PDH Course H131

Regional Detention Pond Design
An Eclectic Approach

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2007

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Woodmansee Park

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I. <u>Background and Purpose</u>

The purpose of this course is to present some ideas and suggestions for designing regional detention basins. Woodmansee Park , located in the City of Salem, Oregon, is taken as an example for you to use in the course. You may assume that the City's current Storm Drain Design Standards provide general guidance but do not offer specific methods for the design such regional facilities.

In this course you will attempt to do that.

Woodmansee Park, located in southeast Salem, Oregon offers some unique advantages as a site for regional stormwater detention. Because the Park is already publically owned with borders on both sides of West Pringle Creek no land acqusition costs would be incurred. The park topography is already naturally "bowl shaped" and the Park's upstream location makes it ideal for protection of downstream properties from Idylwood Drive northward. These two advantages, and others are discussed at the conclusion of these calculations, may persuade you that the Park is a prime candidate for a regional detention basin.

This course is based on a belief that stormwater detention, stream restoration, groundwater recharge, multi purpose public land use and similar community goals are most fairly and efficiently achieved through a system of carefully planned, designed, and publically owned and operated, regional detention basins.

The following detailed calculations for preliminarily sizing the detention pond are mainly in the form of Excel spreadsheets. Some calculations were done using the computer programs ***SMADA , Hydraflow 2007, and NFF*** each of which are described under the "references" section below. You would need copies of these programs to completely check the results but they are not necessary to your understanding of the design approach.

Because many of the summaries of results are in the form of Excel spreadsheets you will need that program to view and print them for easy reference.

<u>II Overview of the Seven Step Process</u>

1. ***Gather basin and precipitation data for the watershed***

2. ***Select an initial design storm; duration, probability and distribution***

3. ***Calculate the pre and post development runoff hydrographs and the ratio of the two peak flows***

4. ***Estimate required storage volume based on TR-55***

5. ***Postulate a detention pond size and volume***

6. ***Route a series of storms through the hypothetical pond***

7. ***Complete final design details and calibrate the hydrologic model using known rainfall and streamflow records***

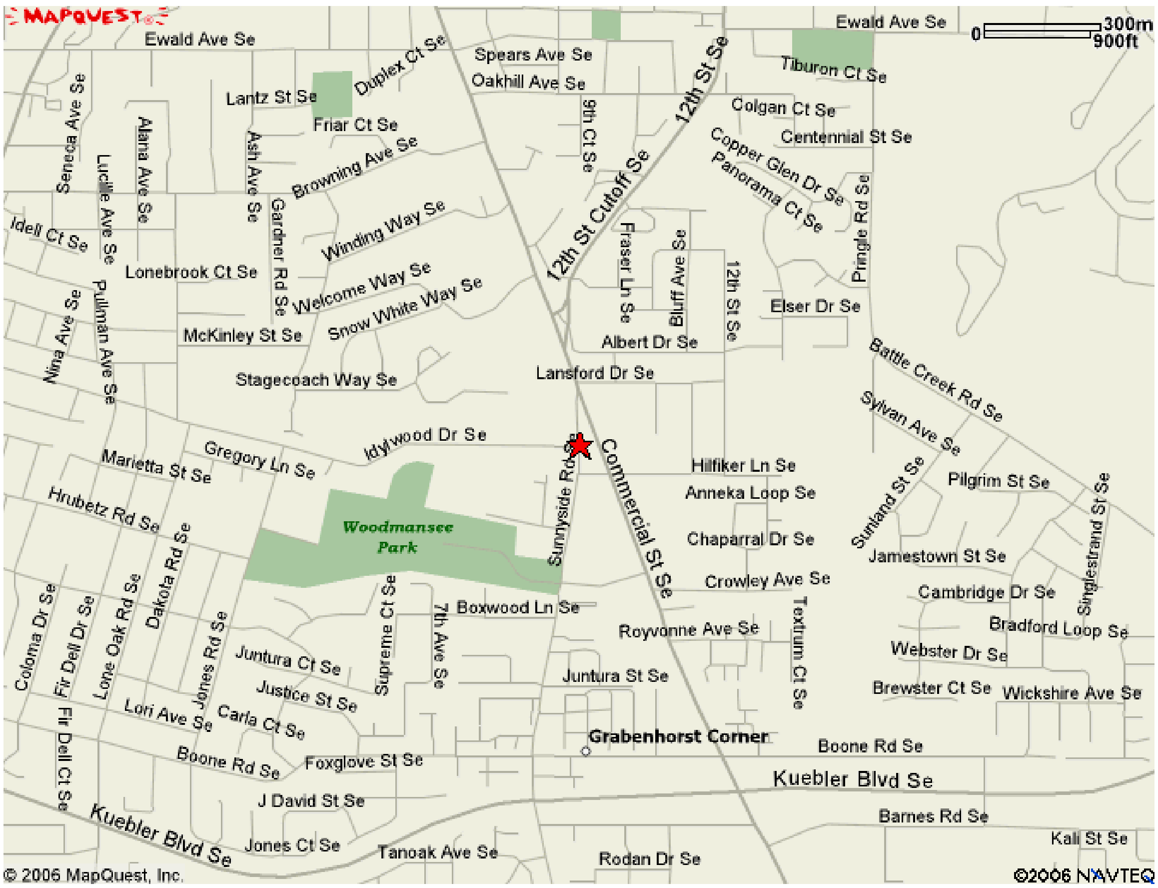


Figure 1-1 VICINITY MAP

II. Basin Description

Figure 1-1 shows the general location of Woodmansee Park . To begin our calculations, the drainage basin shown in **Figure 2-1** has been delineated. Covering 1,015.8 Acres (1.59 square miles) it drains the upper reaches of West Pringle Creek and drains eastward and northward to a culvert crossing Idylwood Drive just west of Sunnyside Road. Other physical basin characteristics are noted in the detailed calculations.

The basin is currently (2006) almost fully developed with a mixture of land uses ranging from commercial , open spaces, single and multi-family residential lots. Pre developed conditions are assumed to be a mixture of farm, pasture and orchard uses. Soils are predominantly Jory soils of Hydrologic Group B.

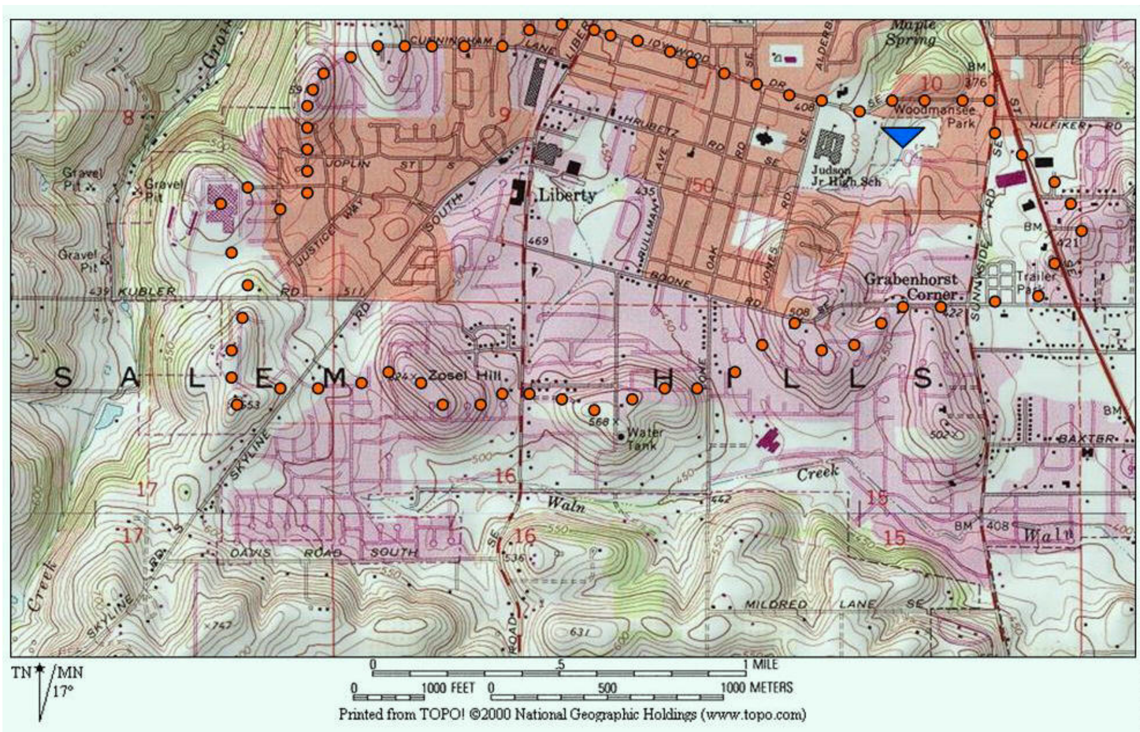


Figure 2-1 Drainage Basin Topography

Completing the available mapping, the City of Salem has an excellent Geographic Information system from which **Figure 2-2** has been created.

Remember, if you are using Adobe Reader you can zoom in or out on any of the graphics included in this course.

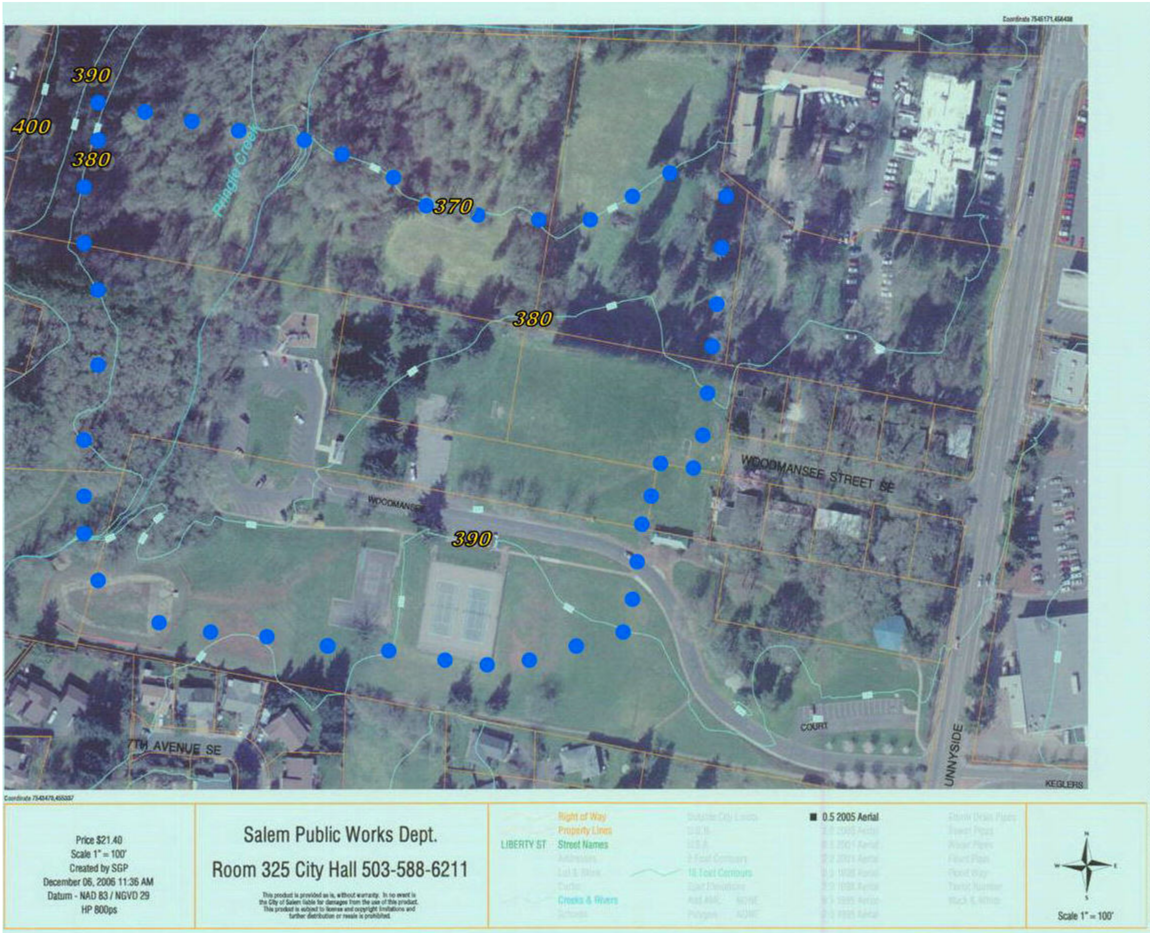


Figure 2-2
Park Topography

The dotted blue outline shows the approximate area available for the detention pond.

III. Salem Area Precipitation Data

Figure 3-1 summarizes the precipitation data used in these calculations and includes references to the sources of that data. This data is taken mostly from the City's Stormwater Master Plan and extended to cover a wider range of storms which, while rare, may be expected to occur during the useful life of the detention basin.

