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Detention Pond Design and Analysis

CE 378 Water Resources Engineering

Contents

Introduction

Wet detention ponds are probably the most common management practice for the control of stormwater runoff quality. If properly designed, constructed, and maintained, they can be very effective in controlling a wide range of pollutants and peak runoff flow rates. There is probably more information concerning the design and performance of detention ponds in the literature than for any other stormwater control device. Wet detention ponds are a very robust method for reducing stormwater pollutants. They typically show significant pollutant reductions as long as a few design-related attributes are met. Many details are available to enhance performance, and safety, that should be followed. Many processes are responsible for the pollutant removals observed in wet detention ponds. Physical sedimentation is the most significant removal mechanism. However, biological and chemical processes can also contribute important pollutant reductions. The extensive use of aquatic plants, in a controlled manner, can provide additional pollutant removals. Wet detention ponds are also suitable for enhancement with chemical and advanced physical processes.

This sectionn discusses one of the most often used and most effective stormwater control practice: wet detention ponds. There are many stormwater control practices, but all are not suitable in every situation. It is important to

understand which controls are suitable for the site conditions and can also achieve the required goals. This will assist in the realistic evaluation for each practice of: the technical feasibility, implementation costs, and long-term maintenance requirements and costs. It is also important to appreciate that the reliability and performance of many of these controls have not been well established, with some still in the development stage. This is not to say that emerging controls cannot be effective, however, they do not have a large amount of historical data on which to base designs or to be confident that performance criteria will be met under the local conditions. The most promising and best understood stormwater control practices are wet detention ponds. Less reliable in terms of predicting performance, but showing promise, are stormwater filters, wetlands, and percolation basins. Grass swales also have shown great promise during the EPA's Nationwide Urban Runoff Program (NURP) and other research projects. During the last 10 to 30 years, much experience has been gained with many stormwater practices, especially source controls and stream restoration efforts. An effective stormwater management program likely must contain elements of many control practices to be most cost-effective. The combinations of practices that are most efficient for a specific area must be selected based on many site-specific conditions and local objectives. In many cases, wet detention ponds can be an important stormwater control that should be given serious consideration.

Wet detention ponds are also one of the most robust stormwater control practices available. Although a good maintenance program is necessary to ensure the best performance and minimize associated problems, many stormwater ponds have functioned well with minimal maintenance. In addition, as long as certain design guidelines are followed, many design details that are worthwhile to consider do not create critical problems if incorrectly implemented. Finally, it is possible to retrofit stormwater ponds and correct many of these problems as experience dictates. These robust attributes are rare for most stormwater control practices. As an example, a study of 11 types of stormwater quality and quantity control practices used in Prince George's County, Maryland (Metropolitan Washington Council of Governments 1992) was conducted to examine their performance and longevity. They concluded that several types of stormwater control practices had either failed or were not performing as well as intended. Generally, wet ponds, artificial marshes, sand filters, and infiltration trenches achieved moderate to high levels of removal for both particulate and soluble pollutants. Only wet ponds and artificial marshes were found to function for a relatively long time without frequent maintenance. Control practices, which were found to perform poorly, included infiltration basins, porous pavements, grass filters, swales, smaller "pocket" wetlands, extended detention dry ponds, and oil/grit separators. Infiltration stormwater controls had high failure rates that could often be attributed to poor initial site selection and/or lack of proper maintenance. The poor performance of some of the controls was likely a function of poor design, improper installation, inadequate maintenance, and/or unsuitable placement of the control. Greater attention to these details would probably reduce the failure rate of these practices. The wet ponds and artificial marshes were much more robust and functioned adequately under a wider range of marginal conditions.

The majority of stormwater treatment practices are most effective for the removal of particulate forms of pollutants only, especially the settleable solids fraction. Removal of dissolved, or colloidal, pollutants is minimal and therefore pollution prevention or control at the sources offers a more effective way to control the dissolved pollutants. Fortunately, most toxic stormwater pollutants (heavy metals and organic compounds) are mostly association with stormwater particulates (Pitt, *et al*. 1995). Therefore, the removal of the solids will also remove much of the pollutants of interest. Notable exceptions of potential concern include: nitrates, chlorides, zinc, pathogens, 1,3-dichlorobenzene, fluoranthene, and pyrene. Stormwater ponds mostly utilize sedimentation as the main pollutant removal mechanism. However, chemical and biological mechanisms are also available, especially when the pond is appropriately planted with wetland vegetation. Stormwater ponds, while costly, also generally add substantial value to adjacent property, if designed and maintained well. The following are general conclusions pertaining to stormwater detention facilities.

Expected Detention Pond Performance

- Dry ponds have little documented direct water quality benefits due to scouring of bottom sediments. Decreased receiving water velocities will decrease receiving water bank erosion and will improve aquatic habitat, however.
- Wet ponds have been extensively monitored under a wide variety of conditions. If well designed and properly maintained, suspended solids removals of 70 to 90% can be obtained. BOD_5 and COD removals of about 70%, nutrient removals of about 60 to 70%, and heavy metal removals of about 60 to 95% can also be obtained.

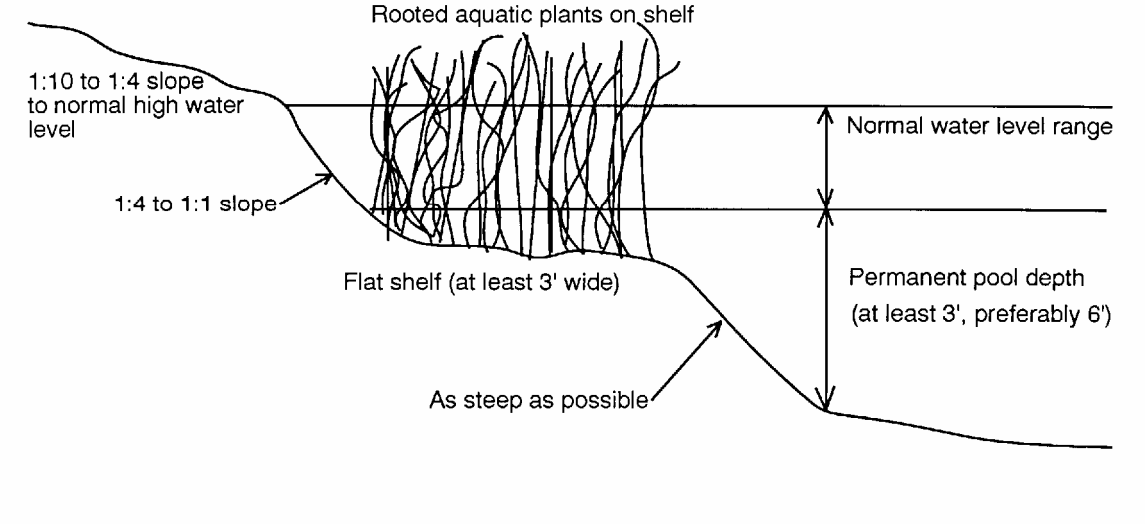
Limited bacteria control (maybe up to 50%) can be expected in the absence of disinfection. Wet ponds can also be designed to obtain significant flood control benefits.

Potential Detention Pond Problems

- Wet ponds can require about three to six years to obtain an ecological balance. During the initial unstable period, excessive algal growths, fish kills, and nuisance odors may occur.
- Wet ponds can have poor water quality and water contact recreation and consumptive fishing should be discouraged.
- Careful watershed-wide planning is needed to insure composite flood control benefits from many ponds in a watershed.

Wet Detention Pond Design Guidelines to Minimize Potential Problems

- Keep pond shape simple to encourage good water circulation. The length should be about three to five times the width for maximum detention efficiency and the inlets and outlets need to be widely spaced to minimize short-circuiting.
- Need at least three and preferably six feet of permanent standing water over most of the pond to protect sediments from scouring, to decrease light penetration (to minimize rooted aquatic plant growths), and to increase winter survival of fish.
- Increase flushing during extended dry periods, possibly with groundwater, to improve water quality. Reduce contaminated baseflows from entering the pond through source controls.
- Proper pond side slopes are very important to improve safety and aesthetics and to minimize mosquito problems and excessive rooted plant growths. An underwater shelf near the pond edge needs to be planted with rooted aquatic plants to prevent children's access to deep water, to improve pond aesthetics, to increase pollutant removals through biochemical processes, and to improve aquatic habitat. If waterfowl are desired users of the pond, then no more than one-half of the pond perimeter should be heavily planted. The following general dimensions for pond side slopes are suggested:



- Outlet structures should be designed for low outflows during low pond depths to maximize particulate retention. Place underwater dams or deeper sediment trapping forebays near pond inlets to decrease required dredging areas. Provide a drain to completely de-water the pond for easier maintenance.

- Protect the inlet and outlet areas from scour erosion and cover the inlets and outlets with appropriate safety gratings. Provide an adequate emergency spillway. Minimize water elevation changes to discourage mosquito problems.

Required Stormwater Detention Pond Maintenance

- If the pond does not require any maintenance, it is not producing very many water quality benefits. Ponds need to be periodically dredged to remove contaminated bottom sediments.
- Plan extra pond depth for sacrificial volume to lengthen dredging intervals (approximately one inch per year, much more in forebays). Also plan for heavy equipment access to pond edges.
- Remove excessive algae and other aquatic plants to prevent decomposition and nutrient cycling and associated nuisance conditions.

Basic Wet Detention Pond Design Guidelines

- Engineering design guidelines (covering such things as foundations, fill materials, embankments, gratings, anti-seep collars, and emergency spillway construction), such as published by the U.S. Natural Resources Conservation Service, the Bureau of Reclamation, and the Army Corps of Engineers must be followed.
- Pond size is dictated mostly by desired particle size control and water outflow rate. The following table is an estimate of pond surface requirements for different land uses and conditions. A target for the worst-case control of 5 μm will remove all particles greater than 5 μm under almost all conditions and will result in a long-term median removal of about 2 μm. This control goal corresponds to about 90% suspended solids reductions in urban runoff. A worst-case goal of 20 μm control will result in about 65% suspended solids reductions.

Percent of drainage area required as pond for:

Land Use	5 μm control	20 μm control
Totally paved areas	3.0 percent	1.1 percent
Freeways	2.8	1.0
Industrial areas	2.0	0.8
Commercial areas	1.7	0.6
Institutional areas	1.7	0.6
Residential areas	0.8	0.3
Open space areas	0.6	0.2
Construction sites	1.5	0.5

Wet Detention Pond Costs

- Initial wet detention pond construction costs are roughly estimated to be about $40,000 per acre of pond surface (excluding land costs).
- Maintenance costs are estimated to be about $1500 per pond surface acre per year.

Pond Size Calculation

- The following table shows the minimum pond surface area (acres) required for different freeboard elevations above the invert of 60 degree and 90 degree V-notch weirs, for both 5 and 20 μm particle control:

	60° V-notch weir			90° V-notch weir		
Head	Discharge	Min. surface acres for:		Discharge	Min. surface acres for:	
(feet)	(cfs)	5 µm	20 µm	(cfs)	5 µm	20 µm
0.5	0.25	0.044	0.004	0.45	0.08	0.006
1	1.4	0.25	0.02	2.4	0.42	0.03
1.5	3.9	0.69	0.06	6.7	1.2	0.1
2	8.0	1.4	0.11	14	2.5	0.2
3	22	3.9	0.32	40	7.1	0.6
4	45	7.9	0.65	81	14	1.2

A review of wet detention pond design procedures must include four very important publications that all stormwater managers should have. Tom Schueler's *Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban Best Management Practices* (1987) includes many alternative wet pond designs for various locations and conditions. *Watershed Protection Techniques* is a periodical published by Schueler at the Center for Watershed Protection (Ellicott City, Maryland) and includes many summaries of current stormwater management research, including new developing design procedures and performance data for detention ponds. In addition, Peter Stahre's and Ben Urbonas's book on *Stormwater Detention for Drainage, Water Quality and CSO Management* (1990) includes in-depth discussions on many detention pond design and operational issues. Also, Gary Minton recently published a comprehensive manual on stormwater treatment, *Stormwater Treatment; Biological, Chemical & Engineering Principles* (2002) that stormwater managers should also have access to. In addition, the on-going ACSE BMP database contains a growing number of case studies documenting stormwater control performance from many US locations. This database is located at:
http://www.asce.org/community/waterresources/nsbmpdb.cfm

Reservoir Routing

The discharged water from a detention pond is simply displaced pond water. In some cases, observed outlet water characteristics during a specific storm cannot be related to the inlet water characteristics. If the storm is small, the volume of water coming into the pond can be substantially less than the resident water in the pond. In these cases, the outlet water is mostly "left-over" water from a previous event or from relatively low volume (but long duration) baseflows that had previously entered the pond since the last storm. However, if the storm is large, then the water being discharged from the pond is mostly related to the specific event. Therefore, analyses of detention pond behavior must consider the relative displacement of pond water. Long-term continuous analyses comparing many adjacent storms resulting in seasonal inlet and outlet discharges of pollutants may be more appropriate than monitoring simple paired samples.

The following discussion on routing includes a procedure to examine these pond water displacement considerations and their effects on particulate trapping. The Source Loading and Management Model (WinSLAMM) and the Detention Pond Analysis model (WinDETPOND) include a computerized version of the storage-indication method.

Introduction to the Storage-Indication Method

The pond routing calculation procedure presented in the remainder of this section is based on the Natural Resources Conservation Service Technical Release-20 (TR-20) procedures (SCS 1982), as presented by McCuen (1982). The reservoir routing subroutine in TR-20 (RESVOR) is based on the storage equation:

$$I - O = \frac{\Delta S}{\Delta T}$$

where I is the pond inflow and O is the pond outflow. The difference between the inflow and outflow must be equal to $\Delta S/\Delta T$, the change in pond storage per unit of time. McCuen presents a series of equations and their solutions that require the preparation of a "storage-indication" curve to produce the pond outflow hydrograph. The storage-indication curve is a plot of pond outflow (O) against the corresponding pond storage at that outflow (S) plus 1/2 of the outflow times the time increment. When the pond outflow hydrograph is developed, the upflow velocity procedure described earlier can be used to estimate pond pollutant removal and peak flow rate reduction performance.

Outflow Rates from Discharge Control Devices

The first step in using the storage-indication method is to determine the stage-discharge relationship for the pond under study. This relationship (the rating curve) is the pond outflow rate (expressed in cubic feet per second, or cfs) for different pond water surface elevations (expressed in feet). Figures 1 through 3 are approximate rating curves for several common outlet control weir types for water surface elevation ranges up to six feet above the weir inverts. As an example, Figure 1 shows six separate curves for different lengths of rectangular weirs (from two to 18 feet wide). At a water surface elevation of 2.5 feet above the bottom of the weir (stage), not the bottom of the pond, a three foot wide rectangular weir would discharge about 34 cfs, while a 12 foot wide rectangular weir at this same stage would discharge about 150 cfs. For most applications, other stage-discharge rating curves will need to be developed and used, especially for commonly used broad crested weirs or culverts.

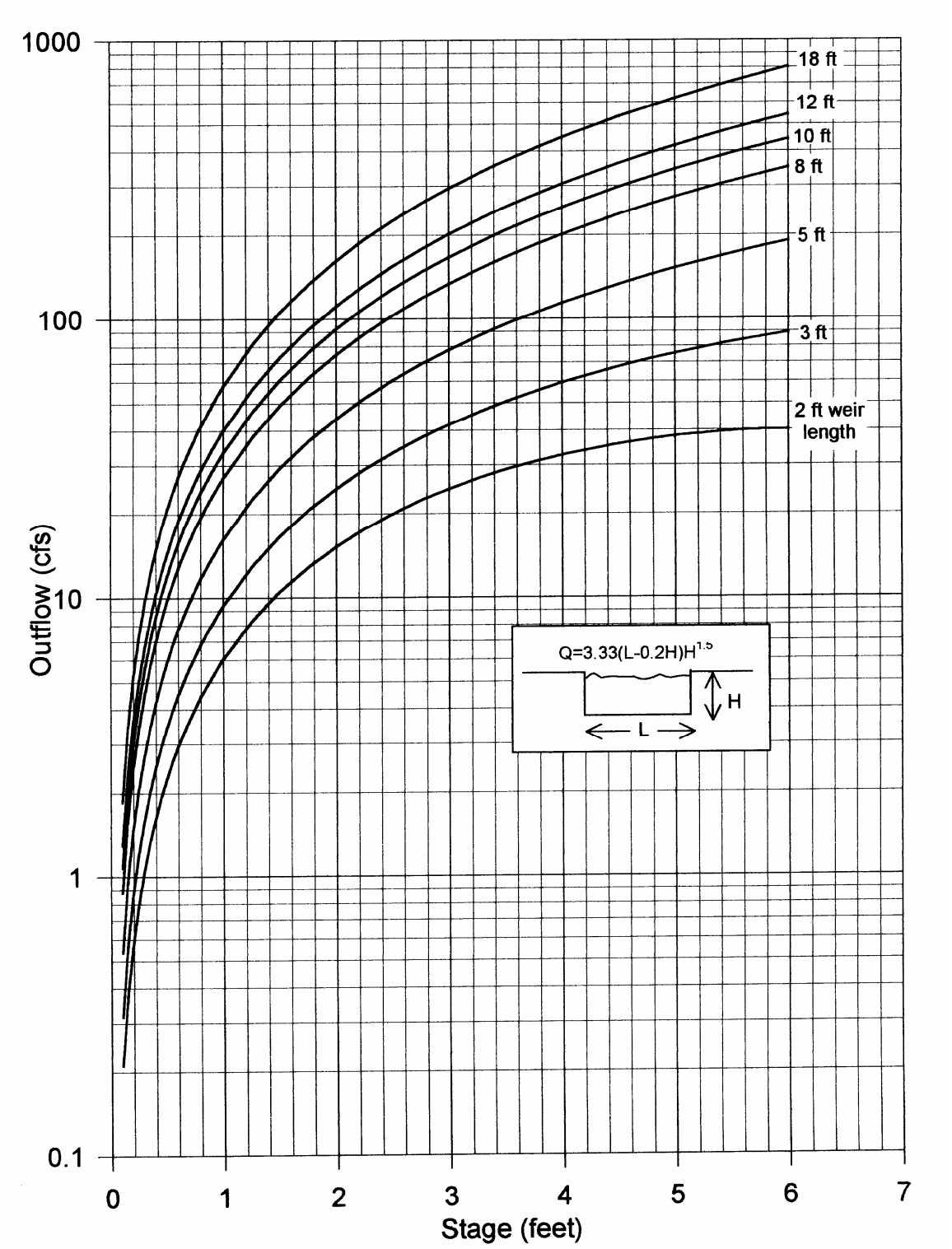


Figure 1. Approximate rating curves for rectangular weirs.

