POWERS BOULEVARD DETENTION FACILITY

FINAL DRAINAGE REPORT

Prepared for:

City of Colorado Springs
Department of Public Works
30 South Nevada
Colorado Springs, Colorado 80903

Prepared by:

Kiowa Engineering Corporation
419 West Bijou Street
Colorado Springs, Colorado 80905-1308

Kiowa Project No. 89.08.16
D12/R61

January, 1990

ENGINEER'S STATEMENT:

The attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the City/County for drainage reports and said report is in conformity with the master plan of the drainage basin. I accept responsibility for any liability caused by any negligent acts, errors or omissions on my part in preparing this report.

Kiowa Engineering Corporation, 419 W. Bijou, Colorado Springs, Colorado 80905-1308

_______________________ 4-12-90
Date

19310

I. INTRODUCTION

Authorization

The preparation of this final drainage report was authorized under the terms of the agreement between the City of Colorado Springs and Kiowa Engineering Corporation dated August 14, 1989.

Purpose and Scope

The purpose of the final drainage report for the Powers Boulevard Detention Facility is to refine the preliminary hydrologic and hydraulic analyses summarized in the Preliminary Design Report. Specifically, the scope of this report is as follows:

1. Address review comments related to the hydrologic analysis contained within the Preliminary Design Report.
2. Refine the hydrologic model used to determine the stage, storage, and discharge relationships for the detention facility.
3. Analyze the hydrologic characteristics related to the sizing of water quality features within the detention facility, based upon climatological data for the Colorado Springs area.
4. Prepare final recommendations for the layout of the detention facility and the various appurtenant structures.

Review comments were received from City utility departments, and from CH2M-Hill, Inc., regarding the design of the detention facility. The assumptions made during the preliminary design report preparation regarding the surface area draining to the facility have been specifically readdressed (reference CH2M-Hill, Inc., letter of October 6, 1989).

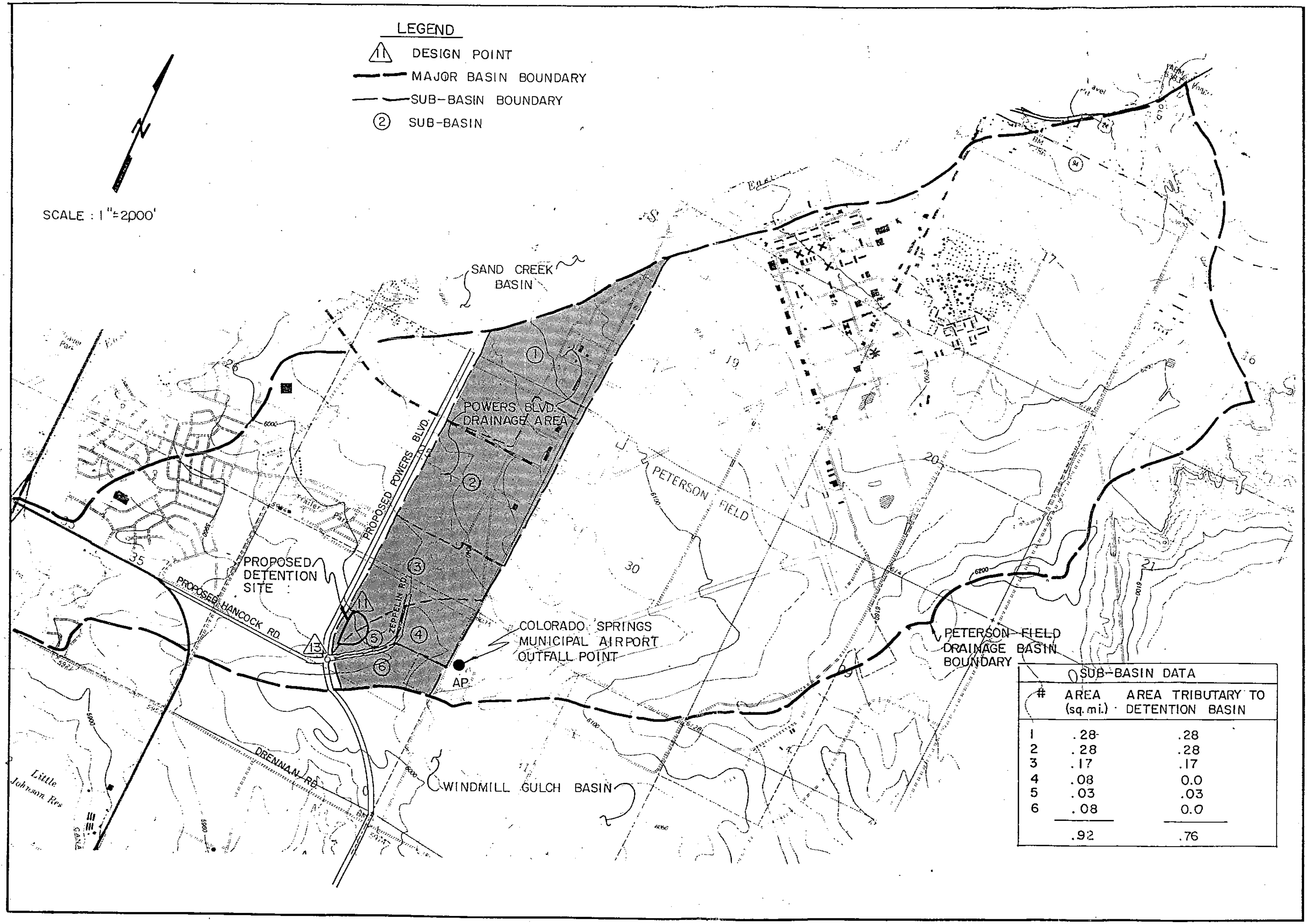
II. HYDROLOGIC ANALYSIS

Shown on Figure 1 is the sub-basin map used to develop the hydrologic model for the sizing of the detention facility. The "Powers Boulevard" drainage area, shown as the shaded area on Figure 1, has been reevaluated. Field visits and further review of the Powers Boulevard Design Plans prepared by CH2M Hill, Inc., were used to confirm the areas to be directly routed to the detention facility. In the Preliminary Design Report, it was assumed that sub-basins 1 through 6 would be tributary to the detention facility (Reference Figure 8, Sub-basin delineation, Powers Boulevard Drainage Report, prepared by CH2M Hill, Inc.). It was confirmed that sub-basins 4 and 6 drain to the existing concrete swale along Zeppelin Road, and it is not practical to route these two basins through the detention facility. Summarized on Table 1 is peak flow data for the revised hydrologic analysis, which eliminated basins 4 and 6. The TR-20 computer output is contained within Appendix A. The peak flow data shown on Table 1 will be used in sizing the detention facility storage area and outlet structure(s).

Water Quality Hydrology

Contained within Appendix B is a description of the analysis which will be used to size the water quality features of the Powers Boulevard Detention Facility. The analysis is based upon climatological data for the Colorado Springs area and provides for a methodology to size water quality pond volumes of an optimum size to store and treat urban runoff.

Based upon the methodology summarized in Appendix B, it has been determined that a water quality storage volume of 32 acre feet should be provided within the Powers Boulevard Detention Facility. This is based upon the precipitation and runoff parameters for a 24-hour storm separation time, and 24-hour release time for the water quality storage area. The depth of the water quality pool will be 3.5 to 4-feet. A 24-hour release time will be used to control the retention time. The water quality pond will be drained by a culvert controlled by an



SUB-BASIN DATA		
#	AREA (sq. mi.)	AREA TRIBUTARY TO DETENTION BASIN
1	.28	.28
2	.28	.28
3	.17	.17
4	.08	0.0
5	.03	.03
6	.08	0.0
	.92	.76

Kiowa Engineering Corporation
419 W. Bijou Street
Colorado Springs, Colorado
80905-1308

POWERS BLVD. DETENTION FACILITY
FINAL DRAINAGE REPORT
DRAINAGE BASIN BOUNDARIES

Project No. 89-08-16
Date: 9/89
Design: RNW
Drawn: EAK
Check:
Revisions:

FIGURE 1

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)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 1													
STRUCTURE 11	ADDHYD	.76	7	2	.10	.0	4.60	24.00	3.29	---	6.07	1896.60	2495.5
STRUCTURE 12	RESVOR	.76	7	2	.10	.0	4.60	24.00	3.29	90.93	6.45	536.42	705.8
STRUCTURE 13	ADDHYD	6.66	7	2	.10	.0	4.60	24.00	1.48	---	6.10	2226.44	334.4
ALTERNATE 1 STORM 2													
STRUCTURE 11	ADDHYD	.76	7	2	.10	.0	3.00	24.00	1.82	---	6.08	1038.10	1365.9
STRUCTURE 12	RESVOR	.76	7	2	.10	.0	3.00	24.00	1.82	88.60	6.61	184.01	242.1
STRUCTURE 13	ADDHYD	.92	7	2	.10	.0	3.00	24.00	1.82	---	6.10	342.62	372.4
STRUCTURE 11	ADDHYD	.76	7	2	.10	.0	2.70	24.00	1.55	---	6.09	883.00	1161.8
STRUCTURE 12	RESVOR	.76	7	2	.10	.0	2.70	24.00	1.55	88.04	6.61	161.78	212.9
STRUCTURE 13	ADDHYD	.92	7	2	.10	.0	2.70	24.00	1.55	---	6.11	285.75	310.6

SUMMARY TABLE 2 - SELECTED MODIFIED ATT-KIN REACH ROUTINGS IN ORDER OF STANDARD EXECUTIVE CONTROL INSTRUCTIONS
(A STAR(*) AFTER VOLUME ABOVE BASE(IN) INDICATES A HYDROGRAPH TRUNCATED AT A VALUE EXCEEDING BASE + 10% OF PEAK
A QUESTION MARK(?) AFTER COEFF.(C) INDICATES PARAMETERS OUTSIDE ACCEPTABLE LIMITS, SEE PREVIOUS WARNINGS)

SEC ID	REACH LENGTH (FT)	INFLOW PEAK (CFS)	INFLOW TIME (HR)	OUTFLOW PEAK (CFS)	OUTFLOW TIME (HR)	OUTFLOW+ INTERV.AREA PEAK (CFS)	OUTFLOW+ INTERV.AREA TIME (HR)	BASE-FLOW (CFS)	VOLUME ABOVE BASE (IN)	MAIN TIME INCR (HR)	ITER-ATION #	Q AND A EQUATION COEFF (X)	Q AND A EQUATION POWER (M)	LENGTH FACTOR (K*)	PEAK RATIO O/I (Q*)	S/Q @PEAK (K) (SEC)	ATT-KIN COEFF (C)	PEAK TRAVEL TIME STOR-AGE (HR)	PEAK TRAVEL TIME KINE-MATIC (HR)
ALTERNATE 1 STORM 1																			
2	2700	666	6.1	666	6.1	---	---	0	3.29	.10	0	3.81	1.41	.015	1.000	113	1.00?	.00	.00
3	3600	1386	6.1	1386	6.1	---	---	0	3.29	.10	0	3.30	1.42	.018	1.000	130	1.00?	.00	.00
4	1335	1584	6.1	1584	6.1	---	---	0	1.19	.10	0	3.42	1.36	.001	1.000	56	1.00?	.00	.00
5	1680	1584	6.1	1584	6.1	---	---	0	1.19	.10	0	3.79	1.36	.002	1.000	65	1.00?	.00	.00
5	1680	212	6.0	212	6.0	---	---	0	3.29	.10	0	3.20	1.42	.015	1.000	106	1.00?	.00	.00
ALTERNATE 1 STORM 2																			

2	2700	361	6.1	361	6.1	---	---	0	1.82	.10	0	3.59	1.43	.017	1.000	132	1.00?	.00	.00
+ 3	3600	764	6.1	764	6.1	---	---	0	1.82	.10	0	3.03	1.44	.022	1.000	151	1.00?	.00	.00
+ 5	1680	116	6.1	116	6.1	---	---	0	1.82	.10	0	3.18	1.43	.019	1.000	126	1.00?	.00	.00
+ 2	2700	306	6.1	306	6.1	---	---	0	1.55	.10	0	3.55	1.43	.018	1.000	138	1.00?	.00	.00
+ 3	3600	650	6.1	650	6.1	---	---	0	1.55	.10	0	2.97	1.45	.023	1.000	157	1.00?	.00	.00
+ 5	1680	100	6.1	100	6.1	---	---	0	1.55	.10	0	3.18	1.43	.020	1.000	132	1.00?	.00	.00

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

..SECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS...24 HR DURATION 1 100YR	2 10YR	
0 STRUCTURE 13	.92			
ALTERNATE 1		2226.44	285.75	COMBINED OUTFLOW
STRUCTURE 12	.76			
ALTERNATE 1		536.42	161.78	OUTFLOW FROM BASIN
STRUCTURE 11	.76			
ALTERNATE 1		1896.60	883.00	INFLOW TO BASIN

MAIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

MAIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

MAIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

..AIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

IIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<

IIN - UNEXPECTED RECORD FOUND(IGNORED) >>> <<<
..ND OF 1 JOBS IN THIS RUN

APPENDIX B

Water Quality Analysis

DETERMINATION OF THE OPTIMAL DETENTION POND SIZE FOR THE CITY OF COLORADO SPRINGS, COLORADO

BY

JAMES C.Y. GUO, PH.D., P.E.

SUBMITTED TO

KIOWA ENGINEERING CORPORATION
DENVER, COLORADO

DECEMBER 27, 1989

CONTENTS

DETERMINATION OF
THE OPTIMAL DETENTION POND SIZE
FOR THE CITY OF COLORADO SPRINGS, COLORADO

Background

Detention pond is an effective tool for runoff water quality and quantity control. The storage of a detention pond reduces peak runoff rate. Therefore, the larger the pond is, the more attenuation on peak flow will result. As a common practice, when designing a flood control detention pond, pond size is determined by a design flood with a specified return period such as a 100 year flood. However, considering water quality control, runoff volume treatment on daily events is more important than peak flow rate attenuation on less frequent events. Using the concept of design flood may result in a hugh storage which may be excessive to daily runoffs.

To determine the proper size of a water quality control pond requires to understand local daily rainfall or runoff characteristics including the statistic spectrum of local rainfall and runoff patterns, precipitation distribution, average time interval between storms, and then a risk cost analysis can be performed. Since rainfall pattern varies from one place to another, in this study, the hourly precipitation data collected at the Station 051778 in the City of Colorado Springs by the National Weather Service was used to apply the methodology developed by the Denver Urban Drainage and Flood Control District to the determination of cost effective water quality pond size. It has found that drainage basin runoff coefficient, pond emptying time, and local mean precipitation are important factors.

Work Description

The computer model, PONDRISK, developed by the Department of Civil Engineering, University of Colorado at Denver was employed to analyze the hourly rainfall data collected in the City of Colorado Springs from 1974 to 1989. The model first computes rainfall statistics and then assesses the treatment capacities for a range of pond sizes. The optimal pond size is determined by its performance effectiveness among the pond sizes studied for each hydrologic cases. In the portion of rainfall statistics, the continuous hourly precipitation record is separated into individual storms using six, 12, 24 and 48 hours as separation time intervals. For instance, when using 12 hours as a separation time, any adjacent hourly precipitations occurred with a time interval less than 12 hours are accounted into one single storm. The computer model accumulates rainfall depth and duration for each storm and then computes statistics for average rainfall depth, duration, intensity and dry hours (time period between two adjacent storms.) among storms identified. The second portion of this study was to convert the point precipitations into runoff volumes using runoff coefficient, C. Namely, Runoff Volume = C *(Precipitation - Infiltration Loss)

The infiltration loss was determined to be 0.1 inch.

