Chapter 22
DRAINAGE, FLOOD PREVENTION, STORMWATER QUALITY AND EROSIONCONTROL

Introduction

The standards, guidelines and criteria presented herein are provided in order to facilitate the planning, design, construction and operation of both public and private drainage control, flood control, stormwater quality and erosion control facilities within the community.

The criteria are not intended as a substitute for good engineering judgment; imagination and ingenuity are encouraged. The thrust of these criteria is toward generalization in order to provide guidance for a large majority of design circumstances, but it must be understood that situations will arise in which these criteria are not appropriate.

The City Engineer or the AMAFCA Executive Engineer, as appropriate, may, in specific cases, require more stringent criteria or allow relaxation of these criteria based on his judgment and sound engineering practice.

The DRB representative from the City Engineer's office acts as the designee of the AMAFCA Executive Engineer except in review of proposals involving major arroyos or platting outside the City Limits where there is no immediately pending proposed annexation.

Summary of Documents Relating to Drainage, Flood Control, Stormwater Quality and Erosion Control

City Ordinances and Policies

a. Drainage Control Ordinance

b. Flood Hazard Control Ordinance

c. Comprehensive Zoning Code

d. Subdivision Ordinance

e. Open Space Policy

Other Jurisdictions

Some development plans will involve coordination with and approval by jurisdictions in addition to the City of Albuquerque because the site drains to or may impact property in their jurisdiction. They are listed below:

AMAFCA
Bernalillo County
MRGCD
NMDOT
USACE

Village of Los Ranchos
Rio Rancho
NPS
BNSF
PNM
New Mexico Gas Company
Other jurisdictions as applicable.

Section 1. Hydrology

INTRODUCTION There have been many methods used in Albuquerque and Bernalillo County to compute run off volumes, peak flow rates and runoff hydro graphs from drainage basins. Any methodology used should be based on measurable conditions, be as simple as possible and produce accurate reproducible results. The methods, graphs, and tables which follow will be used by the City of Albuquerque, Bernalillo County and AMAFCA staff in the review and evaluation of development plans and drainage management plans.

Two basic methods of analysis are presented herein:

a) PART A - describes a simplified procedure for smaller watersheds based on the Rational Method and initial abstraction/uniform infiltration precipitation losses. The procedure is applicable to watersheds up to 40 acres in size, but the procedure may be extended to include larger watersheds with some limitations

b) PART C - describes a unit hydro graph procedure which uses a version of the U.S.D.A. Agricultural Research Service HYMO computer program, modified to utilize initial abstraction/uniform infiltration precipitation losses. The AHYMO computer program developed by the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), and the simplified input procedures available with this program, are also described. This procedure is applicable for small and large watersheds. **PART B** describes the computation of time of concentration and time to peak which are used in PART A and PART C.

There may be conditions in which the procedures and analysis tools described in PART A, or PART C are not applicable or optimal for design. **PART D** describes some additional analysis procedures and some criteria under which alternate procedures will be evaluated.

c) PART E contains a tabulated list of definitions of symbols used in this Section of the D.P.M. and a bibliography.

PART A - PROCEDURE FOR 40 ACRE AND SMALLER BASINS

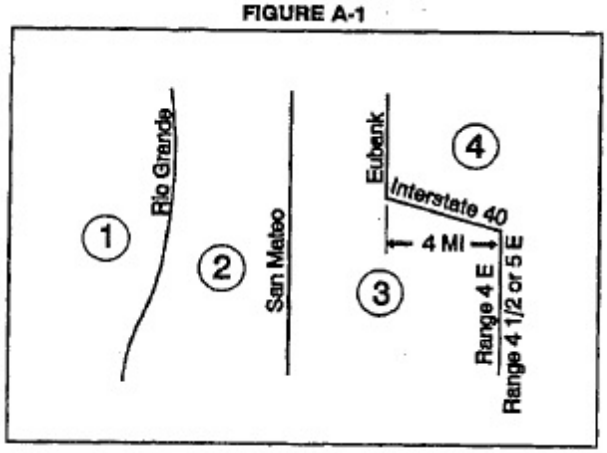
A simplified procedure for projects with sub-basins smaller than 40 acres has been developed based on initial abstraction/uniform infiltration precipitation losses and Rational Method procedures. For this procedure, Bernalillo County has been divided into four (4) Precipitation Zones.

A.1 PRECIPITATION ZONES

Bernalillo County's four precipitation zones are indicated in TABLE A-1 and on FIGURE A-1.

TABLE A-1. PRECIPITATION ZONES	
Zone	**Location**
1	West of the Rio Grande
2	Between the Rio Grande and San Mateo
3	Between San Mateo and Eubank, North of Interstate 40; and between San Mateo and the East boundary of Range 4 East, South of Interstate 40
4	East of Eubank, North of Interstate 40; and East of the East boundary of Range 4 East, South of Interstate 40

FIGURE A-1



Where a watershed extends across a zone boundary, use the zone which contains the largest portion of the watershed.

A.2 DESIGN STORM

The principal design storm is the 100-year 6-hour event defined by the NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Vol. IV - New Mexico. Assume an AMC II condition (a normally dry watershed). For design of retention or detention ponds, storms of 24-hour or longer duration many be required. The 24-hour event is defined by the NOAA Atlas 2. The 4-day and 10-day events can be obtained using the procedures in S.C.S. TSC Technical Note-Hydrology, PO-6 (Rev. 2) The 100-year 60-minute depth is computed by the following formula from Table 11 of NOAA Atlas 2:

$$P_{60} = 0.494 + 0.755 * (P_{360} * P_{360} / P_{1440}) \qquad \text{(a-1)}$$

TABLE A-2. DEPTH (INCHES) AT 100-YEAR STORM					
Zone	P_{60}	P_{360}	P_{1440}	P_{4days}	P_{10days}

1	1.87	2.20	2.66	3.12	3.67
2	2.01	2.35	2.75	3.30	3.95
3	2.14	2.60	3.10	3.95	4.90
4	2.23	2.90	3.65	4.70	5.95

The 2-year 60-minute depth is computed by the following formula from NOAA Atlas 2:

$$P_{60\text{-}2} = -0.011 + 0.942 * (P_{360\text{-}2} * P_{360\text{-}2} / P_{1440\text{-}2}) \qquad \text{(a-2)}$$

Based on fitting a logarithmic curve to the values in Table 12 of NOAA Atlas 2, the 12- minute (0.2 hour) depth was computed to be 50.24 percent of the 60-minute depth:

$$P_{12} = 0.5024 * P_{60} \qquad \text{(a-3)}$$

For certain applications (e.g., street drainage, low flow channels and sediment transport) storms of greater frequency than the 100-year storm must be considered. To estimate precipitation at return periods other than 100 years, multiply the 360-minute or 1440-minute 100-year precipitation amounts by the factors in TABLE A-3.

TABLE A-3. RETURN PERIOD FACTORS	
Return Period (years)	**Factor**
50	0.900
25	0.800
10	0.667
5	0.567
2	0.434

Example A-1
Find the 10-year, 6-hour storm depth for Zone 2. $P_{360\text{-}10} = 2.35 * 0.667 = 1.57$ inches

Example A-2
Find the 2-year, 1-hour storm depth for Zone 3. $P_{360\text{-}2}$ = 2.60 * 0.434 = 1.128 inches $P_{1440\text{-}2}$ = 3.10 * 0.434 = 1.345 inches $P_{60\text{-}2}$ = -0.011 + 0.942* ($P_{360\text{-}2}$* $P_{360\text{-}2}$ / $P_{1440\text{-}2}$) = -0.011+ 0.942*(1.128*1.128/1.345) = 0.880 inches

A.3 LAND TREATMENTS

All land areas are described by one of four basic land treatments or by a combination of the four land treatments.

Land treatments are given in TABLE A-4.

TABLE A-4. LAND TREATMENTS

Treatment	Land Condition
A	Soil uncompacted by human activity with 0 to 10 percent slopes. Native grasses, weeds and shrubs in typical densities with minimal disturbance to grading, ground cover and infiltration capacity.
B	Irrigated lawns, parks and golf courses with 0 to 10 percent slopes. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes greater than 10 percent and less than 20 percent.
C	Soil compacted by human activity. Minimal vegetation. Unpaved parking, roads, trails. Most vacant lots. Gravel or rock on plastic (desert landscaping). Irrigated lawns and parks with slopes greater than 10 percent. Native grasses, weeds and shrubs, and soil uncompacted by human activity with slopes at 20 percent or greater. Native grass, weed and shrub areas with clay or clay loam soils and other soils of very low permeability as classified by SCS Hydrologic Soil Group D.
D	Impervious areas, pavement and roofs.
Most watersheds contain a mix of land treatments. To determine proportional treatments, measure respective subareas	

A.4 ABSTRACTIONS

Initial abstraction is the precipitation depth which must be exceeded before direct runoff begins. Initial abstraction may be intercepted by vegetation, retained in surface depressions, or absorbed on the watershed surface. Initial abstractions are shown in TABLE A-6.

TABLE A-6. INITIAL ABSTRACTION (IA)	
Treatment	**Initial Abstraction (inches)**
A	0.65
B	0.50
C	0.35
D	0.10

Infiltration is the only significant abstraction after the initial abstraction. After initial abstraction is satisfied, treat infiltration as a constant loss rate as specified in TABLE A-7.

TABLE A-7. INFILTRATION (INF)	
Treatment	**Loss Rate (inches/hour)**
A	1.67
B	1.25
C	0.83
D	0.04*
* Treatment D infiltration rate is applicable from 0 to 3 hours; use uniform reduction from 3 to 6 hours, with no infiltration after 6 hours.	

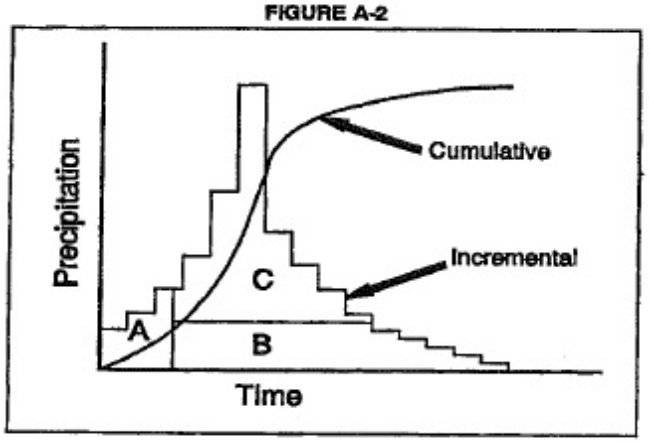
Runoff from a previous event can saturate a channel bed, rendering it minimally pervious for several days. Do not anticipate additional bed losses for design purposes.

A.5 EXCESS PRECIPITATION & VOLUMETRIC RUNOFF

Excess precipitation, E, is the depth of precipitation remaining after abstractions are removed. Excess precipitation does not depend on watershed area.

Excess precipitation is determined by subtracting the initial abstraction and infiltration from the design storm hydro graph. FIGURE A-2 illustrates the development of excess precipitation. The curved line plots cumulative precipitation. Precipitation intensities (in/hr) are shown as a histogram. Initial abstraction is area A. The horizontal line is at a height corresponding to the infiltration rate. Infiltration loss is area B. The remaining histogram, area C, is excess precipitation.

FIGURE A-2



Excess precipitation, E, by zone and treatment is summarized in TABLE A-8.

