Section 2. HYDROLOGY

A. Preface

Southern Sandoval County Flood Control Authority (SSCAFCA) was created in 1990 (first official day was June 1, 1990) by the New Mexico Legislature with specific responsibilities to address flooding problems in greater Sandoval County. SSCAFCA's goals, Mission Statements and Vision Statement were developed by staff and adopted by the Board. They are listed below to ensure that the letter and intent guide development. With these purposes in mind and the urgency to adopt drainage criteria, SSCAFCA unofficially adopted Chapter 22 of the City of Albuquerque Development Process Manual.

In 2007, in an effort to adopt drainage criteria that is more representative of the desires of the SSCAFCA Board, the Board authorized the Executive Engineer to adapt the City of Albuquerque DPM Chapter 22 to meet its needs and desires. With this authorization, SSCAFCA joined with the City of Rio Rancho in establishing drainage criteria that is mutually agreeable to both jurisdictions. SSCAFCA volunteered to take the lead in the creation of Chapter 22 for Southern Sandoval County by establishing a Subcommittee that met weekly. In conjunction with this update, Bohannan-Huston was charged with the task to prepare for adoption changes to the City of Albuquerque DPM and the AMAFCA Sediment and Erosion Design Guide to supplement the work of the Subcommittee and WHPacific and Stantec investigated public domain hydrology models for inclusion in the DPM. The USACE HEC-HMS model was selected and changes prepared to incorporate this public domain model into the document for use in SSCAFCA's jurisdiction.

On July 31, 2009 SSCAFCA adopted the revised Chapter 22 as an allowable procedure for hydrologic analysis and design of flood control structures.

The City of Rio Rancho is in the process of adopting the revised Chapter 22 as an allowable procedure for hydrologic analysis and design of flood control structures.

SSCAFCA and the City of Rio Rancho wish to acknowledge the assistance of the committee members listed below who helped prepare and/or reviewed the document:

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B. Introduction

There have been many methods used in the City of Rio Rancho's and SSCAFCA's jurisdiction to compute runoff volumes, peak flow rates and runoff hydrographs 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 Rio Rancho and SSCAFCA staff in the review and evaluation of development plans and drainage management plans.

Three basic methods of analysis are presented herein:

- **Rational Method -** describes a simplified procedure for smaller watersheds based on the Rational Method. The procedure is applicable to watersheds up to 40 acres in size.

- **Rainfall-runoff modeling with AHYMO -** describes procedures for rainfall-runoff modeling using the AHYMO computer program. AHYMO is a version of the U.S.D.A. Agricultural Research Service HYMO computer program, modified to utilize initial abstraction/uniform infiltration precipitation losses. Rainfall-runoff modeling using AHYMO is applicable for drainage areas between 40 and 320 acres in size.
- **Rainfall-runoff modeling with HEC-HMS-** describes procedures for rainfall-runoff modeling using the U.S. Army Corps of Engineers HEC-HMS software. Rainfall-runoff modeling using HEC-HMS is applicable for drainage areas greater than 40 acres in size.

C. Symbols and Definitions

When evaluating 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 AHYMO 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)

R.....storage coefficient (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

√.....square root operator

DEFINITIONS

100-year Design Storm **–** means a storm as defined by the Drainage Ordinance and DPM.

ADA **–** Americans with Disabilities Act.

Amendment **–** Change to an effective FEMA map resulting in the exclusion of an individual structure or a legally described parcel of undeveloped land that was inadvertently included in the SFHA.

Amenities **–** Improvements that may enhance the citizens' enjoyment of the outdoors including, but not limited to, trails, view points with benches, wildlife and plant habitat, educational/informational signage, and trailheads.

Applicant **–** means any Developer seeking to construct drainage facilities under this Procedure.
Base Flood Elevation (BFE) **-** Elevation of the 100-year (1-percent annual chance) flood, in feet, referenced to the National Geodetic Vertical Datum.

Benefit **–** means, for the purpose of this Procedure, the provision of a drainage outfall or flood control facility that serves the property.

Benefited Area **–** means the tracts or parcels of land within a drainage basin benefited by the proposed drainage or flood control facilities.

BMP **–** Best Management Practice.

Certificate of Completion and Acceptance **–** means a document issued by the City/SSCAFCA in a format prescribed in the Development Process Manual which certifies that the public infrastructure improvements required for a development have been satisfactorily completed by the developer and are accepted by the City, for maintenance and public use.

City/County Floodplain Administrator **–** Public official who is designated by the community to coordinate the community's participation in the National Flood Insurance Program.

Consulting Engineer **–** means a professional engineer competent in surface water hydrology and hydraulics duly licensed under the laws of the State of New Mexico who is under contract with an Applicant or the City/SSCAFCA to design drainage facilities.

Cost Allocation **–** means a cost allocated to new development in order to fund and/or recoup the costs of drainage facilities necessitated by and attributable to the new development.

Cost Allocation Table **–** means the list or roll of all tracts or parcels of property within the benefited area and the amount to be allocated against each tract or parcel as determined in accordance with this Procedure.

Dams – Storm water retention/detention structures approved and controlled by the Office of the State Engineer (i.e., containing a storage volume equal to or greater than 50 acre feet and/or a berm height of 25' or greater).

Depth of Bury – the vertical distance between the top of the utility line and the bottom of the arroyo, whether the utility is in the arroyo or adjacent to the arroyo, at the time of consideration.

Developer – means any individual, estate, trust, receiver, cooperative association, club, corporation, company, firm, partnership, joint venture, syndicate, political subdivision or other public or private entity engaging in the platting, subdivision, filling, grading, excavating, or construction of structures.

DEVEX – the runoff with existing platting, full development, unpaved streets, and drainage conveyance. If available, DEVEX flows shall be taken from SSCAFCA approved WMP's.

DPM – Development Process Manual.

Drainage Basin – means the land area from which storm water shall drain to an acceptable outfall.

Drainage Facilities – means public facilities used for conducting storm waters to, through and from a drainage basin to the point of final destination, and any related improvements, as defined in the Allocation Plan including, but not limited to, any or all of the following: bridges, pipes, conduits, culverts, crossing structures, arroyos, waterways, inlets, swales, ditches, gulches, channels, temporary or permanent retention and detention areas, water quality features, lateral erosion line and stability measures removal and/or replacement of existing facilities, as well as easements and rights-of-way necessary to accommodate the same.

Encroachment – Construction, placement of fill, or similar alteration of topography in the flood plain that reduces the area available to convey flood waters.

Federal Emergency Management Agency (FEMA) – Government Agency that regulates FIRM maps.

Floodway – Channel of a stream or other watercourse, plus any adjacent flood plain areas that must be kept free of encroachment so that the 100-year flood discharge can be conveyed without cumulatively increasing the elevation of the 100-year flood more than zero feet.

Floodway Fringe – Portion of the 100-year flood plain that is not within the floodway and in which development and other forms of encroachment are allowed.

Flood Boundary and Floodway Map (FBFM) – Flood plain management map issued by FEMA that depicts, based on detailed analyses, the boundaries of the 100- and 500-year floods and the limits of the 100-year floodway. Replaced by FIRM.

Flood Insurance Rate Map (FIRM) **–** Insurance and flood plain management map issued by FEMA that, based on detailed analyses, identifies areas of 100-year flood hazard in a community. Also shown are BFEs, actuarial insurance rate zones, delineations of the 100- and 500-year flood boundaries, and, on some FIRMS, the 100-year floodway. The Flood Insurance Rate Map enables the community to enter the Regulatory Phase of the National Flood Insurance Program.

Flood Plain **–** Any land area susceptible to being inundated by water from any source, or areas adjacent to a watercourse or other body of water that are subject to inundation by flood waters.

Gross Pollutants **–** litter, vegetation, coarse sediment and floatable debris. For the local Municipal Separate Storm Sewer System (MS4), the gross pollutant treatment size is defined as 1-3/4" and larger.

HDPE **–** High Density Polyethylene.

Infrastructure Allocation Drainage Management Plan **or** ***Allocation Plan*** **–** means a comprehensive analysis of the discharge rate volume, frequency, and course of stormwaters within one or more drainage basins or watershed resulting from a new development and used to identify required drainage facilities so that an equitable cost distribution for drainage facilities may be allocated against benefited properties. The Allocation Plan shall be prepared in accordance with this Procedure.

LEE **–** Lateral Erosion Envelope.

Letter of Map Amendment (LOMA) **–** Official determination by FEMA that a specific structure or portion of a property is not within a 100-year flood zone; amends the effective FIRM map.

Letter of Map Revision (LOMR) **–** Official determination by FEMA that revises Base Flood Elevations, flood insurance rate zones, flood boundaries, or floodways as shown on an effective FIRM map.

National Flood Insurance Program (NFIP) **–** Federal regulatory program under which flood-prone areas are identified and flood insurance is provided to the owners of property in flood-prone areas.

New Development **–** means the proposed subdivision of land, reconstruction, redevelopment, conversion, structural alteration, relocation or enlargement of any structure; or any proposed use or extension of the use of land affecting drainage within the benefited area, including but not limited to proposed buildings or other structures, site plan requests, grading, paving, filling, or excavation.

NPDES **–** National Pollutant Discharge Elimination System.

Open Space **–** means publicly owned or controlled lands set aside for Open Space purposes.

Ponds **–** Smaller storm water retention/detention structures not approved or controlled by the Office of the State Engineer (i.e., containing a storage volume less than 50 acre feet and/or a berm height of less than 25'.

Revision **–** Change to any of the information that is depicted on an effective NFIP map, which is accomplished by a LOMR or by a Physical map revision.

SAS ECZ **–** Sanitary Sewer Line Erosion Control Zone, the Depth of Scour for the 100-year DEVEX event.

Scour Depth **–** Cumulative scour depth including consideration of contraction scour and local scour as defined in Sections 3.4 and 3.5 of the Sediment and Erosion Design Guide.

Special Flood Hazard Area (SFHA) **–** Area inundated by the base (100-year) flood, which carries any of several A or V zone designations.

SSCAFCA **–** Southern Sandoval County Arroyo Flood Control Authority.

Storm Water Quality Constituents **–** dissolved and suspended nutrients, metals, oils, greases, biological agents, etc.

Storm Water Quality Treatment Rate (SWQR) **–** the peak rate of flow from the water quality storm event.

Storm Water Quality Treatment Volume (SWQV) **–** the treatment volume from the water quality storm event.

Temporary Drainage Facility **–** means a nonpermanent drainage control, flood control or erosion control facility constructed as part of a phased project or to serve until such time as a permanent facility is in place, including, but not limited to, desilting ponds, berms, diversions, channels, detention ponds, bank protection and channel stabilization measures.

Water Quality Storm Event **–** 0.6 inches of precipitation within a six-hour period. This is approximately equivalent to the average annual precipitation event and represents the 80th percentile rainfall event (i.e., approximately 80% of the total annual rainfall occurs in storm events with 0.6" or smaller precipitation depth).

Watershed Park **–** A comprehensive, connected system of joint use amenities along the arroyos in Southern Sandoval County.

Witness Post **–** A post identifying the location and depth of the utility that will remain in its location through a storm event.

WMP's **–** Watershed Management Plans.

D. Rational Method

D.1 INTRODUCTION

The Rational Method formula is a commonly used, simplified method of estimating peak discharge for small uniform drainage areas. This method is typically used to size drainage structures for the peak discharge of a given return period. Extensions of this method can be used to estimate runoff volume and the shape of the runoff hydrograph to design drainage facilities and / or design a drainage structure that requires routing of the hydrograph through the structure.

The Rational Equation is expressed as follows:

$$Q = CiA \tag{D-1}$$

where:
- Q = maximum rate of runoff, in cfs
- C = runoff coefficient
- i = average rainfall intensity, in inches / hour
- A = drainage area, in acres

D.2 ASSUMPTIONS

The following assumptions are inherent when using the Rational Equation:

1. The peak flow occurs when the entire watershed is contributing to the flow,
2. The rainfall intensity is the same over the entire watershed,
3. The rainfall intensity is uniform over a duration equal to the time of concentration, and
4. The frequency of the computed peak flow is the same as that of the rainfall intensity (e.g. the 25-year rainfall intensity is assumed to produce the 25-year peak flow).

D.3 LIMITATIONS

The following limitations shall apply to the Rational Method for use in the SSCAFCA jurisdiction. Drainage areas that do not meet the following conditions will require the use of an appropriate rainfall-runoff method as outlined in Sections E or F.

1. The total drainage area cannot exceed 40 acres in size,
2. The land treatment within the contributing watershed must be fairly consistent over the entire drainage area and uniformly distributed throughout the area, and
3. The contributing drainage area cannot have drainage structures or other facilities upstream of the point of interest that require flood routing.

D.4 RUNOFF COEFFICIENTS

Perhaps the most important variable in the Rational Method equation is the runoff coefficient. The runoff coefficient represents the fraction of rainfall that appears as surface runoff from a watershed. Thus, the runoff coefficient is, by default, also a measure of the fraction of rainfall lost to depression storage, infiltration and evaporation with infiltration being the primary loss component. This fraction is largely independent of rainfall intensity or volume from impervious areas. However, for pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of runoff. Therefore, the selection of a runoff coefficient that is appropriate for the storm, soil type, land cover and land use conditions is critical.

Runoff coefficients are based on a characterization of the watershed area into land treatment classifications. Four land treatment classifications have been created that typify the conditions in the SSCAFCA jurisdiction. Descriptions of the land treatment classifications are provided in Table D-1. Three of the land treatment classifications (A, B and C) are for pervious conditions. The forth classification (D) is for impervious areas. Runoff coefficients for each land treatment type are listed in Table D-2.

TABLE D-1. 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 mixture of land treatments. To determine proportional treatments, measure respective subareas. In lieu of specific measurement for treatment D, the areal percentages in TABLE D-3 may be employed.	

For watersheds with multiple land treatment types present, an area averaged runoff coefficient should be used as input to Equation D-1. The area average can be a simple arithmetic average, as seen in the equation below.

$$C = \frac{A_A C_A + A_B C_B + A_C C_C + A_D C_D}{A_A + A_B + A_C + A_D}$$

TABLE D-2. RATIONAL METHOD RUNOFF COEFFICIENT, C				
Recurrence Interval	**Land Treatment**			
Years	**A**	**B**	**C**	**D**
500	0.56	0.62	0.66	0.93
100	0.27	0.43	0.61	0.93
50	0.20	0.35	0.58	0.93
25	0.14	0.31	0.56	0.92
10	0.08	0.24	0.47	0.92
5	0.01	0.10	0.40	0.92
2	0.00	0.02	0.26	0.92
1	0.00	0.00	0.06	0.90

TABLE D-3 SSCAFCA TREATMENT TYPE PERCENTAGE SUMMARY

Parcel Description	Treatments				Methodology/Notes
	A	B	C	D	
1/8 Acre	0%	15%	15%	70%	DPM, Chapter 22.2, Table A-4 for D
1/6 Acre	0%	28%	15%	57%	Northern Meadows Master Plan
1/4 Acre	0%	30%	28%	42%	DPM, and followed SSCAFCA lead on B&C
1/2 Acre	10%	33%	30%	27%	SSCAFCA
1 Acre	43%	20%	20%	17%	SSCAFCA
Single Family Residential N=units/acre, N6					7*√((N*N) + (5*N))
Estate Lots (btwn 1-5ac)	60%	15%	15%	10%	DPM for 2.5 acre lot
M-1 (Light Industrial)	0%	15%	15%	70%	DPM for D, split B & C
Vacant Res./Undevel.	79%	8%	8%	5%	DPM for 5 acre lot
Arroyo	100%	0%	0%	0%	DPM
Major Roads	0%	0%	10%	90%	DPM
School	10%	20%	20%	50%	DPM
Commercial/Industrial	0%	0%	15%	85%	DPM average of Heavy Industrial and Commercial
Open Space	100%	0%	0%	0%	DPM
Parks, Sports and Rec	0%	85%	0%	15%	DPM
Landfill	0%	0%	100%	0%	All disturbed ground
Multi-Family	0%	15%	15%	70%	DPM-Multiple Unit Res. Attached
Northern Meadows	0%	28%	15%	57%	Northern Meadows Master Plan
Drainage Ponds	0%	0%	100%	0%	
County Platted (1)	18.7%	29.5%	27.0%	24.8%	(used Basin P12_104 as typical)
County Unplatted (2)	95%	5%	0%	0%	DPM

NOTES

1. County Platted area is defined as the area between CORR boundary and Rio Rancho Estates boundary.
2. County Unplatted area is defined as the area outside the city limits and the Rio Rancho Estates limits. It is considered to be existing conditions.
3. All roads are assumed to be paved.

D.5 TIME OF CONCENTRATION

Time of concentration is defined as the 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 (concentration point). The units for time of concentration are time, in hours. This implies that the time of concentration flow path may not be the longest physical length, but the length that results in the longest time.

Time of concentration is calculated using the SCS Upland Method. The Upland Method is the summation of flow travel time for the series of unique flow characteristics that occur along the overall basin flow path length. The Upland Method travel time equation is:

$$T_c = \sum_{i=1}^{n} \left(\frac{L_i}{36{,}000 * K_i * \sqrt{S_i}} \right) \qquad \text{(D-2)}$$

Where: T_c = Time of concentration, in hours
L_i = Length of each unique surface flow conveyance condition, in feet
K_i = Conveyance factor from Table D-4
S_i = Slope of the flow path, in feet per foot

TABLE D-4. 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.

D.6 INTENSITY

Rainfall intensity, i, in Equation D-1 is estimated in inches/hour for the specified recurrence interval. The rainfall intensity is uniform over a duration equal to the time of concentration for the drainage area.

For most drainage areas less than or equal to 40 acres in size, it can be assumed that the time of concentration for drainage areas up to 40 acres in size will not exceed 15-minutes. Rainfall intensities for time of a time of concentration of 15-minutes are listed in Table D-5. Rainfall intensities listed in Table D-5 are based on precipitation values for the SSCAFCA jurisdiction derived from NOAA Atlas 14, Precipitation - Frequency Atlas of the United States, Volume 1: Semiarid Southwest (Arizona, Southeast California, Nevada, New Mexico, Utah).

TABLE D-5. RAINFALL INTENSITY

Recurrence Interval Years	Intensity in/hr
500	5.7
100	4.4
50	3.9
25	3.4
10	2.8
5	2.3
2	1.7
1	1.4

D.7 RUNOFF VOLUME

Runoff volumes for drainage areas less than or equal to 40 acres in size can be estimated using a modified form of the Rational Method Equation. That equation is as follows.

$$V = C\frac{P}{12}A \qquad \text{(D-3)}$$

where: V = runoff volume, in acre-feet

C = weighted runoff coefficient derived from Table D-2

P = rainfall depth, in inches from Table D-6

A = drainage area, in acres

Rainfall depths for Equation D-3 are listed in Table D-6. The rainfall depths provided in Table D-6 are for multiple recurrence intervals and storm durations. Those values are adapted from NOAA Atlas 14, Precipitation - Frequency Atlas of the United States, Volume 1: Semiarid Southwest (Arizona, Southeast California, Nevada, New Mexico, Utah). For all other recurrence intervals and / or storm durations, point precipitation depths are to be obtained directly from the National Weather Service through the NOAA 14 Precipitation Frequency Data Server web site

found at http://hdsc.nws.noaa.gov/hdsc/pfds/sa/nm_pfds.html. At this web site point precipitation values for frequencies up to 1,000 years and duration up to 60 days can be obtained by entering the latitude and longitude of the watershed of interest.