In the computation, it was assured that before the beginning of each storm, the pond is empty; in other words, the pond emptying hour is equal to the storm separation time. The corresponding average release rate from the pond is determined by the ratio of pond volume to pond emptying time. Whenever, the pond becomes full, the difference between the incoming runoff and the released runoff is considered untreated and overflown. For a selected pond size, the program computes the runoff capture rate which is defined as the ratio of treated runoff volume to the total runoff volume throughout the entire precipitation record.

Results

In this study, there were three runoff coefficients, 0.2, 0.5 and 0.9, used to determine the optimal detention pond sizes expressed in inches/square foot. The detailed explanation of the pond performance optimization methodology can be found in the Appendix A. Results of this study, as tabulated, the statistics of rainfall characteristics vary with respect to the storm separation time interval. The optimal runoff capture rates for different runoff coefficients are around 85% which means that 85% of runoff volume would be treated if the optimal pond size was used.

RAIN DURATION AND DEPTH STATISTICS FOR COLORADO SPRINGS

STORM SEPARATION TIME INTERVAL IN HOURS	DURATION MEAN HOURS	DURATION S.D. HOURS	DURATION SKEWNESS	PRECIPITATION MEAN INCH	PRECIPITATION S.D. INCH	PRECIPITATION SKEWNESS
6.000	5.400	6.860	2.760	0.450	0.470	3.180
12.000	7.530	9.820	2.340	0.460	0.480	3.000
24.000	16.260	20.380	2.220	0.572	0.617	2.828
48.000	32.790	44.420	2.570	0.684	0.751	2.600

RAIN INTENSITY AND DRY HOURS STATISTICS FOR COLORADO SPRINGS

STORM SEPARATION TIME INTERVAL IN HOURS	INTENSITY MEAN IN/HR	INTENSITY S.D. IN/HR	INTENSITY SKEWNESS	TIME INTERVAL MEAN HOURS	TIME INTERVAL S.D. HOURS	TIME INTERVAL SKEWNESS
6.000	1.850	4.480	3.990	92.600	116.900	2.640
12.000	0.078	0.154	11.490	105.900	120.500	2.510
24.000	0.045	0.077	4.480	136.600	126.200	2.320
48.000	0.026	0.047	6.044	168.900	129.200	2.250

NOTE: RAIN SEPARATION TIME= THE MINIMUM TIME INTERVAL BETWEEN TWO ADJACENT RAIN STORMS ON A CONTINEOUS RECORD.

TIME INTERVAL= DRY HOURS BETWEEN ADJACENT RAINSTORMS.

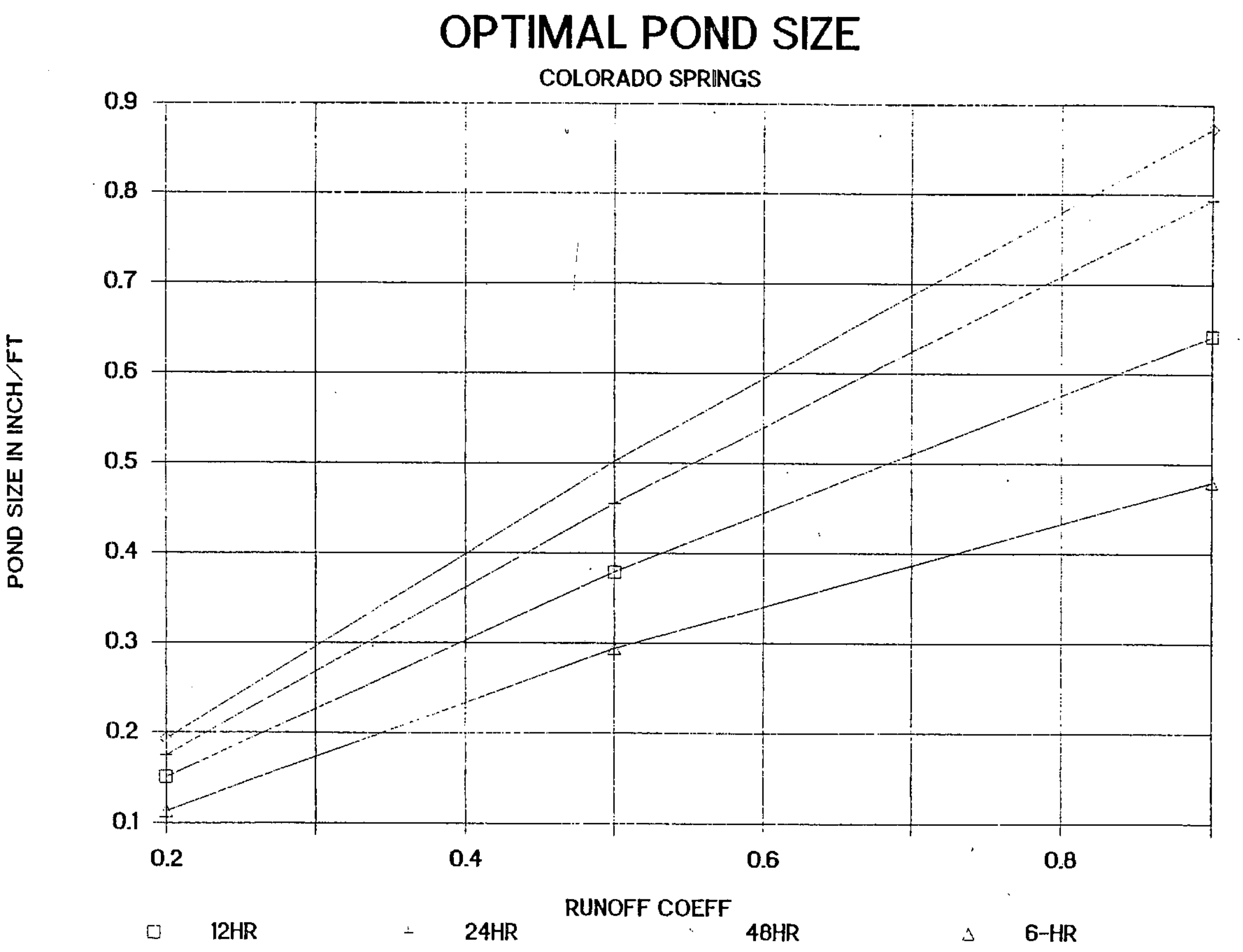
OPTIMAL POND SIZE AND RUNOFF CAPTURE RATE
FOR COLORADO SPRINGS

POND EMPTYING TIME HOURS	C=0.2 PONDSIZE TO MEAN PRECIPI	CAPTURE RATE %	C=0.5 PONDSIZE TO MEAN PRECIPI	CAPTURE RATE %	C=0.9 PONDSIZE TO MEAN PRECIPI	CAPTURE RATE %
6.000	0.257	82.79	0.652	83.57	1.060	82.39
12.000	0.325	86.10	0.816	86.19	1.380	84.97
24.000	0.305	85.36	0.795	86.30	1.390	85.60
48.000	0.277	81.67	0.718	82.84	1.250	87.27

NOTE: C= RUNOFF COEFFICIENT.
CAPTURE RATE= RUNOFF TREATED VOLUME/TOTAL RUNOFF VOLUME

OPTIMAL POND SIZE IN INCHES/SQ FOOT

RUNOFF COEFF	POND EMPTYING TIME IN HOURS 6.000	12.000	24.000	48.000
0.200	0.113	0.151	0.175	0.193
0.500	0.294	0.379	0.455	0.502
0.900	0.480	0.642	0.794	0.873


OPTIMAL POND SIZE
COLORADO SPRINGS
POND SIZE IN INCH/FT
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.2
0.4
0.6
0.8
RUNOFF COEFF
12HR
24HR
48HR
6-HR

Design Example

A detention pond, located in the City of Colorado Springs, is designed to have emptying hours of 24 hours for a drainage basin of 100 acres and runoff coefficient of 0.9. According to the results of this study, using 24 hours as storm separation time, the mean precipitation is 0.572 inch with an average duration of 20.4 hours and intensity of 0.045 inch/hour. The most effective pond size to the mean precipitation is 1.390 which is equivalent to 0.794 inch/square foot or 6.62 acre-foot, 100 acre * (0.794/12) foot, for this drainage basin. The average release rate from this pond is

Pond Volume/Emptying Time = 6.62 acre-ft/24 hour=3.34 cfs

According to the computed statistics, this pond shall have a runoff volume capture rate of 85.60%.

Summary

This study has been successfully performed for the Colorado Springs areas using the methodology developed by the University of Colorado at Denver and the Denver Urban Drainage and Flood Control District. The City of Colorado Springs is one of major metropolitan areas in the State of Colorado. Results from this study shall help engineers to further understand the local rainfall and runoff patterns and to optimize the use of detention pond facility. Living in this fast paced modern society, development of new understanding of our natural environment shall definitely help engineers make more proper decisions, especially for civil engineers who ought to work with the natural environment.

APPENDIX A.

TECHNICAL PAPER:

OPTIMIZATION OF STORMWATER QUALITY CAPTURE VOLUME

OPTIMIZATION OF STORMWATER QUALITY CAPTURE VOLUME

Ben Urbonas, P.E.[1], James C.Y. Guo, Ph.D., P.E.[2]
and L. Scott Tucker, P.E.[3], all M.ASCE

ABSTRACT

There is a need for rational, scientifically based, methods to size urban stormwater runoff facilities for the purpose of water quality enhancement. This paper describes a procedure that utilizes hydrologic principles for optimizing the capture volume. This procedure takes recorded precipitation data and processes it using a quasi-continuous simulation method to determine the number of storm events and total of storm runoff volume being captured within the period being studied. The application of this procedure is illustrated using a 40 year hourly rainfall record at the Denver Raingauge.

INTRODUCTION

The practice of urban stormwater management has until recently focused primarily on quantity issues such as drainage and flood control. Flooding of streets, streams, and rivers has been the main concern. Local governments have constructed thousands of miles of curb, gutter, road side ditches, and storm sewers to convey stormwaters as quickly and efficiently as possible to the nearest stream. This practice along with the increase in impervious surfaces accompanied by urbanization increases the volume and peak flow of runoff for any given rainfall event.

[1] Chief, Master Planning Program, Urban Drainage and Flood Control District, Denver, Colorado.

[2] Associate Professor, University of Colorado at Denver

[3] Executive Director, Urban Drainage and Flood Control District, Denver, Colorado.

Because development results in greater surface runoff rates when compared with undeveloped land, it is common for local governments to attempt mitigating these runoff increases by requiring developers to construct on-site stormwater detention facilities. The concept is to hold back runoff for a short period from each development in small ponds, on parking lots, or wherever space can be found at the site to temporarily store the water. However, on-site detention criteria varies considerably from community to community, the impact of muliples of on-site facilities is uncertain, and long term maintenance is is not a sure thing when it comes to these randomly placed on-site detention facilities.

The alternative to developer constructed on-site detention facilities is regional detention sites. Most people agree that regoinal facilities are more cost efficient and are much more likely to be properly maintained because they would be owned and operated by a public entity. While preferred, it is difficult to fund regional detention. As a result, individual on-site detention requirements are still commonly enforced and the use of on-site detention is the most common approach.

Urban stormwater management, however, is changing quite rapidly from a focus on quantity to a focus on quantity and quality. Two basic issues have and are exerting considerable influence for this change. The first is a fundamental heightening of environmental awareness and concern by the public. There seems to be public support for environmental programs. Stormwater quality in general is probably not a serious problem in relation to concerns such as global warming, Love Canal, sludge disposal, or the Alaska oil spill, and except in some specific situations the impact of urban stormwater on receiving water bodies is not documented or understood. Nevertheless, urban stormwater along with non-point runoff from non-urban sources contribute pollutants to the receiving waters and efforts to do something about it are slowly picking up support and momentum.

The second factor causing a shift toward urban stormwater quality is the Water Quality Act of 1987 (WQA), which amended the Federal Water Pollution Control Act. The WQA of 1987 is a reflection of the public's support for pollution control, and such legislation gives focus and direction to general issues. The WQA requires the Environmental Protection Agency (EPA) to develop a National Pollutant Discharge Elimination System (NPDES) permit program for separate urban stormwater discharges. How the 1987 WQA may impact the citizens, communities, local governments, industry, consultants and the water quality across the United States is yet to be seen. Nevertheless, local governments and industries throughout the United States have a mandate from Congress to control pollutants in urban runoff to the "maximum extent practicable" (MEP). This hopefully means that Congress expects solutions to be practical, pragmatic, and economical.

In order to be practical and effective it is important that technologies for dealing with urban stormwater runoff be available that get the job done. Several simple technologies are emerging that willl be able to be used to remove pollutants from urban stormwater (Urbonas and Roesner, 1986), (Roesner, Urbonas and Sonnen, 1989). These include detention and retention basins, infiltration and percolation at the source of runoff, wetlands, sand filters, and combinations of these techniques. It is important to realize that the same design criteria used to design detention ponds to reduce peak flows cannot be used to design detention and retention basins for stormwater quality purposes.

It is clear from reading the 1986 and 1989 references cited above that the size of runoff event to be captured and treated is a critical factor in the design of stormwater quality detention and retention basins. For example, if the design runoff event is too small, the effectiveness will be reduced because too many storms will exceed the capacity of the facility. Or if the design event is too large, the smaller runoff events will tend to empty faster than desired for adequate settling of pollutants. Thus the larger basins may not provide the needed retention time for the predominant number of smaller events.

A balance between the storage size and water quality treatment effectiveness is needed. Grizzard et. al. (1986) reported results from a field study of basins with extended detention times in the Washington, D. C. area. Based on their observations they suggested that these basins provide good levels of treatment when they are sized to have an average drain time of 24 hours, which equates to a 40 hour drain time for a brim-full basin.

EPA (1986) suggested an analytical methodology for estimating the removal efficiencies of sediments in ponds that have surcharge storage above a permanent pool. Subsequently, Schueler (1987) suggested that the surcharge volume be equivalent to the average runoff event volume. Analysis by the authors in Denver using the EPA analysis technique indicates that wet ponds can be very effective in removing settleable pollutants (i.e., annual TSS removal rates in excess of 80 percent). However, this analysis was limited to ponds that have brim-full surcharge volume equal to one-half inch of runoff from the tributary impervious surfaces, with this volume being drained in 12 hours. Never-the-less, there remains little rationale for the sizing of the capture volume that results in reasonable pollutant load removal while providing reasonably sized cost effective facilities.

Until recently, the primary interest was in drainage and flood control. As a result, the focus was on the larger storm events such as the 2- to 100-year floods. Although drainage and flood control engineers traditionally consider the 2-year event as small, at least in the Denver area it is larger than 95 percent of all the runoff events that typically occur in an urban watershed. Also, through experience we have learned that a detention facility designed to control a 100-year, or even a 2-year flood has little, if any, effect on water quality. Thus, focusing on the traditional drainage design storms is not practical or desirable when considering stormwater quality.

This paper will discuss a method that can be used to find a point of diminishing returns for the sizing of water quality detention facilities. It utilizes rainstorm records as its base instead of synthesized design storms. An example based on the National Weather Service long term precipitation record in Denver is used to illustrate the suggested methodology.

MAXIMIZATION OF STORMWATER RUNOFF CAPTURE VOLUME

Rain Point Diagram.

In 1976 von den Herik (1976) suggested in Holland a rainfall data-based method for estimating runoff volumes. This method is based on long term record of total rainfall and duration of storms. Subsequently Pecher (1978 & 1979) suggested modifications to von den Herik's work to use in the sizing of detention facilities through the use of a Rain Point Diagram (RPD). The authors modified the original method to transform the RPD to a Runoff Volume Point Diagram (RVPD) by multiplying the individual rainstorm depths on the RPD by the runoff coefficient of the tributary watershed.