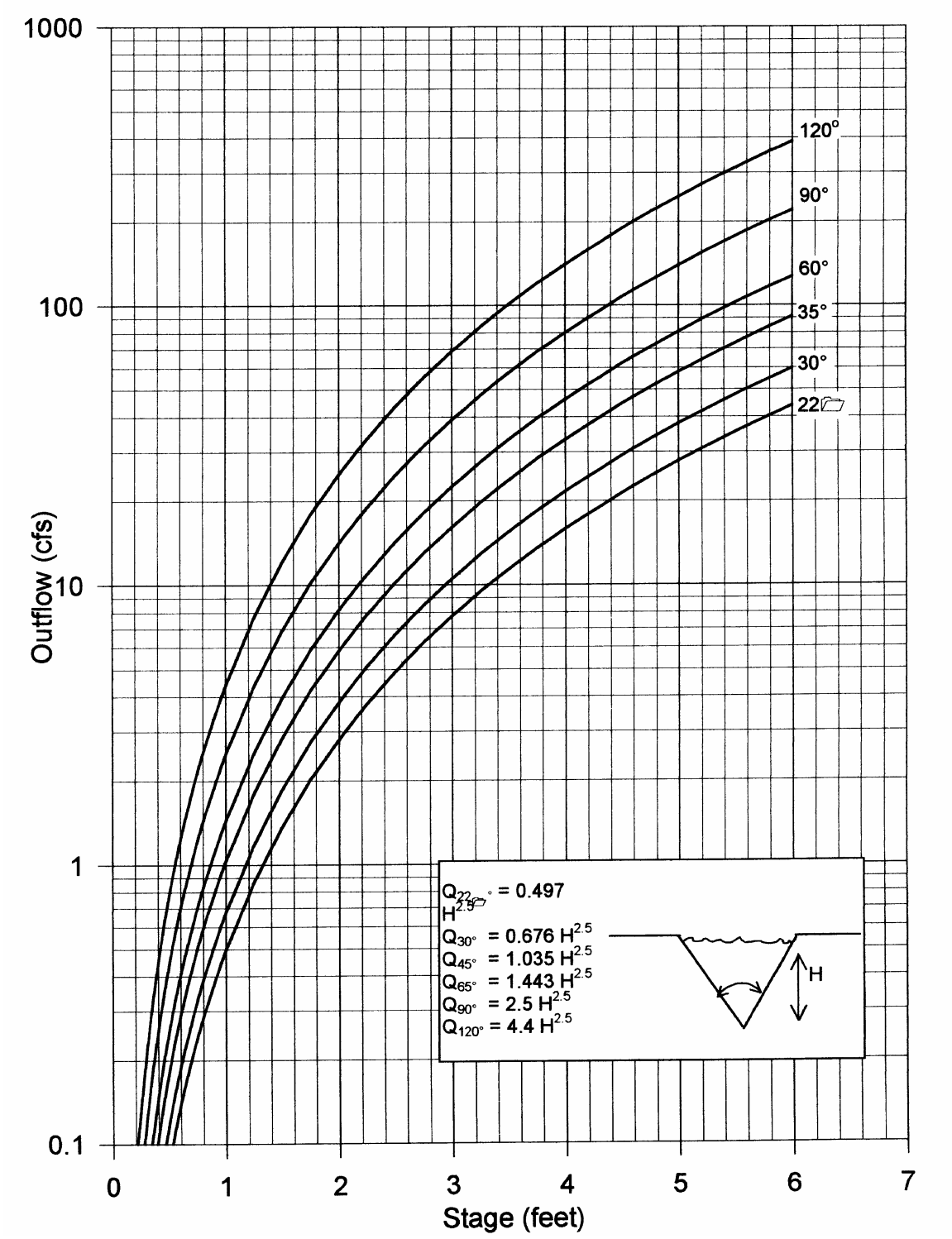


Figure 2. Approximate rating curves for V-notch weirs.

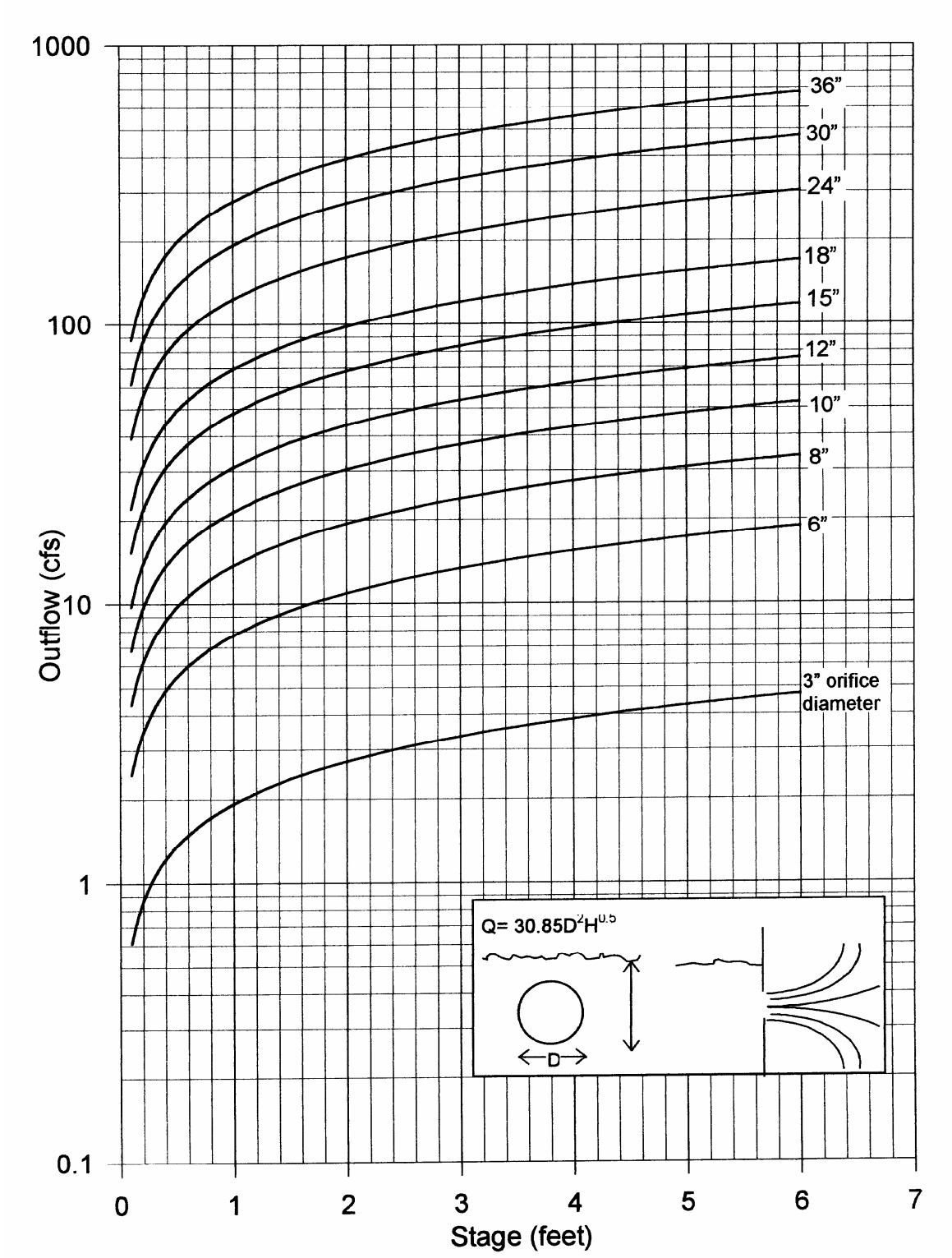


Figure 3. Approximate rating curves for orifice discharges (only applies after the orifice if covered completely).

Stage-Area and Storage-Indication Curve Development

The relationship between the pond stage and the surface area for the pond under study is also needed in order to calculate the storage volume available for specific pond stages. Figure 4 is an example stage-area curve developed from topographic maps of the Monroe Street detention pond in Madison, Wisconsin. The normal pond wet surface is at 13 feet (arbitrary datum) and the emergency spillway is located at 16 feet, for a resultant useable stage range of three feet.

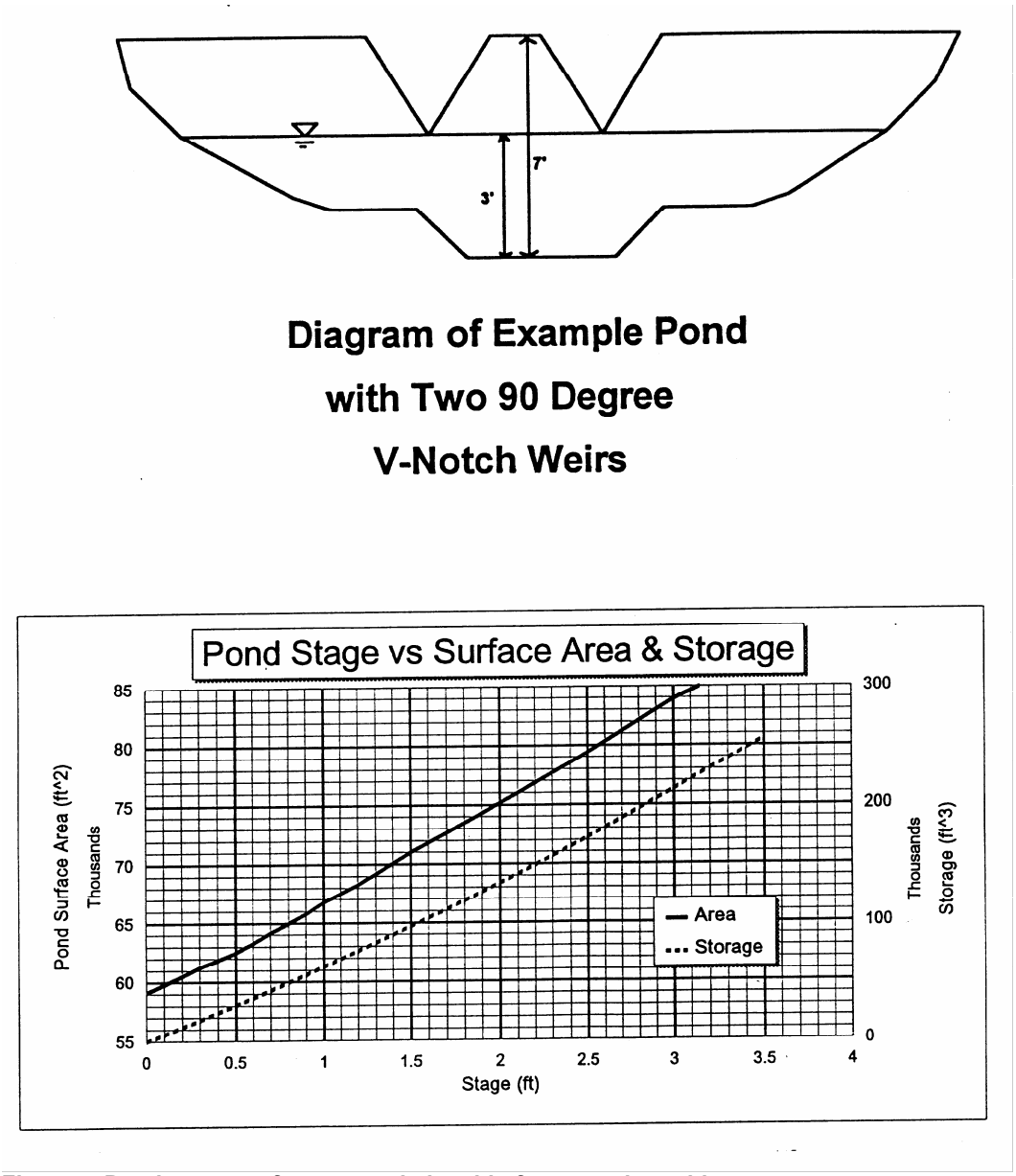


Figure 4. Pond-stage surface area relationship for example problem.

Table 1 shows the calculations used to produce the storage-indication figure (Figure 5) for the Monroe St. pond. This example assumes some pond modifications: two 90^{o} V-notch weirs, with a maximum stage range increased to 3.5 feet available before the emergency spillway is activated. The storage calculations assume an initial storage

value of zero at the bottom of the V-notch weirs (13.0 feet). The time increment used in these calculations is ten minutes, or 600 seconds. The storage-indication curve shown as Figure 5 is therefore a plot of pond outflow (cfs) verses pond storage plus 300 (1/2 of 600 seconds) times the outflow rate. The storage-indication figure must also include the stage verses outflow and storage verses outflow curves (also from Table 1).

Table 1. Calculation of Storage-Indication Relationships for Example Pond and 1.5-Inch, 3-Hour Rain.

Datum Stage (H) (ft)	Discharge Rate[1] (O) (ft^3/sec)	Surface Area (ft^2)	Storage (S) (ft^2)	S + ½ OΔt (see footnote 2)
0	0	59,100	0	0
0.1	0.016	59,800	5,980	5,985
0.2	0.09	60,500	12,100	12,130
0.3	0.25	61,250	18,375	18,450
0.4	0.51	61,850	24,740	24,890
0.5	0.88	62,520	31,260	31,520
0.6	1.4	63,300	37,980	38,400
0.7	2.1	64,200	44,940	45,570
0.8	2.9	65,000	52,000	52,870
0.9	3.8	65,800	59,200	60,340
1.0	5.0	66,767	66,770	68,270
1.2	7.9	68,300	82,000	84,370
1.5	14	71,000	107,000	111,200
1.8	22	73,500	130,000	136,600
2.0	28	75,148	150,300	158,700
2.5	49	79,400	200,000	214,700
3.0	78	83,928	251,800	275,200
3.5	115	87,500	306,300	340,800

[1] Using two 90° V-notch weirs:
$Q = 2(2.5H^{2.5})$

[2] $S + ½ O \Delta t = S + O (½ \Delta t) = S + 300 (O)$
$\Delta t = 600$ seconds

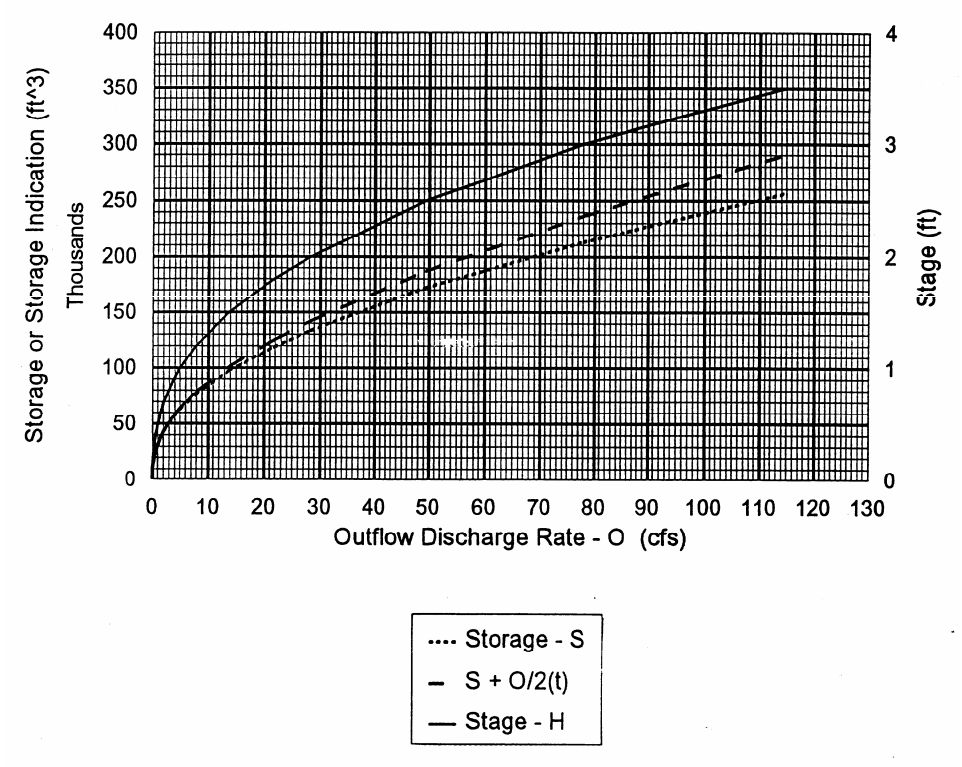


Figure 5. Pond-stage/storage indication curve for example problem.

Storage-Indication Calculation Procedure

Table 2 shows the calculations necessary to develop the pond outflow hydrograph and the plot of particle removal, for a triangular inflow hydrograph resulting from a 1.5 inch, 3-hour rain. Columns A through J of this table (to develop the outflow hydrograph and pond surface area) need to be calculated by rows (horizontally), while columns K through O (to calculate the upflow velocity and associated particulate removals) can be calculated vertically, based on the previously calculated column values. It should be noted that columns C through F are offset between the indicated time values and not for the specific times shown in column A. All of the starting values (time zero) in columns B (the beginning inflow rate), G (the beginning outflow rate), H (the pond storage volume above the normal wet pond water surface elevation), and I (the pond stage) are zero for this example.

Table 2a. Pond Performance Calculations for Example 1.5-Inch, 3-Hour Rain

A Time (min)	B Inflow (cfs)	C Average inflow for increment	D Average inflow volume (avg. inflow x time period)	E Previous storage minus incremental outflow S-0.5(O)Δt	F Previous storage plus incremental outflow S+0.5(O)Δt	G Outflow (O) (cfs)	H Storage (S) (ft^2)	I Pond stage (ft)	J Pond surface area (ft^2)
0	0					0	0	0	59,000
		4.5	2,700	0	2,700				
10	9					0.01	3,000	0.1	60,000
		13.5	8,100	2,997	11,100				
20	18					0.09	12,100	0.2	60,400
		22.5	13,500	12,073	25,600				
30	27					0.51	24,740	0.4	62,000
		31.5	18,900	24,590	43,490				
40	36					1.0	44,000	0.7	64,100
		40.5	24,300	43,700	68,000				
50	45					5.1	66,770	1.0	66,800
		50.0	30,000	65,240	95,240				
60	55					10	95,000	1.4	70,000
		59.5	35,700	93,500	129,200				
70	64					19	125,000	1.8	73,500
		68.5	41,100	119,300	160,400				
80	73					30	155,000	2.1	76,000
		77.5	46,500	146,000	192,500				
90	82					41	180,000	2.3	77,800
		86.5	51,900	167,700	219,600				
100	91					52	205,000	2.6	80,200
		95.5	57,300	189,400	246,700				
110	100					63	225,000	2.8	81,800
		95.5	57,300	206,100	263,400				
120	91					71	240,000	2.9	82,700
		86.5	51,900	218,700	270,600				
130	82					77	250,000	3.0	83,700
		77.5	46,500	226,900	273,400				
140	73					78	250,000	3.0	83,800
		68.5	46,100	226,600	267,700				
150	64					73	245,000	2.9	82,700
		59.5	35,700	223,100	258,800				
160	55					69	240,000	2.8	81,800
		50.0	30,000	219,300	249,300				
170	45					65	230,000	2.7	81,800
		40.5	24,300	210,500	234,800				
180	36					58	220,000	2.6	80,200
		31.5	18,900	202,600	221,500				
190	27					52	205,000	2.5	79,400
		22.5	13,500	189,400	202,900				
200	18					44	185,000	2.4	78,600
		13.5	8,100	171,800	180,000				
210	9					36	170,000	2.2	76,900
		4.5	2,700	159,200	162,000				
220	0					29	152,000	2.0	75,200
		0	0	143,300	143,300				
230	0					22	135,000	1.8	73,500
		0	0	128,400	128,400				
240	0					18	125,000	1.7	72,700
		0	0	119,600	119,600				

Table 2a. Pond Performance Calculations for Example 1.5-Inch, 3-Hour Rain (Continued).