*(**NOTE:** In this table and several tables which follow, corresponding values for 2- and 10-year storms are shown in brackets below each 100-year value)*

TABLE A-8. EXCESS PRECIPITATION, E (INCHES) - 6 HOUR STORM				
Zone	100-YR.. **Treatment** [2-YR., 10-YR.]			
	A	B	C	D

1	0.44 [0.00, 0.08]	0.67 [0.01, 0.22]	0.99 [0.12, 0.44]	1.97 [0.72, 1.24]
2	0.53 [0.00, 0.13]	0.78 [0.02, 0.28]	1.13 [0.15, 0.52]	2.12 [0.79, 1.34]
3	0.66 [0.00, 0.19]	0.92 [0.06, 0.36]	1.29 [0.20, 0.62]	2.36 [0.89, 1.50]
4	0.80 [0.02, 0.28]	1.08 [0.11, 0.46]	1.46 [0.27, 0.73]	2.64 [1.01, 1.69]

To determine the volume of runoff,

1) Determine the area in each treatment, A_A, A_B, A_C, A_D

2) Compute the weighted excess precipitation, E

$$\text{Weighted } E = \frac{E_A A_A + E_B A_B + E_C A_C + E_D A_D}{A_A + A_B + A_C + A_D} \qquad \text{(a-5)}$$

3) Multiply the weighted E by the watershed area.

$$V_{360} \text{ (as volume)} = \text{weighted } E * (A_A + A_B + A_C + A_D) \qquad \text{(a-6)}$$

EXAMPLE A-3
Find the 100-year V_{360} for 30 acres in zone 1. Eight acres are treatment A, 10 acres are treatment B, 5 acres are treatment C, and 7 acres are treatment D. Weighted E = ((8 * 0.44) + (10 * 0.67) + (5 * 0.99) + (7 * 1.97)) / 30 = 0.965 inches Volume = (0.965 * 30) / 12 = 2.41 acre-ft. = V_{360}

For ponds which hold water for longer than 6 hours, longer duration storms are required to establish runoff volumes. Since the additional precipitation is assumed to occur over a long period, the additional volume is based on the runoff from the impervious areas only.

For 24-hour storms:

$$V_{1440} = V_{360} + A_D * (P_{1440} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-7)}$$

For 4-day storms:

$$V_{4DAYS} = V_{360} + A_D * (P_{4DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-8)}$$

For 10-day storms:

$$V_{10DAYS} = V_{360} + A_D * (P_{10DAYS} - P_{360}) / 12 \text{ in/ft} \qquad \text{(a-9)}$$

EXAMPLE A-4
Find the 100-year 24-hour and 4-day runoff volume, V_{1440} and V_{4days}, for the area in Example A-3. V_{360} = 2.41 acre-feet V_{1440} = 2.41 + 7 ac * (2.66 - 2.20) / 12 = 2.68 acre-feet V_{4DAYS} = 2.41 + 7 ac * (3.12 - 2.20) / 12 = 2.95 acre-feet

A.6 PEAK DISCHARGE RATE FOR SMALL WATERSHEDS

Small watersheds are less than or equal to 40 acres.

Peak Discharge

Using a 0.2-hour (12-minute) time of concentration peak discharge, Q_P, per acre is the volume of excess precipitation in the heaviest 12-minute portion of the storm, divided by the time increment 12 minutes, and multiplied by an attenuation factor. The attenuation factor (0.59 for treatment A, 0.67 for treatment B, 0.75 for treatment C and 0.93 for treatment D) describes the effect of routing. Determine the peak discharge using the values in TABLE A-9, which have been adjusted to consider the effects of initial abstraction.

TABLE A-9. PEAK DISCHARGE (CFS/ACRE)

Zone	100-YR. **Treatment** [2-YR., 10-YR.]			
	A	**B**	**C**	**D**
1	1.29 [0.00, 0.24]	2.03 [0.03, 0.76]	2.87 [0.47, 1.49]	4.37 [1.69, 2.89]
2	1.56 [0.00, 0.38]	2.28 [0.08, 0.95]	3.14 [0.60, 1.71]	4.70 [1.86, 3.14]
3	1.87 [0.00, 0.58]	2.60 [0.21,1.19]	3.45 [0.78, 2.00]	5.02 [2.04, 3.39]
4	2.20 [0.05, 0.87]	2.92 [0.38, 1.45]	3.73 [1.00, 2.26]	5.25 [2.17, 3.57]

To determine the peak rate of discharge,

1) Determine the area in each treatment, A_A, A_B, A_C and A_D.

2) Multiply the peak rate for each treatment by the respective areas and sum to compute the total Q_P.

$$\text{Total } Q_P = Q_{PA}A_A + Q_{PB}A_B + Q_{PC}A_C + Q_{PD}A_D \qquad \text{(a-10)}$$

Example A-5
Find 100-year Q_P for 14 acres in zone 1. The four land treatments are: 3 acres in treatment A, 5 acres in treatment B, 2 acres in treatment C and 4 acres in treatment D. Total Q_P = (1.29 * 3) + (2.03 * 5) + (2.87 * 2) + (4.37 * 4) = 37.24 cfs

3) Approximately the same results can be achieved by a Rational Method solution. The 0.2-hour (12-minute) peak intensities, I, are given in TABLE A-10 and Rational Method coefficients, C, are given in TABLE A-11.

$$\text{Total } Q_P = (C_A * I * A_A) + (C_B * I * A_B) + (C_C * I * A_C) + (C_D * I * A_D) \quad \text{(a-11)}$$

TABLE A-10. PEAK INTENSITY (IN/HR at t_c = 0.2 hour)

Zone	**Intensity** 100-YR. [2-YR., 10-YR.]
1	4.70 [1.84, 3.14]
2	5.05 [2.04, 3.41]
3	5.38 [2.21, 3.65]
4	5.61 [2.34, 3.83]

TABLE A-11. RATIONAL METHOD COEFFICIENT, C

Zone	**Treatment** 100-YR. [2-YR., 10-YR.]			
	A	B	C	D
1	0.27 [0.00, 0.08]	0.43 [0.02, 0.24]	0.61 [0.26, 0.47]	0.93 [0.92, 0.92]
2	0.31 [0.00, 0.11]	0.45 [0.04, 0.28]	0.62 [0.29, 0.50]	0.93 [0.91, 0.92]
3	0.35 [0.00, 0.16]	0.48 [0.10, 0.33]	0.64 [0.35, 0.55]	0.93 [0.92, 0.93]

4	0.39 [0.02, 0.23]	0.52 [0.16, 0.38]	0.66 [0.43, 0.59]	0.94 [0.93, 0.93]

(Note the quote from the ASCE Manual and Report on Engineering Practice No. 37 (1969): The commonly reported Rational C values "are applicable for storms to 5- to 10-yr. frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff." Thus higher C's realized under heavy precipitation might be expected.)

Example A-6

Recompute Example A-5 using the Rational Method**.**

Q = C I A

= (0.27 * 4.70 *3) + (0.43 * 4.70 * 5) + (0.61 * 4.70 * 2) + (0.93 * 4.70 * 4)

= 37.13 cfs

A.7 USE OF RATIONAL METHOD FOR WATERSHEDS LARGER THAN 40 ACRES

Peak rates of discharge may be computed for watersheds larger than 40 acres by using the Rational Method Coefficients (C's) from TABLE A-11 and modifying the Intensity (in/hr) for a larger time of concentration (t_c). This method may be used to establish peak flow rates for off-site flow areas when sizing channels, pipes and road crossings. On-site areas should be divided into 40 acre or smaller sub- basins and should not use this procedure. For watersheds larger than 40 acres, the rational method should not be used to establish allowable historic flow rates since it will tend to give somewhat larger values than those computed by unit hydro graph procedures.

The procedures outlined in PART B should be used to compute the time of concentration (t_C).

Then compute the Intensity (in/hour), using the time of concentration, t_C (hours), and the 60-minute precipitation depth, P_{60} (inches), with the following formula:

$$I = 0.726 * (\log_{10} (24.6 * t_c)) * (1/t_c) * P_{60} \qquad \text{(a-12)}$$

Do not use this formula for t_c larger than 2.0 hours.

EXAMPLE A-7

Find Q_P for a 100-year storm at a 120 acre watershed in zone 3, with a 2600 feet shallow concentrated flow upper subreach at 0.015 ft/ft slope and 1200 feet natural channel lower subreach at 0.02 ft/ft slope. The watershed is 50 percent treatment A, 20 percent treatment B, 10 percent treatment C and 20 percent treatment D.

Compute the time of concentration using Table B-1 from PART B

as follows:

With a reach length longer than 2000 feet, use K = 3 for the portion below the first 2000 feet.

Since total reach length (2600 + 1200) is less than 4000 feet use equations b-1 and b-2 from PART B

t_c = ((2000 / (10 * 2 * ($0.015^{0.5}$))) + (600 / (10 * 3 ($0.015^{0.5}$))) +

(1200 / (10 * 3 * ($0.02^{0.5}$)))) / 3600 = 0.3507 hours

Compute the Intensity, I, using equation a-12 as follows:

I=0.726*log10(24.6*0.3507)*(1/0.3507)*2.14

=4.15 inshces/hour

Using equation a-11 and the percentage of treatment types:

A_A= 120 * 0.50 = 60 acres A_B = 120 * 0.20 = 24 acres
A_c = 120 * 0.10 = 12 inches A_D = 120 * 0.20 = 24 acres

Q_P = (0.35 * 4.15 * 60) + (0.48 * 4.15 * 24) + (0.64 * 4.15 * 12) + (0.93 * 4.15 * 24) = 259.46 cfs

A.8 HYDROGRAPH FOR SMALL WATERSHED

Base time, t_B, for a small watershed hydrograph is,

$$t_B = (2.107 * E * A_T / Q_P) - (0.25 * A_D / A_T) \quad \text{(a-13)}$$

Where t_B is in hours, E is the excess precipitation in inches (from TABLE A-8), Q_P is the peak flow in cfs, A_D is the area in treatment D, and A_T is the total area in acres. Using the time of concentration, t_c (hours), the time to peak in hours is:

$$t_P = (0.7 * t_C) + ((1.6 - (A_D / A_T)) / 12) \quad \text{(a-14)}$$

Continue the peak for 0.25 * A_D / A_T hours. When A_D is zero, the hydrograph will be triangular. When A_D is not zero, the hydrograph will be trapezoidal. FIGURE A-3 shows the hydrograph in graphic form.

Example A-8
Determine the hydrograph for Example A-5. A_T = 14.0 acres A_D = 4.0 acres t_C = 0.2 hour Q_P = 37.24 cfs E = ((3 * .44) + (5 * .67) + (2 * .99) + (4 * 1.97)) / (3 + 5 + 2 + 4) = 1.038 inches t_P = (0.7 * 0.2) + (1.6 - (4 / 14)) / 12) = 0.2495 hours t_B = (2.017 * 1.038 * 14 / 37.24) - (0.25 * 4 / 14) = 0.7157 hours Duration of peak = 0.25 * 4 / 14 = 0.0714 hours

PART B - TIME OF CONCENTRATION, LAG TIME, AND TIME TO PEAK

There is a delay, after a brief heavy rain over a watershed, before the runoff reaches its maximum. The length of time it takes for runoff from a watershed to reach an analysis point effects the peak runoff rate, with shorter times producing higher peak flow for a constant runoff volume. The velocity at which water can flow through a watershed and the length of flow path are used to determine the time factors. Time of concentration, lag time, and time to peak are three related watershed parameters that are used to determine peak rates of runoff.