You will use this data in this course but it is suggested you look beyond the specifics of **Figure 3-1**. In your own practice you have probably accumulated Intensity-Duration-Frequency curves for your geographic area. Think of **Figure 3-1** as being a similar resource but more useful in that it covers a much wider range of possible storms. <u>Consider developing your own RDF curves and adding them to your hydrolgy tool kit.</u>

slerdf.xls Revised 8-22-07

Salem, OR
Rainfall Depth - Duration - Frequency Curves
n = 1 / Annual Exceedence Probability in Decimal Percent

Duration		Rainfall - inches								Notes
Hours	Days	n = 1	n = 2	n = 5	n=10	n = 25	n = 50	n = 100	*n = 500*	e
0.08333	0.003	0.05	0.07	0.08	0.10	0.13	0.15	0.17	*0.27*	
0.25	0.01	0.10	0.13	0.16	0.20	0.25	0.28	0.31	*0.48*	
1	0.04	0.25	0.31	0.39	0.47	0.56	0.63	0.70	*1.01*	
2	0.08	0.35	0.48	0.60	0.71	0.84	0.95	1.05	*1.46*	a
3	0.13	0.50	0.63	0.78	0.91	1.07	1.20	1.32	*1.81*	
6	0.25	1.00	1.10	1.40	1.60	1.90	2.10	2.30	*2.62*	b
24	1.00	2.00	2.70	3.20	3.50	4.00	4.40	4.70	*5.46*	d
48	2.00	3.00	3.60	4.50	5.00	5.70	6.30	6.80	*7.89*	b
72	3.00	3.71	4.63	5.64	6.18	7.04	7.72	8.29	*9.79*	
96	4.00	4.50	5.40	6.70	7.30	8.20	9.10	9.80	*11.40*	b
168	7.00	6.00	7.30	9.10	9.90	11.10	12.20	13.20	*15.34*	b
240	10.00	7.00	8.90	11.10	12.00	13.40	14.70	15.80	*18.54*	c

Notes: a. ODOT Hydraulics Manual
b. Precipitation- Frequency Atlas of the Western United States, NOAA 1973
c. Technical Paper 49, US Department of Commerce, Weather Bureau, 1963
d. Salem Storm Drain Master Plan, 2000
e. Extrapolated and smoothed

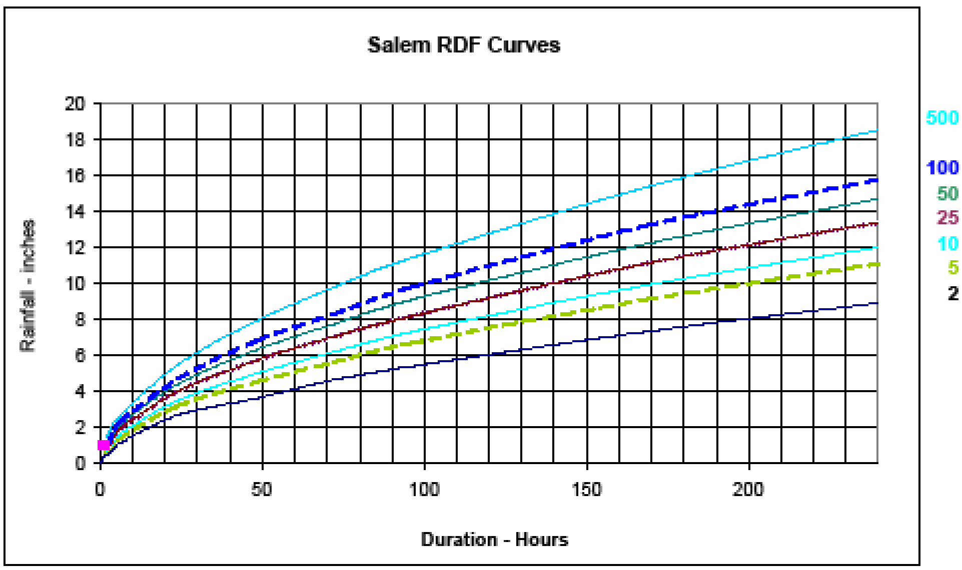


Figure 3-1

It would be easy, looking at **Fig. 3.1**, to conclude that the Salem area precipitation is predictable with an accuracy of plus or minus a few hundreths of an inch. This is not true, of course, as the example below will show.

If you live in one of the several States that have recently updated their precipitation data you can visit NOAA's website:

http://www.weather.gov/oh/hdsc/ or

http://lwf.ncdc.noaa.gov/oa/ncdc.html

There you will find more complete data which is updated and can be searched by latitude and longitude, station name, or watershed. The most useul information you would find there is the range, called the 90% confidence level, of the data. An error range of 30 to 40 percent is not uncommon.

Finally, **Figure 3-1** may be viewed as an array of possible storms we might use to "test" our detention pond. With a personal computer and one or more of the programs listed above this can be done quickly and easily. For this course we won't use all 96 storms but we'll select about a dozen, including at least one historical storm from February 1996.

Figure 3-2 below illustrates the kind and range of data available for some parts of the United States.

Precipitation Frequency Data Server http://hdsc.nws.noaa.gov/cgi-bin/hdsc/buildout.perl?type=pf&units=us&series=pd&statename...

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

California 33.677 N 116.41 W 856 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 4
G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2006
Extracted: Tue Sep 11 2007

Confidence Limits | Seasonality | Location Maps | Other Info. | GIS data | Maps | Help | Docs | U.S. Map

Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.13	0.20	0.25	0.33	0.41	0.55	0.62	0.82	1.01	1.07	1.09	1.19	1.31	1.41	1.61	1.83	2.06	2.23
2	0.18	0.27	0.34	0.46	0.56	0.74	0.84	1.10	1.37	1.46	1.47	1.61	1.78	1.92	2.20	2.50	2.83	3.07
5	0.28	0.43	0.53	0.71	0.88	1.13	1.26	1.62	2.01	2.18	2.20	2.41	2.67	2.89	3.31	3.76	4.26	4.65
10	0.37	0.56	0.70	0.94	1.16	1.46	1.61	2.05	2.51	2.75	2.78	3.07	3.38	3.65	4.18	4.72	5.35	5.85
25	0.51	0.78	0.96	1.30	1.61	1.98	2.15	2.66	3.23	3.56	3.65	4.03	4.40	4.77	5.42	6.09	6.87	7.52
50	0.64	0.97	1.21	1.63	2.01	2.44	2.62	3.19	3.81	4.23	4.39	4.84	5.24	5.69	6.44	7.19	8.08	8.86
100	0.79	1.21	1.50	2.01	2.49	2.96	3.15	3.76	4.45	4.96	5.21	5.75	6.17	6.70	7.54	8.38	9.36	10.28
200	0.97	1.48	1.83	2.47	3.06	3.58	3.75	4.39	5.13	5.74	6.12	6.75	7.18	7.80	8.72	9.65	10.71	11.78
500	1.26	1.92	2.38	3.20	3.96	4.53	4.67	5.33	6.11	6.88	7.47	8.24	8.65	9.41	10.43	11.46	12.62	13.90
1000	1.52	2.32	2.88	3.88	4.80	5.39	5.49	6.13	6.93	7.83	8.63	9.51	9.88	10.77	11.84	12.93	14.15	15.60

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.

1 of 6 9/11/2007 11:10 AM

IV. Local Area Regression Equations and the NFF

The **National Flood Frequency (NFF)** program may be used to calculate peak flows based on stream gage records. The City of Salem Design Standards contain a series of regression equations developed in 1983 by the USGS. While seldom used for detention basin design, such equations are available for all 50 states and are very often used for Flood Studies on larger streams and rivers. It is important to understand that they are based not on precipitation data but on flow data from gaged streams. Because there are very few stream gages, and their records are short, the NFF equations are even less statistically reliable than rainfall records.

As a very approximate first step, the ***NFF*** program is used here to estimate peak flows and annual exceedence probabilities (AEP).

A second reason for using ***NFF*** is that flows on the Willamette River and several of its tributaries in Salem have been calculated using it for many Flood Studies, both by FEMA and its Contractors and are considered conservative. Indeed, they are often more conservative than Rational Method flows.

<u>These flow estimates are not used as the basis for design but are included to provide a “reasonableness “ check on the published flows and volumes calculated by others.</u>

Calculations using these equations are summarized on the following pages.

Regression equations represent a statistical approach to design, as opposed to more deterministic methods like the Rational Method. They have a standard error of estimate on the order of plus or minus 16 to 84 percent. This error might be improved through the use of additional years of record and re-analysis. In some parts of the US, efforts have been made to do this but funding may not be available in your area to accomplish this anytime soon.

Still, this method is believed to be reliable within the wide limits stated. Consistently applied, it would be possible for designers to check each others' work with reasonable hope of agreement.

The NFF also estimates the maximum probable flow according to Crippen and Bue which for our basin is about 8860 cfs. Although this is a very large, and unlikely, flow it could be used to size the emergency spillway of the detention pond.

But, large errors will remain in any analysis. These errors simply reflect the random nature of rainfall. In nearly all cases, flows are reported to no more than three significant figures. Two, or even one, significant figure is about the best one can hope for. Any better precision is simply not possible.

V. An Eclectic Approach to the Design

Of the many methods used by Engineers, and others, to estimate probable flood flows only a few are widely accepted. These include the NRCS (National Resource Conservation

Service)Synthetic Hydrograph, the Santa Barbara Urban Hydrograph, the National Flood Frequency Program (NFF), HEC-HMS, SWMM, WIN TR-55 and some others.

For this report the NRCS-484 method has been chosen. The reasons for this choice are detailed in **Appendix A.**

Appendix A is part of this course and the Quiz includes questions related to this appendix.

There are also many narrowly applied "local" methods which are limited to certain geographical areas or recognized only by some reviewing agencies such as State Departments of Transportation. These methods are usually prescriptive "standards" having little theoretical or empirical basis. Such methods are not considered by most Engineers suitable for the design of any but the smallest, "local" detention basins. Even if you are required to use such methods you will find the design approach suggested here to be a useful check on the adequacy of your design.

In the many available hydrologic models only a few features are common to all. Two that are most common are land area and land use. Other factors enter in but these two are common to nearly all methods. Many models recognize that storm probability and duration are important factors and affect the size and cost of any detention facility, whether local or regional.

Some of the most sophisticated methods make use of the power of the personal computer and excellent aids such as modern Geographic Information Systems (GIS), NEXTEL Radar precipitation data, and the like but such programs are often expensive, and require very large amounts of detailed input data and training in the use of the program. Usually such progrms are difficult to set up for smaller projects. In this welter of "data" it is easy to believe that the bigger, more complicated analysis is somehow more "scientific", or "accurate" than other available methods. <u>This is not true.</u>

Back to our example. Some of the really important storm events in the Willamette Valley are probably the longer duration storms such as those which occurred in 1964-65, and in February and November of 1996. These storms, lasting 3 to 10 days or more, typically occur late in the rainy season when ground is saturated, snow covered, or frozen, thus accelerating runoff and increasing the liklihood of significant flood damage.

This all leads to a suspicion, at least, that a regional detention pond should be designed for a fairly long duration storm, say 3 days or more, with an annual exceedence probablity of about 1 %. An even lower probabilty storm may prove, in the final design, to be necessary to achieve acceptable factors of safety where downstream areas are particulary vulnerable to flood damage.

Finally, with the method outlined below, the initial design is "tested" by routing a range of rainfall events through the trial pond and reviewing the results carefully. This is a trial and error approach but only a few trials are needed before patterns emerge allowing the designer to move quickly to a final configuration.

Only in this way can a comprehensive assessment of the design be made and prudent judgement applied. Conventional wisdom on this has long asserted that we cannot economically design local and regional stormwater facilities for low probability, infrequent events. Like much other conventional wisdom little, if any, evidence is ever presented to support this notion.

In the design which follows, a regional pond is preliminarily designed with the first goal of reducing outflow to the predevelopment level, as near as we can determine what that was. Land cover, rather than soil type, is more important in most cases according to the work of many others in making that educated guess.

A second goal is to determine the answer to the question of what might happen if flows exceeding the "design storm" enter the basin. Will it fail structurally or hydraulically ? If so, what might the consequences of that be?

The latter question, of course, cannot be answered for a hypothetical drainage basin, but for an actual drainage basin and pond it can be answered, at least qualitatively. Even an approximate answer can be very useful in assessing the risks of failure.

The design approach suggested here may be thought of as having seven main steps. Numerous arbitrary decisions are made in the course of the design but each such decision is tested to determine if it seems reasonable. The method uses widely recognized computer models and a variety of what are believed to be the better ideas drawn from many fields including statistics and stream restoration guidelines.

<u>Perhaps the most important advantage of this method is that the selected model may be calibrated based on actual rainfall events.</u>

<u>Step 1 Calculate Peak Flows</u>

Calculate peak flows (for both predeveloped (Rural) and developed (Urban) conditions) for the basin for a range of probabilities from 50% to 0.2% AEP (2 Year to 500 Year) events using the NFF program.
These are summarized below:

Rural Estimate: Rural 1
Basin Drainage Area: 1.59 mi^2
1 Region
Region: Willamette_Region
2-Year 24 Hour Precipitation = 2.5 in
Crippen & Bue Region 17

Urban Estimate: Urban 1
Basin Drainage Area: 1.59 mi^2
1 Region
Region: National Urban
Channel Slope = 66 ft per mi
2-year 24-hour Precipitation = 2.5 in
Basin Storage = 0 percent
Basin Development Factor = 6 dimensionless

Flood Peak Discharges, in cubic feet per second

Estimate	Recurrence Interval, yrs	Peak, cfs	Standard Error, %
Rural 1	2	62.4	33
	5	97.1	33
	10	123	33
	25	160	34
	50	190	36
	100	222	37
	500	306	
Urban 1	2	326	38
	5	430	37
	10	507	38
	25	603	40
	50	698	42
	100	803	44
	500	1040	49

Step 2 Select a Design Storm

Arbitraily, we'll calculate the 100 year 96 hour storm flows and volumes using the NRCS-484 method. The actual rainfall amount (9.8 inches) and pattern from the NRCS Type IA storm is used. This may be thought of as the initial "design storm". Subsequent calculations may show that it is not the governing storm event but it often serves as a useful place to begin the design / analysis. (Storm Types established by NRCS are illustrated in Appendix A).

Note that any one of many possible storms might have been selected as a "design storm" . Because Steps 4, 5 and 6 require other test storms the choice of the initial design storm is not critical. At least one of the selected test storms will probably govern the final design.