A Time (min)	B Inflow (cfs)	C Average inflow for increment	D Average inflow volume (avg. inflow x time period)	E Previous storage minus incremental outflow S-0.5(O)Δt	F Previous storage plus incremental outflow S+0.5(O)Δt	G Outflow (O) (cfs)	H Storage (S) (ft^2)	I Pond stage (ft)	J Pond surface area (ft^2)
250	0					16	115,000	1.6	71,900
		0	0	110,200	110,200				
260	0					13	105,000	1.5	71,000
		0	0	101,100	101,100				
270	0					11	100,000	1.4	70,000
		0	0	96,700	96,700				
280	0					10	95,000	1.3	69,200
		0	0	92,000	92,000				
290	0					9	90,000	1.3	69,200
		0	0	87,300	87,300				
300	0					8	85,000	1.2	68,500
	Maximum = 100 cfs		Total = 660,000			Max. = 78 Total = 981			

Table 2b. Particle Removal Performance Calculations for Example 1.5-inch, 3-hr Rain

A Time (min.)	B Inflow (cfs)	G Outflow (O) (cfs)	J Pond surface area (ft^2)	K Upflow velocity (ft/sec)	L Critical particle size (μm)	M Weighted particle size (outflow x size)	N Percent suspended solids control	O Weighted control (outflow x control)
0	0	0	59,000	0	-	0	100	0
10	9	0.01	60,000	1.7×10^{-7}	0.3	0.003	100	1
20	18	0.09	60,400	1.5×10^{-6}	0.6	0.05	100	9
30	27	0.51	62,000	8.2×10^{-6}	1.3	0.66	99	50
40	36	1.0	64,000	1.6×10^{-5}	1.8	1.8	98	98
50	45	5.1	66,800	7.6×10^{-5}	3.8	19.4	91	464
60	55	10	70,000	1.4×10^{-4}	5.1	51	88	880
70	64	19	73,500	2.6×10^{-4}	7	133	84	1,596
80	73	30	76,000	4.0×10^{-4}	8	240	82	2,460
90	82	41	77,800	5.3×10^{-4}	10	410	78	3,200
100	91	52	80,200	6.5×10^{-4}	11	572	75	3,900
110	100	63	81,800	7.7×10^{-4}	12	756	73	4,600
120	91	71	82,700	8.6×10^{-4}	12	852	73	5,180
130	82	77	83,700	9.2×10^{-4}	13	1,000	71	5,470
140	73	78	83,800	9.3×10^{-4}	13	1,010	71	5,540
150	64	73	82,700	8.8×10^{-4}	13	949	71	5,180
160	55	69	81,800	8.4×10^{-4}	12	830	73	5,040
170	45	65	81,800	8.0×10^{-4}	12	780	73	4,750
180	36	58	80,200	7.2×10^{-4}	11	638	75	4,350
190	27	52	79,400	6.6×10^{-4}	11	572	75	3,900
200	18	44	78,600	5.6×10^{-4}	10	440	78	3,430
210	9	36	76,900	4.7×10^{-4}	9	320	80	2,880
220	0	29	75,200	3.9×10^{-4}	8	232	82	2,380
230	0	22	73,500	3.0×10^{-4}	7.5	170	83	1,830
240	0	18	72,700	2.5×10^{-4}	6.5	120	85	1,530
250	0	16	71,900	2.2×10^{-4}	6	96	86	1,380
260	0	13	71,000	1.8×10^{-4}	6	78	86	1,120
270	0	11	70,000	1.6×10^{-4}	5.5	61	88	968
280	0	10	69,200	1.5×10^{-4}	5.3	53	88	880
290	0	9	69,200	1.3×10^{-4}	5	45	89	800
300	0	8	68,500	1.2×10^{-4}	4.7	38	89	710
						Total = 10,468		Total = 74,576

Peak reduction factor: PRF = 1- $[(Q_{o\,max})/(Q_{i\,max})]$ = 1 – [(78)/(100)] = 0.22
Weighted average critical particle size =[total (outflow x size)]/[total (outflow)] = 10,468/981 = 10.7 μm
Weighted average suspended solids control =[total (outflow x control)]/[total (outflow)] = 74,576/981 = 76%

Column A shows the times at ten minute increments for five hours (300 minutes) since the start of the runoff. Column B is the pond inflow hydrograph (instantaneous flow rates at each time increment). The calculation of the inflow hydrograph is shown on Table 3. The inflow runoff rates can be estimated using WinTR-55 for a design storm, or by any other method, or from an observed hydrograph. Table 3 shows how the example Monroe Street detention pond watershed is divided into these three major land surfaces and how the average runoff rates are calculated for the storms under consideration.

Table 3. Rain and Inlet Hydrograph Characteristics for Example

Rain depth (in.)	Rain intensity (in/hr)	Rain duration (hrs)	Return frequency (years)	Average cfs/acre			Average cfs (for total area)[1]			Total avg. flow (cfs)	Total flow volume (10^3 ft^3)	Runoff duration (hrs)	Peak 5-min. flow (cfs)	Time to peak flow (hrs)	Total storm volume as a fraction of pond base storage[2]
				Imperv.	Pervious	Imper. To Pervious	Imperv.	Pervious	Imper. To Pervious						
0.1	0.03	3	<1	0.013	0.0001	0.006	0.8	0.01	0.4	1.2	16	3.6	2.4	1.8	0.1
0.5	0.17	3	<1	0.16	0.0035	0.08	10	0.4	3	13	180	3.6	26	1.8	1.0
0.9	1.8	0.5	1	1.8	0.06	0.45	113	8	28	150	320	0.6	300	0.3	1.8
1.1	1.1	1	1	1.0	0.045	0.5	63	6	32	100	430	1.2	200	0.6	2.4
1.3	0.7	2	1	0.7	0.03	0.35	44	4	22	70	610	2.4	140	1.2	3.4
1.5	0.5	3	1	0.5	0.025	0.25	32	3	16	50	640	3.6	100	1.8	3.6
1.7	0.3	6	1	0.3	0.015	0.15	19	2	9	30	780	7.2	60	3.6	4.3
2.0	0.2	12	1	0.2	0.012	0.10	13	2	6	20	1,100	14	40	7	6.1
2.3	0.1	24	1	0.1	0.006	0.05	6	1	3	10	1,100	29	20	14.5	6.1
5.5	0.23	24	100	0.23	0.05	0.12	14	6	8	28	2,900	29	56	14.5	16

Areas for different land cover types:

Impervious area: 63 acres
Pervious area: 126 acres
Impervious area draining to pervious area: 63 acres

Pond base storage (normally wet volume) is about 180,000 ft^3.

Urban hydrographs can be represented with a simple triangular shape (as shown on Figure 6), with a peak runoff rate equal to about twice the average runoff rate and with the runoff duration about 20 percent longer than the rain duration (Pitt and McLean 1986). This simplification is reasonable for most small to intermediate rains, especially when the effects of a relatively large series of individual rains on a pond are to be evaluated statistically, instead of describing the pond performance associated with a single "design" storm. For larger rains, the ratio of the peak to average flow usually increases to about 3.4. This higher ratio can be represented using a multiple triangular hydrograph, similar to that used by the SCS. The peak flow rate in this example (1.5 inch, 3 hour rain) is assumed to be about 100 cfs and occurs at 1.8 hours into the runoff period. Of course, any hydrograph shape can be used in these calculations. This triangular shape is used in SLAMM as a simplification when evaluating very large numbers of storms. However, WinDETPOND is a more detailed detention pond program that allows any runoff hydrograph to be evaluated (if manually entered). Pond leakage, groundwater intrusion, evaporation, or any other additional water losses or inflows can be added or subtracted from the pond inflow hydrograph, if desired, and are included in the computer programs.

Column C shows the average runoff rates (cfs) for the two adjacent time increments. Column D shows the incremental incoming runoff volume (cubic feet) for each time increment (average inflow runoff rate, from column C, times the increment time, or 600 seconds). Column E shows the previous storage volume minus one-half of the outflow rate times the time increment (one-half of the outflow volume). The first value shown in this column (for the increment 0 to 10 minutes) is zero because the previous storage and outflow rate values (for time 0) are both 0: 0 - 1/2 (0) (600) = 0 - 0 = 0. The second value in column E (for the time increment 10 to 20 minutes) is: 3,000 - 1/2 (0.01) (600) = 3,000 - 3 = 2,997. Before this second value in column E can be calculated, the previous outflow rate (O) and pond storage (S) values (for time 10 minutes) must be calculated.

Column F is the Column E value plus the Column D value (increment inflow). The first value shown in Column F is therefore equal to the first value shown in Column D (2700 for this example). The second value in column F (for the time increment 10 to 20 minutes) is 8,100 + 2,997 = 11,100.

Column G (pond outflow rate, O) and column H (pond storage, S) also start as 0 values at time 0. Later values in these columns are obtained from the storage-indication curve, using the column F value for the previous time increment. The 2,700 value in column F (representing S + 1/2 (O) (dt)) is used in Figure 5 to obtain a corresponding pond outflow rate of about 0.01 cfs and a pond storage volume of about 3,000 cubic feet.

The stage values in column I are obtained from the stage-discharge curve (shown in tabular form on Table 1 for this example), using the corresponding outflow rates from column G. The pond surface area values are obtained from the stage-area curve (Figure 4), using the corresponding stage values from column I.

The particle removal calculations are based on the previously described upflow velocity method, using the "instantaneous" pond surface area values (from column J) and outflow rate values (from column G). Column K shows the upflow velocities (in feet per second) calculated by dividing the outflow rate values (column G) by the corresponding pond surface area values (from column J). Column L shows the sizes of the critical particles (the smallest particles that would settle below the bottom of the outfall structure and therefore be "retained") and are estimated from Figure 5 based on these upflow velocities. Column M shows the outflow rate weighting of these particle sizes (critical particle size times the outflow rate). In this example, the "flow-weighted" critical particle size is about 11 μm.

Column N shows the estimated particulate residue percentage removals, based on a particle size distribution from Figure 5. Column O shows the flow-weighted calculations. For this example, a particulate residue reduction of about 75 percent may be expected.

The results of these calculations can be effectively presented on several graphs. Figure 6 compares the inlet and outlet hydrographs, Figure 7 shows the stage elevations above the permanent pool and the upflow velocities, and Figure 8 shows the critical particle sizes controlled and the estimated percentage control of particulate residue for this example.

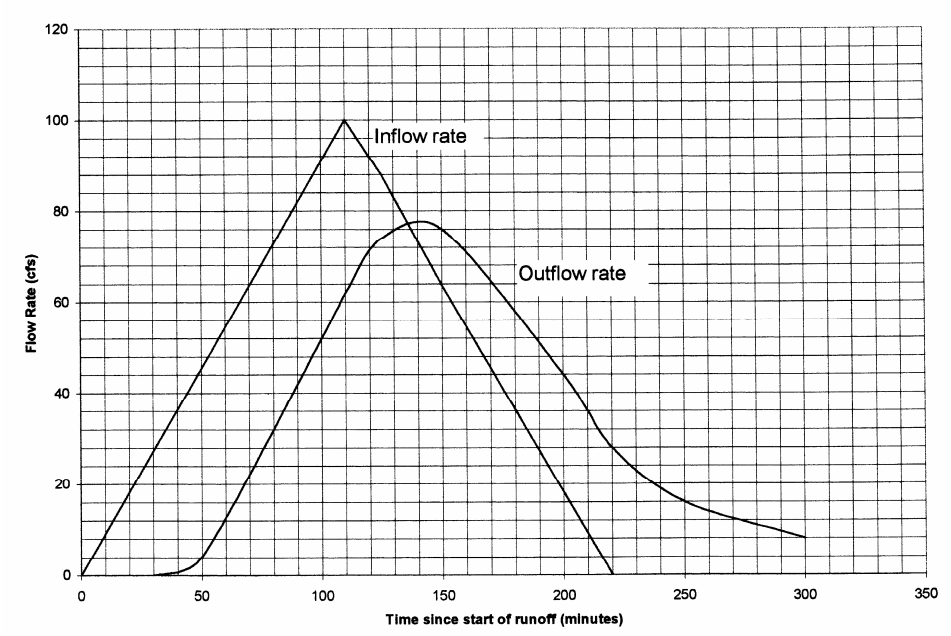


Figure 6. Inflow and outflow hydrographs for example problem.

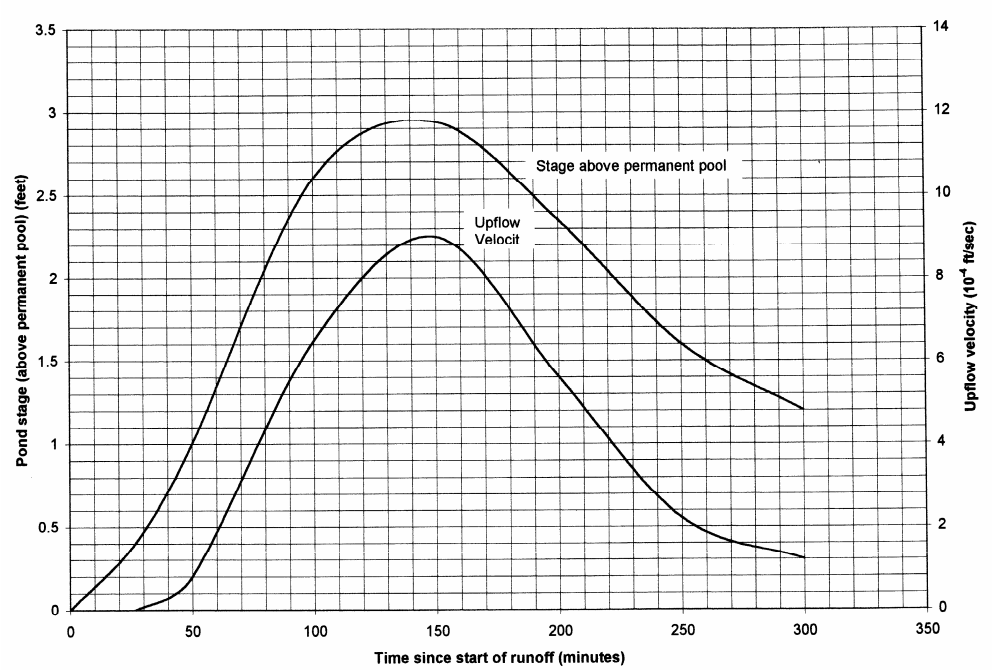


Figure 7. Stage and upflow velocity plots for example problem.

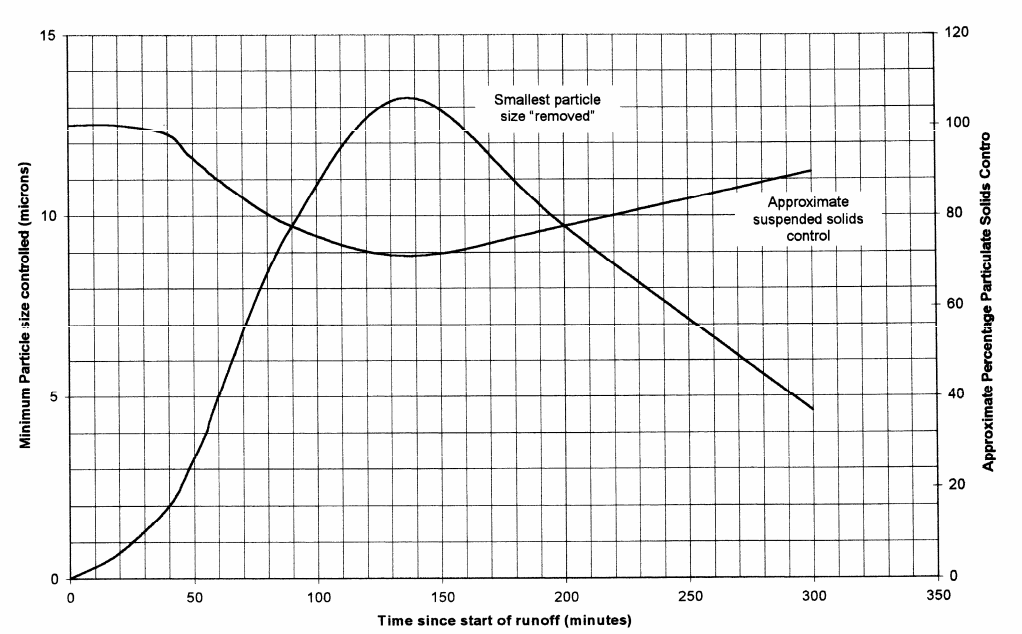


Figure 8. Particle sizes and percentage suspended solids removed for example problem.

The Use of WinTR-55 for Detention Pond Analyses

The complete User Guide for TR-55 (1986 version) can be downloaded from: http://www.wcc.nrcs.usda.gov/water/quality/common/tr55/tr55.pdf. According to the NRCS (2002), Technical Release 55 (TR-55) *Urban Hydrology for Small Watersheds* was first issued in January 1975 as a simplified procedure to calculate the storm runoff volume, peak rate of discharge, hydrographs and storage volumes required for stormwater management structures (SCS 1975). This initial version involved manual methods and assumed the Type II rainfall distribution for all calculations. In June 1986, major revisions were made in TR-55 by adding three additional rainfall distributions (Type I, IA and III) and programming the computations. Time of concentration was estimated by splitting the hydraulic flow path into separate flow phases (SCS 1986). This 1986 version is the last non-computerized version and has been widely used for drainage design in urban areas.

Even though the manual version of TR-55 is currently being phased out, its use may still be of interest for many situations, especially as WinTR-55 is still an official "beta" version. In addition, the User Guide for TR-55 (SCS 1986) contains a more through description of the basic processes included in the model.

Only the following site characteristics are needed to use TR-55: drainage area, curve number (CN), and time of concentration (Tc). With this information, it is possible to develop a hydrograph for a specific design storm. If in a complex drainage area, the watershed can be subdivided into subwatersheds for routing the flows through the system. TR-55 and WinTR-55 handle watershed routing quite differently, with WinTR-55 conducting a more through routing approach, similar to the method used in TR-20. However, the WinTR-55 "structures" module (dealing with ponds) has some serious shortcomings in the available outlet structure descriptions. The basic TR-20 is therefore recommended by NRCS when more detailed analyses are needed. The following discussion summarizes many of the basic features and approaches of WinTR-55 for the analysis of detention ponds in watershed analyses.

This example application of WinTR-55 demonstrates the use of this new program in evaluating detention ponds in a watershed with multiple subdrainage areas. As noted in the User Guide, WinTR-55 has some limitations compared to the more comprehensive TR-20 program. In the analysis of detention ponds ("structures"), the most important limitation is the availability of only 3 types of outlet structures (a broad-crested weir, a 90^{o} weir, and a pipe outlet). The greatest concern is how the pipe outlet is considered (a short-tube approximation approach). The USDA

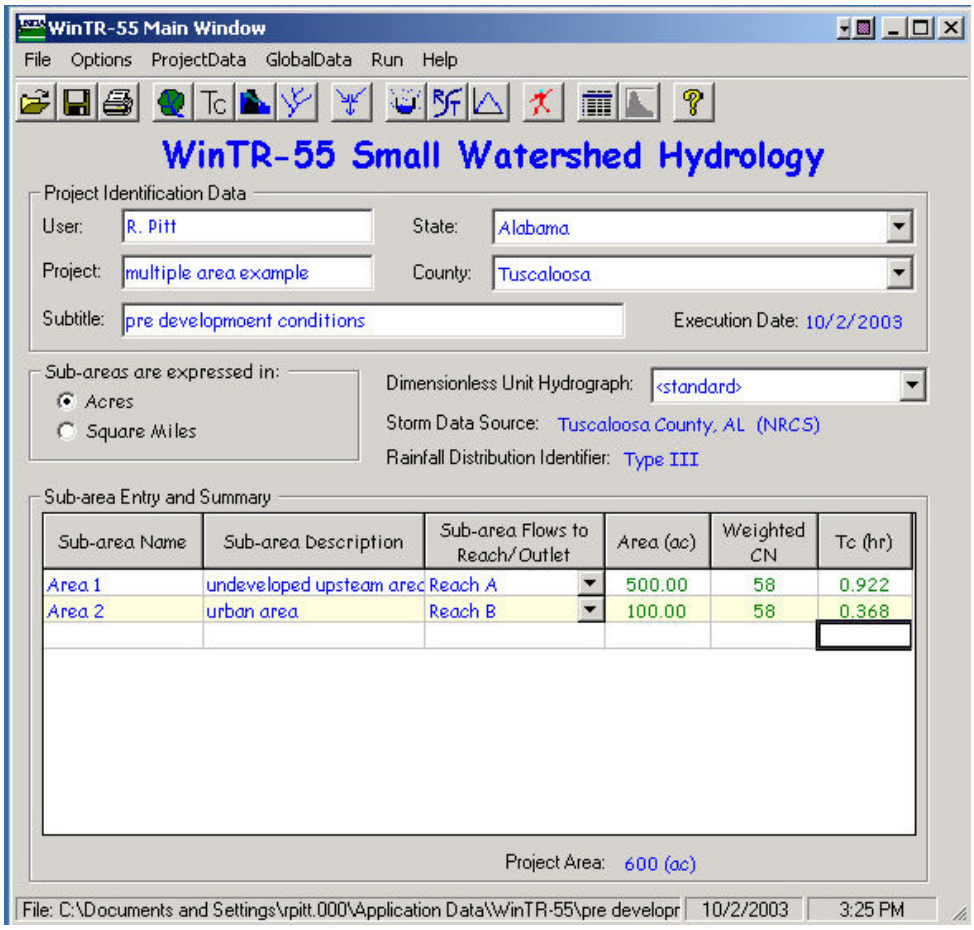
WinTR-55 team explained this as follows: This approximation uses the pipe diameter and the head on the pipe as the total head (permanent pool elevation to outlet invert plus 1/2 diameter). This estimation of head, coupled with a slightly different orifice flow coefficient (0.6 instead of 0.8), essentially cancel each other out and the result is a higher discharge estimate for one type of pipe material versus a slightly lower discharge estimate for another. The pipe materials checked were reinforced concrete and corrugated metal. Overall, the estimated differences were very small. The future version of the User Manual will be rewritten to reflect this short tube flow assumption with a disclaimer that if the user needs a more exact estimate they should use a different tool (SITES or a user-estimated rating in TR-20). They also stated that Version 1.0 of WinTR-55 will keep the existing short-tube approximation for pipe outlets of structures. A future version 2.0 of WinTR-55 will likely have the ability to enter a user-provided stage-storage-discharge rating curve or more complete pipe rating curves.