TABLE D-6. RECURRENCE INTERVAL POINT PRECIPITATION DEPTHS				
Recurrence Interval	**Duration**			
Years	**15-Minute**	**1-Hour**	**6-Hour**	**24-Hour**
500	1.42	2.37	3.01	3.57
100	1.10	1.84	2.37	2.90
50	0.97	1.62	2.11	2.57
25	0.85	1.42	1.86	2.29
10	0.70	1.16	1.54	1.90
5	0.58	0.97	1.31	1.66
2	0.43	0.72	1.02	1.32
1	0.34	0.56	0.81	1.05

D.8 RUNOFF HYDROGRAPH

A runoff hydrograph can be synthesized for drainage areas less than or equal to 40 acres based on the Rational Method. This procedure is to be used where routing of the storm inflow through a drainage structure is desired, such as for the design of a detention basin. The procedure is based on an idealized hydrograph shape, drainage area time of concentration and the Rational Method peak discharge. The shape of the hydrograph is shown in Figure D-1. Equations for deriving the runoff hydrograph shape are as follows:

$$t_B = \left(2.017\frac{C * P * A_T}{Q_P}\right) - \left(0.25\frac{A_D}{A_T}\right) \qquad \text{(D-4)}$$

where: t_B = time base, in hours

C = runoff coefficient from Table D-2

P = rainfall depth, in inches from Table D-6

Q_P = Rational Method peak discharge, in cfs

A_D = area in land treatment type D, in acres

A_T = drainage area, in acres

$$t_p = 0.7 * T_c + \frac{1.6 - A_D / A_T}{12} \qquad \text{(D-5)}$$

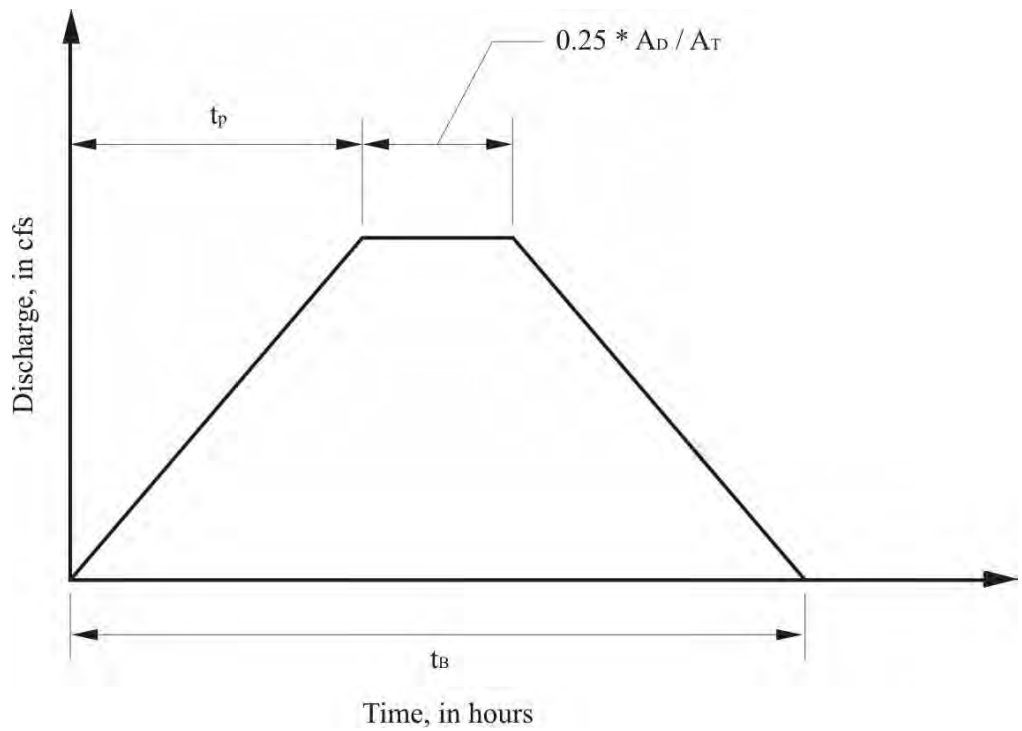
where: t_p = time to peak in hours

T_c = time of concentration from Eqn. D-2, in hours

A_D = area in land treatment type D, in acres

A_T = drainage area, in acres

FIGURE D-1. RATIONAL METHOD RUNOFF HYDROGRAPH SHAPE



D.9 PROCEDURE

A runoff hydrograph can be synthesized for drainage areas less than or equal to 40 acres based on the Rational Method. This procedure is to be used where routing of the storm inflow through a drainage structure is desired, such as for the design of a detention basin.

To estimate peak discharge,

1. Determine the drainage area for the point of interest.
2. Calculate the area of each unique land treatment type or zoning classification.

3. Using the percent area of each land treatment type, calculate the area averaged runoff coefficient using the data from Table D-2.
4. For the desired frequency, select the maximum intensity from Table D-5.
5. Calculate the peak discharge using Equation D-1:

To estimate runoff volume,

1. For the desired storm frequency and duration, select the rainfall depth from Table D-6
2. Calculate the runoff coefficient using the procedures for estimating peak discharge
3. Calculate runoff volume using Equation D-3

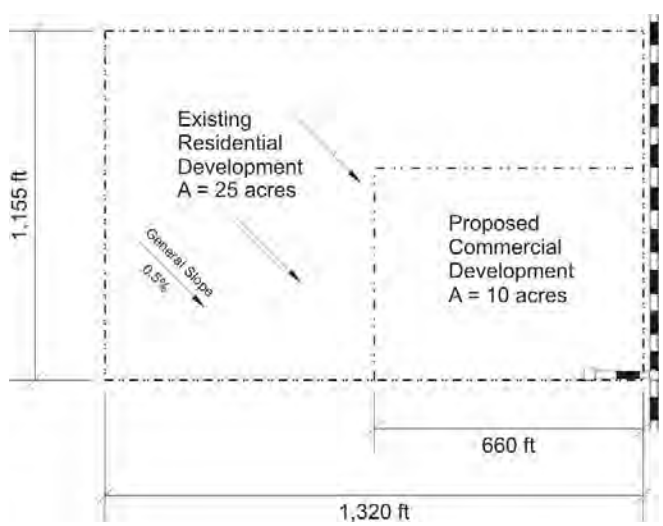
To estimate the Rational Method runoff hydrograph,

1. Calculate the peak discharge using the above procedures
2. From an appropriate map of the drainage area, delineate the time of concentration flow path and measure the length, in feet.
3. Select K from Table D-4
4. Measure the average flow path slope, S
5. Calculate the time of concentration using Equation D-2
6. Calculate the time base of the runoff hydrograph using Equation D-4
7. Calculate the time to peak using Equation D-5
8. Construct the hydrograph starting at time = 0 hours with a discharge of 0 cfs.

D.10 EXAMPLE

Runoff from an existing residential development collects at a roadway intersection. A new storm drain lateral is to be constructed as part of a proposed commercial development (see the following figure). Calculate the following:

1. 10-year peak discharge for the storm drain lateral.
2. Storage volume necessary to temporarily store the entire runoff volume from the 100-year, 6-hour storm.
3. Compute a runoff hydrograph for design of a detention basin to meter the 100-year flow into the storm drain.



Peak Discharge

1. Calculate the weighted runoff coefficient

   From Table D-3, Land Treatment Type percentages for the two parcel descriptions are:

Parcel Description	Area acres	Percent of Land Treatment Type A	B	C	D
1/8 Acre	25	0	15	15	70
Commercial / Industrial	10	0	0	15	85

   From Table D-2, runoff coefficients for a 10-year frequency storm are:

   - $C_B = 0.24$
   - $C_C = 0.47$
   - $C_D = 0.92$

   Area of each Land Treatment Type is calculated as:

   - $Area_B = (0.15)(25) + (0)(10) =$ 3.75 acres
   - $Area_C = (0.15)(25) + (0.15)(10) =$ 5.25 acres
   - $Area_D = (0.70)(25) + (0.80)(10) =$ 26.0 acres

   Total Area = 35.0 acres

   Weighted runoff coefficient (C) is:

$$C = \frac{(3.75)(0.24) + (5.25)(0.47) + (26)(0.92)}{35}$$

$$C = 0.78$$

2. From Table D-5, the rainfall intensity (assuming $T_c \leq 15$ minutes) = 2.8 in/hr

3. Calculate the peak discharge using Equation D-1

$$Q = CiA$$

$$Q = (0.78)(2.8)(35)$$

$$Q = 77\ cfs$$

Note: It is recommended that all flow rates be rounded up to the nearest single unit (e.g. 76.44 cfs is rounded to 77 cfs).

100-Year, 6-hour Runoff Volume

1. From Table D-6, 100-year, 6-hour rainfall depth = 2.37 inches

2. Calculate the weighted runoff coefficient for the 100-year event

   From Table D-3, Land Treatment Type percentages for the two parcel descriptions are:

Parcel Description	Area acres	Percent of Land Treatment Type A	B	C	D
1/8 Acre	25	0	15	15	70
Commercial / Industrial	10	0	0	15	85

   From Table D-2, runoff coefficients for a 100-year frequency storm are:

   - $C_B = 0.43$
   - $C_C = 0.61$
   - $C_D = 0.93$

   Area of each Land Treatment Type is calculated as:

   - $Area_B = (0.15)(25) + (0)(10) =$ 3.75 acres
   - $Area_C = (0.15)(25) + (0.15)(10) =$ 5.25 acres
   - $Area_D = (0.70)(25) + (0.85)(10) =$ 26.0 acres

   Total Area = 35.0 acres

   Weighted runoff coefficient (C) is:

$$C = \frac{(3.75)(0.43) + (5.25)(0.61) + (26)(0.93)}{35}$$

$$C = 0.83$$

3. Calculate the runoff volume using Equation D-3

$$V = C\frac{P}{12}A$$

$$V = (0.83)\left(\frac{2.37}{12}\right)(35)$$

$$V = 5.7 \textit{ acre-feet}$$

Runoff Hydrograph

1. Calculate Time of Concentration assuming a total length of 2,475 feet (1,155 + 1,320) and a channel will be constructed to convey runoff along the boundary of the commercial development to the storm drain inlet.

   From Table D-4, select conveyance factors for each conveyance condition

   A. K_1 = 2 (Shallow concentrated flow within residential area)
   B. K_2 = 3 (Street flow, storm sewers and open channels for commercial area)

   From Equation D-2, T_c is:

$$T_c = \sum_{i=1}^{n} \left(\frac{L_i}{36{,}000 * K_i * \sqrt{S_i}} \right)$$

$$T_c = \left[\frac{(1{,}155 + 660)}{(36{,}000)(2)\sqrt{0.005}} + \frac{660}{(36{,}000)(3)\sqrt{0.005}} \right]$$

$$T_c = (0.36 + 0.09)$$

T_c = 0.45 hours (27 minutes)

   Note: assumption of a 15 minute T_c for estimating the 10-year peak discharge is reasonable and conservative based on the 100-year T_c of 27 minutes.

2. Calculate the 100-year peak discharge using Equation D-1 and an intensity of 4.4 in/hr taken from Table D-5

$Q = CiA$

Q = (0.83)(4.4)(35)

Q = 128 *cfs*

3. Calculate the shape of the runoff hydrograph time base using Equation D-4 and time to peak using Equation D-5

$$t_B = \left(2.017 \frac{C * P * A_T}{Q_P} \right) - \left(0.25 \frac{A_D}{A_T} \right)$$

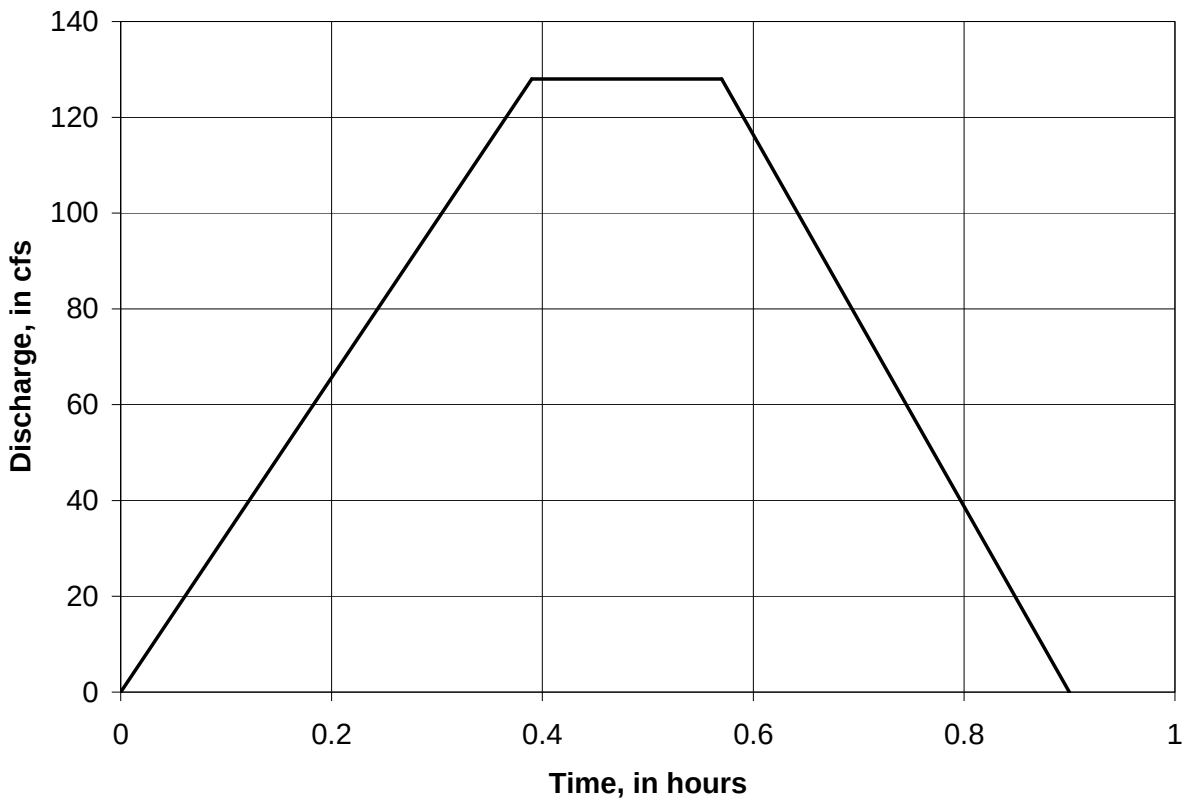
$$t_B = \left(2.017 \frac{(0.83)(2.37)(35)}{128} \right) - (0.25)\left(\frac{26}{35} \right)$$

$t_B = 0.90$ *hours*

$$t_p = 0.7 * T_c + \frac{1.6 - A_D / A_T}{12}$$

$$t_p = 0.7\left(\frac{27}{60}\right) + \frac{1.6 - \left(26/35\right)}{12}$$

$t_p = 0.39$ *hours*



E. Rainfall-Runoff Modeling: AHYMO

E.1 INTRODUCTION

Rainfall-runoff modeling for drainage areas greater than 40 acres and less than 320 acres in size may be conducted using the AHYMO computer program. AHYMO is an arid lands hydrologic model based on the HYMO computer program. The HYMO program was developed by Jimmy R. Williams and Roy W. Hann, Jr. in the early 1970s for the USDA Agricultural Research Service in cooperation with the Texas Agricultural Experiment Station, Texas A&M University. During the 1980s, HYMO was reformulated, enhanced and renamed to AHYMO by Cliff Anderson to simulate rainfall-runoff processes characteristic of the Albuquerque area. The current version of the program was issued in 1997.

Rainfall-runoff methodologies encoded into AHYMO are described in the following sections. In addition, techniques and procedures for developing the necessary input to AHYMO are discussed in the following sections.

E.2 DESIGN RAINFALL CRITERIA

For design hydrology, the characteristics of the major flood producing storm are simulated using a synthetic storm. Components of a synthetic storm are basin average rainfall depth and temporal distribution. Information and procedures for developing the design rainfall criteria for storms other than the Probable Maximum Precipitation are provided in the following sections.

E.2.1 Depth

The principal design storm for peak flow determination is the 100-year, 6-hour event. For analysis and design of retention ponds and detention dams, the 100-year, 24-hour storm is to be used. Additional design analysis may be required if the structure falls under the jurisdiction of the New Mexico Office of State Engineer, Dam Safety Bureau. Point precipitation depths for the 100-year storm to be used within the SSCAFCA jurisdiction are provided in Table E-1. Those values are adapted from NOAA Atlas 14, Precipitation - Frequency Atlas of the United States, Volume 1: Semiarid Southwest (Arizona, Southeast California, Nevada, New Mexico, Utah).

For determining sediment transport and for analysis of watersheds with complex routing conditions, other storm frequencies and durations may be required. Point precipitation depths for use in the SSCAFCA jurisdiction for multiple recurrence intervals and storm durations are listed in Table E-1. For all other recurrence intervals and storm durations, point precipitation depths are to be obtained directly from the National Weather Service through the NOAA 14 Precipitation Frequency Data Server website found at http://hdsc.nws.noaa.gov/hdsc/pfds/sa/nm_pfds.html. At this web site point precipitation values for frequencies up to 1,000 years and duration up to 60 days can be obtained by entering the latitude and longitude of the watershed of interest.

TABLE E-1. RECURRENCE INTERVAL POINT PRECIPITATION DEPTHS				
Recurrence Interval	**Duration**			
Years	**15-Minute**	**1-Hour**	**6-Hour**	**24-Hour**
500	1.42	2.37	3.01	3.57
100	1.10	1.84	2.37	2.90
50	0.97	1.62	2.11	2.57
25	0.85	1.42	1.86	2.29
10	0.70	1.16	1.54	1.90
5	0.58	0.97	1.31	1.66
2	0.43	0.72	1.02	1.32
1	0.34	0.56	0.81	1.05

E.2.2 Temporal Distribution

Basin average rainfall for 100-year, 6- and 24-hour storms is distributed temporally using a suite of equations; E-1 through E-6. The equations are a function of the 1-, 6- and 24-hour basin average depths. The design rainfall distribution is front loaded with the peak intensity set at 85.3 minutes (hour 1.42) regardless of storm duration. This distribution results in approximately 80 percent of the total depth occurring in less than one hour. For the 6-hour storm the distribution of rainfall is determined using the first 5 of the 6 equations. For the 24-hour storm, all 6 equations are used. To illustrate the shape of the pattern, the 6-hour storm distribution using the depths from Table E-1 for a 20 square mile watershed is shown in Figure E-2.

$$P_T = 2.334 * \left(P_{360} - P_{60}\right) * \left(1.5^A - \left(1.5 - \frac{t}{60}\right)^A\right) \qquad \text{For } 0 \le t \le 60 \quad \text{(E-1)}$$

$$P_T = P_{T=60} + P_{60} * 0.4754 * \left(0.5^{0.09} - \left(1.5 - \frac{t}{60}\right)^{0.09}\right) \qquad \text{For } 60 < t < 67 \quad \text{(E-2)}$$

$$P_T = P_{T=60} + P_{60} * \left(0.0001818182 * (t - 60) + 0.000018338 * (t - 60)^{3.2}\right) \qquad \text{For } 67 \le t < 85.3 \quad \text{(E-3)}$$

$$P_T = P_{T=60} + P_{60} * \left(0.07 * (t - 60) - 1.1886 - 0.0404768 * (t - 85)^{1.0985865}\right) \qquad \text{For } 85.3 \le t < 120 \quad \text{(E-4)}$$

$$P_T = P_{360} + \left(P_{T=60} + P_{60} - P_{360}\right)\frac{4.4^{3A} - \left(t/60 - 1.6\right)^{3A}}{4.4^{3A} - 0.4^{3A}}$$ For $120 \le t \le 360$ (E-5)

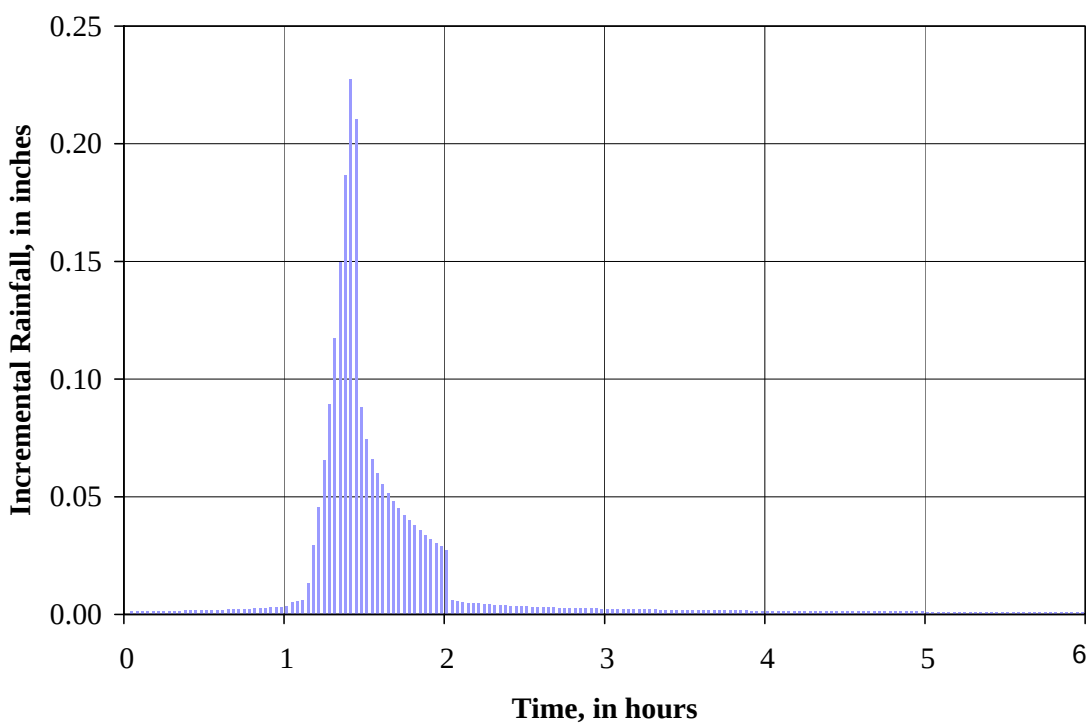
$$P_T = P_{1440} + \left(P_{360} - P_{1440}\right) * \frac{30^{B} - \left(t/60 + 6\right)^{B}}{30^{B} - 12^{B}}$$ For $360 < t < 1440$ (E-6)

Where: $$A = \frac{Log\left(P_{360}/P_{60}\right)}{Log(6.0)}$$

$$B = \frac{Log\left(P_{1440}/P_{360}\right)}{Log(4.0)}$$

These equations are implemented in the AHYMO program by specifying P_{60}, P_{360}, and P_{1440} with the RAINFALL command. See the AHYMO users manual for additional information at www.ahymo.com.

FIGURE E-1. 100-YR 6-HR RAINFALL HYETOGRAPH



E.2.3 Procedure

A. For design events up to the 500-year and storm durations up to the 24-hour
    1. Select the point rainfall depths from Table E-1
    2. In AHYMO
        a. For a 6-hour storm code the RAINFALL command with the following:
            1. Distribution type = 1
            2. 1-hour rainfall depth from Step 1
            3. 6-hour rainfall depth from Step 1
            4. Incremental time, DT, of 0.033333 hours
        b. For a 24-hour storm code the RAINFALL command with the following:
            1. Distribution type = 2
            2. 1-hour adjusted rainfall depth from Step 1
            3. 6-hour adjusted rainfall depth from Step 1
            4. 24-hour adjusted rainfall depth from Step 1
            5. Incremental time, DT, of 0.05 hours

B. For design storms with durations other than 6- or 24-hours, submit to SSCAFCA/City Engineer in writing a recommendation for depth-area reduction and time distribution of the rainfall for the selected storm event.