Using the computer program SMADA an initial storage volume calculation can be made. The details of that calculation may be found below, in abbreviated form, and under Step 5. That calculation yields a developed runoff volume of about **633 Ac-ft.**

SMADA Output Pre developed Conditions

Hydrograph Type :SCS 484 Hydrograph

Time (hr)	Time HHMM	Rain (in)	C Rain (in)	Infiltration (in)	Excess (in)	Excess (cfs)	Outflow
0.250	00015	0.012	0.012	0.012	0.000	0.000	0.000

0.500	00030	0.012	0.024	0.012	0.000	0.000	0.000
0.750	00045	0.012	0.037	0.012	0.000	0.000	0.000
1.000	00100	0.012	0.049	0.012	0.000	0.000	0.000
.	.	.	.	.	.	.	.
	.	.	.	.	.	.	. .
	.	.	.	.	.	.	. .
98.25	00215	0.000	9.800	0.000	0.000	0.000	11.359
98.50	00230	0.000	9.800	0.000	0.000	0.000	8.117
98.75	00245	0.000	9.800	0.000	0.000	0.000	5.414
99.00	00300	0.000	9.800	0.000	0.000	0.000	3.251
99.25	00315	0.000	9.800	0.000	0.000	0.000	1.628
99.50	00330	0.000	9.800	0.000	0.000	0.000	0.545
		9.800	5.327	4.472	4.472		

Totals for Watershed in inches over 1015.80 acres

Rational Coefficient = 0.456 ***Peak Flow (cfs) = 203.91***

SMADA Output Developed Conditions

Hydrograph Type :SCS 484 Hydrograph

Time (hr)	Time HHMM	Rain (in)	C Rain (in)	Infiltration (in)	Excess (in)	Excess (cfs)	Outflow (cfs)
0.250	00015	0.012	0.012	0.006	0.006	25.086	1.183
0.500	00030	0.012	0.024	0.006	0.006	25.086	3.549
0.750	00045	0.012	0.037	0.006	0.006	25.086	7.098
1.000	00100	0.012	0.049	0.006	0.006	25.086	11.829
1.250	00115	0.012	0.061	0.006	0.006	25.086	15.851
1.500	00130	0.012	0.073	0.006	0.006	25.086	19.164
1.750	00145	0.012	0.086	0.006	0.006	25.086	21.768
2.000	00200	0.012	0.098	0.006	0.006	25.086	23.661
2.250	00215	0.015	0.113	0.007	0.007	30.103	25.082
	.	.	.	.	.	.	.
	.	.	.	.	.	.	. .
	.	.	.	.	.	.	. .
96.00	00000	0.013	9.800	0.001	0.012	49.752	49.716
96.25	00015	0.000	9.800	0.000	0.000	0.000	47.378
96.50	00030	0.000	9.800	0.000	0.000	0.000	42.694
96.75	00045	0.000	9.800	0.000	0.000	0.000	35.663
97.00	00100	0.000	9.800	0.000	0.000	0.000	26.284
97.25	00115	0.000	9.800	0.000	0.000	0.000	18.310
97.50	00130	0.000	9.800	0.000	0.000	0.000	11.743
97.75	00145	0.000	9.800	0.000	0.000	0.000	6.581
98.00	00200	0.000	9.800	0.000	0.000	0.000	2.826
98.25	00215	0.000	9.800	0.000	0.000	0.000	0.477
		9.800	**2.376**	**7.422**	**7.422**		

Totals for Watershed in inches over 1015.80 acres
Rational Coefficient = 0.758 ***Peak Flow (cfs) = 417.97***

Step 3 Estimate Required Storage Volume

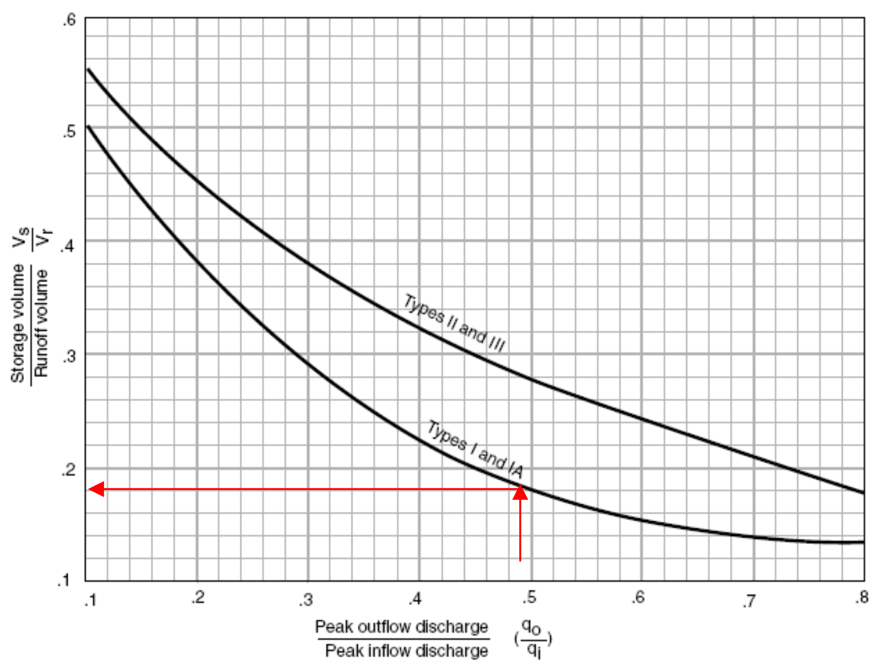
Using **Figure 6.1** , from TR-55 calculate the required storage volume, Vs:

Taking the ratio of the predeveloped and developed flows we get:

Qpre/Qdev = 204/418 = 0.49

Now, use Figure 6.1:

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



Vs/Vr = 0.18

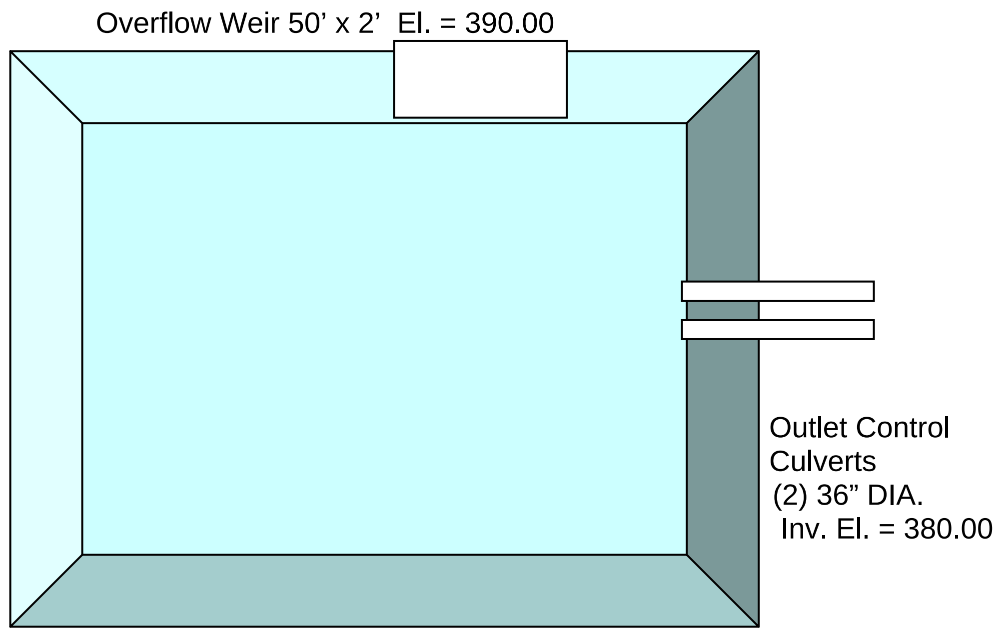
(Strictly speaking, Figure 6.1 is intended for use only with flows calculated using the TR-55 method. That rule is "bent" here to arrive at a fairly conservative estimate of the initial volume.)
From this we can estimate the required storage volume to be:

Vs = (0.18)(633 af) = 114 acre feet

We'll use 100 ac-ft for our initial pond volume.

Here's a sketch of our hypothetical pond:

Step 4 - Sketch Initial Pond Design



Bottom Dimensions: 626' x 626'
Bottom Elevation = 380'
Design Depth: 10'
Sidewall Slopes: 4H to 1V Freeboard: 2' Volume Full: 3,918,760 cf = 89.96 Ac-ft

Figure 4-2

Using Hydraflow Hyrographs (2007) we can easily develop the stage storage and stage ouflow relationships for this pond . These are shown below as **Figures 4-3 and 4-4.**

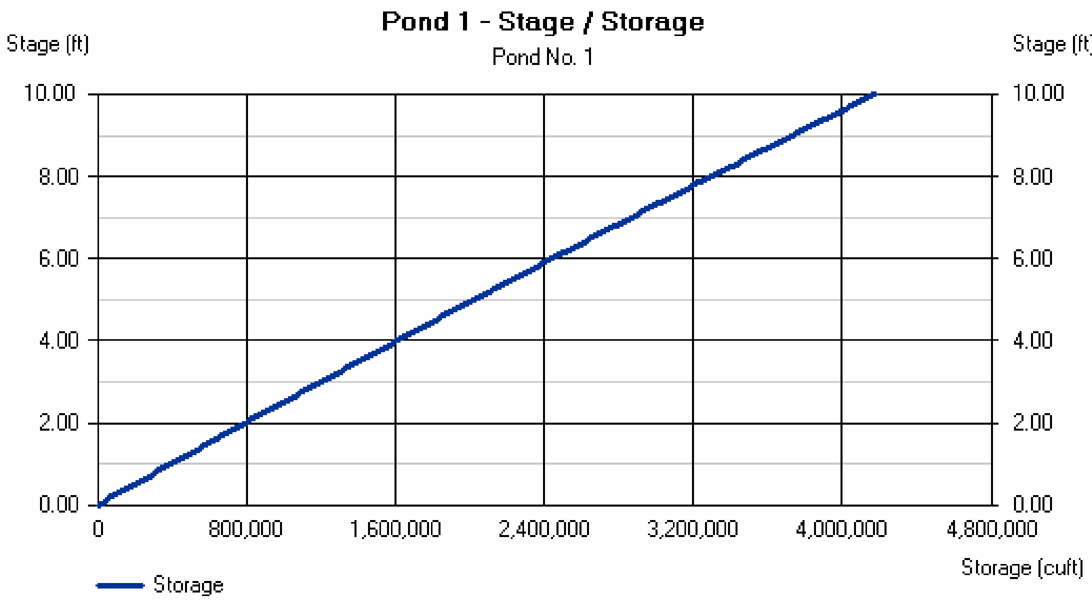


Figure 4-3

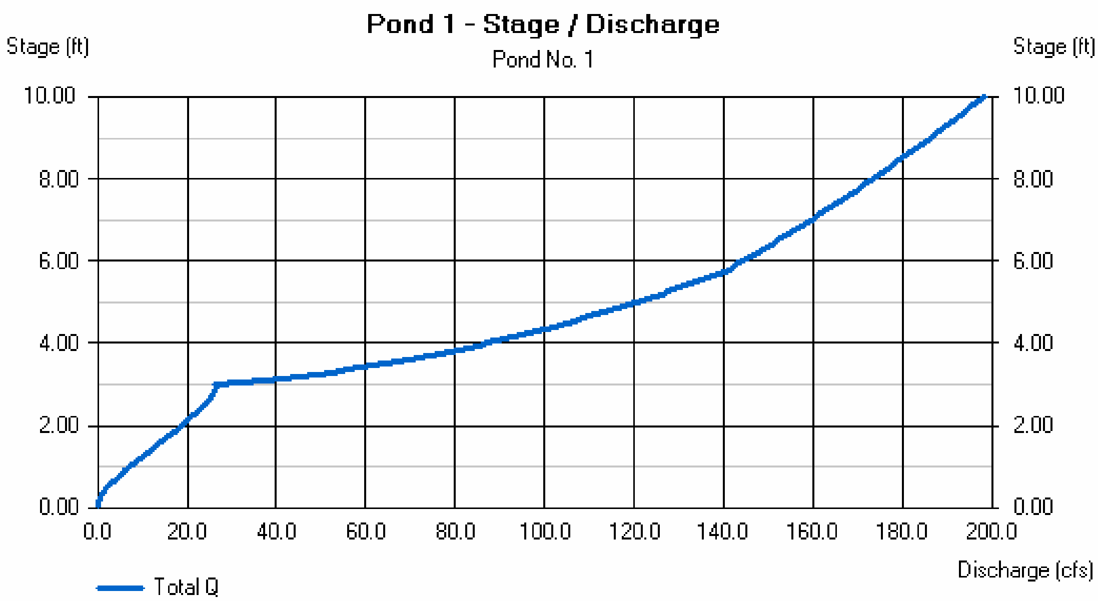


Figure 4-4

Step 5 Route Series of Storms through pond

In this step a series of hypothetical storms are routed through this basin. These storms range in duration from 1 hour to 288 hours (12 days). They also range from 0.2% to 50% AEP. Because rainfall data is available, the February 1996 storm is also included.

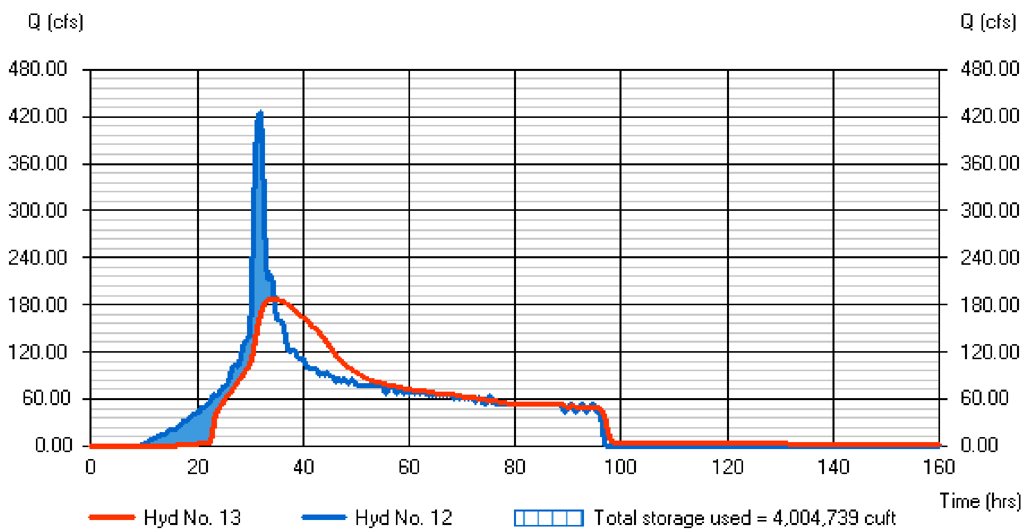
Note: The detailed calculations for these routings are voluminous so only the graphical results are presented here.