WinTR-55 is a great improvement over the older TR-55 in that more accurate channel and reservoir routing is provided. This Windows version of the program is also very easy to use and the provided graphical output options enable efficient and rapid evaluations.

This simple example is comprised of two subwatersheds, a 500 acre undeveloped area and an adjacent 100 acre developing area. Specific characteristics of these areas (soils, land use breakdowns, channel characteristics, etc.) are provided in the following discussion. Initially, the pre-development conditions are examined, followed by developed conditions. A preliminary design of a detention pond is then evaluated to attempt to provide similar discharge peak flows from the developed watershed portion after development as before development.

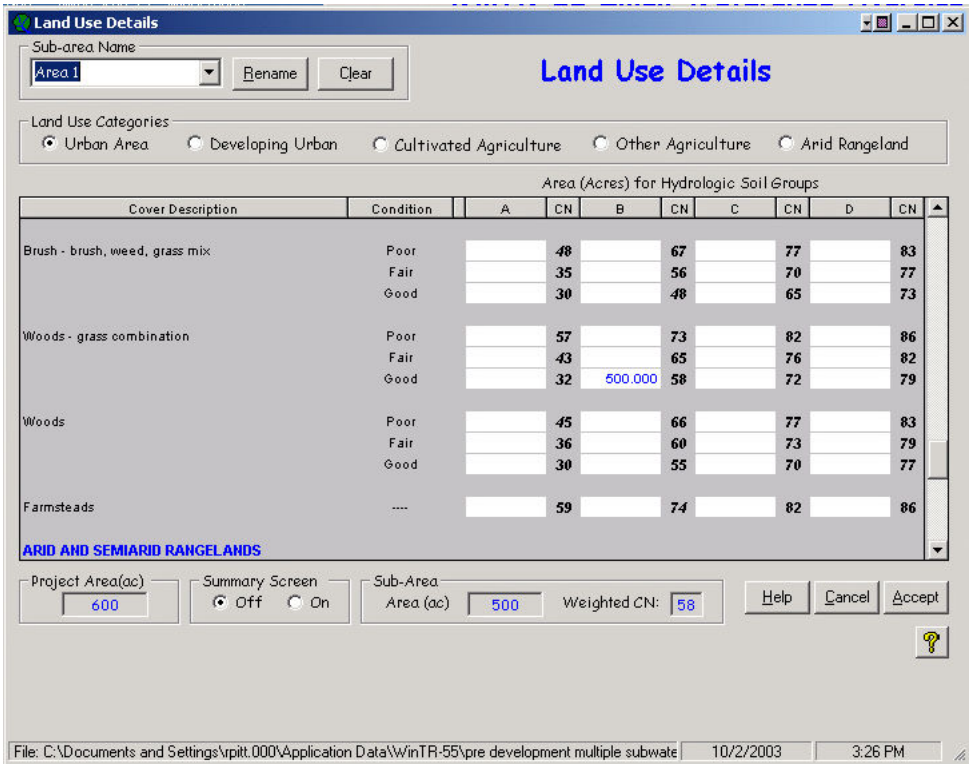
Predevelopment Conditions

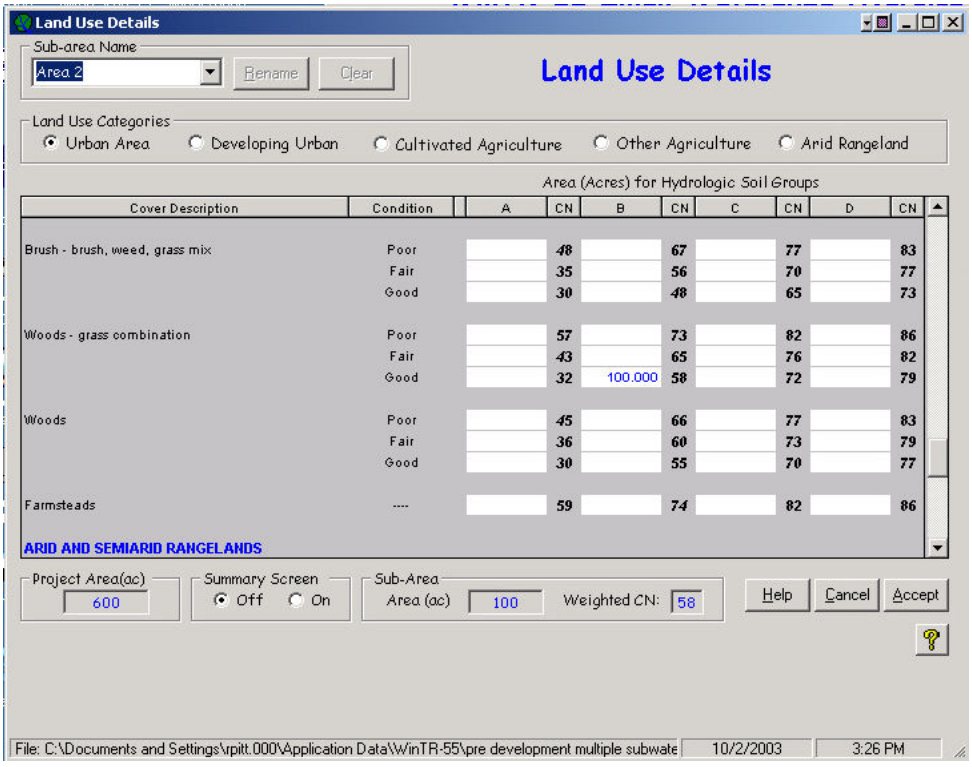
The following screen shows the basic site conditions. The screen also shows the location of the area (Tuscaloosa County, Alabama), the selection of the standard dimensionless hydrograph, the selection of the area units, and labels. The drop-down “options” menu was also used to select “English” units (actually US customary units). The area, CN, and Tc values area entered and calculated in other screens and the information was automatically transferred to this screen.



In order to enter the area, CN, and Tc screens, double-click on one of the cells in the columns under the desired label. The following screen is opened when either the area or CN column is selected. The screen shows the complete listing of available land uses and surface covers for each of the 4 hydrological soil groups (scrolling is needed to see all the options). Type in the area associated with each condition for each area. In this example, the pre-development condition is woods-grass combination in good condition, with B soils. Area 1 is 500 acres in size, while Area 2 (the developing area) is 100 acres in size. These pre-development conditions are the same throughout the sub-areas, but it is possible to select a variety of conditions and have the program automatically weight the overall CN. If desired, it is possible to directly enter the CN value without using the calculator.

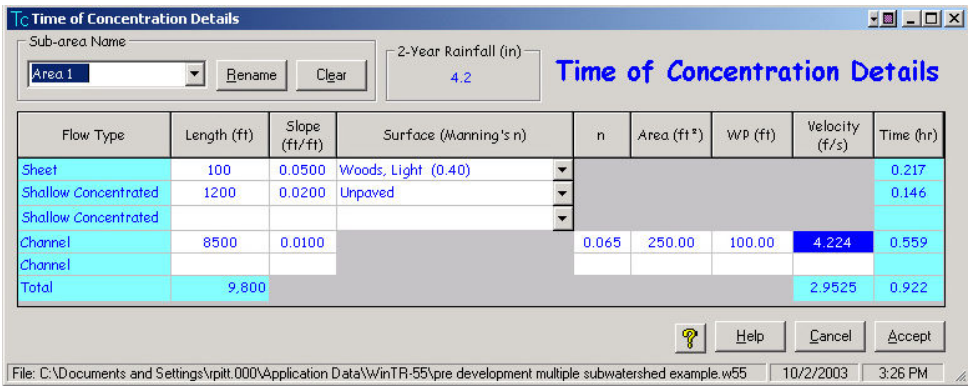
Although not noted in the WinTR-55 User Guide, the prior TR-55 guidance recommended that the range of CNs for one area should be relatively narrow, with no more than an extreme difference of 5 in the CNs for any area. If the CN values varied by more than 5, it was recommended that the sub-area be further divided to place the extreme values in separate sub-areas. This was recommended to enable more accurate routing of sub-area flows compared to using a composite CN based on a wide range of individual values.

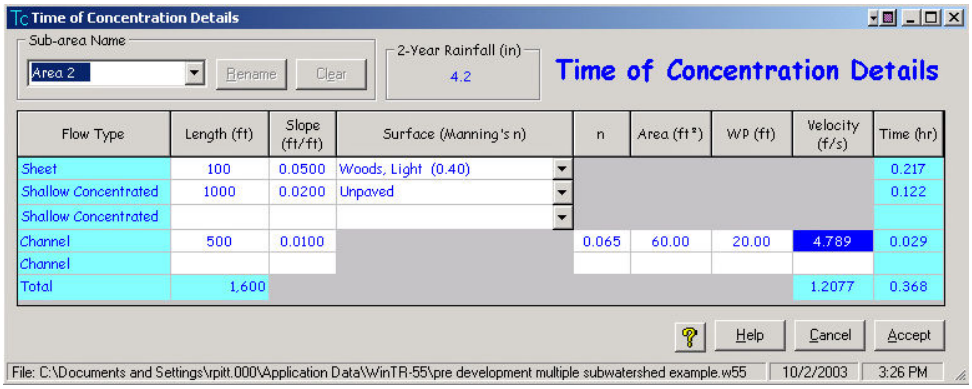

Land Use Details
Sub-area Name
Area 1
Rename
Clear
Land Use Details
Land Use Categories
Urban Area
Developing Urban
Cultivated Agriculture
Other Agriculture
Arid Rangeland
Area (Acres) for Hydrologic Soil Groups
Cover Description
Condition
A
CN
B
CN
C
CN
D
CN
Brush - brush, weed, grass mix
Poor 48 67 77 83
Fair 35 56 70 77
Good 30 48 65 73
Woods - grass combination
Poor 57 73 82 86
Fair 43 65 76 82
Good 32 500.000 58 72 79
Woods
Poor 45 66 77 83
Fair 36 60 73 79
Good 30 55 70 77
Farmsteads
---- 59 74 82 86
ARID AND SEMIARID RANGELANDS
Project Area(ac)
600
Summary Screen
Off
On
Sub-Area
Area (ac)
500
Weighted CN:
58
Help
Cancel
Accept
File: C:\Documents and Settings\rpitt.000\Application Data\WinTR-55\pre development multiple subwate
10/2/2003
3:26 PM


Land Use Details
Sub-area Name
Area 2
Rename
Clear
Land Use Details
Land Use Categories
Urban Area
Developing Urban
Cultivated Agriculture
Other Agriculture
Arid Rangeland
Area (Acres) for Hydrologic Soil Groups
Cover Description
Condition
A
CN
B
CN
C
CN
D
CN
Brush - brush, weed, grass mix
Poor 48 67 77 83
Fair 35 56 70 77
Good 30 48 65 73
Woods - grass combination
Poor 57 73 82 86
Fair 43 65 76 82
Good 32 100.000 58 72 79
Woods
Poor 45 66 77 83
Fair 36 60 73 79
Good 30 55 70 77
Farmsteads
---- 59 74 82 86
ARID AND SEMIARID RANGELANDS
Project Area(ac)
600
Summary Screen
Off
On
Sub-Area
Area (ac)
100
Weighted CN:
58
Help
Cancel
Accept
File: C:\Documents and Settings\rpitt.000\Application Data\WinTR-55\pre development multiple subwate
10/2/2003
3:26 PM

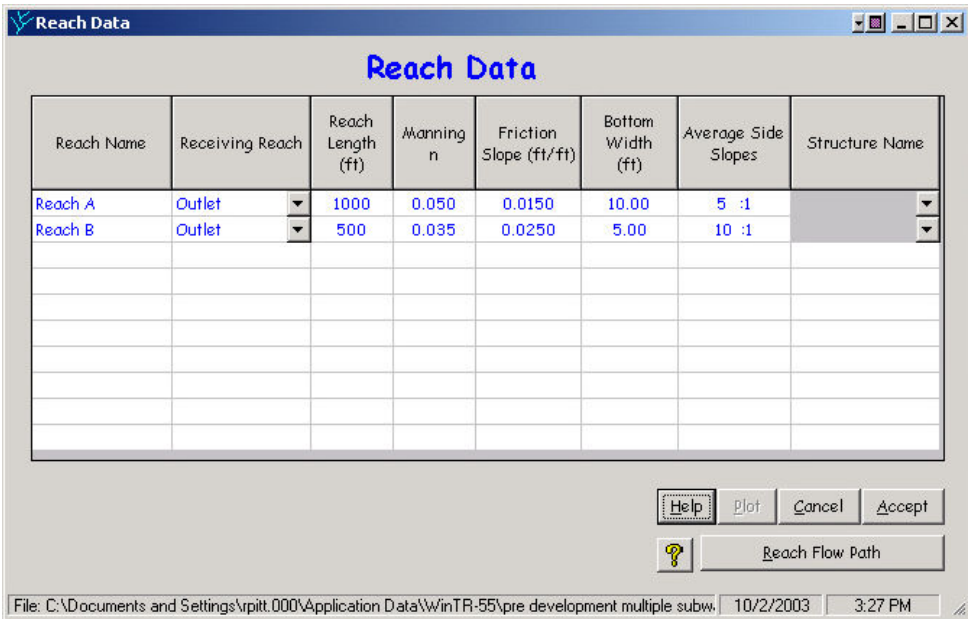
The Tc calculation screen can be opened in the same way, by double-clicking on any cell under the Tc column. If available, the Tc can be directly entered without using the calculator. The following screens show the examples for sub-areas 1 and 2 (selected by using the drop-down option under "sub-area name"). The flow path described on the screens needs to be pre-determined to be the critical Tc flow path (the path that requires the longest time for water drainage, not the physically longest flow path necessarily).

As in TR-55, the Tc can be comprised of three components. The sheet flow length is now restricted to a maximum length of 100 ft. Prior TR-55 guidance allowed a maximum length of 300 ft, but this was thought to be excessive by the WinTR-55 development team. The "Surface (Manning's n)" menu lists the available sheetflow roughness values. These are substantially different than what would be appropriate for channel flow conditions for rougher material. Smooth surfaces have similar values. The shallow concentrated flow surface drop-down options are restricted to "paved" and "unpaved." Two shallow concentrated flow segments are allowed. There are also two channel segments allowed. These are usually designated as streams on USGS topographic maps.

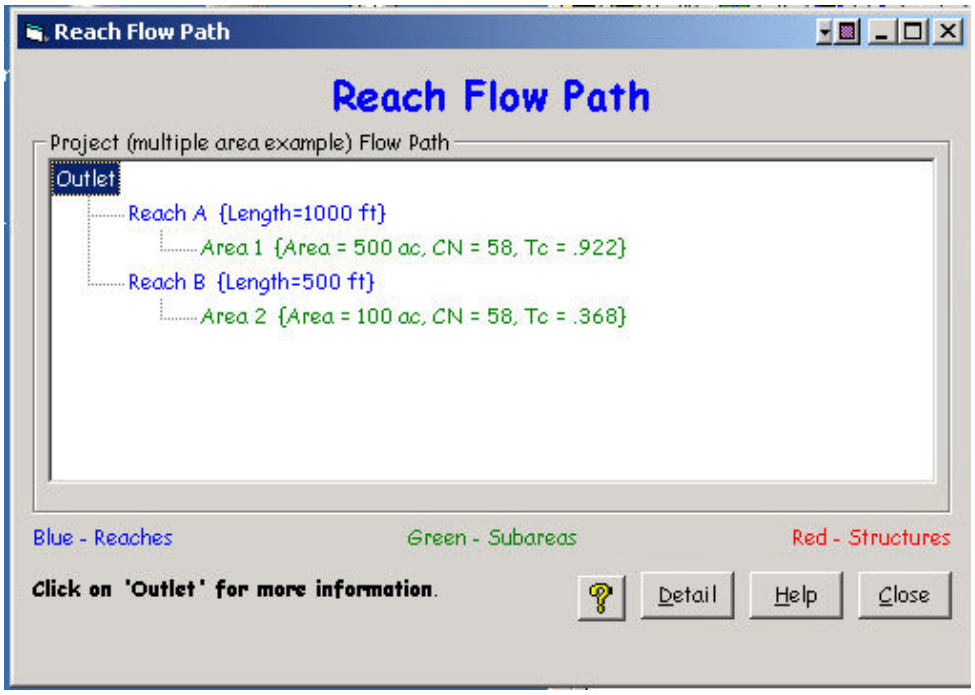




The "Reach Data" also needs to be entered. These are not the channels described on the Tc screen. The Tc channels are located within the sub-areas. The Reach channels are the channels into which the sub-areas discharge (and as noted on the opening screen). This screen also asks for the receiving reach into which each reach discharges. It is also possible to designate the outlet as the receiving reach, as in this example. This screen is also used to designate a reach as a structure ("reaches" can be either channels or detention ponds, with the appropriate routing procedure used). If the structure has already been described, then the structure name will appear on the structure name drop-down menu.

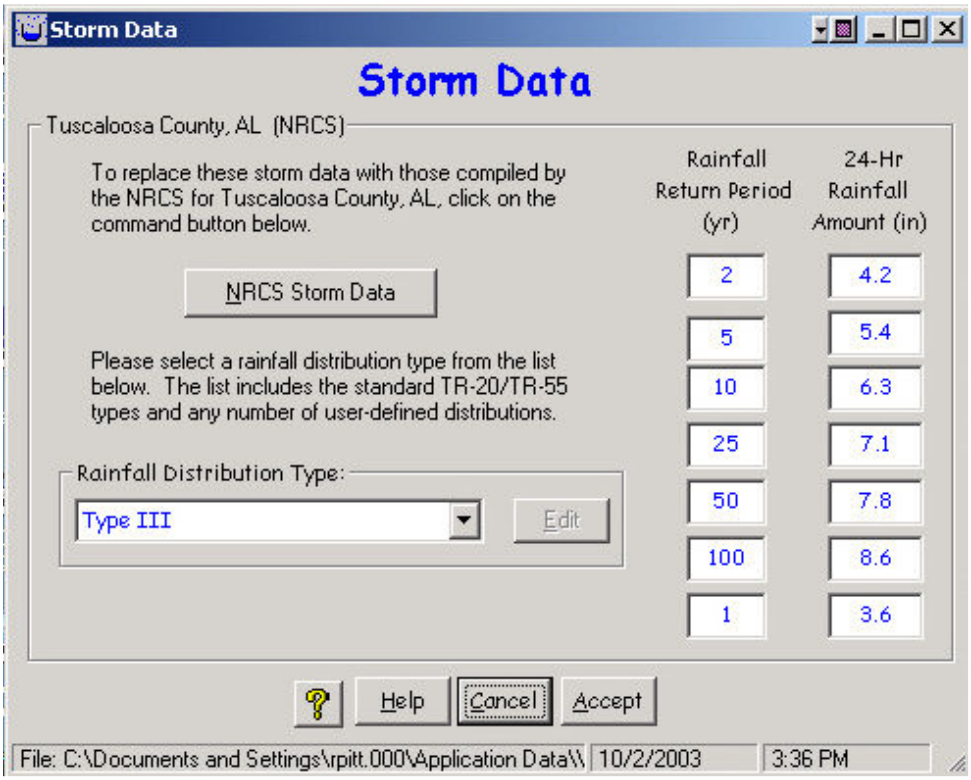


The "Reach Flow Path" should be selected to confirm that the model interpreted the entered area and reach connections correctly. This screen shows the basic watershed area conditions, plus shows the reaches each sub area flows into, plus shows how the reaches are combined as they flow downstream. It is possible to construct and evaluate a very complex set of sub-areas for evaluation. This example is about as simple as possible and still show how pond and sub area hydrographs can be combined.