E.3 RAINFALL LOSS

Rainfall losses are generally considered to be the result of evaporation of water from the land surface, interception of rainfall by vegetal cover, depression storage on the land surface and the infiltration of water into the soil matrix. The magnitude of rainfall loss is typically expressed as an equivalent uniform depth in inches. By a mass balance, rainfall minus losses equals rainfall excess. Estimation of rainfall loss is an important element in flood analyses that must be clearly understood and estimated with care.

E.3.1 Land Treatment

Estimation of rainfall losses are based on a characterization of the watershed area into land treatment classifications. Four land treatment classifications have been created that typify the conditions in the SSCAFCA jurisdiction. Descriptions of the land treatment classifications are provided in Table E-2. Three of the land treatment classifications (A, B and C) are for pervious conditions. The forth classification (D) is for impervious areas.

TABLE E-2. 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. In lieu of specific measurement for treatment D, the areal percentages in Table E-3 may be employed.	

Of the land treatment classifications listed in Table E-2, only treatment type A represents land in its natural, undisturbed state. Land treatment classifications B and C describe conditions that have been impacted by some form of urbanization. Urban areas within a watershed usually contain a mix of the land treatment types. Ideally, the specific area of each land treatment type can be measured from available information. In lieu of specific measurement for each unique land treatment type that occurs within urban areas, generalized percentages based on zoning classifications can be used. Average land treatment type percentages associated with various zoning designations are listed in Table E-3.

TABLE E-3 SSCAFCA TREATMENT TYPE PERCENTAGE SUMMARY

Parcel Description	Treatments				Methodology/Notes
	A	B	C	D	
1/8 Acre	0%	15%	15%	70%	DPM, Chapter 22.2, Table A-4 for D
1/6 Acre	0%	28%	15%	57%	Northern Meadows Master Plan
1/4 Acre	0%	30%	28%	42%	DPM, and followed SSCAFCA lead on B&C
1/2 Acre	10%	33%	30%	27%	SSCAFCA
1 Acre	43%	20%	20%	17%	SSCAFCA
Single Family Residential N=units/acre, N6					7*√((N*N) + (5*N))
Estate Lots (btwn 1-5ac)	60%	15%	15%	10%	DPM for 2.5 acre lot
M-1 (Light Industrial)	0%	15%	15%	70%	DPM for D, split B & C
Vacant Res./Undevel.	79%	8%	8%	5%	DPM for 5 acre lot
Arroyo	100%	0%	0%	0%	DPM
Major Roads	0%	0%	10%	90%	DPM
School	10%	20%	20%	50%	DPM
Commercial/Industrial	0%	0%	15%	85%	DPM average of Heavy Industrial and Commercial
Open Space	100%	0%	0%	0%	DPM
Parks, Sports and Rec	0%	85%	0%	15%	DPM
Landfill	0%	0%	100%	0%	All disturbed ground
Multi-Family	0%	15%	15%	70%	DPM-Multiple Unit Res. Attached
Northern Meadows	0%	28%	15%	57%	Northern Meadows Master Plan
Drainage Ponds	0%	0%	100%	0%	
County Platted (1)	18.7%	29.5%	27.0%	24.8%	(used Basin P12_104 as typical)
County Unplatted (2)	95%	5%	0%	0%	DPM

NOTES

1. County Platted area is defined as the area between CORR boundary and Rio Rancho Estates boundary.
2. County Unplatted area is defined as the area outside the city limits and the Rio Rancho Estates limits. It is considered to be existing conditions.
3. All roads are assumed to be paved.

E.3.2 Initial Abstraction and Infiltration Loss

Simulation of rainfall loss is accomplished using an initial loss coupled with a loss rate. This combined methodology is a two parameter model. The first parameter is the Initial Abstraction (IA). The initial abstraction is the summation of all losses other than infiltration and is applied at the beginning of the storm event. The second parameter is the Infiltration rate (INF) of the soil matrix at saturation. Infiltration losses begin once the initial abstraction is completely satisfied. For pervious conditions, the infiltration rate is constant. For impervious conditions, the infiltration rate is constant up to hour 3 of the design storm. After hour 3 and until hour 6, the infiltration rate is linearly reduced to zero. Beyond hour 6, no infiltration occurs. The constant loss is only applied once the Initial Abstraction is satisfied. An illustration of the application of this method is provided in Figure E-2.

Recommended values for the Initial Abstraction and Infiltration rate are assigned to each land treatment type and are listed in Table E-4. For watersheds and subbasins with multiple, unique land treatment types an arithmetic area averaged value for IA and INF is to be calculated.

FIGURE E-2. REPRESENTATION OF RAINFALL LOSS METHODOLOGY

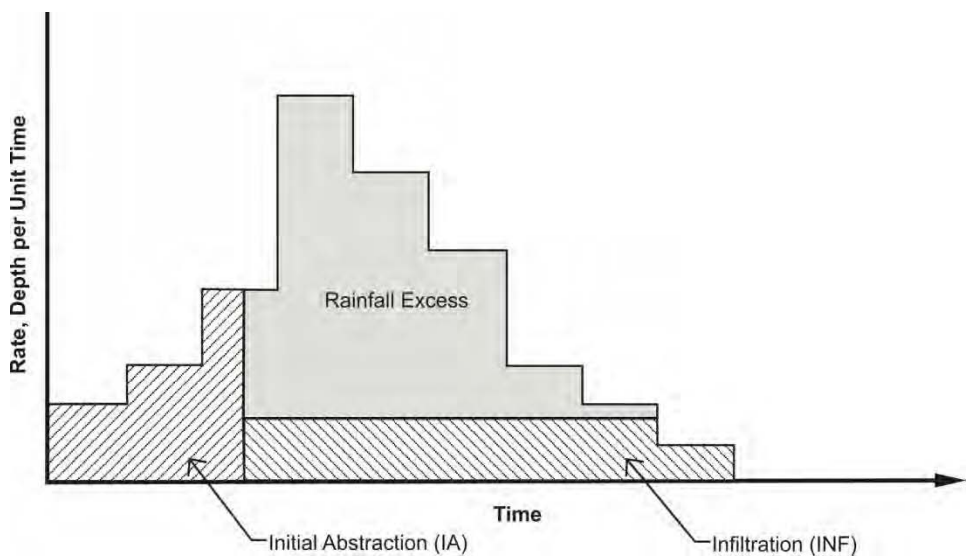


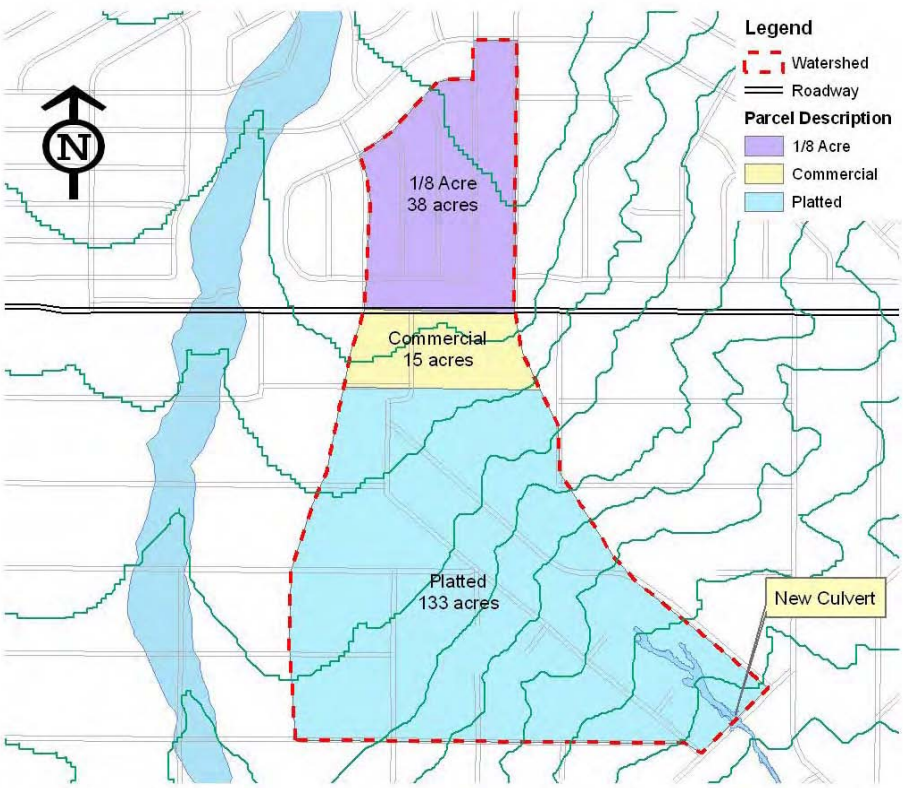
TABLE E-4. INITIAL AND CONSTANT LOSS PARAMETERS		
Land Treatment	**Initial Abstraction (inches)**	**Infiltration (inches/hour)**
A	0.65	1.67
B	0.50	1.25
C	0.35	0.83
D	0.10	0.04

E.3.3 Procedure

1. For each subbasin, calculate the area of each unique land treatment type or zoning classification.
2. Calculate the area weighted percentage of each land treatment type.
3. In AHYMO, for each subbasin code in the percent area of each land treatment type in the COMPUTE NM HYD command.(See AHYMO Users Manual)

E.3.4 Example

A new culvert is to be constructed to convey the 100-year, 6-hour storm at the location shown in the following figure. Compute the rainfall loss parameters for the contributing watershed.



1. From Table E-3, percentage of Land Treatment Types for each parcel within the watershed are:

Parcel Description	Area acres	Percent of Land Treatment Type A	B	C	D
1/8 Acre	38	0	15	15	70
Commercial / Industrial	15	0	0	15	85
Platted	133	18.7	29.5	27.0	24.8

Area of each Land Treatment Type is calculated as:

- $Area_A$ = (0)(38) + (0)(15) + (0.187)(133) = 24.9 acres
- $Area_B$ = (0.15)(38) + (0)(15) + (0.295)(133) = 44.9 acres
- $Area_C$ = (0.15)(38) + (0.15)(15) + (0.27)(133) = 43.9 acres
- $Area_D$ = (0.70)(38) + (0.85)(15) + (0.248)(133) = 72.3 acres

Total Area = 186.0 acres

2. Using values of IA from Table E-4, calculate the weighted value of IA

$$IA = \frac{(24.9)(0.65)+(44.9)(0.50)+(43.9)(0.35)+(72.3)(0.10)}{(24.9+44.9+43.9+72.3)}$$

IA = 0.33 inches

3. Using values of INF from Table E-4, calculate the weighted value of INF

$$INF = \frac{(24.9)(1.67)+(44.9)(1.25)+(43.9)(0.83)+(72.3)(0.04)}{(24.9+44.9+43.9+72.3)}$$

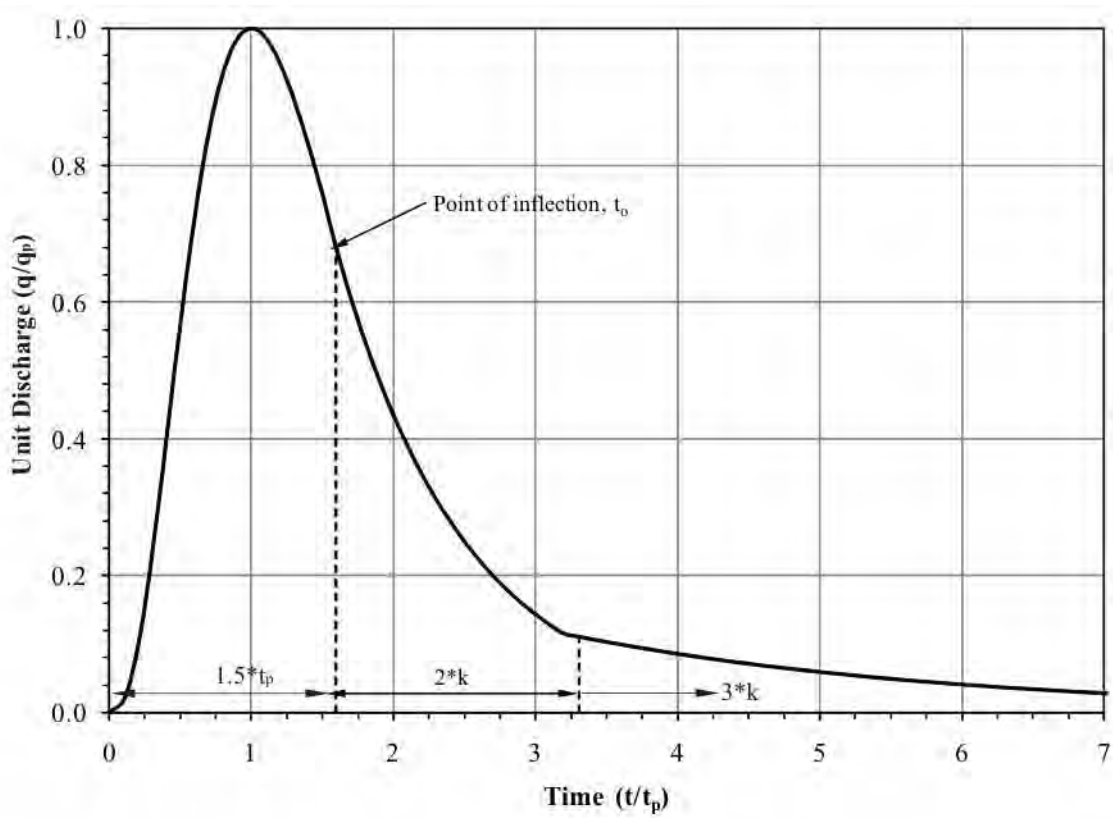
INF = 0.74 in/hr

E.4 UNIT HYDROGRAPH

Rainfall excess generated during a storm event is routed across the basin surface and eventually begins to concentrate at a downstream location (concentration point). The routing process results in the transformation of rainfall excess to a runoff hydrograph. Simulation of rainfall excess transformation is typically accomplished using the concept of a unit hydrograph. A unit hydrograph is defined as the hydrograph of one inch of direct runoff from a storm of a specified duration for a particular basin. Every watershed will have a different unit hydrograph that reflects the topography, land use, and other unique characteristics of the individual watershed. Different unit hydrographs will also be produced for the same watershed for different durations of rainfall excess.

For most watersheds, sufficient data (rainfall and runoff records) does not exist to develop unit hydrographs specific to the watershed. Therefore, indirect methods are used to develop a unit hydrograph. Such unit hydrographs are called synthetic unit hydrographs. The synthetic unit hydrograph encoded in AHYMO is dimensionless and can be defined by two numeric parameters; Time to Peak (t_p) and Recession Constant (k). The shape of the AHYMO dimensionless unit hydrograph is broken into three time segments as illustrated in Figure E-3.

FIGURE E-3. AHYMO DIMENSIONLESS UNIT HYDROGRAPH



E.4.1 Time to Peak

Time to peak is defined as the time from the beginning of unit rainfall excess to the time of the peak flow of the unit runoff hydrograph. It is assumed to be a constant ratio of the time of concentration as given by Equation E-7. Time of concentration (T_c) is defined as the 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 (concentration point). The units for time of concentration are time, in hours. This implies that the time of concentration flow path may not be the longest physical length, but the length that results in the longest time.

$$t_p = \left(\frac{2}{3}\right) * T_c \qquad \text{(E-7)}$$

Time of concentration is calculated using one of three equations. Selection of the appropriate equation is based on the time of concentration flow path length (in time). Regardless of the selected equation, time of concentration should not be less than 12 minutes.

For basins with flow path lengths less than 4,000 feet the SCS Upland Method is used. The Upland Method is the summation of flow travel time for the series of unique flow characteristics that occur along the overall basin flow path length. The Upland Method travel time equation is:

$$T_c = \sum_{i=1}^{n} \left(\frac{L_i}{36{,}000 * K_i * \sqrt{S_i}} \right) \qquad \text{(E-8)}$$

Where: T_c = Time of concentration, in hours
L_i = Length of each unique surface flow conveyance condition, in feet
K_i = Conveyance factor from Table E-5
S_i = Slope of the flow path, in feet per foot

TABLE E-5. 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 basins with flow path lengths longer than 4,000 feet the following equation should be used for calculating time of concentration:

$$T_c = \left(\frac{12{,}000 - L}{72{,}000 * K * \sqrt{S}} + \frac{(L - 4{,}000) * K_n * \left(L_{ca} / L \right)^{0.33}}{552.2 * S^{0.165}} \right) \qquad \text{(E-9)}$$

Where: L = Flow path length, in feet
L_{ca} = Distance along L from point of concentration to a point opposite the centroid of the basin, in feet

K = Conveyance factor from Table E-5
K_n = Basin factor, from Table E-6
S = Slope of flow path, in feet per foot

TABLE E-6. 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)

Calculation of a basin time of concentration is a function of flow path length and, by extension, basin area. Therefore, basin / subbasin delineation is a key consideration that must be addressed early on in the modeling process as it not only influences unit hydrograph parameter estimation but rainfall loss parameters as well. Wherever possible, subbasin delineation should be based on the best available topographic mapping and, if available, detailed aerial photography. For some areas, field investigation may also be necessary to verify subbasin boundaries particularly in urban or distributary areas. The breakdown of a watershed into subbasins should consider the following:

- The subbasin sizes should be as uniform as possible.
- Subbasins should have fairly homogeneous land use and geographic characteristics. For example: mountain, hillslope and valley areas should be delineated separately where possible.
- Soils, vegetation and land treatment characteristics should be fairly homogeneous.
- Subbasins size should be commensurate with the intended use of the model. For example, if the model is to be used for the evaluation and / or design of drainage infrastructure, the subbasin size should be fairly small so that runoff magnitudes are known at multiple locations within the watershed. For drainage management plans, the subbasin size should in general not be greater than 1.5 mi^2 or less than 0.1 mi^2.

E.4.2 Time of Concentration for Steep Slopes and Natural Channels

The equations used to compute time of concentration may result in 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 equations identified in Section E.4.1 can result in 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 not be used for the following conditions:

- Slopes flatter than 0.04 foot per foot.
- Channels with irrigated grass, riprap, soil cement, gabion, or concrete lining which cannot be clearly identified as natural or naturalistic.
- The hydraulic design of channels or channel elements. The purpose of 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:

$$S' = 0.052467 + 0.062627\,S - 0.18197\,e^{-62.375S} \qquad \text{(E-10)}$$

Where: S = Measured slope, in feet per foot
S′ = Adjusted slope, in feet per foot

The conveyance factors (K) for the Upland Method should be checked to make sure that appropriate Froude Numbers are maintained. The Basin Factors, K_n, from Table E-6 remain applicable when using equations E-8 and E-9 with the adjusted slope computed by equation E-10. To adjust the conveyance factor (K) it is necessary to estimate the peak flow rate from the watershed. Using estimated conveyance factors (K) from Table E-5 and the Rational Method procedures outlined in Part D, an estimated peak flow rate for the basin (Q_p) can be computed. The following formulas are then used to compute conveyance factor adjustment:

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

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

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

- if K > K' then K= K'
- if K' ≥ K ≥ K" then K = K (no adjustment)

- if K < K" then K= K"

This is an iterative process that is to be repeated until the computed value of Q_p is within 10 percent of original value of Q_p.

E.4.3 Recession Constant

The recession constant is a function of drainage area, rainfall depth and land cover treatment. A value of k is calculated for each land cover treatment present in the watershed. Two sets of equations are provided for the estimation of k. Selection of the appropriate set is based on basin area.

For drainage basins less than or equal to 40 acres in size, k is calculated separately for each land treatment type using Equations E-13 through E-16. For basins with multiple land cover treatments, an arithmetically area-weighted value is calculated for the pervious areas (land treatment types A, B and C) with a separate calculation for land treatment type D. Regardless of the land treatment type or combinations of land treatment type within the basin, the calculated value of k must be no greater than $1.35t_p$ and no less than $0.545t_p$. The following are equations for calculating land treatment types.

Land Treatment Type A

$$k = \begin{cases} t_p\left(1.58159 - 0.18912P_{60}\right) & \text{For } P_{60} < 2.10 \text{ inches} \\ t_p\left(0.98204 + 0.09638P_{60}\right) & \text{For } P_{60} \geq 2.10 \text{ inches} \end{cases} \qquad \text{(E-13)}$$

Land Treatment Type B

$$k = \begin{cases} t_p\left(1.22953 - 0.132P_{60}\right) & \text{For } P_{60} < 1.89 \text{ inches} \\ t_p\left(0.8090 + 0.0905P_{60}\right) & \text{For } P_{60} \geq 1.89 \text{ inches} \end{cases} \qquad \text{(E-14)}$$

Land Treatment Type C

$$k = \begin{cases} t_p\left(0.90392 - 0.07488P_{60}\right) & \text{For } P_{60} < 1.68 \text{ inches} \\ t_p\left(0.63596 + 0.08462P_{60}\right) & \text{For } P_{60} \geq 1.68 \text{ inches} \end{cases} \qquad \text{(E-15)}$$

Land Treatment Type D

$$k = \begin{cases} 0.5450 t_p & \text{For } P_{60} < 1.33 \text{ inches} \\ t_p\left(0.31048 + 0.7356 P_{60}\right) & \text{For } P_{60} \geq 1.33 \text{ inches} \end{cases} \quad \text{(E-16)}$$

For drainage basins greater than or equal to 200 acres in size, k is calculated separately for each land treatment type using Equations E-17 through E-20. For basins with multiple land cover treatments, an arithmetically area-weighted value is calculated for the pervious areas (land treatment types A, B and C) with a separate calculation for land treatment type D. Regardless of the land treatment type or combinations of land treatment type within the basin, the calculated value of k must not be greater than $1.30t_p$.