<Fig51.xls>

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Hydrograph Routing through Pond
100 Year - 96 Hour - Type IA Storm - AMC II



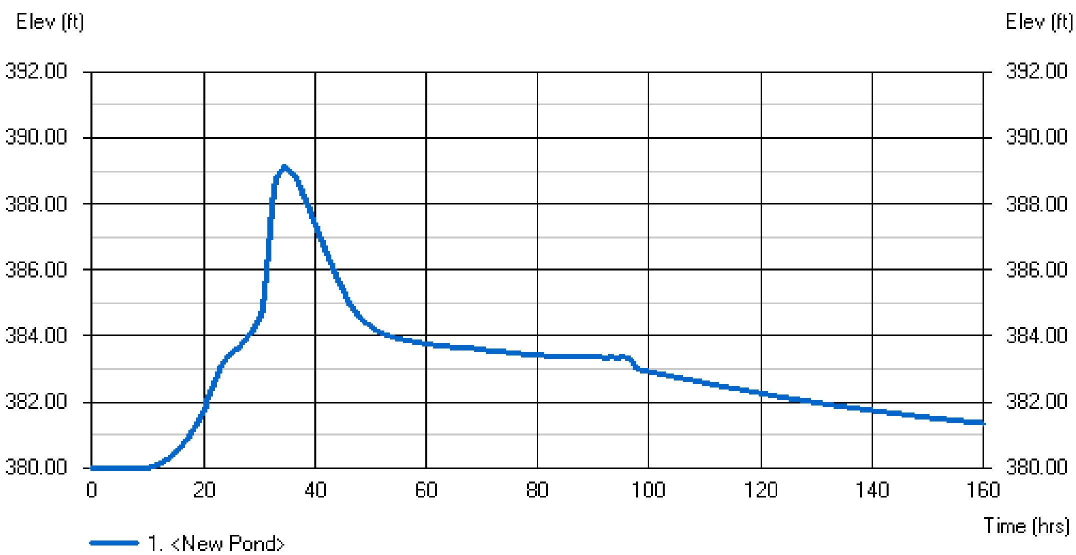


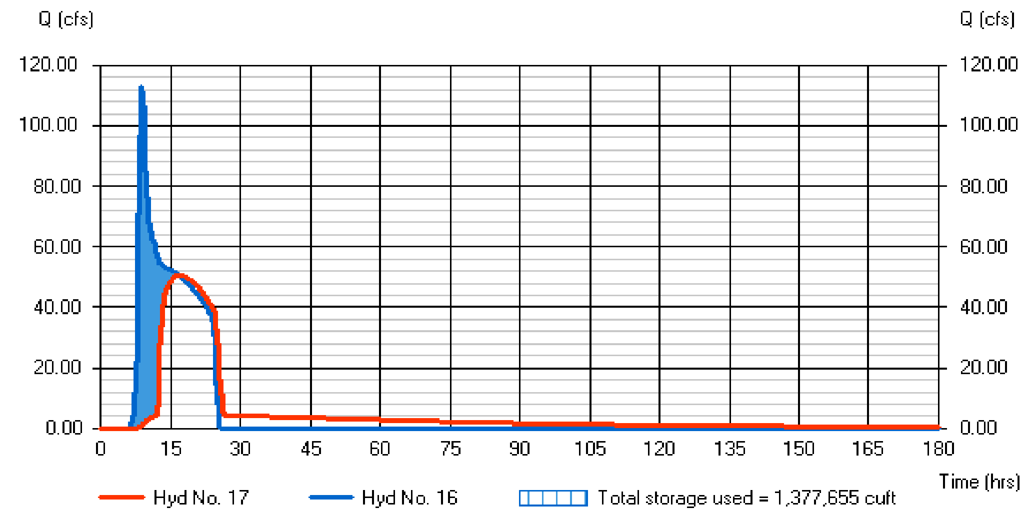
Figure 5.1 - Hydrograph Routing

<Fig52.xls>

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Hydrograph Routing through Pond
2 Year - 24 Hour - Type IA Storm - AMC II



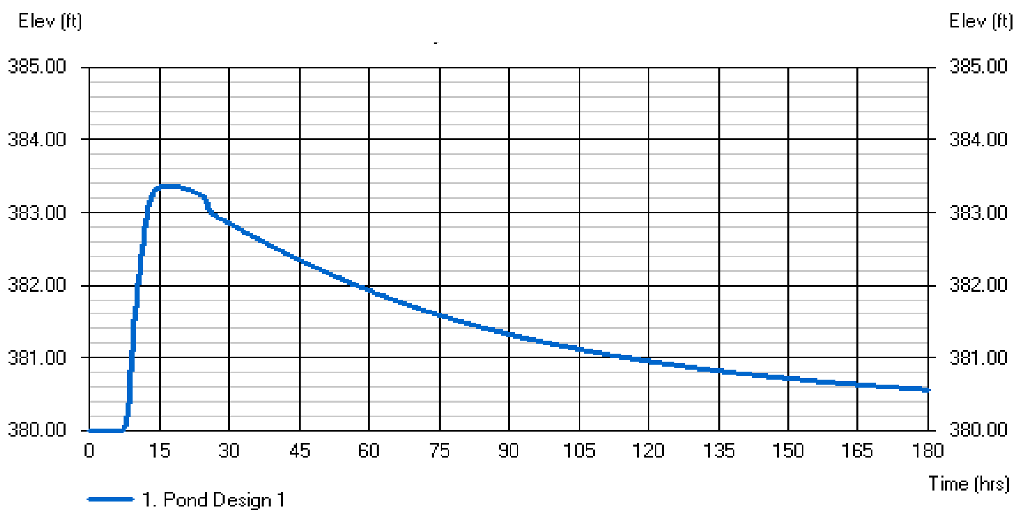


Figure 5.2 - Hydrograph Routing

<Fig53.xls>

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Hydrograph Routing through Pond
2 Year - 72 Hour - Type IA Storm - AMC III

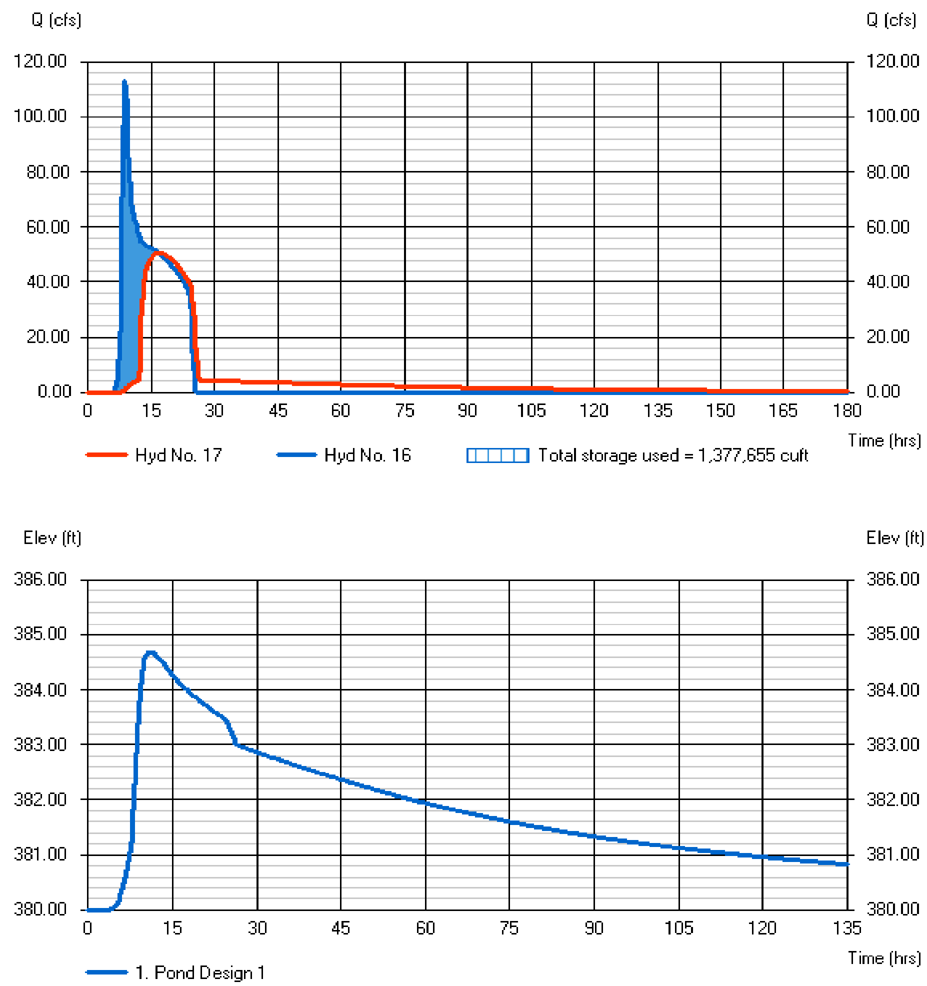


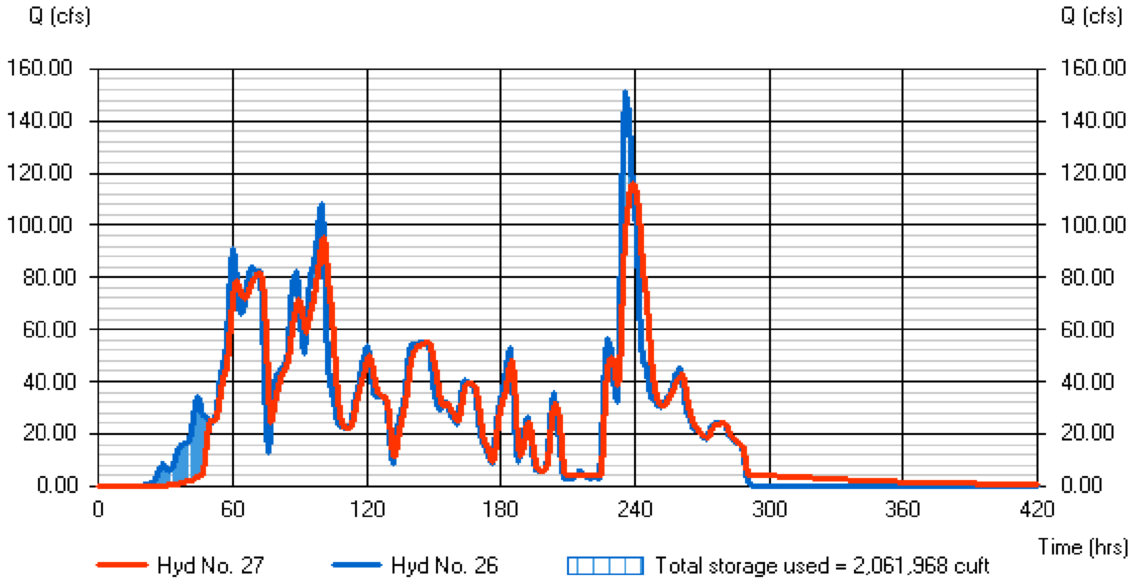
Figure 5.3 - Hydrograph Routing

<Fig54.xls>

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Hydrograph Routing through Pond
10 Year - 288 Hour - Feb. 1996 Storm Type - AMC II



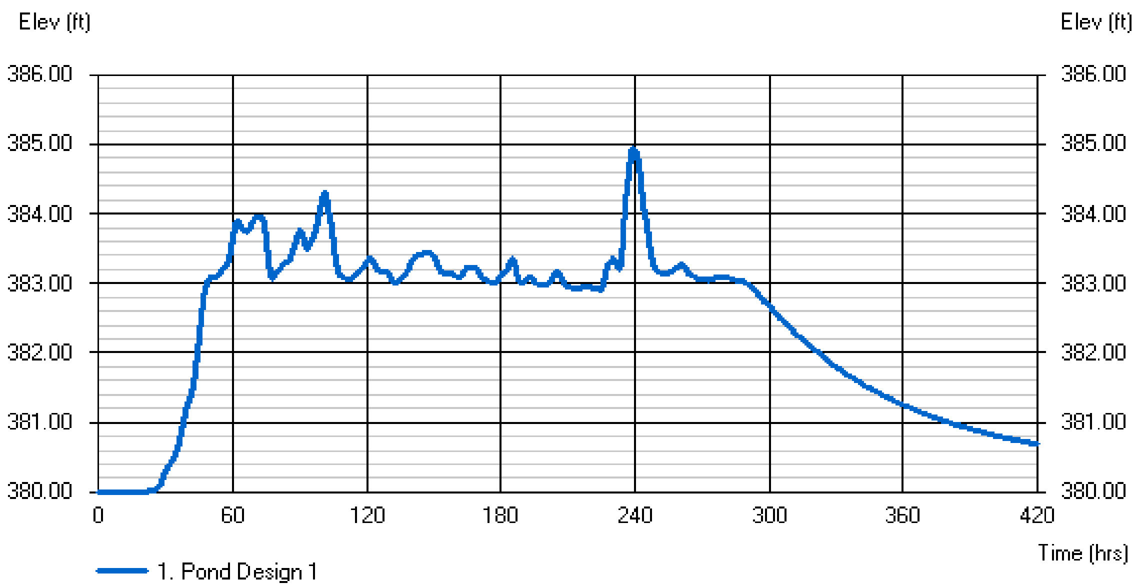


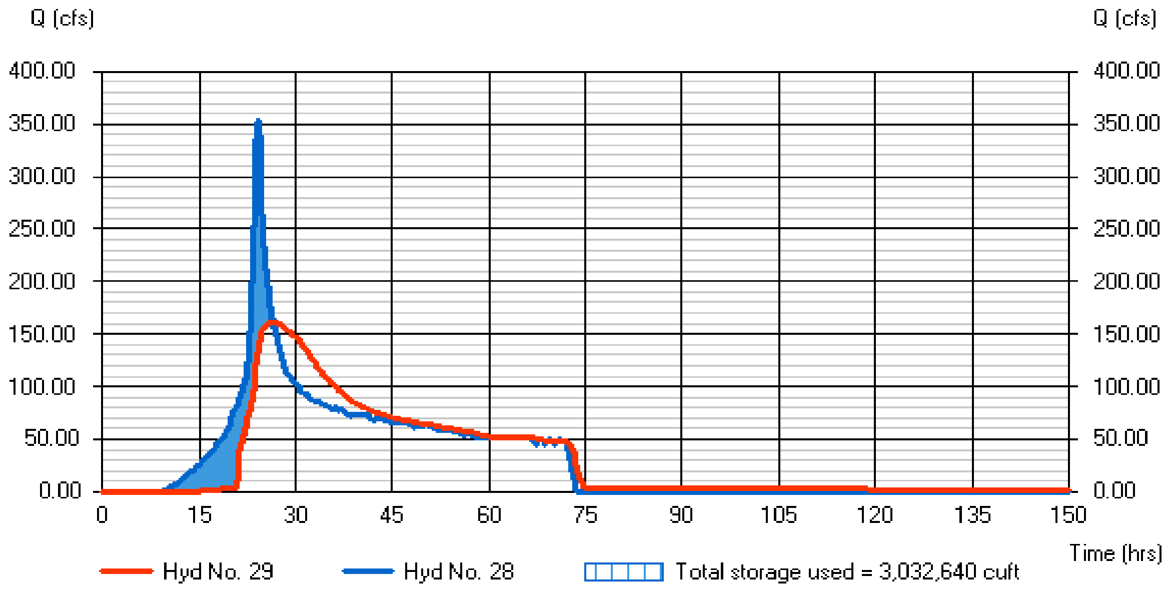
Figure 5.4 - Hydrograph Routing

<Fig55.xls>

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Hydrograph Routing through Pond
25 Year - 72 Hour - Type IA Storm - AMC II