Finally, the "Storm Data" must be selected, or entered. The following screen is available under the "GlobalData" drop down main menu. If the "NRCS Storm Data" button is selected, the standard 24-hr rainfall amounts and

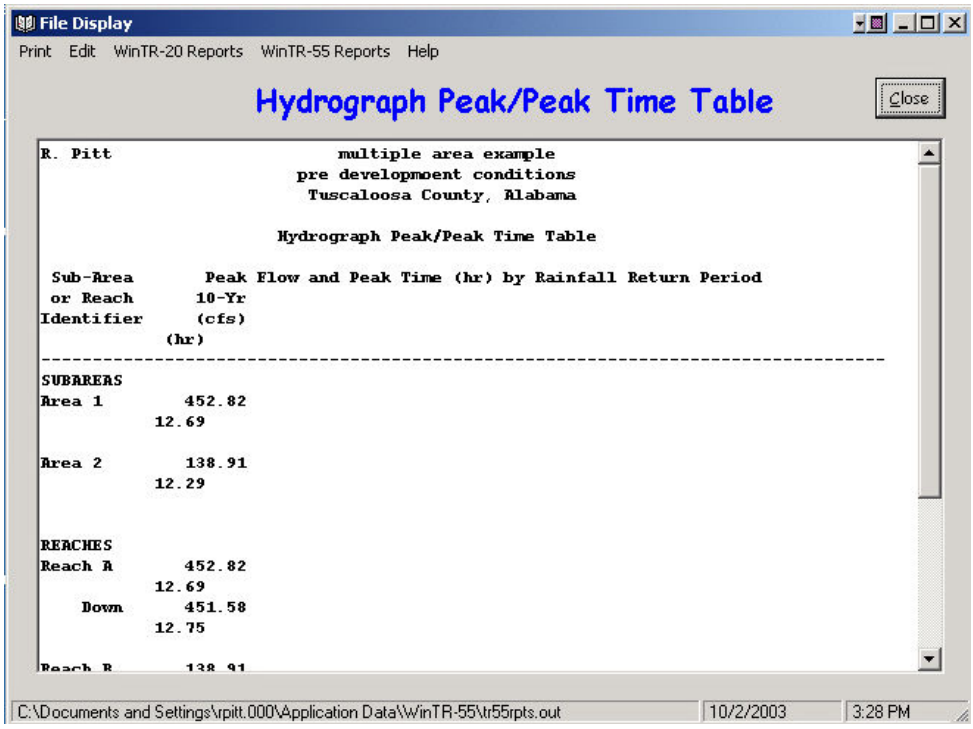
appropriate Rainfall Distribution Type are used, corresponding to the county selected on the first program screen. It is possible to enter other rainfall amounts. WinTR-55 is an event model that is used for individual design storms, although WinTR-55 can examine the entire set, or a sub-set, of the standard storms. Although the 24-hr rainfall amounts are used, the critical rain intensity corresponding to the Tc is actually used to normalize the dimensionless hydrograph.



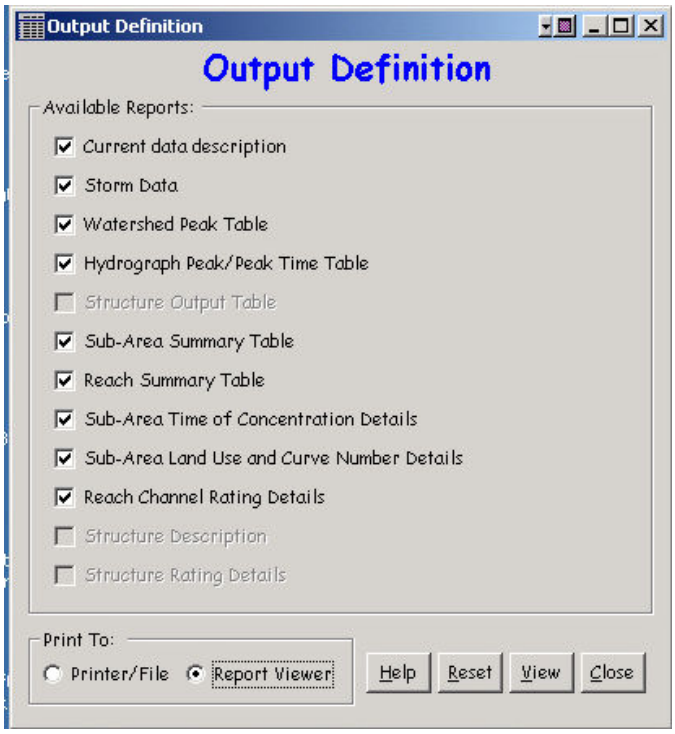
The "Run" icon is then selected and the following screen appears. This screen is used to select which event(s) are to be evaluated.

When the "Run" button is selected, after clicking on each desired rain, the program calculates the site runoff and routes it through each reach. An embedded version of TR-20 is actually used to conduct the analyses, being much better than the prior manual TR-55 procedures which required rather crude increments of important site factors. The

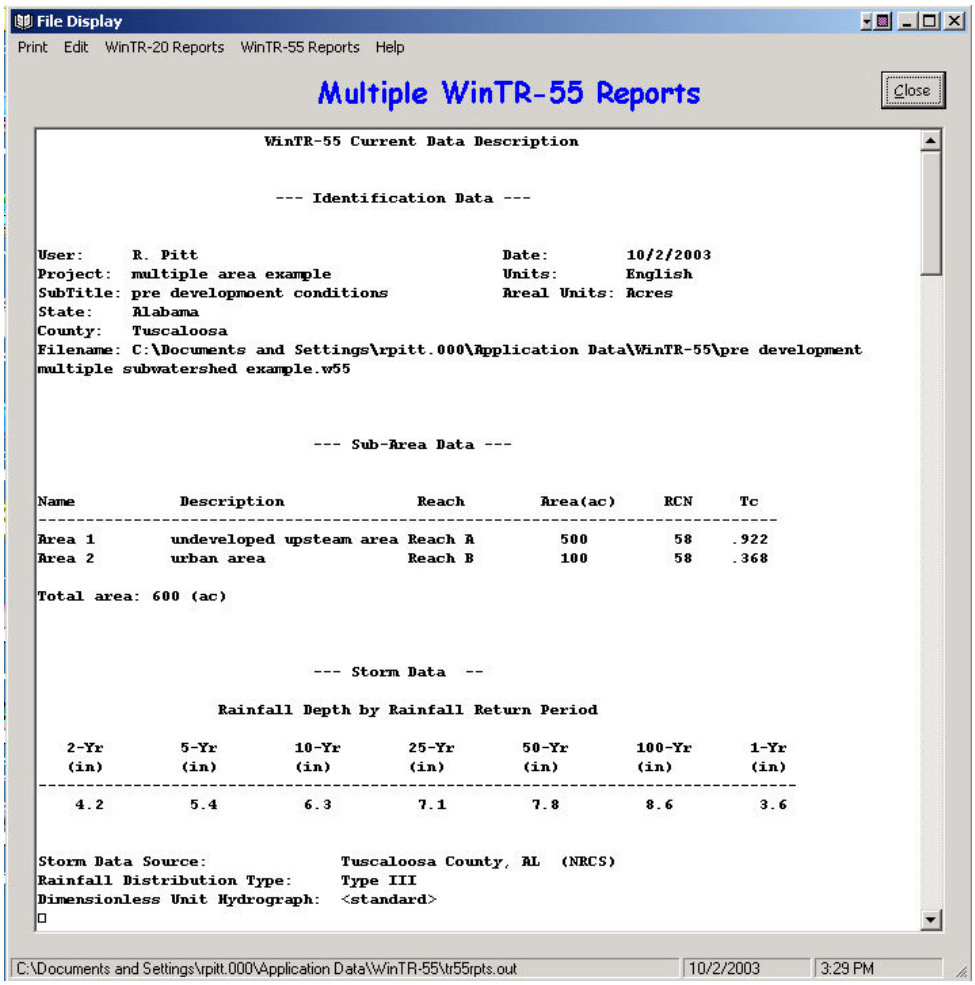
following screen is then automatically displayed after a run. This screen displays the TR-20 output screen, showing the peak runoff conditions and times. It is also possible to select "WinTR-20 Reports" for more detailed output information.



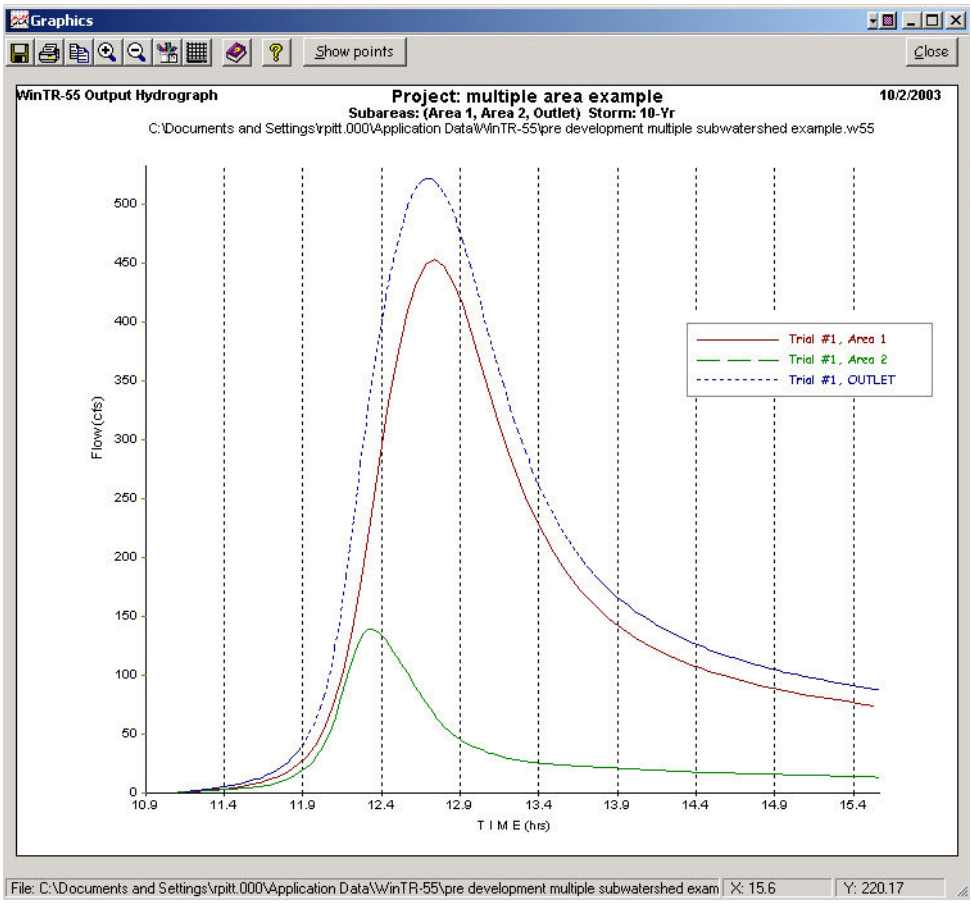
The "Output Definition", or report writer, icon displays the following screen. This allows specific information to be produced in a written report, or displayed on the computer screen.



The following is an on-screen report:

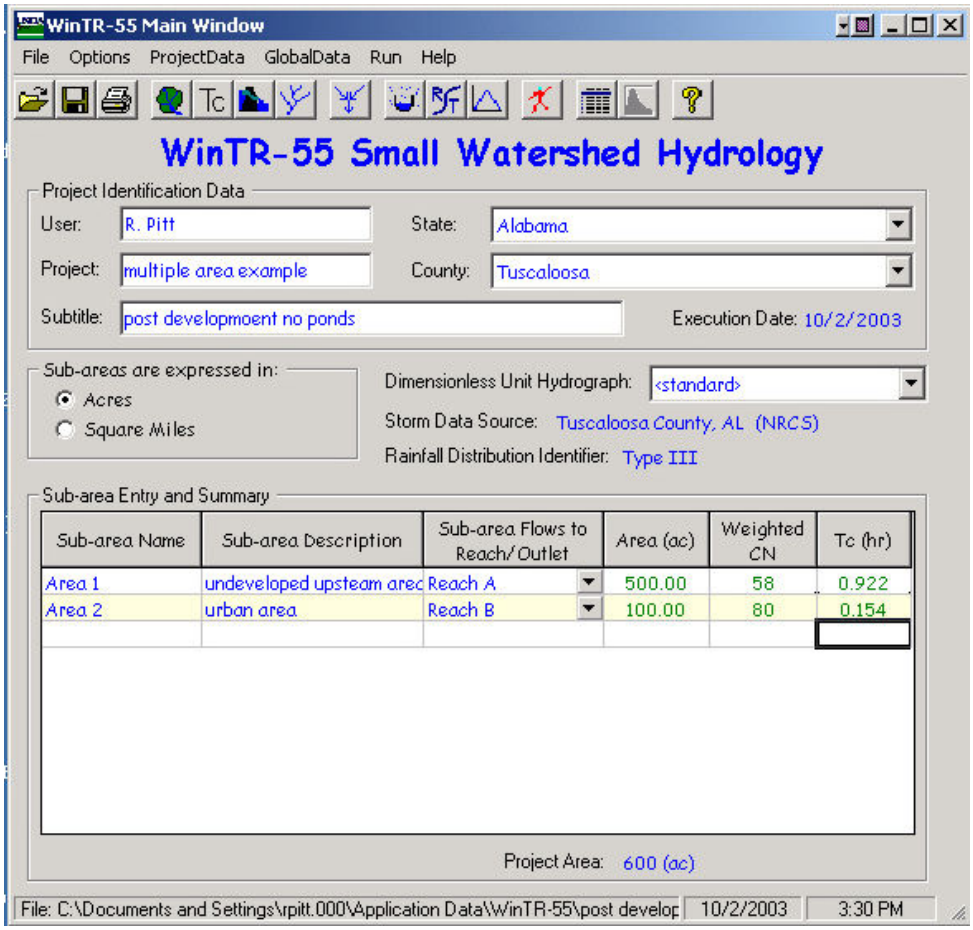


The following is the hydrograph that can be plotted by selecting the next to last icon on the top tool bar. The selection screen allows different hydrographs to be displayed. This plot shows how the pre-development hydrographs from the two sub-areas join for the complete hydrograph. The 10-year storm (having a 10% chance of occurring in any one year) produces a peak flow of about 139 cfs in the developing watershed. The upland sub-area peak flow was about 453 cfs, while these combined to create a total basin peak flow of about 522 cfs.

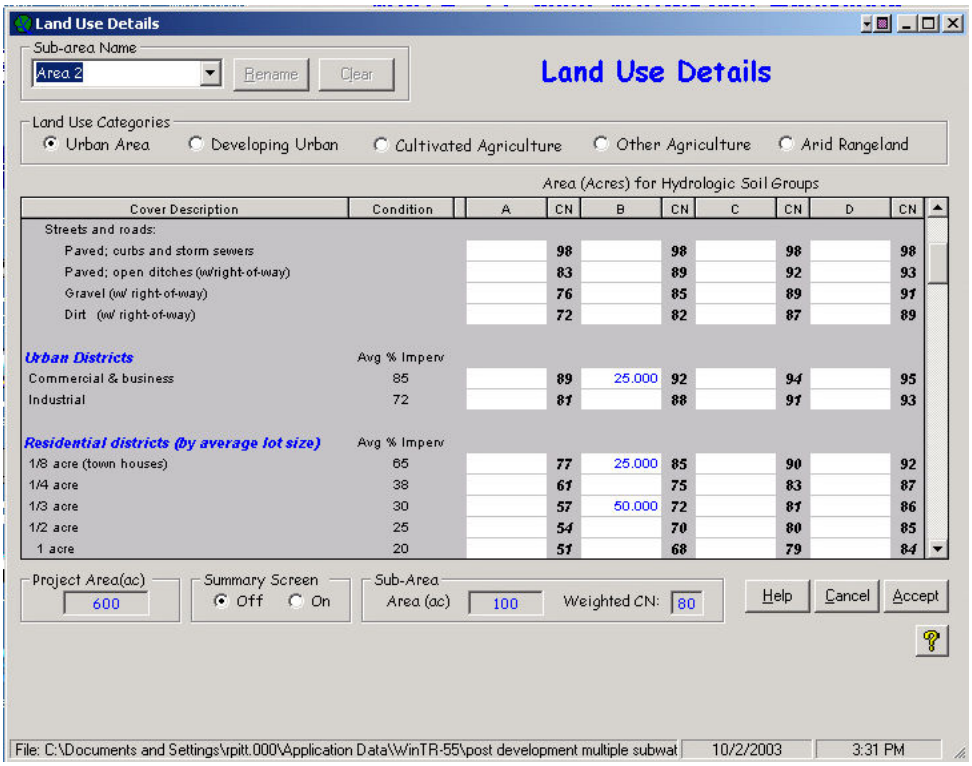


Post Development Conditions

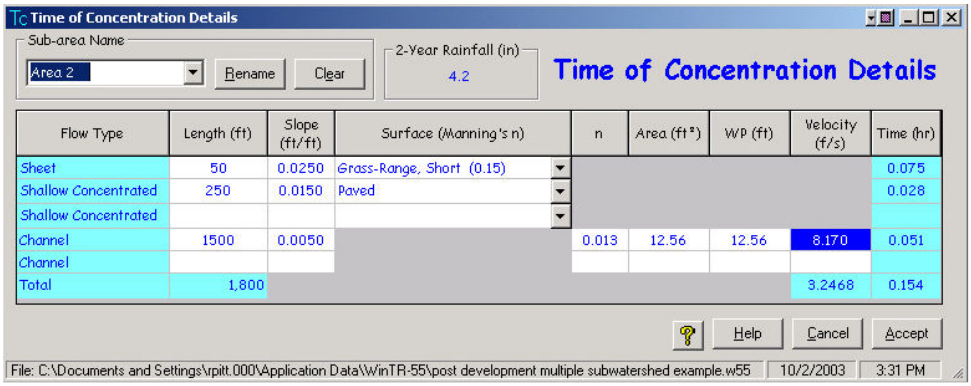
The pre-development file was edited and re-saved (using the "save as" option under the file drop-down menu) to reflect developed conditions in sub-area 2, as shown on the following screens:



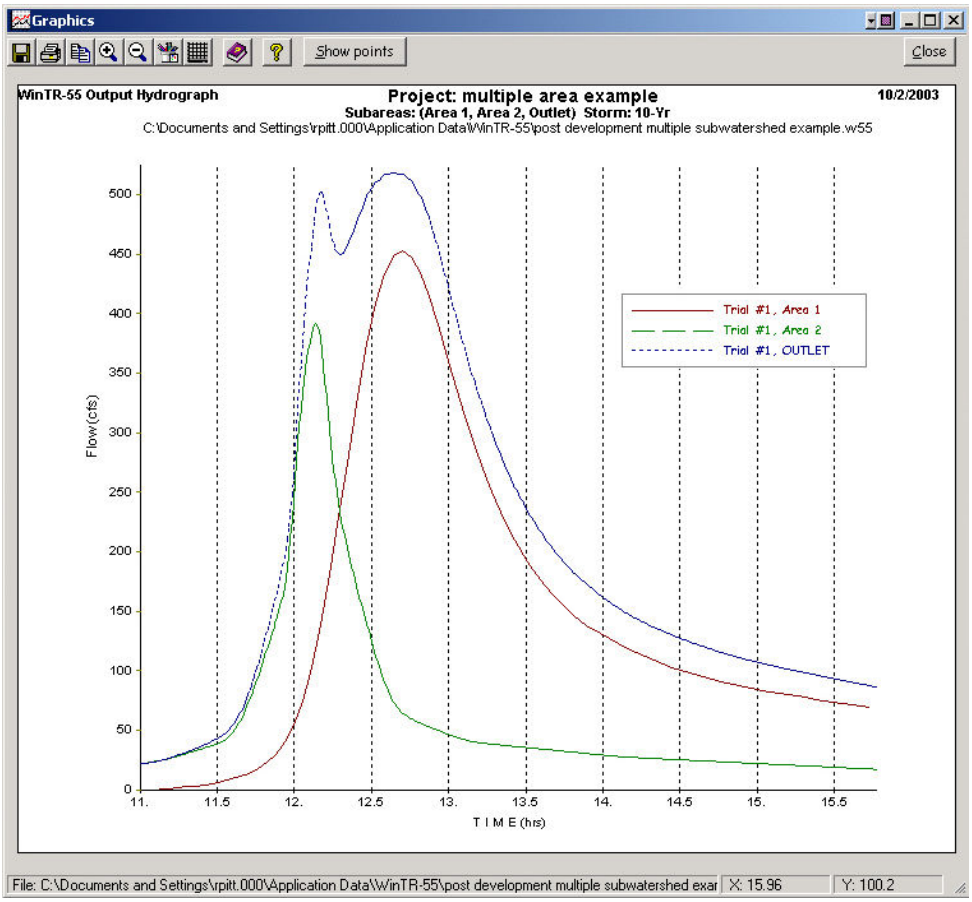
The developed 100 acre sub-area is comprised of 25 acres of commercial, 25 acres of town houses, and 50 acres of 1/3 acre lot residential areas (notice that the individual CNs range from 72 to 92, much broader than a difference of 5. Therefore, this area should be further sub-divided to separate the individual land uses, if possible. They were not in this simple example though).