The PVPD method approximates continuous modelling without setting up a continuous model. The method requires combining individual recorded hourly or 15 minute rainfall increments in a given period of record into separate storm depth totals. Separate storms are identified by a period of time when no rainfall occurs. Very small storms that are not likely to produce runoff can be then be purged from the record. Rainfall storm totals were then converted to runoff depths (i.e., volumes) by multiplying the rainfall depth by the watersheds runoff coefficient (C).

Because the RVPD procedure has not taken into account the effects of several successive rainstorms, it would have a tendency to underestimate the capture effectivenes of detention facilites that have very low release rates. This is because the volume captured during one storm may not be fully drained before the next storm occurs. The RVPD assumes an empty basin for each event.

The procedures used to develop the RVPD method and a case study using the Denver rain gage data will be discussed subsequently. However, to illustrate the use of the RVPD a plot of 63 storms is shown in Figure 1, where the individual storm runoff depth in inches is plotted agaist storm duration. A runoff capture envelope is also plotted on this same figure. This captured storage envelope is bases on the "brim-full" volume of the detention facility and its emptying time. In Figure 1 the runoff capture envelope is based on a detention basin that has a brim-full capacity of 0.3 watershed inches which can be emtied throughthe outlet in 12 hours (sometimes called drawdown time).

All the points above the capture volume envelope line represent individual storms that have sufficient runoff to exceed the available storage volume (i.e., brim-full volume) of the detention facility. Obviously, plotting and counting all points for a long record of rainstorms is a very tedious job. As a result, the authors developed a software package to perform this task.

While this procedure is a simplification of a continuous modelling process, the results should be sufficiently accurate for general planning purposes. This conclusion is supported by the fact that the true accuracy of hydrologic calculations is significantly less than the precision implied by stormwater hydrology models (ASCE, 1984) that are commonly used.

To compensate for storms that may be closely spaced, the authors used a storm separation interval equal to one-half of the emptying time of the brim-full volume. In other words, a storm was defined as separate from a previous storm when this separation condition was satisfied between the end of the last recorded rainfall increment and the beginning of the next one.

The sensitivity of the storm separation period was tested using a storm separation period equal to the brim-full volume emptying time. Virtually no difference was found in the capture volume effectiveness between the separation set at brim-full and one-half of the brim-full emptying time. Such sensitivity tests are suggested whenever other precipitation data are used for this procedure.

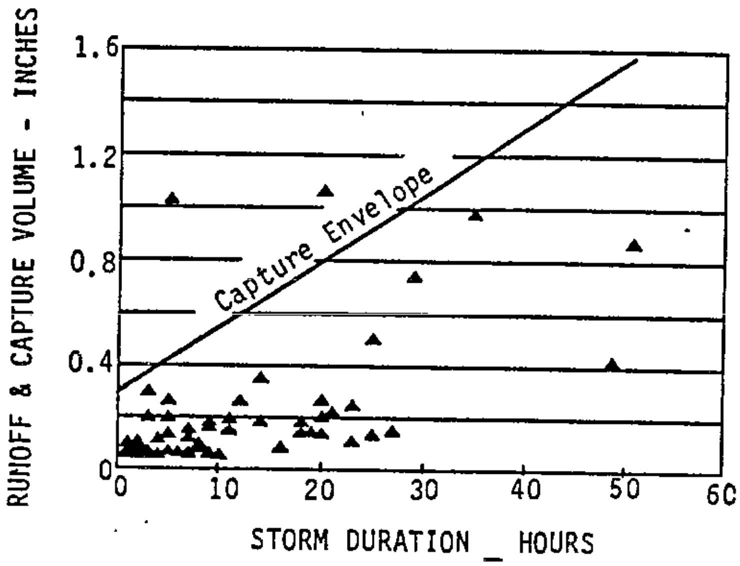


Figure 1. Runoff Volume Point Diagram and Capture Volume Envelope. (1-inch = 24.5 millimeters)

Storage Volume Optimization Procedure

After the total rainfall record is separated into individual storm events, the runoff volume for each storm can be estimated using:

$$V_r = C\ P_t \tag{1}$$

in which, V_r = total runoff volume for a storm, in watershed inches or meters

C = runoff coefficient

P_t = total precipitation over the watershed for the storm in inches of meters.

For a given detention pond or basin that has a brim-full volume V_r with an emptying time T_e, its average release rate, q, is

$$q = V_r\ /\ T_e \tag{2}$$

The runoff volume capture capacity, V_m, of the detention basin for any storm may be estimated using:

$$V_m = V_r + q\ T_d \tag{3}$$

in which, T_d = storm duration. The function ($q\ T_d$) represents the storage beyond the brim-full volume that becomes available during the strom as the result of releases from the basin during the storm's duration.

The actual runoff volume captured and processed for quality improvement through the basin for a given storm is equal to V_r, namely storm runoff volume, when V_r is less than V_m; otherwise it is equal to V_m with the excess runoff volume assumed to overflow without any treatment. Adding the volumes captured for all the storms occurring during the record period gives the total volume captured and treated, V_t, within the period. Thus, the volume capture ratio for the period of rainfall record is defined as,

$$R_v = V_t \ / \ V_{tr} \tag{4}$$

in which, R_v = volume capture ratio for the record period
V_t = total volume captured during the period
V_{tr}= total runoff volume during the same period.

Similarly, the runoff event capture ratio is defined:

$$R_e = N_f \ / \ N \tag{5}$$

in which, R_e = runoff event capture ratio for the period
N_f = number of runoff events that are less than or equal to V_m in runoff volume
N = total number of runoff events.

For the total set of runoff events in the record there is a detention volume that will capture all of the runoff events of record. For practical reasons this maximum pond volume, P_m, was defined to be equal to the 99.9 percent probability runoff event volume for the record period. For the Denver raingage period of record studied (1944-1984) this is equal to to the runoff from 3.04 inches (77.2 mm) of precipitation, or 6.9 times the precipitation of an average runoff producing storm for this period of record. This 99.9 percentile value, namely P_m, was then used to normalize all pond sizes being tested using the following equation:

$$P_r = P \ / \ P_m \tag{6}$$

in which, P_r = relative pond size normalized to P_m
P = pond size being tested
P_m = maximum runoff volume (i.e., 99.9% probability).

The maximization procedure incrementally increases the relative (i.e., normalized) pond size and calculates the runoff volume and event capture ratios (i.e., R_v and R_e) using the RVPD method. Figure 2 illustrates an example of the results of such an analysis using the precipitation record at the Denver gauge between 1944 and 1984. In this example the capture volume was maximized using storms defined by a 6-hour period of separation, 12-hour emptying time for the brim-full basin, and a runoff coefficient C = 0.5 for the watershed.

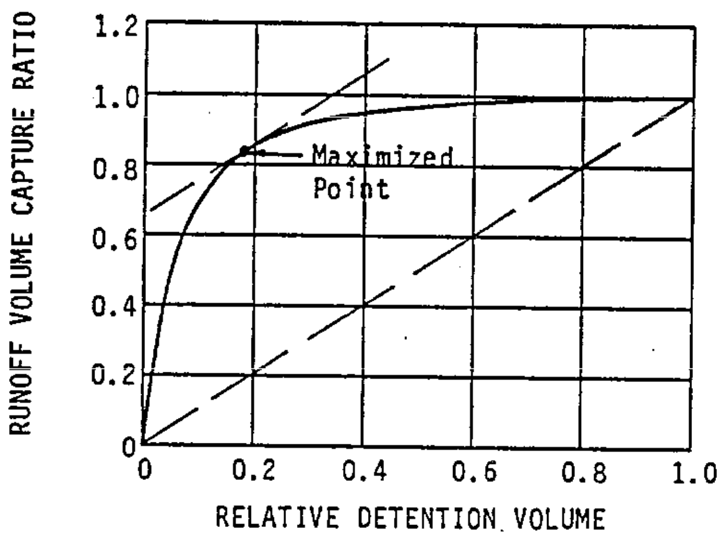


Figure 2. Maximizing Capture Volume.

The maximized pond size occurs where the 1:1 slope is tangent to the runoff capture rate function. Before this point is reached the capture rate increases faster than the relative capure volume size. After this point is reached the increases in the capture rate become less than than corresponding increases in relative capture volume size. In other words, when the point of maximization is passed, diminishing returns are experienced if the capture volume is increased any further. In Figure 2 example, the maximized point occurs when the relative capture volume is equal to 0.18. At this point we capture in total and release slowly approximately 82 percent of the entire runoff depth that has occured during the 40 year study period. This relative capture volume is then converted to actual volume using Equation 6, namely,

$$P = P_r \, P_m$$
$$= (0.18)\ (0.5\ 3.04)$$
$$= 0.27 \text{ watershed inches (6.86 millimeters)}$$

in which, 0.5 is the watershed's runoff coefficient and P_m = 3.04 inches (77.2 mm), namely the depth of rain during the 99.9 percent probability storm.

CASE STUDY USING DENVER RAIN GAUGE DATA

Developing Regional Detention Sizing Guidelines.

The authors investigated the Denver Gauge precipitation data using several storm separation periods, which has been defined as the time between the end of one storm and the beginning of the next. A statistical summary of rainfall characteristics for all storms that exceeded a total of 0.1 inch (2.54 mm) is given in Table 1. A 0.1 inch (2.54 mm) "filter" was used to eliminate from the record the very small storms, of which most are likely not to produce runoff. The urban rainfall and runoff data in the Denver area indicate that approximately 0.08 to 0.15 inches (2.03 to 3.81 mm) of rainfall depth is the point of incipient runoff.

TABLE 1. DENVER RAIN GAUGE HOURLY DATA SUMMARY 1944-1984 STORMS LARGER THAN 0.1 INCHES (2.54 mm) IN DEPTH

SEPARATION BASIS FOR NEW STORM (HOURS)	NUMBER OF STORMS	AVERAGE DEPTH (INCHES)	AVERAGE STORM DURATION (HOURS)	AVERAGE TIME BETWEEN STORMS (HOURS)	NUMBER OF STORMS SMALLER THAN AV.	PERCENT OF STORMS SMALLER THAN AV.
1	1131	0.39*	7	267	802	70.9
3	1091	0.42*	9	275	782	71.7
6	1084	0.44*	11	275	766	70.7
12	1056	0.46*	14	280	748	70.8
24	983	0.51*	23	293	686	69.8
48	876	0.58*	43	310	613	70.0

* Multiply values by 25.4 to convert to millimeters.

A skewed statistical distribution exists with more than two-thirds of the storms having less precipitation than the 40 year average storm depth. Appearently in the Denver area the average runoff producing rain storm depth is a relatively large event.

The distribution of all (i.e., unfiltered) storms vs. total storm precipitation depth when individual storms are defined by a six hours separation period is shown in Figure 3. Note that sixty percent of the precipitation events produced 0.1-inches (2.54 mm) or less of rainfall depth. Over ninety percent of all recorded storms had 0.5-inches or less of rainfall depth. This indicates that the focus, at least in the Denver area should be on the smaller, more frequently occurring storms whenever water quality is being considered.

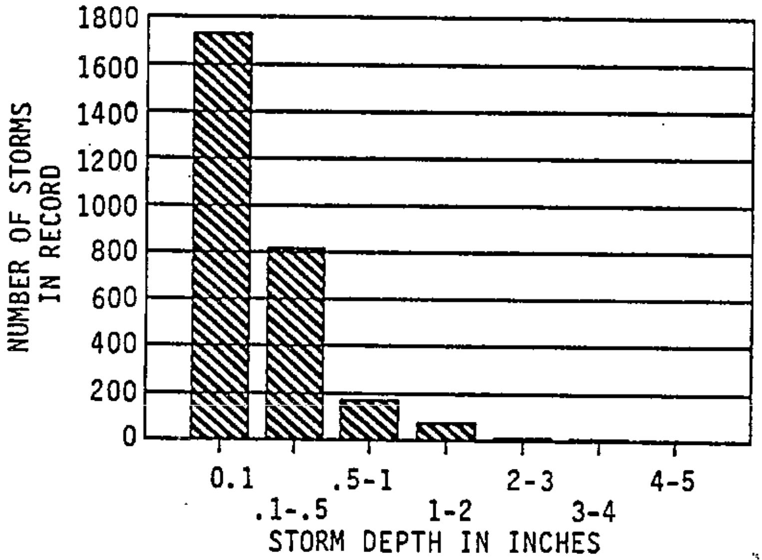


Figure 3. Number of Storms in Denver vs. Storm depth. (One inch = 25.4 millimeters)

Once the precipitation and runoff probabilities were understood, an attempt was made to find a simple yet reasonably accurate relationships for approximating the maximized capture volume of water quality detention basins. As described earlier, the maximized point was defined when additional storage resulted in rapidly diminishing numbers of storms or in the storm runoff volume being totally captured. The final result of this analysis is illustrated in Figure 4, which relates the maximized capture volume to the watershed's runoff coefficient. Separate relationships are shown for the brim-full storage volume emptying time of 12-, 24- and 40-hours.

The captured volume ratio for this relationship exceeds 80 percent and the storm event capture ratio exceeds 86 percent. The storm event capture ratio is of greater importance to the receiving waters because it is the frequency of the shock loads that has the greatest negative effect on the aquatic life in the receiving streams. On the other hand, examination of the precipitation records (i.e., Figure 3) indicates that the volume capture ratio is influenced significantly by the very few very large storms. During these very large runoff events catastrophic flooding is likely to be of primary concern and stromwater quality. It should also be noted that even in these larger events some degree of capture and treatment occurs, although at somewhat reduced efficiency since the detention capacity is exceeded.

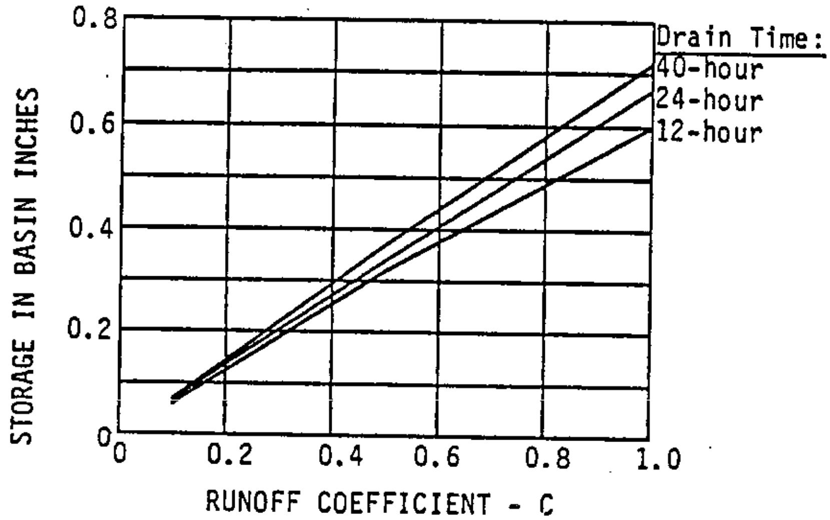


Figure 4. Maximized Capture Volume for Water Quality, Denver Rain Gauge 1944-84 Period. (One inch = 25.4 millimeters)

SENSITIVITY OF PROCEDURE

Capture Volume

Understanding the sensitivity of the event capture ratios to a change in the design capture volume (i.e.. brim-full volume) helps to rationally size water quality facilities. To help define this sensitivity a watershed having a runoff coefficient of C = 1.0 and a storage basin having the maximized volume draining in 12 hours was analyzed. The design capture volume of the basin was increased and decreased in increments and the results were normalized around the maximized volume point. Figure 5 illustrates the findings for this particular case. Although the results varied somewhat between similar tests, the trend was virtually the same for each test that were made using the Denver rain rauge data.