B.1 DEFINITIONS

The three time parameters used are defined as follows:

time of concentration (t_c) = time it takes for runoff to travel from the hydraulically most distant part of the watershed basin to the basin outlet or point of analysis

Lag time (L_G) = time from the center of unit rainfall excess to the time that 50 percent of the volume of unit runoff from the drainage basin has passed the concentration point or point of analysis.

time to peak (t_p) = time from the beginning of unit rainfall excess to the time of the peak flow of the unit runoff hydrograph.

The three time parameters can be computed using the procedures identified in this section. The peak discharge rates and intensity factors identified in TABLES A-9 and A-10 (PART A) were

computed using a time of concentration (t_c) of 0.2 hour. The procedures in Part C require the computation of time to peak (t_p) as specified herein.

B.2 COMPUTATION OF TIME OF CONCENTRATION

Three different equations are used to compute time of concentration (tc) for larger watersheds. For subbasin reach lengths shorter than 4000 feet the SCS Upland Method is used; for subbasin reach lengths longer than 12000 feet the USDI Bureau of Reclamation lag time equation is used. A transition equation is used for subbasin reach lengths between 4000 and 12000 feet.

Consideration should be given to splitting large watersheds into smaller subbasins with reach lengths less than 4000 feet. Smaller subbasins will allow more accurate modeling of channels and basin topography, and should provide for greater modeling accuracy.

1) For subbasin reach lengths less than 4000 feet:

Compute time of concentration, t_c (hours), for the entire (pervious and impervious) watershed by the SCS Upland Method, the sum of the travel times in the subreaches comprising the longest flow path to the watershed outlet.

$$t_c = (L_1 / V_1 + L_2 / V_2 + ... + L_x / V_x) / 3600 \text{ sec/hour} \qquad \text{(b-1)}$$

and, $(L_1 + L_2 + ... + L_x) < 4000$ feet

where L_X is the subreach length (feet) and v is the velocity (feet/sec) in that subreach, as determined by the following equation:

$$v = K * \sqrt{(s * 100)} = 10 * K * \sqrt{(s)} \qquad \text{(b-2)}$$

where s is the slope in foot per foot, and K depends upon the conveyance condition, as shown in TABLE B-1. If t_C is computed to be less than 0.2 hours, use t_C = 0.2 hours.

TABLE B-1. CONVEYANCE FACTORS

K	Conveyance Condition
0.7	Turf, landscaped areas and undisturbed natural areas (sheet flow* only).
1	Bare or disturbed soil areas and paved areas (sheet flow* only).
2	Shallow concentrated flow (paved or unpaved).
3	Street flow, storm sewers and natural channels, and that portion of subbasins (without constructed channels) below the upper 2000 feet for subbasins longer than 2000 feet.
4	Constructed channels (for example: riprap, soil cement or concrete lined channels).
* Sheet flow is flow over plane surfaces, with flow depths up to 0.1	

feet.

Sheet flow applies only to the upper 400 feet (maximum) of a subbasin.

For composite reaches where the basin slope is <u>not</u> uniform, the composite basin conveyance condition, K, can be computed using the following equation:

$$K = (L / \sqrt{s}) / (L_1 / (K_1 * \sqrt{s_1}) + L_2 / (K_2 * \sqrt{s_2}) + ... + L_x / (K_x * \sqrt{s_x})) \quad \text{(b-4)}$$

where: $L = L_1 + L_2 + ... + L_x$

and, $s = (L_1 * s_1 + L_2 * s_2 + ... + L_x * s_x) / L$ (b-5)

2.) For subbasin reach lengths between 4000 and 12000 feet:

Compute the time of concentration, t_C (hours), for the entire watershed using the following equation:

$$t_C = ((12000 - L) / (72000 * K * s^{0.5})) + ((L - 4000) * K_N * (L_{CA} / L)^{0.33} / (552.2 * s^{0.165})) \quad \text{(b-6)}$$

where:

K = Conveyance factor from TABLE B-1. For composite reaches, K is computed using equation b-3 or b-4.

L = distance of longest watercourse, in feet.

L_{CA} = distance along L from point of concentration to a point opposite centroid of drainage basin, in feet.

s = overall slope of L, in foot per foot. For composite reaches s is computed using equation b-5.

K_N = a basin factor based on an estimate of the weighted, by stream length, average Manning's n value for the principal watercourses in the drainage basin. For the Albuquerque area, values of K_N may be estimated from TABLE B-2.

TABLE B-2. LAG EQUATION BASIN FACTORS

K_N	Basin Condition
0.042	Mountain Brush and Juniper
0.033	Desert Terrain (Desert Brush)
0.025	Low Density Urban (Minimum improvements to watershed channels)

0.021	Medium Density Urban (Flow in streets, storm sewers and improved channels)
0.016	High Density Urban (Concrete and rip-rap lined channels)

3.) For subbasin reach length greater than 12000 feet:

Compute the time of concentration, t_C (hours), for the entire watershed by first computing the lag time using the USDI Bureau of Reclamation lag time equation as follows:

$$L_G = 26 * K_N * ((L * L_{CA} / (5280^2 * (s * 5280)^{0.5}))^{0.33})$$

where: L_G = Lag time, in hours (b-7)

The time of concentration, t_c (hours), is computed from the lag time, L_G (hours), by the following equation:

$$t_c = (4 / 3) * L_G \quad \text{(b-8)}$$

The time of concentration, t_C (hours), is computed from the lag time, LG (hours), by the following equation:

B.3 COMPUTATION OF TIME TO PEAK

For the procedures outlined in PART C , the time to peak (t_P) is assumed to be a constant ratio of the time of concentration (t_C). The following equation is used to compute time to peak:

$$t_P = (2 / 3) * t_C \quad \text{(b-9)}$$

Example B-1
Find the time of concentration (t_c) for a watershed with a 4000 feet desert terrain upper reach (shallow concentrated flow) at 0.015 ft/ft slope and a 3000 feet low density urban lower reach (streets and natural channels) at 0.02 ft/ft slope. The distance to the centroid point is 60% of the total reach length. $L = 4000 + 3000 = 7000$ ft $L_{CA} / L = 0.60$ $s = (0.015 * 4000 + 0.02 * 3000) / 7000 = 0.01714$ foot per foot $K_N = (0.033 * 4000 + 0.025 * 3000) / 7000 = 0.030$ From equation b-4: $K = (7000 / (0.01714^{0.5})) / ((2000 / (2 * (0.015^{0.5}))) + (2000 / (3 * (0.015^{0.5}))) + (3000 / (3 * (0.02^{0.5})))) = 2.59$ $t_c = ((12000 - 7000) / (72000 * 2.59 * 0.01714^{0.5})) + ((7000 - 4000) * 0.030 * 0.60^{0.33} / (552.2 * 0.01714^{0.165})) = 0.2048 + 0.2694 = 0.4742$ hours

Example B-2
Find the time of concentration (t_c), lag time (L_G) and time to peak (t_p) for a watershed with a 8000 feet desert terrain upper reach at 0.015 ft/ft slope and a 6000 feet low density urban lower reach at 0.02 ft/ft slope. The distance to the centroid point is 60% of the total reach length. $L = 8000 + 6000 = 14000$ feet $L_{CA} = 0.60 * 14000 = 8400$ feet Use equation b-5: $s = (0.015 * 8000 + 0.02 * 6000) / 14000 = 0.01714$ ft/ft $K_N = (0.033 * 8000 + 0.025 * 6000) / 14000 = 0.030$ Use equation b-7: $L_G = 26 * 0.030 * ((14000 * 8400 / (5280^2 * (0.01714 * 5280)^{0.5}))^{0.33})$ $= 0.596$ hours $t_c = (4/3) * 0.596 = 0.795$ hours $t_p = (2/3) * 0.795 = 0.530$ hours

B.4 TIME OF CONCENTRATION FOR STEEP SLOPES AND NATURAL CHANNELS

The procedures used to compute time of concentration (tc) as described in Section B.2 may compute values that are too small to be sustained for natural channel conditions. In natural channels, flows become unstable when a Froude Number of 1.0 is approached. The procedures identified in Section B-2 may compute flow velocities for steep slopes that indicate supercritical flow conditions, even though such supercritical flows cannot be sustained for natural channels.

For steep slopes, natural channels will likely experience chute and pool conditions with a hydraulic jump occurring at the downstream end of chute areas; or will experience a series of cascading flows with very steep drops interspersed with flatter channel sections.

For the purposes of this section, steep slopes are defined as those greater than 0.04 foot per foot. The procedures outlined in this section should <u>no</u>t be used for the following conditions:

a. Slopes flatter than 0.04 foot per foot.

b. Channels with irrigated grass, riprap, soil cement, gabion, or concrete lining which cannot be clearly identified as natural or naturalistic.

c. The hydraulic design of channels or channel elements. The purpose this section is to define procedures for hydrologic analysis only. The design of facilities adjacent to or within channels with chute and pool conditions cannot be analyzed with the simplified procedures identified herein. It may be necessary to design such facilities for the supercritical flows of chutes (for sediment transport, local scour, stable material size) and for the hydraulic jump of pool conditions (for maximum water surface elevation and flood protection).

The slope of steep natural watercourses should be adjusted to account for the effective slope that can be sustained. The slope adjustment procedures identified in the Denver - Urban

Drainage and Flood Control District (UDFCD) Urban Storm Drainage Criteria Manual (Figure 4-1, Runoff chapter, 1990) are applicable for the slope adjustment identified herein. In addition, channel conveyance factors (K) should be checked to make sure that appropriate equivalent Froude Numbers are maintained. The UDFCD Figure 4-1 can be approximated by the following equation:

$$s' = 0.052467 + (0.063627 * s) - 0.18197 * e^{(-62.375 * s)} \qquad \text{(b-10)}$$

where: s = measured slope (foot per foot)

and, s' = adjusted slope (foot per foot)

The conveyance factors (K) for the upland method should be checked to make sure that appropriate Froude Numbers are maintained. To accomplish this, it is necessary to estimate the peak flow rate from the watershed. Using estimated conveyance factors (K) from Table B-1 and the procedures outlined in Part A, an estimated peak flow rate for the basin (Q_P) can be computed. The following formulas are then used to compute conveyance factor adjustment:

$$K' = 0.302 * s'^{(-0.5)} * Q_p^{(0.18)} \qquad \text{(b-11)}$$

$$K'' = 0.207 * s'^{(-0.5)} * Q_p^{(0.18)} \qquad \text{(b-12)}$$

An adjusted conveyance factor (K) is then obtained based on the following:

if $K > K'$ then $K = K'$

if $K' \geq K \geq K''$ then $K = K$ (no adjustment)

if $K < K''$ then $K = K''$

Recompute Q_P based on the revised conveyance factor (K) using the procedures in Part A or Part C as appropriate. If the recomputed Q_P is within 10 percent of the Q_P used to compute K' and K", the estimate is sufficiently accurate. If the recomputed Q_P is more than 10 percent from the Q_P used to compute K' and K", repeat the process using the revised Q_P.

The Lag Equation Basin Factors, K_N, from, TABLE B-2 remain applicable when using equations b-6 and b-7 with the adjusted slope computed by equation b-10.

