Stormwater Management

Pond Design Example

For

Extended Detention Wet Pond

Richard A. Claytor, Jr., P.E.

The Center for Watershed Protection
8737 Colesville Road, Suite 300
Silver Spring, MD 20910

December 1995

Acknowledgement:

The preparation of this document was made possible, in part, through a cooperative Agreement with the U.S. Environmental Protection Agency. The contents do not necessarily reflect the views and policies of EPA, nor does the mention of trade names or commercial products constitute endorsement or recommendation for use.

The author would like to thank the following individuals who contributed to the preparation of this document: Mr. Tom Davenport, Mr. John Galli, Mr. Dan Rogers, Mr. Tom Schueler, and Mr. Mike Wiley. In addition, the author would like to extend special appreciation to Mr. Tim Schueler for his insightful comments and feedback.

The following stormwater management pond design example sets forth a step-by-step approach for the design of a stormwater management wet extended detention pond in the mid-Atlantic piedmont region of the United States. The design criteria is representative of one jurisdiction within this region. The design approach will be adaptable to other regions of the country and other jurisdictions, but the specific criteria will vary from jurisdiction to jurisdiction. Designers must recognize this in adapting this to the specific design criteria.

Stormwater Management
Pond Design Example

Table of Contents

DESIGN EXAMPLE:

Introduction

A developer is planning on developing a townhouse project located in suburban Maryland, approximately 30 miles northwest of Washington D.C. in Frederick County. This development is within the Bush Creek watershed which is tributary to the Monocacy River (see Figure 1). The site consists of a 51 acre area draining an approximately 70 acre watershed. The project site is 7500± feet upstream from a County road and there are no houses or other structures downstream from the property. Since the project is draining to Bush Creek, a Maryland Use I Waterway (warm water fishery), a wet pond with extended detention (ED) is proposed to provide both water quality and water quantity controls. The site is an existing pasture which is drained via a shallow swale. There are no existing wetlands present on the property and the site is in agricultural production. Therefore, there will be no disturbance to existing forests. The development proposal is to construct approximately 335 townhouses, a network of roadways and parking lots and a community center. The developer has retained you to design the stormwater management facility to serve the entire project and to obtain the necessary approvals and permits.

First Steps

The initial steps are to assemble the necessary information to design the facility and to confirm the local design criteria. It may be wise to set up a meeting with the local reviewing agency to confirm the local design criteria. The purpose of this meeting should be to establish the requirements for water quality storage, attenuation requirements for water quantity controls, safety storm requirements and the pond "hazard classification" analysis requirements prior to making wrong assumptions and having to re-do design work. The designer should also visit the site whenever possible to familiarize themselves with the specific characteristics of the site, and to ensure that design data provided reflects the current conditions in the field.

Since this facility is located in Frederick County, Maryland the following criteria will be used for the design example: Note that different jurisdictions have different criteria, It is up to the designer to be familiar with the applicable criteria where he/she is working.

Water Quality

A) Permanent Pool

The Water Quality Treatment Volume (WQV) is computed using a locally acceptable method (e.g., 1.25" x R_v x DA). where: R_v is defined as the volumetric runoff coefficient and DA is the drainage area.

Fifty percent of WQV is then allocated for the permanent pool volume and 50% is allocated for extended detention (ED).

1) The permanent pool maximum depth should not generally exceed 8 feet, with a average

depth of 4 to 6 feet. The length to width ratio should be approximately 2:1. Shallow aquatic shelves should be provided for planting aquatic vegetation over an area of approximately 10% of the permanent pool surface area and along 70% of the shoreline. A 15 foot wide safety bench should be provided at the extended detention water surface elevation.

2) The sediment forebay for a wet ED pond should be sized for a minimum of 0.1" per impervious acre of drainage area with an optimal volume of 0.25" per impervious acre. The forebay volume should be in addition to the permanent pool volume and should consist of a separate cell, formed by an earthen berm, gabion or concrete weir wall.

B) Extended Detention

Fifty percent of WQV is allocated for the extended detention (ED) volume. The recommended maximum ED water surface should be limited to three feet above the permanent pool elevation. This minimizes shoreline disturbance caused by changing water surface elevations.

Stormwater Quantity Management

For the 2 year and 10 year storms the maximum release rate after development can be no larger than the pre-developed 2 year and 10 year peak discharge from the project site. The principal spillway must pass at least the 5 year storm.

Safety Storm

For the 100 year storm, the pond must safely pass the post-developed peak discharge with a minimum of one foot of freeboard (for this example, for ponds without an emergency spillway a two foot minimum freeboard is required).

The minimum information needed for the hydrologic analysis of this example is as follows:

1. Topography of watershed limits, USGS 7.5 series is usually acceptable.
2. USDA Soils Survey for county in question (Frederick Co. MD for this example).
3. Zoning or Land Use maps depicting ultimate land uses (schematic development plan for this example with off-site land use defined).
4. Exiting vegetation conditions (recent aerial photograph is often acceptable).
5. Topographic information or detail of downstream conditions for "hazard classification" analysis.

The first task is to identify the watershed boundary, transfer these limits to the soils survey map, and planimeter the area to establish the total drainage area to the pond site. For our example, this area is 72.4 acres (See Figure 1). Note that some of the contributing area to the pond is from off-site areas, these areas must be accounted for in your design calculations. This analysis will be discussed further in the computation of the runoff Curve Number (CN).

Hydrologic Computations

There are several acceptable hydrological methods utilized in the design of ponds. The Unit Hydrograph Method, SCS Methods, USGS Methods are a few examples. For this example, SCS TR-55, "Urban Hydrology for Small Watersheds" (June 1986) and SCS TR-20, "Project Formulation, Hydrology" are used to compute inflow hydrographs and storm routings. Other computer simulation models are available to compute inflow hydrographs. Examples include: The Army Corps of Engineer's, HEC-1; U.S. EPA, SWMM; The Penn State Runoff Model (PSRM), to name a few.

Runoff Curve Number (CN)

The initial variable which needs to be computed is the CN for both pre-developed and post-developed conditions. To compute CNs, the hydrologic soil groups (HSG) and land use are needed. The USDA, Soils Surveys delineate and label (soil symbol only) soil types for each county in the country. The designer must match up the symbol with the name, apply the appropriate HSG, and compute the area. TR-55, Appendix A, provides the HSG designation for United States soils. Figure 1 shows the soil symbols, HSG and watershed boundary for this example.

Once this step is complete, the pre-developed and post-developed land uses need to be determined. The pre-developed land use can be determined by reviewing a recent aerial photograph of the area and is usually combined with a field visit to the site. Some jurisdictions allow pre-developed conditions to be "present condition", that is, whatever the existing land use is at the time of plan preparation. Other jurisdictions require a land use representative of a undisturbed area such as "woods" or "meadow". This criteria will usually provide for a more restrictive release rate and requires more quantity control storage. To determine the future land use, the designer must refer to a zoning or ultimate land use map. In our example the ultimate allowable land use proposed is a townhouse development. See Figure 1.

Since some of the drainage area to the pond comes from off-site, the designer must consider how this will be handled in sizing the pond. Most jurisdictions do not require off-site areas to be managed by a proposed facility. Some jurisdictions require management for the total drainage area to a pond. When this is the case, only two alternatives need to be analyzed, pre-developed and post-developed conditions for both on-site and off-site.

A common approach is to consider the off-site area as "present condition" when computing the pre-developed and post developed conditions for the storm frequencies which require management (the 2 year and 10 year in our example). The "ultimate condition" must be analyzed separately for the safety storm frequencies (the 100 year storm for our example). For this condition, the entire watershed is considered ultimately-developed. This approach assumes that on-site management (back to present or pre-developed conditions) will be provided by future ponds for those off-site properties which drain to the proposed facility, but that off-site management will not provide for the safety storm event. In our example, it has been assumed that the off-site areas are in meadow in the pre-developed condition to compute a more restrictive release rate and therefore a more effective management strategy.

It should be noted that in our example, only one homogeneous watershed for both pre-developed and post-developed conditions is analyzed. The shape of the watershed is fairly uniform and there is predominately only one land-use. If the shape or land use distribution is not uniform, more than one subarea will likely be necessary.

Time of Concentration (t_c)

The next step is to compute the time of concentration (t_c) for both the pre-developed and post-developed conditions. The travel time (t_t) for this example is 0 since there is only one homogeneous watershed. The procedure for computing the t_c is outlined in detail in Chapter 3 of TR-55. There are a few rules of thumb that are worth noting. First, the location of the travel path should be one which is representative of the majority of the drainage area. The length of overland sheet flow should be limited to 150 feet for pre-developed conditions and 100 feet for post-developed conditions. This appears to more closely represent what is actually taking place in the field. It is very rare that water doesn't concentrate within these distances. The designer should be able to justify lengths beyond those stated above. Finally, in computing channel velocities, the designer should verify the typical cross-sectional geometry for both pre-developed and post-developed conditions. Notice should be taken that the pre-developed condition may represent a condition not as degraded as the current conditions and therefore the existing channel geometry may be more disturbed than is desired to achieve satisfactory stormwater controls.

Inflow Hydrographs

The final step in the hydrologic analysis involves computing the pre-developed and post-developed inflow hydrographs. In our example, the computer program TR-20 has been used. The pre-developed rate is computed to determine the required release rate from the facility. The post-developed inflow Hydrograph will form the basis of computing the required storage for management of the selected storm frequencies (2 year and 10 year storms). The input requirements for TR-20 are reasonably simple. To compute the runoff hydrographs, the only input data requirements are drainage area (DA), curve number (CN) and time of concentration (t_c). The TR-20 user manual "Project Formulation, Hydrology" provides detailed guidance on the input requirements. The following pages represent the CN, t_c, and drainage area analysis followed by the TR-20 printout with the complete inflow hydrographs for the 2,10 and 100 year storms for pre-developed, post-developed, and ultimate-developed conditions.

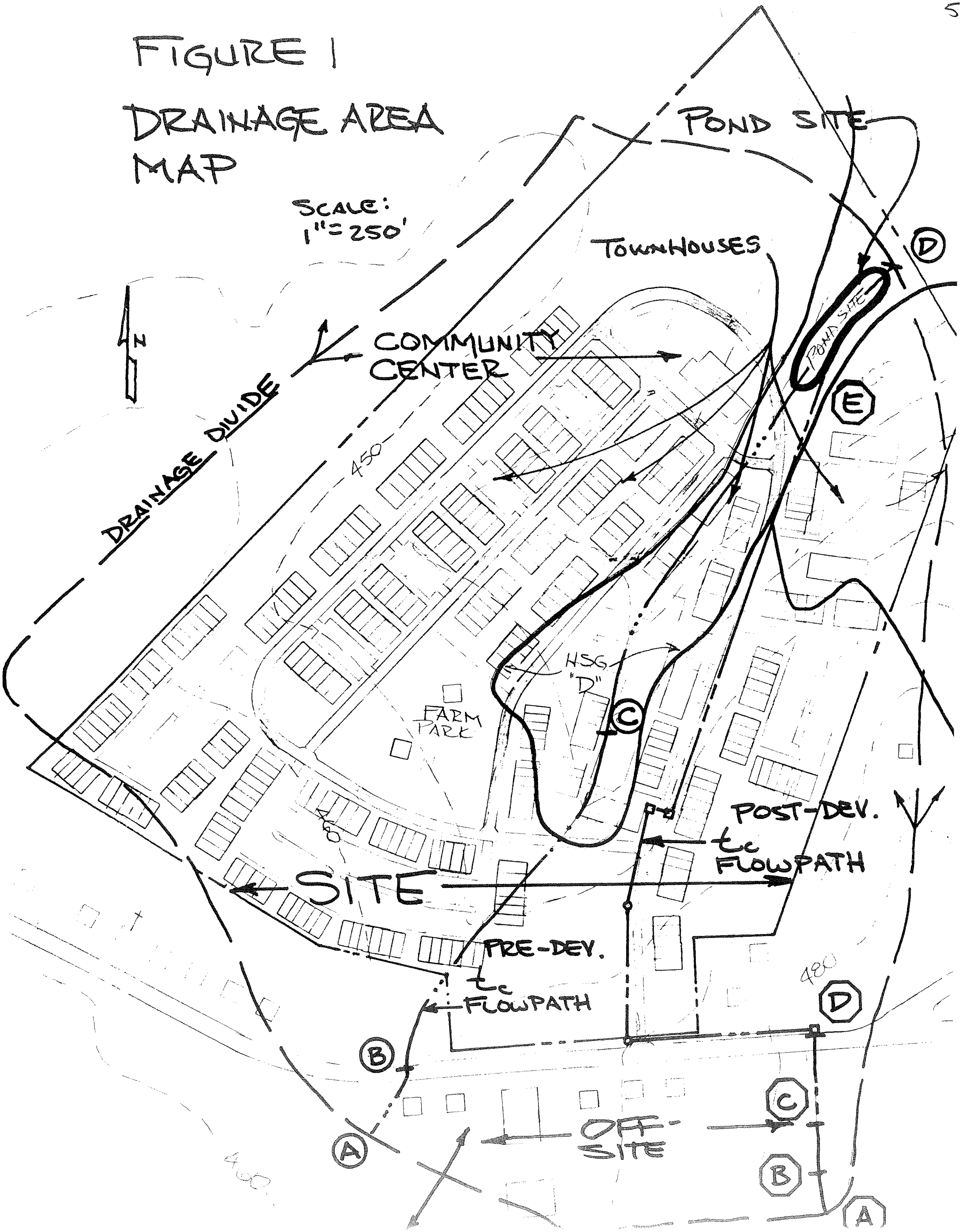


FIGURE 1
DRAINAGE AREA MAP
SCALE: 1"=250'
N
POND SITE
TOWNHOUSES
D
COMMUNITY CENTER
DRAINAGE DIVIDE
POND SITE
E
450
HSG "D"
C
FARM PARK
POST-DEV. tc FLOWPATH
SITE
PRE-DEV. tc FLOWPATH
480
D
B
C
OFF-SITE
A
B
A
460

G

Worksheet 2: Runoff curve number and runoff

Project DESIGN EXAMPLE By RAC Date 12/94

Location FREDERICK CO. MD Checked ____ Date ____

Circle one: Present Developed PRE-DEV. (MEADOW)

1. Runoff curve number (CN)

Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN [1/] Table 2-2	CN [1/] Fig. 2-3	CN [1/] Fig. 2-4	Area ☒ acres ☐ mi^2 ☐ %	Product of CN x area
GLENELG CHESTER ELIOAK MANOR B	MEADOW (GOOD)	58			60.8	3526.4
LINGANORE C	MEADOW (GOOD)	71			4.4	312.4
WORSHAM D	MEADOW (GOOD)	78			7.2	561.6
				Totals =	72.4	4400.4

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{4400.4}{72.4} = \underline{60.78}$; Use CN = $\boxed{61}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	100
Rainfall, P (24-hour) (FRED. CO. MD.)	in	3.1	5.0	7.0
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	0.40	1.37	2.70

Worksheet 2: Runoff curve number and runoff

Project DESIGN EXAMPLE By RAC Date 12/94

Location FREDERICK CO. MD Checked ______ Date ______

Circle one: Present Developed POST DEV. CONDITIONS (ONSITE ONLY)

• FOR 2 AND 10 YEAR MANAGEMENT

1. Runoff curve number (CN)

	Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN 1/ Table 2-2	CN 1/ Fig. 2-3	CN 1/ Fig. 2-4	Area ☒ acres ☐ mi^2 ☐ %	Product of CN x area
ONSITE 49.6 AC	GLENELG CHESTER ELIOAK MANOR B	TOWNHOUSES; 45% IMP. ROADS (GOOD H. COND)	* 78			36.8	2870.4
	GLENELG CHESTER ELIOAK MANOR B	COMMUNITY CENTER; (POOL, BATHHOUSE, ETC) 65% IMP. (GOOD H.C.)	85			1.6	136.0
	LINGANORE C	TOWNHOUSES: 45% IMP. (GOOD H. COND.)	85			4.0	340.0
	WORSHAM D	TOWNHOUSES: 45% IMP. (GOOD H.C.)	88			6.7	589.6
	WORSHAM D	COMMUNITY CENTER 65% IMP. (GOOD H.C)	92			0.5	46.0
OFFSITE 22.8 AC	B	ASSUMED AS PRESENT CONDITION (10% IMP)	65			22.4	1456.0
	C	ASSUMED AS PRESENT CONDITION (PASTURE)	74			0.4	29.6
					Totals =	72.4	5467.6

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{5467.6}{72.4} = 75.52$; Use CN = $\boxed{76}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr	2	10	
Rainfall, P (24-hour)	in	3.1	5.0	
Runoff, Q .... (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in	1.08	2.53	

* SAMPLE CN COMP.: CN = 0.45(98) + 0.55(61) = 77.65
USE 78

Worksheet 2: Runoff curve number and runoff

Project DESIGN EXAMPLE By RAL Date 12/94

Location FREDERICK CO. MD Checked ____ Date ____

Circle one: Present Developed ULTIMATE CONDITIONS (OFFSITE + ONSITE)

FOR 100 YEAR SAFETY STORM

1. Runoff curve number (CN)

	Soil name and hydrologic group (appendix A)	Cover description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connected impervious area ratio)	CN [1/] Table 2-2	CN [1/] Fig. 2-3	CN [1/] Fig. 2-4	Area ☒ acres ☐ mi^2 ☐ %	Product of CN x area
ONSITE 49.6 AC	→	FROM PREVIOUS COMPUTATION (FOR 2 AND 10 YR MANAGEMENT)	80.28			49.6	3982.0
OFFSITE 22.8 AC.	B	COMMERCIAL (85% IMP)	92			6.6	607.2
	B	TOWNHOUSES (45% IMP)	78			15.8	1232.4
	C	TOWNHOUSES (45% IMP)	85			0.4	34.0
					Totals =	72.4	5855.6

1/ Use only one CN source per line.

CN (weighted) = $\frac{\text{total product}}{\text{total area}} = \frac{5855.6}{72.4} =$ 80.87; Use CN = $\boxed{81}$

2. Runoff

		Storm #1	Storm #2	Storm #3
Frequency	yr			100
Rainfall, P (24-hour)	in			7.0
Runoff, Q (Use P and CN with table 2-1, fig. 2-1, or eqs. 2-3 and 2-4.)	in			4.81

(210-VI-TR-55, Second Ed., June 1986)

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project DESIGN EXAMPLE By RAC Date 12/94

Location FREDERICK CO, MD Checked ____ Date ____

Circle one: Present Developed PRE-DEV. CONDITIONS

Circle one: T_c T_t through subarea

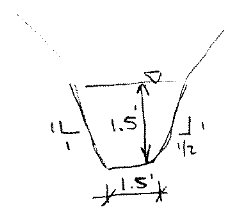
NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)		Segment ID		A-B			
1.	Surface description (table 3-1)			DENSE GRASS			
2.	Manning's roughness coeff., n (table 3-1)			0.24			
3.	Flow length, L (total L $\leq$ 300 ft)		ft	150'			
4.	Two-yr 24-hr rainfall, P_2		in	3.1"			
5.	Land slope, s		ft/ft	0.02			
6.	$T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\, s^{0.4}}$	Compute T_t	hr	0.33	+	—	= 0.33

Shallow concentrated flow		Segment ID		B-C			
7.	Surface description (paved or unpaved)			UNPAVED			
8.	Flow length, L		ft	820			
9.	Watercourse slope, s		ft/ft	.045			
10.	Average velocity, V (figure 3-1)		ft/s	3.4			
11.	$T_t = \frac{L}{3600\ V}$	Compute T_t	hr	0.07	+	–	= 0.07

Channel flow		Segment ID		C-D			
12.	Cross sectional flow area, a		ft^2	3.94			
13.	Wetted perimeter, p_w		ft	5.30			
14.	Hydraulic radius, $r = \frac{a}{p_w}$	Compute r	ft	0.743			
15.	Channel slope, s		ft/ft	.027			
16.	Manning's roughness coeff., n			.04			
17.	$V = \frac{1.49\, r^{2/3}\, s^{1/2}}{n}$	Compute V	ft/s	5.0			
18.	Flow length, L		ft	1100			
19.	$T_t = \frac{L}{3600\ V}$	Compute T_t	hr	0.06	+	–	= .06
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)		hr				0.46



Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project DESIGN EXAMPLE By RAL Date 12/94

Location FREDERICK CO MD Checked ______ Date ______

Circle one: Present Developed ULTIMATE DEVELOPMENT.