At the ratio of 1.0 on the abcissa, the capture volume has to be almost doubled to capture an additional 10 persent of the runoff events in the fecord. On the other hand, reducing the capture volume by 25 percent results in the reduction of only eight percent in the runoff events that are not captured in total. It needs to be understood that failure to capture a runoff event in total does not mean that the facility will not remove suspended solids. Suspended solids will be removed, but at a somewhat diminished efficiency. The sensitivity of the facility's solids capture efficiency will be discussed next.

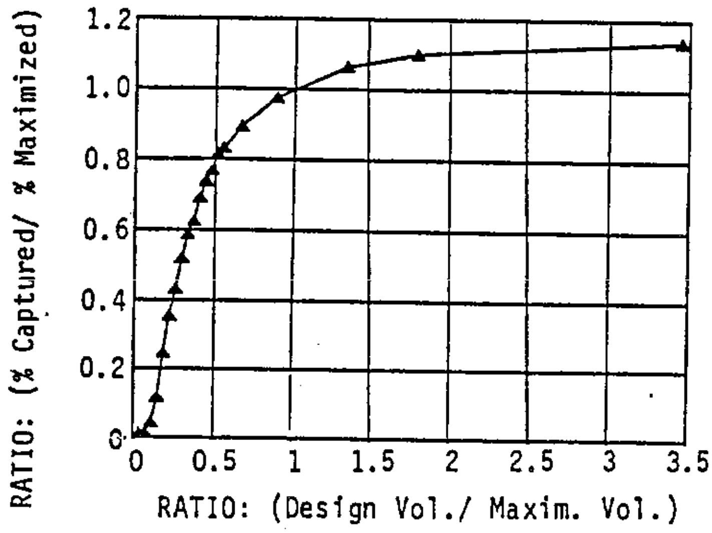


Figure 5. Sensitivity of Capture Volume Size.

Removal of Suspended Sediments

An attempt was made to test the sensitivity of the surcharge detention volume above the permanent pool level on the annual removal rates of total suspended solids in stormwater. For lack of local data on sediment settling velocities, the data given by EPA (1986) was use for several capture volume sizes. Estimates were made of the dynamic removals during the runoff events and the quiescent removals in the pond between storms. When using a surcharge capture volume equal to 70 percent of the maximized volume, the annual removal of TSS by the pond is estimated at 86 percent. This compares to an estimated rate of 88 percent annual removal of TSS when using the maximized capture volume, and only a 90 percent removal rate when using twice the maximized volume.

It appears from the preliminary estimates made using the Denver rain gauge records that it is posible to reduce the capture volume for a wet detention pond and see virtually no effect on the annual removal efficiency of the facility. Figure 5 suggests that the the design volume could be set 25 to 35 percent less than the maximized capture volume. Obviously this suggestion needs more testing. If verified, savings in the construction of water quality enhancement facilities should be possible. Continuous modelling and field testing are suggested as posible methods to test this premise.

Extending the Design Procedure

It is clear from the sensitivity analysis that the capture volume may be reduced somewhat from the maximized point without a significant loss in effectiveness. The designer or the water quality administrator may want to target the capture volume size to serve a runoff event of a desired recurrence probability such as the 85%, 80% or lesser runoff event. Figure 6 illustrates the type of relationships that can be developed if such a goal is desired. Obviously economics and practicality of the capture volume size should be considered when selecting the stormwater quality sizing criteria.

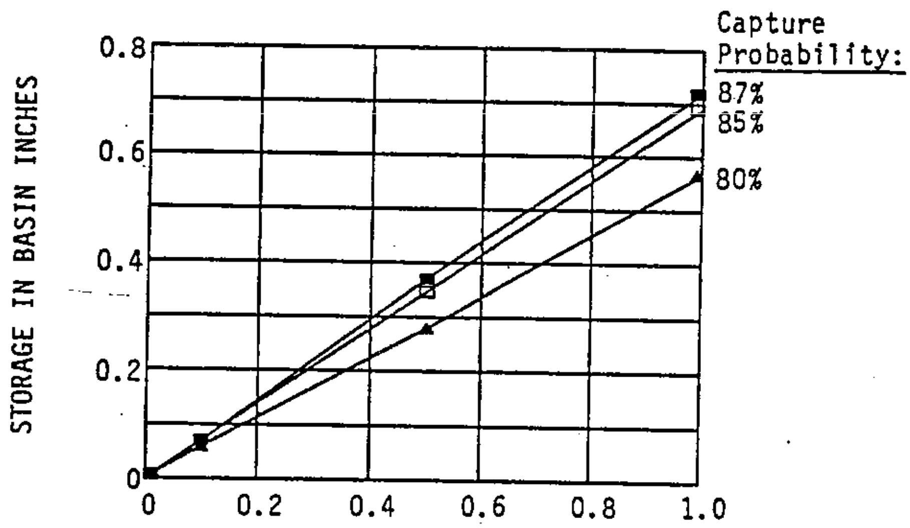


Figure 6. Capture Volumes for a 40-hour Drain Time and Several Runoff Event Capture Probabilities.

From our analysis of the Denver rain gauge data, it looks reasonable, logical and prudent to target the capture of approximately 80th percentile runoff event. This means that the detention facility can be reduced by about 25 to 30 percent in size make it more affordable, while still capturing in total 92 percent of the storm events. When the reduced detention facility is analyzed for impact on the average annual removal in total suspended solids, the difference from the maximized size in water quality being released to the receiving waters is practically not measurable. In other words, the 80 percentile capture volume should provide very good long term TSS removal rates. Also, basins of this size should fit easily within either on-site detention facilities designed for control of runoff peaks or within most landscaping areas of new developments.

At the same time, the removal of dissolved nutrients, such as phosphorous or nitrates, is primarily the function of residence time within the permanent water pool of the "wet pond" between storms. Increasing the capture volume above this pool should have little effect on the removal efficiencies of these compounds. Similarly, "dry ponds" have limited removal efficiencies of dissolved nutrients since their primary removal mechanism is sedimentation (Grizzard, et. al., 1986; Schueler, 1987; Roesner, et. al., 1988; Stahre and Urbonas, 1988).

DETERMINATION OF RUNOFF COEFFICIENT

Using Figure 4 or Figure 6 it is possible to quickly estimate an effective size of a stormwater quality detention basin. Since the engineer has to address smaller runoff events when dealing with stormwater quality, an appropriate runoff coefficient needs to be used. In 1982 EPA published data as part of the NURP study on rainfall depth vs. runoff volume. Although EPA did acknowledge some regional differences, much of the United States was found to be well represented by the data plotted in Figure 7. The curve in this figure is a third order regressed polynomial with the regression coefficient $R^2 = 0.79$. This value of R^2 implies a reasonably strong correlation between the watershed imperviousness, I, in percent and the runoff coefficient, C, for the range of data collected by EPA. Since the NURP study covered two year period, in our opinion this relationship is justified for 2-year recurrence probability and smaller storms.

EXAMPLE OF BASIN SIZING

An example is used next to demonstrate how to determine a "maximized" capture volume for an extended detention basin. A 100 acre (40.5 hectares) multi-family residential tributary watershed that has 60 percent of its area covered by impervious surfaces is used as the example conditions.

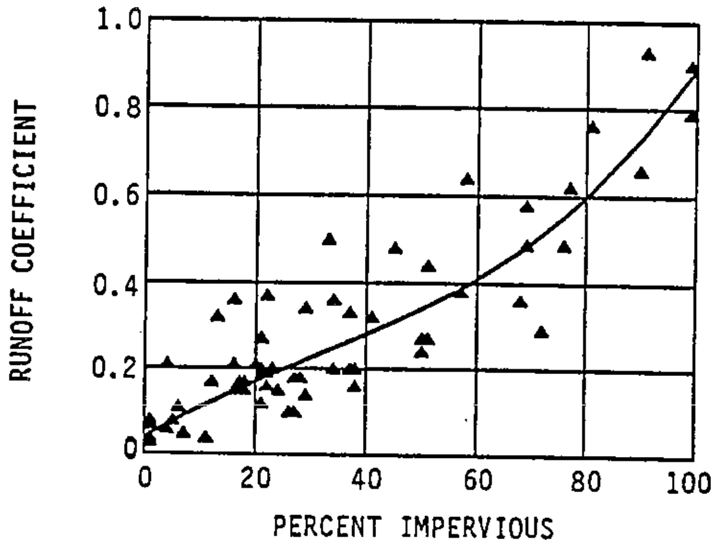


Figure 7. Runoff Coefficient Based on NURP Data for 2-year and Smaller Storms.

Using Figure 7 the runoff coefficient for the watershed, C = 0.4, is estimated. A well performing extended detention basin, according to Grizzard, et. al. (1986), needs to capture approximately the mean seasonal runoff and release it over a 24 hour period, which they suggested could be accomplished if the brim-full volume is drained in 40 to 48 hours. Thus, using the 80 percentile curve on Figure 6 and a brim-full drain time of 40 hours a design volume of 0.22 watershed inches (7.62 mm) is obtained. This is the runoff from a 0.55 inch (14 mm) storm and equates to 1.8 acre feet (2,300 cubic meters) of storage.

CONCLUSIONS

An investigation of sizing stormwater quality facilities for maximized capture of stormwater runoff events and their performance in removing settleable pollutants revealed that simplified design guidelines are possible. These guidelines can be developed using local or regional rain gauge records.

The procedure for the development of these simplified guidelines uses a Runoff Volume Point Diagram method to approximate a continuous simulation process in combination with an optimization routine. This procedure was converted by the authors into computer software.

Using the Denver rain gauge for the testing of this procedure, a figure was prepared that relates a watershed's runoff coefficient, required capture volume and the drain time for this volume. The procedure consists of the following steps:

1. Reduce the recorded rain gauge record (preferably hourly or 15-minute record) to a Rain Point Diagram using several storm separation periods.

2. Transform these Rain Point Diagrams into a Runoff Volume Point Diagrams by multiplying the individual rainfall depths by the watershed's Runoff Coefficient. This can be done for three or more values of C, such as C = 0.1, 0.5 and 1.0 to provide several points on the final design curves.

3. Process the Runoff Volume Point Diagrams through the optimization procedure described earlier using several capture volumes and brim-full storage volume drain times. Suggest using a Runoff Volume Point Diagram that was prepared using a time of storm separation equal to one-half of the desired brim-full drain time.

4. Plot all of the results on a figure similar to Figure 4 for the specific precipitation gauge being used.

5. Perform sensitivity analysis and if appropriate offer options for the sizing of capture volume for several levels of capture probability (eg. Figure 6) and/or TSS removal.

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von den Herik, A.G., "Water Pollution by Storm Overflow From Mixed Sewer Systems," Berichte der ATV, No. 28, Bonn, 1976. (In German)

Table 1. Summary of Discharges with Detention.

Design Point	Description	Area (sq.mi.)	24-hour (cfs) (2) 10-year	24-hour (cfs) (2) 100-year
AP	Airport Outfall	5.74	770	1630 (1)
11 in	Powers Boulevard Basin	.76	1040	1900
11 out		.76	370	540
13	Combined Powers Boulevard/Airport Basins	6.5	510	2440

(1) Assumes future Airport detention basins in-place.

(2) 24-hour storm duration controls peak flow and volume for Powers Boulevard Detention Facility design.

orifice (or other flow control device), and will outfall to the existing box culvert under Powers Boulevard. A final TR-20 run will be compiled for the detention facility, which will account for the water quality pool volume. For the purposes of this analysis, the water quality storage area has been assumed to be empty at the time of a 100-year storm event.

III. HYDRAULICS

The control of the developed inflow to the proposed detention facility will be achieved by extending the existing twin, 6-foot by 10-foot box culvert under Powers Boulevard into the detention area, and constructing a drop inlet structure. The inlet structure will be sized to convey the 100-year peak discharge from the detention basin to the flow shown on Table 1. The drop inlet will be protected with a trash rack, and will discharge into one or both of the bays of the existing box culvert. Presented on Figure 1 is a detail of the drop inlet structure. Control of the water quality pond level will be accomplished through a separate drop inlet structure, with a peak flow capacity equal to the discharge required to drain the pool in no more than 24 hours. This inlet will discharge into the 100-year drop inlet. The estimated rate of discharge is 16 cubic feet per second, based upon the volume obtained using the methodology presented in Appendix B.

The emergency spillway has been sized to convey the developed 100-year peak flow out of the pond, assuming that the principal outlet is blocked. A riprap weir, of approximately 400 feet in length and a 100-year depth of 1.5 feet, has been sized for the detention basin. The crest elevation has been set at 92.5, which is approximately 1.8 feet higher than the low point of Powers Boulevard adjacent to the detention basin (i.e., Powers Boulevard Station 345+11.75). The crest of the emergency overflow weir will be centered at the low point of proposed Powers Boulevard.

Because of the elevation of the low point of the proposed roadway, the embankment/excavation alternative presented in the Preliminary Design Report is recommended for further design. An embankment of approximately 2000-feet in length, with a maximum elevation of 94.0 will be required for the detention facility. The embankment will form the emergency overflow crest, and can be constructed from materials excavated from the active storage area of the detention facility. A 15-foot crest width will be used. A maintenance trail will follow the crest.

A concrete channel will convey the majority of the developed runoff to the detention basin (Reference CH2M-Hill, Inc., Powers Boulevard, Phase I Design Plans, Sheets 26 and 27). Flow from this channel will pass through an energy dissipation/debris collection structure and then spread into the water quality pool area with a channel transition structure. A trickle channel will be required within the water quality pool to convey very low flows to the water quality outlet structure. Cross slopes within the water quality area will be no more than 0.5 percent.

It is recommended that a forebay be constructed within the water quality pond. The forebay will be formed by constructing a berm across the mid-section of the water quality storage area. The forebay will act to further limit the area where routine (annual) maintenance must be conducted. The forebay will be drained by culverts passing under the berm which form the two bays, or all of the water quality pool. A hard surface maintenance trail will be constructed on top of this berm, which will be capable of withstanding an overtopping event. The forebay will primarily catch the more frequent rainfall events which are not of sufficient volume to entirely fill the water quality pool.

Presented on Figure 1 is the conceptual layout of the detention facility, and the various structures which will be required to operate and maintain the detention basin. Quantity cost and estimates for the facility depicted will be prepared during the later preliminary design phases.