Example B-3

Compute the time of concentration (t_c) for a natural basin having a length of 4,000 feet and a uniform slope of 0.12 foot per foot. The basin is estimated to have a peak flow of 600 cfs using the procedures in PART A .

s = 0.12 foot per foot Q_P = 600 cfs

Compute the adjusted slope using equation b-10.

$$s' = 0.052467 + 0.063627 * 0.12 - 0.18197 * (e^{(-62.375 * 0.12)})$$

$$= 0.052764 + 0.007635 - 0.000102 = 0.0603 \text{ ft/ft}$$

Compute conveyance factors from Table B-1 and equation b-3

$$K = 4000 / (300 / 0.7 + 1700 / 2.0 + 2000 / 3.0) = 2.056$$

From equations b-11 and b-12

$K' = 0.302 * (.0603)^{(-0.5)} * (600)^{0.18} = 3.89$

$K'' = 0.207 * (.0603)^{(-0.5)} * (600)^{0.18} = 2.66$

Since K < K" then use K = 2.66

From equation b-1 and b-2

$V = 10 * 2.66 * (0.0603^{0.5}) = 6.53$ ft/sec

$t_C = (4000 / 6.53) / 3600 = 0.170$ hour (Use 0.200 hour minimum.)

The Q_P should then be recomputed using the revised t_c and the procedures in PART A or PART C.

B.5 CHANNEL ROUTING FOR STEEP SLOPES AND NATURAL CHANNELS

The procedures outlined to compute time of concentration for steep natural channels in Section B.4 are also applicable for hydrologic routing of hydrographs through channel segments. The restrictions which limit the procedure only to natural channels with slopes steeper than 0.04 foot per foot are also applicable here. The procedures are not applicable to the hydraulic design of channel structures.

Equation b-10 can be used to obtain an adjusted slope for the channel segment. The Manning's roughness (n) for the channel should be checked to make sure that appropriate Froude Numbers are maintained. It is necessary to estimate the peak flow rate (Q_P) for the watershed channel segment to perform this check. An analysis without a Manning's roughness adjustment may be used for the initial estimate. The following formula is then used to compute the Manning's roughness adjustment:

$$n' = 0.122 * s'^{(0.5)} * Q_P^{(0.06)} \qquad \text{(b- 13)}$$

An adjusted Manning's roughness (n) is then obtained based on the following:

if n < n' then n = n'

if n ≥ n' then n = n (no adjustment)

Recompute the Q_p based on the revised Manning's roughness (n). If the recomputed Q_p is within 30 percent of the Q_p used to compute n', the estimate is sufficiently accurate. If the recomputed Q_p is more than 30 percent from the Qp used to compute n", repeat the process using the revised Q_p.

Example B-4

A channel segment immediately downstream of the basin in Example B-3 has a slope of 0.08 foot per foot. The channel has an apparent Manning's roughness of 0.035. Compute the equivalent channel slope and Manning's roughness for use in hydrologic routing.

s = 0.08 foot per foot $Q_p = 600$ cfs

$s' = 0.052467 + 0.063627 * 0.08 - 0.18197 * e^{(-62.375*0.08)}$

$= 0.052467 + 0.005090 - 0.001238 = 0.0563$ ft/ft

Use equivalent slope = 0.0563 ft/ft in routing computation.
From equation b-13
$n' = 0.122 * (.0563)^{0.5} * (600)^{0.06} = 0.0425$
Since $n < n'$, then use $n = 0.0425$

PART C - PROCEDURE FOR SMALL AND LARGE WATERSHEDS

A unit hydrograph procedure is used for major drainage area analysis and for sub-basins larger than 40 acres. To compute peak discharge the HYMO computer program (Problem-Oriented Computer Language for Hydrologic Modeling, by J.R. Williams and R.W. Hann, Jr., ARS-S-9, 1973) with subsequent modifications by the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) is used. **The PART C procedure may be utilized for small watersheds (40 acres or less) in place of the procedures specified in PART A**

C.1 THE HYMO COMPUTER PROGRAM

HYMO was developed for rural watersheds exceeding 320 acres. Some modification to HYMO is required for it to be used for small watersheds modeled as components of large watersheds, or for urban watersheds. The following items have been implemented as standard modifications to the HYMO procedures:

1. Employ precipitation and land treatment-specific HYMO recession coefficients, k.

2. Provide a consistent range of peak flows, Q_p for large and small watersheds by adjusting k with sub-basin area.

3. Employ an overland-flow or lag time based time of concentration.

4. Compute runoff by initial abstraction - uniform infiltration methodology. Do not use the standard SCS curve number methodology for computation of runoff.

5. Compute hydrographs by the split watershed method. Compute a hydrograph for the impervious portion of the sub-basin and a separate hydrograph for the pervious portion; then add the pervious and impervious hydrographs to compute the sub-basin hydrograph.

6. Increase hydrograph storage to allow up to 600 hydrograph point values, and increase rainfall storage to allow up to 500 incremental values.

7. Do not automatically adjust Manning's roughness coefficient for depth of flow in channel routing computations.

Extensive enhancements and revisions have been added to the original A.R.S. version of the HYMO program to implement the standard modifications and add new program features. However, the revised HYMO program will execute the original test data from the A.R.S. version of HYMO.

Modifications to the original program FORTRAN source code are required to implement the standard modifications. A PC version (MS-DOS) of the modified program, including FORTRAN source code, is available at the cost of reproduction from the City of Albuquerque, AMAFCA and Bernalillo County. Mini-computer and mainframe computer revisions of the program are available from local engineering consultants.

An enhanced version of the program, titled AHYMO, is also available at nominal cost from AMAFCA. The AHYMO program is distributed subject to AMAFCA's policy for the "Sale and Distribution of AHYMO Software" (Resolution No. 1991-6). The enhanced version (AMAFCA's AHYMO) can store twenty (20) hydrographs; the original and revised version can store six (6) hydrographs. The enhanced version adds the RAINFALL and COMPUTE NM HYD functions, and provides a computation summary table. Both the revised version and the enhanced version can store 600 hydrograph point values. The enhanced version of the program is not required to use the procedures in PART B, although such use may simplify data entry and computation.

C.2 ZONES

The unit hydrograph procedure should not utilize precipitation zones from TABLE A-1 or FIGURE A-1 of PART A. The precipitation amounts are obtained from the NOAA Atlas 2, or figures in PART C and the equations in PART C. Program parameters are obtained based on basin characteristics and precipitation quantities.

C.3 DESIGN STORM

1) The principal design storm for peak flow determination is the 100-year 6-hour event defined by the NOAA Atlas 2, Precipitation - Frequency Atlas of the Western United States, Vol. IV - New Mexico. Storms of other frequencies or durations are required for design or analysis of volume sensitive facilities, when examining sediment transport, and for complex routing conditions. The following statement from the Federal Emergency Management Agency (FEMA) should be used to provide guidance when selecting a storm duration:

"FEMA's position regarding the duration of rainfall is that the storm must extend for a period long enough to include all rainfall excess when the volume of the runoff hydrograph is an important consideration. This includes conditions when detention storage is involved, when sediment processes are a significant factor, and when combining and routing subbasin hydrographs to obtain watershed runoff. When the peak flow is the primary concern, and it is established that the use of a longer duration storm would not increase the peak flow, shorter duration storms are acceptable." 2) The 100-year six hour, P_{360}, and the 24-hour, P_{1440}, precipitations are obtained from Figures 24 and 30 of NOAA Atlas 2. The one hour precipitation, P_{60}, is computed using the procedures in TABLE 11 of NOAA Atlas 2 and equations a-1 or a-2 herein. Values for P_{60}, P_{360} and P_{1440} in Bernalillo County should be obtained from FIGURES C-1, C-2, and C-3 respectively. Outside of Bernalillo County, NOAA Atlas 2 should be utilized.

3) Compute the depth of cumulative precipitation (also called the mass rainfall or rainfall distribution), P_T, at any time, t < or = 1440 minutes, as follows:

$$P_T = 2.334 * (P_{360} - P_{60}) * (1.5^A - (1.5 - t / 60)^A), \qquad 0\ t < 60 \qquad \text{(c-1)}$$

$P_T = P_{T=60} + P_{60} * 0.4754 * (0.5^{0.09} - (1.5 - t / 60)^{0.09})$, 60 t < 67 (c-2)

$P_T = P_{T=60} + P_{60} * 0.0001818182 * (t - 60) + 0.000018338 * (t-60)^{3.2}$, 67 t < 85.3 (c-3)

$P_T = P_{T=60} + P_{60} * 0.07 * (t - 60) - 1.1886 - 0.0404768 * (t-85)^{1.0985865}$, 85.3 t < 120 (c-4)

$P_T - P_{360} + (P_{T=60} + P_{60} - P_{360}) * (4.4^{(3*A)} - (t / 60 - 1.6)^{(3*A)})$
$/ (4.4^{(3*A)} - 0.4^{(3*A)})$, 120 t t 360 (c-5)

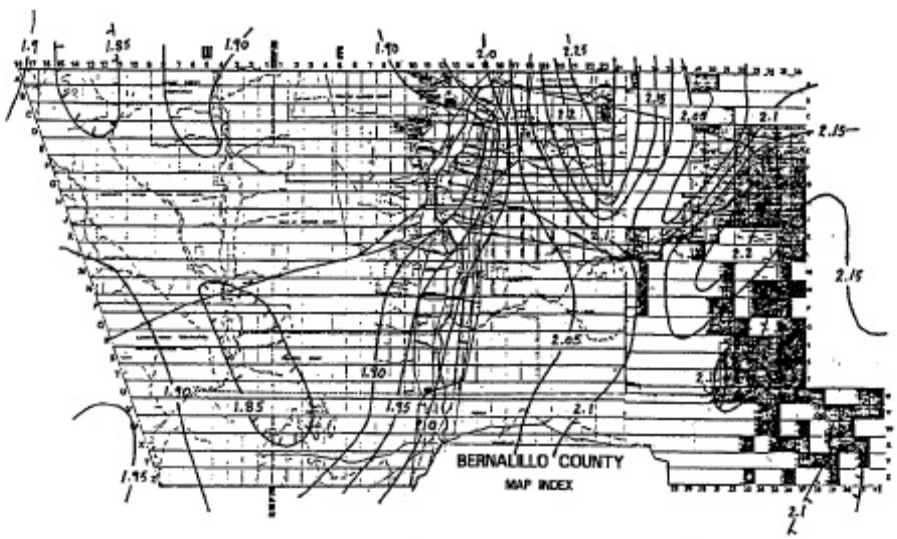
$P_T - P_{1440} + (P_{360} - P_{1440}) * (30^B - (t / 60 + 6)^B) / (30^B - 12^B)$, 360 < t 1440 (c-6)

where $A = \log_{10}(P_{360} / P_{60}) / \log_{10}(6.0)$

and $B = \log_{10}(P_{1440} / P_{360}) / \log_{10}(4.0)$

Figure C-1 - P_{60}

Isopluvials of 100-year, 1-hour precipitation.