Circle one: T_c T_t through subarea

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

	Units				Total
Segment ID		A-B			
1. Surface description (table 3-1)		GRASS			
2. Manning's roughness coeff., n (table 3-1) ..		0.15			
3. Flow length, L (total L ≤ 300 ft)	ft	100'			
4. Two-yr 24-hr rainfall, P_2	in	3.1			
5. Land slope, s	ft/ft	0.02			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t	hr	0.17	+	–	= 0.17

Shallow concentrated flow

	Units				Total
Segment ID		B-C		C-D	
7. Surface description (paved or unpaved)		UNPAVED		PAVED	
8. Flow length, L	ft	100'		200	
9. Watercourse slope, s	ft/ft	.02		.02	
10. Average velocity, V (figure 3-1)	ft/s	2.3		2.9	
11. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.01	+	0.02	= 0.03

Channel flow

*ASSUME AVG S.D. SIZE = 24'

	Units				Total
Segment ID		D-E*			
12. Cross sectional flow area, a	ft^2	3.14			
13. Wetted perimeter, p_w	ft	6.28			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft	0.50			
15. Channel slope, s	ft/ft	.028			
16. Manning's roughness coeff., n		0.013			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s	12.1			
18. Flow length, L	ft	1900			
19. $T_t = \frac{L}{3600\ V}$ Compute T_t	hr	0.04	+	–	= 04
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr				0.24

TR-55 COMPUTER VERSION - PRE-DEV. CONDITIONS

PEAK DISCHARGE SUMMARY

JOB:	DESIGN EXAMPLE			RAC
DRAINAGE AREA NAME	TOTAL DA, PRE-DEVELOPED COND			30-Mar-95
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
MEADOW	GLENELG	B	58	60.80 Ac.
MEADOW	LINGANORE	C	71	4.40 Ac.
MEADOW	WORSHAM	D	78	7.20 Ac.
		AREA SUBTOTALS:		72.40 Ac.

Time of Concentration 2-Yr 24 Hr Rainfall = 3.1 In	Surface Cover Cross Section	Manning 'n Wetted Pe	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	dense grass	'n'=0.24	150 Ft.	2.00% 0.33 Hrs
Shallow Flow	UNPAVED		820 Ft. 3.42 F.P.S.	4.50% 0.07 Hrs.
Channel Flow Hydraulic Radius =0.74	3.9 SqFt	'n'=0.040 5.3 Ft.	1100 Ft. 5.02 F.P.S.	2.70% 0.06 Hrs.
Total Area in Acres =	72.40 Ac.	Total Shee	Total Shallow	Total Channel
Weighted CN =	61	Flow=	Flow=	Flow =
Time Of Concentration =	0.46 Hrs.	0.33 Hrs.	0.07 Hrs.	0.06 Hrs.
Pond Factor =	1	RAINFALL TYPE II		

STORM	*Precipitation* (P) inches	*Runoff* (Q)	Qp, PEAK DISCHARGE	TOTAL STORM Volumes
1 Year	**2.5 In.**	**0.2 In.**	**5.3 CFS**	51,480 Cu. Ft.
2 Year	**3.1 In.**	**0.4 In.**	**15.8 CFS**	106,125 Cu. Ft.
5 Year	**4.0 In.**	**0.8 In.**	**40 CFS**	213,529 Cu. Ft.
10 Year	**5.0 In.**	**1.4 In.**	**74 CFS**	359,817 Cu. Ft.
25 Year	**5.4 In.**	**1.6 In.**	**89 CFS**	424,538 Cu. Ft.
50 Year	**6.1 In.**	**2.1 In.**	**117 CFS**	544,736 Cu. Ft.
100 Year	**7.0 In.**	**2.7 In.**	**156 CFS**	710,104 Cu. Ft.

PEAK DISCHARGE SUMMARY				
JOB:	DESIGN EXAMPLE			RAC
DRAINAGE AREA NAME	TOTAL DA, POST-DEVELOPED COND.			30-Mar-95
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
ONSITE				
TOWNHS. (45% IMP)	GLENELG, et.al	B	78	36.80 Ac.
COMM CNTR (65% I)	GLENELG, et.al.	B	85	1.60 Ac.
TOWNHS. (45% IMP)	LINGANORE	C	85	4.00 Ac.
TOWNHS. (45% IMP)	WORSHAM	D	88	6.70 Ac.
COMM CNTR (65%I)	WORSHAM	D	92	0.50 Ac.
OFFSITE				
PRESENT COND. *	GLENELG, et.al.	B	65	22.40 Ac.
PRESENT COND. *	LINGANORE	C	74	0.40 Ac.
*OFFSITE IS ASSMD.				
PRESENT COND.				
FOR STORMMGT.				
		AREA SUBTOTALS:		72.40 Ac.

Time of Concentration 2-Yr 24 Hr Rainfall = 3.1 In	Surface Cover Cross Section	Manning 'n Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	short grass	'n'=0.15	100 Ft.	2.00% 0.17 Hrs
Shallow Flow (a)	UNPAVED		100 Ft. 2.28 F.P.S.	2.00% 0.01 Hrs.
(b)	PAVED		200 Ft. 2.87 F.P.S.	2.00% 0.02 Hrs.
Channel Flow Hydraulic Radius =0.50	3.1 SqFt	'n'=0.013 6.3 Ft.	1900 Ft. #########	2.80% 0.04 Hrs.
Total Area in Acres =	72.40 Ac.	Total Sheet Flow=	Total Shallow Flow=	Total Channel Flow =
Weighted CN =	76			
Time Of Concentration =	0.24 Hrs.	0.17 Hrs.	0.03 Hrs.	0.04 Hrs.
Pond Factor =	1	RAINFALL TYPE II		

STORM	***Precipitation*** **(P) inches**	***Runoff*** **(Q)**	**Qp, PEAK DISCHARGE**	**TOTAL STORM Volumes**
1 Year	**2.5 In.**	**0.7 In.**	**52.0 CFS**	182,534 Cu. Ft.
2 Year	**3.1 In.**	**1.1 In.**	**84.1 CFS**	284,616 Cu. Ft.
5 Year	**4.0 In.**	**1.7 In.**	**140 CFS**	456,909 Cu. Ft.
10 Year	**5.0 In.**	**2.5 In.**	**209 CFS**	666,364 Cu. Ft.
25 Year	**5.4 In.**	**2.9 In.**	**238 CFS**	753,916 Cu. Ft.
50 Year	**6.1 In.**	**3.5 In.**	**290 CFS**	911,053 Cu. Ft.
100 Year	**7.0 In.**	**4.3 In.**	**358 CFS**	#############

TR-55
COMPUTER
VERSION

POST-DEV.
CONDITIONS

PEAK DISCHARGE SUMMARY				
JOB:	DESIGN EXAMPLE			RAC
DRAINAGE AREA NAME	TOTAL DA, ULTIMATE CONDITIONS			30-Mar-95
COVER DESCRIPTION	SOIL NAME	GROUP A,B,C,D?	CN from TABLE 2-2	AREA (In acres)
ONSITE				
TOWNHS. (45% IMP)	GLENELG, et.al	B	78	36.80 Ac.
COMM CNTR (65% I)	GLENELG, et.al.	B	85	1.60 Ac.
TOWNHS. (45% IMP)	LINGANORE	C	85	4.00 Ac.
TOWNHS. (45% IMP)	WORSHAM	D	88	6.70 Ac.
COMM CNTR (65%I)	WORSHAM	D	92	0.50 Ac.
OFFSITE *				
COMMERCIAL (85%I)	GLENELG, et.al.	B	92	6.60 Ac.
TOWNHS. (45% IMP)	GLENELG, et.al.	B	78	15.80 Ac.
TOWNHS. (45% IMP)	LINGANORE	C	85	0.40 Ac.
*OFFSITE AS ULT.				
FOR DAM SAFETY				
		AREA SUBTOTALS:		72.40 Ac.

Time of Concentration 2-Yr 24 Hr Rainfall = 3.1 In	Surface Cover Cross Section	Manning 'n Wetted Per	Flow Length Avg Velocity	Slope Tt (Hrs)
Sheet Flow	short grass	'n'=0.15	100 Ft.	2.00% 0.17 Hrs
Shallow Flow (a)	UNPAVED		100 Ft. 2.28 F.P.S.	2.00% 0.01 Hrs.
(b)	PAVED		200 Ft. 2.87 F.P.S.	2.00% 0.02 Hrs.
Channel Flow Hydraulic Radius =0.50	3.1 SqFt	'n'=0.013 6.3 Ft.	1900 Ft. #########	2.80% 0.04 Hrs.

Total Area in Acres =	72.40 Ac.	Total Shee Flow= 0.17 Hrs.	Total Shallow Flow= 0.03 Hrs.	Total Channel Flow = 0.04 Hrs.
Weighted CN =	81			
Time Of Concentration =	0.24 Hrs.			
Pond Factor =	1	RAINFALL TYPE II		

STORM	***Precipitation*** **(P) inches**	***Runoff*** **(Q)**	**Qp, PEAK DISCHARGE**	**TOTAL STORM Volumes**
1 Year	**2.5 In.**	**0.9 In.**	**74.1 CFS**	247,671 Cu. Ft.
2 Year	**3.1 In.**	**1.4 In.**	**112.4 CFS**	365,523 Cu. Ft.
5 Year	**4.0 In.**	**2.1 In.**	**176 CFS**	557,552 Cu. Ft.
10 Year	**5.0 In.**	**3.0 In.**	**251 CFS**	784,580 Cu. Ft.
25 Year	**5.4 In.**	**3.3 In.**	**281 CFS**	878,145 Cu. Ft.
50 Year	**6.1 In.**	**4.0 In.**	**334 CFS**	#############
100 Year	**7.0 In.**	**4.8 In.**	**404 CFS**	#############

TR-55
COMPUTER
VERSION

ULTIMATE
CONDITIONS

HYDROLOGY RUN

```
****************************************************************************
*                                    * TR 20 S/N    :                        *
*                                    * HMVersion    :  3.40                 *
*              TR 20                 * Date         : 12/21/94              *
*                                    * Time         : 19:48:47              *
*    Project Formulation Hydrology   * Input file   : DESIGN.EXM            *
*                                    * Output file  : OUT                   *
*                                    *                                      *
*                                    *                                      *
****************************************************************************
```

```
XXXXXXX  XXXXXX    XXXXX    XXXXX
   X     X     X  X     X  X     XX
   X     X     X        X  X    X X
   X     XXXXXX       X    X   X  X
   X     X   X      X      X X    X
   X     X    X   X        XX     X
   X     X     X  XXXXXXX   XXXXX
```

```
::::::::::::::::::::::::::::::::::::::::::
::::::::::::::::::::::::::::::::::::::::::
:::                                    :::
:::  Full Microcomputer Implementation :::
:::                by                  :::
:::        Haestad Methods, Inc.       :::
:::                                    :::
::::::::::::::::::::::::::::::::::::::::::
::::::::::::::::::::::::::::::::::::::::::
```

37 Brookside Road * Waterbury, Connecticut 06708 * (203) 755-1666

INPUT - FILE

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20                    FULLPRINT                    NOPLOTS
TITLE     DESIGN EXAMPLE--HYDROLOGY (DESIGN.EXM)    DEC. '94  RAC
TITLE     HYDROLOGIC RUN FOR PRE & POST DEV 2, 10, 100 YR. EVENTS
 6 RUNOFF 1     1     7 0.113      61.0       0.462      1 0   0 0 1 - PRE-DEV.
 6 RUNOFF 1     2     6 0.113      76.0       0.241      1 0   0 0 1 - POST-DEV.
 6 RUNOFF 1     3     5 0.113      81.0       0.241      1 0   0 0 1 - ULT. DEV
  ENDATA
 7 INCREM 6             0.1
 7 COMPUT 7     1     2 0.0        3.1        1.0        2 2   1   1 - 2-YEAR
  ENDCMP 1
 7 COMPUT 7     1     2 0.0        5.0        1.0        2 2   1   2 - 10-YEAR
  ENDCMP 1
 7 COMPUT 7     3     3 0.0        7.0        1.0        2 2   1   3 - 100-YEAR
  ENDCMP 1
  ENDJOB 2