APPENDIX A

Hydrologic Analysis

Input File Powcr.DAT

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

JOB TR-20                                              NOPLOTS
TITLE 001 "POWER DETENTION ALT-6"
 ITLE     FUTURE CONDITION (NOT INCL. BASINS 4 & 6)
 5 RAINFL 7            0.25
 8          0.0        .0005      .0015      .0030      .0045
 8          .006       .008       .010       .012       .0143
 8          .0165      .0188      .021       .0233      .0255
 8          .0278      .0320      .0390      .0460      .0530
 8          .06        .075       .10        .400       .70
 8          .725       .750       .765       .780       .790
 8          .800       .810       .820       .825       .830
 8          .835       .840       .845       .850       .855
 8          .860       .8638      .8675      .8713      .8750
 8          .8788      .8825      .8863      0.8900     0.8938
 8          .8975      .9013      .9050      .9083      .9115
 8          .9148      .9180      .9210      .9240      .9270
 8          .9300      .9325      .9350      .9375      .9400
 8          .9425      .9450      .9475      .9500      .9525
 8          .9550      .9575      .9600      .9625      .9650
 8          .9675      .9700      .9725      .9750      .9775
 8          .9800      .9813      .9825      0.9838     0.9850
 8          .9863      .9875      .9888      .9900      .9913
 8          .9925      .9938      .9950      .9963      .9975
 8          .9988      1.000      1.000      1.000      1.000
 9 ENDTBL
 3 STRUCT      12
 8                     82.5       0.         0.
 8                     83.0       6.         0.4
 8                     84.0       16.        2.3
 8                     85.0       21.        7.
 8                     86.0       50.        12.5
 8                     87.0       100.       19.5
 8                     88.0       160.       30.
 8                     89.0       200.       40.5
 8                     90.0       350.       52.5
 8                     91.0       550.       65.
 8                     91.5       610.       71.0
 8                     92.5       670.       90.
 9 ENDTBL
 2 XSECTN     2        1.0
 8                     6020.      0.         0.
 8                     6021.      76.2       8.5
 8                     6022.      260.9      20.
 8                     6023.      556.7      34.5
 8                     6024.      975.3      52.0
 8                     6025.      1529.5     72.5


********************80-80 LIST OF INPUT DATA (CONTINUED)**********************

 9 ENDTBL
 2 XSECTN     3        1.0
 8                     6020.      0.         0.
 8                     6021.      101.8      11.5
 8                     6022.      338.9      26.
 8                     6023.      704.1      43.5
 8                     6024.      1204.4     64.
 [illegible]
```

```
 9 ENDTBL
 2 XSECTN     4          1.0
 8                       6020.        0.          0.
 8                       6021.        50.4        7.5
 8                       6022.        175.5       18.
 8                       6023.        379.5       31.5
 8                       6024.        673.3       48.
 8                       6025.        1064.6      67.5
 8                       6026.        1569.2      90.
 8                       6027.        2187.7      115.5
 8                       6028.        2944.4      144.
 8                       6028.5       3359.8      159.4
 9 ENDTBL
 2 XSECTN     5          1.0
 8                       6020.        0.          0.
 8                       6021.        56.3        7.5
 8                       6022.        196.2       18.
 8                       6023.        424.2       31.5
 8                       6024.        752.8       48.
 8                       6025.        1190.2      67.5
 8                       6026.        1754.4      90.
 8                       6027.        2445.9      115.5
 8                       6028.        3291.9      144.
 8                       6028.5       3756.4      159.4
 9 ENDTBL
 6 RUNOFF 1     01     5 0.282        88.0        0.43
 6 REACH  3  02   5    6 2700.
 6 RUNOFF 1     02     7 0.279        88.0        0.32
 6 ADDHYD 4     02 6 7 5
 6 REACH  3  03   5    6 3600.
 6 RUNOFF 1     03     7 0.169        88.0        0.39
 6 ADDHYD 4     03 6 7 5
 6 SAVMOV 5     03 5   1
 6 REACH  3  04   2    5 1335.
 6 REACH  3  05   5    4 1680.
 6 RUNOFF 1     04     7 0.08         88.0        0.3
 6 REACH  3  05   7    2 1680.
 6 RUNOFF 1     05     3 0.03         88.0        .29
```

********************80-80 LIST OF INPUT DATA (CONTINUED)*********************

```
 6 SAVMOV 5    03 1   5
 6 ADDHYD 4    11 5 3 6                                       1 1 1 1   1
 6 RUNOFF 1    06     5 0.08        88.0        0.27
 6 RESVOR 2    12 6   3  82.5                                 1 1 1 1   1
 6 ADDHYD 4    13 4 3 1
 6 ADDHYD 4    13 1 2 6
 6 ADDHYD 4    13 6 5 1                                       1 1 1 1   1
 6 RUNOFF 1    07     6 0.045       88.0        0.25
 6 RUNOFF 1    08     7 0.045       88.0        0.28
 6 RUNOFF 1    09     5 0.41        49.0        1.1
 6 ADDHYD 4    10 5 7 6
   ENDATA
 7 READHD 8     2
 7 READHD 9 0.0          .0830        5.738       0.000
 8          0.0          .0           .0          .0          .0
 8          0.0          .0           .0          .0          .0
 8          0.0          .0           .0          .0          .0
 8          0.0          .0           .0          .0          .0
 8          0.0          .0           .0          .0          .02
 8          0.06         .12          .19         .29         .41
 8          0.54         .68          .82         .97         1.12
```

```
8        1.27      1.43      1.58      1.72      1.86
8        2.01      2.15      2.29      2.42      2.54
8        2.66      2.78      2.92      3.13      3.47
8        3.95      4.6       5.5       6.51      7.6
8        8.7       9.7       10.6      11.5      12.3
8        13.1      13.8      14.6      15.9      18.2
8        21.7      26.4      44.4      149.2     367.7
8        672.2     1027.6    1380.7    1614.5    1552.1
8        1355.7    1154.4    980.3     840.4     727.9
8        631.1     550.2     486.1     436.9     400.4
8        372.1     346.2     324.2     306.8     293.5
8        283.6     276.1     270.5     266.3     263.1
8        260.7     259.0     256.3     249.0     240.2
8        232.2     225.5     220.2     216.0     212.7
8        210.      207.9     206.2     204.8     203.9
8        203.2     202.7     202.4     202.0     201.8
8        201.5     201.3     201.2     201.0     200.8
8        200.7     200.3     198.7     196.6     194.6
8        192.8     191.3     190.1     189.3     188.7
8        188.2     187.7     187.2     186.8     186.6
8        186.4     186.3     186.1     185.8     185.6
8        185.5     185.4     185.4     185.2     185.
8        184.8     184.7     184.7     184.7     184.5
8        184.3     184.1     184.1     184.1     184.1
8        183.9     183.8     183.5     182.9     182.1
```

********************80-80 LIST OF INPUT DATA (CONTINUED)*********************

```
8        181.3     180.5     179.8     179.2     178.8
8        178.6     178.4     178.2     177.9     177.6
8        177.2     176.7     176.2     176.8     175.5
8        175.2     174.9     174.8     174.6     174.4
8        174.3     174.2     173.6     172.6     171.8
8        171.0     170.4     169.9     169.5     169.2
8        168.9     168.7     168.5     168.4     168.2
8        168.1     168.1     168.0     167.9     167.9
8        167.9     167.8     167.8     167.8     167.8
8        167.8     167.8     167.8     167.8     167.8
8        167.8     167.8     167.8     167.9     167.9
8        167.9     168.0     168.0     168.1     168.1
8        168.2     168.2     168.3     168.3     168.4
8        168.4     168.5     168.6     168.6     168.7
8        168.8     168.9     168.9     169.0     169.1
8        169.2     169.3     169.3     169.4     169.5
8        169.6     169.7     168.9     167.0     165.1
8        163.3     161.9     160.7     159.9     159.3
8        159.0     158.9     158.4     158.1     158.0
8        158.0     158.1     158.1     158.0     157.9
8        157.9     158.0     158.2     158.2     158.2
8        158.1     158.1     158.2     158.4     158.4
8        158.4     158.3     158.3     158.4     158.6
8        158.7     158.6     158.5     158.8     158.6
8        158.8     158.9     158.8     158.7     158.7
8        158.8     158.9     159.0     159.0     158.9
8        158.2     156.3     154.2     152.3     150.8
8        149.6     148.7     148.0     148.5     147.1
9 ENDTBL
  LIST
  INCREM 6              .100
7 COMPUT 7    01    10 0.0         4.6         1.0         7 2   1   1
  ENDCMP 1
  COMPUT 7    01    10 0.0         3.0         1.0         7 2   1   2
  ENDCMP 1
```

7 COMPUT 7 01 - 10 0.0 2.7 1.0 7 2 1 2
ENDCMP 1
ENDJOB 2

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

1

FILE NO. 1

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

EXECUTIVE CONTROL OPERATION READHD RECORD ID

DISCHARGE HYDROGRAPH, HYDROGRAPH LOCATION 2

STARTING TIME= .00 TIME INCREMENT= .08 DRAINAGE AREA= 5.74 BASE FLOW= .00

.00	.00	.00	.00	.00
.00	.00	.00	.00	.00
.00	.00	.00	.00	.00
.00	.00	.00	.00	.00
.00	.00	.00	.00	.02
.06	.12	.19	.29	.41
.54	.68	.82	.97	1.12
1.27	1.43	1.58	1.72	1.86
2.01	2.15	2.29	2.42	2.54
2.66	2.78	2.92	3.13	3.47
3.95	4.60	5.50	6.51	7.60
8.70	9.70	10.60	11.50	12.30
13.10	13.80	14.60	15.90	18.20
21.70	26.40	44.40	149.20	367.70
672.20	1027.60	1380.70	1614.50	1552.10
1355.70	1154.40	980.30	840.40	727.90
631.10	550.20	486.10	436.90	400.40
372.10	346.20	324.20	306.80	293.50
283.60	276.10	270.50	266.30	263.10
260.70	259.00	256.30	249.00	240.20
232.20	225.50	220.20	216.00	212.70
210.00	207.90	206.20	204.80	203.90
203.20	202.70	202.40	202.00	201.80
201.50	201.30	201.20	201.00	200.80
200.70	200.30	198.70	196.60	194.60
192.80	191.30	190.10	189.30	188.70
188.20	187.70	187.20	186.80	186.60
186.40	186.30	186.10	185.80	185.60
185.50	185.40	185.40	185.20	185.00
184.80	184.70	184.70	184.70	184.50
184.30	184.10	184.10	184.10	184.10
183.90	183.80	183.50	182.90	182.10
181.30	180.50	179.80	179.20	178.80
178.60	178.40	178.20	177.90	177.60
177.20	176.70	176.20	176.80	175.50
175.20	174.90	174.80	174.60	174.40
174.30	174.20	173.60	172.60	171.80
171.00	170.40	169.90	169.50	169.20
168.90	168.70	168.50	168.40	168.20
168.10	168.10	168.00	167.90	167.90
167.90	167.80	167.80	167.80	167.80
167.80	167.80	167.80	167.80	167.80
167.80	167.80	167.80	167.90	167.90

}	167.90	168.00	168.00	168.10	168.10
3	168.20	168.20	168.30	168.30	168.40
8	168.40	168.50	168.60	168.60	168.70
7	168.80	168.90	168.90	169.00	169.10
}	169.20	169.30	169.30	169.40	169.50
8	169.60	169.70	168.90	167.00	165.10
8	163.30	161.90	160.70	159.90	159.30
;	159.00	158.90	158.40	158.10	158.00
J	158.00	158.10	158.10	158.00	157.90
8	157.90	158.00	158.20	158.20	158.20
}	158.10	158.10	158.20	158.40	158.40
;	158.40	158.30	158.30	158.40	158.60
8	158.70	158.60	158.50	158.80	158.60
9	158.80	158.90	158.80	158.70	158.70
;	158.80	158.90	159.00	159.00	158.90
8	158.20	156.30	154.20	152.30	150.80
8	149.60	148.70	148.00	148.50	147.10

ENDTBL

XECUTIVE CONTROL OPERATION LIST RECORD ID

ISTING OF CURRENT DATA

	XSECTN NO.	DRAINAGE AREA
? XSECTN	2	1.0000

	ELEVATION	DISCHARGE	END AREA
8	6020.00	.00	.00
	6021.00	76.20	8.50
	6022.00	260.90	20.00
8	6023.00	556.70	34.50
o	6024.00	975.30	52.00
	6025.00	1529.50	72.50

9 ENDTBL

	XSECTN NO.	DRAINAGE AREA
2 XSECTN	3	1.0000

	ELEVATION	DISCHARGE	END AREA
8	6020.00	.00	.00
8	6021.00	101.80	11.50
	6022.00	338.90	26.00
u	6023.00	704.10	43.50
8	6024.00	1204.40	64.00
	6025.00	1850.50	87.50

ENDTBL

```
          XSECTN NO.      DRAINAGE AREA
 2 XSECTN     4             1.0000

                          ELEVATION  DISCHARGE    END AREA
 3                          6020.00        .00         .00
 8                          6021.00      50.40        7.50
 8                          6022.00     175.50       18.00
 3                          6023.00     379.50       31.50
 3                          6024.00     673.30       48.00
 8                          6025.00    1064.60       67.50
 8                          6026.00    1569.20       90.00
 3                          6027.00    2187.70      115.50
 8                          6028.00    2944.40      144.00
1
```



```
 3                          6028.50    3359.80      159.40
 ENDTBL

          XSECTN NO.      DRAINAGE AREA
 2 XSECTN     5             1.0000

                          ELEVATION  DISCHARGE    END AREA
                            6020.00        .00         .00
 8                          6021.00      56.30        7.50
                            6022.00     196.20       18.00
                            6023.00     424.20       31.50
 8                          6024.00     752.80       48.00
 8                          6025.00    1190.20       67.50
                            6026.00    1754.40       90.00
                            6027.00    2445.90      115.50
 8                          6028.00    3291.90      144.00
                            6028.50    3756.40      159.40
 ENDTBL

          STRUCT NO.      ELEVATION  DISCHARGE     STORAGE
 STRUCT       12

 8                            82.50        .00         .00
                              83.00       6.00         .40
                              84.00      16.00        2.30
 8                            85.00      21.00        7.00
                              86.00      50.00       12.50
                              87.00     100.00       19.50
 8                            88.00     160.00       30.00
 8                            89.00     200.00       40.50
                              90.00     350.00       52.50
                              91.00     550.00       65.00
 8                            91.50     610.00       71.00
                              92.50     670.00       90.00
 ENDTBL

