total flow through riser box for elevations ranging from 100.00 to 108.00 feet. VTPSUHM used to perform these calculations, and the results are shown in Figure 12.

VTPSUHM is used once again, this time to route the 10-year event throu the basin with the two-stage outlet structure. The results are graphically sho

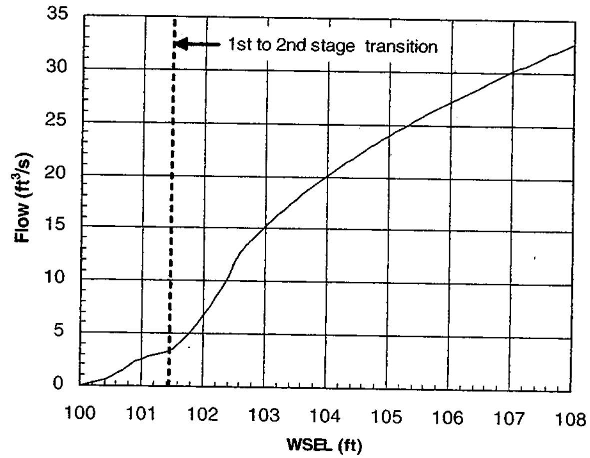

Figure 12.11 Rating curve for first trial geometry of first plus second stages, Example 12.5.

in Figure 12.12. The tabular results (not shown, but similar to Table 12.11) show that the peak pond outflow was 17.4 ft³/s at 12.5 hours. This is a very good result on the first trial, since the target release flow is 17.5 ft³/s. The sizing is accepted as good. From the tabulated computer results, the maximum WSEL in the pond during this routing is 103.40 feet with a maximum storage of 0.8261 ac-ft, which is 35,985 ft³.

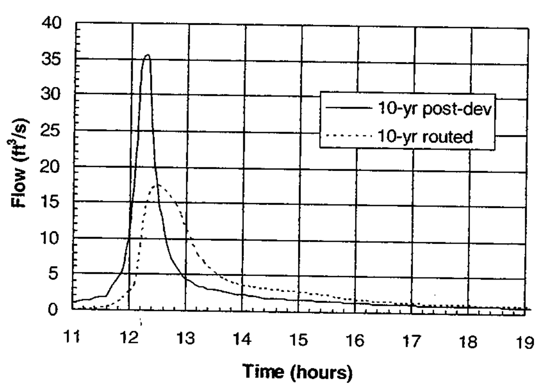

Figure 12.12 Inflow and outflow hydrographs for second-stage routing, Example 12.5.

The summary design of Stage 2 is:

Rectangular orifice: 2.00 ft wide, 1.00 ft high, invert at 101.45 ft

Rating curve equations: Equations 12.5, 12.6, 12.7, and 12.8

q_{peak} = 17.4 ft³/s, S = 35,985 ft, $WSEL$ = 103.40 ft

Stage 3. 100-year Event. The maximum WSEL of the two-stage routin is used to set the invert of the third stage. Elevation 103.45 feet is chose The target release rate is 43.2 ft³/s and the estimated required storage 60,430 ft³. Once more, estimated storage is used with the E S curve to est mate the maximum WSEL in the pond while controlling the third even Figure 12.7 indicates that this WSEL is about 104.9 feet. This suggests th about 1.45 ft (104.9 − 103.45) of head is available to drive flow through t third stage, 2.95 ft (104.9 − 101.95) of head for the second stage, and 4.4 ft (104.9 − 100.417) to drive flow through the first stage. The design flo for the third stage is the target release flow for the 100-year event minus t flow passing through the first and second stages. The first stage flow is est mated using Equation 12.6.

$$q_{o,1} = 3.34(4.48)^{1/2} = 7.07 \text{ ft}^3/\text{s}$$

The second stage flow is estimate using Equation 12.8.

$$q_{o,2} = 9.63(2.95)^{1/2} = 16.54 \text{ ft}^3/\text{s}$$

The third-stage design flow becomes 43.2 − 16.54 − 7.07 = 19.59 ft³/ The last stage will be a weir since it is at the top of the riser. The we length (width) is estimated by solving the weir equation for L.

$$L = \frac{q}{C_w^{3/2}} = \frac{19.59}{3.1(1.45)^{1.5}} = 3.62 \text{ ft}$$

This weir could be one weir of 3.6 feet width or, two weirs of 1.8 fe width each, installed on opposing sides of the concrete riser box. The seco design keeps the possibility of a 2-by-4-foot riser box, which is a standa concrete box dimension. In either case, the weir flow length will be 3.60 for the first sizing trial. This opening is modeled by one equation.

$$q_{w,3} = CLH_w^{3/2} = 3.1(3.6)H_w^{3/2} = 11.16(WSEL - 103.45)^{3/2} \quad (12$$

Once again, a new outlet structure rating curve must be generated. T final rating curve of the second stage design is updated with the addition the third opening. The outflow calculations are restarted at elevation 103.45 feet, and five equations will apply: Equations 12.5 and 12.6, which apply to the first stage, Equations 12.7 and 12.9, which apply to the second stage, and Equation 12.9 which applies to the third stage, summarized here for convenience.