*******************************END OF 80-80 LIST*******************************
```

COMPUTER PROGRAM FOR PROJECT FORMULATION - HYDROLOGY USER NOTES

THE USERS MANUAL FOR THIS PROGRAM IS THE MAY 1982 DRAFT OF TR-20. CHANGES FROM THE 2/14/74 VERSION INCLUDE:

REACH ROUTING - THE MODIFIED ATT-KIN ROUTING PROCEDURE REPLACES THE CONVEX METHOD. INPUT DATA PREPARED FOR PREVIOUS PROGRAM VERSIONS USING CONVEX ROUTING COEFFICIENTS WILL NOT RUN ON THIS VERSION.

THE PREFERRED TYPE OF DATA ENTRY IS CROSS SECTION DATA REPRESENTATIVE OF A REACH. IT IS RECOMMENDED THAT THE OPTIONAL CROSS SECTION DISCHARGE-AREA PLOTS BE OBTAINED WHENEVER NEW CROSS SECTION DATA IS ENTERED. THE PLOTS SHOULD BE CHECKED FOR REASONABLENESS AND ADEQUACY OF INPUT DATA FOR THE COMPUTATION OF "M" VALUES USED IN THE ROUTING PROCEDURE.

GUIDELINES FOR DETERMINING OR ANALYZING REACH LENGTHS AND COEFFICIENTS (X,M) ARE AVAILABLE IN THE USERS MANUAL. SUMMARY TABLE 2 DISPLAYS REACH ROUTING RESULTS AND ROUTING PARAMETERS FOR COMPARISON AND CHECKING.

HYDROGRAPH GENERATION - THE PROCEDURE TO CALCULATE THE INTERNAL TIME INCREMENT AND PEAK TIME OF THE UNIT HYDROGRAPH HAVE BEEN IMPROVED. PEAK DISCHARGES AND TIMES MAY DIFFER FROM THE PREVIOUS VERSION. OUTPUT HYDROGRAPHS ARE STILL INTERPOLATED, PRINTED, AND ROUTED AT THE USER SELECTED MAIN TIME INCREMENT.

INTERMEDIATE PEAKS - METHOD ADDED TO PROVIDE DISCHARGES AT INTERMEDIATE POINTS WITHIN REACHES WITHOUT ROUTING.

OTHER - THIS VERSION CONTAINS SOME ADDITIONS TO THE INPUT AND NUMEROUS MODIFICATIONS TO THE OUTPUT. USER OPTIONS HAVE BEEN MODIFIED AND AUGMENTED ON THE JOB RECORD, RAINTABLES ADDED, ERROR AND WARNING MESSAGES EXPANDED, AND THE SUMMARY TABLES COMPLETELY REVISED. THE HOLDOUT OPTION IS NOT OPERATIONAL AT THIS TIME.

PROGRAM QUESTIONS OR PROBLEMS SHOULD BE DIRECTED TO HYDRAULIC ENGINEERS AT THE SCS NATIONAL TECHNICAL CENTERS:

CHESTER, PA (NORTHEAST) -- 215-499-3933, FORT WORTH, TX (SOUTH) -- 334-5242 (FTS)
LINCOLN, NB (MIDWEST) -- 541-5318 (FTS), PORTLAND, OR (WEST) -- 423-4099 (FTS)
OR HYDROLOGY UNIT, ENGINEERING DIVISION, LANHAM, MD -- 436-7383 (FTS).

PROGRAM CHANGES SINCE MAY 1982:

12/17/82 - CORRECT PEAK RATE FACTOR FOR USER ENTERED DIMHYD
CORRECT REACH ROUTING PEAK TRAVEL TIME PRINTED WITH FULLPRINT OPTION
5/02/83 - CORRECT COMPUTATIONS FOR ---
1. DIVISION OF BASEFLOW IN DIVERT OPERATION
2. HYDROGRAPH VOLUME SPLIT BETWEEN BASEFLOW AND ABOVE BASEFLOW
3. CROSS SECTION DATA PLOTTING POSITION
4. INTERMEDIATE PEAK WHEN "FROM" AREA IS LARGER THAN "THRU" AREA
5. STORAGE ROUTED REACH TRAVEL TIME FOR MULTIPEAK HYDROGRAPH
6. ORDERING "FLOW-FREQ" FILE FROM SUMMARY TABLE #3 DATA
7. BASEFLOW ENTERED WITH READHYD
8. LOW FLOW SPLIT DURING DIVERT PROCEDURE #2 WHEN SECTION RATINGS START AT DIFFERENT ELEVATIONS
ENHANCEMENTS ---
1. REPLACE USER MANUAL ERROR CODES (PAGE 4-9 TO 4-11) WITH MESSAGES
2. LABEL OUTPUT HYDROGRAPH FILES WITH CROSS SECTION/STRUCTURE, ALTERNATE AND STORM NO'S
09/01/83 - CORRECT INPUT AND OUTPUT ERRORS FOR INTERMEDIATE PEAKS
CORRECT COMBINATION OF RATING TABLES FOR DIVERT
CHECK REACH ROUTING PARAMETERS FOR ACCEPTABLE LIMITS

ELIMINATE MINIMUM REACH TRAVEL TIME WHEN ATT-KIN COEFFICIENT EQUALS ONE

2-YEAR

EXECUTIVE CONTROL OPERATION INCREM MAIN TIME INCREMENT = .10 HOURS RECORD ID

EXECUTIVE CONTROL OPERATION COMPUT FROM STRUCTURE 1 TO STRUCTURE 2 RECORD ID
STARTING TIME = .00 RAIN DEPTH = 3.10 RAIN DURATION= 1.00 RAIN TABLE NO.= 2 ANT. MOIST. COND= 2
ALTERNATE NO.= 1 STORM NO.= 1 MAIN TIME INCREMENT = .10 HOURS

OPERATION RUNOFF STRUCTURE 1 — PRE-DEV
OUTPUT HYDROGRAPH= 7
AREA= .11 SQ MI INPUT RUNOFF CURVE= 61. TIME OF CONCENTRATION= .46 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0616 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.25	16.02	(RUNOFF)
23.67	1.06	(RUNOFF)

RUNOFF VOLUME ABOVE BASEFLOW = .40 WATERSHED INCHES, 29.42 CFS-HRS, 2.43 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RUNOFF STRUCTURE 2 - POST-DEV.
OUTPUT HYDROGRAPH= 6
AREA= .11 SQ MI INPUT RUNOFF CURVE= 76. TIME OF CONCENTRATION= .24 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0321 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.07	89.32	(RUNOFF)
23.65	1.85	(RUNOFF)

RUNOFF VOLUME ABOVE BASEFLOW = 1.08 WATERSHED INCHES, 79.05 CFS-HRS, 6.53 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 1 RECORD ID

10 - YEAR

EXECUTIVE CONTROL OPERATION COMPUT FROM STRUCTURE 1 TO STRUCTURE 2 RECORD ID
STARTING TIME = .00 RAIN DEPTH = 5.00 RAIN DURATION= 1.00 RAIN TABLE NO.= 2 ANT. MOIST. COND= 2
ALTERNATE NO.= 1 STORM NO.= 2 MAIN TIME INCREMENT = .10 HOURS

OPERATION RUNOFF STRUCTURE 1 — PRE-DEV.
OUTPUT HYDROGRAPH= 7
AREA= .11 SQ MI INPUT RUNOFF CURVE= 61. TIME OF CONCENTRATION= .46 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0616 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.21	78.36	(RUNOFF)
23.66	2.61	(RUNOFF)

RUNOFF VOLUME ABOVE BASEFLOW = 1.37 WATERSHED INCHES, 99.72 CFS-HRS, 8.24 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RUNOFF STRUCTURE 2 — POST-DEV.
OUTPUT HYDROGRAPH= 6
AREA= .11 SQ MI INPUT RUNOFF CURVE= 76. TIME OF CONCENTRATION= .24 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0321 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.05	212.24	(RUNOFF)
15.16	8.01	(RUNOFF)
16.45	6.97	(RUNOFF)
17.66	5.85	(RUNOFF)
19.65	4.73	(RUNOFF)
23.65	3.60	(RUNOFF)

RUNOFF VOLUME ABOVE BASEFLOW = 2.54 WATERSHED INCHES, 185.11 CFS-HRS, 15.30 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 2 RECORD ID

100 - YEAR

EXECUTIVE CONTROL OPERATION COMPUT FROM STRUCTURE 3 TO STRUCTURE 3 RECORD ID
STARTING TIME = .00 RAIN DEPTH = 7.00 RAIN DURATION= 1.00 RAIN TABLE NO.= 2 ANT. MOIST. COND= 2
ALTERNATE NO.= 1 STORM NO.= 3 MAIN TIME INCREMENT = .10 HOURS

OPERATION RUNOFF STRUCTURE 3 - ULTIMATE DEV.
OUTPUT HYDROGRAPH= 5
AREA= .11 SQ MI INPUT RUNOFF CURVE= 81. TIME OF CONCENTRATION= .24 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0321 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.04	392.06	(RUNOFF)
16.41	11.21	(RUNOFF)
17.65	9.39	(RUNOFF)
19.65	7.55	(RUNOFF)
23.65	5.70	(RUNOFF)

RUNOFF VOLUME ABOVE BASEFLOW = 4.81 WATERSHED INCHES, 350.80 CFS-HRS, 28.99 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 3 RECORD ID

EXECUTIVE CONTROL OPERATION ENDJOB RECORD ID

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 1 STORM 1 — 2-YEAR													
STRUCTURE 1	RUNOFF	.11	2	2	.10	.0	3.10	24.00	.40	---	12.25	16.02	141.8 P
STRUCTURE 2	RUNOFF	.11	2	2	.10	.0	3.10	24.00	1.08	---	12.07	89.32	790.4 P
ALTERNATE 1 STORM 2 - 10-YEAR													
STRUCTURE 1	RUNOFF	.11	2	2	.10	.0	5.00	24.00	1.37	---	12.21	78.36	693.4 P
STRUCTURE 2	RUNOFF	.11	2	2	.10	.0	5.00	24.00	2.54	---	12.05	212.24	1878.3 P
ALTERNATE 1 STORM 3 — 100 YEAR													
STRUCTURE 3	RUNOFF	.11	2	2	.10	.0	7.00	24.00	4.81	---	12.04	392.06	3469.6

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 1	2	3
STRUCTURE 3	.11			
ALTERNATE 1		.00	.00	392.06
STRUCTURE 2	.11			
ALTERNATE 1		89.32	212.24	.00
STRUCTURE 1	.11			
ALTERNATE 1		16.02	78.36	.00

Permanent Pool and Extended Detention Storage Analysis

The next step is to calculate the required storage for water quality. For this example the water quality storage will consist of two components. The permanent pool volume will consist of 50% of WQV, the forebay will consist of a volume of at least 0.1" per impervious acre, and the extended detention (ED) volume will consist of the remaining 50% of WQV.

Permanent Pool

DA=72.4 ac	%Imp. = on-site:		(47.5 ac(.45) + 2.1 ac(.65))/49.6 = .46
	off-site:		(6.6(.85) + 16.2(.45))/22.8 = .57
	total:		(49.6(.46) + 22.8(.57))/72.4 = .49

Use 50% Impervious

for 50% Imp. WQV = 1.25" x R_v x DA

R_v = 0.05 + 0.009(I) = 0.05 + 0.009(50%) = 0.5 Where R_v = the volumetric runoff coefficient, and is computed as derived by Schueler, 1987

WQV = 1.25"(0.5)(72.4 ac)/12"/ft = 3.77 ac-ft

Permanent Pool Volume (V_{pp}):

V_{pp} = 50% of WQV = 0.5(3.77 ac-ft) = 1.89 ac-ft
Minimum length to width ratio = 2:1
Max depth = 8 ft, 4ft to 6ft avg.
70% of shoreline in shallow aquatic shelves

Sediment Forebay

Size forebay to provide between .1" and .25" volume per impervious acre

Minimum Vol. required: .1"(0.5)(72.4 ac)/(12"/ft) = <u>0.3 ac-ft</u>

Extended Detention

Vol. required: ½ * (WQV) = ½ * (3.77 ac-ft) = <u>1.89 ac-ft</u>

Note: The ultimate developed drainage area was used for water quality volume calculations since it is not known what type of water quality controls will ultimately be installed upstream.

This example employs the "kirplunk" method of ED sizing, where the entire ED volume of runoff is assumed to be in the pond all at once. This will not actually be the case since the outlet orifice will be sized to release this volume over a 24 hour duration, but this conservative approach allows for ED control for storm events other than the target rainfall. Designs may incorporate multiple orifice plates or caps, or multiple weirs to allow for various ED release rates.

Stormwater Management Storage Analysis

The next step in the design process is to estimate the stormwater management volume requirements. This is done using TR-55 short cut routing methods (Chapter 6, TR-55, 1986) which will be used as the basis for initially grading the pond and computing the storage-elevation relationship. This preliminary storage analysis provides the first draft information necessary for sizing the principal spillway.

Preliminary Storage for 2 Year Storm Event

From TR-20 hydrologic results: 2 year pre-developed runoff = 16.0 cfs
2 year post-developed discharge = 89.3 cfs

Allowable 2 year release rate = 16.0 cfs

Compute: peak outflow discharge ÷ peak inflow discharge (Q_o/Q_i)
$Q_o/Q_i = 16.0/89.3 = 0.18$
Read from figure 6.1, TR-55, Storage Volume/Runoff Volume (V_s/V_r)
$V_s/V_s = 0.47$: for CN =76, and 3.1" rainfall; $V_r = 1.08"$
Storage Volume: $V_s = 0.47(1.08")(72.4$ ac/12 "/ft) = 3.06 ac-ft.
(note: providing extended detention (for ½ WQV) increases TR-55 estimated required storage for 2 yr storm by approx 10%, use estimated required storage 3.06(1.1) = 3.37 ac-ft)

Preliminary storage for 10 Year Storm Event

10 year pre-developed runoff = 78.4 cfs
10 year post-developed runoff = 212.2 cfs

Allowable 10 year release rate = 78.4 cfs

$Q_o/Q_i = 78.4/212.2 = 0.37$
$V_s/V_r = 0.34$: for CN = 76, and 5.0" rainfall; $V_r = 2.54"$
Storage Volume: $V_s = 0.34(2.54)(72.4/12) = 5.21$ ac-ft
(note: providing extended detention (for $½V_t$) increases TR-55 estimated required storage for 10 year storm by approximately 10%, use estimated required storage 5.21(1.1) = 5.73 ac-ft)

Once the short-cut volumes have been computed, the next step is to initially grade the pond (establish contours), and determine the elevation-storage relationship for the pond. Storage must be provided for the permanent pool, sediment forebay, extended detention, 2 year and 10 year storms and sufficient additional storage to pass the 100 year storm with minimum freeboard. An elevation-storage table and curve is prepared using the average area method for computing volumes. Two tables have been prepared, one for the permanent pool and forebay and the other for stormwater management quantity volume available above the permanent pool (See Figure 2 and 3).

Permanent Pool

Required permanent pool volume = ½ * WQV = 1.89 ac-ft
From elevation-storage table, Figure 2, read elev. 413

Use permanent pool elev... 414.0 (storage = 2.68 ac-ft)

Sediment Forebay

Required minimum sediment forebay volume = 0.3 ac-ft
Want surface elevation at, or above ED elevation.

See Figure 4 for a profile through the center of the pond area, delineating permanent pool and sediment forebay.

Sediment Forebay surface elevation = 417.0, from elevation-storage table, Figure 3, read storage available = 0.62 ac-ft

Dam Breach Analysis

Once the pond contours and embankment height have been initially established, the dam breach potential must be checked. The maximum discharge (Q_{max}) during a breach is computed and compared with downstream conditions to establish the pond hazard classification. In some cases the embankment design can be modified, if necessary, to reduce the hazard classification. Several computer models are acceptable for calculating the breach hydrograph and conducting stream valley routings to establish the hazard classification (e.g., SCS-TR-66 "Simplified Dam-Breach Routing Procedure", U.S COE HEC-1, DAMBRK). In many cases a complete hazard analysis will not be required since the downstream conditions do not warrant a detailed assessment. For this example, since there are no structures, road or railroad crossings within 7,500 feet from the pond, the Q_{max} and the approximate breach depth are computed to document the potential breach area for use in planning potential future downstream developments. If the Q_{max} and breach depth are excessively large, a complete stream valley routing may be necessary to document the point where the breach flow is equal to that of the floodplain. The amount of analysis necessary varies from jurisdiction to jurisdiction, but the ultimate liability is with the designer, so if there is any doubt, a complete analysis should be conducted:

From MD 378, $Q_{max} = 3.2\ H_w^{5/2}$ where: H_w = height of water at the time of breach, measured from the emergency spillway to the low point on the original grade at the center of the pond embankment.

$H_w = 419.8 - 406.5 = 13.3$ ft
$Q_{max} = 3.2 * (13.3)^{5/2} = 2064$ ft^3/sec

For stream valley with:

Approximately 15' bottom width
2:1 valley side slopes
Moderate underbrush, partially wooded, ($n = 0.06$)
Approx. valley slope = 2.0%
$Q_{max} = 2064$ ft^3/sec

Depth will be approximately 8 feet, based on Manning's equation analysis.

Based on the downstream conditions, and the moderate depth of the breach classify the facility as:

SCS-Class "a", Low Hazard (minimal property damage, no loss of life)

Hydraulic Flow Characteristics of Control Structure

The next step is to calculate the hydraulic characteristics of the control release structure. This involves (1) sizing a low flow orifice or weir for ED control, (2) a higher stage orifice or weir for 2 and 10 year quantity control, (2) sizing the riser and barrel (collectively known as the principal spillway), and (4) sizing an emergency spillway.

(1) Extended Detention

Fifty % of WQV is allocated for extended detention or 1.89 ac-ft
This volume is released over a 24 hour duration.
Using the elevation-storage table and curve, estimate the elevation required to store the full ED volume (1.89 ac-ft): Read elev.. 416.6, **use 417.0**

Compute the average release rate (equal to the volume/duration):

Q_{avg} = (1.89 ac-ft(43,560 ft^2/ac))/ (24hr(3600 sec/hr)) = 0.95 cfs

At the full ED elevation, the maximum release rate is assumed to be $Q_{max} = 2 * (Q_{avg})$

$Q_{max} = 2 \cdot (0.95) = 1.9$ cfs @ elev.. 417.0