                     TIME INCREMENT
 DIMHYD                  .0200

 8              .0000      .0300      .1000      .1900      .3100
                .4700      .6600      .8200      .9300      .9900
               1.0000      .9900      .9300      .8600      .7800
                .6800      .5600      .4600      .3900      .3300
```

6	.2800	.2410	.2070	.1740	.1470
8	.1260	.1070	.0910	.0770	.0660
8	.0550	.0470	.0400	.0340	.0290
8	.0250	.0210	.0180	.0150	.0130

1

8	.0110	.0090	.0080	.0070	.0060
8	.0050	.0040	.0030	.0020	.0010
8	.0000	.0000	.0000	.0000	.0000

9 ENDTBL

COMPUTED PEAK RATE FACTOR = 484.00

TABLE NO. TIME INCREMENT
5 RAINFL 1 .5000

8	.0000	.0080	.0170	.0260	.0350
8	.0450	.0550	.0650	.0760	.0870
8	.0990	.1120	.1260	.1400	.1560
8	.1740	.1940	.2190	.2540	.3030
8	.5150	.5830	.6240	.6550	.6820
8	.7060	.7280	.7480	.7660	.7830
8	.7990	.8150	.8300	.8440	.8570
8	.8700	.8820	.8930	.9050	.9160
8	.9260	.9360	.9460	.9560	.9650
8	.9740	.9830	.9920	1.0000	1.0000

ENDTBL

TABLE NO. TIME INCREMENT
RAINFL 2 .2500

8	.0000	.0020	.0050	.0080	.0110
	.0140	.0170	.0200	.0230	.0260
	.0290	.0320	.0350	.0380	.0410
8	.0440	.0480	.0520	.0560	.0600
8	.0640	.0680	.0720	.0760	.0800
	.0850	.0900	.0950	.1000	.1050
8	.1100	.1150	.1200	.1260	.1330
8	.1400	.1470	.1550	.1630	.1720
	.1810	.1910	.2030	.2180	.2360
	.2570	.2830	.3870	.6630	.7070
8	.7350	.7580	.7760	.7910	.8040
8	.8150	.8250	.8340	.8420	.8490
	.8560	.8630	.8690	.8750	.8810
8	.8870	.8930	.8980	.9030	.9080
8	.9130	.9180	.9220	.9260	.9300
	.9340	.9380	.9420	.9460	.9500
	.9530	.9560	.9590	.9620	.9650
8	.9680	.9710	.9740	.9770	.9800
8	.9830	.9860	.9890	.9920	.9950
	.9980	1.0000	1.0000	1.0000	1.0000

9 ENDTBL

1

TABLE NO. TIME INCREMENT
5 RAINFL 3 .5000

	.0000	.0100	.0220	.0360	.0510
	.0670	.0830	.0990	.1160	.1350
8	.1560	.1790	.2040	.2330	.2680
	.3100	.4250	.4800	.5200	.5500
	.5770	.6010	.6230	.6440	.6640
8	.6830	.7010	.7190	.7360	.7530
8	.7690	.7850	.8000	.8150	.8300
	.8440	.8580	.8710	.8840	.8960
	.9080	.9200	.9320	.9440	.9560
8	.9670	.9780	.9890	1.0000	1.0000

ENDTBL

TABLE NO. TIME INCREMENT
RAINFL 4 .5000

	.0000	.0040	.0080	.0120	.0160
8	.0200	.0250	.0300	.0350	.0400
	.0450	.0500	.0550	.0600	.0650
	.0700	.0750	.0810	.0870	.0930
8	.0990	.1050	.1110	.1180	.1250
	.1320	.1400	.1480	.1560	.1650
	.1740	.1840	.1950	.2070	.2200
8	.2360	.2550	.2770	.3030	.4090
8	.5150	.5490	.5830	.6050	.6240
	.6400	.6550	.6690	.6820	.6940
	.7050	.7160	.7270	.7380	.7480
8	.7580	.7670	.7760	.7840	.7920
	.8000	.8080	.8160	.8230	.8300
	.8370	.8440	.8510	.8580	.8640
8	.8700	.8760	.8820	.8880	.8940
8	.9000	.9060	.9110	.9160	.9210
	.9260	.9310	.9360	.9410	.9460
	.9510	.9560	.9610	.9660	.9710
8	.9760	.9800	.9840	.9880	.9920
	.9960	1.0000	1.0000	1.0000	1.0000

ENDTBL

TABLE NO. TIME INCREMENT
RAINFL 5 .5000

8	.0000	.0020	.0050	.0080	.0110
	.0140	.0170	.0200	.0230	.0260

8	.0290	.0320	.0350	.0380	.0410
	.0440	.0470	.0510	.0550	.0590
	.0630	.0670	.0710	.0750	.0790
8	.0840	.0890	.0940	.0990	.1040
8	.1090	.1140	.1200	.1260	.1330
	.1400	.1470	.1540	.1620	.1710
	.1810	.1920	.2040	.2170	.2330
8	.2520	.2770	.3180	.6380	.6980
	.7290	.7520	.7700	.7850	.7980
	.8090	.8190	.8290	.8380	.8460
	.8540	.8610	.8680	.8740	.8800

[illegible]	.8920	.8970	.9020	.9070
.9120	.9170	.9210	.9250	.9290
.9330	.9370	.9410	.9450	.9490
.9530	.9570	.9600	.9630	.9660
.9690	.9720	.9750	.9780	.9810
.9840	.9870	.9900	.9930	.9960
.9980	1.0000	1.0000	1.0000	1.0000

9 ENDTBL

TABLE NO.	TIME INCREMENT
5 RAINFL 6	.0200

.0000	.0080	.0162	.0246	.0333
.0425	.0524	.0630	.0743	.0863
.0990	.1124	.1265	.1420	.1595
.1800	.2050	.2550	.3450	.4370
.5300	.6030	.6330	.6600	.6840
.7050	.7240	.7420	.7590	.7750
.7900	.8043	.8180	.8312	.8439
.8561	.8678	.8790	.8898	.9002
.9103	.9201	.9297	.9391	.9483
.9573	.9661	.9747	.9832	.9916
1.0000	1.0000	1.0000	1.0000	1.0000

ENDTBL

TABLE NO.	TIME INCREMENT
RAINFL 7	.2500

.0000	.0005	.0015	.0030	.0045
.0060	.0080	.0100	.0120	.0143
.0165	.0188	.0210	.0233	.0255
.0278	.0320	.0390	.0460	.0530
.0600	.0750	.1000	.4000	.7000
.7250	.7500	.7650	.7800	.7900
.8000	.8100	.8200	.8250	.8300
.8350	.8400	.8450	.8500	.8550

.8600	.8638	.8675	.8713	.8750
.8788	.8825	.8863	.8900	.8938
.8975	.9013	.9050	.9083	.9115
.9148	.9180	.9210	.9240	.9270
.9300	.9325	.9350	.9375	.9400
.9425	.9450	.9475	.9500	.9525
.9550	.9575	.9600	.9625	.9650
.9675	.9700	.9725	.9750	.9775
.9800	.9813	.9825	.9838	.9850
.9863	.9875	.9888	.9900	.9913
.9925	.9938	.9950	.9963	.9975
.9988	1.0000	1.0000	1.0000	1.0000

ENDTBL

STANDARD CONTROL INSTRUCTIONS

```
6 RUNOFF 1    1    5      .2820    88.0000     .43000 0 0 0 0 0
6 REACH  3  2  5   6  2700.0000      .0000     .00000 0 0 0 0 0
6 RUNOFF 1    2    7      .2790    88.0000     .32000 0 0 0 0 0
6 ADDHYD 4    2 6 7 5                                0 0 0 0 0 0
6 REACH  3  3  5   6  3600.0000      .0000     .00000 0 0 0 0 0
6 RUNOFF 1    3    7      .1690    88.0000     .39000 0 0 0 0 0
6 ADDHYD 4    3 6 7 5                                0 0 0 0 0 0
6 SAVMOV 5    3 5  1
6 REACH  3  4  2   5  1335.0000      .0000     .00000 0 0 0 0 0
6 REACH  3  5  5   4  1680.0000      .0000     .00000 0 0 0 0 0
6 RUNOFF 1    4    7      .0800    88.0000     .30000 0 0 0 0 0
6 REACH  3  5  7   2  1680.0000      .0000     .00000 0 0 0 0 0
6 RUNOFF 1    5    3      .0300    88.0000     .29000 0 0 0 0 0
6 SAVMOV 5    3 1  5
6 ADDHYD 4   11 5 3 6                                1 1 1 1 0 1
6 RUNOFF 1    6    5      .0800    88.0000     .27000 0 0 0 0 0
6 RESVOR 2   12 6  3    82.5000                      1 1 1 1 0 1
6 ADDHYD 4   13 4 3 1                                0 0 0 0 0 0
6 ADDHYD 4   13 1 2 6                                0 0 0 0 0 0
6 ADDHYD 4   13 6 5 1                                1 1 1 1 0 1
6 RUNOFF 1    7    6      .0450    88.0000     .25000 0 0 0 0 0
6 RUNOFF 1    8    7      .0450    88.0000     .28000 0 0 0 0 0
6 RUNOFF 1    9    5      .4100    49.0000    1.10000 0 0 0 0 0
  ADDHYD 4   10 5 7 6                                0 0 0 0 0 0
  ENDATA

END OF LISTING
1

TR20 XEQ  2/ 1/90   17:53      "POWER DETENTION ALT-6"                          JOB  1   PASS  1
     REV PC/09/83              FUTURE CONDITION (NOT INCL. BASINS 4 & 6)                 PAGE  11

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

EXECUTIVE CONTROL OPERATION COMPUT                                                     RECORD ID
                                  FROM STRUCTURE  1
                                                  TO STRUCTURE 10
      STARTING TIME =   .00   RAIN DEPTH =  4.60   RAIN DURATION=  1.00   RAIN TABLE NO.= 7   ANT. MOIST. COND= 2
      ALTERNATE NO.= 1        STORM NO.= 1   MAIN TIME INCREMENT =   .10 HOURS

    ***  WARNING  REACH   2 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   3 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   4 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