Land Treatment Type A

$$k = t_p\left(0.854 + 0.5808 * 4.756828^{1-P_{60}}\right) \quad \text{(E-17)}$$

Land Treatment Type B

$$k = t_p\left(0.770 + 0.480 * 4.756828^{1-P_{60}}\right) \quad \text{(E-18)}$$

Land Treatment Type C

$$k = t_p\left(0.686 + 0.3792 * 4.756828^{1-P_{60}}\right) \quad \text{(E-19)}$$

Land Treatment Type D

$$k = t_p\left(0.528 + 0.1896 * 4.756828^{1-P_{60}}\right) \quad \text{(E-20)}$$

For drainage basins between 40 and 200 acres in size, calculate k using the appropriate equations for drainage area up to 40 acres and for drainage areas greater than or equal to 200 acres in size. The basin specific values of k for pervious and impervious areas are then calculated using linear interpolation.

E.4.4 Procedure

1. From an appropriate map of the watershed, delineate the time of concentration flow path for each subbasin and measure the length, in feet.
    a. If the flow path length is less than 4,000 feet, calculate T_c using Equation E-8 with the following:
        i. Select K from Table E-5
        ii. Measure the average flow path slope, S. If the flow path slope is greater than 0.04 feet / foot:
            1. Calculate the adjusted slope using Equation E-10.
            2. Estimate the peak discharge using procedures in Part D

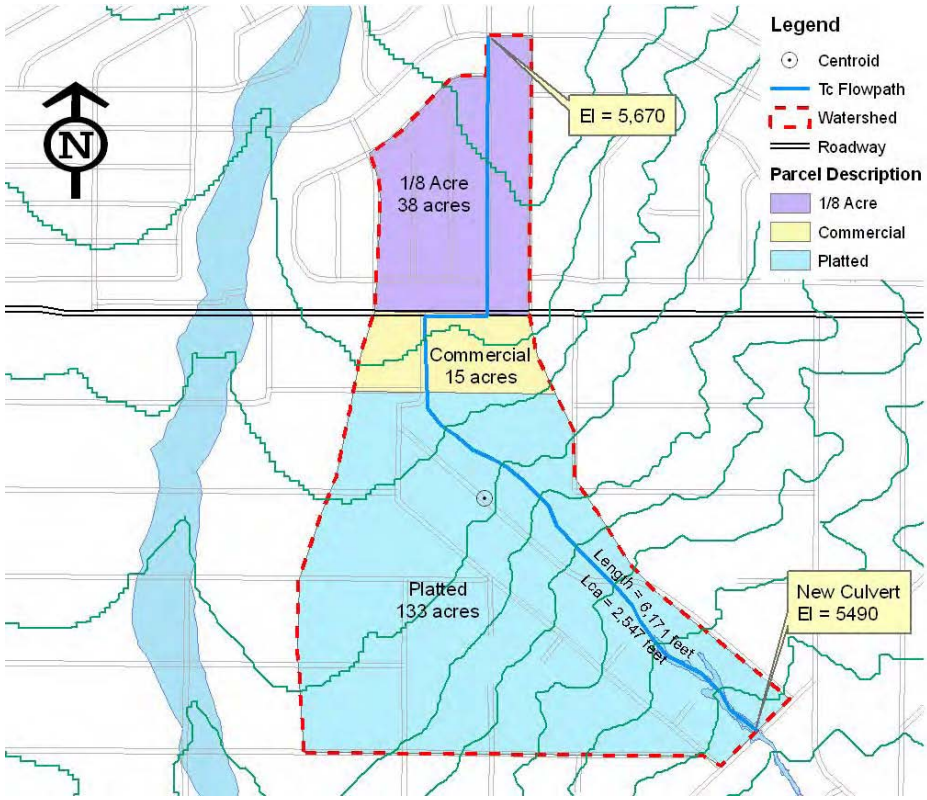
3. Calculate the conveyance factor adjustment range using Equations E-11 and E-12.
4. Recalculate the peak discharge using the procedures in Part D and the adjusted slope and conveyance factor.
5. Repeat steps ii3 and ii4 until the calculated peak discharge is within 10 % of the original value.

b. If the flow path length is longer than 4,000 feet, calculate T_c using Equation E-9 with the following:
   i. Measure L_{ca} and S
   ii. Select appropriate values of K from Table E-5 and K_n from Table E-6

2. Calculate t_p using Equation E-7
3. Calculate k based on the drainage area:
   a. If drainage area is less than or equal to 40 acres in size and contains only one land treatment type, use Equation E-13, E-14 or E-15 as appropriate for the land treatment type present. If multiple land treatment types are present, calculate an arithmetically area-weighted value for pervious areas using Equations E-13 through E-15 and also calculate k for impervious area using Equation E-16.
   b. If drainage area is greater than or equal to 200 acres in size and contains only one land treatment type, use Equation E-17, E-18 or E-19 as appropriate for the land treatment type present. If multiple land treatment types are present, calculate an arithmetically area-weighted value for pervious areas using Equations E-17 through E-19 and also calculate k for impervious area using Equation E-20.
   c. If drainage area is between 40 and 200 acres in size then calculate k according to Step 3a and 3b. Then use linear interpolation to estimate k for the basin drainage area.

E.4.5 Example

A new culvert is to be constructed to convey the 100-year, 6-hour storm at the location shown in the following figure. Compute the unit hydrograph parameters for the contributing watershed.



1. The flow path length is greater than 4,000 feet. Therefore, Equation E-9 is used for the calculating Time of Concentration (T_c). Select K and K_n from Tables E-5 and E-6, respectively.

   C. K = 2 (Shallow concentrated flow within residential area)
   D. K_n = 0.33 (Desert terrain)

2. Using Equation E-9, calculate T_c.

$$T_c = \left(\frac{12{,}000 - 6{,}171}{72{,}000 * 2 * \sqrt{0.029}} + \frac{(6{,}171 - 4{,}000) * 0.033 * \left({2{,}547}/{6{,}171} \right)^{0.33}}{552.2 * 0.029^{0.165}} \right)$$

$$T_c = (0.24 + 0.17)$$

$$T_c = 0.41 \text{ hours}$$

3. Using Equation E-7, calculate t_p.

$$t_p = \left({2}/{3} \right) * T_c$$

$$t_p = \left({2}/{3} \right) * 0.41$$

$$t_p = 0.27 \text{ hours}$$

4. Calculate the recession constant, k, using Equations E-13 through E-20 and the 100-year, 1-hour rainfall depth from Table E-1.

Calculate the k for Land Treatment Type A at 40 and 200 acres

$$k_A^{40} = t_p\left(1.58159 - 0.18912 P_{60}\right)$$

$$k_A^{40} = 0.27 * \left(1.58159 - 0.18912 * 1.84\right)$$

$$k_A^{40} = 0.33 \text{ hours}$$

$$k_A^{200} = t_p\left(0.854 + 0.5808 * 4.756828^{1-P_{60}}\right)$$

$$k_A^{200} = 0.27 * \left(0.854 + 0.5808 * 4.756828^{1-1.84}\right)$$

$$k_A^{200} = 0.27 \text{ hours}$$

Calculate the k for Land Treatment Type B at 40 and 200 acres

$$k_B^{40} = t_p\left(1.22953 - 0.132 P_{60}\right)$$

$$k_B^{40} = 0.27 * \left(1.22953 - 0.132 * 1.84\right)$$

$$k_B^{40} = 0.27 \text{ hours}$$

$$k_B^{200} = t_p\left(0.770 + 0.480 * 4.756828^{1-P_{60}}\right)$$

$$k_B^{200} = 0.27 * \left(0.770 + 0.480 * 4.756828^{1-1.84}\right)$$

$$k_B^{200} = 0.24 \text{ hours}$$

Calculate the k for Land Treatment Type C at 40 and 200 acres

$$k_C^{40} = t_p\left(0.63596 + 0.08462 P_{60}\right)$$

$$k_C^{40} = 0.27 * \left(0.63596 + 0.08462 * 1.84\right)$$

$$k_C^{40} = 0.21 \text{ hours}$$

$$k_C^{200} = t_p\left(0.686 + 0.3792 * 4.756828^{1-P_{60}}\right)$$

$$k_C^{200} = 0.27 * \left(0.686 + 0.3792 * 4.756828^{1-1.84}\right)$$

$$k_C^{200} = 0.21 \text{ hours}$$

Calculate the k for Land Treatment Type D at 40 and 200 acres

$$k_D^{40} = t_p\left(0.31048 + 0.07356 P_{60}\right)$$

$$k_D^{40} = 0.27 * \left(0.31048 + 0.07356 * 1.84\right)$$

$$k_D^{40} = 0.12 \text{ hours}$$

$$k_D^{200} = t_p\left(0.528 + 0.1896 * 4.756828^{1-P_{60}}\right)$$

$$k_D^{200} = 0.27 * \left(0.528 + 0.1896 * 4.756828^{1-1.84}\right)$$

$$k_D^{200} = 0.16 \text{ hours}$$

Calculate the weighted k for the pervious area at 40 and 200 acres

$$k_p^{40} = \frac{(24.9)(0.33) + (44.9)(0.27) + (43.9)(0.21)}{(24.9 + 44.9 + 43.9)}$$

$$k_p^{40} = 0.26 \text{ hours}$$

$$k_p^{200} = \frac{(24.9)(0.27) + (44.9)(0.24) + (43.9)(0.21)}{(24.9 + 44.9 + 43.9)}$$

$$k_p^{200} = 0.23 \text{ hours}$$

Calculate the weighted k for the pervious portion of the watershed using linear interpolation

$$k_p = 0.23 - (0.23 - 0.26) * \left(\frac{200 - 186}{200 - 40}\right)$$

k_p = 0.233 hours

Calculate the weighted k for the impervious portion of the watershed using linear interpolation

$$k_p = 0.16 - (0.16 - 0.12) * \left(\frac{200 - 186}{200 - 40}\right)$$

k_p = 0.157 hours

E.5 CHANNEL ROUTING

Hydrologic channel routing describes the movement of a floodwave (hydrograph) along a watercourse. For most natural rivers, as a floodwave passes through a given reach, the peak of the outflow hydrograph is attenuated and delayed due to flow resistance in the channel and the storage capacity of the river reach. In urban environments, runoff is often conveyed in manmade features such as roadways, storm drains and engineered channels that minimize hydrograph attenuation.

Channel routing is used in flood hydrology models, such as AHYMO, when the watershed is modeled with multiple subbasins and runoff from the upper subbasins must be translated through a channel or system of channels to the watershed outlet. The channel routing method in AHYMO is the Muskingum-Cunge methodology.

The Muskingum-Cunge channel routing is a physically based methodology that solves the continuity and diffusive form of the momentum equation based on the physical channel properties and the inflow hydrograph. The solution procedure involves the discretization of the equations in both time and space (length). The discretized time and distance step size influence the accuracy and stability of the solution.

E.5.1 Physical Parameters

The physical parameters required for the Muskingum-Cunge channel routing are: reach length, flow resistance factor, friction slope and the channel geometry. One limitation of this method is that it cannot account for the effects of backwater. Therefore, the friction slope can be approximated using the average bed slope.

The channel reach length and average bed slope should be estimated from the best available mapping. If there are significant changes in the bed slope over the length of the channel routing reach, a weighted average slope should be estimated or multiple reach lengths used. Also, if the channel bed slope exceeds 0.04 feet per foot then the procedures in Section E.4.2 should be followed.

Hydrologic routing calculations are based on a single cross section that describes the average geometry for the entire reach. The representative geometry can be any prismatic open channel configuration, including a circular section, as well as an irregular channel. Typically, the channel geometry is derived from a single location along the reach that is representative of the overall channel geometry. Channel geometry can be estimated using available topographic mapping or from field survey.

E.5.2 Roughness Coefficients

Flow resistance in the channel and overbank flow area is simulated using Manning's roughness coefficients. Flow resistance is affected by many factors including bed material size, bed form, irregularities in the cross section, depth of flow, vegetation, channel alignment, channel shape, obstructions to flow and the quantity of sediment of being transported in

suspension or as bed load. In general, all factors that retard flow and increase turbulent mixing tend to increase Manning's n-values. Manning's roughness coefficients appropriate for hydrologic routing are listed in Table E-7 and are, in general, taken from the SSCAFCA Sediment and Erosion Design Guide (MEI, 2008). Use of roughness coefficients other than those listed in Table E-7 must be estimated using the information and procedures in the Sediment and Erosion Design Guide and approved by SSCAFCA.

TABLE E-7. MANNING'S ROUGHNESS COEFFICIENTS	
Channel or Floodplain Type	**n-value**
Sand bed arroyos	0.055
Tined concrete	0.018
Shotcrete	0.025
Reinforced concrete pipe	0.013
Trowled concrete	0.013
No-joint cast-in-place concrete pipe	0.014
Reinforced concrete box	0.015
Reinforced concrete arch	0.015
Streets	0.017
Flush grouted riprap	0.020
Corrugated metal pipe	0.025
Grass-lined channels (sodded & irrigated)	0.025
Earth-lined channels (smooth)	0.030
Wire-tied riprap	0.040
Medium weight dumped riprap	0.045
Grouted riprap (exposed rock)	0.045
Jetty type riprap (D50 > 24")	0.050

E.5.3 Procedure

1. From an appropriate map of the watershed, measure the routing reach length in feet and estimate the friction slope as the channel bed slope in feet per foot.
2. Select and identify cross sectional geometry that represents that average hydraulic conditions of the reach. If a single cross section cannot be identified that represents

the average hydraulic conditions, break the reach into multiple sections and treat each as a unique element in AHYMO.

3. Conduct a field reconnaissance of the watershed and routing reaches to observe the flow resistance characteristics.
4. Select an appropriate Manning's roughness coefficient for the channel and overbank flow areas using Table E-7.

E.6 SEDIMENT BULKING

Flow bulking occurs when sediment is eroded from the land surface and entrained into the flowing water. Entrained sediment has the effect of increasing the runoff volume and flow rate. Within this jurisdiction there is potential for high sediment yields. For undeveloped watersheds the bulking factor is 18%. Similarly, sediment yield from developed areas shall be 6%. Developed conditions are those areas that have paved roads with curb and gutter. Given the high potential for surface erosion, all watershed models will include flow bulking.

E.6.1 Procedure

In AHYMO, flow bulking for sediment is simulated using a ratio. The ratio is applied to direct runoff estimated for each subbasin. The bulking factor is applied globally using the SEDIMENT BULK Command. The bulking factor specified on this command is used for all subsequent runoff calculation until changed by another SEDIMENT BULK Command.

F. Rainfall-Runoff Modeling: HEC-HMS

F.1 INTRODUCTION

Rainfall-runoff modeling for drainage areas greater than 320 acres in size is to be conducted using the U.S. Army Corps of Engineers HEC-HMS software. HEC-HMS can also be applied to drainage areas between 40 and 320 acres in size. HEC-HMS is the successor to HEC-1 and has been in use since 1998. HEC-HMS is a public domain software that is part of the Hydrologic Engineering Center's Next Generation Software Development Project. Input to HEC-HMS is to be developed using the recommended methodologies, techniques and procedures presented in the following sections.

F.2 DESIGN RAINFALL CRITERIA

For design hydrology, the characteristics of the major flood producing storm are simulated using a synthetic storm. Components of a synthetic storm are basin average rainfall depth and temporal distribution. Information and procedures for developing the design rainfall criteria for storms other than the Probable Maximum Precipitation are provided in the following sections.

F.2.1 Depth

The principal design storm for peak flow determination is the 100-year, 6-hour event. For analysis and design of retention ponds and detention dams, the 100-year, 24-hour storm is to be used unless the structure falls under the jurisdiction of the New Mexico Office of State Engineer, Dam Safety Bureau. Point precipitation depths for the 100-year storm to be used within the SSCAFCA jurisdiction are provided in Table F-1. Those values are adapted from NOAA Atlas 14, Precipitation - Frequency Atlas of the United States, Volume 1: Semiarid Southwest (Arizona, Southeast California, Nevada, New Mexico, Utah).

For determining sediment transport and for analysis of watersheds with complex routing conditions, other storm frequencies and durations may be required. Point precipitation depths for use in the SSCAFCA jurisdiction for multiple recurrence intervals and storm durations are listed in Table F-1. For all other recurrence intervals and storm durations, point precipitation depths are to be obtained directly from the National Weather Service through the NOAA 14 Precipitation Frequency Data Server web site found at http://hdsc.nws.noaa.gov/hdsc/pfds/sa/nm_pfds.html. At this web site point precipitation values for frequencies up to 1,000 years and duration up to 60 days can be obtained by entering the latitude and longitude of the watershed of interest.

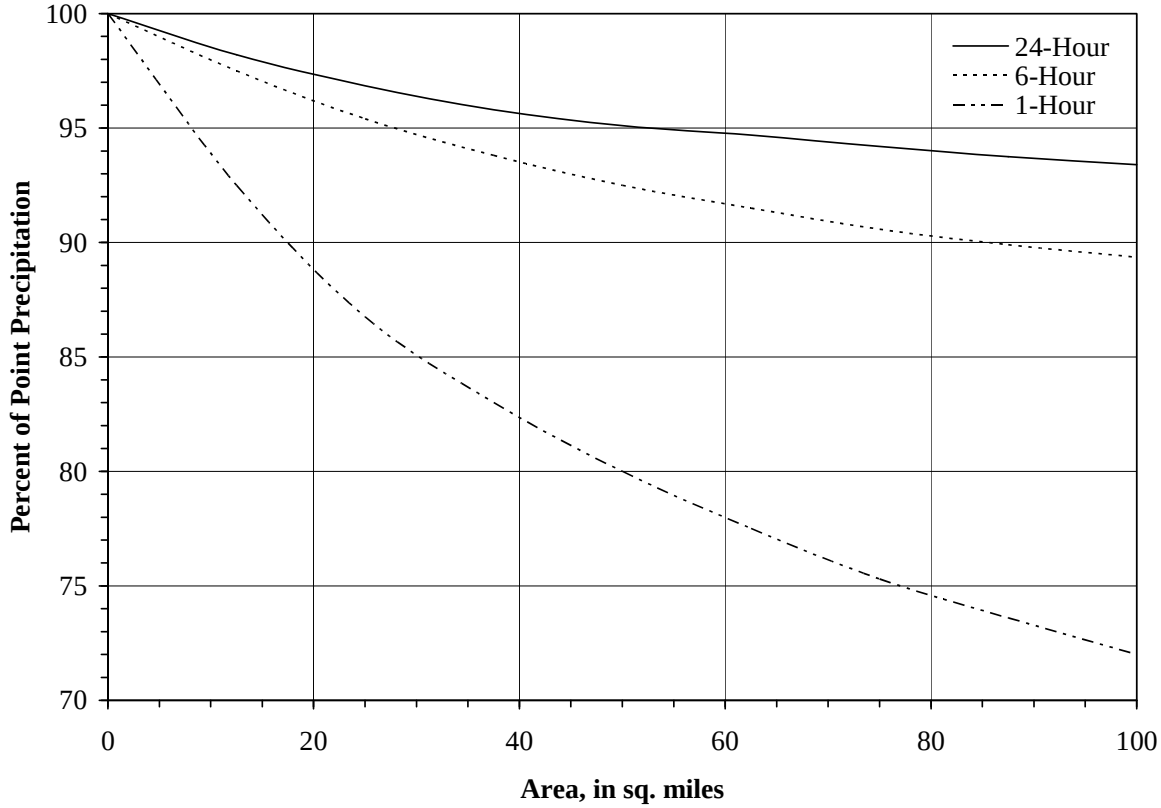
TABLE F-1. RECURRENCE INTERVAL POINT PRECIPITATION DEPTHS				
Recurrence Interval	**Duration**			
Years	**15-Minute**	**1-Hour**	**6-Hour**	**24-Hour**
500	1.42	2.37	3.01	3.57
100	1.10	1.84	2.37	2.90
50	0.97	1.62	2.11	2.57
25	0.85	1.42	1.86	2.29
10	0.70	1.16	1.54	1.90
5	0.58	0.97	1.31	1.66
2	0.43	0.72	1.02	1.32
1	0.34	0.56	0.81	1.05

F.2.2 Depth-Area-Reduction

The rainfall depths listed in Table F-1 or obtained from the NOAA 14 Precipitation Frequency Data Server web site are point rainfall depths for specified durations. This depth is not the areal-averaged rainfall over the basin that would occur during a storm. For uncontrolled watersheds (those areas not controlled by dams, ponds and / or partial diversions), a reduction factor is used to convert the point rainfall to an equivalent uniform depth over the entire watershed. Reduction factors for converting point rainfall depths to basin averaged rainfall are depicted graphically in Figure F-1. That figure is adapted from NOAA Atlas 2 Precipitation-Frequency Atlas of the Western United States, Vol. IV - New Mexico.