Compute required low flow orifice size: (use orifice equation, reference Brater and King, "Handbook of Hydraulics")

$Q = C * A \cdot (2gh)^{1/2}$

Try 6" diameter orifice C = 0.6, A = 0.196 ft^2

h = 417.0 - (414.0 + d/2) = 417.0 - (414.0 + 0.5'/2) = 2.75'

Q = 0.6 * (.196 ft^2) · [(64.4 ft/sec^2)(2.75 ft)]$^{1/2}$ = 1.56 ft^3/sec, 1.56 < 1.90 OK

Result: **Use 6" diameter orifice, invert elevation, 414.0**

(2.0) 2 Year Stormwater Management

Set invert elevation at ED water surface elevation (417.0)
Allowable release rate = 16.0 cfs (from pre-developed hydrology)

From preliminary storage calculations, for storage of 3.37 ac-ft, read elevation = 417.8 (Figure 3)
At elevation 417.8; h_{ED} = 3.55', $Q_{ED\ orifice}$ = 1.8 cfs, therefore,
2 year slot maximum release rate = 16.0 cfs - 1.8 cfs = 14.2 cfs.

Compute required 2 year slot orifice or weir size: (use orifice equation or weir equation, Brater and King "Handbook of Hydraulics")

$Q_o = C * A * (2gh)^{1/2}$ or $Q_w = C * L * H^{3/2}$

Try 4' x 1' horizontal slot, with invert set at 417.0

Orifice: C = 0.6 A = 4 ft^2

$h = 417.8 - (417.0 + 0.5) = 0.3'$

$Q_o = 0.6 * (4.0\ ft^2) * [(64.4\ ft/sec^2)(0.3\ ft)]^{1/2} = 10.5\ cfs + 1.8\ cfs = 12.3\ cfs$ (which is < 16.0 cfs)

Weir: $C = 3.1$ $L = 4\ ft$

$H = 417.8 - 417.0 = 0.8\ ft$

$Q_w = 3.1(4\ ft)(0.8)^{3/2} = 8.9\ cfs + 1.8\ cfs = 10.7\ cfs$ (which is < 16.0 cfs)

Note: both the orifice equation and the weir equation are presented here, the weir equation is used for elevations below the top of the slot (elev. 418) and the orifice equation is used for submerged conditions (above elev. 418).

Result: **<u>Use 4 ft x 1 ft slot, invert elevation, 417.0</u>**

(2.1) <u>10 Year Stormwater Management</u>

Set invert elevation above 2 year elevation at 418.0
Allowable 10 year release rate = 78.3 cfs (pre-developed hydrology)

From preliminary storage calculations, for storage = 5.73 ac-ft, read elevation 419.5 (Figure 3)
At elev.. 419.5 $Q_{ED\ orifice} = 2.2$ cfs and $Q_{2yr\ slot} = 27.2$ cfs, therefore 10 year slot maximum release rate = 78.3 cfs - (2.2 cfs + 27.2 cfs) = 48.9 cfs

Compute required 10 year slot size

Try 10' x 2' slot (actually, 2 - 5' x 2' slots)

Weir: $C = 3.1$ $L = 10\ ft$

$H = 419.5 - 418.0 = 1.5\ ft$

$Q_w = 3.1(10\ ft)(1.5\ ft)^{3/2} = 57.0\ cfs > 48.9\ cfs$ (may be too large, but want the barrel to control flow if possible; this is explained below)

Orifice: $C = 0.6$ $A = 20\ ft^2$

$h = 419.5 - (418.0 + 2.0'/2) = 0.5\ ft$

$Q_o = 0.6 * (20\ ft) * [(64.4\ ft/sec^2)(0.5\ ft)]^{1/2} = 68.1\ cfs > 57.0\ cfs$ from weir equation, which is good because we want the barrel to control flow before the high stage riser slot goes from weir flow to orifice flow. See Storage-Elevation-Discharge Data table to verify that barrel controls flow prior to riser going from weir to orifice flow.

Result: **<u>Use two 5 ft x 2 ft slots, invert elevation, 418.0</u>**

(3.0) **Riser**

Size riser to accommodate all flow through control openings (See Figure 5)

- 6" ED Orifice
- 4' x 1' 2-Year Slot

- Two 5' x 2' 10-Year Slots
- 27" RCP Barrel (see following computations)

Try 8' x 5' Reinforced Concrete Box

Check orifice control at elevation 419.5 (this condition is rarely a limiting factor)

$C = 0.6$ $A = 40\ ft^2$

$h = 419.5 - 418.0 = 1.5'$

$Q_{base\ orifice} = 0.6 * (40\ ft^2) * [(64.4\ ft/sec^2)(1.5)]^{1/2} = 236$ cfs which is >> slot orifice

Result: **Use 8' x 5' Reinforced Concrete Box Riser**

(3.1) Barrel

Upstream invert = 405.0
Downstream invert = 403.65

The barrel inverts are established based on existing topography at the outfall and the relative elevation of the pond bottom.

At elevation 419.5 the barrel should control flow and release less than 78.3 cfs.

Try 27" RCP Barrel

Inlet Control Condition

Use Federal Highway Administration Culvert Charts or alternative computer program (such as HY-8) to check inlet capacity.

For 27" RCP, $H_w/d = (419.5 - 405.0)/2.25\ ft = 6.44$, for Chart No. 2, (Headwater Depth for Concrete Pipe Culverts with Inlet Control), entrance condition (1), read Q = 70 cfs±

70 cfs < 78.3 cfs

Outlet Control Condition

Use SCS pipe flow equation (based on Bernoulli equation) from NEH Section 5, ES-42

$Q = A * [2gh/(1 + k_m + k_pL)]^{1/2}$ $A = 3.98\ ft^2$, k_m (coeff. of minor losses) = 1.0, k_p (head loss coeff. of circular pipe flowing full) = .01016, L (pipe length) = 81 ft

$h = 419.5 - (403.65 + 2.25/2) = 14.73$

$Q = 3.98\ ft^2 * [(64.4\ ft/sec^2)(14.73)/2 + .01016(81)]^{1/2} = 73.0$ cfs > 70 cfs so barrel is in inlet control

Result: **Use 27" RCP Barrel**

(4) Emergency Spillway

Set invert elevation above 10 year water surface elevation (419.7)
Set crest elevation = 419.8

Size spillway to pass ultimate 100 year discharge (full off-site development) with at least 1 foot of freeboard to top of embankment (Q_{100} inflow = 392.0 cfs).

Try 40' wide, vegetated, emergency spillway with 3:1 side slopes

Top of dam = 422.5, so at elev. 421.5, Q_{ES} must be large enough to attenuate 100 year storm.

Using SCS, Design Data for Earth Spillways, Ref. Engineering Field Manual (RTSC-NE-ENG. 1110),

H = 421.5 - 419.8 = 1.7 ft

Q_{ES} = 222.0 cfs

$Q_{Principal\ Spillway}$ = 77 cfs

Q_{ES} + Q_{PS} = 299 cfs, which is less than 392cfs, however pond storage attenuation will ensure passage of ultimate 100 year flow (see TR-20, routing for specific elevations and discharges)

Hydraulic Computation Equations: Summary

(1) **Extended Detention - 6" orifice**

$Q = C * A * (2gh)^{1/2}$ where: $C = 0.6$, $A = 0.196\ ft^2$, and $h = w.s.e. - 414.25$

$\underline{Q = 0.943 * h^{1/2}}$

(2.0) **2 Year Slot - 4' x 1' slot**

Orifice:

$Q = C * A * (2gh)^{1/2}$ where: $C = 0.6$, $A = 4.0\ ft^2$, and $h = w.s.e. - 417.5$

$\underline{Q = 19.25 * h^{1/2}}$

Weir:

$Q_w = C * L * H^{3/2}$ where $C = 3.1$, $L = 4.0$ ft, and $H = w.s.e. - 417.0$

$\underline{Q_w = 12.4 * H^{3/2}}$

(2.1) **10 Year Slot - two 5' x 2' slots**

Weir:

$Q_w = C * L * H^{3/2}$ where $C = 3.1$, $L = 10.0$ ft, and $H = w.s.e. - 418.0$

$\underline{Q_w = 31.0 * H^{3/2}}$

Orifice:

$Q = C * A * (2gh)^{1/2}$ where: $C = 0.6$, $A = 20.0\ ft^2$, and $h = w.s.e. - 419.0$

$\underline{Q = 96.25 * h^{1/2}}$

(3.0) **Riser - 8' x 5' box**

Orifice:

$Q_{base\ orifice} = C * A * (2gh)^{1/2}$ where: $C = 0.6$, $A = 40.0\ ft^2$, and $h = w.s.e. - 418.0$

$\underline{Q_{base\ orifice} = 192.61 * h^{1/2}}$

(note: slot orifice more restrictive than base, use slot orifice in storage-elevation-discharge data table)

(3.1) **Barrel - 27" RCP**

Inlet control:

Use FHA Culvert Chart No. 2 where, $H_w/D = (w.s.e. - 405.0)/2.25$

Outlet control:

$Q = A* [(2gh/1 + k_m + k_pL)]^{1/2}$ where $A = 3.98\ ft^2$, k_m (coeff. of minor losses) = 1.0, k_p (head loss coeff. of circular pipe flowing full) = .01016, L (pipe length) = 81 ft, and h = w.s.e. - 404.78

$\underline{Q = 19.00 * h^{1/2}}$

(4) **Emergency Spillway**

Use Engineering Field Manual, Design Data for Earth Spillways

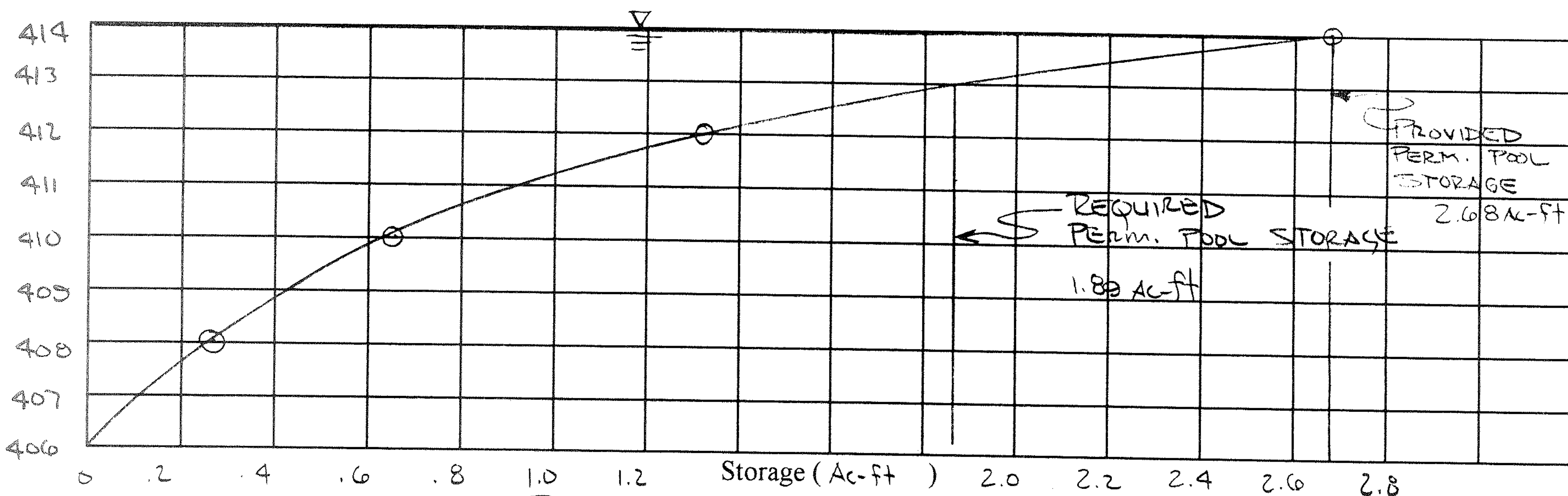
where $H_p = w.s.e. - 419.8$

Figure 2

Design Example
Permanent Pool Volume
Sediment Forebay Volume

Elevation - Storage Data

	Elevation (MSL)	Area (in²)	Area (ft²)	Average Area (ft²)	Depth (ft)	Volume (ft³)	Σ Volume (ft³)	Σ Volume (ac-ft)	Σ Volume above permanent pool (ac-ft)
Perm Pool	406	1.51	3775						
	408	3.20	8000	5887.50	2	11,775	11,775	0.27	
	410	5.34	13,350	10,675.00	2	21,350	33,125	0.76	
	412	7.88	19,700	16,525.00	2	33,050	66,175	1.52	
	414	12.31	30,775	25,237.50	2	50,475	116,650	2.68	
Forebay	412	1.06	2500						
	414	1.84	4600	3550	2	7,100	7,100	0.16	
	417	3.43	8575	6587.50	3	19,762.50	26,862.5	0.62	



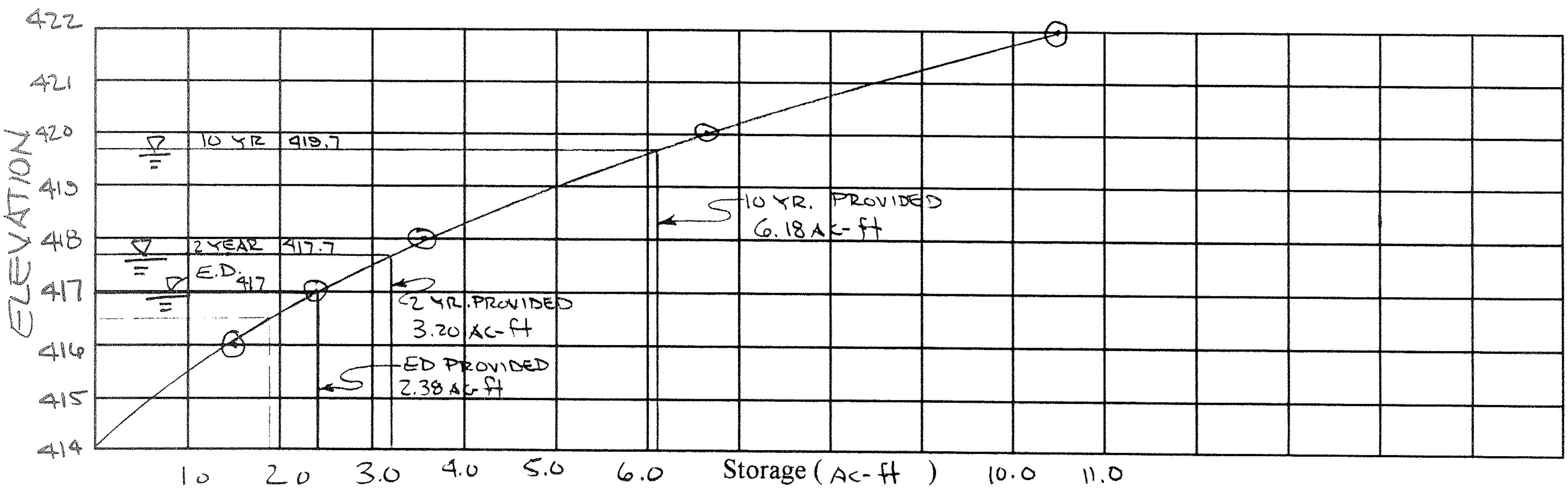
Permanent-Pool Elevation-Storage Curve

FIGURE 3

ELEVATION-STORAGE DATA AND CURVE ABOVE PERMANENT POOL

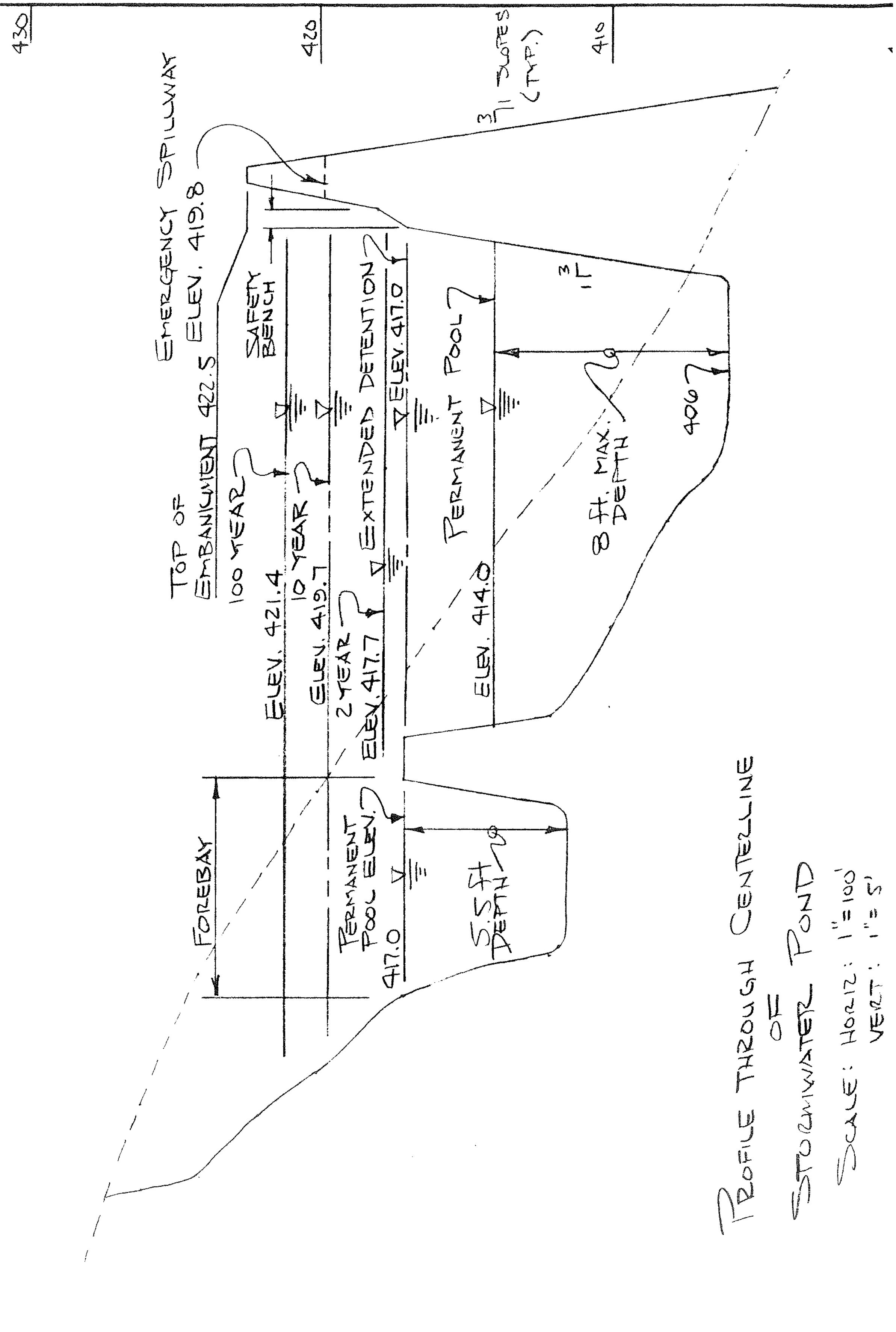
Elevation - Storage Data

Elevation (MSL)	Area (in²)	Area (ft²)	Average Area (ft²)	Depth (ft)	Volume (ft³)	Σ Volume (ft³)	Σ Volume (ac-ft)	Σ Volume above permanent pool (ac-ft)
414	12.31	30,775						
416	14.43	36,075	33,425	2	66,850.0	66,850		1.53
417.	15.28	38,200	37,137.50	1	37,137.50	103,987.5		2.38
418	25.21	63,025	50,612.5	1	50,412.50	154,600		3.55
420	28.72	71,800	67,412.5	2	134,825.0	289,425		6.64
422	38.06	95,150	83,475	2	166,950	456,375		10.48



Center for Watershed Protection
8737 Colesville Road, Suite 300
Silver Spring, MD 20910

FIGURE 4



PROFILE THROUGH CENTERLINE OF STORMWATER POND

SCALE: HORIZ: 1" = 100'
VERT: 1" = 5'

FIGURE 5

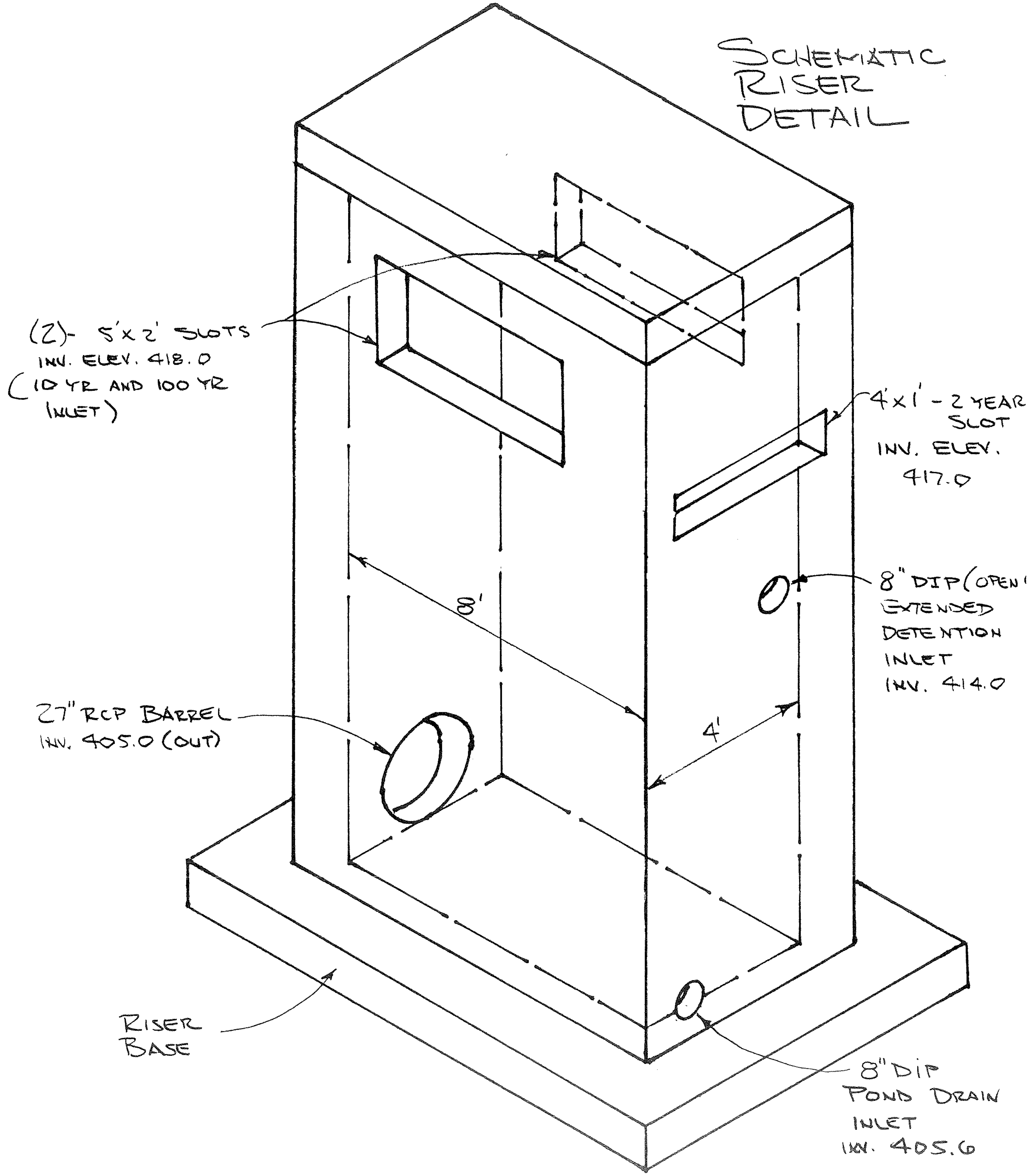
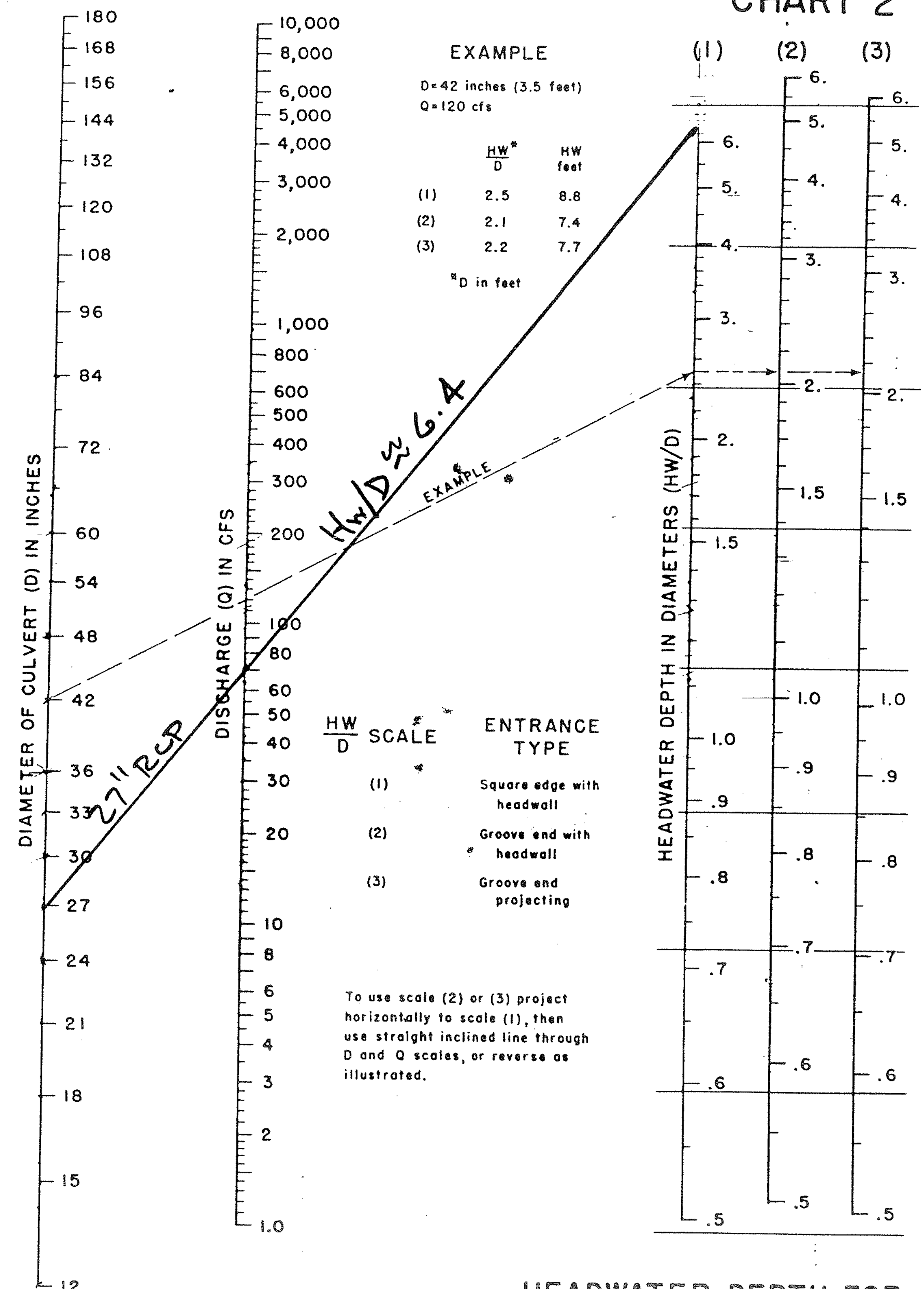

SCHEMATIC
RISER
DETAIL
(2)- 5'x 2' SLOTS
INV. ELEV. 418.0
(10 YR AND 100 YR
INLET)
4'x1' - 2 YEAR
SLOT
INV. ELEV.
417.0
8" DIP (OPEN
EXTENDED
DETENTION
INLET
INV. 414.0
8'
4'
27" RCP BARREL
INV. 405.0 (OUT)
RISER
BASE
8" DIP
POND DRAIN
INLET
INV. 405.6

FIGURE 6

CHART 2



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

DESIGN DATA FOR EARTH SPILLWAYS

SIDE SLOPE 3:1
VEGETATED n=0.040

WIDTH → 40'

STAGE (Hp) IN FEET	SPILLWAY VARIABLES	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	7	8	9	10	12	[illegible]	14	16	17	18	20	21	22	24	[illegible]	[illegible]	[illegible]
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	11	12	14	16	[illegible]	20	22	24	25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	V	2.9	2.9	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	[illegible]	[illegible]
0.7	Q	11	14	16	19	21	24	26	29	31	33	36	38	41	43	[illegible]	[illegible]	49
	V	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	40	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	14	17	20	23	26	30	32	35	38	42	45	47	50	52	55	[illegible]	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	19	22	25	29	32	36	40	43	47	51	55	59	63	66	69	73	76
	V	3.7	3.7	3.8	3.8	3.8	3.8	3.6	3.8	3.8	3.8	3.8	3.8	3.6	3.8	3.8	3.8	3.8
	S	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	48	48	46	48	48	48	48	46	48	48	49	49	49	49	49	49	49
1.0	Q	20	27	30	35	38	43	48	52	56	61	64	69	74	79	82	86	90
	V	3.9	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	X	52	52	52	52	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	25	31	34	40	45	48	54	60	65	70	74	79	84	90	95	100	105
	V	4.1	4.1	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	56	56	56	56	56	56	56	56	56	56	56	56	56	56	56
1.2	Q	30	37	42	47	52	59	65	71	76	82	88	92	99	105	110	116	122
	V	4.3	4.3	4.4	4.4	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	59	59	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60
1.3	Q	35	42	48	55	62	68	75	82	89	95	101	109	116	122	127	134	140
	V	4.5	4.5	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	63	63	64	64	64	64	64	64	64	64	64	64	64	64	64	64	64
1.4	Q	40	48	56	64	70	78	86	93	100	108	114	121	130	138	146	152	159
	V	4.7	4.7	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9
	S	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	67	67	67	68	68	68	68	68	68	68	68	68	68	68	68	66	68
1.5	Q	46	54	63	71	82	88	96	106	113	121	128	136	144	154	164	173	180
	V	4.8	4.9	4.9	4.9	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	72	72	72	72	72	72	72	72	72	72	72	72	72	72	72	72	72
1.6	Q	52	60	70	81	90	100	110	118	129	137	145	155	162	172	182	192	201
	V	4.9	5.0	[illegible]	5.1	5.1	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	76	76	76	76	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	58	66	75	90	100	115	[illegible]	132	141	149	160	168	179	190	201	212	222
	V	5.0	5.2	5.2	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4
	S	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	80	80	80	80	80	[illegible]	80	80	80	80	80	80	80	80	81	81	81
1.8	Q	65	[illegible]	[illegible]	100	110	122	133	145	155	166	175	188	196	208	220	232	243
	V	5.2	5.3	5.4	5.4	5.4	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5
	S	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84	84	85
1.9	Q	72	84	[illegible]	110	124	134	148	160	172	182	192	204	218	232	240	253	266
	V	5.4	5.4	5.5	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	86	87	88	88	88	88	88	88	88	88	89	89	89	89	89	89	89
2.0	Q	[illegible]	94	[illegible]	120	133	148	160	172	186	197	212	223	236	252	265	276	[illegible]
	V	5.5	5.6	5.6	5.7	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.8
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	90	92	92	92	92	92	92	92	92	92	92	92	92	93	93	93	93
2.1	Q	86	102	116	132	146	160	176	190	202	217	229	241	255	271	286	300	315
	V	5.6	5.7	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	94	95	95	95	95	95	95	95	96	96	96	96	97	97	97	97	97
2.2	Q	95	112	125	143	158	176	192	204	220	234	246	263	280	297	313	326	340
	V	5.7	5.8	5.9	5.9	6.0	6.0	6.0	6.0	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	99	99	99	100	100	101	101	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	102	102	102	102	102
2.3	Q	104	120	138	155	172	190	205	222	236	251	265	283	295	319	336	354	367
	V	5.8	5.9	6.0	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.2	6.2	6.2	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	103	104	104	105	105	105	106	106	106	105	106	105	106	106	106	106	106
2.4	Q	114	132	[illegible]	166	185	205	222	237	252	269	288	303	321	340	359	378	392
	V	6.0	6.0	[illegible]	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.4	6.4	6.4	6.4
	S	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	108	108	109	109	109	109	109	[illegible]	[illegible]	110	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Hp →

Exhibit 11-3.1

REFERENCE

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
ENGINEERING & WATERSHED PLANNING UNIT
BROOMALL PENNSYLVANIA

TSC-NE-ENG.
1110
SHEET 3 OF 11

TABLE 1

Storage - Elevation - Discharge Data

Elevation (MSL)	Storage (ac-ft)	Low (1) Flow (EXT. DET.)		Riser (3.0): (2.0) 2 YEAR SLOT		Riser (3.0): High Stage Slot (2.1) Orifice		Riser (3.0): High Stage Slot (2.1) Weir		(3.1) Barrel 27" RCP: Inlet FHWA CULVERT CH.		(3.1) Barrel 27" RCP: Pipe SCS PIPE EQ.		Emergency Spillway (4) 40' WIDE		Total Discharge
		H (ft)	Q (cfs)	H (ft)	Q (cfs)	H (ft)	Q (cfs)	H (ft)	Q (cfs)	Hw/D (ft)	Q (cfs)	H (ft)	Q (cfs)	H (ft)	Q (cfs)	(cfs)
414.0	0	–	–													0
414.5	0.38	0.25	0.5													0.5
415.0	0.77	0.75	0.8													0.8
416.0	1.53	1.75	1.2													1.2
417.0	2.38	2.75	1.6	0.0	0.0	(INVERT 2 YEAR SLOT)										1.6
417.5	2.97	3.25	1.7	0.5*	4.4											6.1
418.0	3.55	3.75	1.8	1.0*	12.4	0	0	0	0	(INVERT 10 YEAR SLOT)						14.2
419.0	5.10	4.75	2.1	1.5**	23.6	–		1.0	31	6.2	69	13.88	71.0			56.7
419.5	5.87	5.25	2.2 t	2.0**	27.2 t	–		1.5	57.0 t	6.4	70 t	14.38	72.3			70.0
419.8	6.33	–	–	2.3	29.2	–		1.8	74.9	6.6	71	14.68	73.0	0	0	71.0
420.0	6.64	–	–	2.5*+	30.4	1.0	36.3	2.0	87.7	6.7	72.5	14.88	73.5	0.2	5±	77.5
420.5	7.60	–	–	–	–	–	–	–	–	6.9	74	15.38	74.7	0.7	49	123
421.0	8.56	–	–	–	–	–	–	–	–	7.1	75	15.88	75.9	1.2	122	197
421.5	9.52	–	–	–	–	–	–	–	–	7.3	76.5	16.38	77.1	1.7	222	298.5

Project Name: DESIGN EXAMPLE Date: MARCH '95 By: RAC

* WEIR FLOW
** ORIFICE

(2.1) NOTE: NUMBERS IN PARENTHESES CORRESPOND TO HYDRAULIC EQUATIONS ON PAGE 31