    ***  WARNING  REACH   5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT  ***

OPERATION ADDHYD   STRUCTURE 11

         PEAK TIME(HRS)          PEAK DISCHARGE(CFS)          PEAK ELEVATION(FEET)
             6.07                      1896.60                      (NULL)
```

[illegible]	41.75	(NULL)
12.85	31.65	(NULL)
13.83	27.55	(NULL)
19.86	21.20	(NULL)
23.85	10.77	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.20	1.32	4.08	9.72	19.65	102.35	421.77	923.23	1411.73
6.00	DISCHG	1778.56	1878.35	1519.89	1002.83	631.70	430.47	319.22	243.67	191.29	159.30
7.00	DISCHG	141.64	129.75	115.04	101.13	92.24	87.71	85.37	84.15	83.51	83.22
8.00	DISCHG	82.96	79.87	70.04	58.57	50.67	46.37	44.12	42.95	42.32	41.98
9.00	DISCHG	41.81	41.73	41.69	41.68	41.68	41.70	41.71	41.72	41.73	41.74
10.00	DISCHG	41.72	41.00	38.63	35.86	33.82	32.55	31.88	31.68	31.68	31.60
11.00	DISCHG	31.39	31.27	31.37	31.53	31.54	31.38	31.28	31.39	31.56	31.57
12.00	DISCHG	31.41	31.32	31.42	31.59	31.60	31.44	31.35	31.45	31.63	31.63
13.00	DISCHG	31.46	31.07	30.17	29.17	28.37	27.78	27.46	27.44	27.54	27.51
14.00	DISCHG	27.33	27.04	26.53	26.02	25.67	25.49	25.39	25.34	25.32	25.31
15.00	DISCHG	25.29	24.98	23.98	22.82	22.02	21.58	21.35	21.24	21.17	21.14
16.00	DISCHG	21.12	21.12	21.11	21.11	21.11	21.12	21.12	21.12	21.12	21.13
17.00	DISCHG	21.13	21.13	21.13	21.14	21.14	21.14	21.14	21.15	21.15	21.15
18.00	DISCHG	21.15	21.16	21.16	21.16	21.16	21.16	21.17	21.17	21.17	21.17
19.00	DISCHG	21.18	21.18	21.18	21.18	21.18	21.19	21.19	21.19	21.19	21.20
20.00	DISCHG	21.17	20.43	18.01	15.17	13.10	11.82	11.13	10.91	10.91	10.82
21.00	DISCHG	10.61	10.48	10.57	10.73	10.73	10.56	10.46	10.56	10.73	10.73
22.00	DISCHG	10.57	10.46	10.57	10.73	10.73	10.57	10.47	10.57	10.73	10.73

23.00	DISCHG	10.57	10.47	10.57	10.74	10.74	10.57	10.47	10.57	10.74	10.74
24.00	DISCHG	10.54	9.62	7.11	4.25	2.27	1.20	.63	.34	.18	.09
25.00	DISCHG	.04	.02	.01	.00						

RUNOFF VOLUME ABOVE BASEFLOW = 3.29 WATERSHED INCHES, 1613.43 CFS-HRS, 133.33 ACRE-FEET; BASEFLOW = .00 CFS

PERATION RESVOR STRUCTURE 12

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.45	536.42	90.93

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.01	.10	.40	1.16	2.74	7.29	16.44	26.76	84.16
5.00	ELEV	82.50	82.50	82.51	82.53	82.60	82.73	83.13	84.09	85.20	86.68
6.00	DISCHG	156.97	230.97	381.80	490.89	531.37	531.34	511.93	483.34	450.37	416.25
6.00	ELEV	87.95	89.21	90.16	90.70	90.91	90.91	90.81	90.67	90.50	90.33
7.00	DISCHG	383.28	352.58	329.43	307.69	286.96	267.61	249.82	233.61	218.90	205.58
7.00	ELEV	90.17	90.01	89.86	89.72	89.58	89.45	89.33	89.22	89.13	89.04
8.00	DISCHG	197.96	194.35	190.65	186.73	182.64	178.48	174.35	170.30	166.34	162.49
8.00	ELEV	88.95	88.86	88.77	88.67	88.57	88.46	88.36	88.26	88.16	88.06
9.00	DISCHG	158.14	152.78	147.65	142.76	138.10	133.65	129.41	125.36	121.50	117.82
9.00	ELEV	87.97	87.88	87.79	87.71	87.63	87.56	87.49	87.42	87.36	87.30
10.00	DISCHG	114.31	110.95	107.67	104.42	101.21	97.60	93.85	90.29	86.93	83.76
10.00	ELEV	87.24	87.18	87.13	87.07	87.02	86.95	86.88	86.81	86.74	86.68
11.00	DISCHG	80.76	77.93	75.26	72.74	70.38	68.15	66.04	64.05	62.18	60.43
11.00	ELEV	86.62	86.56	86.51	86.45	86.41	86.36	86.32	86.28	86.24	86.21
12.00	DISCHG	58.77	57.20	55.71	54.33	53.02	51.79	50.62	49.64	48.87	48.13
12.00	ELEV	86.18	86.14	86.11	86.09	86.06	86.04	86.01	85.99	85.96	85.94
13.00	DISCHG	47.43	46.74	46.05	45.35	44.64	43.94	43.24	42.57	41.93	41.31
13.00	ELEV	85.91	85.89	85.86	85.84	85.82	85.79	85.77	85.74	85.72	85.70
14.00	DISCHG	40.72	40.14	39.57	39.01	38.44	37.90	37.36	36.85	36.36	35.89
14.00	ELEV	[illegible]	85.66	85.64	85.62	85.60	85.58	85.56	85.55	85.53	85.51

15.00	DISCHG	35.44	35.00	34.55	34.07	33.58	33.07	32.58	32.10	31.63	31.19
15.00	ELEV	85.50	85.48	85.47	85.45	85.43	85.42	85.40	85.38	85.37	85.35
16.00	DISCHG	30.76	30.35	29.95	29.58	29.22	28.87	28.54	28.22	27.92	27.63
16.00	ELEV	85.34	85.32	85.31	85.30	85.28	85.27	85.26	85.25	85.24	85.23
17.00	DISCHG	27.35	27.09	26.83	26.59	26.36	26.14	25.92	25.72	25.52	25.34
17.00	ELEV	85.22	85.21	85.20	85.19	85.18	85.18	85.17	85.16	85.16	85.15
18.00	DISCHG	25.16	24.99	24.82	24.67	24.52	24.38	24.24	24.11	23.98	23.86
18.00	ELEV	85.14	85.14	85.13	85.13	85.12	85.12	85.11	85.11	85.10	85.10
19.00	DISCHG	23.75	23.64	23.53	23.43	23.34	23.25	23.16	23.07	22.99	22.92
19.00	ELEV	85.09	85.09	85.09	85.08	85.08	85.08	85.07	85.07	85.07	85.07
20.00	DISCHG	22.84	22.76	22.61	22.35	22.00	21.59	21.16	20.94	20.86	20.77
20.00	ELEV	85.06	85.06	85.06	85.05	85.03	85.02	85.01	84.99	84.97	84.95
21.00	DISCHG	20.68	20.59	20.50	20.42	20.33	20.25	20.16	20.08	20.00	19.91
21.00	ELEV	84.94	84.92	84.90	84.88	84.87	84.85	84.83	84.82	84.80	84.78
22.00	DISCHG	19.83	19.75	19.67	19.59	19.51	19.44	19.36	19.28	19.21	19.13
22.00	ELEV	84.77	84.75	84.73	84.72	84.70	84.69	84.67	84.66	84.64	84.63

23.00	DISCHG	19.06	18.98	18.91	18.84	18.77	18.69	18.62	18.55	18.48	18.42
23.00	ELEV	84.61	84.60	84.58	84.57	84.55	84.54	84.52	84.51	84.50	84.48
24.00	DISCHG	18.35	18.28	18.19	18.08	17.95	17.81	17.66	17.51	17.36	17.21
24.00	ELEV	84.47	84.46	84.44	84.42	84.39	84.36	84.33	84.30	84.27	84.24
25.00	DISCHG	17.06	16.91	16.76	16.61	16.47	16.32	16.18	16.04	15.51	14.85
25.00	ELEV	84.21	84.18	84.15	84.12	84.09	84.06	84.04	84.01	83.95	83.88
26.00	DISCHG	14.22	13.61	13.03	12.48	11.95	11.44	10.95	10.48	10.04	9.61
26.00	ELEV	83.82	83.76	83.70	83.65	83.59	83.54	83.50	83.45	83.40	83.36
27.00	DISCHG	9.20	8.81	8.43	8.08	7.73	7.40	7.09	6.79	6.50	6.22
27.00	ELEV	83.32	83.28	83.24	83.21	83.17	83.14	83.11	83.08	83.05	83.02
28.00	DISCHG	5.88	5.19	4.59	4.05	3.58	3.16	2.79	2.47	2.18	1.92
28.00	ELEV	82.99	82.93	82.88	82.84	82.80	82.76	82.73	82.71	82.68	82.66
29.00	DISCHG	1.70	1.50	1.33	1.17	1.03	.91	.81	.71	.63	.56
29.00	ELEV	82.64	82.63	82.61	82.60	82.59	82.58	82.57	82.56	82.55	82.55

RUNOFF VOLUME ABOVE BASEFLOW = 3.29 WATERSHED INCHES, 1611.83 CFS-HRS, 133.20 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION ADDHYD STRUCTURE 13

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.10	2226.44	(NULL)
19.95	197.17	(NULL)
23.81	179.74	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = 6.66 SQ.MI.

2.00	DISCHG	.02	.08	.16	.26	.40	.56	.73	.90	1.08	1.26
3.00	DISCHG	1.45	1.63	1.80	1.97	2.14	2.31	2.46	2.61	2.75	2.92
4.00	DISCHG	3.20	3.66	4.34	5.33	6.52	7.84	9.12	10.26	11.35	12.33
5.00	DISCHG	13.27	14.24	15.99	19.53	26.02	39.63	134.01	450.43	930.24	1507.72
6.00	DISCHG	2035.53	2226.19	2062.21	1804.08	1580.43	1396.91	1237.74	1093.52	971.72	876.05
7.00	DISCHG	800.79	735.83	681.10	634.67	597.12	566.25	540.31	518.35	499.53	483.43
8.00	DISCHG	473.38	462.73	445.87	429.63	416.62	406.06	397.22	389.60	382.98	377.14
9.00	DISCHG	371.32	365.00	359.20	353.92	348.83	344.09	339.56	335.36	331.28	327.39
10.00	DISCHG	323.61	318.68	312.26	305.99	300.39	294.95	289.98	285.67	281.73	277.93
11.00	DISCHG	274.33	271.15	268.29	265.68	263.01	260.42	258.14	256.08	254.22	252.19
12.00	DISCHG	250.24	248.50	247.05	245.66	244.09	242.57	241.31	240.38	239.61	238.65
13.00	DISCHG	237.64	236.22	234.34	232.46	230.69	229.10	227.83	226.93	226.08	225.15
14.00	DISCHG	224.14	223.04	221.74	220.78	219.96	218.52	217.62	216.96	216.22	215.56
15.00	DISCHG	214.99	213.85	211.94	210.24	208.78	207.58	206.55	205.70	204.91	204.22
16.00	DISCHG	203.62	203.00	202.50	202.08	201.60	201.22	200.89	200.47	200.17	199.88
17.00	DISCHG	199.60	199.34	199.08	198.84	198.61	198.39	198.18	198.07	197.88	197.76

18.00	DISCHG	197.01	197.54	197.41	197.32	197.24	197.13	197.10	197.00	196.99	196.92
19.00	DISCHG	196.90	196.91	196.89	196.85	196.87	196.90	196.92	196.87	196.91	196.96
20.00	DISCHG	197.00	195.52	192.31	189.20	186.67	184.60	183.17	182.38	182.08	181.52
21.00	DISCHG	180.98	180.78	180.79	180.80	180.60	180.35	180.28	180.41	180.48	180.37
22.00	DISCHG	180.13	180.07	180.21	180.27	180.17	179.94	179.88	180.02	180.16	180.00
23.00	DISCHG	179.79	179.91	179.86	179.99	179.83	179.60	179.55	179.65	179.74	179.67
24.00	DISCHG	179.34	177.68	174.41	171.29	168.99	17.89	17.69	17.52	17.36	17.21

1

25.00	DISCHG	17.06	16.91	16.76	16.61	16.47	16.32	16.18	16.04	15.51	14.85
26.00	DISCHG	14.22	13.61	13.03	12.48	11.95	11.44	10.95	10.48	10.04	9.61
27.00	DISCHG	9.20	8.81	8.43	8.08	7.73	7.40	7.09	6.79	6.50	6.22
28.00	DISCHG	5.88	5.19	4.59	4.05	3.58	3.16	2.79	2.47	2.18	1.92
29.00	DISCHG	1.70	1.50	1.33	1.17	1.03	.91	.81	.71	.63	.56

RUNOFF VOLUME ABOVE BASEFLOW = 1.48 WATERSHED INCHES, 6345.51 CFS-HRS, 524.39 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP RECORD ID

COMPUTATIONS COMPLETED FOR PASS 1

EXECUTIVE CONTROL OPERATION COMPUT RECORD ID

FROM STRUCTURE 1

TO STRUCTURE 10

STARTING TIME = .00 RAIN DEPTH = 3.00 RAIN DURATION= 1.00 RAIN TABLE NO.= 7 ANT. MOIST. COND= 2

ALTERNATE NO.= 1 STORM NO.= 2 MAIN TIME INCREMENT = .10 HOURS

*** WARNING REACH 2 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 3 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

PERATION ADDHYD STRUCTURE 11

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.08	1038.10	(NULL)
9.91	25.29	(NULL)
12.86	19.28	(NULL)
13.83	16.80	(NULL)
19.87	13.01	(NULL)
23.85	6.62	(NULL)

IME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.28	25.47	156.94	407.55	694.02
6.00	DISCHG	939.01	1034.86	854.39	569.21	361.18	248.33	185.95	143.04	112.98	94.60
7.00	DISCHG	84.48	77.60	68.92	60.66	55.37	52.69	51.32	50.61	50.26	50.11
8.00	DISCHG	49.98	48.15	42.23	35.33	30.58	27.99	26.64	25.94	25.57	25.37
9.00	DISCHG	25.27	25.23	25.22	25.21	25.22	25.24	25.25	25.27	25.28	25.29
10.00	DISCHG	25.28	24.85	23.42	21.74	20.51	19.75	19.34	19.22	19.23	19.18
11.00	DISCHG	19.06	18.99	19.05	19.16	19.16	19.07	19.02	19.08	19.19	19.20
12.00	DISCHG	19.11	19.05	19.12	19.22	19.23	19.14	19.08	19.15	19.26	19.27
13.00	DISCHG	19.17	18.93	18.38	17.78	17.30	16.94	16.74	16.73	16.80	16.78
14.00	DISCHG	16.67	16.50	16.19	15.88	15.67	15.56	15.50	15.48	15.46	15.46
15.00	DISCHG	15.45	15.26	14.65	13.94	13.45	13.19	13.05	12.98	12.94	12.93
16.00	DISCHG	12.92	12.91	12.91	12.91	12.92	12.92	12.92	12.93	12.93	12.93

17.00	DISCHG	12.93	12.94	12.94	12.94	12.94	12.95	12.95	12.95	12.95	12.96

18.00	DISCHG	12.96	12.96	12.97	12.97	12.97	12.97	12.98	12.98	12.98	12.98
19.00	DISCHG	12.99	12.99	12.99	12.99	13.00	13.00	13.00	13.00	13.01	13.01
20.00	DISCHG	13.00	12.54	11.06	9.31	8.04	7.26	6.83	6.70	6.70	6.65
21.00	DISCHG	6.52	6.44	6.49	6.59	6.59	6.49	6.43	6.49	6.59	6.59
22.00	DISCHG	6.49	6.43	6.49	6.60	6.60	6.50	6.43	6.50	6.60	6.60
23.00	DISCHG	6.50	6.43	6.50	6.60	6.60	6.50	6.44	6.50	6.60	6.61
24.00	DISCHG	6.48	5.91	4.37	2.61	1.40	.74	.39	.21	.11	.06
25.00	DISCHG	.03	.01	.00							

RUNOFF VOLUME ABOVE BASEFLOW = 1.82 WATERSHED INCHES, 891.53 CFS-HRS, 73.68 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RESVOR STRUCTURE 12

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.61	184.01	88.60

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.02	1.52	8.18	16.79	23.27
5.00	ELEV	82.50	82.50	82.50	82.50	82.50	82.50	82.63	83.22	84.16	85.08
6.00	DISCHG	59.55	110.24	148.73	169.88	179.04	182.93	183.99	183.39	181.67	179.26
6.00	ELEV	86.19	87.17	87.81	88.25	88.48	88.57	88.60	88.58	88.54	88.48
7.00	DISCHG	176.48	173.52	170.41	167.14	163.76	160.35	155.53	150.70	146.08	141.65
7.00	ELEV	88.41	88.34	88.26	88.18	88.09	88.01	87.93	87.85	87.77	87.69
8.00	DISCHG	137.43	133.35	129.28	125.11	120.86	116.63	112.51	108.53	104.71	101.06
8.00	ELEV	87.62	87.56	87.49	87.42	87.35	87.28	87.21	87.14	87.08	87.02
9.00	DISCHG	96.97	92.86	88.98	85.32	81.88	78.63	75.57	72.68	69.97	67.40
9.00	ELEV	86.94	86.86	86.78	86.71	86.64	86.57	86.51	86.45	86.40	86.35
10.00	DISCHG	64.99	62.70	60.49	58.32	56.18	54.12	52.13	50.25	48.86	47.60
10.00	ELEV	86.30	86.25	86.21	86.17	86.12	86.08	86.04	86.00	85.96	85.92
11.00	DISCHG	46.38	45.22	44.10	43.03	42.02	41.04	40.10	39.20	38.35	37.53
11.00	ELEV	85.88	85.84	85.80	85.76	85.72	85.69	85.66	85.63	85.60	85.57
12.00	DISCHG	36.75	35.99	35.27	34.59	33.93	33.30	32.70	32.12	31.57	31.04
12.00	ELEV	85.54	85.52	85.49	85.47	85.45	85.42	85.40	85.38	85.36	85.35
13.00	DISCHG	30.54	30.05	29.56	29.07	28.58	28.09	27.61	27.15	26.71	26.28
13.00	ELEV	85.33	85.31	85.30	85.28	85.26	85.24	85.23	85.21	85.20	85.18
14.00	DISCHG	25.87	25.48	25.09	24.70	24.32	23.95	23.59	23.25	22.91	22.60
14.00	ELEV	85.17	85.15	85.14	85.13	85.11	85.10	85.09	85.08	85.07	85.06
15.00	DISCHG	22.29	22.00	21.70	21.38	21.05	20.94	20.87	20.81	20.74	20.67
15.00	ELEV	85.04	85.03	85.02	85.01	85.00	84.99	84.97	84.96	84.95	84.93
16.00	DISCHG	20.60	20.53	20.47	20.40	20.34	20.27	20.21	20.14	20.08	20.02
16.00	ELEV	84.92	84.91	84.89	84.88	84.87	84.85	84.84	84.83	84.82	84.80
17.00	DISCHG	19.95	19.89	19.83	19.77	19.71	19.65	19.59	19.54	19.48	19.42