From Figures 24 and 30, *NOAA Atlas 2, Volume IV*, and Y_{100} from equation on Table 11

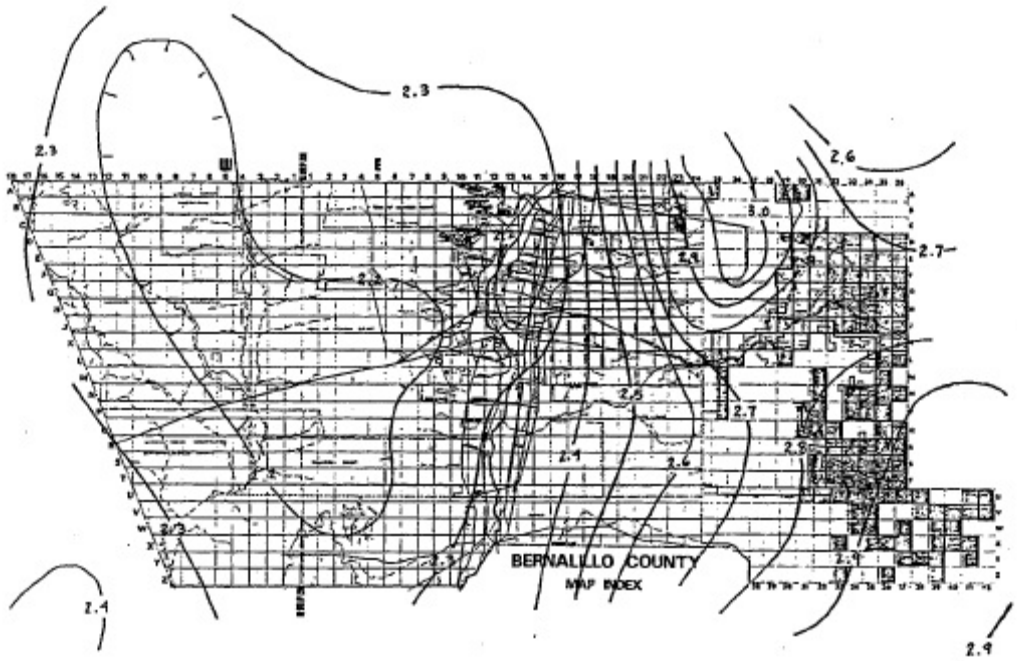


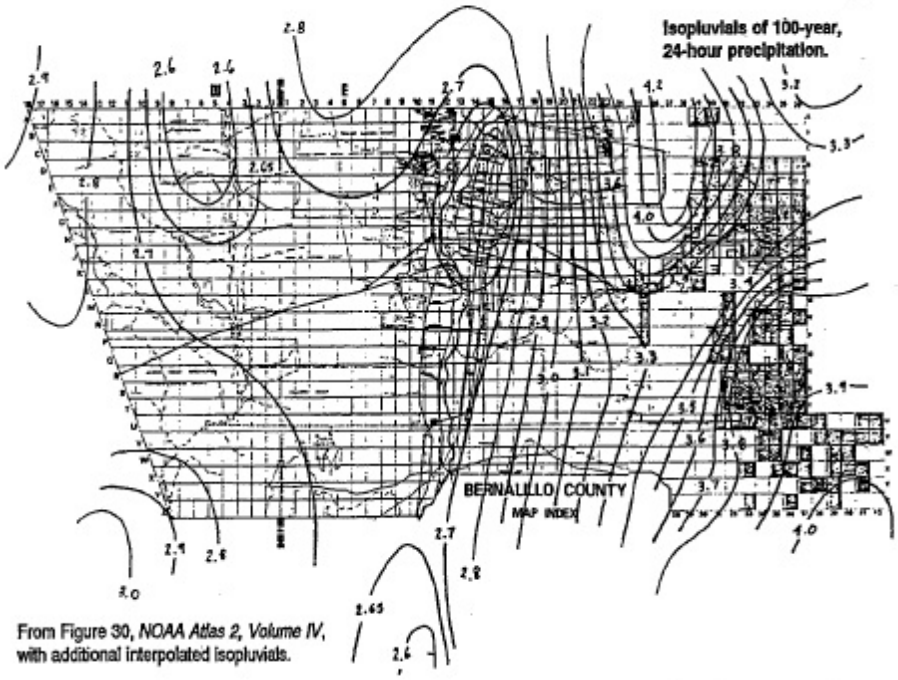
Figure C-2 - P_{360}

Isopluvials of 100-year, 6-hour precipitation.

From Figure 24, *NOAA Atlas 2, Volume IV*, with additional interpolated isopluvials.

Figure C-3 - P_{1440}

Isopluvials of 100-year, 24-hour precipitation.



From Figure 30, *NOAA Atlas 2, Volume IV*, with additional interpolated isopluvials.

3) Some applications must consider storms of greater frequency than the 100-year storm. To estimate the 360-minute and 1440-minute precipitation at return periods more frequent than 100-years, use the following formula:

$$P_{N\text{-}YEAR} = \{1 - (\log_{10}(100/year) * 0.333\}\} * P_{N\text{-}100}, \quad 2 \leq = year < 100 \qquad \text{(c-7)}$$

The return period factors from TABLE A-3 may also be utilized.

To estimate the 60-minute precipitation at return periods more frequent that 100-years, use the following formula:

$$P_{60\text{-}YEAR} = \{0.494 - (\log_{10}(100/year) * 0.505 / \log_{10}(50))\}$$

$$+ \{0.755 + (\log_{10}(100/year) * 0.187 / \log_{10}(50))\}$$

$$\{* 1 - (\log_{10}(100/year) * 0.333)\}$$

$$* (P_{360\text{-}100}) * (P_{360\text{-}100}) / (P_{1440\text{-}100}), \quad 2 \leq = year < 100 \qquad \text{(c-8)}$$

4) Analysis of retention ponds and detention dams may require storm durations longer than 24-hours. The procedures in S.C.S. TSC Technical Note - Hydrology PO-6 (Rev. 2) may be used to determine precipitation for storm durations between 1 day and 10 days. In Bernalillo County, the following equation may be used in place of the method in PO-6 to determine the 100-year 10-day storm.

$$P_{10days} = 10.0 - (24.9 / (P_{1440})^{1.4}) \qquad \text{(c-9)}$$

The 100-year storm between 1-day and 10-days can be computed with the following equation:

$$P_D = P_{1440} + (0.469 * \log_{10}(D)) + (0.059 * (D-1)) * (P_{10days} - P_{1440}) \qquad \text{(c-10)}$$

where D = duration in days

Example C-1
Determine the 1-hour, 6-hour, 24-hour, 4-day and 10-day precipitation for a 100-year frequency storm at a subbasin located on page E-20 of the Albuquerque/Bernalillo County Zone Atlas. Also determine the 1-hour, 6-hour and 24-hour precipitation for a 10-year frequency storm at this location. From Figures C-1, C-2, and C-3 P_{60} = 2.15 inches P_{360} = 2.57 inches P_{1440} = 3.02 inches $P_{10days} = 10.0 - 24.9 / 3.02^{1.4} = 4.70$ inches $P_{4days} = 3.02 + (0.469 * \log_{10}(4) + 0.059 * (4-1)) * (4.70 - 3.02)$ $= 3.79$ inches $P_{360\text{-}10yr} = (1 - (\log_{10}(100 / 10) * 0.333)) * 2.57 = 1.71$ inches $P_{1440\text{-}10yr} = (1 - (\log_{10}(100 / 10) * 0.333)) * 3.02 = 2.01$ inches $P_{60\text{-}10y} = 0.494 - (\log_{10}(100 / 10) * 0.505 / \log_{10}(50))$

$+ .755 + (\log_{10}(100 / 10) * 0.187 / \log_{10}(50))$
$* 1 - (\log_{10}(100 / 10) * 0.333) * 2.57 * 2.57 / 3.02$
$= 0.1968 + 0.8651 * 0.667 * 2.57 * 2.57 / 3.02$
$P_{60\text{-}10yr} = 1.46$ inches

5) When evaluating uncontrolled watersheds larger than five (5) square miles, the precipitation amounts may be reduced by multiplying the precipitation amounts from FIGURES C-1, C-2, and C-3by the "Percent of Point Precipitation" obtained from FIGURE C-4. Uncontrolled watersheds means those areas not controlled by dams, ponds or partial diversions.

For large complex watersheds which include several dams and partial diversions, the method of successive subtraction may provide more detailed modeling of the watershed. The U.S. Bureau of Reclamation computer program SUCSUB, is a useful tool when utilizing this procedure. The user manual for SUCSUB is contained in the U.S. Bureau of Reclamation, Personal Computer Programs. Estimating Probable Maximum Precipitation and Precipitation Frequency-Duration in the Western United States (1989).

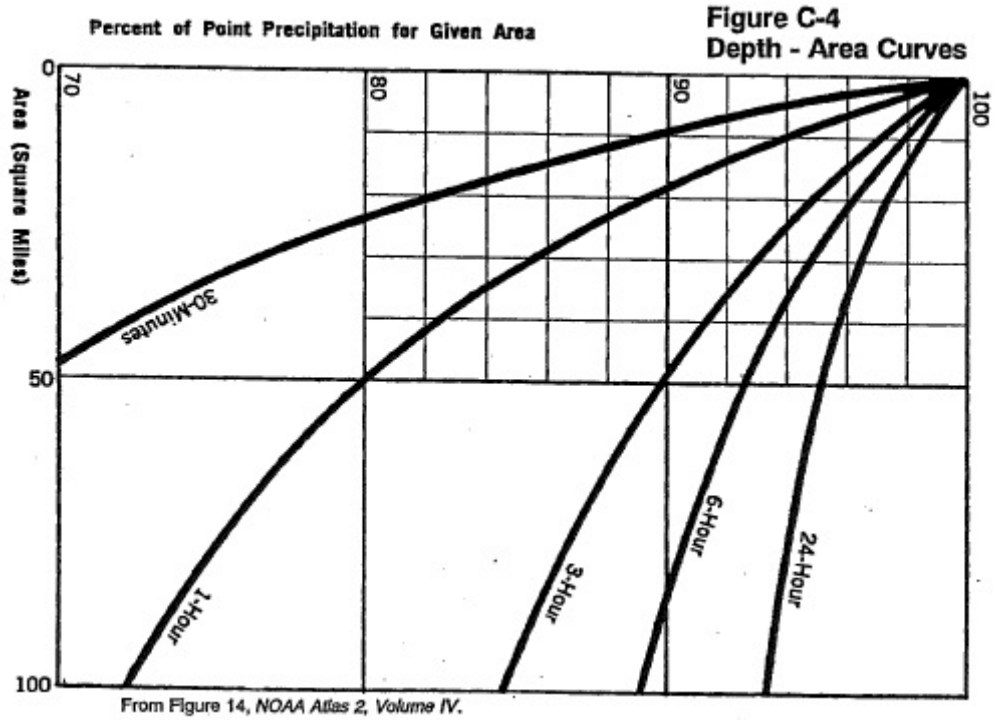
The use of FIGURE C-4 is appropriate for sizing major dams, channels and arroyos but may not be appropriate for sizing channel inlets, side drainage and storm sewers associated with these major facilities. Use of a single "Percent of Point Precipitation" value for large drainage studies may cause flows in the upper reaches of the study area to be under estimated. It may be necessary to evaluate major projects with and without area reduction factors and to establish the capacity of intermediate facilities based on a ratio of the values obtained.