```
*****************************************************************************
*                                      * TR 20 S/N     :                      *
*                                      * HMVersion     :  3.40              *
*               TR 20                  * Date          :  9/21/95           *
*                                      * Time          : 13:38:12           *
*    Project Formulation Hydrology     * Input file    : DESIGNEX.ROT       *
*                                      * Output file   : OUT                *
*                                      *                                    *
*                                      *                                    *
*****************************************************************************
```

STORMWATER
MANAGEMENT
ROUTING
RUN

```
XXXXXXX  XXXXXX    XXXXX    XXXXX
   X     X     X  X     X  X    XX
   X     X     X        X  X   X X
   X     XXXXXX       X    X  X  X
   X     X   X      X      X X   X
   X     X    X   X        XX    X
   X     X     X  XXXXXXX   XXXXX
```

```
::::::::::::::::::::::::::::::::::::::::::
::::::::::::::::::::::::::::::::::::::::::
:::                                    :::
:::  Full Microcomputer Implementation :::
:::               by                   :::
:::       Haestad Methods, Inc.        :::
:::                                    :::
::::::::::::::::::::::::::::::::::::::::::
::::::::::::::::::::::::::::::::::::::::::
```

37 Brookside Road * Waterbury, Connecticut 06708 * (203) 755-1666

```
******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************  INPUT FILE

JOB TR-20                    FULLPRINT                 NOPLOTS
TITLE     DESIGN EXAMPLE--ROUTING (DESIGNEX.ROT)   MAR. '95  RAC
TITLE     ROUTING RUN FOR POST DEV. FOR 2, 10,& ULT. DEV. FOR 100 YR
 3 STRUCT       2
 8                     414.0     0.0       0.0
 8                     414.5     0.5       0.38
 8                     415.0     0.8       0.77
 8                     416.0     1.2       1.53
 8                     417.0     1.6       2.38
 8                     417.5     6.1       2.97   <--- HYDRAULIC
 8                     418.0     14.2      3.55        CHARACTERISTICS
 8                     419.0     56.7      5.10        DATA
 8                     419.5     70.0      5.87
 8                     419.8     71.0      6.33
 8                     420.0     77.5      6.64
 8                     420.5     123.0     7.60
 8                     421.0     197.0     8.56
 8                     421.5     298.5     9.52
 9 ENDTBL
 6 RUNOFF 1     1      6 0.113   76.0      0.241     1 1   1 0 1    POST DEV. INFLOW
 6 RESVOR 2     2 6    7 414.0                       1 1 1 1 0 1    POST DEV. ROUTING
 6 RUNOFF 1     3      1 0.113   81.0      0.241     1 1   1 0 1    ULT. DEV. INFLOW
 6 RESVOR 2     2 1    2 414.0                       1 1 1 1 0 1    ULT. DEV. ROUTING
   ENDATA
 7 INCREM 6            0.1
 7 COMPUT 7     1      2 0.0     3.1       1.0       2 2   1   1     2-YEAR
   ENDCMP 1
 7 COMPUT 7     1      2 0.0     5.0       1.0       2 2   1   2     10-YEAR
   ENDCMP 1
 7 COMPUT 7     3      2 0.0     7.0       1.0       2 2   1   3     100-YEAR
   ENDCMP 1
   ENDJOB 2