17.00	ELEV	84.79	84.78	84.77	84.75	84.74	84.73	84.72	84.71	84.70	84.68
18.00	DISCHG	19.36	19.31	19.25	19.20	19.14	19.09	19.04	18.98	18.93	18.88
18.00	ELEV	84.67	84.66	84.65	84.64	84.63	84.62	84.61	84.60	84.59	84.58
19.00	DISCHG	18.83	18.78	18.73	18.67	18.63	18.58	18.53	18.48	18.43	18.38
19.00	ELEV	84.57	84.56	84.55	84.53	84.53	84.52	84.51	84.50	84.49	84.48
20.00	DISCHG	18.34	18.29	18.23	18.16	18.08	17.99	17.89	17.79	17.70	17.60

20.00	ELEV	84.47	84.46	84.45	84.43	84.42	84.40	84.38	84.36	84.34	84.32
21.00	DISCHG	17.50	17.41	17.31	17.22	17.12	17.03	16.94	16.85	16.76	16.67
21.00	ELEV	84.30	84.28	84.26	84.24	84.22	84.21	84.19	84.17	84.15	84.13
22.00	DISCHG	16.58	16.49	16.40	16.32	16.23	16.15	16.06	15.89	15.49	15.11
22.00	ELEV	84.12	84.10	84.08	84.06	84.05	84.03	84.01	83.99	83.95	83.91
23.00	DISCHG	14.75	14.40	14.06	13.74	13.43	13.14	12.86	12.59	12.33	12.09
23.00	ELEV	83.87	83.84	83.81	83.77	83.74	83.71	83.69	83.66	83.63	83.61
24.00	DISCHG	11.85	11.61	11.33	11.00	10.62	10.21	9.80	9.40	9.00	8.62
24.00	ELEV	83.58	83.56	83.53	83.50	83.46	83.42	83.38	83.34	83.30	83.26
25.00	DISCHG	8.26	7.91	7.57	7.25	6.94	6.64	6.36	6.09	5.54	4.89
25.00	ELEV	83.23	83.19	83.16	83.12	83.09	83.06	83.04	83.01	82.96	82.91
26.00	DISCHG	4.32	3.81	3.37	2.98	2.63	2.32	2.05	1.81	1.60	1.41
26.00	ELEV	82.86	82.82	82.78	82.75	82.72	82.69	82.67	82.65	82.63	82.62
27.00	DISCHG	1.25	1.10	.97	.86	.76	.67	.59	.52	.46	.41
27.00	ELEV	82.60	82.59	82.58	82.57	82.56	82.56	82.55	82.54	82.54	82.53
28.00	DISCHG	.36	.32	.28	.25	.22	.19	.17	.15	.13	.12
28.00	ELEV	82.53	82.53	82.52	82.52	82.52	82.52	82.51	82.51	82.51	82.51
29.00	DISCHG	.10	.09	.08	.07	.06	.06	.05	.04	.04	.03
29.00	ELEV	82.51	82.51	82.51	82.51	82.51	82.50	82.50	82.50	82.50	82.50

RUNOFF VOLUME ABOVE BASEFLOW = 1.82 WATERSHED INCHES, 891.03 CFS-HRS, 73.63 ACRE-FEET; BASEFLOW = .00 CFS

WARNING - NO HYDROGRAPH IN INPUT LOCATION 4 OR 3 IN ADDHYD OPERATION*
STRUCTURE 13

OPERATION ADDHYD STRUCTURE 13

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.10	342.62	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .92 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.14	10.03	61.20	139.95	210.31
6.00	DISCHG	293.95	342.55	300.79	250.24	227.89	218.46	212.48	205.75	199.95	195.84
7.00	DISCHG	192.44	188.62	183.55	178.75	174.69	171.02	166.09	161.22	156.59	152.17
8.00	DISCHG	147.95	143.23	137.25	131.56	126.63	122.12	117.88	113.86	110.02	106.36
9.00	DISCHG	102.27	98.16	94.29	90.63	87.19	83.95	80.89	78.01	75.29	72.73
10.00	DISCHG	70.32	67.87	65.20	62.65	60.33	58.16	56.14	54.27	52.91	51.62
11.00	DISCHG	50.38	49.21	48.12	47.08	46.05	45.04	44.10	43.23	42.40	41.57
12.00	DISCHG	40.76	40.00	39.31	38.65	37.98	37.32	36.71	36.16	35.64	35.10
13.00	DISCHG	34.56	34.00	33.35	32.73	32.15	31.60	31.10	30.67	30.25	29.81
14.00	DISCHG	29.37	28.93	28.45	28.00	27.59	27.21	26.85	26.50	26.17	25.85
15.00	DISCHG	25.54	25.18	24.69	24.21	23.82	23.68	23.60	23.53	23.45	23.39
16.00	DISCHG	23.32	23.25	23.19	23.12	23.06	22.99	22.93	22.86	22.80	22.74
17.00	DISCHG	22.68	22.62	22.56	22.50	22.44	22.38	22.32	22.26	22.21	22.15
18.00	DISCHG	22.09	22.04	21.98	21.93	21.87	21.82	21.77	21.72	21.66	21.61
19.00	DISCHG	21.56	21.51	21.46	21.41	21.36	21.31	21.26	21.22	21.17	21.12
20.00	DISCHG	21.07	20.86	20.33	19.87	19.59	19.39	19.26	19.18	19.10	18.98
21.00	DISCHG	18.85	18.75	18.69	18.62	18.51	18.38	18.28	18.22	18.16	18.05
22.00	DISCHG	17.93	17.84	17.78	17.72	17.62	17.50	17.41	17.27	16.90	16.50
23.00	DISCHG	16.10	15.74	15.44	15.14	14.82	14.50	14.21	13.97	13.73	13.47
24.00	DISCHG	13.20	12.77	12.02	11.29	10.74	10.26	9.82	9.40	9.00	8.62

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25.00	DISCHG	8.26	7.91	7.57	7.25	6.94	6.64	6.36	6.09	5.54	4.89
26.00	DISCHG	4.32	3.81	3.37	2.98	2.63	2.32	2.05	1.81	1.60	1.41
27.00	DISCHG	1.25	1.10	.97	.86	.76	.67	.59	.52	.46	.41
28.00	DISCHG	.36	.32	.28	.25	.22	.19	.17	.15	.13	.12
29.00	DISCHG	.10	.09	.08	.07	.06	.06	.05	.04	.04	.03

RUNOFF VOLUME ABOVE BASEFLOW = 1.62 WATERSHED INCHES, 1078.78 CFS-HRS, 89.15 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP RECORD ID

+ COMPUTATIONS COMPLETED FOR PASS 2

XECUTIVE CONTROL OPERATION COMPUT RECORD ID

+ FROM STRUCTURE 1

+ TO STRUCTURE 10

STARTING TIME = .00 RAIN DEPTH = 2.70 RAIN DURATION= 1.00 RAIN TABLE NO.= 7 ANT. MOIST. COND= 2

ALTERNATE NO.= 1 STORM NO.= 2 MAIN TIME INCREMENT = .10 HOURS

*** WARNING REACH 2 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 3 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

*** WARNING REACH 5 ATT-KIN COEFF.(C) GREATER THAN 0.667, CONSIDER REDUCING MAIN TIME INCREMENT ***

PERATION ADDHYD STRUCTURE 11

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.09	883.00	(NULL)
9.92	22.18	(NULL)
12.86	16.94	(NULL)
13.83	14.77	(NULL)
19.87	11.46	(NULL)
23.85	5.84	(NULL)

IME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.04	16.93	117.12	322.79	570.01
6.00	DISCHG	789.12	881.32	732.16	489.24	311.15	214.53	161.11	124.22	98.31	82.45
7.00	DISCHG	73.72	67.78	60.23	53.03	48.42	46.08	44.89	44.28	43.99	43.86
8.00	DISCHG	43.76	42.16	36.98	30.94	26.78	24.52	23.34	22.73	22.41	22.24
9.00	DISCHG	22.15	22.12	22.10	22.11	22.12	22.13	22.14	22.16	22.17	22.18
10.00	DISCHG	22.18	21.80	20.55	19.08	18.00	17.33	16.97	16.87	16.88	16.84
11.00	DISCHG	16.73	16.67	16.73	16.82	16.82	16.74	16.70	16.76	16.85	16.86
12.00	DISCHG	16.78	16.73	16.79	16.89	16.89	16.81	16.77	16.83	16.92	16.93
13.00	DISCHG	16.84	16.63	16.16	15.62	15.20	14.89	14.72	14.71	14.77	14.75
14.00	DISCHG	14.66	14.51	14.24	13.96	13.78	13.68	13.63	13.61	13.60	13.59
15.00	DISCHG	13.59	13.42	12.89	12.26	11.83	11.60	11.48	11.42	11.39	11.37
16.00	DISCHG	11.36	11.36	11.36	11.36	11.37	11.37	11.37	11.37	11.38	11.38
17.00	DISCHG	11.38	11.39	11.39	11.39	11.39	11.40	11.40	11.40	11.40	11.41

18.00	DISCHG	11.41	11.41	11.41	11.42	11.42	11.42	11.43	11.43	11.43	11.43
19.00	DISCHG	11.44	11.44	11.44	11.44	11.45	11.45	11.45	11.45	11.46	11.46
20.00	DISCHG	11.45	11.05	9.74	8.21	7.08	6.39	6.02	5.90	5.90	5.85
21.00	DISCHG	5.74	5.67	5.72	5.81	5.81	5.72	5.66	5.72	5.81	5.81
22.00	DISCHG	5.72	5.67	5.72	5.81	5.81	5.73	5.67	5.72	5.81	5.81
23.00	DISCHG	5.73	5.67	5.73	5.82	5.82	5.73	5.67	5.73	5.82	5.82
24.00	DISCHG	5.71	5.21	3.85	2.30	1.23	.65	.34	.18	.10	.05
25.00	DISCHG	.02	.01	.00							

RUNOFF VOLUME ABOVE BASEFLOW = 1.55 WATERSHED INCHES, 762.06 CFS-HRS, 62.98 ACRE-FEET; BASEFLOW = .00 CFS

OPERATION RESVOR STRUCTURE 12

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.61	161.78	88.04

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .76 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.00	.99	6.98	16.01	19.78
5.00	ELEV	82.50	82.50	82.50	82.50	82.50	82.50	82.58	83.10	84.00	84.76
6.00	DISCHG	43.18	86.25	122.18	144.72	156.50	160.95	161.78	161.19	159.46	156.28
6.00	ELEV	85.76	86.73	87.37	87.75	87.94	88.02	88.04	88.03	87.99	87.94
7.00	DISCHG	152.67	148.89	144.97	140.90	136.74	132.61	128.59	124.71	121.00	117.44
7.00	ELEV	87.88	87.81	87.75	87.68	87.61	87.54	87.48	87.41	87.35	87.29
8.00	DISCHG	114.04	110.76	107.48	104.09	100.62	96.47	92.31	88.34	84.57	81.00
8.00	ELEV	87.23	87.18	87.12	87.07	87.01	86.93	86.85	86.77	86.69	86.62
9.00	DISCHG	77.63	74.44	71.44	68.61	65.95	63.43	61.07	58.83	56.73	54.75
9.00	ELEV	86.55	86.49	86.43	86.37	86.32	86.27	86.22	86.18	86.13	86.10
10.00	DISCHG	52.88	51.11	49.55	48.28	47.01	45.76	44.54	43.36	42.23	41.15
10.00	ELEV	86.06	86.02	85.98	85.94	85.90	85.85	85.81	85.77	85.73	85.69
11.00	DISCHG	40.11	39.11	38.16	37.25	36.37	35.54	34.74	33.97	33.24	32.54
11.00	ELEV	85.66	85.62	85.59	85.56	85.53	85.50	85.47	85.45	85.42	85.40
12.00	DISCHG	31.87	31.22	30.61	30.02	29.46	28.92	28.40	27.91	27.44	26.99
12.00	ELEV	85.37	85.35	85.33	85.31	85.29	85.27	85.26	85.24	85.22	85.21
13.00	DISCHG	26.56	26.14	25.73	25.31	24.88	24.46	24.05	23.65	23.27	22.91
13.00	ELEV	85.19	85.18	85.16	85.15	85.13	85.12	85.11	85.09	85.08	85.07
14.00	DISCHG	22.56	22.22	21.89	21.55	21.23	20.98	20.92	20.85	20.79	20.73
14.00	ELEV	85.05	85.04	85.03	85.02	85.01	85.00	84.98	84.97	84.96	84.95
15.00	DISCHG	20.66	20.60	20.54	20.47	20.39	20.32	20.24	20.16	20.09	20.01
15.00	ELEV	84.93	84.92	84.91	84.89	84.88	84.86	84.85	84.83	84.82	84.80
16.00	DISCHG	19.93	19.86	19.79	19.71	19.64	19.57	19.49	19.42	19.35	19.28
16.00	ELEV	84.79	84.77	84.76	84.74	84.73	84.71	84.70	84.68	84.67	84.66
17.00	DISCHG	19.21	19.15	19.08	19.01	18.94	18.88	18.81	18.75	18.68	18.62
17.00	ELEV	84.64	84.63	84.62	84.60	84.59	84.58	84.56	84.55	84.54	84.52
18.00	DISCHG	18.56	18.49	18.43	18.37	18.31	18.25	18.19	18.13	18.07	18.01
18.00	ELEV	84.51	84.50	84.49	84.47	84.46	84.45	84.44	84.43	84.41	84.40
19.00	DISCHG	17.96	17.90	17.84	17.79	17.73	17.68	17.62	17.57	17.51	17.46
19.00	ELEV	84.39	84.38	84.37	84.36	84.35	84.34	84.32	84.31	84.30	84.29
20.00	DISCHG	17.41	17.35	17.29	17.22	17.14	17.05	16.95	16.85	16.76	16.66

20.00	ELEV	84.28	84.27	84.26	84.24	84.23	84.21	84.19	84.17	84.15	84.13
21.00	DISCHG	16.57	16.47	16.38	16.29	16.19	16.10	16.01	15.62	15.20	14.80
21.00	ELEV	84.11	84.09	84.08	84.06	84.04	84.02	84.00	83.96	83.92	83.88
22.00	DISCHG	14.41	14.04	13.69	13.35	13.03	12.72	12.42	12.13	11.86	11.61
22.00	ELEV	83.84	83.80	83.77	83.73	83.70	83.67	83.64	83.61	83.59	83.56
23.00	DISCHG	11.36	11.12	10.89	10.67	10.46	10.26	10.07	9.88	9.71	9.54
23.00	ELEV	83.54	83.51	83.49	83.47	83.45	83.43	83.41	83.39	83.37	83.35
24.00	DISCHG	9.38	9.21	9.01	8.76	8.46	8.14	7.82	7.50	7.18	6.88
24.00	ELEV	83.34	83.32	83.30	83.28	83.25	83.21	83.18	83.15	83.12	83.09
25.00	DISCHG	6.59	6.31	6.04	5.41	4.78	4.22	3.73	3.29	2.91	2.57
25.00	ELEV	83.06	83.03	83.00	82.95	82.90	82.85	82.81	82.77	82.74	82.71
26.00	DISCHG	2.27	2.00	1.77	1.56	1.38	1.22	1.08	.95	.84	.74
26.00	ELEV	82.69	82.67	82.65	82.63	82.62	82.60	82.59	82.58	82.57	82.56
27.00	DISCHG	.66	.58	.51	.45	.40	.35	.31	.27	.24	.21
27.00	ELEV	82.55	82.55	82.54	82.54	82.53	82.53	82.53	82.52	82.52	82.52
28.00	DISCHG	.19	.17	.15	.13	.12	.10	.09	.08	.07	.06
28.00	ELEV	82.52	82.51	82.51	82.51	82.51	82.51	82.51	82.51	82.51	82.51
29.00	DISCHG	.05	.05	.04	.04	.03	.03	.03	.02	.02	.02
29.00	ELEV	82.50	82.50	82.50	82.50	82.50	82.50	82.50	82.50	82.50	82.50

RUNOFF VOLUME ABOVE BASEFLOW = 1.55 WATERSHED INCHES, 761.68 CFS-HRS, 62.94 ACRE-FEET; BASEFLOW = .00 CFS

WARNING - NO HYDROGRAPH IN INPUT LOCATION 4 OR 3 IN ADDHYD OPERATION
STRUCTURE 13

OPERATION ADDHYD STRUCTURE 13

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
6.11	285.75	(NULL)

TIME(HRS) FIRST HYDROGRAPH POINT = .00 HOURS TIME INCREMENT = .10 HOURS DRAINAGE AREA = .92 SQ.MI.

5.00	DISCHG	.00	.00	.00	.00	.00	.03	6.51	46.86	114.79	175.22
6.00	DISCHG	242.04	285.54	253.20	214.11	198.81	191.82	186.59	180.69	175.42	170.76
7.00	DISCHG	166.61	162.09	156.46	151.05	146.30	141.94	137.83	133.92	130.20	126.65
8.00	DISCHG	123.25	119.42	114.46	109.74	105.67	101.28	97.02	93.00	89.21	85.64
9.00	DISCHG	82.27	79.09	76.09	73.27	70.61	68.10	65.73	63.50	61.40	59.42
10.00	DISCHG	57.56	55.65	53.68	52.09	50.65	49.31	48.06	46.89	45.78	44.69
11.00	DISCHG	43.62	42.62	41.69	40.80	39.92	39.05	38.25	37.51	36.80	36.09
12.00	DISCHG	35.39	34.74	34.15	33.59	33.02	32.45	31.93	31.46	31.01	30.55
13.00	DISCHG	30.09	29.61	29.05	28.52	28.02	27.55	27.12	26.75	26.39	26.01
14.00	DISCHG	25.63	25.25	24.84	24.45	24.10	23.85	23.78	23.71	23.65	23.59
15.00	DISCHG	23.52	23.40	23.17	22.96	22.82	22.72	22.64	22.56	22.48	22.40
16.00	DISCHG	22.33	22.25	22.18	22.10	22.03	21.96	21.89	21.82	21.75	21.68
17.00	DISCHG	21.61	21.54	21.48	21.41	21.34	21.28	21.21	21.15	21.08	21.02
18.00	DISCHG	20.96	20.90	20.83	20.77	20.71	20.65	20.59	20.54	20.48	20.42
19.00	DISCHG	20.36	20.31	20.25	20.20	20.14	20.09	20.03	19.98	19.93	19.87
20.00	DISCHG	19.82	19.62	19.14	18.73	18.47	18.28	18.15	18.07	17.99	17.88
21.00	DISCHG	17.76	17.66	17.59	17.52	17.41	17.29	17.20	16.83	16.43	16.02
22.00	DISCHG	15.60	15.23	14.90	14.58	14.25	13.91	13.61	13.35	13.10	12.83
23.00	DISCHG	12.55	12.30	12.10	11.91	11.68	11.46	11.26	11.10	10.95	10.77
24.00	DISCHG	10.57	10.24	9.62	9.02	8.57	8.19	7.84	7.50	7.19	6.88

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25.00	DISCHG	6.59	6.31	6.04	5.41	4.78	4.22	3.73	3.29	2.91	2.57
26.00	DISCHG	2.27	2.00	1.77	1.56	1.38	1.22	1.08	.95	.84	.74
27.00	DISCHG	.66	.58	.51	.45	.40	.35	.31	.27	.24	.21
28.00	DISCHG	.19	.17	.15	.13	.12	.10	.09	.08	.07	.06
29.00	DISCHG	.05	.05	.04	.04	.03	.03	.03	.02	.02	.02

RUNOFF VOLUME ABOVE BASEFLOW = 1.55 WATERSHED INCHES, 922.15 CFS-HRS, 76.21 ACRE-FEET; BASEFLOW = .00 CFS

EXECUTIVE CONTROL OPERATION ENDCMP RECORD ID

COMPUTATIONS COMPLETED FOR PASS 3

EXECUTIVE CONTROL OPERATION ENDJOB RECORD ID

1

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