The Tc factors also changed substantially for sub-area 2 after development:



When the same 10-year storm was evaluated, the following hydrograph was produced:



The upper sub-area (#1) had the same hydrograph characteristics, but the urbanized sub-area (#2) had a substantial increase in runoff volume and peak flow rate. The above composite hydrographs also show that the peaks are much more separated after development, with the hydrograph of the developed area to develop and recede much faster than the slower responding upper area sub-area. The developed area now has a peak flow rate of 391 cfs, but because the hydrograph components are more separated than for pre-developed conditions, the overall total peak hydrograph actually decreases slightly, to about 518 cfs.

Post Development with Pond

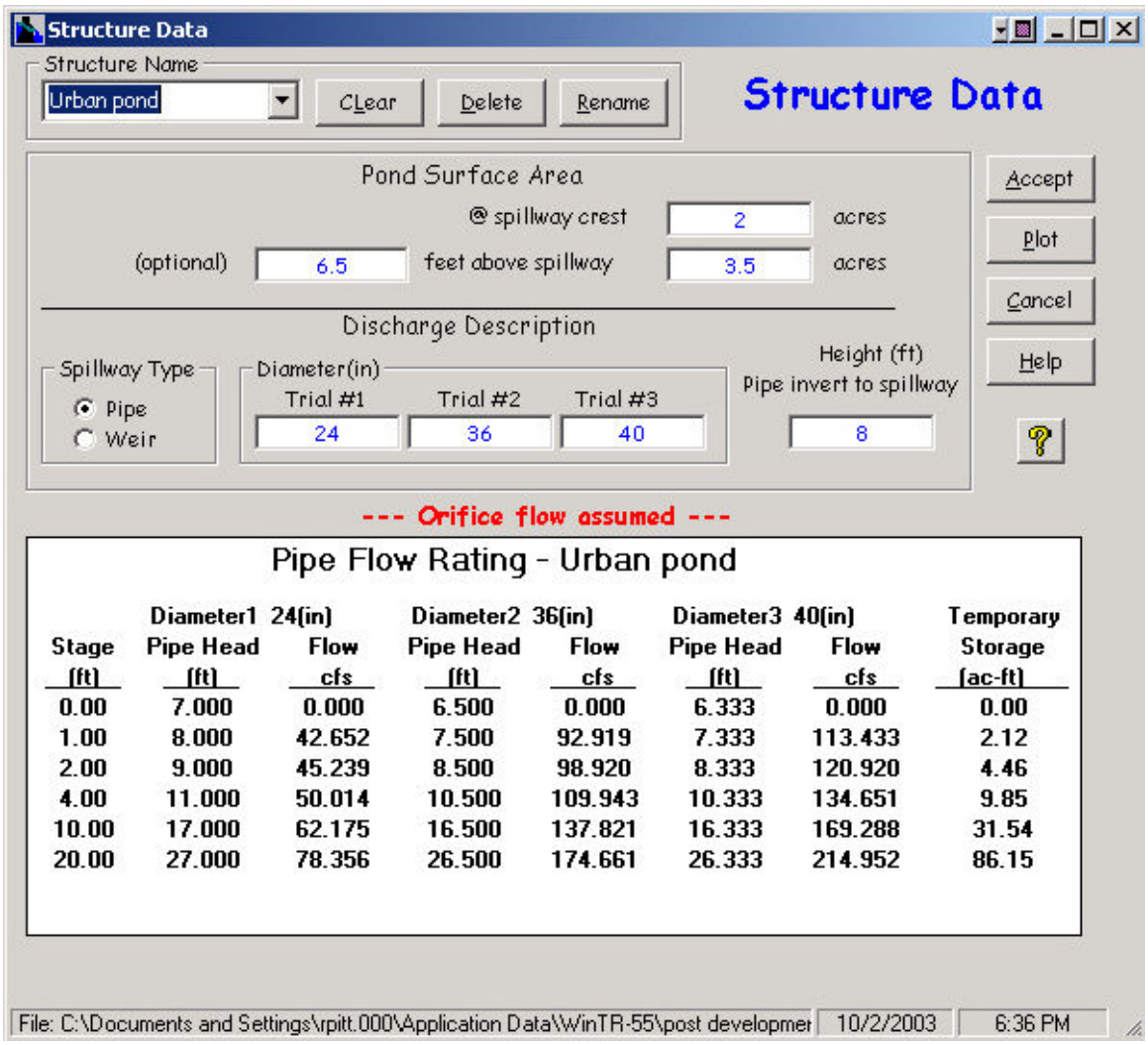
Even though the total area peak flows are actually less after development with no pond, the hypothetical site development standards still required a detention pond to reduce the post-development peak flow to the pre-development levels for the area undergoing development. The WinTR-55 manual suggests a simplified approach to size the needed pond based on the difference in the runoff volumes for pre and post-development conditions, and restricting the pond outlet device to the pre-development flow. Appendix 1 presents Chapter 6 of the older TR-55 that included a graphical method to select the pond storage and outfall structure characteristics.

The "WinTR-20 Reports" lists the runoff depth, in watershed inches. The pre-development runoff was reported to be 1.95 inches (over 100 acres). This corresponds to about 16.2 acre-feet. The post-development runoff depth was about 4.05 inches (also over the same 100 acres), corresponding to about 33.8 acre-feet. The difference (and "required" pond storage) is therefore 17.6 acre-feet. The maximum pond discharge was the pre-development peak flow (for the 10-year storm for this example) of 139 cfs.

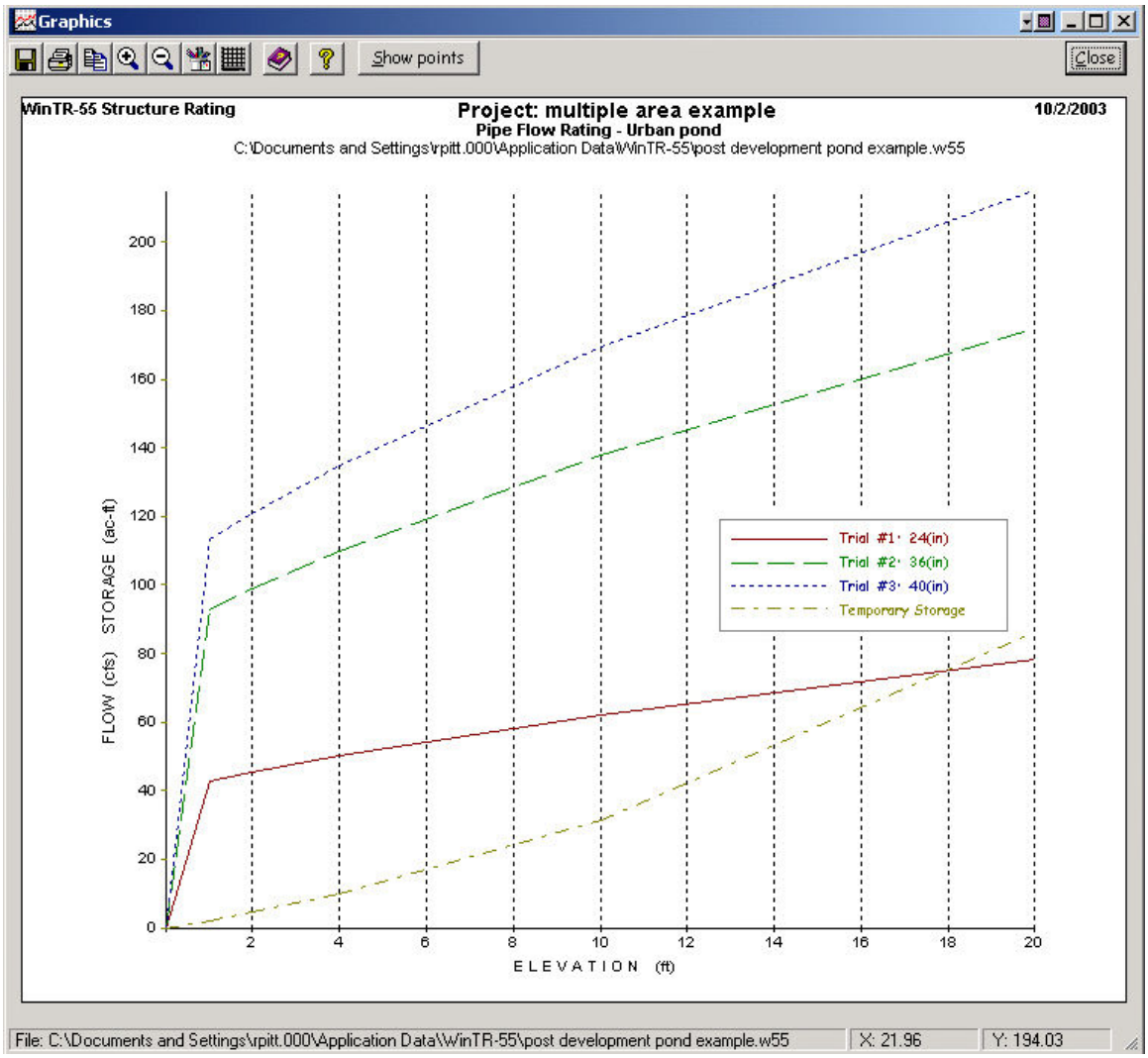
The pond size can then be crudely sized using these values. However, this was to be a multi-purpose pond, also providing water quality benefits. A rough guide for the pond surface area (the bottom of the storage layer) for water

quality benefits can be estimated to be about 3% of the watershed paved area, plus 0.5% of the watershed pervious area. The CN menu presented the watershed % imperiousness areas for each development category. The commercial area is assumed to be 85% impervious, the high density residential area to be 65% impervious, and the low density residential area to be 30% impervious. A simple calculation resulted in a pond bottom area (the actual surface of the permanent pool, which needs to be at least 3 feet deep), of 1.78 acres. A value of 2 acres will therefore be used. If this portion of the pond is 6.5 feet deep, and the top area is 3.5 acres, the pond side slopes would be about 7.3:1 (H:V), a reasonable value, to provide about 17.6 acre-feet of storage.

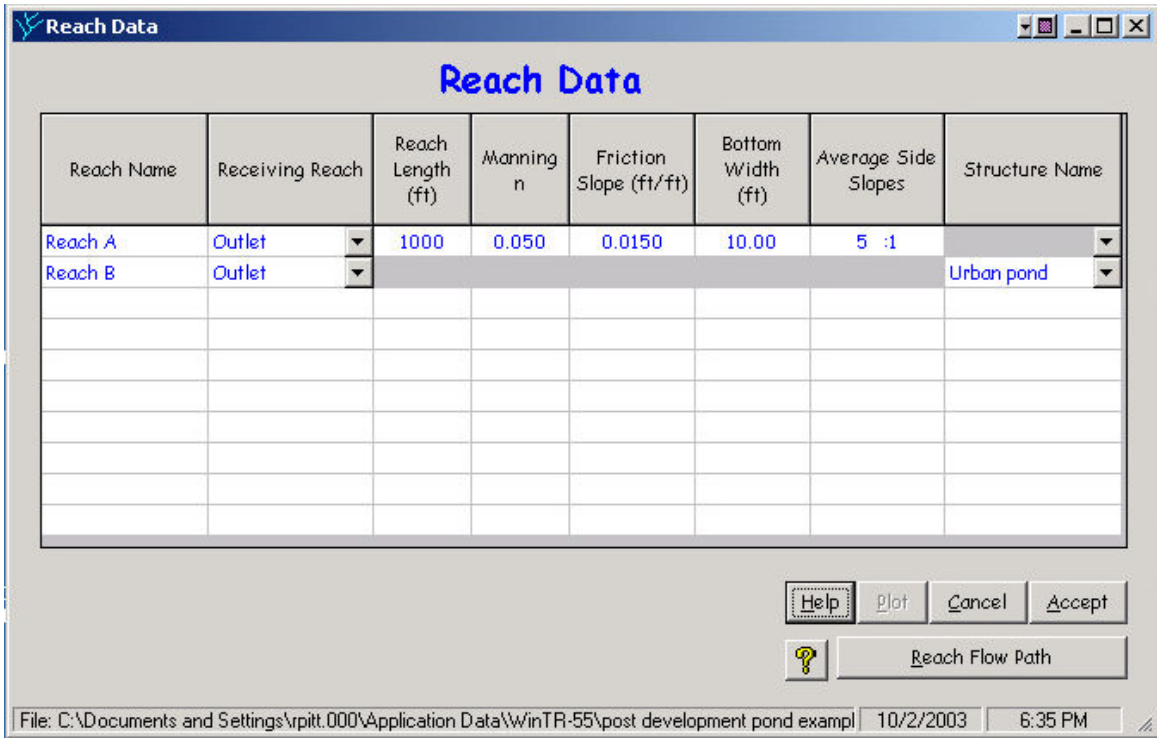
The first step was to describe the pond and to edit the post-development file to change Reach B from a channel to a pond. The following is the description of the pond "structure" using the "Structure Data" top menu bar option. The pond surface areas are described using the above calculated estimates. The area is 2 acres at the depth where the discharge begins, and is 3.5 acres in area 6.5 feet above this spillway elevation. WinTR-55 will assume a deeper pond as needed (above 6.5 feet) but will use this side slope. If the upper area was not entered (it is an optional value), the pond is assumed to then have vertical side slopes (not a good idea). The "Discharge Description" is based on the spillway type selected, either a pipe (using the pipe approach previously described), or a weir. If a weir is selected, it can be a broad-crested weir and the weir length entered. If a 0 value is entered for the weir length, the model will assume a 90^{o} V-notch weir. If a pipe spillway is selected (as in this example), the pipe diameter (in inches) is given, ranging from 6 to 60 inches. When a pipe is selected, the height from the invert of the discharge end of the pipe to the spillway elevation is also needed for the simplified equation. This height must be at least twice the diameter of the pipe. Up to three pipe diameters (or weir lengths) can be entered. The model will evaluate all three options, making the selection of the choice easier. As the dimensions are entered, the rating curves (flow vs. height) and storage below the elevations are displayed. This is a good indication of the correct spillway size, as the maximum discharge close to the desired pond depth can be observed. In this case, the 40 inch pipe has the desired discharge of 139 cfs at a stage slightly above 4 feet, and well under 10 feet. The 36 inch pipe option would need about 10 feet of stage (greater than planned), while the 24 inch pipe would require even more (more than 20 ft). Therefore, it is expected that the 3^{rd} pipe option, the 40 inch pipe would work best.



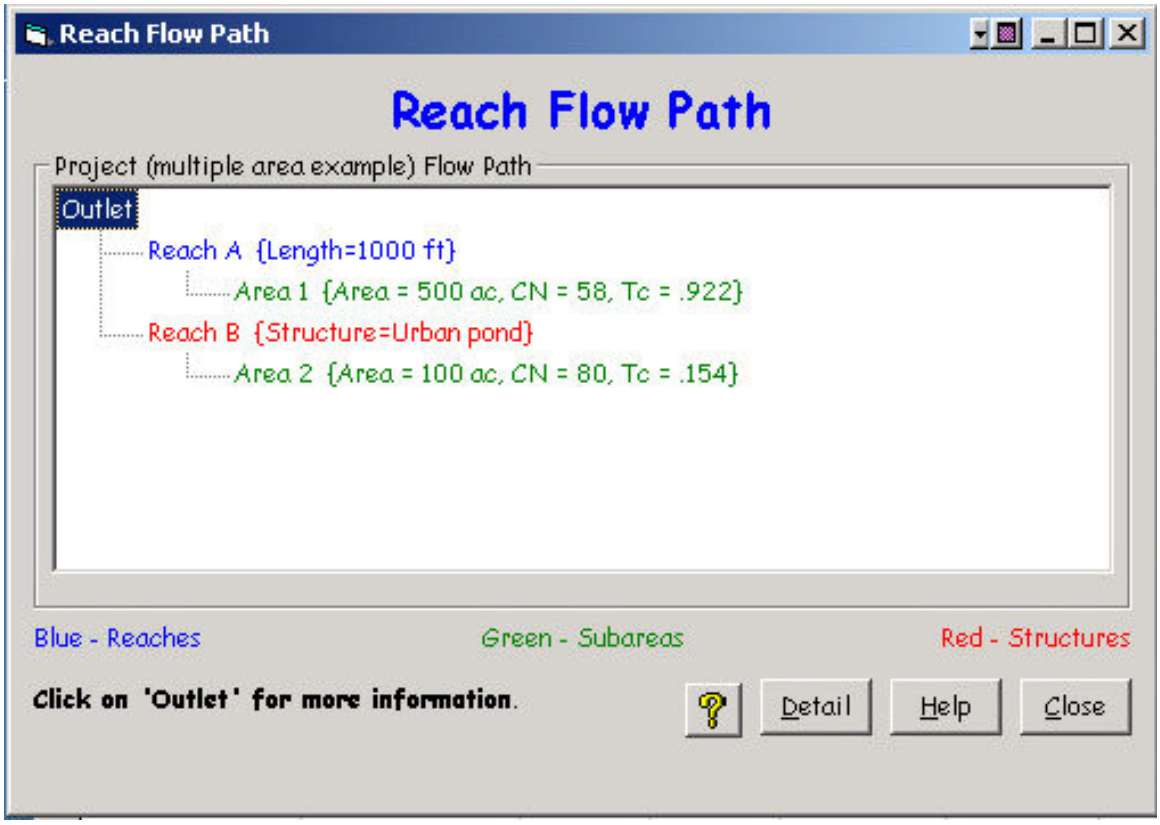
A rating curve can also be plotted for each outlet option if the "Plot" option is selected on the structure screen. This plot confirms that the 40 inch pipe discharge would require about 5 feet of the available pond stage.