*******************************END OF 80-80 LIST*********************************
```

COMPUTER PROGRAM FOR PROJECT FORMULATION - HYDROLOGY USER NOTES

THE USERS MANUAL FOR THIS PROGRAM IS THE MAY 1982 DRAFT OF TR-20. CHANGES FROM THE 2/14/74 VERSION INCLUDE:

REACH ROUTING - THE MODIFIED ATT-KIN ROUTING PROCEDURE REPLACES THE CONVEX METHOD. INPUT DATA PREPARED FOR PREVIOUS PROGRAM VERSIONS USING CONVEX ROUTING COEFFICIENTS WILL NOT RUN ON THIS VERSION.

THE PREFERRED TYPE OF DATA ENTRY IS CROSS SECTION DATA REPRESENTATIVE OF A REACH. IT IS RECOMMENDED THAT THE OPTIONAL CROSS SECTION DISCHARGE-AREA PLOTS BE OBTAINED WHENEVER NEW CROSS SECTION DATA IS ENTERED. THE PLOTS SHOULD BE CHECKED FOR REASONABLENESS AND ADEQUACY OF INPUT DATA FOR THE COMPUTATION OF "M" VALUES USED IN THE ROUTING PROCEDURE.

GUIDELINES FOR DETERMINING OR ANALYZING REACH LENGTHS AND COEFFICIENTS (X,M) ARE AVAILABLE IN THE USERS MANUAL. SUMMARY TABLE 2 DISPLAYS REACH ROUTING RESULTS AND ROUTING PARAMETERS FOR COMPARISON AND CHECKING.

HYDROGRAPH GENERATION - THE PROCEDURE TO CALCULATE THE INTERNAL TIME INCREMENT AND PEAK TIME OF THE UNIT HYDROGRAPH HAVE BEEN IMPROVED. PEAK DISCHARGES AND TIMES MAY DIFFER FROM THE PREVIOUS VERSION. OUTPUT HYDROGRAPHS ARE STILL INTERPOLATED, PRINTED, AND ROUTED AT THE USER SELECTED MAIN TIME INCREMENT.

INTERMEDIATE PEAKS - METHOD ADDED TO PROVIDE DISCHARGES AT INTERMEDIATE POINTS WITHIN REACHES WITHOUT ROUTING.

OTHER - THIS VERSION CONTAINS SOME ADDITIONS TO THE INPUT AND NUMEROUS MODIFICATIONS TO THE OUTPUT. USER OPTIONS HAVE BEEN MODIFIED AND AUGMENTED ON THE JOB RECORD, RAINTABLES ADDED, ERROR AND WARNING MESSAGES EXPANDED, AND THE SUMMARY TABLES COMPLETELY REVISED. THE HOLDOUT OPTION IS NOT OPERATIONAL AT THIS TIME.

PROGRAM QUESTIONS OR PROBLEMS SHOULD BE DIRECTED TO HYDRAULIC ENGINEERS AT THE SCS NATIONAL TECHNICAL CENTERS:

CHESTER, PA (NORTHEAST) -- 215-499-3933, FORT WORTH, TX (SOUTH) -- 334-5242 (FTS)
LINCOLN, NB (MIDWEST) -- 541-5318 (FTS), PORTLAND, OR (WEST) -- 423-4099 (FTS)
OR HYDROLOGY UNIT, ENGINEERING DIVISION, LANHAM, MD -- 436-7383 (FTS).

PROGRAM CHANGES SINCE MAY 1982:

12/17/82 - CORRECT PEAK RATE FACTOR FOR USER ENTERED DIMHYD
CORRECT REACH ROUTING PEAK TRAVEL TIME PRINTED WITH FULLPRINT OPTION

5/02/83 - CORRECT COMPUTATIONS FOR ---
1. DIVISION OF BASEFLOW IN DIVERT OPERATION
2. HYDROGRAPH VOLUME SPLIT BETWEEN BASEFLOW AND ABOVE BASEFLOW
3. CROSS SECTION DATA PLOTTING POSITION
4. INTERMEDIATE PEAK WHEN "FROM" AREA IS LARGER THAN "THRU" AREA
5. STORAGE ROUTED REACH TRAVEL TIME FOR MULTIPEAK HYDROGRAPH
6. ORDERING "FLOW-FREQ" FILE FROM SUMMARY TABLE #3 DATA
7. BASEFLOW ENTERED WITH READHYD
8. LOW FLOW SPLIT DURING DIVERT PROCEDURE #2 WHEN SECTION RATINGS START AT DIFFERENT ELEVATIONS

ENHANCEMENTS ---
1. REPLACE USER MANUAL ERROR CODES (PAGE 4-9 TO 4-11) WITH MESSAGES
2. LABEL OUTPUT HYDROGRAPH FILES WITH CROSS SECTION/STRUCTURE, ALTERNATE AND STORM NO'S

09/01/83 - CORRECT INPUT AND OUTPUT ERRORS FOR INTERMEDIATE PEAKS
CORRECT COMBINATION OF RATING TABLES FOR DIVERT
CHECK REACH ROUTING PARAMETERS FOR ACCEPTABLE LIMITS

ELIMINATE MINIMUM REACH TRAVEL TIME WHEN ATT-KIN COEFFICIENT EQUALS ONE

Z-YEAR

EXECUTIVE CONTROL OPERATION INCREM MAIN TIME INCREMENT = .10 HOURS RECORD ID

EXECUTIVE CONTROL OPERATION COMPUT FROM STRUCTURE 1 TO STRUCTURE 2 RECORD ID
STARTING TIME = .00 RAIN DEPTH = 3.10 RAIN DURATION= 1.00 RAIN TABLE NO.= 2 ANT. MOIST. COND= 2
ALTERNATE NO.= 1 STORM NO.= 1 MAIN TIME INCREMENT = .10 HOURS

OPERATION RUNOFF STRUCTURE 1 Z-YEAR INFLOW
OUTPUT HYDROGRAPH= 6
AREA= .11 SQ MI INPUT RUNOFF CURVE= 76. TIME OF CONCENTRATION= .24 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0321 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.07	89.32	(RUNOFF)
23.65	1.85	(RUNOFF)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .11 SQ.MI.

10.00	DISCHG	.00	.00	.00	.00	.00	.00	.01	.06	.18	.36
11.00	DISCHG	.57	.82	1.13	1.46	1.92	2.45	4.93	11.79	20.67	46.62
12.00	DISCHG	80.83	87.13	54.86	33.21	22.93	17.42	14.94	13.12	12.22	11.11
13.00	DISCHG	10.11	9.47	8.69	8.24	7.73	7.31	6.97	6.46	6.16	5.88
14.00	DISCHG	5.66	5.48	5.22	5.07	4.83	4.61	4.43	4.17	4.03	3.98
15.00	DISCHG	3.97	3.97	3.98	3.96	3.76	3.56	3.48	3.45	3.45	3.45
16.00	DISCHG	3.45	3.46	3.47	3.47	3.48	3.48	3.38	3.14	3.00	2.95
17.00	DISCHG	2.93	2.93	2.93	2.93	2.94	2.94	2.94	2.95	2.93	2.72
18.00	DISCHG	2.50	2.42	2.39	2.38	2.38	2.38	2.38	2.38	2.38	2.39
19.00	DISCHG	2.39	2.39	2.39	2.40	2.40	2.40	2.40	2.40	2.38	2.17
20.00	DISCHG	1.95	1.86	1.83	1.82	1.81	1.81	1.82	1.82	1.82	1.82
21.00	DISCHG	1.82	1.82	1.82	1.82	1.83	1.83	1.83	1.83	1.83	1.83
22.00	DISCHG	1.83	1.83	1.83	1.84	1.84	1.84	1.84	1.84	1.84	1.84
23.00	DISCHG	1.84	1.85	1.85	1.85	1.85	1.85	1.85	1.85	1.83	1.61
24.00	DISCHG	1.38	1.07	.49	.18	.06	.02	.01	.00		

RUNOFF VOLUME ABOVE BASEFLOW = 1.08 WATERSHED INCHES, 79.05 CFS-HRS, 6.53 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RESVOR STRUCTURE 2 - ROUTING
INPUT HYDROGRAPH= 6 OUTPUT HYDROGRAPH= 7
SURFACE ELEVATION= 414.00

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
13.12	9.28	417.70

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .11 SQ.MI.

11.00	DISCHG	.01	.02	.03	.04	.06	.08	.12	.21	.38	.64
11.00	ELEV	414.01	414.02	414.03	414.04	414.06	414.08	414.12	414.21	414.38	414.74
12.00	DISCHG	.97	1.31	1.58	3.93	5.41	6.47	7.53	8.24	8.73	9.05
12.00	ELEV	415.41	416.28	416.96	417.26	417.42	417.52	417.59	417.63	417.66	417.68

13.00	DISCHG	9.22	9.28	9.26	9.17	9.04	8.88	8.69	8.47	8.24	7.99
13.00	ELEV	417.69	417.70	417.69	417.69	417.68	417.67	417.66	417.65	417.63	417.62
14.00	DISCHG	7.75	7.51	7.28	7.05	6.82	6.59	6.36	6.14	6.00	5.87
14.00	ELEV	417.60	417.59	417.57	417.56	417.54	417.53	417.52	417.50	417.49	417.47
15.00	DISCHG	5.76	5.65	5.55	5.45	5.35	5.25	5.14	5.04	4.94	4.85
15.00	ELEV	417.46	417.45	417.44	417.43	417.42	417.41	417.39	417.38	417.37	417.36
16.00	DISCHG	4.77	4.69	4.61	4.54	4.48	4.42	4.36	4.29	4.21	4.14
16.00	ELEV	417.35	417.34	417.33	417.33	417.32	417.31	417.31	417.30	417.29	417.28
17.00	DISCHG	4.06	4.00	3.93	3.87	3.81	3.76	3.71	3.66	3.62	3.57
17.00	ELEV	417.27	417.27	417.26	417.25	417.25	417.24	417.23	417.23	417.22	417.22
18.00	DISCHG	3.51	3.45	3.38	3.32	3.26	3.21	3.16	3.11	3.07	3.03
18.00	ELEV	417.21	417.21	417.20	417.19	417.18	417.18	417.17	417.17	417.16	417.16
19.00	DISCHG	2.99	2.95	2.92	2.88	2.85	2.83	2.80	2.78	2.75	2.72
19.00	ELEV	417.15	417.15	417.15	417.14	417.14	417.14	417.13	417.13	417.13	417.12
20.00	DISCHG	2.68	2.64	2.59	2.54	2.50	2.45	2.42	2.38	2.34	2.31
20.00	ELEV	417.12	417.12	417.11	417.10	417.10	417.09	417.09	417.09	417.08	417.08
21.00	DISCHG	2.28	2.25	2.23	2.20	2.18	2.16	2.14	2.12	2.10	2.08
21.00	ELEV	417.08	417.07	417.07	417.07	417.06	417.06	417.06	417.06	417.06	417.05
22.00	DISCHG	2.07	2.05	2.04	2.03	2.02	2.01	2.00	1.99	1.98	1.97
22.00	ELEV	417.05	417.05	417.05	417.05	417.05	417.05	417.04	417.04	417.04	417.04
23.00	DISCHG	1.96	1.95	1.95	1.94	1.94	1.93	1.93	1.92	1.92	1.90
23.00	ELEV	417.04	417.04	417.04	417.04	417.04	417.04	417.04	417.04	417.04	417.03
24.00	DISCHG	1.88	1.84	1.77	1.69	1.60	1.59	1.59	1.58	1.57	1.57
24.00	ELEV	417.03	417.03	417.02	417.01	417.00	416.98	416.97	416.95	416.94	416.92
25.00	DISCHG	1.56	1.56	1.55	1.54	1.54	1.53	1.53	1.52	1.51	1.51
25.00	ELEV	416.91	416.89	416.88	416.86	416.85	416.83	416.82	416.80	416.79	416.77
26.00	DISCHG	1.50	1.50	1.49	1.49	1.48	1.47	1.47	1.46	1.46	1.45
26.00	ELEV	416.76	416.74	416.73	416.71	416.70	416.69	416.67	416.66	416.64	416.63
27.00	DISCHG	1.45	1.44	1.43	1.43	1.42	1.42	1.41	1.41	1.40	1.40
27.00	ELEV	416.61	416.60	416.59	416.57	416.56	416.54	416.53	416.52	416.50	416.49
28.00	DISCHG	1.39	1.39	1.38	1.37	1.37	1.36	1.36	1.35	1.35	1.34
28.00	ELEV	416.48	416.46	416.45	416.44	416.42	416.41	416.40	416.38	416.37	416.36
29.00	DISCHG	1.34	1.33	1.33	1.32	1.32	1.31	1.31	1.30	1.30	1.29
29.00	ELEV	416.34	416.33	416.32	416.31	416.29	416.28	416.27	416.25	416.24	416.23

RUNOFF VOLUME ABOVE BASEFLOW = .80 WATERSHED INCHES, 58.03 CFS-HRS, 4.80 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 1 RECORD ID

10-YEAR

EXECUTIVE CONTROL OPERATION COMPUT FROM STRUCTURE 1 TO STRUCTURE 2 RECORD ID
STARTING TIME = .00 RAIN DEPTH = 5.00 RAIN DURATION= 1.00 RAIN TABLE NO.= 2 ANT. MOIST. COND= 2
ALTERNATE NO.= 1 STORM NO.= 2 MAIN TIME INCREMENT = .10 HOURS

OPERATION RUNOFF STRUCTURE 1 - INFLOW
OUTPUT HYDROGRAPH= 6
AREA= .11 SQ MI INPUT RUNOFF CURVE= 76. TIME OF CONCENTRATION= .24 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0321 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.05	212.24	(RUNOFF)
15.16	8.01	(RUNOFF)
16.45	6.97	(RUNOFF)
17.66	5.85	(RUNOFF)
19.65	4.73	(RUNOFF)
23.65	3.60	(RUNOFF)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .11 SQ.MI.

8.00	DISCHG	.00	.00	.00	.00	.01	.06	.14	.22	.31	.39
9.00	DISCHG	.48	.58	.71	.83	.94	1.05	1.18	1.36	1.52	1.65
10.00	DISCHG	1.78	1.94	2.16	2.38	2.73	3.11	3.53	4.18	4.72	5.47
11.00	DISCHG	6.23	6.99	7.98	8.85	10.30	11.85	20.47	42.63	65.51	127.87
12.00	DISCHG	200.57	203.40	124.56	73.71	49.89	37.35	31.68	27.64	25.62	23.20
13.00	DISCHG	21.05	19.66	17.99	17.01	15.93	15.03	14.30	13.23	12.61	12.02
14.00	DISCHG	11.54	11.17	10.63	10.29	9.80	9.35	8.98	8.43	8.14	8.04
15.00	DISCHG	8.01	8.00	8.01	7.96	7.57	7.15	6.99	6.93	6.91	6.91
16.00	DISCHG	6.92	6.92	6.93	6.94	6.94	6.94	6.74	6.25	5.96	5.87
17.00	DISCHG	5.83	5.82	5.82	5.83	5.83	5.83	5.84	5.84	5.80	5.38
18.00	DISCHG	4.95	4.78	4.72	4.70	4.70	4.70	4.70	4.70	4.70	4.71
19.00	DISCHG	4.71	4.71	4.71	4.72	4.72	4.72	4.72	4.73	4.67	4.26
20.00	DISCHG	3.82	3.65	3.59	3.56	3.56	3.56	3.56	3.56	3.56	3.56
21.00	DISCHG	3.56	3.56	3.57	3.57	3.57	3.57	3.57	3.57	3.57	3.57
22.00	DISCHG	3.58	3.58	3.58	3.58	3.58	3.58	3.58	3.58	3.59	3.59
23.00	DISCHG	3.59	3.59	3.59	3.59	3.59	3.59	3.60	3.60	3.55	3.12
24.00	DISCHG	2.67	2.08	.96	.34	.12	.04	.01	.00		

RUNOFF VOLUME ABOVE BASEFLOW = 2.54 WATERSHED INCHES, 185.11 CFS-HRS, 15.30 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RESVOR STRUCTURE 2 - ROUTING
INPUT HYDROGRAPH= 6 OUTPUT HYDROGRAPH= 7
SURFACE ELEVATION= 414.00

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.34	70.53	419.66

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .11 SQ.MI.

8.00	DISCHG	.00	.00	.00	.00	.00	.00	.00	.00	.01	.01

8.00	ELEV	414.00	414.00	414.00	414.00	414.00	414.00	414.00	414.00	414.01	414.01
9.00	DISCHG	.01	.02	.03	.04	.04	.05	.07	.08	.09	.11
9.00	ELEV	414.01	414.02	414.03	414.04	414.04	414.05	414.07	414.08	414.09	414.11
10.00	DISCHG	.13	.15	.17	.19	.21	.24	.28	.32	.36	.41
10.00	ELEV	414.13	414.15	414.17	414.19	414.21	414.24	414.28	414.32	414.36	414.41
11.00	DISCHG	.47	.52	.57	.62	.67	.74	.82	.96	1.19	1.56
11.00	ELEV	414.47	414.54	414.61	414.69	414.79	414.90	415.06	415.40	415.97	416.90
12.00	DISCHG	15.14	53.17	69.16	70.42	70.27	68.45	63.93	59.36	54.10	48.06
12.00	ELEV	418.02	418.92	419.47	419.63	419.58	419.44	419.27	419.10	418.94	418.80
13.00	DISCHG	42.78	38.22	34.27	30.86	27.93	25.40	23.21	21.29	19.59	18.11
13.00	ELEV	418.67	418.57	418.47	418.39	418.32	418.26	418.21	418.17	418.13	418.09
14.00	DISCHG	16.82	15.71	14.73	14.02	13.58	13.15	12.71	12.27	11.84	11.43
14.00	ELEV	418.06	418.04	418.01	417.99	417.96	417.93	417.91	417.88	417.85	417.83
15.00	DISCHG	11.06	10.72	10.43	10.16	9.90	9.62	9.34	9.08	8.85	8.64
15.00	ELEV	417.81	417.79	417.77	417.75	417.73	417.72	417.70	417.68	417.67	417.66
16.00	DISCHG	8.45	8.28	8.13	8.00	7.89	7.78	7.68	7.55	7.39	7.23
16.00	ELEV	417.64	417.63	417.63	417.62	417.61	417.60	417.60	417.59	417.58	417.57
17.00	DISCHG	7.08	6.94	6.82	6.71	6.62	6.53	6.46	6.39	6.33	6.25
17.00	ELEV	417.56	417.55	417.54	417.54	417.53	417.53	417.52	417.52	417.51	417.51
18.00	DISCHG	6.13	6.04	5.96	5.88	5.81	5.74	5.68	5.62	5.56	5.51
18.00	ELEV	417.50	417.49	417.48	417.48	417.47	417.46	417.45	417.45	417.44	417.43
19.00	DISCHG	5.46	5.42	5.37	5.33	5.29	5.26	5.23	5.20	5.17	5.12
19.00	ELEV	417.43	417.42	417.42	417.41	417.41	417.41	417.40	417.40	417.40	417.39
20.00	DISCHG	5.06	4.98	4.89	4.81	4.74	4.66	4.60	4.53	4.47	4.42
20.00	ELEV	417.38	417.38	417.37	417.36	417.35	417.34	417.33	417.33	417.32	417.31
21.00	DISCHG	4.37	4.32	4.27	4.23	4.19	4.15	4.11	4.08	4.05	4.02
21.00	ELEV	417.31	417.30	417.30	417.29	417.29	417.28	417.28	417.28	417.27	417.27
22.00	DISCHG	3.99	3.97	3.94	3.92	3.90	3.88	3.86	3.85	3.83	3.82
22.00	ELEV	417.27	417.26	417.26	417.26	417.26	417.25	417.25	417.25	417.25	417.25
23.00	DISCHG	3.80	3.79	3.78	3.76	3.75	3.74	3.74	3.73	3.72	3.69
23.00	ELEV	417.24	417.24	417.24	417.24	417.24	417.24	417.24	417.24	417.24	417.23
24.00	DISCHG	3.65	3.57	3.44	3.27	3.09	2.90	2.73	2.56	2.40	2.26
24.00	ELEV	417.23	417.22	417.20	417.19	417.17	417.14	417.13	417.11	417.09	417.07
25.00	DISCHG	2.12	1.99	1.87	1.75	1.65	1.60	1.59	1.58	1.58	1.57
25.00	ELEV	417.06	417.04	417.03	417.02	417.01	416.99	416.98	416.96	416.95	416.93
26.00	DISCHG	1.57	1.56	1.55	1.55	1.54	1.54	1.53	1.52	1.52	1.51
26.00	ELEV	416.91	416.90	416.88	416.87	416.85	416.84	416.82	416.81	416.79	416.78
27.00	DISCHG	1.51	1.50	1.49	1.49	1.48	1.48	1.47	1.47	1.46	1.45
27.00	ELEV	416.77	416.75	416.74	416.72	416.71	416.69	416.68	416.66	416.65	416.64
28.00	DISCHG	1.45	1.44	1.44	1.43	1.43	1.42	1.42	1.41	1.40	1.40
28.00	ELEV	416.62	416.61	416.59	416.58	416.57	416.55	416.54	416.52	416.51	416.50
29.00	DISCHG	1.39	1.39	1.38	1.38	1.37	1.37	1.36	1.36	1.35	1.35
29.00	ELEV	416.48	416.47	416.46	416.44	416.43	416.42	416.40	416.39	416.38	416.36

RUNOFF VOLUME ABOVE BASEFLOW = 2.23 WATERSHED INCHES, 162.59 CFS-HRS, 13.44 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 2 RECORD ID

100 YEAR

EXECUTIVE CONTROL OPERATION COMPUT FROM STRUCTURE 3 TO STRUCTURE 2 RECORD ID
STARTING TIME = .00 RAIN DEPTH = 7.00 RAIN DURATION= 1.00 RAIN TABLE NO.= 2 ANT. MOIST. COND= 2
ALTERNATE NO.= 1 STORM NO.= 3 MAIN TIME INCREMENT = .10 HOURS

OPERATION RUNOFF STRUCTURE 3 — INFLOW