C.4 HYDROGRAPH COMPUTATION USING HYMO

The steps which follow contain the procedure for computation of a hydrograph using the HYMO computer program. The original HYMO program developed by A.R.S. must be modified to compute runoff based on initial abstraction/uniform infiltration in order to use this procedure. The procedure described in this section utilizes the COMPUTE HYD function of the standard HYMO computer 'program. 1) Normally, only the 6-hour storm is input into the HYMO computer program (t•360 min). HYMO recession coefficients have been calibrated using a rainfall distribution incremental time of 0.033333 hours (2 minutes). Applications requiring consideration of pond or dam storage, and large watersheds, will likely require use of a 24-hour or longer storm. A rainfall distribution incremental time of 0.05 hours (3 minutes) should be used in 24-hour storm applications. The HYMO recession coefficients are also applicable to a 3 minute incremental time2) Determine the areas in each of the pervious land treatments (A, B, and C) and the area of impervious land treatment (D). TABLE A-4 contains a definition of the four treatments. TABLE A-5 may be used to guide the determination of the percentage of treatment D where more specific data is not available. The areas are identified herein as $A_{A, AB, AC, \text{and } AD}$2a) Compute the initial abstraction and infiltration for the pervious treatments (A, B, and C) using the values in TABLE A-6 and TABLE A-7 weighted by the respective areal proportion.

IA-Pervious = $(0.65 * A_A + 0.50 * A_B + 0.35 * A_C) / (A_A + A_B + A_C)$ (c-11)

INF-Pervious = $(1.67 * A_A + 1.25 * A_B + 0.83 * A_C) / (A_A + A_B + A_C)$ (c-12)



2b) The initial abstraction and infiltration for the impervious areas (land treatment D) are obtained directly from TABLES A-6 and A-7.IA - Impervious = 0.10

INF - Impervious = 0.04 at $0 \leq t < 180$ min.

= $0.08 - (0.04 * t / 180)$ at $180 \leq t < 360$

= 0.0 at $360 \leq t$

The revised version of the HYMO program includes this infiltration rate formulation3) Compute the time of concentration, t_C, using equations b-1 and b-2, or equation b-6, or equations b-7 and b-8, and the factors in TABLES B-1 or B-2. A single t_c is computed for the entire (pervious and impervious) watershed.

Consideration should be given to splitting large watersheds into smaller subbasins with reach lengths less than 4000 feet. Smaller subbasins will allow more accurate modeling of channels and basin topography, and should provide for greater modeling accuracy4) Set time to peak, t_p, equal to 2/3 t_c, but never less than 0.133333 hours (8.0 minutes5) The shape of the unit hydrograph is determined by the t_p and recession constant k. The value of k/t_p is used to calibrate the hydrograph peak flow rate. The value of k/tp is computed as a function of the sub-basin area, land treatment (IA and INF and precipitation (P_{60})

5a) For a watershed with a total area ($A_A + A_B + A_C + A_D$) between 0 and 40 acres, use the following equation:

k/t_{p40} = MAX 0, (1.19 + (1.4 * IA) - P_{60}) * (0.035 + (0.150 * INF))

+ (1 + INF) * 0.44 + (P_{60} - 2.0) * (0.073 * (0.014 * INF)) (c-13)

where: MAX(a,b,) = Maximum Value of a or b

and k/t_{p40} (maximum) = 1.35,

and k/t_{p40} (minimum) = 0.545

For the four treatment types, equation c-13 can be reformulated as a set of linear equations as follows:

i) Treatment A with: IA = 0.65 INF = 1.67

@ P_{60} <2.10, k/t_{p-40} = 1.58159 - (P_{60} * .18912) (c-14)

@ $P_{60} \geq$ 2.10, k/t_{p-40} = 0.98204 + (P_{60} * .09638) (c-15)

k/t_{p40} (maximum) = 1.35

ii) Treatment B with: IA = 0.50 INF = 1.25

@P_{60} <1.89, k/t_{p40} = 1.22953 - (P_{60} * .1320) (c-16)

@$P_{60} \geq$1.89, k/t_{p40} = 0.80900 + (P_{60} * .0905) (c-17)

iii) Treatment C with: IA = 0.35 INF = 0.83

@P_{60} <1.68, k/t_{p40} = 0.90392 - (P_{60} * .07488) (c-18)

@$P_{60} \geq$1.68, k/t_{p40} = 0.63596 + (P_{60} * .08462) (c-19)

iv) Treatment D with: IA = 0.10 INF = 0.04

@P_{60} < 1.33, k/t_{p40} = 0.5450 (c-20)

@$P_{60} \geq$ 1.33, k/t_{p40} = 0.31048 + (P_{60} * .07356) (c-21)

k/t_{p40}(minimum) = 0.545

5b) For a watershed with a total area ($A_A + A_B + A_C + A_D$) larger than 200 acres, use the following equation:

k/t_{p200} = 0.52 + 0.20 * INF + (0.18 + 0.24 * INF) * ($2^{(2.25 * (1 - P_{60}))}$) (c-22)

The value of k/t_{p200} computed by equation c-22 is set to a maximum value of 1.30.

For the four land treatment types, equation c-22 can be reformulated as follows:

i) Treatment A with: INF =1.67

k/t_{p200} = 0.854 + 0.5808 * ($4.756828^{(1.0 - P_{60})}$) (c-23)

ii) Treatment B with: INF = 1.25

k/t_{p200} = 0.770 + 0.4800 * ($4.756828^{(1.0 - P_{60})}$) (c-24)

iii) Treatment C with: INF = 0.83

$k/t_{p200} = 0.686 + 0.3792 * (4.756828^{(1.0 - P_{60})})$ (c-25)

iv) Treatment D with: INF = 0.04

$k/t_{p200} = 0.528 + 0.1896 * (4.756828^{(1.0 - P_{60})})$ (c-26)

5c) For watershed with a total area smaller than 40 acres or larger than 200 acres, compute the k/t_p for the pervious areas ($A_A + A_B + A_C$) by using equations c-14 through c-19; or c-23, c-24 and c-25 weighted by the respective areal proportionk/t_p - pervious = $(k/t_{p@A} * A_A + k/t_{p@B} * A_B + k/t_{p@C} * A_C) / (A_A + A_B + A_C)$ (c-27)

The k/t_p for impervious areas is computed from equation c-20, c-21 or c-26 depending of the sub-basin area and P60.

5d) For watersheds between 40 and 200 acres (pervious and impervious), compute the k/tp as follows:

i) Compute k/t_{p40} for the pervious areas ($A_A + A_B + A_C$) using equation c-27 and equations c-14 through c-19, as applicable.

Computer k/t_{p40} for the impervious area (A_D) using equation c-20 or c-21.

ii) Computer k/t_{p200} for the pervious areas ($A_A + A_B + A_C$) using equation c-27 and equations c-23, c-24 and c-25.

Computer k/t_{p200} for the impervious area (A_D) using equation c-26.

iii) Compute the area-adjusted k/t_p ratio by linear interpolation for the impervious and pervious areas by separate computations using the following equation:

$k/t_p = k/t_{p40} + (A-40) * (k/t_{p200} - k/t_{p40}) / 160$ (c-28)

where $A = A_A + A_B + A_C + A_D$ (acres)

5e) Compute the k for the pervious and impervious areas separately by multiplying the k/t_p ratio by the t_p computed in step 4, herein6) The COMPUTE HYD function of the HYMO computer program is used to compute the watershed hydrograph. Separate hydrographs are computed for the pervious and impervious portions of the watershed, then the two separate hydrographs (pervious and impervious) are added to obtain a combined hydrograph.

6a) The following data for the pervious area hydrograph is input into the COMPUTE HYD function of HYMO:

ID NUMBER = Internal storage location (1 to 6)

HYD NUMBER = Hydrograph identification number

DT = Time increment of mass rainfall input and hydrograph output (Use 0.033333 hrs for a 6-hour storm and 0.05 hrs for a 24-hour storm.) DRAINAGE

AREA = $A_A + A_B + A_C$ for the pervious portion of the watershed

IA = Initial Abstraction computed by equation c-11 (Input as a negative number.)

INF = Uniform Infiltration computed by equation c-12 (Input as a negative number.)

k = The recession constant for the pervious area computed by the procedures in Section 5a through 5e (Input as a negative number.)

t_p = Time to peak for the entire watershed (hrs) which is equal to 2/3 t_c (Input as a negative number.)

RAINFALL = The mass rainfall data computed from equations c-1 through c-5 (using a 2 minute time increment for a 6-hour storm) or equations c-1 through c-6 (using a 3 minute time increment for a 24-hour storm). 6b) The following data for the impervious area hydrograph is input into the COMPUTE HYD function of HYMO:

ID NUMBER = Internal storage location (1 to 6, and different from the number used for the pervious area)

HYD NUMBER = Hydrograph identification number

DT = Time increment **(**Usethe same value as the prervious area**)**

DRAINAGE AREA = $A_{D,}$ for the impervious portion of the watershed

IA = Initial Abstraction or 0.10 for impervious areas. (Input as a **negative** number.)

INF = Uniform Infiltration or 0.04 for impervious areas. (Input as a **positive** number to reduce the impervious infiltration rate after 3.0 hours.)

k = The recession constant for the impervious area computed by the procedures in Section 5a through 5e (Input as a **negative** number.)

t_p = Time to peak for the entire watershed (hours). (Use the same value as the pervious area, input as a **negative** number.) RAINFALL = The mass rainfall; input a "-1" for this input to indicate use of the previous mass rainfall.

6c) The pervious hydrograph and the impervious hydrograph are added using the ADD HYD function of the HYMO computer program. The hydrograph values are output using the PRINT HYD function.

Example C-2

Find Q_P (100-year storm) for 1120 acres centered at Sheet E-10 of the City/County Zone Atlas. Land treatments are 240 acres of treatment A, 400 acres of treatment B, 160 acres of treatment C, and 320 acres of treatment D. These areas are proportional to the treatments in Example A-5. The flow path is: 600 ft. mountain brush (400 ft. bare ground sheet flow and 200 ft. shallow concentrated flow), s = 0.05 ft/ft; 2000 ft. desert terrain (shallow concentrated flow), s = 0.03 ft/ft; and 6000 ft. medium density urban (streets, and storm sewers), s = 0.02 ft/ft. The watershed centroid is at 55% of the total basin length. Do not adjust t_p for steep slopes with this example.

<u>Pervious portion</u>

Pervious Area = 240 + 400 + 160 = 800 acres = 1.25 sq. mi.

Composite IA = (240 * 0.65 + 400 * 0.50 + 160 * 0.35) / 800 = 0.515 inches

Composite INF.= (240 * 1.67 + 400 * 1.25 + 160 * 0.83) / 800 = 1.292 inches/hour

L = 600 + 2000 + 6000 = 8600 feet

L_{CA} = 0.55 * 8600 = 4730 feet

s = (0.05 * 600 + 0.03 * 2000 + 0.02 * 6000) / 8600 = 0.0244 ft/ft

Use K = 3.0 for portions of basin below first 2000 feet

K = (8600 / (0.0244)) / ((400 / (1* Sq. rt.(0.05))) + (200/(2* Sq. rt.(0.05))) +
(1400 / (2*Sq. rt.(0.03))) + (600 / (3*(0.03))) + ((6000/3*Sq. rt.(0.02)))) = 2.552

K_N = (.042 * 600 + .033 * 2000 + .021 * 6000) / 8600 = 0.0253

t_c = ((12000 - 8600) / (72000 * 2.552 * $0.0244^{0.5}$)) + ((8600 - 4000) * 0.0253 * $0.55^{0.33}$ / (552.2 * $0.0244^{0.165}$))
= 0.1185 + 0.3193 = 0.4378 hours

t_p = 2/3 * 0.4378 = 0.292 hours

Since the total area (1120 acres) is greater than 200 acres use equations c-23, c-24 and c-25, to compute the k/t_p for each treatment type.