The use of Figure F-1 is appropriate for sizing major dams, channels and arroyos but is usually not appropriate for sizing channel inlets, side drainage and storm sewers associated with these major facilities. Use of a single depth-area reduction factor 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.

FIGURE F-1. 100-YR DEPTH-AREA REDUCTION FACTORS



F.2.3 Temporal Distribution

Basin average rainfall for 100-year, 6- and 24-hour storms is distributed temporally using a suite of equations; F-1 through F-6. The equations are a function of the 1-, 6- and 24-hour basin average depths. The design rainfall distribution is front loaded with the peak intensity set at 85.3 minutes (hour 1.42) regardless of storm duration. This distribution results in approximately 80 percent of the total depth occurring in less than one hour. For the 6-hour storm the distribution of rainfall is determined using the first 5 of the 6 equations. For the 24-hour storm, all 6 equations are used. To illustrate the shape of the pattern, the 6-hour storm distribution using the depths from Table F-1 for a 20 square mile watershed is shown in Figure F-2.

$$P_T = 2.334 * (P_{360} - P_{60}) * \left(1.5^A - \left(1.5 - \frac{t}{60}\right)^A\right)$$ For $0 \le t \le 60$ (F-1)

$$P_T = P_{T=60} + P_{60} * 0.4754 * \left(0.5^{0.09} - \left(1.5 - \frac{t}{60}\right)^{0.09}\right)$$ For $60 < t < 67$ (F-2)

$$P_T = P_{T=60} + P_{60} * \left(0.0001818182 * (t - 60) + 0.000018338 * (t - 60)^{3.2}\right)$$ For $67 \le t < 85.3$ (F-3)

$$P_T = P_{T=60} + P_{60} * \left(0.07 * (t - 60) - 1.1886 - 0.0404768 * (t - 85)^{1.0985865}\right)$$ For $85.3 \le t < 120$ (F-4)

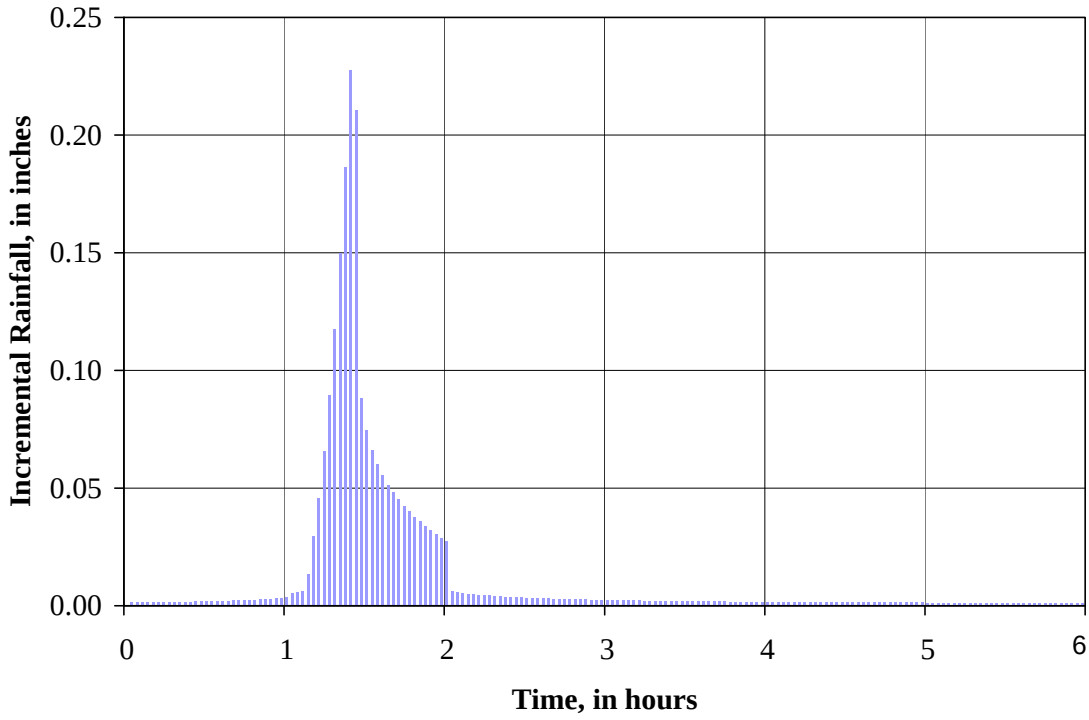
$$P_T = P_{360} + (P_{T=60} + P_{60} - P_{360}) \frac{4.4^{3A} - \left(t/60 - 1.6\right)^{3A}}{4.4^{3A} - 0.4^{3A}}$$ For $120 \le t \le 360$ (F-5)

$$P_T = P_{1440} + (P_{360} - P_{1440}) * \frac{30^B - \left(t/60 + 6\right)^B}{30^B - 12^B}$$ For $360 < t < 1440$ (F-6)

Where: $$A = \frac{Log\left(P_{360}/P_{60}\right)}{Log(6.0)}$$

$$B = \frac{Log\left(P_{1440}/P_{360}\right)}{Log(4.0)}$$

FIGURE F-2. 100-YR 6-HOUR RAINFALL HYETOGRAPH



F.2.4 Procedure

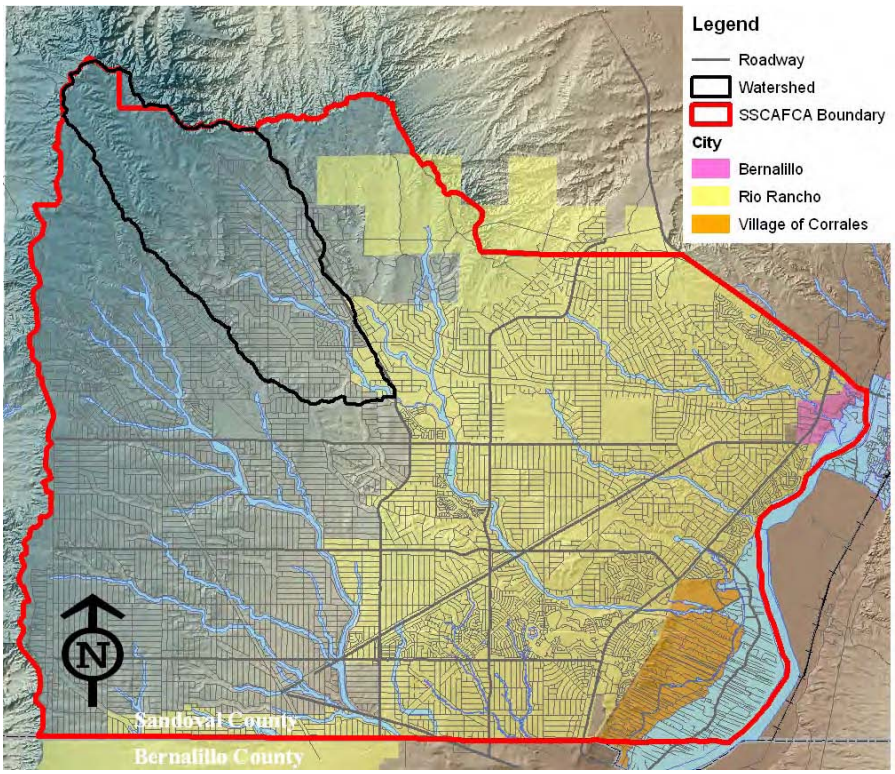
A. For design events up to the 500-year and storm durations up to the 24-hour
   1. Select the point rainfall depths from Table F-1
   2. Using Figure F-1, determine the depth-area adjustment factor for each duration using the total watershed area.
   3. Areally reduce the point precipitation depths from Table F-1 using the depth-area adjustment factors from Figure F-1.
   4. Obtain the rainfall distribution from SSCAFCA.
   5. In HEC-HMS
      a. Code the distribution in as time distribution data.
      b. Select the "Specified Hyetograph" as the Meterological Model
      c. Select "Yes" to include subbasins

B. For design storms with durations other than 6- or 24-hours, submit in writing a recommendation to SSCAFCA for depth-area reduction and time distribution of the rainfall for the selected storm event.

F.2.5 Example

Compute the 100-year, 6-hour storm design rainfall data for the watershed shown in the following figure. The watershed area is approximately 20.5 square miles.

FIGURE F-3. EXAMPLE WATERSHED MAP

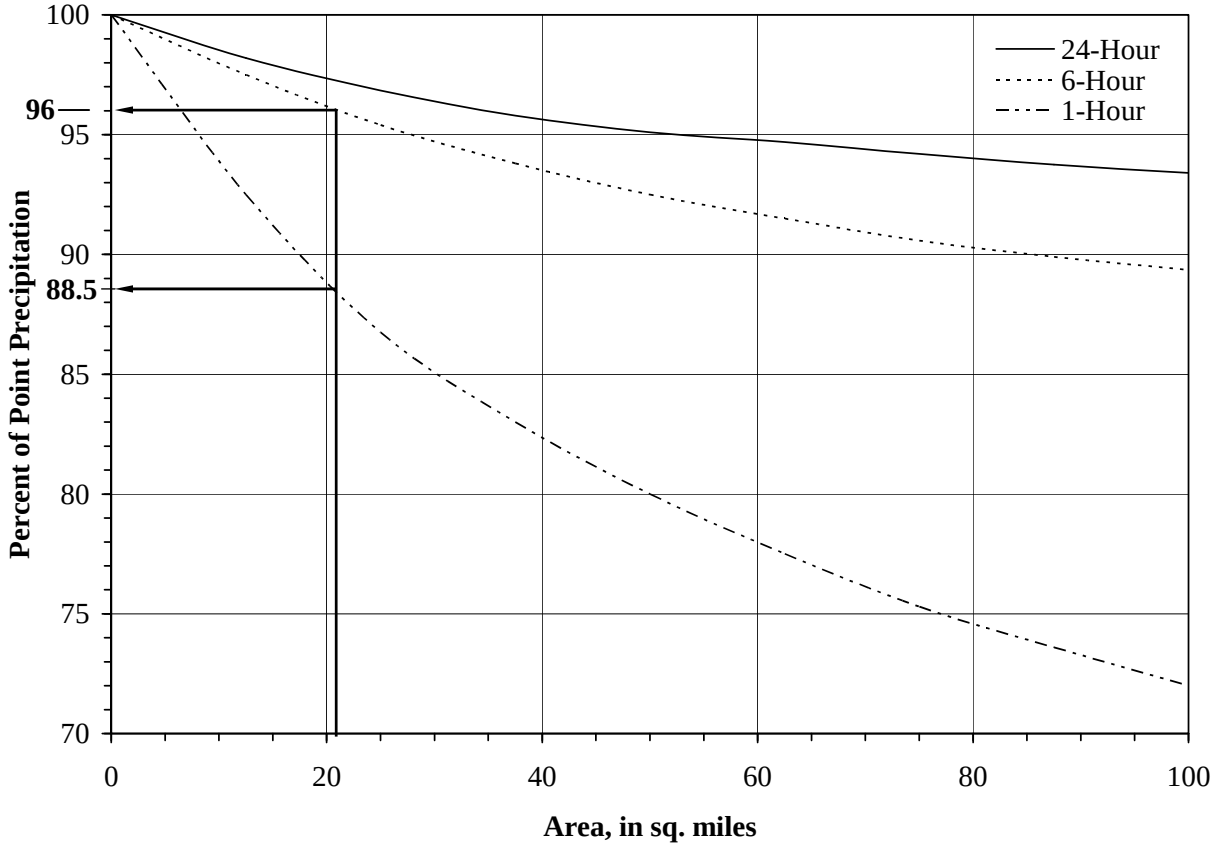


1. 100-year point rainfall depths from Table F-1 are:

   F. 100-year, 1-hour = 1.84 inches
   G. 100-year, 6-hour = 2.37 inches

2. Estimate depth-area reduction factors for the watershed area of 20.5 square miles using Figure F-1

FIGURE F-4. EXAMPLE WATERSHED DEPTH-AREA REDUCTION



3. Calculate the equivalent uniform rainfall depth

$$P_{60}^{100} = (1.84)(0.885) = 1.63 \text{ inches}$$

$$P_{360}^{100} = (2.37)(0.960) = 2.28 \text{ inches}$$

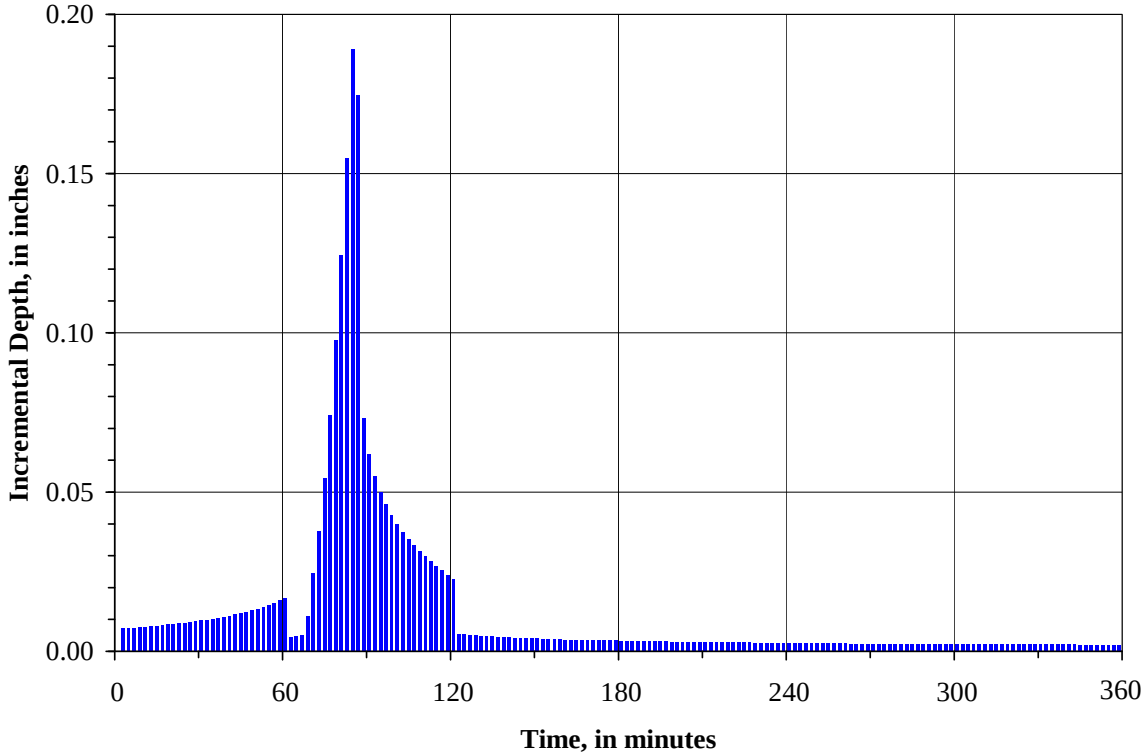
4. Calculate the cumulative rainfall mass curve using Equations F-1 through F-5 for the 6-hour storm. The computation time interval is 2 minutes.

Sample – To be obtained directly form SSCAFCA for use in hydrologic modeling.

CUMULATIVE RAINFALL DISTRIBUTION

Time hours	Rainfall inches	Time hours	Rainfall inches	Time hours	Rainfall inches	Time hours	Rainfall inches
0	0.000	92	1.459	184	2.065	276	2.190
2	0.007	94	1.509	186	2.068	278	2.192
4	0.014	96	1.555	188	2.071	280	2.194
6	0.021	98	1.598	190	2.074	282	2.197
8	0.028	100	1.638	192	2.078	284	2.199
10	0.036	102	1.676	194	2.081	286	2.201
12	0.043	104	1.712	196	2.084	288	2.203
14	0.051	106	1.745	198	2.087	290	2.206
16	0.059	108	1.777	200	2.090	292	2.208
18	0.067	110	1.807	202	2.093	294	2.210
20	0.075	112	1.835	204	2.096	296	2.213
22	0.084	114	1.862	206	2.099	298	2.215
24	0.092	116	1.887	208	2.101	300	2.217
26	0.101	118	1.911	210	2.104	302	2.219
28	0.110	120	1.934	212	2.107	304	2.221
30	0.120	122	1.940	214	2.110	306	2.224
32	0.129	124	1.945	216	2.113	308	2.226
34	0.139	126	1.951	218	2.116	310	2.228
36	0.149	128	1.956	220	2.118	312	2.230
38	0.160	130	1.961	222	2.121	314	2.232
40	0.171	132	1.965	224	2.124	316	2.235
42	0.182	134	1.970	226	2.127	318	2.237
44	0.193	136	1.975	228	2.129	320	2.239
46	0.205	138	1.979	230	2.132	322	2.241
48	0.218	140	1.984	232	2.135	324	2.243
50	0.231	142	1.988	234	2.137	326	2.245
52	0.244	144	1.992	236	2.140	328	2.247
54	0.258	146	1.996	238	2.143	330	2.249
56	0.273	148	2.000	240	2.145	332	2.251
58	0.288	150	2.004	242	2.148	334	2.254
60	0.304	152	2.008	244	2.150	336	2.256
62	0.309	154	2.012	246	2.153	338	2.258
64	0.314	156	2.016	248	2.155	340	2.260
66	0.319	158	2.020	250	2.158	342	2.262
68	0.330	160	2.024	252	2.160	344	2.264
70	0.355	162	2.027	254	2.163	346	2.266
72	0.393	164	2.031	256	2.165	348	2.268
74	0.448	166	2.035	258	2.168	350	2.270
76	0.522	168	2.038	260	2.170	352	2.272
78	0.620	170	2.042	262	2.173	354	2.274
80	0.746	172	2.045	264	2.175	356	2.276
82	0.902	174	2.048	266	2.178	358	2.278
84	1.092	176	2.052	268	2.180	360	2.280
86	1.268	178	2.055	270	2.182		
88	1.341	180	2.058	272	2.185		
90	1.403	182	2.062	274	2.187		

FIGURE F-5. 100-YR 6-HOUR RAINFALL HYETOGRAPH



F.3 RAINFALL LOSS

Rainfall losses are generally considered to be the result of evaporation of water from the land surface, interception of rainfall by vegetal cover, depression storage on the land surface and the infiltration of water into the soil matrix. The magnitude of rainfall loss is typically expressed as an equivalent uniform depth in inches. By a mass balance, rainfall minus losses equals rainfall excess. Estimation of rainfall loss is an important element in flood analyses that must be clearly understood and estimated with care.

F.3.1 Land Treatment

Estimation of rainfall losses are based on a characterization of the watershed area into land treatment classifications. Four land treatment classifications have been created that typify the conditions in the SSCAFCA jurisdiction. Descriptions of the land treatment classifications are provided in Table F-2. Three of the land treatment classifications (A, B and C) are for pervious conditions. The fourth classification (D) is for impervious areas.

TABLE F-2. 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. In lieu of specific measurement for treatment D, the areal percentages in Table F-3 may be employed.	

Of the land treatment classifications listed in Table F-2, only treatment type A represents land in its natural, undisturbed state. Land treatment classifications B and C describe conditions that have been impacted by some form of urbanization. Urban areas within a watershed usually contain a mix of the land treatment types. Ideally, the specific area of each land treatment type can be measured from available information. In lieu of specific measurement for each unique land treatment type that occurs within urban areas, generalized percentages based on zoning classifications can be used. Average land treatment type percentages associated with various zoning designations are listed in Table F-3.

TABLE F-3 SSCAFCA TREATMENT TYPE PERCENTAGE SUMMARY

Parcel Description	Treatments				Methodology/Notes
	A	B	C	D	
1/8 Acre	0%	15%	15%	70%	DPM, Chapter 22.2, Table A-4 for D
1/6 Acre	0%	28%	15%	57%	Northern Meadows Master Plan
1/4 Acre	0%	30%	28%	42%	DPM, and followed SSCAFCA lead on B&C
1/2 Acre	10%	33%	30%	27%	SSCAFCA
1 Acre	43%	20%	20%	17%	SSCAFCA
Single Family Residential N=units/acre, N6					7*√((N*N) + (5*N))
Estate Lots (btwn 1-5ac)	60%	15%	15%	10%	DPM for 2.5 acre lot
M-1 (Light Industrial)	0%	15%	15%	70%	DPM for D, split B & C
Vacant Res./Undevel.	79%	8%	8%	5%	DPM for 5 acre lot
Arroyo	100%	0%	0%	0%	DPM
Major Roads	0%	0%	10%	90%	DPM
School	10%	20%	20%	50%	DPM
Commercial/Industrial	0%	0%	15%	85%	DPM average of Heavy Industrial and Commercial
Open Space	100%	0%	0%	0%	DPM
Parks, Sports and Rec	0%	85%	0%	15%	DPM
Landfill	0%	0%	100%	0%	All disturbed ground
Multi-Family	0%	15%	15%	70%	DPM-Multiple Unit Res. Attached
Northern Meadows	0%	28%	15%	57%	Northern Meadows Master Plan
Drainage Ponds	0%	0%	100%	0%	
County Platted (1)	18.7%	29.5%	27.0%	24.8%	(used Basin P12_104 as typical)
County Unplatted (2)	95%	5%	0%	0%	DPM

NOTES

1. County Platted area is defined as the area between CORR boundary and Rio Rancho Estates boundary.
2. County Unplatted area is defined as the area outside the city limits and the Rio Rancho Estates limits. It is considered to be existing conditions.
3. All roads are assumed to be paved.