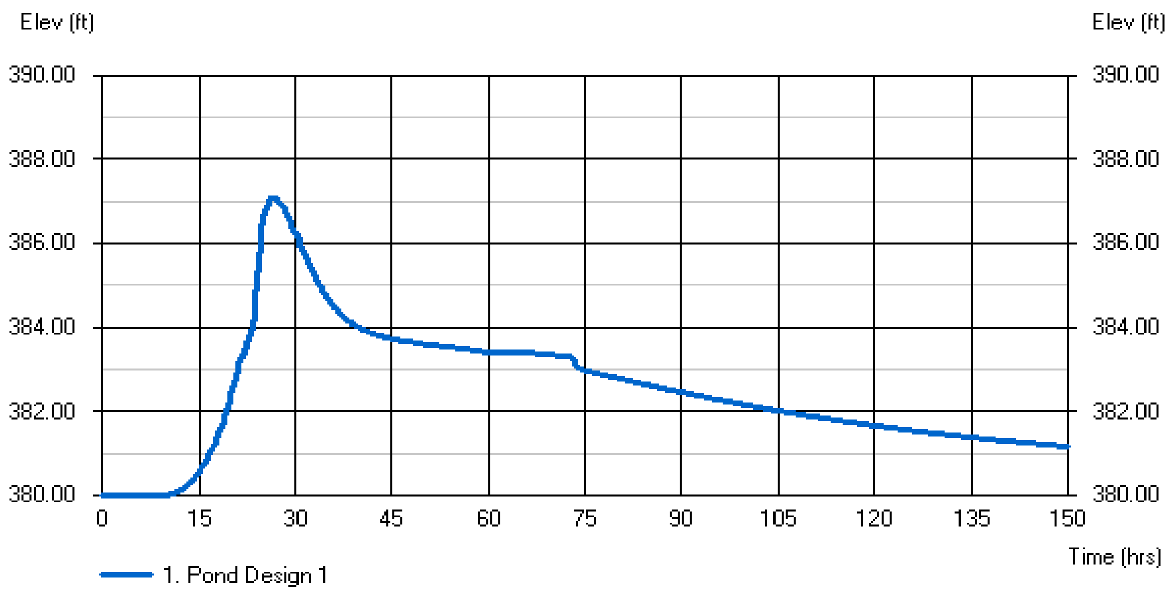


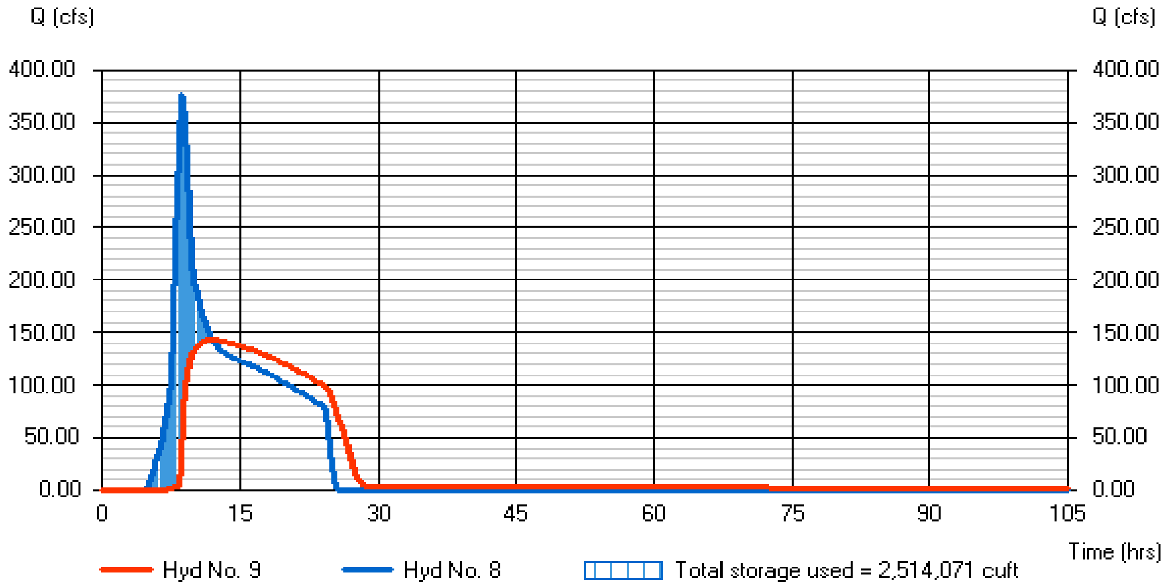
Figure 5.5 - Hydrograph Routing

<Fig56.xls>

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Hydrograph Routing through Pond
100 Year - 24 Hour - Type IA Storm - AMC II



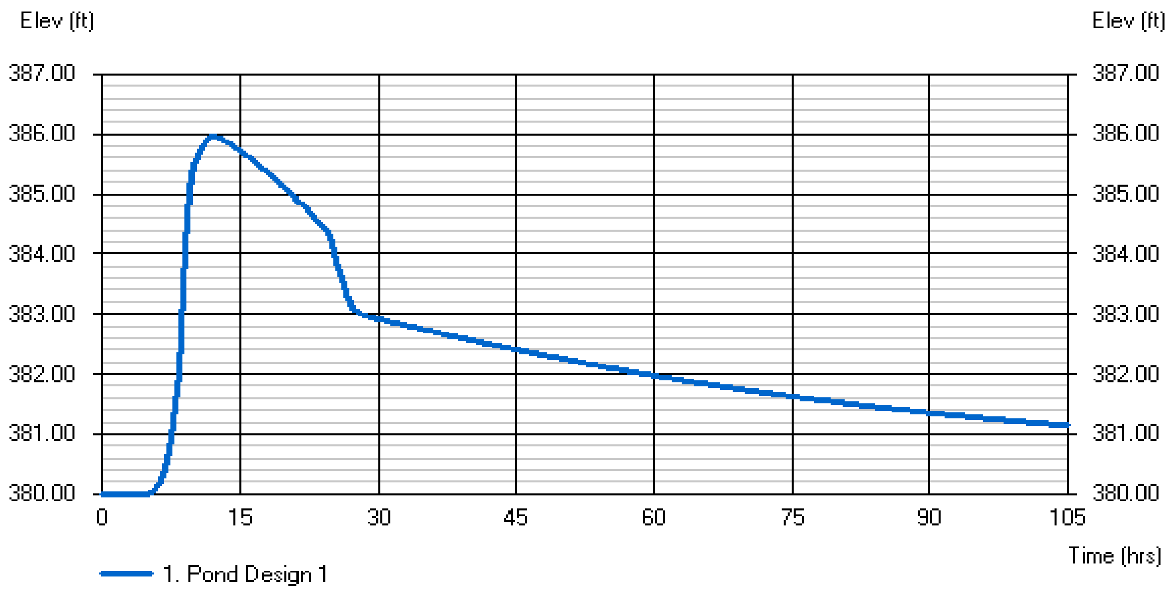


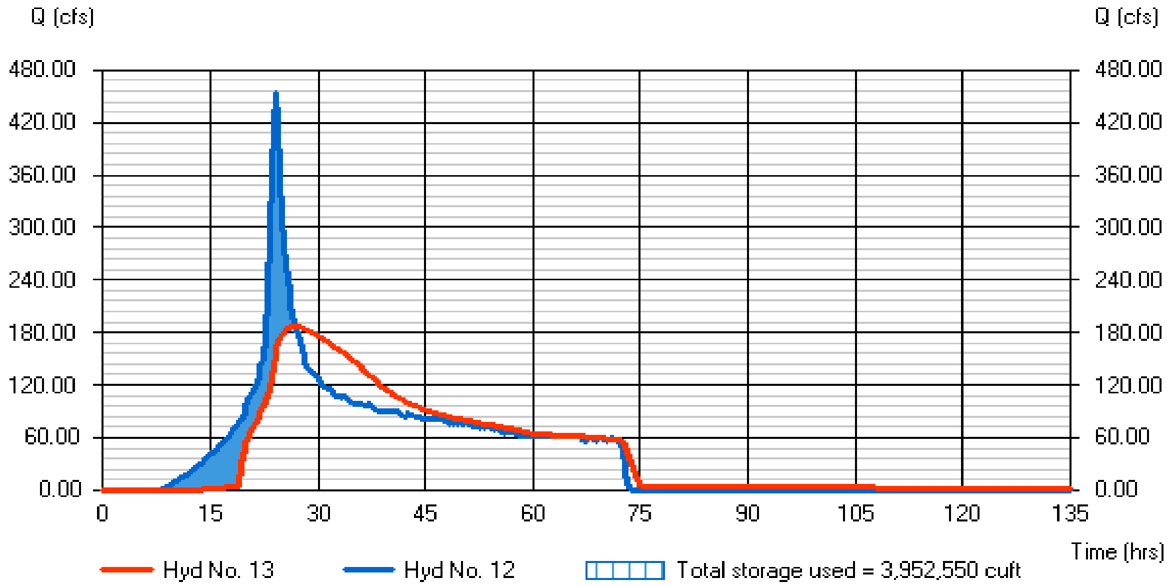
Figure 5.6 - Hydrograph Routing

<Fig57.xls>

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Hydrograph Routing through Pond
100 Year - 72 Hour - Type IA Storm - AMC II



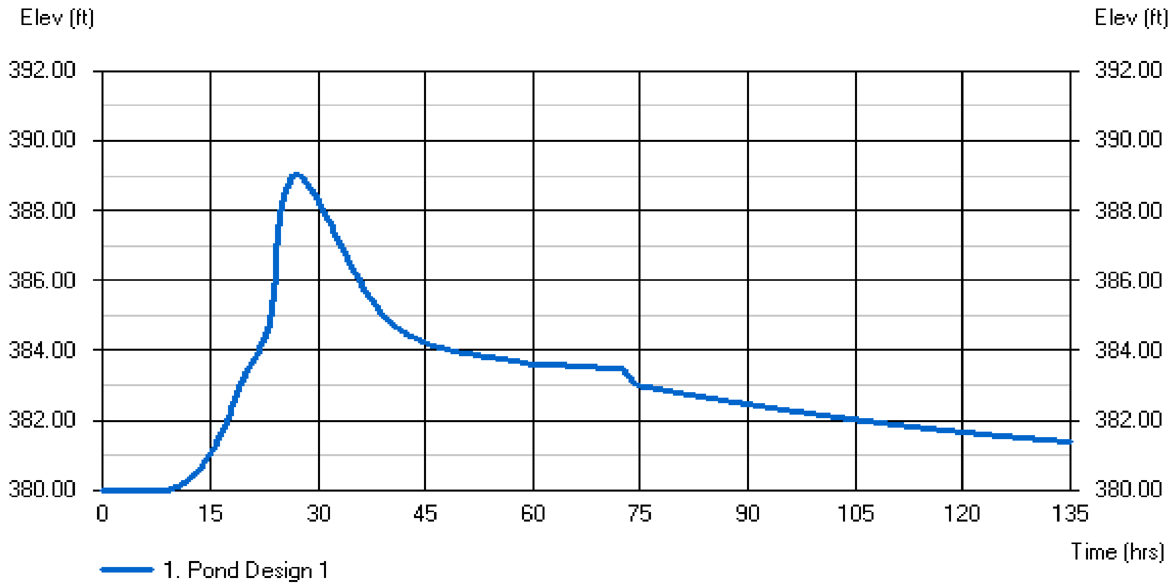


Figure 5.7 - Hydrograph Routing

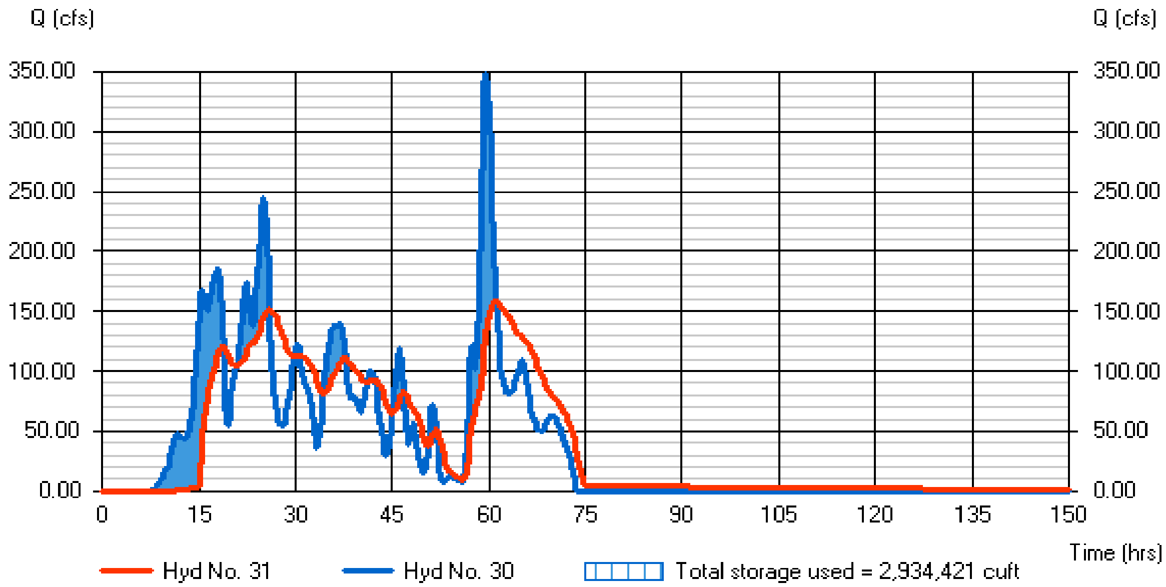
<Fig58.xls>

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Hydrograph Routing through Pond