OUTPUT HYDROGRAPH= 1
AREA= .11 SQ MI INPUT RUNOFF CURVE= 81. TIME OF CONCENTRATION= .24 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= .0321 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.04	392.06	(RUNOFF)
16.41	11.21	(RUNOFF)
17.65	9.39	(RUNOFF)
19.65	7.55	(RUNOFF)
23.65	5.70	(RUNOFF)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .11 SQ.MI.

5.00	DISCHG	.00	.00	.00	.01	.04	.10	.17	.24	.32	.39
6.00	DISCHG	.46	.56	.71	.85	.96	1.07	1.17	1.27	1.37	1.47
7.00	DISCHG	1.57	1.66	1.75	1.84	1.93	2.02	2.11	2.19	2.28	2.36
8.00	DISCHG	2.44	2.62	2.94	3.19	3.55	3.91	4.14	4.31	4.45	4.59
9.00	DISCHG	4.73	4.99	5.44	5.76	5.98	6.16	6.47	6.98	7.36	7.60
10.00	DISCHG	7.80	8.15	8.72	9.21	10.19	11.17	12.25	13.98	15.26	17.07
11.00	DISCHG	18.81	20.42	22.59	24.33	27.45	30.68	50.48	99.51	143.78	258.82
12.00	DISCHG	380.49	369.59	221.80	129.01	86.02	63.66	53.54	46.46	42.90	38.76
13.00	DISCHG	35.08	32.68	29.85	28.18	26.35	24.81	23.58	21.78	20.73	19.74
14.00	DISCHG	18.94	18.31	17.41	16.84	16.01	15.27	14.65	13.75	13.26	13.09
15.00	DISCHG	13.03	13.02	13.01	12.94	12.28	11.60	11.33	11.23	11.20	11.19
16.00	DISCHG	11.19	11.20	11.20	11.20	11.21	11.21	10.87	10.08	9.61	9.45
17.00	DISCHG	9.39	9.37	9.37	9.37	9.37	9.37	9.38	9.38	9.31	8.63
18.00	DISCHG	7.94	7.67	7.57	7.53	7.52	7.52	7.52	7.52	7.52	7.53
19.00	DISCHG	7.53	7.53	7.53	7.53	7.53	7.54	7.54	7.54	7.46	6.79
20.00	DISCHG	6.10	5.82	5.71	5.68	5.67	5.66	5.66	5.66	5.67	5.67
21.00	DISCHG	5.67	5.67	5.67	5.67	5.67	5.67	5.67	5.67	5.67	5.67
22.00	DISCHG	5.68	5.68	5.68	5.68	5.68	5.68	5.68	5.68	5.68	5.68
23.00	DISCHG	5.68	5.68	5.69	5.69	5.69	5.69	5.69	5.69	5.61	4.93
24.00	DISCHG	4.22	3.28	1.51	.54	.19	.07	.02	.00		

RUNOFF VOLUME ABOVE BASEFLOW = 4.81 WATERSHED INCHES, 350.80 CFS-HRS, 28.99 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RESVOR STRUCTURE 2 — ROUTING
INPUT HYDROGRAPH= 1 OUTPUT HYDROGRAPH= 2
SURFACE ELEVATION= 414.00

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.18	279.92	421.41

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .11 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.00	.00	.00	.01	.01
5.00	ELEV	414.00	414.00	414.00	414.00	414.00	414.00	414.00	414.00	414.01	414.01
6.00	DISCHG	.02	.02	.03	.04	.05	.06	.07	.08	.09	.11
6.00	ELEV	414.02	414.02	414.03	414.04	414.05	414.06	414.07	414.08	414.09	414.11
7.00	DISCHG	.12	.14	.16	.17	.19	.21	.23	.25	.27	.30
7.00	ELEV	414.12	414.14	414.16	414.17	414.19	414.21	414.23	414.25	414.27	414.30
8.00	DISCHG	.32	.34	.37	.40	.43	.47	.50	.53	.55	.58
8.00	ELEV	414.32	414.34	414.37	414.40	414.43	414.47	414.50	414.54	414.58	414.63
9.00	DISCHG	.60	.63	.66	.69	.72	.76	.79	.82	.85	.88
9.00	ELEV	414.67	414.71	414.76	414.82	414.87	414.93	414.99	415.05	415.12	415.19
10.00	DISCHG	.91	.94	.97	1.00	1.04	1.08	1.13	1.18	1.24	1.29
10.00	ELEV	415.26	415.34	415.42	415.51	415.60	415.71	415.82	415.95	416.09	416.23
11.00	DISCHG	1.36	1.43	1.51	1.59	2.96	4.55	7.27	15.06	36.76	65.57
11.00	ELEV	416.39	416.57	416.77	416.98	417.15	417.33	417.57	418.02	418.53	419.33
12.00	DISCHG	118.09	251.00	278.18	215.68	159.58	120.04	99.92	83.56	74.25	70.77
12.00	ELEV	420.45	421.27	421.40	421.09	420.75	420.47	420.25	420.07	419.90	419.73
13.00	DISCHG	70.17	66.40	61.72	57.36	51.58	46.29	41.79	37.90	34.52	31.61
13.00	ELEV	419.55	419.36	419.19	419.02	418.88	418.76	418.65	418.56	418.48	418.41
14.00	DISCHG	29.11	26.98	25.12	23.49	22.05	20.75	19.57	18.48	17.47	16.59
14.00	ELEV	418.35	418.30	418.26	418.22	418.18	418.15	418.13	418.10	418.08	418.06
15.00	DISCHG	15.87	15.29	14.83	14.45	14.13	13.90	13.63	13.37	13.14	12.93
15.00	ELEV	418.04	418.03	418.01	418.01	418.00	417.98	417.96	417.95	417.93	417.92
16.00	DISCHG	12.74	12.57	12.42	12.29	12.17	12.06	11.95	11.79	11.58	11.35
16.00	ELEV	417.91	417.90	417.89	417.88	417.87	417.87	417.86	417.85	417.84	417.82
17.00	DISCHG	11.14	10.95	10.78	10.63	10.49	10.37	10.26	10.16	10.07	9.95
17.00	ELEV	417.81	417.80	417.79	417.78	417.77	417.76	417.76	417.75	417.75	417.74
18.00	DISCHG	9.77	9.56	9.35	9.15	8.97	8.81	8.67	8.55	8.44	8.34
18.00	ELEV	417.73	417.71	417.70	417.69	417.68	417.67	417.66	417.65	417.64	417.64
19.00	DISCHG	8.25	8.17	8.10	8.04	7.98	7.93	7.89	7.85	7.81	7.74
19.00	ELEV	417.63	417.63	417.62	417.62	417.62	417.61	417.61	417.61	417.61	417.60
20.00	DISCHG	7.60	7.42	7.24	7.07	6.92	6.78	6.66	6.55	6.45	6.37
20.00	ELEV	417.59	417.58	417.57	417.56	417.55	417.54	417.53	417.53	417.52	417.52
21.00	DISCHG	6.29	6.22	6.16	6.11	6.08	6.05	6.03	6.01	5.99	5.97
21.00	ELEV	417.51	417.51	417.50	417.50	417.50	417.49	417.49	417.49	417.49	417.49
22.00	DISCHG	5.95	5.93	5.92	5.90	5.89	5.88	5.86	5.85	5.84	5.83
22.00	ELEV	417.48	417.48	417.48	417.48	417.48	417.48	417.47	417.47	417.47	417.47
23.00	DISCHG	5.82	5.82	5.81	5.80	5.79	5.79	5.78	5.78	5.77	5.74
23.00	ELEV	417.47	417.47	417.47	417.47	417.47	417.47	417.46	417.46	417.46	417.46
24.00	DISCHG	5.67	5.55	5.36	5.09	4.80	4.52	4.24	3.99	3.74	3.51
24.00	ELEV	417.45	417.44	417.42	417.39	417.36	417.32	417.29	417.27	417.24	417.21
25.00	DISCHG	3.30	3.10	2.91	2.73	2.56	2.41	2.26	2.12	1.99	1.87
25.00	ELEV	417.19	417.17	417.15	417.13	417.11	417.09	417.07	417.06	417.04	417.03
26.00	DISCHG	1.76	1.65	1.60	1.59	1.58	1.58	1.57	1.57	1.56	1.55
26.00	ELEV	417.02	417.01	416.99	416.98	416.96	416.95	416.93	416.91	416.90	416.88
27.00	DISCHG	1.55	1.54	1.54	1.53	1.52	1.52	1.51	1.51	1.50	1.49
27.00	ELEV	416.87	416.85	416.84	416.82	416.81	416.79	416.78	416.77	416.75	416.74
28.00	DISCHG	1.49	1.48	1.48	1.47	1.47	1.46	1.45	1.45	1.44	1.44

28.00	ELEV	416.72	416.71	416.69	416.68	416.66	416.65	416.64	416.62	416.61	416.59
29.00	DISCHG	1.43	1.43	1.42	1.42	1.41	1.40	1.40	1.39	1.39	1.38
29.00	ELEV	416.58	416.57	416.55	416.54	416.52	416.51	416.50	416.48	416.47	416.46

RUNOFF VOLUME ABOVE BASEFLOW = 4.49 WATERSHED INCHES, 327.25 CFS-HRS, 27.04 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 3 RECORD ID

EXECUTIVE CONTROL OPERATION ENDJOB RECORD ID

SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH
A QUESTION MARK(?) INDICATES A HYDROGRAPH WITH PEAK AS LAST POINT.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	AMOUNT (IN)	DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	TIME (HR)	RATE (CFS)	RATE (CSM)
ALTERNATE 1 STORM 1 - 2-YEAR													
STRUCTURE 1	RUNOFF	.11	2	2	.10	.0	3.10	24.00	1.08	---	12.07	89.32	790.4
STRUCTURE 2	RESVOR	.11	2	2	.10	.0	3.10	24.00	.80	417.70	13.12	9.28	82.1
ALTERNATE 1 STORM 2 - 10-YEAR													
STRUCTURE 1	RUNOFF	.11	2	2	.10	.0	5.00	24.00	2.54	---	12.05	212.24	1878.3
STRUCTURE 2	RESVOR	.11	2	2	.10	.0	5.00	24.00	2.23	419.66	12.34	70.53	624.2
ALTERNATE 1 STORM 3 - 100 YEAR													
STRUCTURE 3	RUNOFF	.11	2	2	.10	.0	7.00	24.00	4.81	---	12.04	392.06	3469.6
STRUCTURE 2	RESVOR	.11	2	2	.10	.0	7.00	24.00	4.49	421.41	12.18	279.92	2477.1

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS.......... 1	2	3
STRUCTURE 3	.11			
ALTERNATE 1		.00	.00	392.06
STRUCTURE 2	.11			
ALTERNATE 1		9.28	70.53	279.92
STRUCTURE 1	.11			
ALTERNATE 1		89.32	212.24	.00

Other computations

Outlet Channel Sizing:

Size outlet channel for the 10 year design storm release rate

Q_{10} = 70.5 ft^3/sec
Slope = 1.6%
for rip rap with d_{50} = 18", use Manning's n - 0.041
Design channel to have depth less than ½ * barrel diameter = 27"/2 = 13.5" = 1.13 ft

From open channel flow charts (Federal Highway Administration, "Design Charts for Open-Channel Flow", Hydraulic Design Series No. 3, 1980) or computer program solving Manning's equation. Figure 8, enclosed.
Use trapezoidal channel, with 12' bottom width, 2:1 side slopes, depth = 1.15 ±ft, velocity = 4.4 ft/sec

From U.S. Army, Corp of Engr's, Waterway Experimental Station, Technical Report No H-74-9, page A12:

$d_{50}/d = 0.02 * D/TW * (Q/D^{5/2})^{4/3}$ where:
d_{50} = medium rip rap size, in ft
D = barrel pipe size, in ft
TW = tailwater depth (or depth of flow in channel), in ft
Q = discharge rate, in ft^3/sec

$d_{50} = [0.02 * (2.25'/1.15') * (70.5 \text{ cfs}/2.25'^{5/2})^{4/3}] * 2.25' = 1.72 \text{ ft} = 20.6"$
use d_{50} = 18" (a common size)

required length:

$L/D = 1.8 * (Q/D^{5/2}) + 7$ where:
L = required length, in ft
D = barrel pipe size, in ft
Q = discharge rate, in ft^3/sec

$L = [1.8 * (70.5 \text{ cfs}/2.25'^{5/2}) + 7] * 2.25' = 53.4$ ft, say 54 ft

Use trapezoidal channel, with 12' bottom width, 2:1 side slopes, d_{50} = 18", length = 54'

FIGURE 8

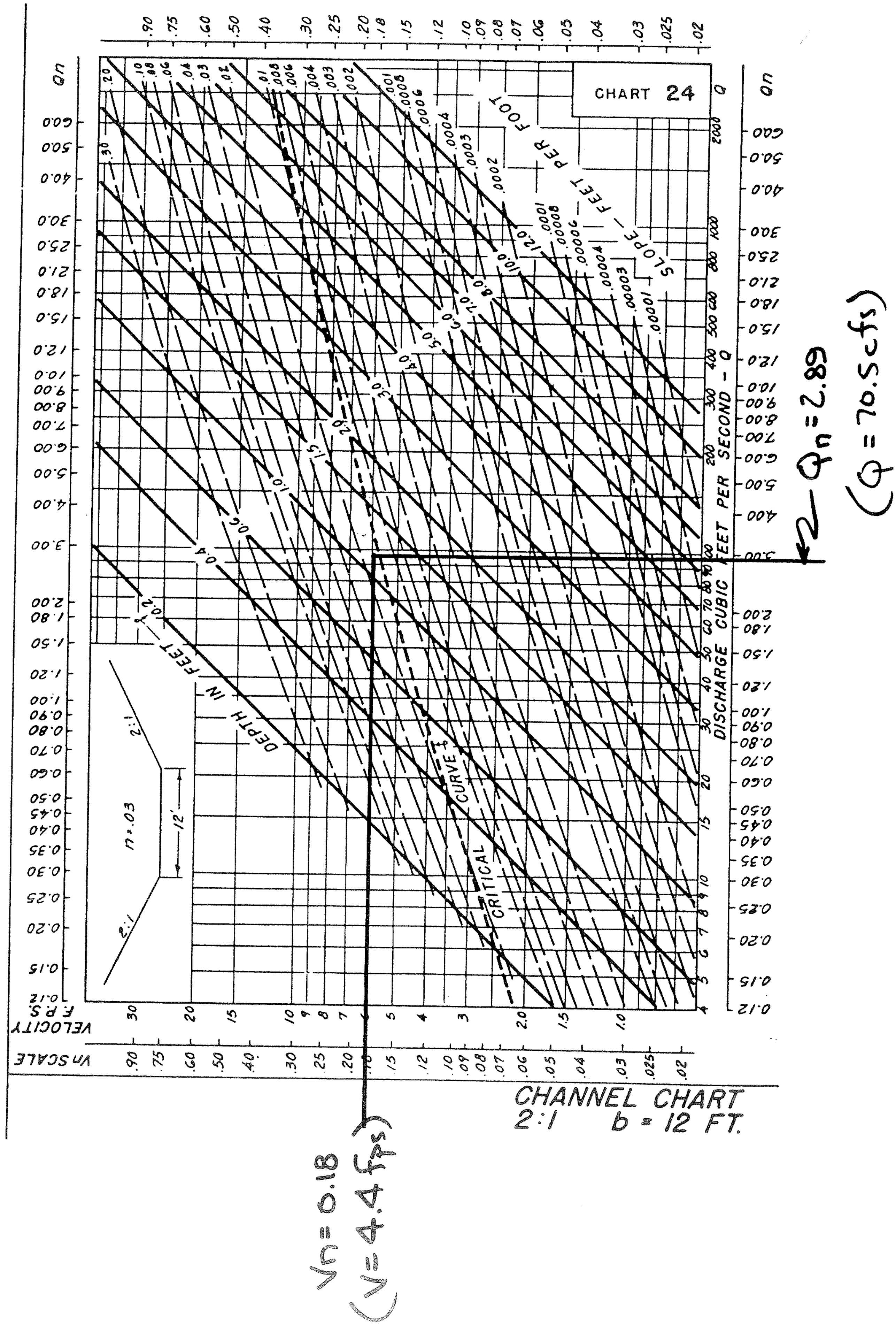


CHART 24
CHANNEL CHART
2:1 b = 12 FT.
n = .03
12'
2:1
DISCHARGE CUBIC FEET PER SECOND - Q
VELOCITY F.P.S.
Vn SCALE
Qn
DEPTH IN FEET
SLOPE — FEET PER FOOT
CRITICAL CURVE
Qn = 2.89
(Q = 70.5 cfs)
Vn = 0.18
(V = 4.4 fps)

FIGURE 9

55

APPENDIX A-39

DESIGN PROCEDURE FOR RIPRAP-LINED CHANNELS

This design of riprap-lined channels is from the National Cooperative Highway Research Program Report No. 108, entitled "Tentative Design Procedure for Riprap-Lined Channels." It is based on the tractive force method and covers the design of riprap in two basic channel shapes, trapezoidal and triangular. (19)

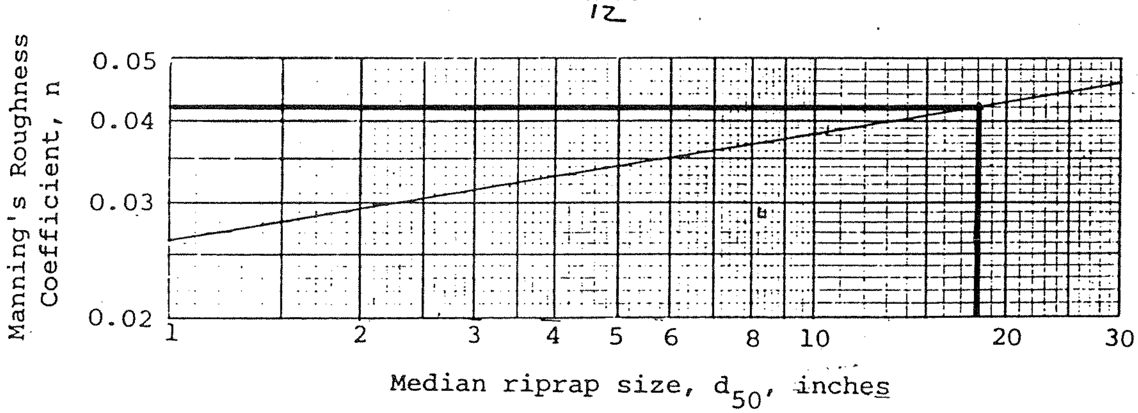
NOTE: This procedure is for the uniform flow in channels and is not to be used for design of riprap deenergizing devices immediately downstream from such high velocity devices as pipes and culverts. See the Standard and Specification for Storm Drain Outlet Protection.

The method in Report No. 108 (design procedure beginning on p. 18) gives a simple and direct solution to the design of trapezoidal channels including channel carrying capacity, channel geometry and the riprap lining. The publication is a very good reference and design aid.

The procedure presented in this Appendix is based on the assumption that the channel is already designed and the remaining problem is to determine the riprap size that would be stable in the channel. The designer would first determine the channel dimensions by the use of Manning's equation. The n value for use in Manning's equation is estimated by estimating a riprap size and then determining the corresponding n value for the riprapped channel from Curve 1, below.

Curve 1 - Manning's "n" for Riprap-Lined Channels

$$n = 0.0395\left(\frac{d_{50}}{12}\right)^{1/6}$$



Water Balance Analysis

Check maximum drawdown during periods of high evaporation and during an extended period of no appreciable rainfall.
The change in storage within a pond = Inflows - Outflows
Potential inflows: Runoff, baseflow, and rainfall
Potential outflows: Infiltration, surface overflow and evaporation (also evapotranspiration)
Assume no inflow from baseflow, no losses for infiltration and because only the permanent pool volume is being evaluated, no losses for surface overflows:
therefore, storage = runoff - evaporation

For permanent pool volume = 2.68 acre-ft = 0.44 watershed inches, a rainfall event yielding 0.44" or more runoff will fill pond.

Evaporation for Maryland: (From Ferguson and Debo, "On-Site Stormwater Management", 1990):

Month	April	May	June	July	Aug.	Sept.
Precip. (ft)	0.30	0.35	0.32	0.36	0.38	0.31
Evap. (ft)	0..36	0.44	0.52	0.54	0.46	0.35

Look at worst case: July
Runoff volume = P * E where
P = precipitation
E = Efficiency of runoff (assumed to be ratio of SCS runoff depth to rainfall depth for 2-year storm)
E = 1.01"/3.1" = .33

Inflow: 0.36 ft * .33 = .119 ft * (72.4 ac/12"/ft) = 0.72 ac-ft
Outflow: surface area * evap losses = 0.70 ac * (0.54 ft) = 0.38 ac-ft

Therefore, drainage area is adequate to support wet pond during normal conditions

For extended period with no rainfall: (assume 45 days during June-July period)
Avg. evaporation: (0.52 ft +0.54 ft)/2 =0.53 ft/ 30.5 days = 0.017 ft/day
for 45 days, loss = 45 * .017 ft/day = 0.78 ft

Assume permanent pool will drop between .8 ft to 1.0 ft for this period. Specify vegetation for the aquatic shelves (to 12") which can tolerate periods of draw down.

Permanent Pool Drain Pipe Sizing:

Design to drain within a 24 hour period

Volume of pool = 2.68 ac-ft * (43,560 ft^2/ac) = 116,741 ft^3
Avg release rate (Q_{avg}) = 116,741 ft^3/ 24 hr * (3600 sec/hr) = 1.35 ft^3/sec
At pond surface, head is max. so release rate (Q) is max., assume $Q_{max} = 2 * Q_{avg}$
$Q_{max} = 2 * 1.35$ cfs = 2.70 ft^3/sec

Try 8" DIP pond drain: Use orifice equation: $Q = cA * (2gh)^{1/2}$
A = .349 ft^2, c = 0.6, h = 414.0- 407.0 = 7.0 ft

$Q = 0.6 * (0.349\ ft^2) * (64.4 * 7.0\ ft)^{1/2} = 4.4\ ft^3/sec$
which is greater than 2.7 ft^3/sec

Use 8" Ductile Iron Pipe Pond Drain

Seepage Control Sizing:

Use a sand filter diaphragm for seepage control. See enclosed material (Appendix A), "Filter Diaphragm Design considerations", Van Aller, 1990, for sample calculations and sizing criteria. Sizing is based on SCS-TR-60, "Earth Dams and Reservoirs", 1984, SCS-SNTC-Technical Note No. 709, SCS-Soil Mechanics Note 1, and SCS-Soil Mechanics Note 3.

Dimensions:
Horizontal: 3 * barrel outside diameter
Vertical, above conduit: 3 * barrel outside diameter
Vertical, below conduit: 2' minimum for firm foundation soils, 1.5 * barrel outside diameter for soft foundation soils.
Thickness: 3 feet minimum
Location: downstream from cutoff trench

Provide drainage pathway to outlet from diaphragm, use 18" deep sand drainage blanket surrounding a 4" PVC/gravel drain system, from sand diaphragm to outlet adjacent to barrel end-section.

Additional computations (not included in Design Example)

Structural concrete design for riser, endwalls (if necessary), and cradle
Slope stability analysis, underdrain or toedrain piping
Anti-floatation computations for riser
Inflow channels to pond
Detailed facility construction cost estimate

Overflow spillway between forebay and pond
Geotechnical investigation and report

Design References Cited in Design Example:

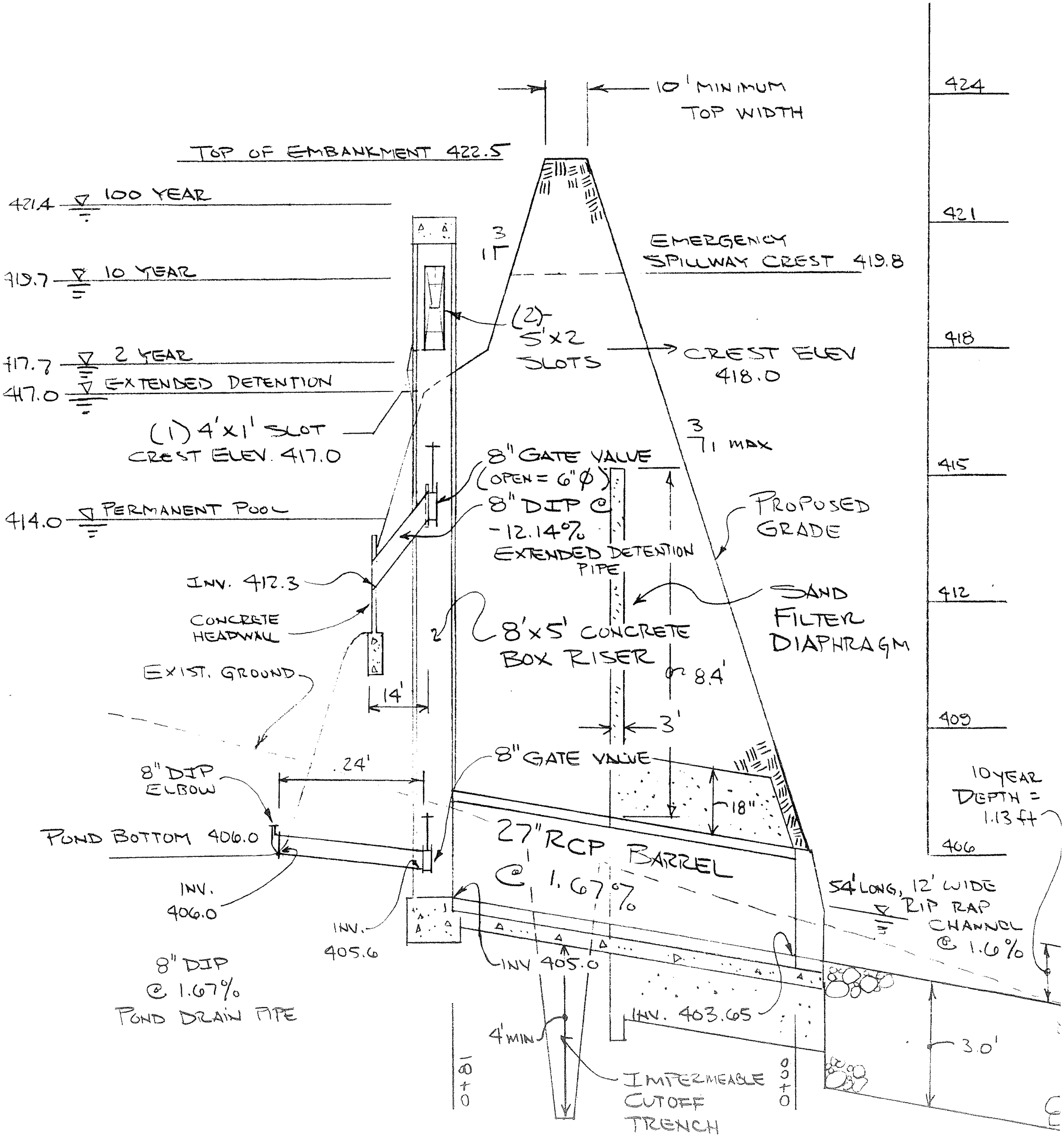