F.3.2 Initial and Constant Loss

Simulation of rainfall loss in HEC-HMS is accomplished using the Initial and Constant Loss Method. The Initial and Constant Loss Methodology is a two parameter model. The first parameter is the Initial Abstraction (IA). The initial abstraction is the summation of all losses other than infiltration and is applied at the beginning of the storm event. The second parameter is the constant loss. The constant loss is the Infiltration rate (INF) of the soil matrix at saturation. The constant loss is only applied once the Initial Abstraction is satisfied. An illustration of the application of this method is provided in Figure F-6.

Recommended values for the Initial Abstraction and Infiltration rate are assigned to each pervious land treatment type and are listed in Table F-4. For watersheds and subbasins with multiple unique land treatment types an arithmetic area averaged value for IA and INF is to be calculated.

FIGURE F-6. REPRESENTATION OF THE INITIAL AND CONSTANT LOSS METHODOLOGY

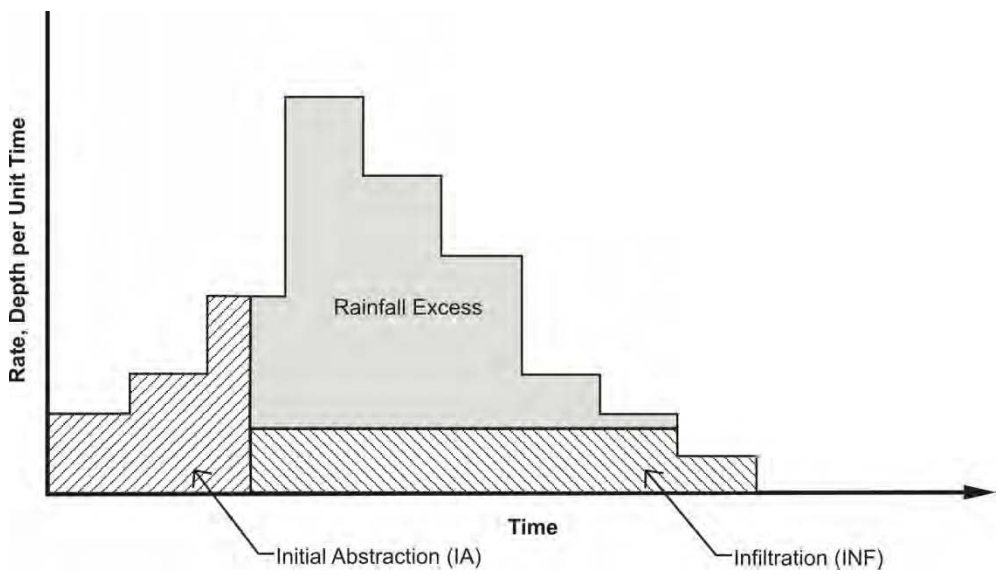


TABLE F-4. INITIAL AND CONSTANT LOSS PARAMETERS		
Land Treatment	**Initial Abstraction (inches)**	**Infiltration (inches/hour)**
A	0.65	1.67
B	0.50	1.25
C	0.35	0.83

F.3.3 Impervious Area

For the Initial and Constant Loss Method as employed in HEC-HMS, it is assumed that there are no losses associated with impervious area (land treatment type D) and rainfall over the impervious area is converted directly to rainfall excess. The percentage of rainfall converted directly to excess is the same as the percent area of land treatment type D. Computationally, rainfall to be converted directly to excess occurs prior to any loss calculations for each model time step. The rainfall not converted directly to excess is then available to the loss calculations.

F.3.4 Procedure

1. For each subbasin, calculate the area of each unique land treatment type or zoning classification.
2. Using the percent area of each pervious area land treatment type, calculate the area averaged value of IA and INF using the data from Table F-4 for each subbasin.
3. For each subbasin sum the percent impervious area as the percent area of Land Treatment Type D.
4. In HEC-HMS, for each subbasin within the Basin Model:
    a. code the subbasin area average value of IA as the Initial Loss.
    b. code the subbasin area average value of INF as the Constant Rate.
    c. code the total percent area of Land Treatment Type D as the impervious percentage.

F.3.5 Example

Calculate the rainfall loss parameters for a 20.5 square mile watershed using the following data:

Parcel Description	**Area sq. miles**
1/8 acre	**1.0**
Platted	**11.5**
Unplatted	**8.0**

1. From Table F-3, percentage of Land Treatment Types for each parcel within the watershed are:

Parcel Description	**Area acres**	**Percent of Land Treatment Type**			
		A	**B**	**C**	**D**
1/8 Acre	1.0	0	15	15	70
Commercial / Industrial	11.5	18.7	29.5	27.0	24.8
Platted	8.0	95	5	0	0

Area of each Land Treatment Type is calculated as:

- $Area_A = (0)(1.0) + (0.187)(11.5) + (0.95)(8.0) =$ 9.8 sq. miles
- $Area_B = (0.15)(1.0) + (0.295)(11.5) + (0.05)(8.0) =$ 3.9 sq. miles
- $Area_C = (0.15)(1.0) + (0.270)(11.5) + (0.0)(8.0) =$ 3.3 sq. miles
- $Area_D = (0.70)(1.0) + (0.248)(11.5) + (0.0)(8.0) =$ 3.5 sq. miles

Total Area = 20.5 sq. miles

2. Using values of IA from Table F-4, calculate the weighted value of IA

$$IA = \frac{(9.8)(0.65) + (3.9)(0.50) + (3.3)(0.35)}{(9.8 + 3.9 + 3.3)}$$

IA = 0.56 inches

3. Using values of INF from Table E-4, calculate the weighted value of

$$INF = \frac{(9.8)(1.67) + (3.9)(1.25) + (3.3)(0.83)}{(9.8 + 3.9 + 3.3)}$$

INF = 1.41 in/hr

4. Assign the impervious area as the percent area of Land Treatment Type D

$$\textit{Percent Impervious} = \left(3.5 / 20.5\right) = 17.1\%$$

F.4 UNIT HYDROGRAPH

Rainfall excess generated during a storm event is routed across the basin surface and eventually begins to concentrate at a downstream location (concentration point). The routing process results in the transformation of rainfall excess to a runoff hydrograph. Simulation of rainfall excess transformation is typically accomplished using the concept of a unit hydrograph. A unit hydrograph is defined as the hydrograph of one inch of direct runoff from a storm of a specified duration for a particular basin. Every watershed will have a different unit hydrograph that reflects the topography, land use, and other unique characteristics of the individual watershed. Different unit hydrographs will also be produced for the same watershed for different durations of rainfall excess.

For most watersheds, sufficient data (rainfall and runoff records) does not exist to develop unit hydrographs specific to the watershed. Therefore, indirect methods are used to develop a unit hydrograph. Such unit hydrographs are called synthetic unit hydrographs. The synthetic unit hydrograph method in HEC-HMS that is to be used to transform rainfall excess to a runoff hydrograph is the Clark unit hydrograph.

The Clark unit hydrograph is analogous to the routing of an inflow hydrograph through a reservoir. The inflow hydrograph, called the translation hydrograph in the Clark method, is determined from the temporal and spatial distribution of rainfall excess over a basin. The translation hydrograph is then routed by a form of the continuity equation. The Clark method uses two numeric parameters; Time of Concentration (T_c) and Storage Coefficient (R) and a graphical parameter, the time-area relation. The time-area relation defines the relation between the accumulated area of a basin and the time it takes for runoff from that area to reach the basin outlet. In the current version of HEC-HMS, the time-area relation is hard coded and cannot be changed by the user.

F.4.1 Time of Concentration

Time of concentration is defined as the 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 (concentration point). The units for time of concentration are time, in hours. This implies that the time of concentration flow path may not be the longest physical length, but the length that results in the longest time.

Time of concentration is calculated using one of three equations. Selection of the appropriate equation is based on the time of concentration flow path length (in time). Regardless of the selected equation, time of concentration should not be less than 8 minutes.

For basins with flow path lengths less than 4,000 feet the SCS Upland Method is used. The Upland Method is the summation of flow travel time for the series of unique flow characteristics that occur along the overall basin flow path length. The Upland Method travel time equation is:

$$T_c = \frac{2}{3} * \sum_{i=1}^{n} \left(\frac{L_i}{36{,}000 * K_i * \sqrt{S_i}} \right) \qquad \text{(F-7)}$$

Where: T_c = Time of concentration, in hours
L_i = Length of each unique surface flow conveyance condition, in feet
K_i = Conveyance factor from Table F-5
S_i = Slope of the flow path, in feet per foot

TABLE F-5. 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, natural channels, and arroyos 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 basins with flow path lengths greater than 12,000 feet the time of concentration is calculated using a form of the basin lag equation. Coefficients and exponents follow USDI Bureau of Reclamation recommendations.

$$T_c = \frac{8}{9} * 26 K_n \left(\frac{L * L_{ca}}{5280^2 * \sqrt{5280 * S}} \right)^{0.33} \qquad \text{(F-8)}$$

Where: T_c = Time of concentration, in hours
L = Flow path length, in feet
L_{ca} = Distance along L from point of concentration to a point opposite the centroid of the basin, in feet
K_n = Basin factor, from Table F-6
S = Slope of flow path, in feet per foot

K_n in Equation F-8 is a measure of the hydraulic efficiency of the watershed to convey runoff to the basin outlet. This is analogous to a Manning's roughness coefficient. Selection of K_n should reflect the conditions of all the watercourse in the basin that convey runoff to the outlet.

TABLE F-6. 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)

For basins with flow path lengths between 4,000 and 12,000 feet a transition equation is used that is a composite of equations F-7 and F-8. This transition equation is expressed as:

$$T_c = \left(\frac{2}{3}\right) * \left(\frac{12{,}000 - L}{72{,}000 * K * \sqrt{S}} + \frac{(L - 4{,}000) * K_n * \left(L_{ca} / L \right)^{0.33}}{552.2 * S^{0.165}} \right) \qquad \text{(F-9)}$$

Where: T_c = Time of concentration, in hours

L = Flow path length, in feet

L_{ca} = Distance along L from point of concentration to a point opposite the centroid of the basin, in feet

K = Conveyance factor from Table F-5. For composite reaches, K is computed using equation F-9a or F-9b as discussed below.

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/Rio Rancho area, values of K_n may be estimated from Table F-6.

S = Slope of flow path, in feet per foot. For composite reaches, s is computed using equation F-9c (weighted average) as discussed below.

For composite reaches where the basin slope is uniform, the composite basin conveyance condition, K, can be computed using the following equation:

$$K = \frac{L}{(L_1/K_1 + L_2/K_2 + \cdots + L_x/K_x)} \qquad \text{(F-9a)}$$

Where L = $L_1 + L_2 + \cdots + L_x$

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:

$$K = \frac{(L/\sqrt{s})}{(L_1/(K_1 * \sqrt{s_1}) + L_2/(K_2 * \sqrt{s_2}) + \cdots + L_x/(K_x * \sqrt{s_x}))} \qquad \text{(F-9b)}$$

Where L = $L_1 + L_2 + \ldots + L_x$

And, s= $$\frac{(L1 * s1 + L2 * s2 + \cdots + Lx * sx)}{L} \qquad \text{(F-9c)}$$

Calculation of a basin time of concentration is a function of flow path length and by extension basin area. Therefore, basin / subbasin delineation is a key consideration that must be addressed early on in the modeling process as it not only influences unit hydrograph parameter estimation but rainfall loss parameters as well. Wherever possible, subbasin delineation should be based on the best available topographic mapping and if available detailed aerial photography. For some areas, field investigation may also be necessary to verify subbasin boundaries particularly in urban or distributary areas. The breakdown of a watershed into subbasins should consider the following:

- The subbasin sizes should be as uniform as possible.
- Subbasins should have fairly homogeneous land use and geographic characteristics. For example: mountain, hillslope and valley areas should be separated by subbasin where possible.
- Soils, vegetation and land treatment characteristics should be fairly homogeneous.
- Subbasins size should be commensurate with the intended use of the model. For example, if the model is to be used for the evaluation and / or design of drainage infrastructure, the subbasin size should be fairly small so that runoff magnitudes are known at multiple locations within the watershed. For drainage management plans, the subbasin size shall in general not be greater than 1.5 mi^2 or less than 0.1 mi^2.

F.4.2 Time of Concentration for Steep Slopes and Natural Channels

The equations used to compute time of concentration may result in 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 equations identified in Section A.3.1 can result in 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:

- Slopes flatter than 0.04 foot per foot.
- Channels with irrigated grass, riprap, soil cement, gabion, or concrete lining which cannot be clearly identified as natural or naturalistic.
- The hydraulic design of channels or channel elements. The purpose of 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) <u>Urban Storm Drainage Criteria Manual</u> (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:

$$S' = 0.052467 + 0.062627\,S - 0.18197\,e^{-62.375S} \qquad \text{(F-10)}$$

Where: S = Measured slope, in feet per foot
S′ = Adjusted slope, in feet per foot

The conveyance factors (K) for the Upland Method should be checked to make sure that appropriate Froude Numbers are maintained. The Lag Equation Basin Factors, K_n, from Table F-6 remain applicable when using equations F-8 and F-9 with the adjusted slope computed by equation F-10. To adjust the conveyance factor (K) it is necessary to estimate the peak flow rate from the watershed. Using estimated conveyance factors (K) from Table F-5 and the procedures outlined in Part D, an estimated peak flow rate for the basin (Q_p) can be computed. The following formulas are then used to compute conveyance factor adjustment:

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

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

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

- if K > K' then K= K'
- if K' ≥ K ≥ K" then K = K (no adjustment)
- if K < K" then K= K"

This is an iterative process that is to be repeated until the computed value of Q_p is within 10 percent of original value of Q_p.

F.4.3 Storage Coefficient

The storage coefficient describes the effect that temporary storage in the basin has on the hydrograph. The storage coefficient has the units of time and is interrelated with time of concentration. The temporary storage potential of runoff for a basin is also influenced by the land treatment conditions present. The equation for estimating the storage coefficient is:

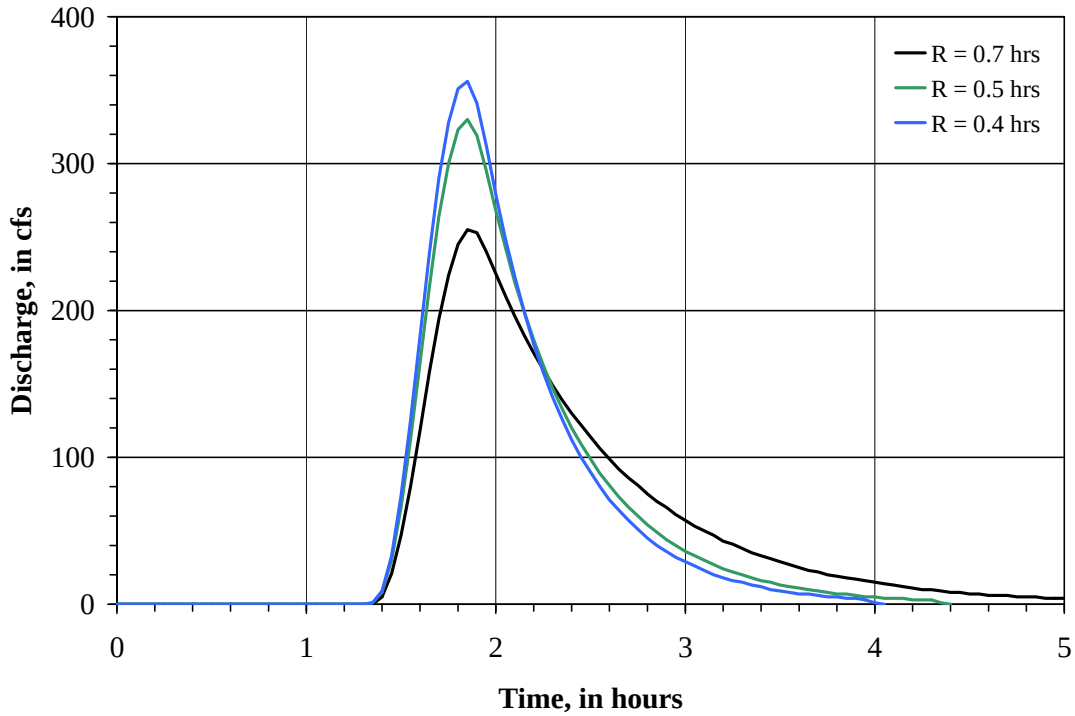
$$R = 1.165 * T_c \left(INF^{0.45} - IA^{1.4} \left(\frac{D}{100} \right)^{0.40} \right) \qquad \text{(F-13)}$$

Where: R = Storage coefficient, in hours
T_c = Time of concentration, in hours (from Eqn. F-7, F-8 or F-9)
INF = Infiltration loss rate for the subbasin, in in/hr
IA = Initial abstraction for the basin, in inches
D = Land treatment type D, expressed in percent

Land treatment conditions (impervious area in particular), influence the storage coefficient in that as the degree of development increases, the storage coefficient decreases. This results in a decrease in the time that runoff is stored in the basin. Thus a greater proportion of runoff volume is conveyed to the basin outlet over a shorter time period, resulting in a higher peak discharge. This is illustrated in Figure F-7. In that figure runoff hydrographs are plotted for a hypothetical

basin 1 square mile in size. Reducing the storage coefficient while holding all other parameters constant results in the compression of the time distribution of runoff and thus an increase in peak discharge.

FIGURE F-7. INFLUENCE OF WATERSHED STORAGE ON THE RUNOFF HYDROGRAPH



F.4.4 Procedure

1. Delineate the time of concentration flow path for each subbasin and measure the length, in feet.
    a. If the flow path length is less than 4,000 feet, calculate T_c using Equation F-7 with the following:
        i. Select K from Table F-5
        ii. Measure the average flow path slope, S. If the flow path slope is greater than 0.04 feet / foot:
            1. Calculate the adjusted slope using Equation F-10.
            2. Estimate the peak discharge using procedures in Part D
            3. Calculate the conveyance factor adjustment range using Equations F-11 and F-12.
            4. Recalculate the peak discharge using the procedures in Part D and the adjusted slope and conveyance factor.
            5. Repeat steps ii3 and ii4 until the calculated peak discharge is within 10 % of the original value.
    b. If the flow path length is between 4,000 and 12,000 feet, calculate T_c using Equation F-9 with the following:

i. Measure L_{ca} and S
ii. Select appropriate values of K from Table F-5 and K_n from Table F-6

c. If the flow path length is greater than 12,000 feet, calculate T_c using Equation F-8 with the following:
i. Measure L_{ca} and S
ii. Select appropriate values of K_n from Table F-6

2. Calculate the storage coefficient for each subbasin using Equation F-13
3. In HEC-HMS code in the calculated values for time of concentration and storage coefficient for each subbasin.

F.4.5 Example

Calculate the unit hydrograph parameters for a 20.5 square mile watershed based on the following data. Rainfall loss parameters for the watershed are from the example in Section F.3.5.

- Flow path length, L = 8.5 miles
- Length to centroid, L_{ca} = 4.0 miles
- Flow path slope, S = 1.8%

1. Calculate T_c

The flow path length is greater than 12,000 feet. Therefore, use Equation F-8 and assume a value for K_n of 0.033.

$$T_c = \frac{8}{9} * 26 K_n \left(\frac{L * L_{ca}}{5280^2 * \sqrt{5280 * S}} \right)^{0.33}$$

$$T_c = \frac{8}{9} * 26 * (0.033) \left(\frac{8.5 * 4.0}{\sqrt{5280 * 0.018}} \right)^{0.33}$$

T_c 1.15 hours

2. Using Equation F-13, calculate the Clark unit hydrograph storage coefficient, R.

$$R = 1.165 * T_c \left(INF^{0.45} - IA^{1.4} \left(\frac{D}{100} \right)^{0.40} \right)$$

$$R = 1.165 * 1.15 * \left(1.41^{0.45} - 0.56^{1.4} \left(\frac{17.1}{100} \right)^{0.40} \right)$$

R = 1.27 hours

F.5 CHANNEL ROUTING

Hydrologic channel routing describes the movement of a floodwave (hydrograph) along a watercourse. For most natural rivers, as a floodwave passes through a given reach, the peak of the outflow hydrograph is attenuated and delayed due to flow resistance in the channel and the storage capacity of the river reach. In urban environments, runoff is often conveyed in man made features such as roadways, storm drains and engineered channels that minimize hydrograph attenuation.

Channel routing is used in flood hydrology models, such as HEC-HMS, when the watershed is modeled with multiple subbasins and runoff from the upper subbasins must be translated through a channel or system of channels to the watershed outlet. The channel routing method to be used in HEC-HMS is the Muskingum-Cunge methodology.

The Muskingum-Cunge channel routing is a physically based methodology that solves the continuity and diffusive form of the momentum equation based on the physical channel properties and the inflow hydrograph. The solution procedure involves the discretization of the equations in both time and space (length). The discretized time and distance step size influence the accuracy and stability of the solution. In HEC-HMS the time and distance step size are calculated internally.

F.5.1 Physical Parameters

The physical parameters required for the Muskingum-Cunge channel routing are: reach length, flow resistance factor, friction slope and the channel geometry. One limitation of this method is that it cannot account for the effects of backwater. Therefore, the friction slope should be approximated using the average bed slope. Channel geometry can be one of the following:

- Circular
- Trapezoidal
- Rectangular
- Triangular
- 8 point irregular cross section

Although a circular section can be simulated, the Muskingum-Cunge solution assumes open channel flow conditions regardless of the geometric constraint. If the inflow to the routing reach results in the flow depth exceeding approximately 77% of the diameter, HEC-HMS will report a warning message and the routing results should be checked for reasonableness. In particular, the results should be checked for volume conservation.