100 Year - 72 Hour - Feb. 1996 Type Storm - AMC II



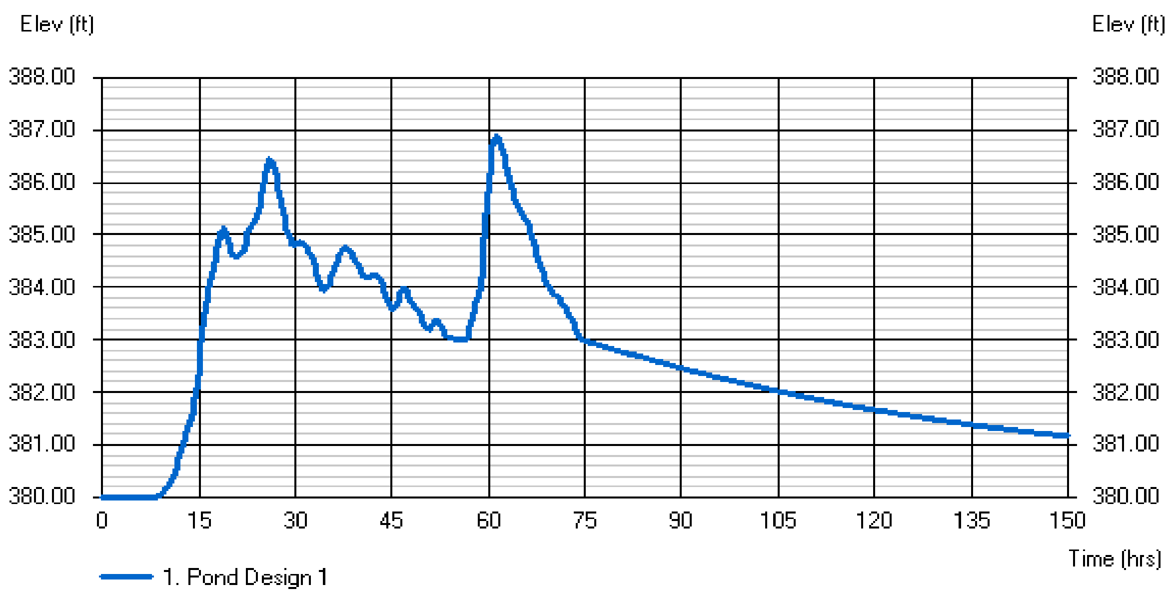


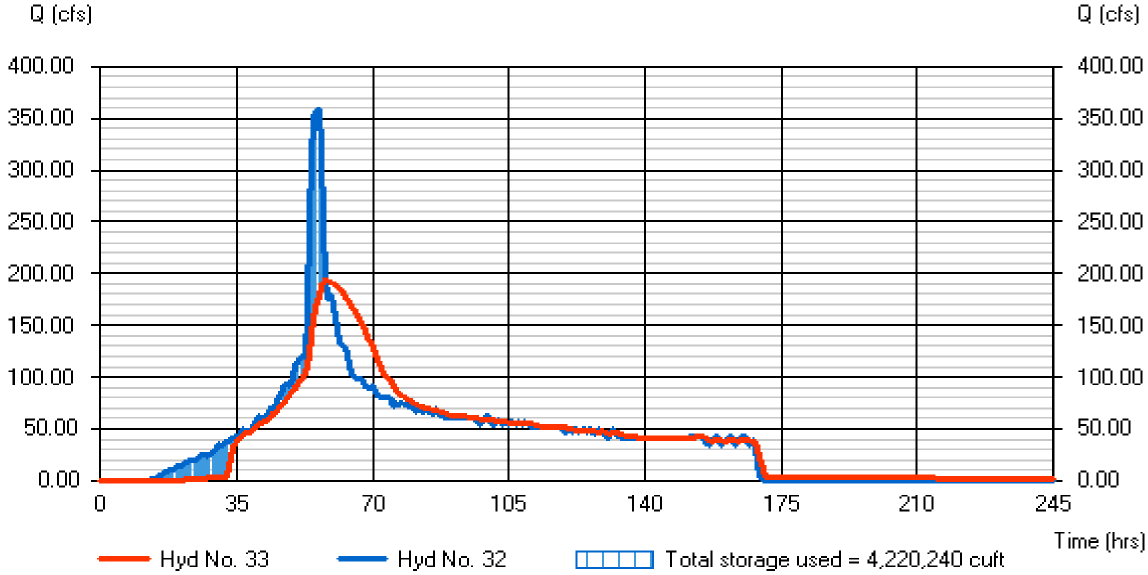
Figure 5.8 - Hydrograph Routing

<Fig59.xls>

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Hydrograph Routing through Pond
100 Year - 168 Hour - Type IA Storm - AMC II



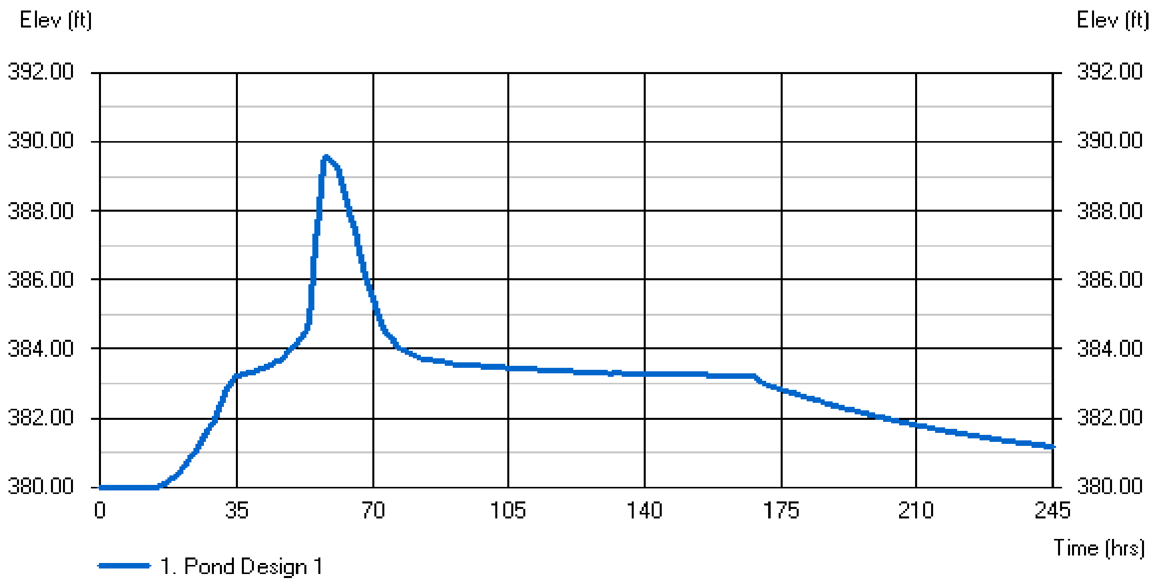


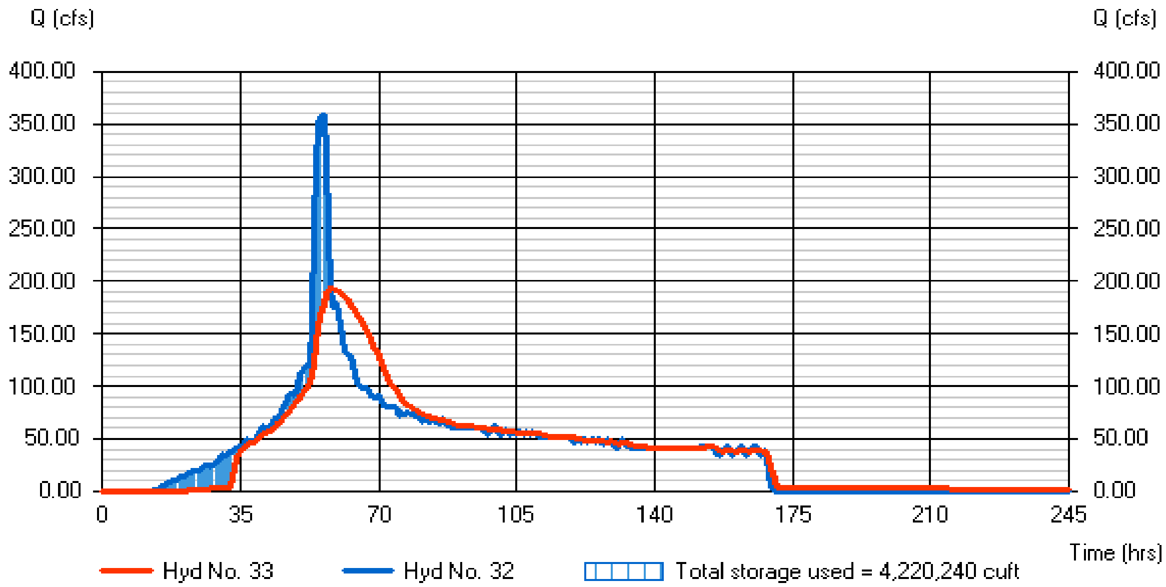
Figure 5.9 - Hydrograph Routing

<Fig59.xls>

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Hydrograph Routing through Pond
100 Year - 168 Hour - Type IA Storm - AMC II



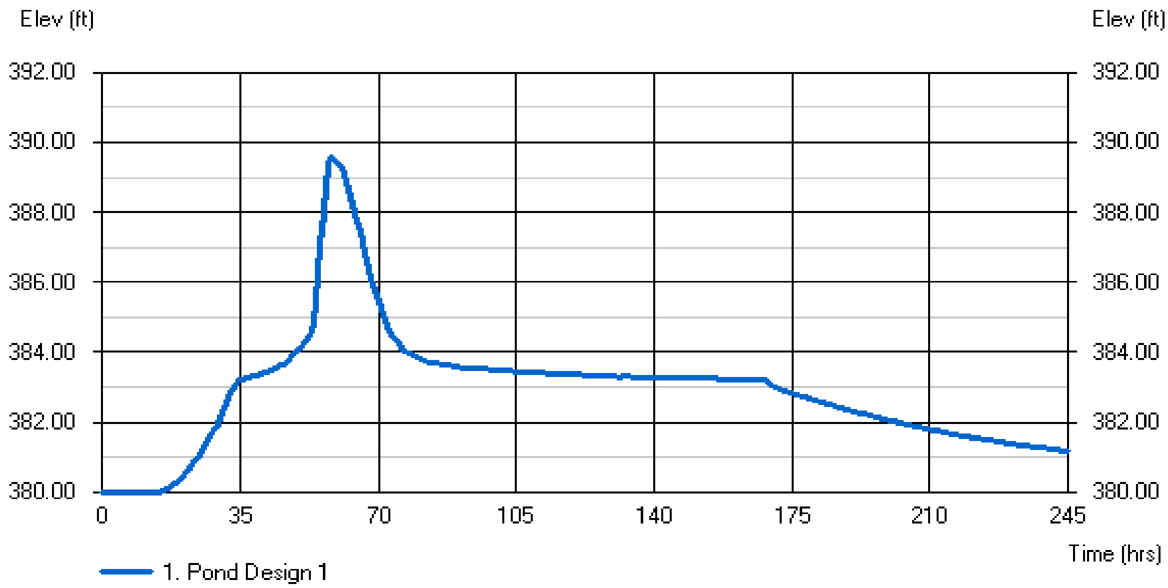


Figure 5.9 - Hydrograph Routing

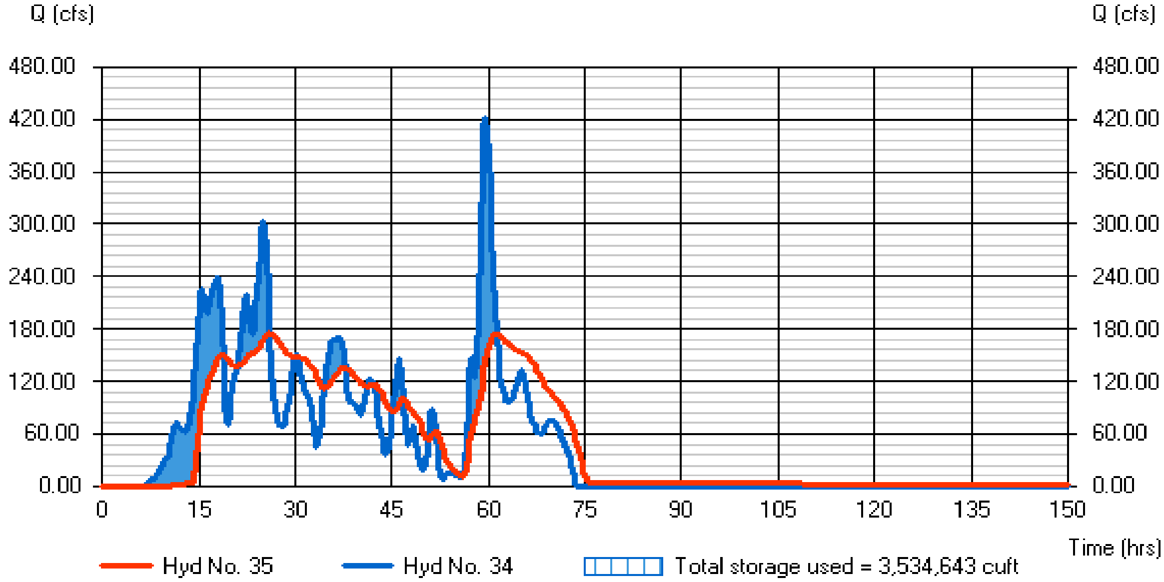
<Fig510.xls>

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Hydrograph Routing through Pond