$$q_{w,1} = 2.58(WSEL - 100.00)^{3/2} \tag{12.5}$$

$$q_{o,1} = 3.34(WSEL - 100.417)^{1/2} \tag{12.6}$$

$$q_{w,2} = 6.20(WSEL - 101.45)^{3/2} \tag{12.7}$$

$$q_{o,2} = 9.63(WSEL - 101.95)^{1/2} \tag{12.8}$$

$$q_{w,3} = 11.16(WSEL - 103.45)^{3/2} \tag{12.9}$$

These equations are used as appropriate to compute total flow through the riser box for elevations ranging from 100.00 to 108.00 feet. Once again, VTPSUHM is used to perform these calculations. The resulting elevation-outflow file is used to route the 100-year hydrograph through the pond. The routing shows that the peak flow out of the basin is 48.1 ft^3/s at 12.4 hours. This flow exceeds the allowable release flow of 43.2 ft^3/s, so the weir is resized. A few more trials on the size of the weir length results in a final weir length of 2.4 feet that produces a routed peak outflow of 42.8 ft^3/s, maximum storage of 1.509 acre-ft (65,732 ft^3), and maximum WSEL of 105.22 ft. This design is accepted as good. The routed 100-year hydrograph is shown in Figure 12.13. The final equation used to model flow through the third stage is changed to Equation 12.10.

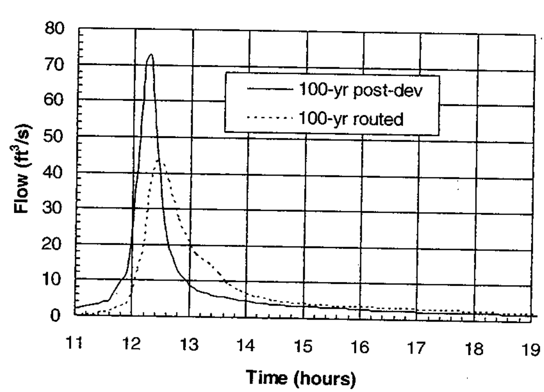

Figure 12.13 Inflow and outflow hydrographs for third-stage routing, Example 12.5.

$$q_{w,3} = 7.44(\mathrm{WSEL} - 103.45)^{3/2} \tag{12.10}$$

The final outlet structure geometry establishes the final design rating curve for the outlet structure and it is plotted for the working elevation range of 100 ft to 107 ft. It is shown in Figure 12.14.

An abridged version of the tabulated output from VTPSUHM for the rating curve of Figure 12.14 is shown in Table 12.12. The original file used in the model had 0.1 hour time steps. Table 12.12 has 0.5 hour time steps simply to reduce the length of the table. The table shows two things of interest. First, the water surface elevation in the riser box, which is backwater caused by the 42-inch outfall culvert, does not exceed 101.45 feet through the riser-box design operating range (100.00 to 105.22 ft), and therefore does not submerge the second or third stage, as was desired early in the design. Second, the assumption of inlet control of the outfall culvert is verified by the program. Third, up to a WSEL of 105.5 ft, the normal depth in the outfall culvert never exceeds 2.82 ft. Thus, a smaller diameter culvert may work and should be evaluated. In this example however, the 42 inch culvert is used in the final design. The final outlet structure design is summarized in Table 12.13 and Figure 12.15.

Emergency Spillway. The detention pond requires an emergency spillway. Typically, it is good practice to provide enough flow area to pass the 100-year postdevelopment hydrograph through the pond, assuming the entire riser-box structure is plugged and nonoperational for the entire routing period. A broad-crested weir is typically used in the design (C = 3.0), with flow depths kept to a minimum. The outlet structure rating curve becomes a one stage structure, with the invert set slightly above the maximum water surface elevation determined in the routing of the 100-year event with the three

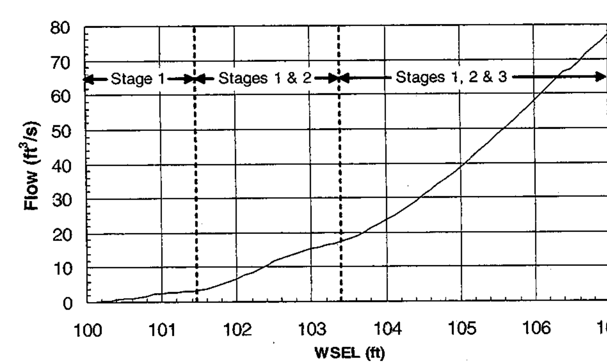

Figure 12.14 Final rating curve for the outlet structure in Example 12.5 showin regions of stage performance.