From FIGURE C-1, the one-hour precipitation, P_{60}, is 1.88 inches.

@ Treatment A

k/t_p = 0.854 + (0.5808 * ($4.756828^{(1.0-1.88)}$)) = 1.0012

@ Treatment B

$k/t_p = 0.770 + (0.4800 * (4.756828^{(1.0-1.88)})) = 0.8917$

@ Treatment C

$k/t_p = 0.686 + (0.3792 * (4.756828^{(1.0-1.88)})) = 0.7821$

Composite $k/t_p = (1.0012 * 240 + 0.8917 * 400 + 0.7821 * 160) / 800 = 0.9026$

$k = 0.292 * 0.9026 = 0.2636$ hours

The 6-hour precipitation, P_{360}, is obtained from FIGURE C-2

A mass rainfall from equations c-1 through c-5 is obtained for a 2 minute incremental time using:

$P_{60} = 1.88$ inches $P_{360} = 2.22$ inches

The values for the pervious area are input into the COMPUTE HYD function of HYMO to obtain the following output for the pervious hydrograph:

$Q_p = 906$ cfs @ 1.700 hrs Volume = 43.4181 Ac Ft

<u>Impervious Portion</u>

Impervious Area = 320 acres = 0.50 sq. mi.

IA = 0.10

INF = 0.04 (0 < t 180 min.)

t_p = 0.292 hr (same as for the pervious area)

Since the total area (1120 acres) is greater than 200 acres, use equation c-26 to compute k/t_p, P_{60} is equal to 1.88 inches.

$k/t_p = 0.528 + (0.1896 * (4.756828^{(1.0-1.88)})) = 0.5761$

$k = 0.292 * .5761 = .1682$

A "-1" is input as the mass rainfall to utilize the previously input rainfall distribution.

The values for the impervious area are input into the COMPUTE HYD function of HYMO to obtain the following:

$Q_p = 924$ cfs @ 1.667 hrs Volume = 52.9338 Ac Ft

The impervious and pervious hydrographs are totaled using the ADD HYD function of HYMO to obtain the combined hydrograph for the basin with:

$Q_p = 1828$ cfs @ 1.667 hrs, Volume = 96.3518 Ac Ft

Example C-3

Find Q_p (100-year storm) for 112 acres in Sheet E-10 of the City/County Zone Atlas. Land treatments are 24 acres of treatment A, 40 acres of treatment B, 16 acres of treatment C, and 32 acres of treatment D. These areas are proportional to the treatments of Example A-5 and Example C-2. The flow path is: 232 ft. bare ground, s = 0.05 ft/ft; 774 ft. shallow concentrated flow, s = 0.03 ft/ft; and 2322 ft. natural channels, s = 0.02 ft/ft.

<u>Pervious portion</u>

Pervious Area = 24 + 40 + 16 = 80 acres = 0.125 sq. mi.

Composite IA = 0.515 (Example C-2)

Composite INF = 1.292 (Example C-2)

t_c= ((232 / (10 * 1 * Sq. rt. 0.05)) + (774 / (10 * 2 * Sq. rt. 0.03)) + (2322 / (10 * 3 * Sq. rt.0.02))) / 3600 = 0.243 hour

t_p = 2/3 (0.243) = 0.162 hour

The watershed area (112 acres) is greater than 40 acres and less than 200 acres, therefore k/t_{p40} and k/t_{p200} must be computed. P_{60} at location E-10 is 1.88 inches, from FIGURE C-1. Equations c-14, c-16 and c-19 are used to compute k/t_{p40} .

@ Treatment A

k/t_{p40} = 1.58159 - ((1.88) * .18912) = 1.2260

@ Treatment B

k/t_{p40} = 1.22953 - ((1.88) * .13200) = 0.9814

@ Treatment C

k/t_{p40} = 0.63596 + ((1.88) * .08462) = 0.7950

Composite k/t_{p40} = (1.2260 * 24 + 0.9814 * 40 + 0.7950 * 16) / 80 = 1.0175

The k/t_{p200} for treatments A, B and C are computed in Example C-2.

Composite k/t_{p200} = (1.0012 * 24 + 0.8917 * 40 + 0.7821 * 16) /80 = 0.9026

The area adjusted k/t_p, ratio is computed from equation c-28.

k/t_p = 1.0175 + {(112 - 40) * (0.9026 - 1.0175) / 160 = 0.9658

k = 0.162 * 0.9658 = 0.1565 hrs

The 6-hour precipitation, P_{360} = 2.22 inches, is obtained from FIGURE C-2

The mass rainfall is obtained from equations c-1 through c-5.

For this example, it is the same as the mass rainfall used in Example C-2.

The values for the pervious area are input into the COMPUTE HYD function of HYMO to obtain the following:

Q_p = 139.9 cfs @ 1.533 hrs, Volume = 4.3418 Ac Ft

<u>Impervious Portion</u>

Impervious Area = 32 acres = 0.05 sq. mi.

IA = 0.10 inches

INF = 0.04 inches/hour (0 < t 180 min.)

t_p = 0.162 hr (same as for the pervious area)

The k/t_p, ratio is computed by first computing k/t_{p40} and k/t_{p200} .

Compute k/t_{p40} from equation c-21.

$k/t_{p40} = 0.31048 + ((1.88) * 0.07356) = 0.4488$

use $k/t_{p40} = 0.545$ (minimum value)

Compute k/t_{p200} from equation c-26.

$k/t_{p200} = 0.528 + (0.1896 * (4.756828^{(1.0-1.88)})) = 0.5761$

The area adjusted k/t_p ratio is computed from equation c-28.

$k/t_p = 0.5450 + (112 - 40) * (0.5761 - 0.5450) / 160 = 0.5590$

$k = 0.162 * 0.5590 = 0.0906$ hrs

A "-1" is input as the mass rainfall to utilize the previously input rainfall distribution.

The values for the impervious area are input into the COMPUTE HYD function of

HYMO to obtain the following:

$Q_p = 127.9$ cfs @ 1.533 hrs, Volume = 5.2934 Ac Ft

The impervious and pervious hydrographs are totaled using the ADD HYD function of HYMO to obtain the combined hydrograph for the watershed with:

$Q_p = 267.7$ cfs @ 1.533 hrs, Volume = 9.6352 Ac Ft

C.5 SIMPLIFIED INPUT FOR HYDROGRAPH COMPUTATION

The relatively complex procedures described in the previous section can be formulated as a sequence of mathematical operations in a computer program. A new function, labeled COMPUTE NM HYD has been developed for the AHYMO computer program (AMAFCA's enhanced version) to simplify the hydrograph computation and use the procedures outlined in the previous section. A new function, labeled RAINFALL, has been developed for AHYMO to simplify the computation of mass rainfall data. The COMPUTE NM HYD and RAINFALL functions are included only on the enhanced version of HYMO available from AMAFCA (AHYMO) and several local engineering consultants. 1) The steps which follow contain the procedure for use of the RAINFALL function:

1a) Determine the one hour and six hour (P_{60} and P_{360}) precipitation from FIGURES C-1 and C-2. If necessary to utilize more frequent storms, use TABLE A-3 and equations c-7 and c-8. For watersheds larger than five (5) square miles, depth-area adjustment may also be applicable. Before the first COMPUTE NM HYD, the program must compute a rainfall distribution; the one hour precipitation (P_{60}) and six hour precipitation (P_{360}) are required input. The program will compute the rainfall distribution at specified time increments using equations c-1 through c-5. Equation c-6 is added for the 24-hour precipitation. It is recommended that the 24-hour (P_{1440}) precipitation amount be computed to allow for the convenient application of a 24-hour rainfall distribution when dams and ponding are to be considered.

1b) Determine the type of distribution to be used.

For a 6-hour rainfall distribution use "TYPE=1".

For a 24-hour rainfall distribution use "TYPE=2".

1c) Determine the incremental time to be used in the computation. Normally 0.033333 hr. (2 min.) is used for the 6-hour rainfall distribution and 0.05 hr. (3 min.) is used for the 24-hour rainfall distribution.

1d) The following data is input into the RAINFALL function of AHYMO:

RAINFALL TYPE = 1 for 6-hour rainfall distribution
= 2 for 24-hour rainfall distribution

QUARTER HOUR PRECIPITATION = 0.0 for TYPE 1 and 2 distributions

ONE HOUR PRECIPITATION = P_{60}

SIX HOUR PRECIPITATION = P_{360}

24-HOUR PRECIPITATION = P_{1440}

DT = Incremental time (hours)

2) The steps which follow contain the procedure for use of the COMPUTE NM HYD function:

2a) Determine the total sub-basin area and the area or percentage of each of the four land treatments (A, B, C and D) using TABLE A-4 and TABLE A-5. Since the COMPUTE NM HYD function reads the total sub-basin area, the land treatments may be computed as ratios of the total area (0 to 1), as percentages of the total area (0 to 100) or as areas in acres or square miles.

2b) Compute the time of concentration, t_c, for the entire sub-basin using equations b-1 and b-2, or equation b-6, or equations b-7 and b-8, and the factors in TABLES B-1 or B-2. Set the time to peak, t_P, equal to 2/3 t_C, but never less than 0.133333 hours (8.0 minutes). The value of t_P, is input into the COMPUTE NM HYD function.

Consideration should be given to splitting large watersheds into smaller subbasins with reach lengths less than 4000 feet. Smaller sub-basins will allow more accurate modeling of channels and basin topography, and should provide for greater modeling accuracy.

2c) The COMPUTE NM HYD function is utilized to compute the sub-basin hydrograph. The computation of k/t_p, ratios, infiltration, initial abstraction, and separate pervious and impervious hydrographs is all performed by the program. The following data is input into the COMPUTE NM HYD function of AHYMO:

ID NUMBER = Internal storage location (1 to 20 with AHYMO by AMAFCA)

HYD NUMBER = Hydrograph identification number

DRAINAGE AREA = The total sub-basin area (in square miles)

AREA OF LAND TREATMENT A,B,C AND D = As a percentage of total (0 to 100), as a ratio (0 to 1), or use areas in square miles or acres (four values are input)

t_p = Time to peak for the entire sub-basin (hours) which is equal to 2/3 t_C. (Input as a negative or positive number.)

MASS RAINFALL = if the RAINFALL function was previously used, input a "-1"

The hydrograph values are output using the PRINT HYD function of AHYMO

Example C-4
Compute the hydrograph using RAINFALL and COMPUTE NM HYD from the data in Example C-3. From FIGURES C-1, C-2 and C-3 the precipitation at location E-10 is obtained as: P_{60} = 1.88 in P_{360} = 2.22 in P_{1440} = 2.68 in Use a TYPE = 1 rainfall distribution and DT = 0.033333 hrs These values are input into the RAINFALL function of AHYMO. TOTAL AREA = 24 + 40 + 16 + 32 = 112 acres = 0.175 sq mi Compute the land treatment percentage: Percent A = (24/112) * 100 = 21.43 Percent B = (40/112) * 100 = 35.71 Percent C = (16/112) * 100 = 14.29 Percent D = (32/112) * 100 = 28.57 As an alternative, the acreage may also be used as input. t_c = 0.243 hr t_p, = (2/3) t_c, = (2/3) * 0.243 = 0.162 hr The above values are input into the COMPUTE NM HYD function of AHYMO to obtain the following: Q_p = 267.8 cfs @ 1.533 hrs, Volume = 9.6351 Ac Ft

PART D - USE OF ALTERNATE PROCEDURES

Hydrology methods other than those specified in Parts A through D may be appropriate for local conditions and may be acceptable to the City and other reviewing agencies. The use of alternate procedures should be reviewed with the agency who will be approving the hydrology early in the project to establish that such alternate procedures are acceptable and to establish

specific parameters. A predesign meeting with the City Engineer or designee is required prior to submitting the drainage report.