500 Year - 72 Hour - Feb. 1996 Type Storm - AMC II



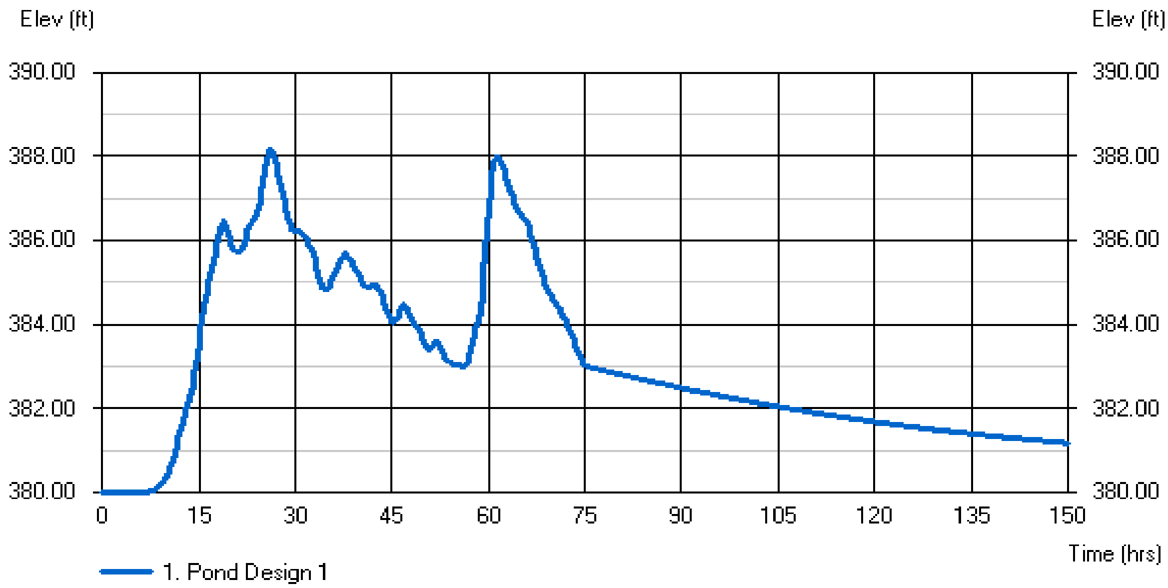


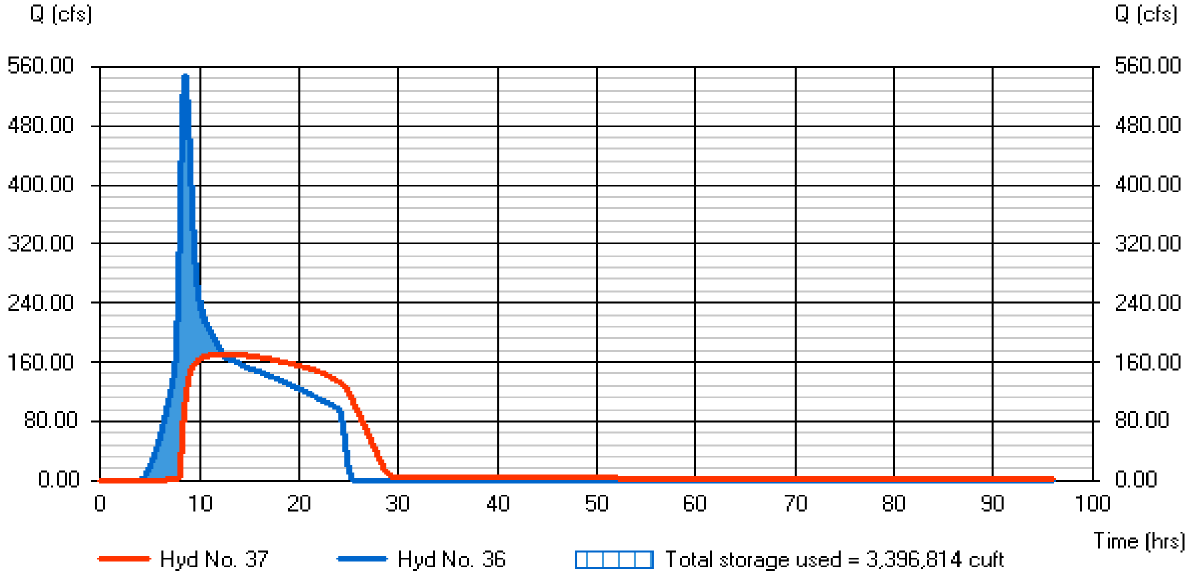
Figure 5.10 - Hydrograph Routing

<Fig511.xls>

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Date: 11-5-07
Design: rwf
Check: rp
Job No.

Hydrograph Routing through Pond
500 Year - 24 Hour - Type IA Storm - AMC II



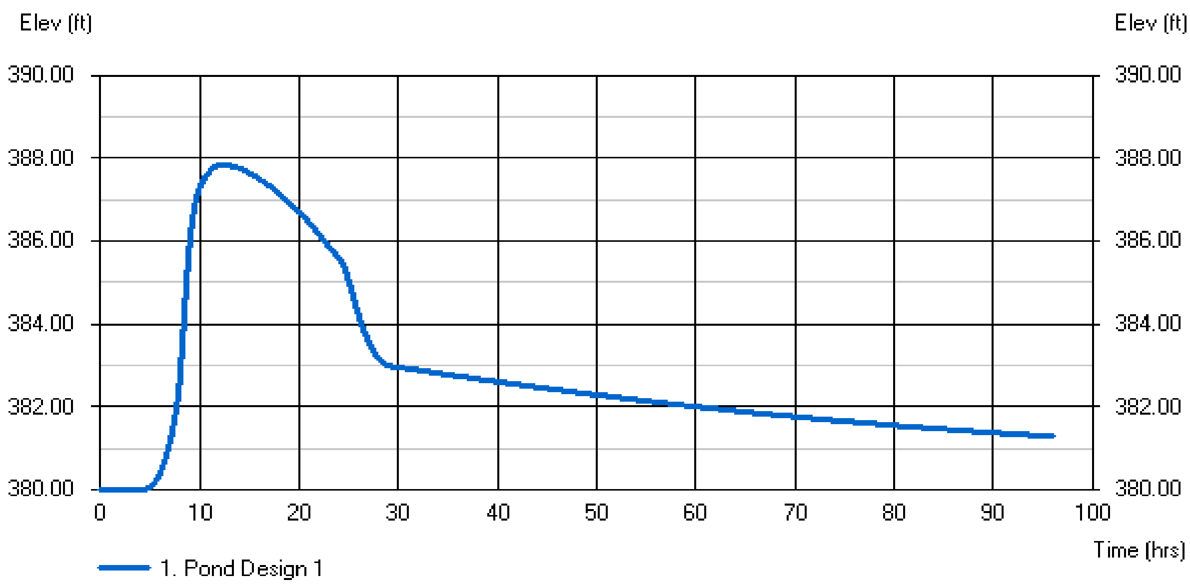


Figure 5.11 - Hydrograph Routing

R3Fig512.xls

Date: 11-5-07
Design: RWF
Check: RP
Job No.

Project: Regional Detention Pond on Pringle Creek
Location: Woodmansee Park, Salem, OR
Client: PDHOnline.org

Site Data:

Pervious Area	507.9 Ac	22124124 SF	50.00%
Imperv. Area	507.9	22124124	50.00%
Totals	1015.8 Ac	44248248 SF	100.00%

Initial Design for 1% AEP - 72 Hour Storm
Detain 100 yr Developed runoff (Use TR-55 Method)
CN undeveloped(Jory soils) = 58
CN developed(weighted average) = 78 (AMC II) or CN = 90 (AMC III)
Hydrologic Group B
Tc = 60 min (Developed)
Tc = 90 min (Undeveloped)
The computer programs Hydraflow and NFF were used to calculate the following flows and volumes:

Pond Performance Summary

Flows and Runoff Volumes

						Inflow		Pond Outflow		
n	AEP (%) *	Duration (hours)	Rainfall Distribution	Rainfall (inches)	AMC/CN	Peak Flow (cfs)	Runoff (inches)	Peak Flow (cfs)	Pond WS Elev	Flow Ratio
100	1%	96	NRCS IA	9.8	II/78	426	6.63	188	389.1	0.44
2	50%	72	NRCS IA	2.7	II/78	113	0.92	50	383.4	0.45
2	50%	72	NRCS IA	2.7	III/90	135	1.71	65	383.6	0.49
10	10%	288	Feb '96	13.0	II/78	152	9.61	116	385.0	0.76
25	4%	72	NRCS IA	7.0	II/78	353	4.46	161	387.1	0.46
100	1%	24	NRCS IA	4.7	II/78	377	2.46	143	386.0	0.38
100	1%	72	NRCS IA	8.3	II/78	454	5.66	186	389.0	0.41
100	1%	72	Feb '96	8.3	II/78	348	5.73	158	386.9	0.45
100	1%	168	NRCS IA	13.2	II/78	358	9.68	193	389.6	0.54
500	0.20%	72	Feb '96	9.8	II/78	421	7.14	175	388.1	0.42
500	0.20%	24	NRCS IA	5.5	II/78	500	3.14	171	387.8	0.34
100	1%	24	NFF	Unk *	Unk	803	Unk	Unk	Unk	

Notes:
In final run included 0.2% AEP Storm to check freeboard
No Overflows

Figure 5.12 Pond Routing Summary

The number and range of "test storms" is arbitrary but should include any storm likely to occur during the service life of the detention basin. With the aid of such programs as HydraFlow the calculations for pond routing take very little time so quite a wide array of test storms may be quickly investigated and summarized as hasl been done in **Figure 5.12, above**.

Step 6 Review routing results

This is a critical step and will guide the final hydraulic design in many ways. Unlike prescriptive standards, this step requires thoughtfiul analysis, judgement and searching for alternatives, opportunities and possible errors. This requires more work than simply applying a set of fixed rules to a problem but the reward for this extra effort is almost always a better design.

For each of the selected test storms we can then ask:

What peak flow reduction is necessary to protect downstream properties?

Does the basin attenuate peak flow by at least 30 %?
If not, can the outflow be accomodated by the existing downstream system or can that system be economically upgraded ?

Does the Basin overflow ?

If the overflow would cause little or no damage, proceed to Final Design
If overflow would be unacceptable, resize the pond or enlarge or decrease the size of the outlet control device and re-run the routings until the performance of the basin is acceptable for all but the most extreme rain events.

No pond can be economically built to meet any possible extreme event. The only thing we can do is minimize the risk, but that risk can never be zero.

Looking at **Figure 5.12** it is apparent that our trial basin is about large enough. During only one of the test storms does it come close to overflow; namely during the 168 hour , 1% AEP storm if the rainfall pattern is close to matching the SCS IA distribution. Given the errors inherent in any uncalibrated hydrolgy model we should consider this the critical storm for design since overflow is very likely.

We can now move to Step 7.

Step 7 - Final (Almost) Model Calibration

Although listed last, Step 7 is the only step which can ensure a reliable model. Thus it is probably the most important difference between this and the numerous other design approaches in use.

To calibrate the model it will be necessary to use existing rain gages in either the West Pringle Creek Basin or an adjacent basin. At least one, and preferably three, recording rain gages and recording flow meters should be used.

This step is not easy to accomplish and requires a great deal of work .The details of the calibration are not covered in this course but calibration is essential if we are to have confidence in our model. **Reference 11** is an example of model calibration which will suggest to you the complexity of the process.

The meters should be monitored before, during and after completion of the basin design and will ensure that the chosen hydraulic model (NRCS – 484) is correctly converting rainfall to runoff. If not, it should be possible to adjust the model or switch to a model which produces closer results .

Once the model is calibrated we can then proceed with some confidence to a final design .

Final design should include consideration of at least the following:

Determine final dimensions, shape and volume and a rerun a final test series
Determine Outflow control devices (fixed or variable). Simpler is better and self operating is better than mechanical
Groundwater Recharge potential or hazards
Environmental concerns
Water Quality enhancement potential

Fish Passage, if applicable
Aesthetics
Ease of Maintenance
Operation/Automation/Controls
Possible multiple uses of the facilities
Impacts upon adjacent land uses
Consequences of hydraulic failure
Consequences of structural failure
Calculate the probability of the design flow being equaled or exceeded within the lifetime of the structure .

This design is “final” only in the sense that the needed storage volume has been determined. The final construction plans must include the actual size and shape of the pond, details of the outlet control structures and a great deal more.

The purpose of the last Hydraflow run illustrated in **Figure 7.1** is to “test” the limits of performance of this particular detention pond. The question posed is:

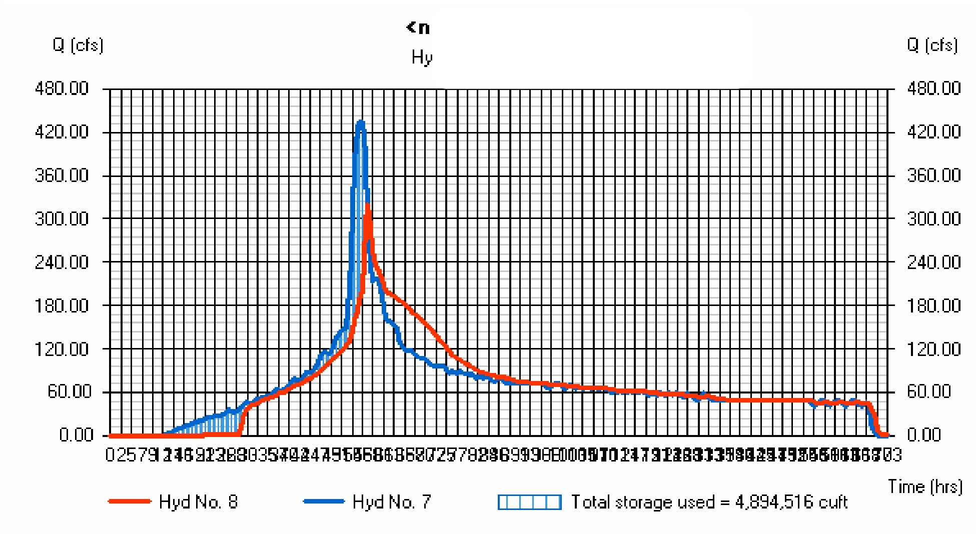
“When Will It Fail ?”

<Fig71.xls>

Project: WOODMANSEE PARK
Location: City of Salem, OR
Client: PDH Online

Date: 11-1(
Design: rw
Check: rp
Job No.

Hydrograph Routing through Pond
540 Year - 168 Hour - Type IA Storm - AMC II



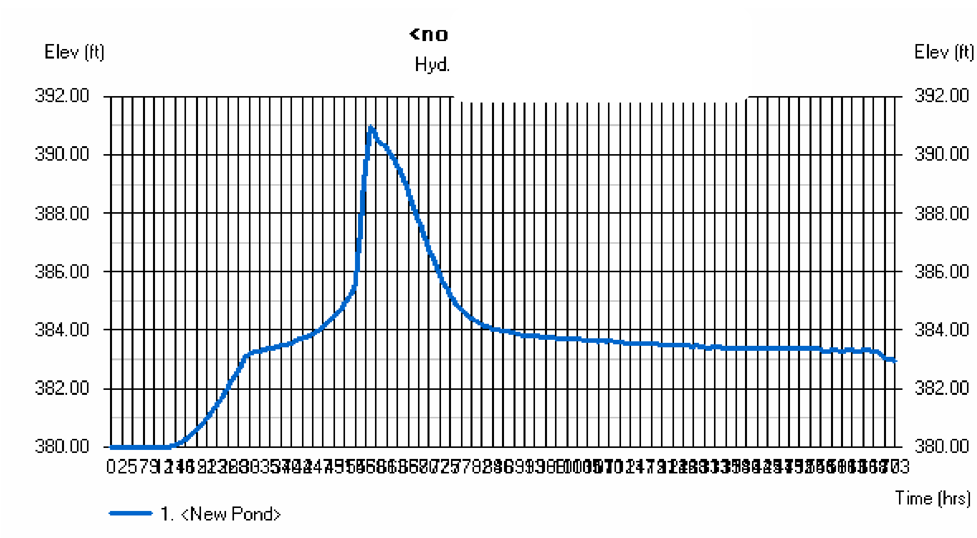


Figure 7.1 - Hydrograph Routing

We'll define hydraulic failure as uncontrolled overflow. Based on our pond design water surface with 1 foot of freeboard this would occur if the pond reached an elevation of 391.00 feet.