TABLE 12.12 Outlet Structure Rating Curve Calculations for Example 12.5 from VTPSUHM

Basin WSEL (ft)	Basin Outflow (ft^3/s)	Riser Box WSEL (ft)	Tailwater Elevation (ft)	Outfall Culvert Control Type
100.00	0.00	98.00	N/A	INLET
100.50	0.92	98.34	N/A	INLET
101.00	2.55	98.59	N/A	INLET
101.50	3.55	98.70	N/A	INLET
102.00	6.74	98.98	N/A	INLET
102.50	11.96	99.32	N/A	INLET
103.00	15.24	99.50	N/A	INLET
103.50	18.10	99.64	N/A	INLET
104.00	23.61	99.88	N/A	INLET
104.50	30.62	100.17	N/A	INLET
105.00	38.87	100.49	N/A	INLET
105.50	48.13	100.82	N/A	INLET
106.00	58.07	101.18	N/A	INLET
106.50	67.12	102.30	N/A	INLET
107.00	77.28	102.91	N/A	INLET

opening riser box. The invert elevation is chosen to be set at 105.25 feet. The length of weir is estimated by setting the available head (flow depth) to 0.5 feet. This is an arbitrary flow depth through the weir, chosen simply on erosion concerns. In general, lower depths mean less chance for scour to occur in the emergency spillway channel. The estimated spillway length is computed using Equation 4.19 solved for L.

$$L = \frac{q}{CH_w^{3/2}} = \frac{72.6}{3.0(0.5)^{1.5}} = 68.4 \text{ ft}$$

A spillway length of 70 feet is used as a first trial. VTPSUHM is used to create a single stage rating curve for this spillway. The 100-year postdevelopment hydrograph is then routed through the detention pond and outlet struc-

TABLE 12.13 Summary of Outlet Structure Geometry for Example 12.5

Stage	Geometric Shape	Dimensions	C or n	Invert Elevation (ft)
1	rectangular orifice	0.833 ft × 0.833 ft	0.60	100.00
2	rectangular orifice	2.00 ft W × 1.00 ft H	0.60	101.45
3	rectangular weir	2.40 ft W	3.1	103.45
4	outfall culvert	42 inch diameter	0.013	98.00

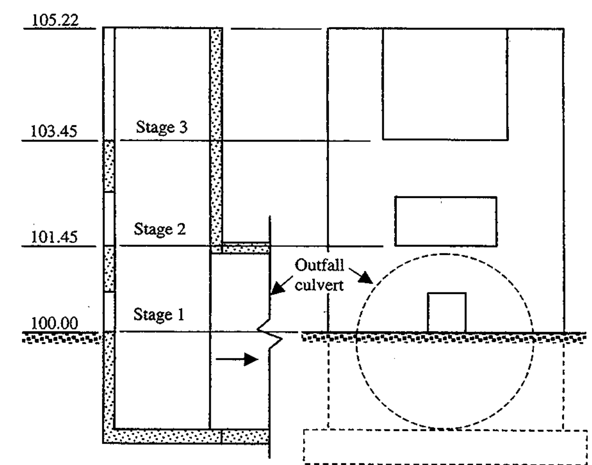

Figure 12.15 Outlet geometry for the final design of the riser box of Example 12

ture to determine the actual water surface elevation obtained while passi the 100-year event. The results of the routing from VTPSUHM shows tl the maximum water surface elevation in the routing is 105.71 feet, whi creates a flow depth through the spillway as approximately 0.49 feet. Tl size is adequate if the dimensions can be made to fit the site. If either t flow depth or width is unacceptable, the dimensions must be changed. F example, if a spillway width closer to 25 feet is more desirable, it can examined very quickly in the computer model. VTPSUHM shows that a foot wide spillway results in a maximum water surface elevation of 106. feet, creating a spillway flow depth of about 0.85 feet.

The depth of flow in the emergency spillway channel will be somethi less than the estimated flow depth (water surface elevation minus spillw invert elevation), since the static energy in the pond will be converted to fl depth plus velocity head in the spillway channel. The flow velocity shou be checked to make sure it is not erosive. If it is erosive, or near erosive, erosion resistant liner must be placed in the spillway to protect the po embankment from failure.

There are other aspects of pond design that should be considered, such embankment stability, outfall culvert piping, riser box trash racks, and an vortex devices. For very small ponds, some of these elements may not critical. However, all of these elements should be considered, taking in account the potential for property loss or damage to downstream property the pond or outlet structure was to fail.

This book is printed on acid-free paper. ♾

Published by John Wiley & Sons, Inc., Hoboken, New Jersey
Published simultaneously in Canada.

Library of Congress Cataloging-in-Publication Data:
Seybert, Thomas A.
Stormwater management for land development: methods and calculations for quantity control / Thomas A. Seybert.
p. cm.
ISBN-13: 978-0-471-72177-2 (cloth)
ISBN-10: 0-471-72177-8 (cloth)
1. Urban runoff—Management. 2. Land use—Environmental aspects. 3. Water quality management. I. Title.
TD657.S49 2006
628.2—dc22
2006000885

Printed in the United States of America

10 9 8 7 6 5 4 3 2 1

To my loving wife and best friend, Chris