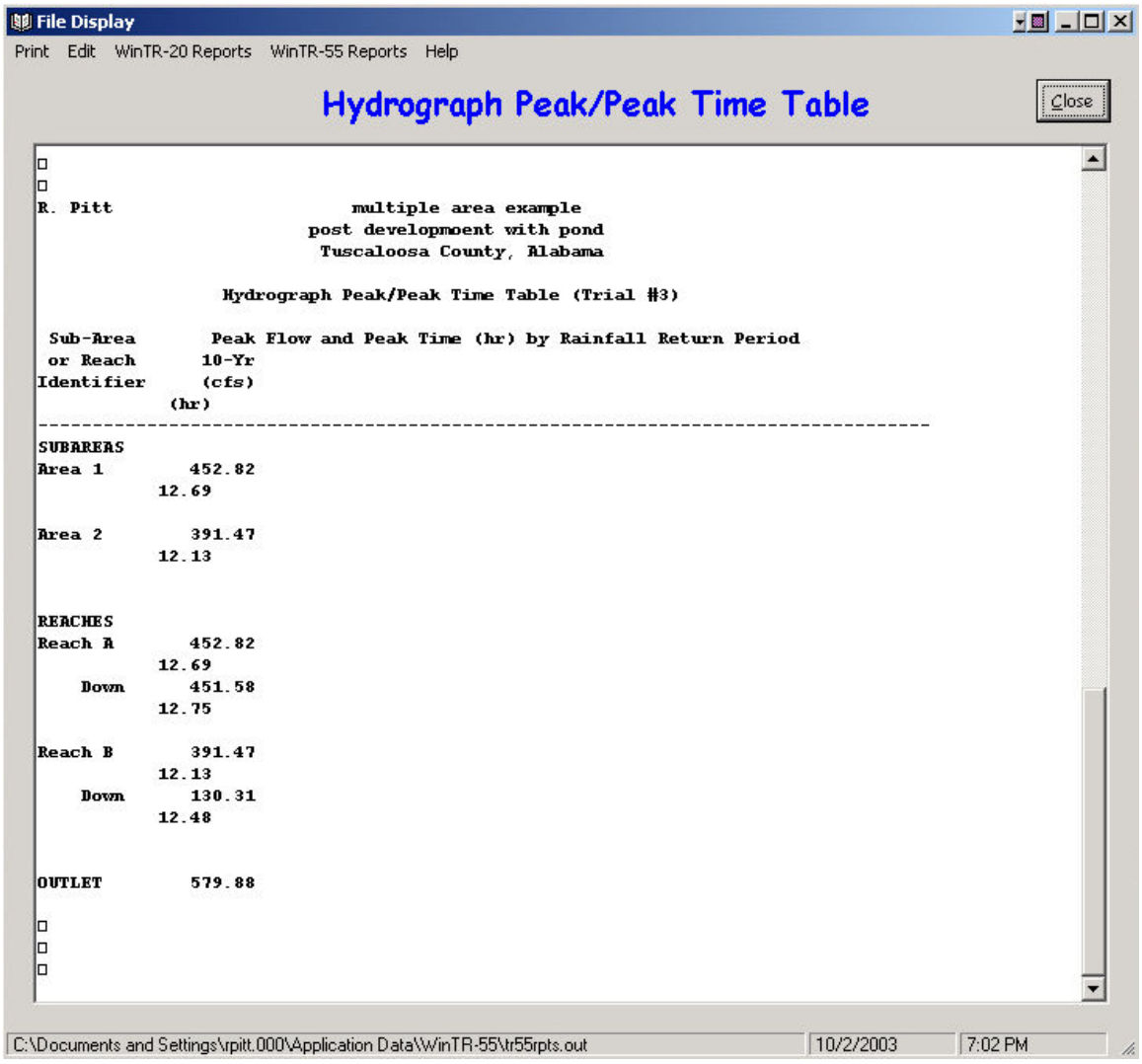
The reach is then modified to be a pond instead of a creek. The “Structure Name” drop-down menu in the appropriate cell is used to select the available pond name (available after the “accept” button on the pond menu is clicked). The creek data, if previously on the reach data menu row for the named reach that is now a pond, needs to be deleted.



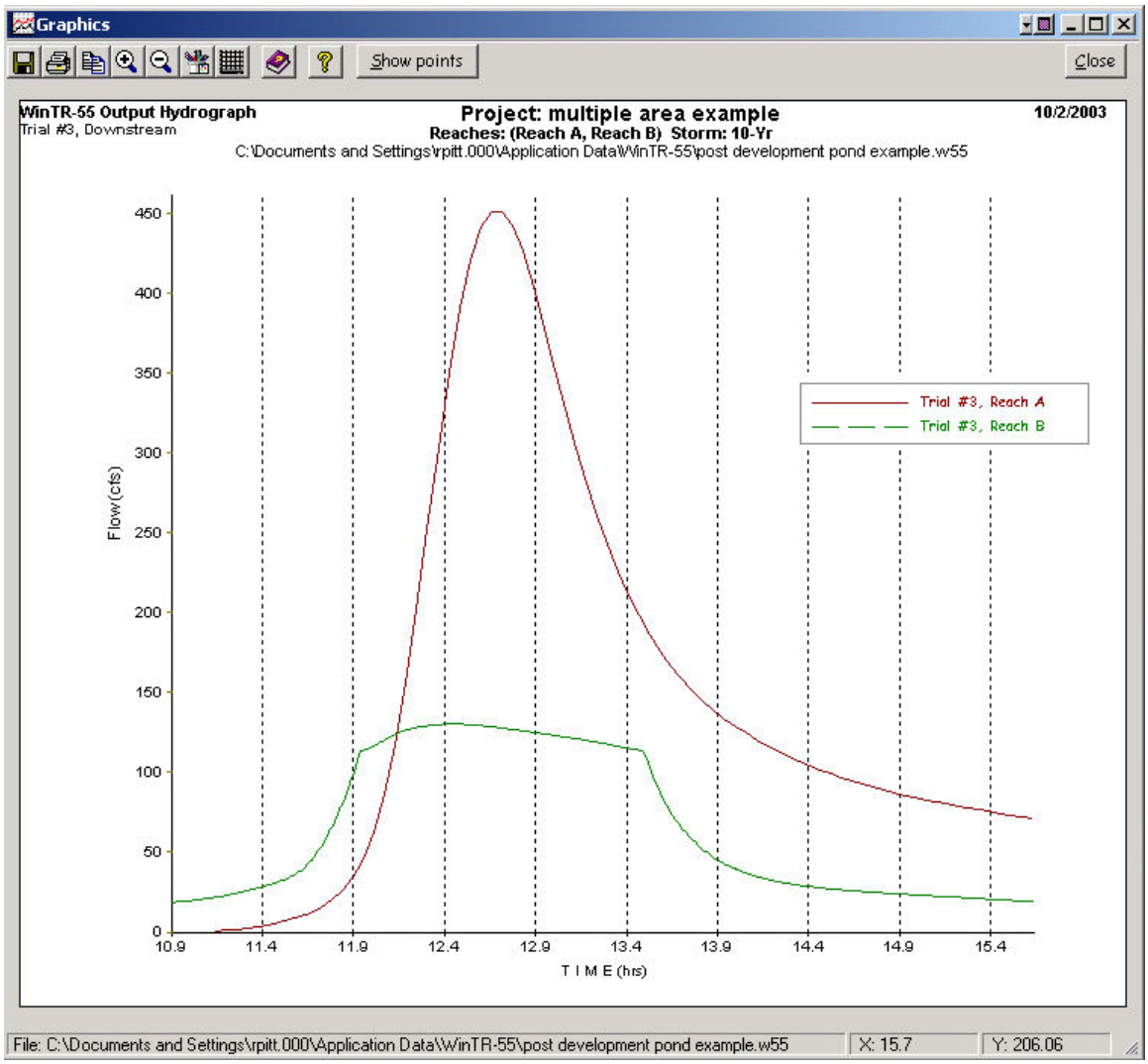
The "Reach Flow Path" screen (selected from the "Project Data" drop down menu) can also be selected to ensure that the model has the outfall, reaches and areas correctly connected:



Upon program execution, the data can be reviewed to verify if any of the spillway options were suitable. The following table shows that trial #3 (the 40 inch pipe) reduces the reach B influent flow (391 cfs) down to about 130 cfs, close enough to the desired maximum peak flow. Unfortunately, the outfall peak flow is shown to be about 580 cfs, substantially greater than the predevelopment peak flow of 521 cfs and the post development peak flow, with no pond, of 518 cfs.



The following plot of the reach hydrographs indicate how this occurred. The water from subarea 2 was delayed in the detention pond (Reach B) and was discharged so that its peak rate closely coincided in time with the undeveloped hydrograph from subarea 1 (Reach A), causing a larger peak flow than if the water was not detained.



This example illustrated how a detention pond can be evaluated for a developing area, how it can be designed for multiple objectives, and how these objectives may, or may not, be realized in a watershed. The simple application of detention pond standards may not always provide the desired downstream benefits. A basin-wide hydrologic analysis (the above example was a crude and simple example) is needed to ensure that ponds area sized and located correctly to provide the desired benefits. Obviously, the above example was a set-up to illustrate this issue. However, it would be relatively easy to modify the pond to still provide the desired water quality benefits, while not exasperating the flood control objective. A change in the pond spillway device to allow the pond to empty more rapidly would solve this problem. In most cases, detention ponds providing large amounts of storage for flood control should be located in upper reaches of watersheds to lessen these problems.

Summary

WinTR-55 is probably the simplest (and cheapest!) model that can be used to examine basin-wide hydraulic issues. It is relatively simple to use and is based on conventional drainage design procedures. Future improvements in the spillway options will make it more accurate. If more precise analyses are needed, TR-20, or more sophisticated models should be used. It must also be emphasized that WinTR-55 (and TR-20) are not suitable models for water quality evaluations. The curve number approach is not applicable for the moderate-sized events that are responsible

for the vast majority of pollutant discharges, continuous simulations for long periods are needed to understand the complex behavior of pollutant discharges under a wide range of environmental conditions, and particle routing (including scour from shallow and dry ponds) is needed to predict the level of pollutant control that may be achieved in detention ponds. However, multiple tools can be used together to better understand how multiple (and often times, conflicting) objectives can be met.

Important Internet Links

Alabama Rainfall Atlas:
http://bama.ua.edu/~rain/

WinTR-55 computer program (windows beta version):
http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-wintr55.html

TR-55 1986 documentation:
ftp://ftp.wcc.nrcs.usda.gov/downloads/hydrology_hydraulics/tr55/tr55.pdf

TR-20 computer program (new windows beta version):
http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-wintr20.html

National Engineering Handbook, Part 630 HYDROLOGY
http://www.wcc.nrcs.usda.gov/hydro/hydro-techref-neh-630.html

US Army Corps of Engineers, Hydrologic Management System User Guide (HEC HMS) (replacement for HEC-1):
http://www.hec.usace.army.mil/software/hec-hms/hechms-hechms.html

US Army Corps of Engineers, River Analysis System User Guide for water surface profile calculations (HEC RAS) (replacement for HEC-2):
http://www.hec.usace.army.mil/software/hec-ras/hecras-hecras.html

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NRCS. *TR-20 System: User Documentation*. United States Department of Agriculture, Natural Resources Conservation Service. 105 pp. 2002b (draft).

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Pitt, R., J. Lantrip, R. Harrison, C. Henry, and D. Hue. *Infiltration through Disturbed Urban Soils and Compost-Amended Soil Effects on Runoff Quality and Quantity*. U.S. Environmental Protection Agency, Water Supply and Water Resources Division, National Risk Management Research Laboratory. EPA 600/R-00/016. Cincinnati, Ohio. 231 pgs. December 1999.

Ponce, V.M., *Engineering Hydrology*, Prentice Hall, 640 pages. 1989.

SCS. *Urban Hydrology for Small Watersheds.* Technical Release 55, US Department of Agriculture, Soil Conservation Service. 91 pp. 1975.

SCS (now NRCS). *Urban Hydrology for Small Watersheds*. US Dept. of Agric., Soil Conservation Service. 156 pgs. 1986.

SCS. *Time of Concentration,* Hydrology Technical Note No. N4. United States Department of Agriculture, Soil Conservation Service, Northeast National Technical Center. 12 pp. 1986.

Thronson, R.E. *Comparative Costs of Erosion and Sediment Control, Construction Activities*. U.S. Environmental Protection Agency. EPA430/9-73-016. Washington, D.C. 1973.

Welle, P.I., Woodward, D. E., Fox Moody, H.*, A Dimensionless Unit Hydrograph for the Delmarva Peninsula*, Paper No. 80-2013, ASAE 1980 Summer Meeting, 18 pp. 1980.

Appendix 1. Chapter 6 from TR-55: Hydrology for Small Urban Watersheds

The attached material is chapter 6 from TR-55 (SCS 1986). This chapter has historically been used to estimate the storage volumes and to select the outlet devices (structures) needed to control pond discharges within certain limits. These methods have usually been assumed to result in storage volumes larger than necessary. In addition, the use of detention facilities to control post-development peak discharges to pre-development levels has not been found to significantly reduce the effects of urbanization on receiving waters, or to adequately control downstream flooding. The use of continuous simulation tools (such as SWMM) and the examination of the energy distribution of the pond discharges may be a more suitable method to determine the necessary flows to minimize problems. However, this method can still be used to size a pond to achieve a desired peak flow rate limit. The sized pond can then be examined with WinTR-55 to see how the pond interacts with flows from throughtout the watershed for a single "design" storm. Finally, a continuous simulation model needs to be used to examine pond performance over a broad range of conditions and to examine the energy profile of the discharged water. A continuous water quality pond model, such as WinDETPOND, also needs to be used to examine pollutant capture and effluent water quality conditions.

Chapter 6 Storage Volume for Detention Basins

As rural areas become urbanized, the resulting increases in peak discharges can adversely affect downstream flood plains. Increasingly, planners, developers, and the public want these downstream areas to be protected. Many local governments are adopting ordinances to control the type of development and its allowable impacts on the watershed. One of the most common controls requires that postdevelopment discharges do not exceed present-condition discharges for one or more storm frequencies at specified points along a channel.

This chapter discusses ways to manage peak discharges by delaying runoff. It also presents a procedure for estimating the storage capacity required to maintain the peaks within a specified level.

Efforts to reduce the effects of increased runoff from urban areas have been innovative and diverse. Many methods have been used effectively, such as infiltration trenches, porous pavement, rooftop storage, and cisterns. But these solutions can be expensive or require site conditions that cannot be provided.

The detention basin is the most widely used measure for controlling peak discharge. It is generally the least expensive and most reliable of the measures that have been considered. It can be designed to fit a wide variety of sites and can accommodate multiple-outlet spillways to meet requirements for multifrequency control of outflow. Measures other than a detention basin may be preferred in some locations; their omission here is not intended to discourage their use. Any device selected, however, should be assessed as to its function, maintenance needs, and impart.

Estimating the effect of storage

When a detention basin is installed, hydrologic routing procedures can be used to estimate the effect on hydrographs. Both the TR-20 (SCS 1983) and DAMS2 (SCS 1982) computer programs provide accurate methods of analysis. Programmable calculator and computer programs are available for routing hydrographs through dams.

This chapter contains a manual method for quick estimates if the effects of temporary detention on peak discharges. The method is based on average storage and routing effects for many structures.

Figure 6-1 relates two ratios: peak outflow to peak inflow discharge (q_o/q_i) and storage volume runoff volume (V_s/V_r) for all rainfall distributions.

The relationships in figure 6-1 were determined on the basis of single stage outflow devices. Some were controlled by pipe flow, others by weir flow. Verification runs were made using multiple stage outflow devices, and the variance was similar to that in the base data. The method can therefore be used for both single- and multiple-stage outflow devices. The only constraints are that (1) each stage requires a design storm and a computation of the storage required for it and (2) the discharge if the upper stage(s) includes the discharge of the lower stage(s).

The brevity of the procedure allows the planner to examine many combinations of detention basins. When combined with the Tabular Hydrograph method, the procedure's usefulness is increased. Its principal use is to develop preliminary indications of storage adequacy and to allocate control to a group of detention basins. It is also adequate, however, for final design of small detention basins.

Input requirements and procedures

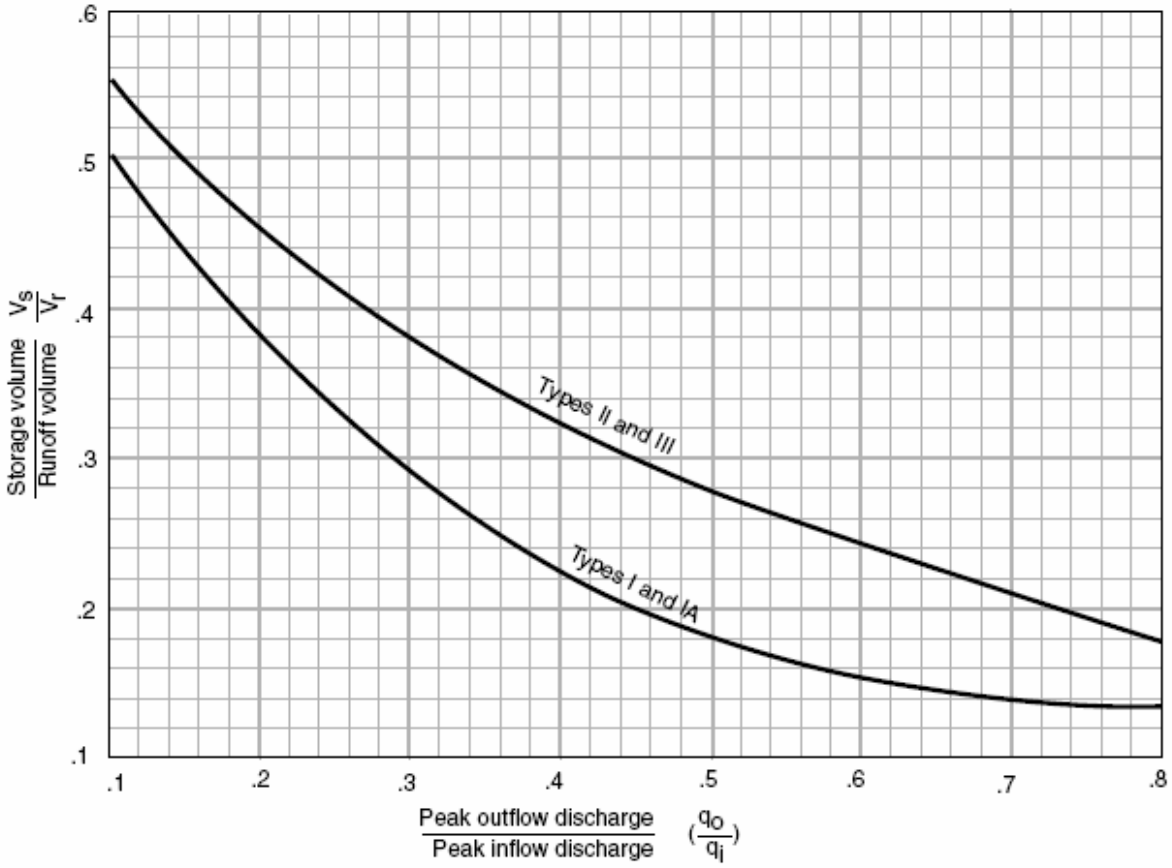
Use figure 6-1 estimate storage volume (V_s) required or peak outflow discharge (q_o). The most frequent application is to estimate V_s, for which the required inputs are runoff volume (V_r), q_o, and peak inflow discharge (q_i). To estimate q_o, the required inputs are V_r, V_s, and q_i.

Estimating V_s

Use worksheet 6a to estimate V_s, storage volume required, by the following procedure.

1. Determine q_o. Many factors may dictate the selection of peak outflow discharge. The most common is to limit downstream discharges to a desired level, such as predevelopment discharge. Another factor may be that the outflow device has already been selected.
2. Estimate q_i by procedures in chapters 4 or 5. Do not use peak discharges developed by other procedure. When using the Tabular Hydrograph method to estimate q_i for a subarea, only use peak discharge associated with $T_t = 0$.

Figure 6-1 Approximate detention basin routing for rainfall types I, IA, II, and III



3. Compute q_o/q_i and determine V_s/V_r from figure 6-1.
4. Q (in inches) was determined when computing q_i in step 2, but now it must be converted to the units in which V_s is to be expressed—most likely, acre-feet or cubic feet. The most common conversion of Q to V_r is expressed in acre-feet:

$$V_r = 53.33Q(A_m) \qquad \text{[eq. 6-1]}$$

where
V_r = runoff volume (acre-ft)
Q = runoff (in)
A_m = drainage area (mi^2), and
53.33 = conversion factor from in-mi^2 to acre-ft.

5. Use the results of steps 3 to 4 to compute V_s:

$$V_S = V_r\left(\frac{V_s}{V_r}\right) \qquad \text{[eq. 6-2]}$$

where
V_s = storage volume required (acre-ft).

6. The stage in the detention basin corresponding to V_s must be equal to the stage used to generate q_o. In most situations a minor modification of the outflow device can be made. If the device has been preselected. repeat the calculations with a modified q_o value.

Estimating q_o

Use worksheet 6b to estimate q_o, required peak outflow discharge, by the following procedure.