When using the 8-point irregular cross section, the cross section must be exactly 8 points. Additionally, the 3rd and 6th point of the cross section defines the break in Manning's n-values for the overbank and channel areas.

F.5.2 Roughness Coefficients

Flow resistance in the channel and overbank flow area is simulated using Manning's roughness coefficients. Flow resistance is affected by many factors including bed material size, bed form, irregularities in the cross section, depth of flow, vegetation, channel alignment, channel shape, obstructions to flow and the quantity of sediment of being transported in suspension or as bed load. In general, all factors that retard flow and increase turbulent mixing tend to increase Manning's n-values. Manning's roughness coefficients appropriate for hydrologic routing are listed in Table F-7 and are, in general, taken from the SSCAFCA Sediment and Erosion Design Guide (MEI, 2008). Use of roughness coefficients other than those listed in Table E-7 must be estimated using the information and procedures in the Sediment and Erosion Design Guide and approved by SSCAFCA.

TABLE F-7. MANNING'S ROUGHNESS COEFFICIENTS	
Channel or Floodplain Type	**n-value**
Sand bed arroyos	0.055
Tined concrete	0.018
Shotcrete	0.025
Reinforced concrete pipe	0.013
Trowled concrete	0.013
No-joint cast-in-place concrete pipe	0.014
Reinforced concrete box	0.015
Reinforced concrete arch	0.015
Streets	0.017
Flush grouted riprap	0.020
Corrugated metal pipe	0.025
Grass-lined channels (sodded & irrigated)	0.025
Earth-lined channels (smooth)	0.030
Wire-tied riprap	0.040
Medium weight dumped riprap	0.045
Grouted riprap (exposed rock)	0.045
Jetty type riprap (D50 > 24")	0.050

F.5.3 Procedure

1. From an appropriate map of the watershed, measure the routing reach length in feet and estimate the friction slope as the channel bed slope in feet per foot.
2. Select a cross sectional geometry that represents that average hydraulic conditions of the reach. If a single cross section cannot be identified that represents the average hydraulic conditions, break the reach into multiple sections and treat each as a unique element in HEC-HMS.
3. Conduct a field reconnaissance of the watershed and routing reaches to observe the flow resistance characteristics.
4. Select an appropriate Manning's roughness coefficient for the channel and overbank flow areas using Table F-7.

F.6 SEDIMENT BULKING

Flow bulking occurs when sediment is eroded from the land surface and entrained into the flowing water. Entrained sediment has the effect of increasing the runoff volume and flow rate. Within this jurisdiction there is potential for high sediment yields. Studies indicate that the sediment yield from undeveloped watersheds can result in bulking factors up to 18%. Similarly, sediment yield from developed areas can result in bulking factors up to 6% for developed conditions. Developed conditions are those areas that have paved roads with curb and gutter. Given the high potential for surface erosion, all watershed models will include flow bulking.

F.6.1 Procedure

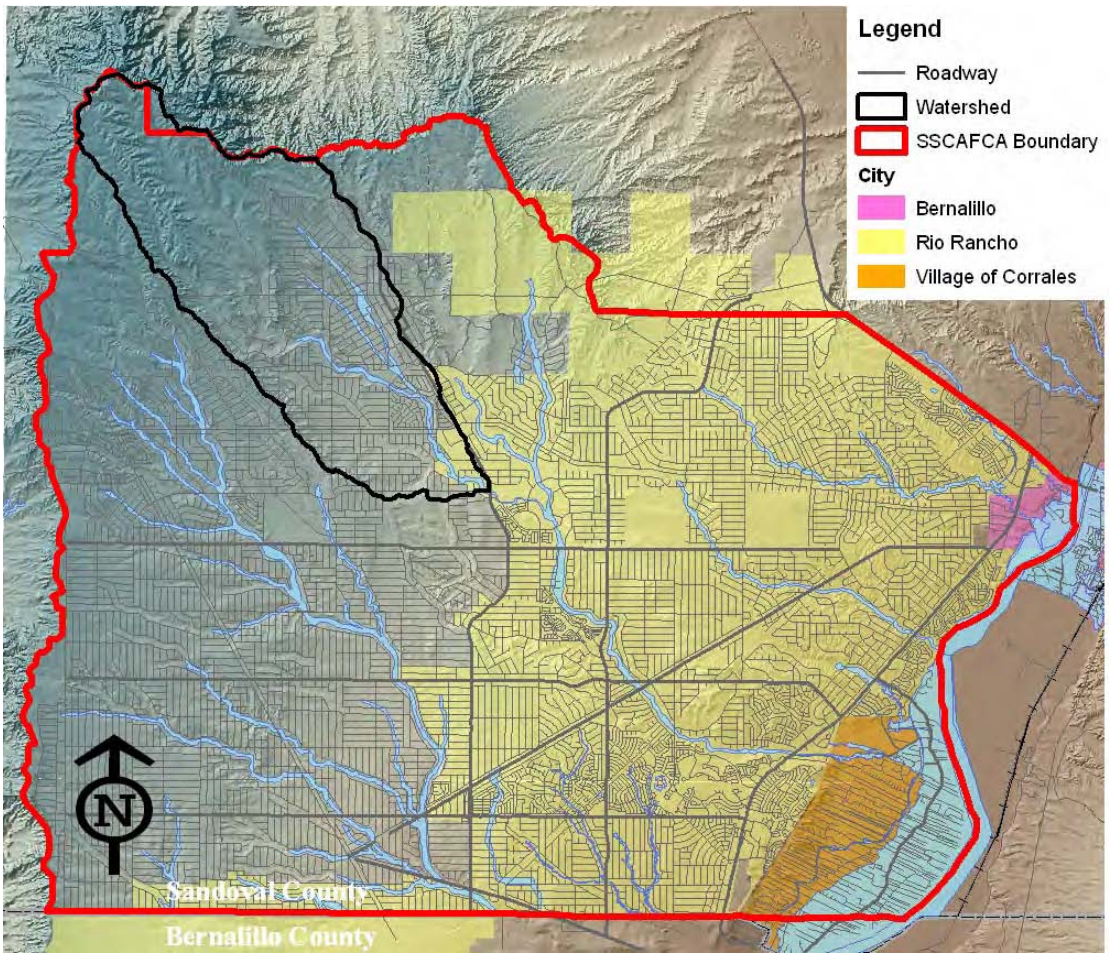
In HEC-HMS, flow bulking for sediment is simulated using a ratio. The ratio is applied to direct runoff estimated for each subbasin. There are two approaches for coding ratios in HEC-HMS. The first is a global assignment. For this option, only one ratio can be applied. Therefore, this option can only be applied to watersheds that are entirely undeveloped or developed. A globally assigned ratio is applied through the computation options for each run.

The second approach for simulating flow bulking due to sediment in HEC-HMS is to apply the appropriate ratio for each subbasin within the watershed. This option is to be used for watersheds with both undeveloped and developed areas.

F.7 HEC-HMS EXAMPLE

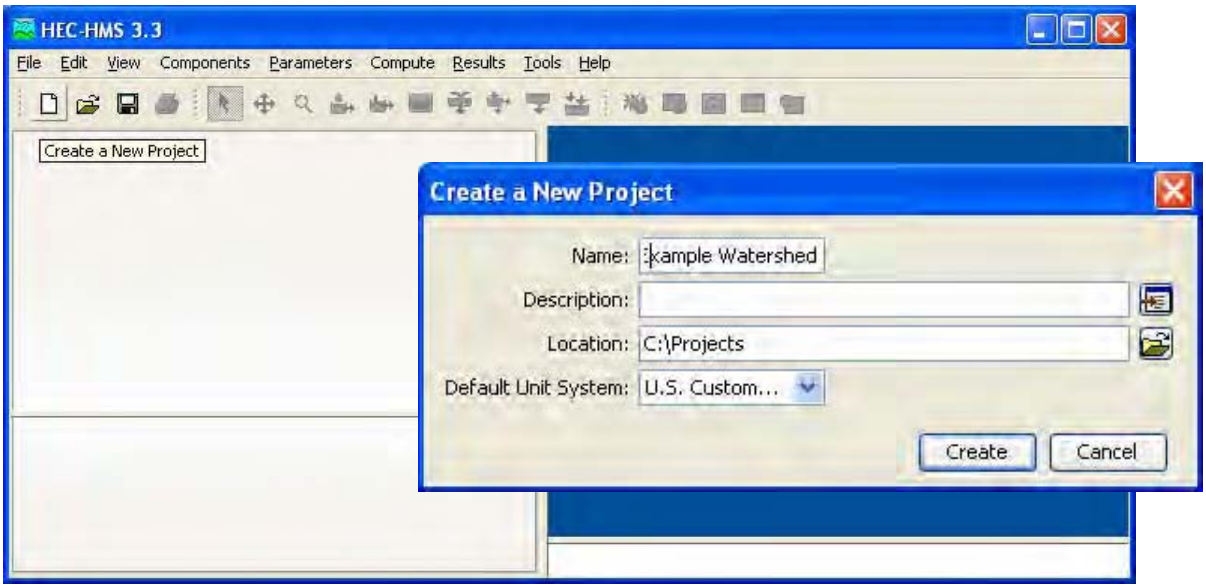
A new roadway crossing is needed for Rainbow Blvd. at Montoyas Arroyo. The new crossing must be designed to covey the 100-year, 6-hour peak flow without overtopping. The contributing drainage area at the roadway crossing is approximately 20.5 square miles. Compute the peak discharge for watershed at Rainbow Blvd.

FIGURE F-8. EXAMPLE WATERSHED MAP

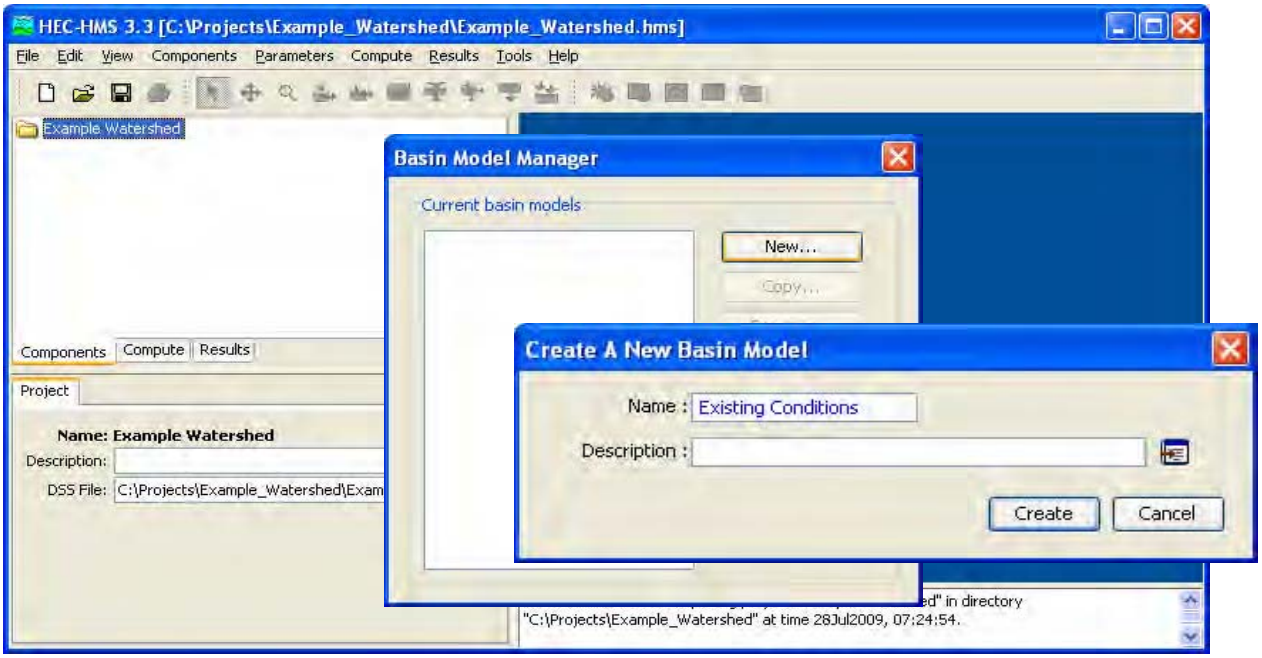


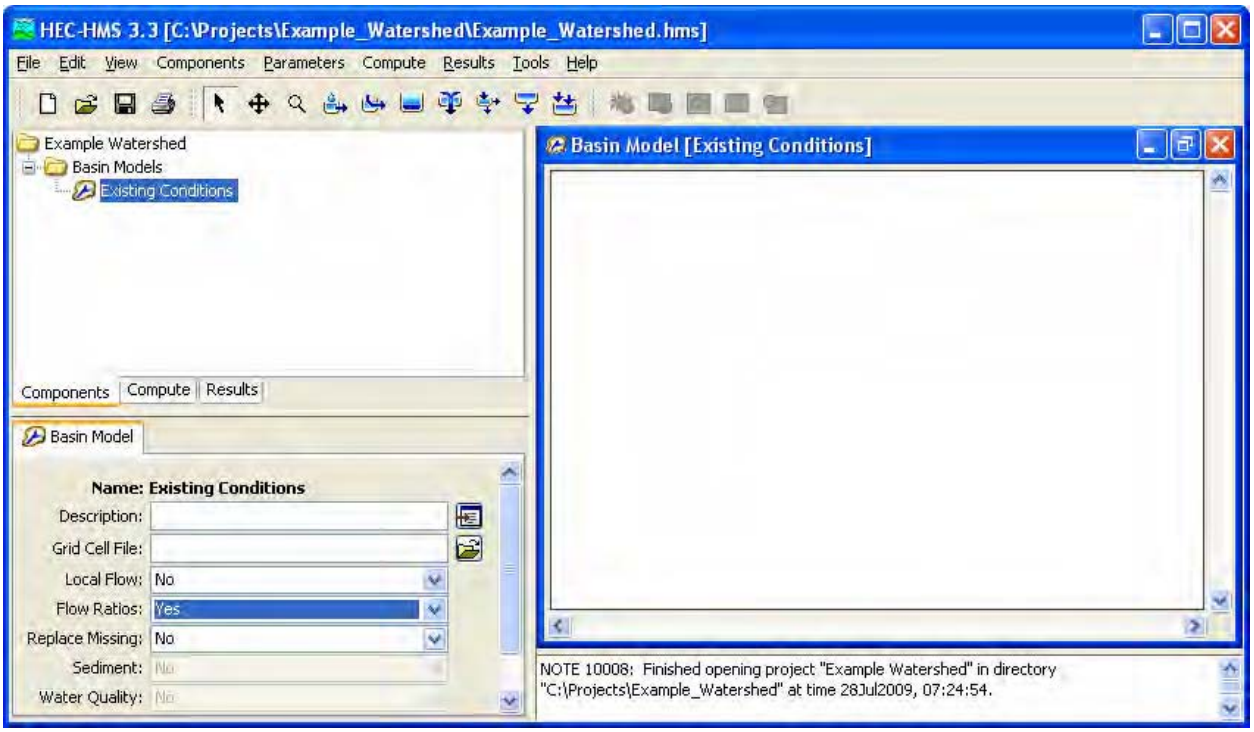
F.8.1 Project Setup

1. Create a new project and provide the following:
    a. Project name (e.g. Example Watershed)
    b. Path to model data
    c. Default system of units

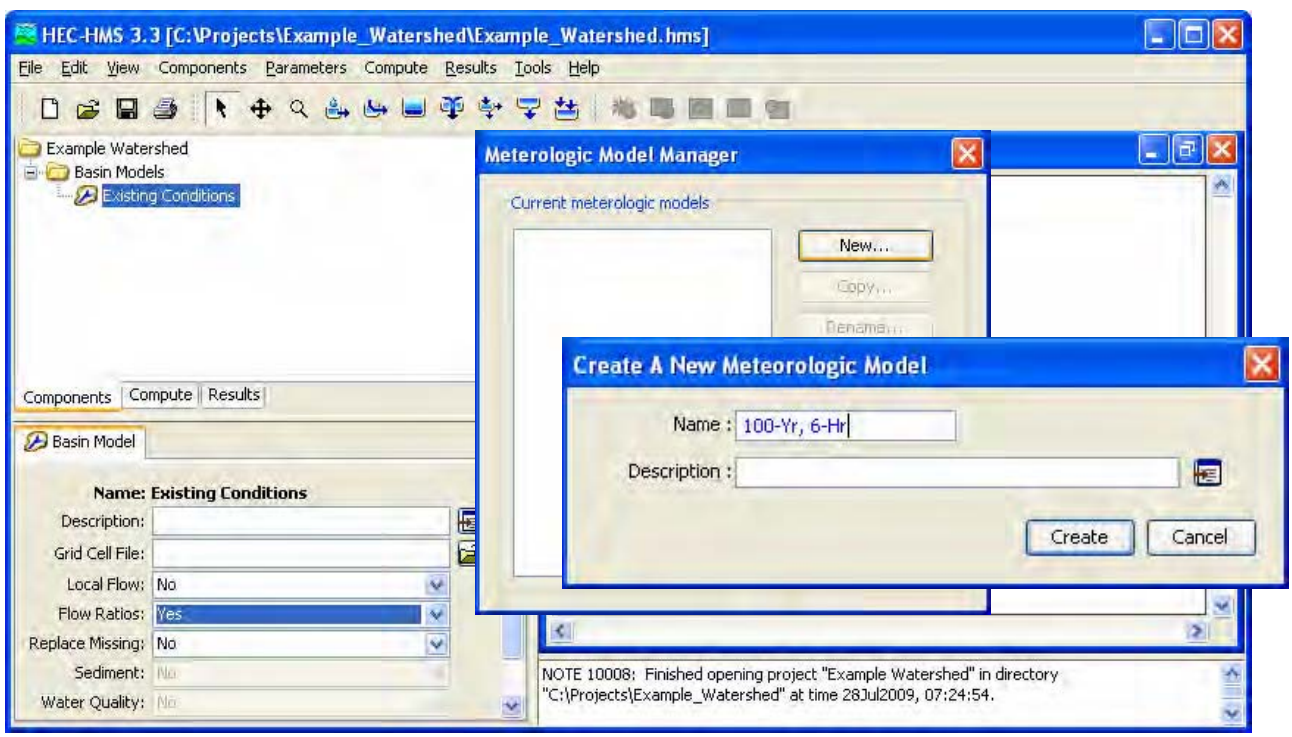


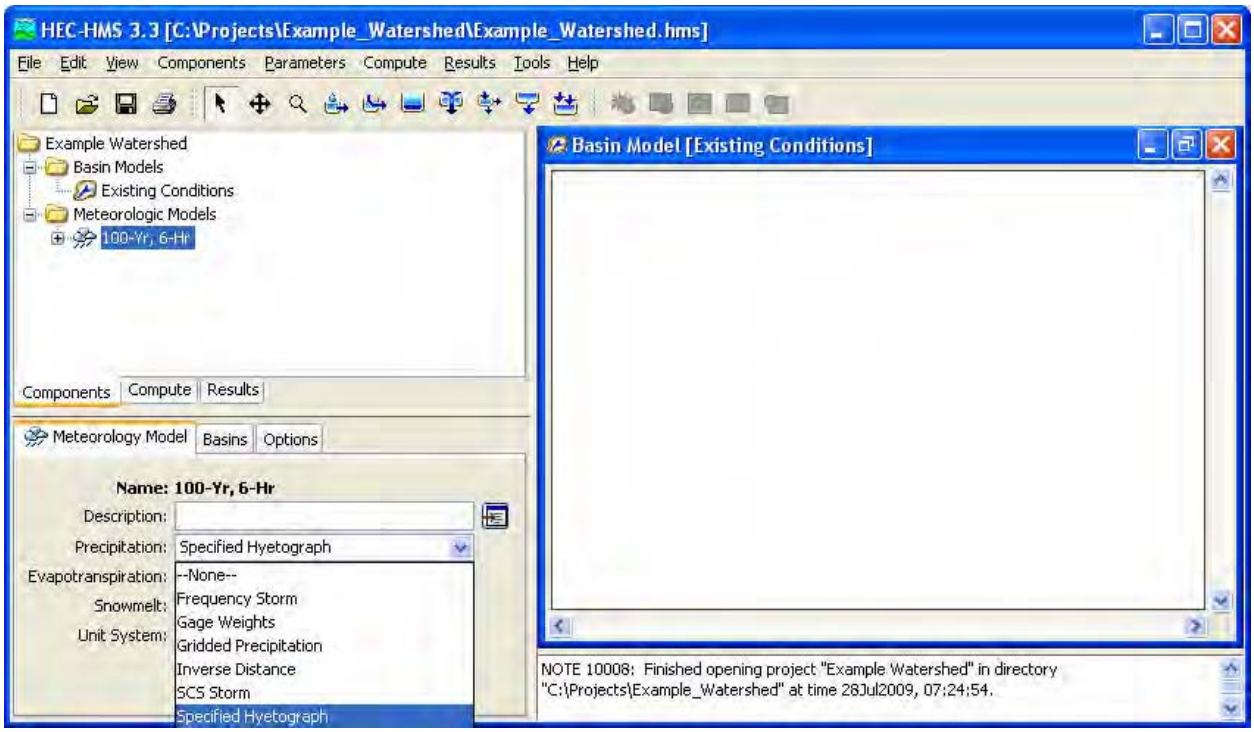
2. Create a Basin Model: From the *Components* pull down menu, select *Basin Model Manager*
    a. Select *New*
    b. Enter a name for the basin model (e.g. Existing Conditions)
    c. In the *Component Editor*, select "Yes" in the *Flow Ratio* list box