After several trials it may be seen that a 168 hour storm with 15.5 inches of rain comes very close to producing ultimate hydraulic failure of the pond. The Type IA rainfall distribution was used for this routing. It results in the water level in the pond rising to 390.95 feet; within .05 feet of complete overflow. Any further rise in water level would result in uncontrolled overflow with probable damage to downstrean structures and properties which might be vulnerable to water damage.

Does this warrant an increase in "free board" ?

From a rough extrapolation of our Rainfall-Duration- Frequency curves (**Fig. 3-1**) we can estimate that the "n" value of this storm is approximately 540. Or, stated another way, the Annual Exceedence probability of this storm is 1 / 540 = .00185 or 0.185 percent.

If we assume a useful life of 50 years for the pond we can calculate the probability of this storm being equaled or exceeded in 50 years using:

$$P_x = 1 - (1 - \frac{1}{n})^x$$

Where: P_x is the probability of occurrence in x number of years

1/n = the Probability of Occurrence in any one year expressed as a decimal percent
"n" is the so called "return period"

For example, if we want to calculate the probability of occurrence of the 540 year flood over the next 50 years the calculations would be:

$$P_{50} = 1 - (1 - \frac{1}{540})^{50}$$
$$P_{50} = 1 - (1 - 0.00185)^{50}$$
$$P_{50} = 1 - (0.99815)^{50}$$
$$P_{50} = 1 - (0.91157)$$
$$P_{50} = 0.0884$$

In other words, there is only a 8.84, say 9 percent, probability that the "ultimate failure storm" will occur one or more times over the next 50 years.

This is equivalent to saying that we can be 91% confident that the system <u>will not</u> fail in the next 50 years This is not "certainty" but it is a very high level of confidence for a hydraulic structure and confirms that the basis of the design is indeed conservative.

Also, it suggests that an increase in freeboard is probably not necessary.

Appendix A – Why NRCS- 484 ?

In the late 1960s and early 1970s, the then Soils Conservation Service (SCS) developed computer models for the hydrologic analysis of small urban watersheds. These early programs preceeded the proliferation of personal computers which are now as ubiquitous as the slide rule was in the 1950s and 1960s.

The main models, known as TR-20 and TR-55 have become familiar to Engineers, Hydrologists and others over the decades and have been "ported over" to PCs so that they are now readily available and enjoy near universal acceptance. More than that, they are essentially "freeware", having been paid for by U.S. taxpayer dollars.

The SCS later became the NRCS (National Resource Conservation Service) but the models have been refined, improved and thoroughly tested by millions of users.

The NRCS hydrology is as easy to understand as the simpler Rational Method and is only a little more complicated to use. The data necessary to create a storm hydrograph in NRCS - 484 consists of a rainfall and pattern, some knowledge of the soils and land cover of the drainage basin, and some information about the frequency of the storm. All of these are easy to find and verify and much of it is available on the internet.

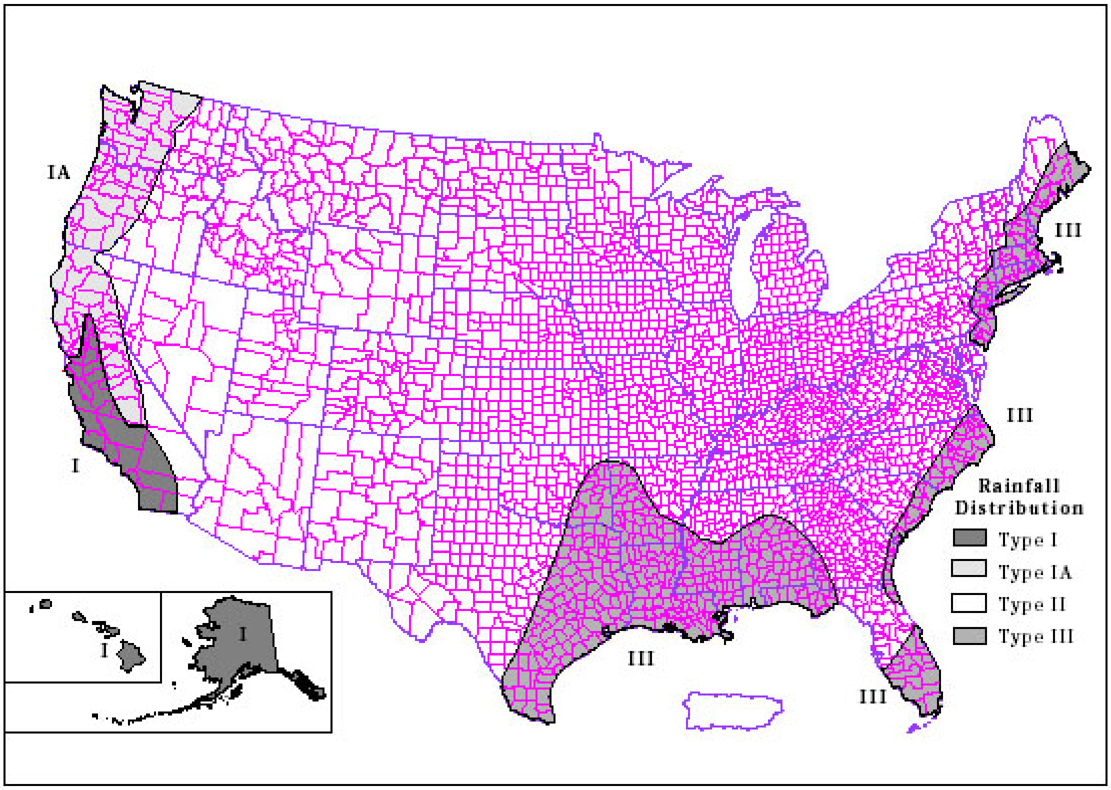
Available mapping is usually similarly available. For most projects in the United States, U.S.G.S Quadrangle maps are sufficiently detailed for hydrologic studies. If better mapping is available it can be used with some improvement in the results.

Weather data too is available, often for free on the internet. The quality of that data is, and always will be, the limiting factor on the accuracy of the estimates made from it. As an example, the data in **Figure 3-2** was downloaded from NOAA's Hydrometerological Studies Center.

Unfortunately, data at this level of detail is not yet available for the Oregon example chosen for this course. If it was, we can be reasonably certain that it too would contain errors of estimate at the 90% confidence level of the same order of magnitude (+ or – 20 %) . For example, the 1% AEP – 24 hour storm for Salem is believed to be about 4.7 inches. The NOAA data suggests that this number should be viewed as having an error of plus or minus 20 percent. Thus Salem's 4.7 inches should be considered accurate only in the range from 3.8 inches to 5.6 inches. Any flows derived from the 4.7 inch figure cannot be any more accurate than this .

Perhaps the greatest weakness of the NRCS – 484 method is its lack of a purely scientific basis. It is not based on first principles of science but rather, is derived from experiments , hunches, trials and a few lucky guesses along the way. It is what Engineers love and scientists hate , an empirical method. In fact, it is only a little more "scientific" than the Rational Method.

<u>But this disadvantage is largely compensated for by the fact that the method can be calibrated to closely match the behavior of any real world watershed.</u>
Also, the NRCS method provides "storm types" based on geographic location as illustrated on the map below:



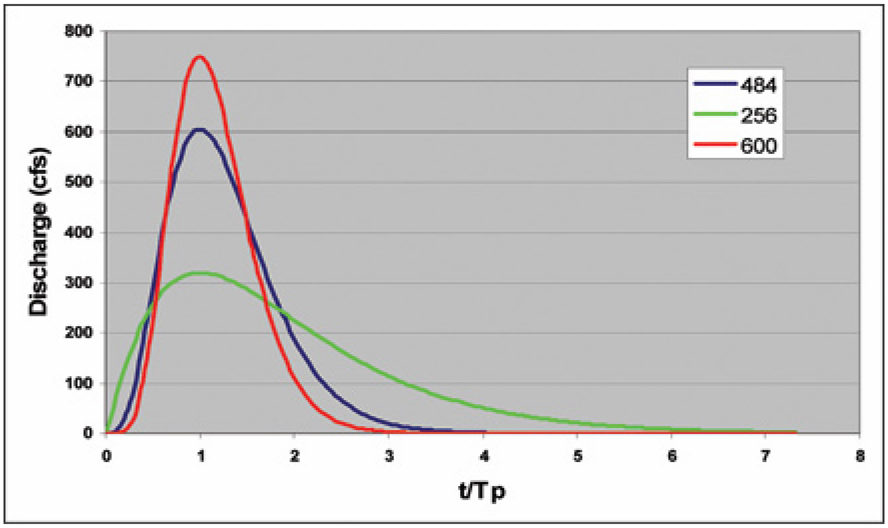
Before returning to the original question, one other side issue may be dealt with. The "484" in the name of the method is what the NRCS calls the "attenuation factor". It may be thought of as a kind of average for a typical urban drainage basin with rolling topography. For other situations, some investigators suggest the following adjustments:

Basin Characteristics	Attenuation Factor
Rural, flat	150
Rural, gently sloping	200
Rural, rolling hills	300
Mixed urban and rural	400
Mixed urban-rural, gently sloping	484
Urban, steeply sloping	575
Rational Formula	645

Because most developable urban basins are gently sloping and a mix of rural and urban land uses the 484 factor seems appropriate to use for the "developed" condition. But for the pre-developed conditions it may be advisable to adjust this factor to something closer to 300 to 150.

This all suggests that calibrating the NRCS – 484 model may be as simple as adjusting this attenuation factor. Unfortunately, it is NOT that simple (Reference 9).

We can't know this without the network of rain and flow gaging stations recommended in the main body of this course.



The graph above is taken from Reference 10 and illustrates the effect of the attenuation factor.

Back to the main question; Why NRCS - 484 ?

1. It is widely available and free
2. It is well understood and easily checked
3. It is as "scientific" as any other available method
4. It demands little data that is not easily accessible
5. It can, and has been, programmed for use on almost any Personal Computer
6. It is capable of performing an almost unlimited number of "what if " scenarios in a short period of time
7. It has few limitations on basin size and may be used for basins as large as several thousand acres
8. It can be calibrated to produce reasonable results under almost any "real life" situation

The following is quoted directly from Reference 10, Voodoo Hydrology.:

" Conclusions and Recommendations

"What conclusions can we draw from all this talk of voodoo hydrology? One colleague, after reading this article, considered a career change to day trading—reasoning it had fewer unknowns and was less risky.

"First of all, we must understand that urban hydrology is an inexact science where we are simply trying to get close to the right answer. We are dealing with probabilities and risk, a changing land-use environment, and many real-world factors that can alter the answer. The applications we may encounter can vary radically. Therefore, it behooves us to better understand the inner workings of the black boxes we commonly use. And we should understand how the common computer packages we use for design employ these methods.

"As local governments, we should establish, very carefully, appropriate application of the most common tools (these and others) and require consistent adherence to the best science available. We should not be reluctant to disallow inappropriate practices or application packages even if they have had long use in our local community. For example, use of an unaltered Modified Rational Approach for detention design should be checked against better methods and adjusted as appropriate to give reasonable design parameters. Nor should we employ unrealistic overdesign in an attempt to cover all eventualities. I know of one community that required 100-year in-bank ditches everywhere within a residential subdivision. The result looked like miniature grand canyons everywhere with a meandering trickle stream somewhere way down in the bottom attacking the toe of the crumbling banks.

"Specific recommendations include:

1. To the extent practical, local communities or collections of communities should seek to "calibrate" standard methods to local conditions. This can be done using measured data and regression equations. Alternately, other more appropriate hydrologic methods can be used as a substitute for parts of the common methods. For example, some communities use different rainfall distributions, infiltration methods, or modified Curve Number charts.
2. Local communities must understand how voodoo practitioners can "cheat" with the various methods and must establish ranges of applicability for the "knobs" in the various methods. For example, does sheet flow really travel 300 feet across grassy areas on steeper slopes? Not in this universe.
3. For the most important hydraulic structures in the community (e.g., large ponds, dams, channels), more exacting standards should be required, including continuous simulation and making maximum use of measured data.
4. Use of "percent removal" criteria should be carefully qualified using the best science and monitoring information available—again calibrated to the local area.
5. A local community should recognize that not all structural controls are created equal and should establish a pollution-reduction criterion that is reasonably effective and does more than hand waving. Define MEP in a way that can be reasonably attained, reasonably maintained, and easily reviewed. No one wants to argue over every site.

"And, finally, look to further automation in our ability to simply and accurately perform better and more accurate calculations. The day is fast approaching when we all will be able to model whole systems using continuous simulation models hard-wired to our local area, use drag-and-drop design approaches, practice "what if" analysis on the fly, perform effective reviews, and let the computer do the crunching while we do the thinking."

Little has changed in hydrology since 1873 when Emil Kuichling first proposed (in the U.S.) the Rational Method. But our ability to juggle numbers rapidly has markedly increased. If we can now increase our <u>understanding</u> of those numbers we will have contributed to the general store of knowledge.

References

**1. The SMADA (Stormwater Management and Design Aid) programs were written to accompany the textbook
Hydrology: Water Quantity and Quality Control 2nd Edition by
M.P. Wanielista, R. Kersten, and R. Eaglin. The text is
available from John Wiley and Sons publishers.**

2. Hydro Cad HydroCAD Software Solutions LLC

3. Hydraflow Hydrographs Intellisolve

"October 18, 2007

"Thank you for visiting the Intelisolve website. We are happy to report that Intelisolve's business has been acquired by Autodesk, Inc., and we will be joining the Autodesk team. We look forward to working with Autodesk to take the Intelisolve technology to the next level.
"Autodesk will provide an update on product availability in the near future.

"Best Regards,

The Intelisolve Team"

4. NFF – The National Flood Frequency Program

ABSTRACT

Estimates of the magnitude and frequency of flood-peak discharges and flood hydrographs are used for a variety of purposes, such as the design of bridges, culverts, and flood-control structures, and for the management and regulation of flood plains. These estimates are often needed at ungaged sites where no observed flood data are available.
To provide simple methods of estimating flood-peak discharges, the U.S. Geological Survey (USGS) has developed and published regression equations for every State, the Commonwealth of Puerto Rico, and a number of metropolitan areas in the United States. These equations have been compiled into the National Flood Frequency (NFF) Program.

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