1. Determine V_s. If the maximum stage in the detention basin is constrained, set V_s by the maximum permissible stage.
2. Compute Q (in inches) by the procedures in chapter 2, and convert it to the same units as V_s (see step 4 in "estimating V_s").
3. Compute V_s/V_r and determine q_o/q_i from figure 6-1.
4. Estimate q_i by the procedures in chapters 4 or 5. Do not use discharges developed by any other method. When using Tabular method to estimate q_i for a subarea, use only the peak discharge associated with $T_t = 0$.
5. From steps 3 to 4, compute q_o:

$$q_o = q_i\left(\frac{q_o}{q_i}\right) \qquad \text{[eq. 6-3]}$$

6. Proportion the outflow device so that the stage at q_o is equal to the stage corresponding to V_s. If q_o cannot be calibrated except in discrete steps (i.e., pipe sizes), repeat the procedure until the stages for q_o and V_s are approximately equal.

Limitations

- This routing method is less accurate as the q_o/q_i ratio approaches the limits shown in figure 6-1. The curves in figure 6-1 depend on the relationship between available storage, outflow device, inflow volume, and shape of the inflow hydrograph. When storage volume (V_s) required is small, the shape of the outflow hydrograph is sensitive to the rate of the inflow hydrograph. Conversely, when V_s is large, the inflow hydrograph shape has little effect on the outflow hydrograph. In such instances, the outflow hydrograph is controlled by the hydraulics of the outflow device and the procedure therefore yields consistent results. When the peak outflow discharge (q_o) approaches the peak flow discharge (q_i) parameters that affect the rate of rise of a hydrograph, such as rainfall volume, curve number, and time of concentration, become especially significant.
- The procedure should not be used to perform final design if an error in storage of 25 percent cannot be tolerated. Figure 6-1 is biased to prevent undersizing of outflow devices, but it may significantly overestimate the required storage capacity. More detailed hydrograph development and routing will often pay for itself through reduced construction costs.

Examples

Four examples illustrate the use of figure 6-1. Examples 6-1 through 6-4, respectively, show estimation of V_s of a two-stage structure, estimation of q_o, and use the Tabular Hydrograph method.

Example 6-1: Estimating V_s, single-stage structure

A development is being planned in a 75-acre (0.1170 mi^2) watershed that outlets into an existing concrete-lined channel designed for present conditions. If the channel capacity is exceeded, damages will be substantial. The watershed is in the type II storm distribution region. The present channel capacity, 180 cfs, was established by computing discharge for the 25-year-frequency storm by the Graphical Peak Discharge method (chapter 4).

The developed-condition peak discharge (q_i) computed by the same method is 360 cfs, and runoff (Q) is 3.4 inches. Since outflow must be held to 180 cfs, a detention basin having that maximum outflow discharge (q_o) will be built at the watershed outlet.

How much storage (V_s) will be required to meet the maximum outflow discharge (q_o) of 180 cfs, and what will be the approximate dimensions of a rectangular weir outflow structure? Figure 6-2 shows how worksheet 6a is used to estimate required storage (V_s = 5.9 acre-ft) and maximum stage (E_{max} = 105.7 ft).

The rectangular weir was chosen for its simplicity; however, several types of outlets can meet the outflow device proportion requirement. Most hydraulic references, along with considerable research data that are available, provide more guidance on variations of outlet devices that can be summarized here.

An outlet device should be proportioned to meet specific objectives. A single-stage device was specified in this example because only one storm was considered. A weir is suitable here because of the low head. The weir crest elevation is 100.00 ft.

Using V_s = 5.9 acre-ft (figure 6-2, step 9) and the elevation-storage curve, the maximum stage (E_{max}) is 105.7 ft.

The rectangular weir equation is

$$q_o = 3.2 L_w H_w^{1.5} \qquad \text{[eq. 6-4]}$$

where

q_o = peak outflow discharge (cfs)
L_w = weir crest length (ft)
H_w = head over weir crest (ft)

H_w and q_o are computed as follows:

$H_w = E_{max}$ – weir crest elevation
= 105.7 - 100.0 = 5.7 ft.

Since q_o is known to be 180 cfs, solving equation 6-4 for L_w yields

$$L_w = \frac{q_o}{3.2 H_w^{1.5}} = \frac{180}{3.2(5.7)^{1.5}} = 4.1 \text{ ft} \qquad \text{[eq. 6-5]}$$

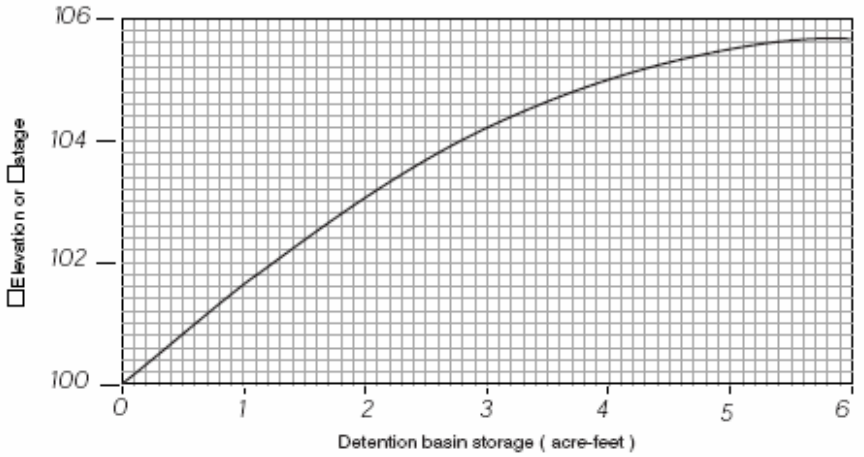
In summary, the outlet structure is a rectangular weir with crest length of 4.1 ft, H_w = 5.7 ft, and q_o = 180 cfs corresponding to a V_s = 5.9 acre-ft.

Figure 6-2 Worksheet 6a for example 6-1

Worksheet 6a: Detention basin storage, peak outflow discharge (q_o) known

Project	Robbinsville	By	SWR	Date	11/5/85
Location	Dyer County, Tennessee	Checked	RGC	Date	11/8/85

Check one: ☐ Present ☒ Developed — Single stage structure



1. Data:
   Drainage area ..... A_m = 0.117 mi²
   Rainfall distribution type (I, IA, II, III) = II

	1st Stage	2nd Stage
2. Frequency ..... yr	25	
3. Peak inflow discharge q_i .....cfs (from worksheet 4 or 5b)	360	
4. Peak outflow discharge q_u ..... cfs	180	
5. Compute $\frac{q_o}{q_i}$	0.50	
6. $\frac{V_s}{V_r}$ ..... (Use $\frac{q_o}{q_i}$ with figure 6-1)	0.28	
7. Runoff, Q ..... in (From worksheet 2)	3.4	
8. Runoff volume V_r ..... ac-ft ($V_r = QA_m$ 53.33)	21.2	
9. Storage volume, V_s ..... ac-ft ($V_s = V_r (\frac{V_s}{V_r})$)	5.9	
10. Maximum storage E_{max} (from plot)	105.7	

1/ 2nd stage q_o includes 1st stage q_o.

Example 6-2 Estimating V_s, Two-stage structure

In addition to the requirements for a 25-year peak outflow discharge of 180 cfs stated in example 6-1, a decision was made to limit the 2-year outflow discharge to 50 cfs because of potential damages to agricultural property below the lined channel. by the method in chapter 4, the estimated 2-year peak discharge for developed conditions will be 91 cfs and runoff (Q) will be 1.5 inches.

Again, a rectangular concrete weir outlaw device was selected; the device could have been another type, but it is important to remember that the flows through the first stage are part of the total discharge of the higher stage.

Figure 6-3 shows how worksheet 6a is used to compute the V_s of 2.4 acre-ft and E_{max} of 103.6 for the stage. E_{max} of 103.6 is the weir crest elevation for the second stage.

Equation 6-5 is again used to compute L_w for the first stage. The weir crest elevation for the first stage is 100.00 ft and q_o = 50 cfs. The first-stage computations for H_w and L_w are

$$H_w = E_{max} \text{ - weir crest elevation}$$
$$= 103.6 - 100.0 = 3.6 \text{ ft;}$$

and, from equation 6-5,

$$L_w = \frac{50}{3.2(3.6)^{1.5}}$$
$$= 2.3 \text{ ft}$$

The second stage is then proportioned to discharge the correct amount at 105.7 feet (fig. 6-2, step 10). Compute the discharge through the first stage for elevation 105.7 feet using

L_w = 2.3 ft (first stage)
and
$H_w = 105.7 - 100.0 = 5.7$ ft

By substituting these values in equation 6-4, discharge (q_o) through the first stage at 105.7 feet is calculated:

$$q_o = 3.2(2.3)(5.7)^{1.5}$$
$$= 100 \text{ ft}^3/\text{s}$$

Now compute the required weir crest length (L_w) for the second stage, using equation 6-5. Since the second stage crest elevation is 103.6 feet,

$$H_w = 105.7 - 103.6$$
$$= 2.1 \text{ ft}$$

and, since q_o for the second stage equals the total discharge from example 6-1 minus discharge through the first stage,

$$q_o = 180 - 100$$
$$= 80 \text{ ft}^3/\text{s}$$

Finally, substituting these H_w and q_o values in equation 6-5 results in

$$L_w = \frac{80}{3.2(2.1)^{1.5}}$$
$$= 8.2 \text{ ft}$$

In summary, the outlet structure is a two-stage rectangular weir with first stage crest length of 2.3 feet at elevation 100.0, and second stage crest length of 8.2 feet at elevation 103.6 feet.

The weir equation used is probably less accurate for the two-stage example than for the single-stage example. The actual second-stage discharge will be slightly more that the one computed. but a discussion of hydraulics of outflow devices is outside the scope of this technical release. Example 6-2 is presented only to illustrate the interrelationship of outflow discharges and storage volume and to show how to develop preliminary estimates of storage requirements for two-stage outlet structures.

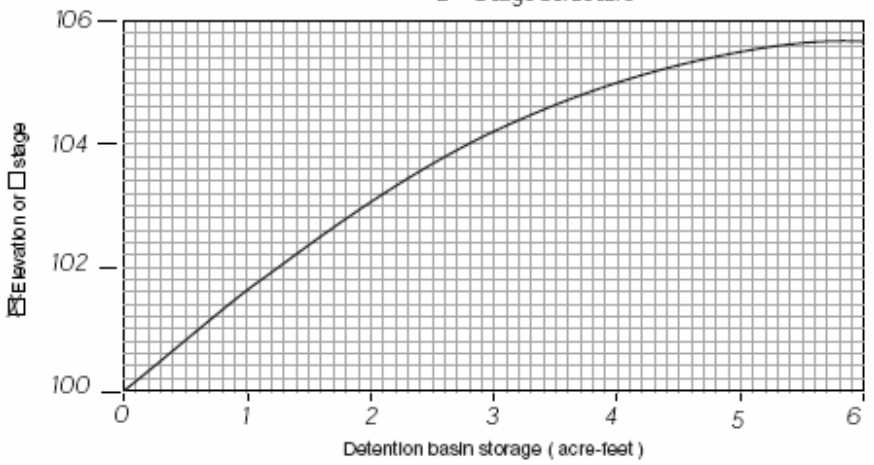
Figure 6-3 Worksheet 6a for example 6-2

Worksheet 6a: Detention basin storage, peak outflow discharge (q_o) known

Project	Robbinsville	By	SWR	Date	11/6/85
Location	Dyer County, Tennessee	Checked	RGC	Date	11/9/85

Check one: ☐ Present ☒ Developed

2 - Stage structure



1. Data:
   Drainage area ..... A_m = 0.117 mi²
   Rainfall distribution type (I, IA, II, III) = II

	1st Stage	2nd Stage
2. Frequency ..... yr	2	25
3. Peak inflow discharge qi .....cfs (from worksheet 4 or 5b)	91	360
4. Peak outflow discharge q_u ..... cfs	50	180
5. Compute $\frac{q_o}{q_i}$	0.55	0.50
6. $\frac{V_s}{V_r}$ (Use $\frac{q_o}{q_i}$ with figure 6-1)	0.26	0.28
7. Runoff, Q ..... in (From worksheet 2)	1.5	3.4
8. Runoff volume V_r ..... ac-ft (V_r = QA_m 53.33)	9.4	21.2
9. Storage volume, V_s ..... ac-ft ($V_s = V_r (\frac{V_s}{V_r})$)	2.4	5.9
10. Maximum storage E_{max} (from plot)	103.6	105.7

[1] 2nd stage q_o includes 1st stage q_o

Example 6-3 Estimating q_o

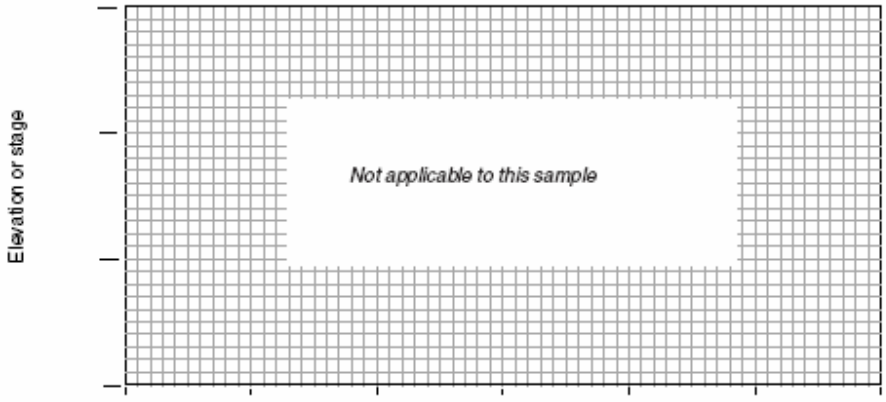
A development is being planned for a 10-acre watershed (0.0156 mi^2). A county ordinance requires that the developed-condition outflow from the watershed for 24-hr, 100-year frequency storm does not exceed the outflow for present conditions. The peak discharge from the watershed for present conditions, 35 cfs, is calculated from procedures in chapter 4. For developed conditions, runoff (Q) is 5.4 inches, peak discharge from the watershed is 42 cfs from procedures in chapter 4, and rainfall distribution is type II. What will be the peak outflow discharge (q_o) from a detention basin that is located at the outlet and has maximum allowable storage volume (V_s) of 35,000 ft^3 and peak inflow discharge (q_i) of 42 cfs? Figure 6-4 shows how worksheet 6b is used to estimate qo as 33 cfs, which is within the 35 cfs limit. An outflow device will be selected to discharge 33 cfs at a stage corresponding to a V_s of 35,000 ft^3.

Figure 6-4 Worksheet 6b for example 6-3

Worksheet 6b: Detention basin storage, storage volume (V_s) known

Project	Woods Acres	By	SWR	Date	11/8/85
Location	Dyer county, Tennessee	Checked	RGC	Date	11/11/85

Check one: ☐ Present ☒ Developed



1. Data:
   Drainage area ..... A_m = 0.0156 mi²
   Rainfall distribution type (I, IA, II, III) = II

	1st Stage	2nd Stage
2. Frequency ..... yr	100	
3. Storage volume V_s ..... ac-ft	0.8	
4. Runoff, Q ..... in (from worksheet 2)	5.4	
5. Runoff volume ..... ac-ft ($V_r = QA_m$ 53.33)	4.5	

6. Compute $\frac{V_s}{V_r}$ 0.18

7. $\frac{q_o}{q_i}$ ..... in 0.78
   (Use $\frac{V_s}{V_r}$ with figure 6-1)

8. Peak inflow discharge q_i ..... in 42
   (From worksheet 4 or 5b)

9. Peak outflow discharge q_o ..... cfs 33 1/
   ($q_o = q_i (\frac{q_o}{q_i})$)

10. Maximum storage E_{max} (from plot) N/A

1/ 2nd stage q_o includes 1st stage q_o

Example 6-4 Estimating V_S, Tabular Hydrograph method

This example builds on examples 5-1 and 5-2. If peak outflow discharge from subarea 7 must not exceed the discharge for present conditions, what will be the storage volume (V_s) required in a detention basin at the outlet of subarea 6?

First, compute the outflow hydrograph without subarea 6 as shown in the table below, which presents developed-condition discharges for example 5-2. (The information in the table is from figure 5-4.)

	Discharge (cfs) at time (hr)								
Sub area	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0
	cfs								
1	7	9	11	16	24	40	78	122	155
2	7	9	12	20	33	55	96	132	132
3	14	29	58	89	106	102	74	46	25
4	19	32	63	114	169	207	193	143	83
5	117	167	205	214	202	175	132	99	70
6 omitted	—	—	—	—	—	—	—		
7	244	167	119	90	72	59	48	40	34
Total without subarea 6	408	413	468	543	606	638	621	582	499

After computing the outflow hydrograph, determine the maximum permissible outflow discharge from subarea 6. The present condition peak discharge at the outlet of subarea 7 is 750 cfs at 14.3 hr (figure 5-2), and the developed condition peak discharge at the outlet of subarea 7 minus subarea 6 is 638 cfs (table above). The difference between these two discharges, 82 cfs, is the maximum outflow discharge (q_o) for the detention basin.

Next, determine the peak discharge for subarea 6 for developed conditions by substituting values in equation 5-1:

$$q = q_t A_m Q \qquad \text{[eq 5-1]}$$

From exhibit 5-II, the largest q_t value is 357 csm/in (exhibit 5-II, sheet 7: T_c = 1.0 hr, T_t = 0, and I_a / P = 0.10 at 12.8 hr). From figure 5-4, $A_m Q$ for subarea 6 is 1.31. Therefore,

$$q = (357)\ (1.31) = 468 \text{ cfs}$$

This q value is, of course, the same as the peak inflow discharge (q_i) into the detention basin.

Finally, use worksheet 6a (fig. 6-5) to compute Vs as 33.2 acre-feet.

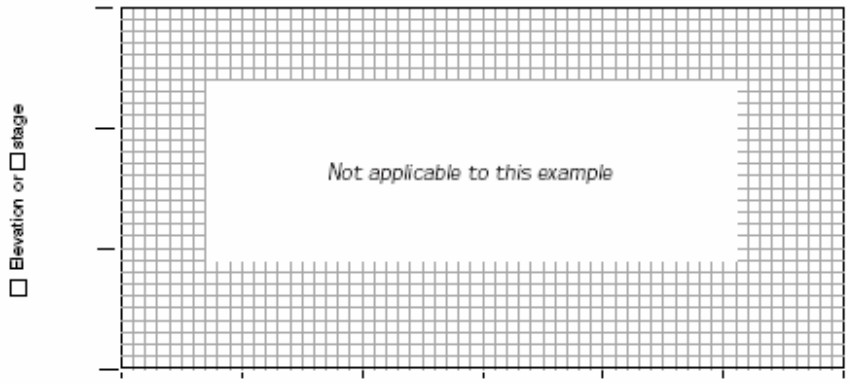
The required storage volume of 33.2 acre-feet is the basis for determining the required stage in the detention basin. This stage is a guide proportioning a spillway that will discharge 82 cfs or less at that storage. The timing or routing effect is not considered because the outflow hydrograph will discharge at near q_o for a significant period.

Figure 6-5 Worksheet 6a for example 6-4

Worksheet 6a: Detention basin storage, peak outflow discharge (q_o) known

Project	Fallswood	By	SWR	Date	11/8/85
Location	Dyer county, Tennessee	Checked	RGC	Date	11/10/85

Check one: ☐ Present ☒ Developed



1. Data:
   Drainage area ..... A_m = 0.40 / 11 mi²
   Rainfall distribution type (I, IA, II, III)

	1st Stage	2nd Stage
2. Frequency ..... yr	25	
3. Peak inflow discharge q_i ..... cfs (from worksheet 4 or 5b)	468	
4. Peak outflow discharge q_u ..... cfs 1/	82	
5. Compute $\frac{q_o}{q_i}$	0.175	
6. $\frac{V_s}{V_r}$ ..... (Use $\frac{q_o}{q_i}$ with figure 6-1)	0.475	
7. Runoff, Q ..... in (From worksheet 2)	3.28	
8. Runoff volume V_r ..... ac-ft ($V_r = QA_m$ 53.33)	69.9	
9. Storage volume, V_s ..... ac-ft ($V_s = V_r (\frac{V_s}{V_r})$)	33.2	
10. Maximum storage E_{max} (from plot)	N/A	

1/ 2nd stage q_o includes 1st stage q_o.