USDA Soils Survey
USGS topographic maps
SCS-TR-55 "Urban Hydrology for Small Watersheds", 1986
SCS-TR-20 "Project Formulation - Hydrology", 1982
Schueler, "Controlling Urban Runoff", 1987
Brater and King, "Handbook of Hydraulics", Sixth Edition, 1976
Federal Highway Administration, "Hydraulic Design of Highway Culverts", 1985
SCS-National Engineering Handbook, Section 5, ES-42
SCS-Engineering Field Manual, "Design Data for Earth Spillways, RTSC-NE-ENG. 1110
SCS-Maryland, Standard and Specifications, Pond, Code 378, 1992
SCS-TR-60, Earth Dams and Reservoirs, 1984
Federal Highway Administration, "Design Charts for Open-Channel Flow", Hydraulic Design Series No. 3, 1980
U.S. Army, Corps of Engineers, Waterway Experimental Station, Technical Report No H-74-9
Ferguson and Debo, "On-Site Stormwater Management", 1990
SCS-SNTC-Technical Note No 709
SCS-Soil Mechanics Note 1
SCS-Soil Mechanics Note 3
Van Aller, "Filter Diaphragm Design Considerations", 1990
Stormworks Software - TR-55

FIGURE 11

Center for Watershed Protection
8737 Colesville Road, Suite 300
Silver Spring, MD 20910

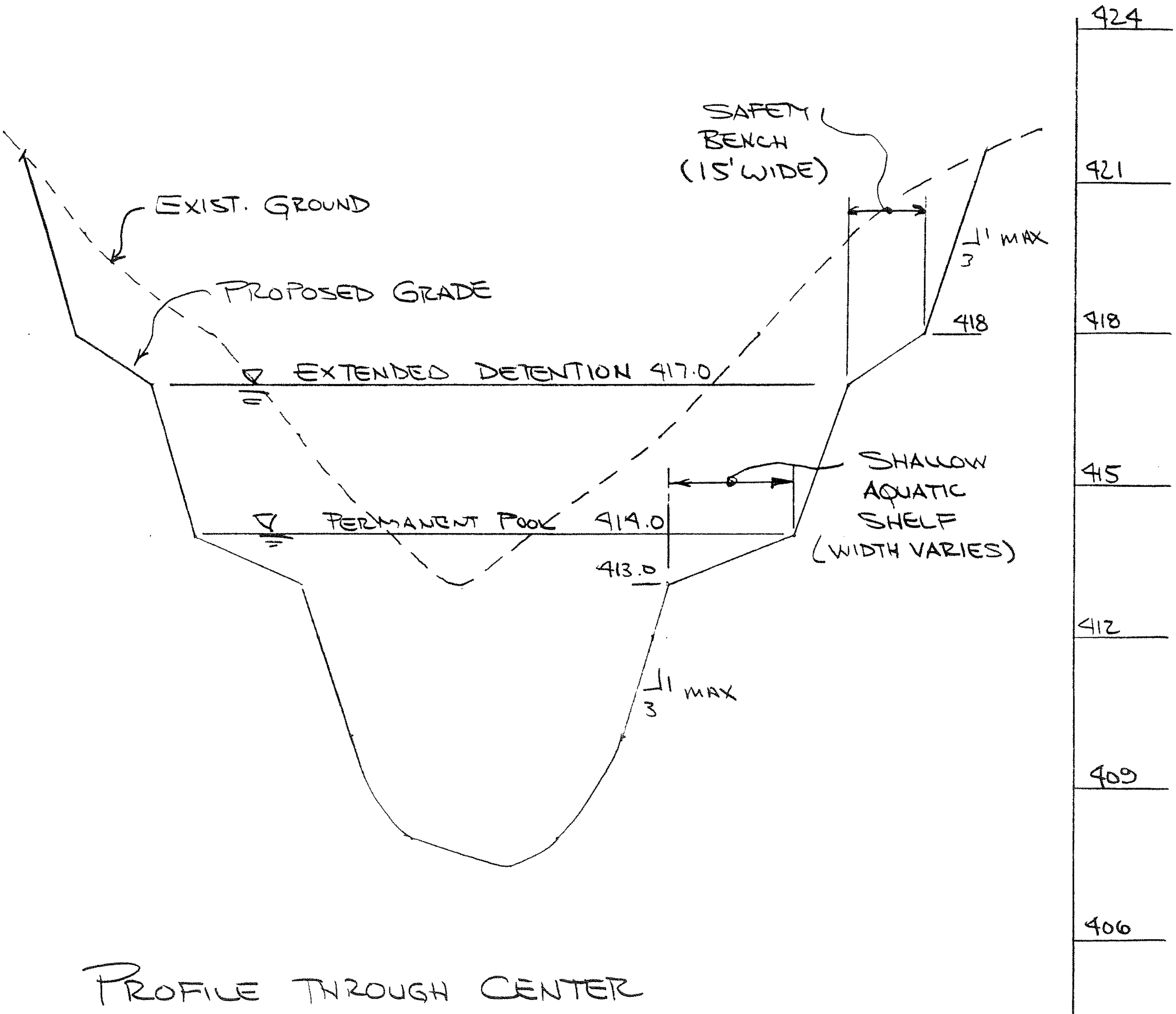
PROFILE THROUGH CENTERLINE OF PRINCIPAL SPILLWAY

SCALE: HORIZ: 1" = 30'
VERT: 1" = 3'



Center for Watershed Protection
8737 Colesville Road, Suite 300
Silver Spring, MD 20910

FIGURE 12



PROFILE THROUGH CENTER OF POND

TYPICAL GRADING

SCALE:
H : 1" = 30'
V : 1" = 3'

APPENDIX A

Filter Diaphragm Design Considerations.

by Harald W. Van Aller, P.E.
Geotechnical Engineer
MD Water Resources Administration
Dam Safety Division
Tawes Building D-3
Annapolis, MD

Presented at the 1990 ASDSO/FEMA
Mid-Atlantic Regional Technical Seminar
Seepage in Dam Safety: Evaluation and Remediation

Introduction

Ray Martin has presented the concept of filter diaphragms for controlling seepage along conduits. Many organizations, including SCS (1985), USBR (1987) and the Maryland Water Resources Administration (MD WRA) are in favor of using these filter zones for controlling seepage along pipes through dams and eliminating the usual anti-seep collars.

The MD WRA is strongly <u>recommending</u> that these filters be used on new dams or remedial repairs in lieu of anti-seep collars, although we are not <u>requiring</u> them. With adequate design and construction supervision, safer structures will result.

Advantages of Filter Diaphragm over Anti-Seep collar:

1. Better compaction of the fill adjacent to the conduit can be obtained if anti-seep collars are not in the way of heavier compaction equipment. Lighter, hand-operated tampers require thinner lifts and more construction time than heavier roller type equipment.

2. Cracks that form in the fill along the conduit will be stopped by the filter and will not propagate completely through the dam. The filter prevents loss of embankment material by "piping" through the crack. The possibility of dam failure is thereby reduced.

3. No time-consuming construction of formwork for concrete anti-seep collars is required. This saves construction time and costs.

4. If a properly designed chimney drain surrounds the pipe as part of the embankment, a separate filter diaphragm is not required.

Disadvantages:

1. Requires careful filter design and suitable filter material must be on-site or imported. However, the design procedure is reasonably straight-forward using SCS-TR-60, SCS-SNTC-Technical Note No. 709, SCS-Soil Mechanics Note 1, and SCS-Soil Mechanics Note 3.

2. Compaction of the filter requires special consideration to prevent settlement of the filter material upon saturation. The recommended installation method involves flooding the filter with water and compacting with vibratory equipment as soon as the water drops below the surface. On small structures (ponds), the filter may be adequately compacted by flooding the sand after placement. (Hall, 1990)

3. Anti-seep collars have been used for a long time and many engineers are comfortable with the concept. Thus, many engineers may be hesitant to try new technology. However, filter diaphragms around conduits for seepage control have been in use for about 20 years. (USBR, 1987)

Additional considerations on filter diaphragms.

Diaphragms can be used on all dam embankments, whether large or small. The SCS Northeast National Technical Center (Chester, PA) requires that diaphragms be used instead of anti-seep collars for "project size" structures as defined in SCS TR-60.[1] They are an option for small ponds approved under the SCS-378 guidelines. (Hall 1990)

However, the MD State Soil Conservation Service office does not endorse use of diaphragms for small ponds because of their concern of the lack of detailed engineering supervision and/or inspection during construction. But every engineer that is designing these ponds needs to convince their client of the importance for construction supervision to prevent the type of failures I mentioned earlier.

A few diaphragm failures have been investigated by the SCS. The failure of a small dam in Texas was attributed to uncontrolled piping along the conduit, and a filter diaphragm that had been constructed failed to protect the dam in this instance. The failure is attributed to settlement of the filter material upon saturation. (Hall, 1990) Perhaps Mr. Talbot, who will be discussing SCS experience with filter diaphragms tomorrow, will discuss some of these failures.

[1] All class (a) dams with a storage-height product greater than 3000, all dams over 35 feet high, and all class (b) and (c) dams.

Filter Design Procedure

Use the worst case scenario, i.e. compare the coarsest filter material to the finest base soil which will be used adjacent to the diaphragm. Remember that the filter must be fine enough to prevent piping of the base soil without clogging and must also be permeable enough to allow for seepage flow to exit quickly.

Sample filter designs are presented in SCS publications SMN-1 and SMN-3. Note that base soil material larger than the No. 4 sieve is not included in filter analysis. Also, TR-60 (page 6-7) states that for base soils with a Plasticity Index (PI) of greater than 15, the maximum $D_{15filter}$ shall be 0.35 mm.

ASTM C-33 sand is often suitable for many fine grained base soils (SMN-1, 1985, page A-2). Gradation of ASTM C-33 Concrete Sand:

Sieve Size	mm	% passing
3/8 in	9.5	100
No. 4	4.75	90 - 100
No. 10	2.00	70 - 100
No. 20	0.850	50 - 85
No. 50	0.300	25 - 50
No. 100	0.150	8 - 30
No. 140	0.106	0 - 15
No. 200	0.075	0 - 5

Gradation of MD SHA (Maryland State Highway Administration) No. 11, which is modified from AASHTO M6 "Fine Aggregate for Portland Cement Concrete" is similar:

Sieve Size	mm	% passing
3/8 in	9.5	100
No. 4	4.75	95 - 100
No. 16	1.18	45 - 85
No. 50	0.300	10 - 30
No. 100	0.150	0 - 10
No. 200	0.075	(use 0 - 5)

It is suggested that each engineer plot the SHA standard gravel gradations on sieve analysis forms and "back figure" a range of acceptable base soil gradations for each. This will reduce the amount of time needed to evaluate filter criteria for a given soil, as the filter will be acceptable if the base soil plots within the pre-determined range.

Filter Diaphragm Dimensions

The horizontal and vertical dimensions of the diaphragm are to be determined in accordance with SCS publications TR-60 and Technical Note No. 709. The resulting dimensions seem rather large, (3 times the pipe diameter all around the pipe) especially if the conduit is more than a few feet in diameter. However, technical Note 709 states that the diaphragm does not need to extend more than five feet beyond the sides of any excavation made to install the conduit.

Note that, in general, any trenches excavated transverse to the dam axis should have side slopes of 2H to 1V or flatter to minimize differential settlements and possible cracking of the embankment. Also note that several of the figures included in Technical Note 709 indicate that it is permissible to construct the conduit on compacted fill. In general this is not good engineering practice and should be avoided if at all possible.

The depth to which the diaphragm is to extend below the conduit depends on how compressible the foundation soil is when compared to the embankment soil. The SCS procedure depends on the determination of a "settlement ratio." [2] Suffice it to say that on firm foundations with a settlement ratio of 0.7 or greater (the settlement ratio for a pipe sitting on rock is 1.0) the filter diaphragm should extend below the pipe to rock, or to a depth of two feet, whichever is encountered first. For softer foundations, the diaphragm should be extended to a depth of 1.5 times the pipe diameter. When in doubt, be conservative.

The minimum diaphragm thickness recommended by Technical Note 709 is 3 feet. This thickness is needed for ease of installation and to minimize the effects of material segregation. Additional thickness may be specified if needed for construction equipment and/or additional seepage capacity.

---

[2] Settlement ratio is defined in SCS publication TR-5. The computation of settlement ratio is complex to say the least. Appendix C of that publication is provided as a handout to assist you in its determination.

Need for a Drain Outlet

TR-60 implies that filters should have a drain outlet to allow seepage flow to exit (page 6-8). If the diaphragm is not tied into the dam's internal drainage system, an outlet should be provided. An exception is made by the SCS literature for dry SWM structures where "*saturated steady state flow will not develop*" as "*[filter] drain capacity is not a necessary design requirement*" (see SMN-1, page 2). A "*drain ... or perforated pipe in the filter is needed if the seepage through the pores of the base soil material exceeds the capacity of the filter.*" However, it would be prudent to include a drain outlet for all diaphragms, as the diaphragm is intended to control excessive leakage from possible cracks rather than normal seepage through the embankment soil.

For slotted and perforated pipes in drains, the opening size must be designed using filter criteria. Cedergren (1989) states that the Army Corps of Engineers' criteria for slotted pipes is $D_{85filter}$/slot width > 1.2. For pipes with circular holes, $D_{85filter}$/hole diameter > 1.0. He also indicates that the USBR recommends $D_{85filter}$/ maximum opening size ≥ 2.

Note that a second filter/drainage layer may be required around the pipe in order to alleviate the need for many very small openings. Don't use fabric wrapped pipe, as the fabric may clog rendering the perforations impenetrable by water. Any open pipe ends should be capped to prevent loss of filter material.

The MD SCS state office has done some field testing and has determined that rigid PVC pipe is more suitable for drain outlets than the commonly available flexible corrugated black polyethylene (PE) pipe. This is because PE pipe can collapse under high fill heights (Thomas, 1990).

Sample Diaphragm Design

A dam is to be built in the Piedmont region of Maryland using local residual soils consisting of micaceous sandy silts and silty sands. The principal spillway pipe will be 48 inch diameter concrete and will be placed in a cradle founded on firm decomposed rock. The dam will be homogeneous and will not have any internal drainage other than the diaphragm.

The procedure presented in SMN-1 is used to design the filter:

Step

1. The fines portion of the embankment soil is non-plastic. A mechanical sieve analysis and hydrometer test on a representative sample of the proposed embankment soil resulted in the following gradation:

Sieve Size	mm	% passing	Adjusted % passing
3/4 in	19.0	100	-
3/8 in	9.5	96	-
No. 4	4.75	91	100
No. 10	2.00	84	92
No. 40	0.425	72	79
No. 60	0.250	65	71
No. 100	0.150	58	64
No. 200	0.075	47	52
Hydrometer	.040	38	42
	.020	23	25
	.007	13	14
	.004	8	9
	.002	5	5

2. Since some of the material is larger than the No. 4 sieve, mathematically remove it from the sample. The adjusted results are also listed in the above table.

3. The new values are plotted. See Figure.

4. Based on Table 1 in SMN-1, the soil is in category 2 with 52% passing the No. 200 sieve.

5. The $D_{15filter}$ is determined from Table 2 in SMN-1. For this soil, the $D_{15filter}$ only needs to be $\leq$ 0.7 mm. And the maximum permissible percent fines (smaller than the No. 200 sieve) in the filter is 5 percent.

6. For permeability, the $D_{15filter}$ must be $\geq 4 \times D_{15base}$. Thus $D_{15filter}$ must be $\geq$ (4 x .0085mm) or .03. But the result is less than 0.1mm, so use 0.1mm.

7. Set maximum filter particle size at 3 inches.

8. Design the filter using the above limits. After plotting the $D_{15filter}$ = 0.1mm and 5% passing No. 200 data points, note that $D_{10filter}$ is less than .5mm. Thus from SMN-1, Table 3, the maximum $D_{90filter}$ is 20mm.

We suspect that ASTM C-33 sand will work. The local supplier says that over the last year, his C-33 material has the following average gradation:

Sieve Size	mm	% passing
3/8"	9.5	100
No. 4	4.75	95.9 - 96.3
No. 8	2.36	83.9 - 84.8
No. 16	1.18	68.9 - 75.3
No. 30	0.600	47.3 - 57.9
No. 50	0.300	17.2 - 22.2
No. 100	0.150	3.0 - 5.0

Now plot these values on the same sheet. It appears that the local C-33 material will make an acceptable filter.

Finally, the guidelines in TR-60 and Technical Note 709 are used to locate the diaphragm. Since there is no internal drainage system into which to tie the diaphragm, an outlet should be provided. This can be accomplished by including an outlet pipe or by extending the filter material to the downstream toe where it can be protected by a layer of riprap.