In general, computer programs which are in the public domain, have available users manuals and established use in the engineering community will most likely be accepted as an alternative. Areas which require special analysis because of unusual terrain conditions, special sediment considerations, unique hydraulic conditions, or extraordinary soil conditions are candidates for alternate procedures. Use of special procedures will be considered when experimental testing and analysis of measured precipitation and runoff conditions indicates that the special procedures will provide more accurate results. The use of proprietary computer programs and programs available only to a small segment of the engineering community will require additional documentation to establish that they are an acceptable alternative. Documentation should include users manuals, discussion of the engineering principals and formulas utilized, and calibration to establish that the methodology is applicable to the local area. The use of an alternate computer program solely on the basis that it gives lower or higher numbers will not be acceptable.

PART E - SYMBOLS AND BIBLIOGRAPHY

E.1 DEFINITIONS OF SYMBOLS

When evaluation equations use the following order of precedence: 1) parentheses, 2) functions (i.e., SIN or LOG), 3) power or square root, 4) multiplication or division, 5) addition or subtraction.

A_A area in land treatment A

A_B area in land treatment B

A_C area in land treatment C

A_D area in land treatment D

A_T total area in sub-basin

Ac Ft acre feet

C Rational Method coefficient

C_A Rational Method coefficient for treatment A

C_B Rational Method coefficient for treatment B

C_C Rational Method coefficient for treatment C

C_D Rational Method coefficient for treatment D

cfs cubic feet per second

CN SCS Curve Number

D duration in days

e base of natural logarithm system = 2.71828

E excess precipitation

E_A excess precipitation for treatment A

E_B excess precipitation for treatment B

E_C excess precipitation for treatment C

E_D excess precipitation for treatment D

EA elevation Adjustment factor for PMP_{60}

Elev elevation (feet)

Ft feet

hr hour

I Rational Method intensity (inches/hour)

IA initial abstraction (inches)

INF infiltration (inches/hour)

K conveyance factor for SCS Upland Method

k recession coefficient for HYMO program

K_N basin factor for lag time equation

K_X conveyance factor for watershed subreach

k/t_{pA} k divided by t_p for treatment A

k/t_{pB} k divided by t_p for treatment B

k/t_{pC} k divided by t_p for treatment C

k/t_{pD} k divided by t_p for treatment D

k/t_{p40} k divided by t_p for 40 acres or smaller area

k/t_{p200} k divided by t_p for 200 acres or larger area

L length of subreach (feet)

L_{CA} distance to centroid of drainage basin (feet)

L_G lag time (hours)

L_X length of watershed subreach

In natural logarithm (base e)

$\log_{10}$ base 10 logarithm

mi^2 square mile(s)

n Manning's roughness coefficient

P_{12} 12-minute precipitation

P_{60} 60-minute precipitation at 100-year storm

$P_{60\text{-}2}$ 60-minute precipitation at 2-year storm

$P_{60\text{-}year}$ 60-minute precipitation at "year" storm

P_{360} 360-minute precipitation at 100-year storm

$P_{360\text{-}2}$ 360-minute precipitation at 2-year storm

$P_{360\text{-}10}$ 360-minute precipitation at 10-year storm

P_{1440} 1440-minute (24-hr) precipitation, 100-year storm

$P_{1440\text{-}2}$ 1440-minute (24-hr) precipitation at 2-year storm

P_D precipitation for "D"-days duration

$P_{N\text{-}100}$ "n"-minute precipitation at 100-year storm

$P_{N\text{-}YEAR}$ "n"-minute precipitation at "year" storm

P_T precipitation at any time, t

PMF Probable Maximum Flood

1/2PMF one-half of the Probable Maximum Flood

PMP_{15} 15-minute Probable Maximum Precipitation

PMP_{60} 60-minute Probable Maximum Precipitation

PMP_{360} 360-minute Probable Maximum Precipitation

PMP_T Probable Maximum Precipitation at anytime, t

Q_P peak discharge (cfs)

Q_{PA} peak discharge rate (cfs/acre) for treatment A

Q_{PB} peak discharge rate (cfs/acre) for treatment B

Q_{PC} peak discharge rate (cfs/acre) for treatment C

Q_{PD} peak discharge rate (cfs/acre) for treatment D

s slope of subreach in foot per foot

t time in minutes

t_B base time for small watershed hydrograph

t_C time of concentration (hours)

t_p time to peak (hours)

v velocity of flow in watershed (feet/sec)

v_x velocity of flow in watershed subreach

V_{360} runoff volume for 360-minute storm

V_{1440} runoff volume for 1440-minute storm

V_{4days} runoff volume for 4-day storm

V_{10days} runoff volume for 10-day storm

y^x y to the x power

+ addition operator

- subtraction operator

* multiplication operator

/ division operator

$\sqrt{\ }$ square root operator

E.2 BIBLIOGRAPHY

Alley, W.M. and Smith, P.E., 1982, *Distributed Routing Rainfall-Runoff Model, Version II, Computer Program Documentation,* Users Manual, Open-File Report 82-344, U.S.D.I. Geological Survey, Denver, Colorado, 56 p.

Anderson, C.E., 1992, *AHYMO*, *Computer Program Users Manual, (AMAFCA Hydrologic Model)*, Albuquerque Metropolitan Arroyo Flood Control Authority, Albuquerque, New Mexico, 44 p.

Anderson, C.E., and Heggen, R.J., 1990, "Evolution of Drainage Criteria, Albuquerque, New Mexico", *Hydraulics/Hydrology of Arid Lands, Proceedings of the International Symposium,* Hydraulics Division, American Society of Civil Engineers, New York, New York, pp. 84-89.

Cudworth, A.G., 1989, *Flood Hydrology Manual,* U.S.D.I. Bureau of Reclamation, Denver, Colorado, 243 p.

Hansen, E.M., Fenn, D.P., Schreiner, L.C., Stodt, R.W., and Miller, J.F., 1988, *Hydrometeorological Report No. 55A, Probable Maximum Precipitation Estimates - United States Between the Continental Divide and the 103rd Meridian,* U.S.D.C. National Oceanic and Atmospheric Administration, Silver Springs, Maryland.

Heggen, R.J., 1987, *Split Ring Infiltration Basic Data Collection and Interpretation,* Report No. PDS 110/210, Bureau of Engineering Research, University of New Mexico, Albuquerque, New Mexico, 28 p.

Huber, W.C., and Dickinson, R.E., 1988, *Storm Water Management Model, Version 4, Users Manual,* Environmental Research Laboratory, U.S. Environmental Protection Agency, Athens, Georgia, 569 p.

James, W.P., 1986, *A & M Watershed Model, Users Manual,* Texas A & M University, College Station, Texas, 281 p.

Miller, J.F., Frederick, R.H., and Tracey, R.J., 1973, *Precipitation-Frequency Atlas of the Western United States, Volume IV, New Mexico,* NOAA Atlas 2, National Oceanic and Atmospheric Administration, Silver Springs, Maryland.

Roesner, L.A., Aldrich, J.A., and Dickinson, R.E., 1988, *Storm Water Management Model, Version 4, Addendum I EXTRAN,* Environmental Research Laboratory, U.S. Environmental Protection Agency, Athens, Georgia, 157 p.

Sabol, G.V., Ward, T.J., and Seiger, A.D., 1982, *Rainfall Infiltration of Selected Soils in the Albuquerque Drainage Area for the Albuquerque Metropolitan Arroyo Flood Control Authority,* Civil Engineering Department, New Mexico State University, Las Cruces, New Mexico, 110 p.

Urban Drainage and Flood Control District, 1969, Update 1990, *Urban Storm Drainage Criteria Manual,* Volume I and II, Denver, Colorado.

U.S. Army Corps of Engineers, 1987, *HEC*-1, *Flood Hydrograph Package, Users Manual,* Hydrologic Engineering Center, Davis, California, 32 p. plus appendices.

U.S.D.A. Soil Conservation Service, 1969, *National Engineering Handbook, Section 4, Hydrology,* Washington, D.C.

U.S.D.A. Soil Conservation Service, 1983, *TR-20 Computer Program for Project Formulation, Hydrology*, draft Second Edition, Lanham, Maryland, 302 p.

U.S.D.A. Soil Conservation Service, 1984, *TR*-48 *DAMS2*, *Structure Site Analysis Computer Program,* draft, Washington, D.C.

U.S.D.A. Soil Conservation Service, 1986, *TR-55Urban Hydrology for Small Watersheds,* Second Edition, Washington, D.C.

U.S.D.I. Bureau of Reclamation, 1973, *Design of Small Dams,* Second Edition, Denver, Colorado, 816 p.

U.S.D.I. Bureau of Reclamation, 1987, *Design of Small Dams,* Third Edition, Denver, Colorado, 860 p.

U.S.D.I. Bureau of Reclamation, Flood Section, 1989, *Personal Computer Programs, Estimating Probable Maximum Precipitation and Precipitation Frequency-Duration in the Western United States,* Denver, Colorado, 72 p.

Williams, J.R., 1968, *Runoff Hydrographs from Small Texas Blacklands Watersheds,* ARS-41-143, U.S.D.A. Agricultural Research Service, Riesel, Texas, 24 p.

Williams, J.R., and Hann, R.W., 1973 HYMO: *Problem-Oriented Computer Language for Hydrologic Modeling, UsersManual*, ARS-S-9, U.S.D.A. Agricultural Research Service, Riesel, Texas.

Woolhiser, D.A., Smith, R.E., and Goodrich, D.C., 1990, *KINEROS, A Kinematic Runoff and Erosion Model, Documentation and User Manual,* ARS-77, U.S.D.A. Agricultural Research Service, Tucson, Arizona, 130 p.

22.2 Site Development Requirements

A. It is beneficial to consider the below listed items when beginning to develop a site:

1. Flood zone-May affect finished floor elevation, locations of structures and increase permit requirements

2. Downstream Capacity-may require onsite ponding

3. Offsite flows- in general, are accepted and conveyed through the site

4. Applicable approved drainage reports and plans-provides previous approvals for downstream capacity and ofsite flows

5. Current Topography-accurate depiction of existing conditions

6. Encumbrances-utility corridors and easements may restrict development

7. Water Quality- First flush and construction runoff

8. Zoning Requirements-may impact grading and drainage requirements on a site

22.3 Grading and Erosion Criteria

A. Slope Criteria

Earth slopes shall confirm to the following criteria:

1. For slopes 3.0 feet high or less, maximum slope should not exceed 2:1 (horizontal to vertical)
2. For slopes greater than 3.0 feet high, maximum slope should not exceed 3:1 (horizontal to vertical) unless stabilized from slope failure through City Engineer approved means. Steeper slopes may be approved subject to a geotechnical recommendation and City concurrence.
3. All slopes shall be protected from erosion, especially when subjected to upland flows.

B. Grading near the Property Line