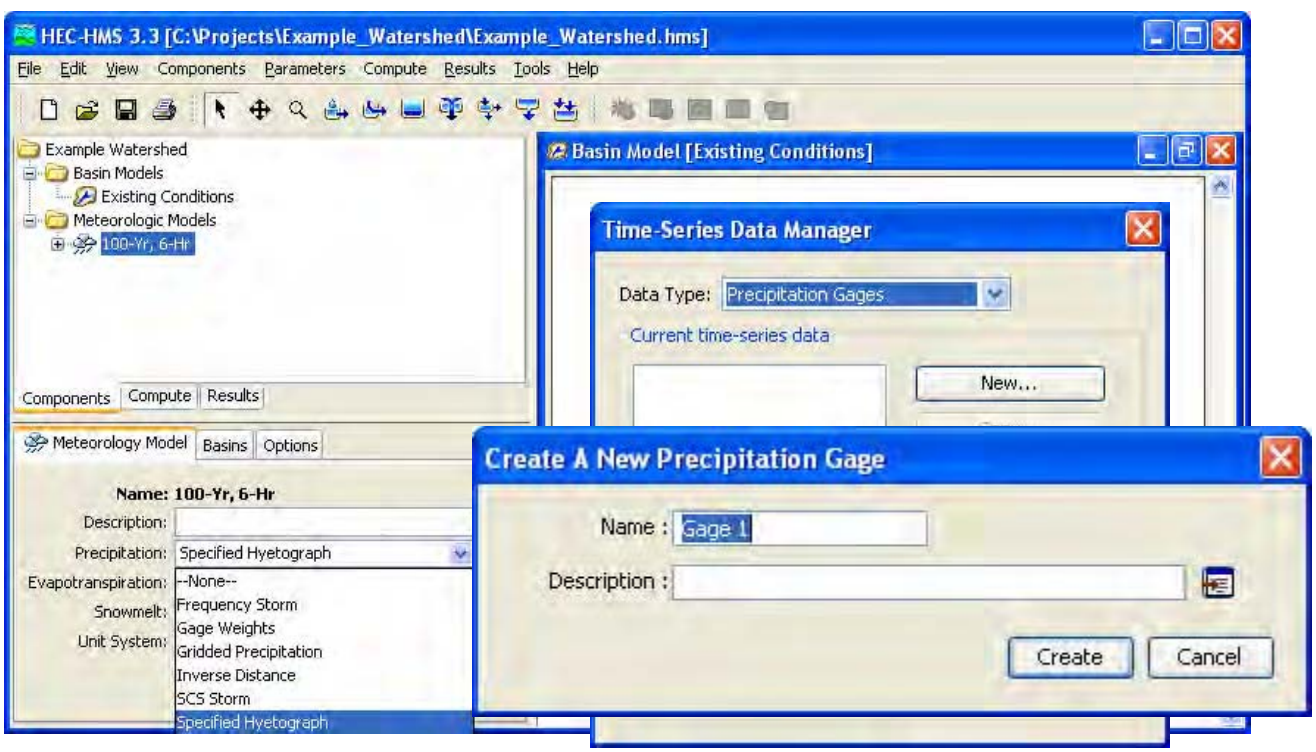


3. Create a Meteorologic Model: From the *Components* pull down menu, select *Meteorologic Model Manager*
    a. Select *New*
    b. Enter a name for the meteorologic model (e.g. 100-Yr, 6-Hr)
    c. In the *Component Editor*, select "Specified Hyetograph" in the *Precipitation* list box





4. Create a precipitation gage: from the *Components* pull down menu, select *Time-Series Data Manager*
    a. With the *Data Type* set to "Precipitation Gages", select *New*
    b. Assign a name for the gage (e.g. Gage-1)



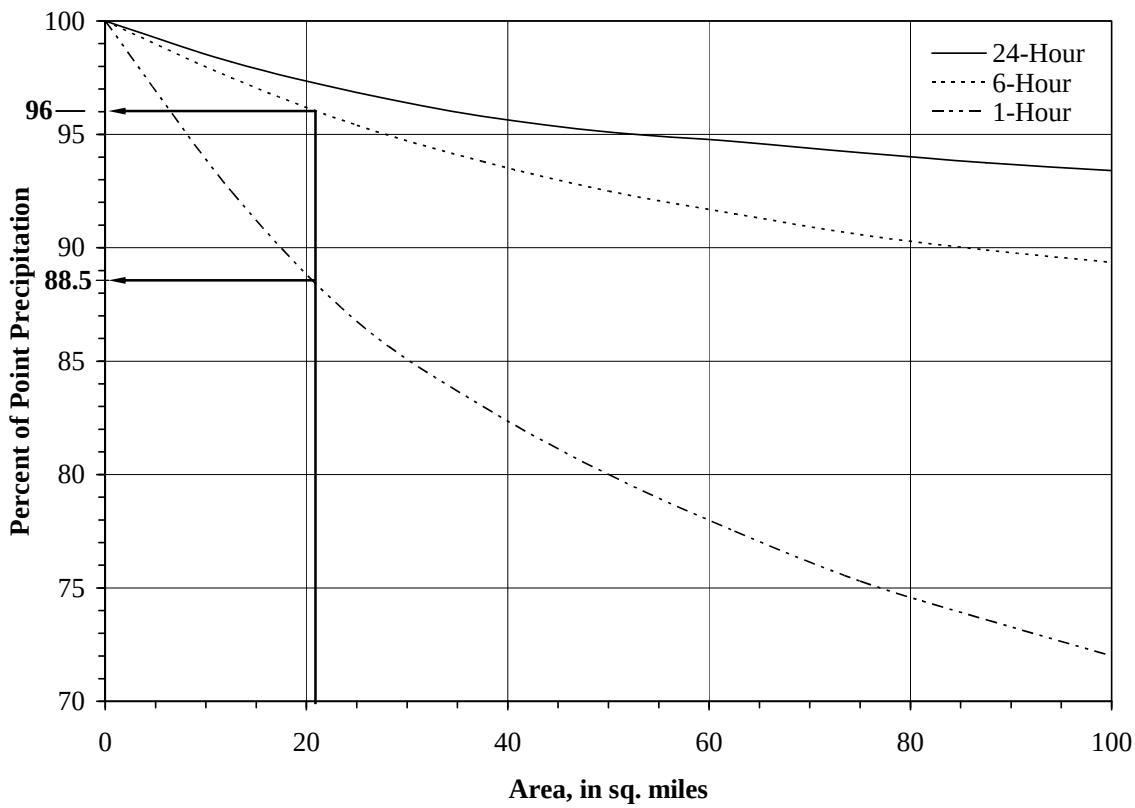
F.8.2 Design Rainfall

Determine the 100-year, 6-hour rainfall data for the watershed, plot the rainfall hyetograph and code the data into the HEC-HMS project.

1. 100-year point rainfall depths taken from Table F-1 are:
   - 100-year, 1-hour = 1.84 inches
   - 100-year, 6-hour = 2.37 inches

2. Estimate depth-area reduction factors for the watershed area of 20.5 square miles using Figure F-1.

FIGURE F-9. EXAMPLE WATERSHED DEPTH-AREA REDUCTION



3. Calculate the equivalent uniform rainfall depth

$$P_1^{100} = (1.84)(0.885) = 1.63 \text{ inches}$$

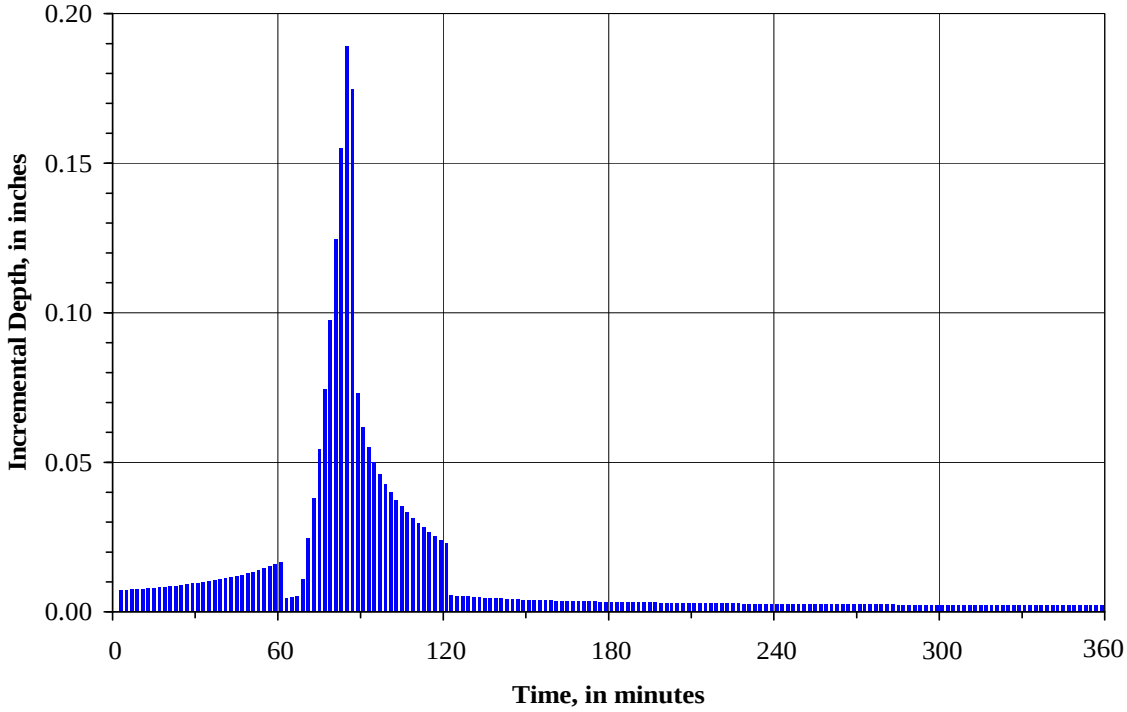
$$P_6^{100} = (2.37)(0.960) = 2.28 \text{ inches}$$

4. Calculate the cumulative rainfall mass curve using Equations F-1 through F-5 for the 6-hour storm. The computation time interval is 2 minutes.

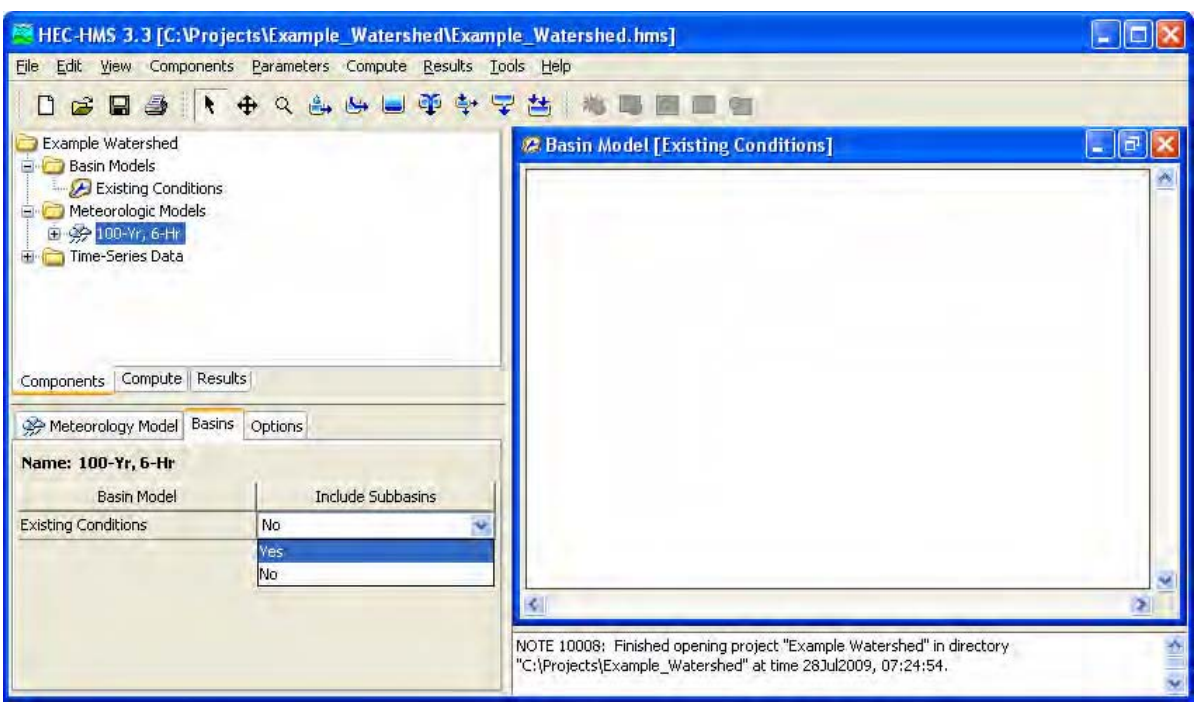
TABLE F-8. CUMULATIVE RAINFALL DISTRIBUTION

Time min	Rainfall inches	Time min	Rainfall inches	Time min	Rainfall inches	Time min	Rainfall inches
0	0.000	92	1.459	184	2.065	276	2.190
2	0.007	94	1.509	186	2.068	278	2.192
4	0.014	96	1.555	188	2.071	280	2.194
6	0.021	98	1.598	190	2.074	282	2.197
8	0.028	100	1.638	192	2.078	284	2.199
10	0.036	102	1.676	194	2.081	286	2.201
12	0.043	104	1.712	196	2.084	288	2.203
14	0.051	106	1.745	198	2.087	290	2.206
16	0.059	108	1.777	200	2.090	292	2.208
18	0.067	110	1.807	202	2.093	294	2.210
20	0.075	112	1.835	204	2.096	296	2.213
22	0.084	114	1.862	206	2.099	298	2.215
24	0.092	116	1.887	208	2.101	300	2.217
26	0.101	118	1.911	210	2.104	302	2.219
28	0.110	120	1.934	212	2.107	304	2.221
30	0.120	122	1.940	214	2.110	306	2.224
32	0.129	124	1.945	216	2.113	308	2.226
34	0.139	126	1.951	218	2.116	310	2.228
36	0.149	128	1.956	220	2.118	312	2.230
38	0.160	130	1.961	222	2.121	314	2.232
40	0.171	132	1.965	224	2.124	316	2.235
42	0.182	134	1.970	226	2.127	318	2.237
44	0.193	136	1.975	228	2.129	320	2.239
46	0.205	138	1.979	230	2.132	322	2.241
48	0.218	140	1.984	232	2.135	324	2.243
50	0.231	142	1.988	234	2.137	326	2.245
52	0.244	144	1.992	236	2.140	328	2.247
54	0.258	146	1.996	238	2.143	330	2.249
56	0.273	148	2.000	240	2.145	332	2.251
58	0.288	150	2.004	242	2.148	334	2.254
60	0.304	152	2.008	244	2.150	336	2.256
62	0.309	154	2.012	246	2.153	338	2.258
64	0.314	156	2.016	248	2.155	340	2.260
66	0.319	158	2.020	250	2.158	342	2.262
68	0.330	160	2.024	252	2.160	344	2.264
70	0.355	162	2.027	254	2.163	346	2.266
72	0.393	164	2.031	256	2.165	348	2.268
74	0.448	166	2.035	258	2.168	350	2.270
76	0.522	168	2.038	260	2.170	352	2.272
78	0.620	170	2.042	262	2.173	354	2.274
80	0.746	172	2.045	264	2.175	356	2.276
82	0.902	174	2.048	266	2.178	358	2.278
84	1.092	176	2.052	268	2.180	360	2.280
86	1.268	178	2.055	270	2.182		
88	1.341	180	2.058	272	2.185		
90	1.403	182	2.062	274	2.187		

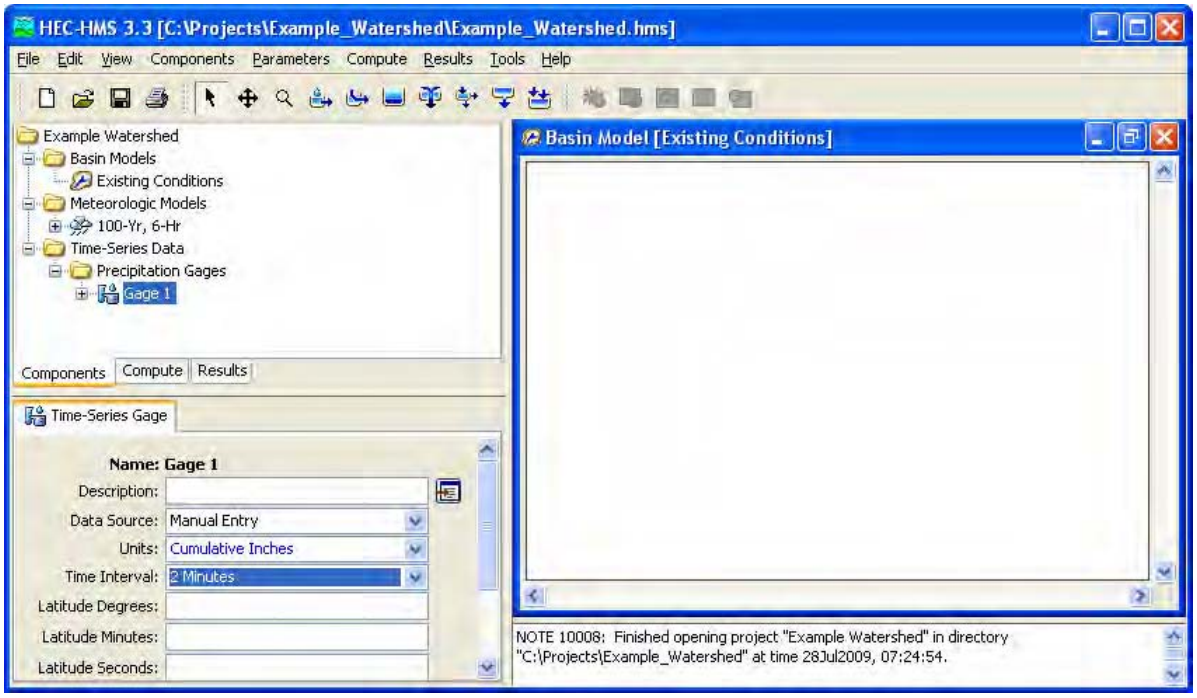
FIGURE F-10. 100-YEAR, 6-HOUR RAINFALL HYETOGRAPH



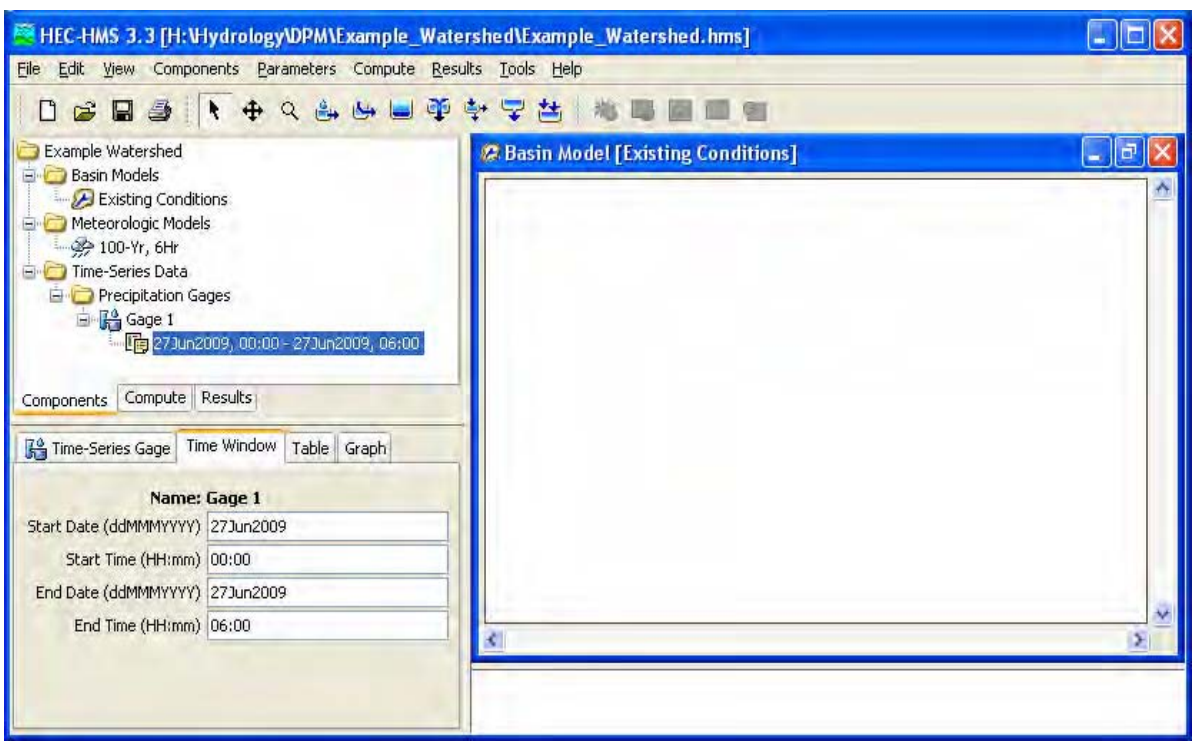
5. Code the cumulative rainfall data into HEC-HMS
    a. In the *Basin* tab of the *Component Editor* for the "100-Yr, 6-Hr" precipitation model, toggle on the "Include Subbasins" option



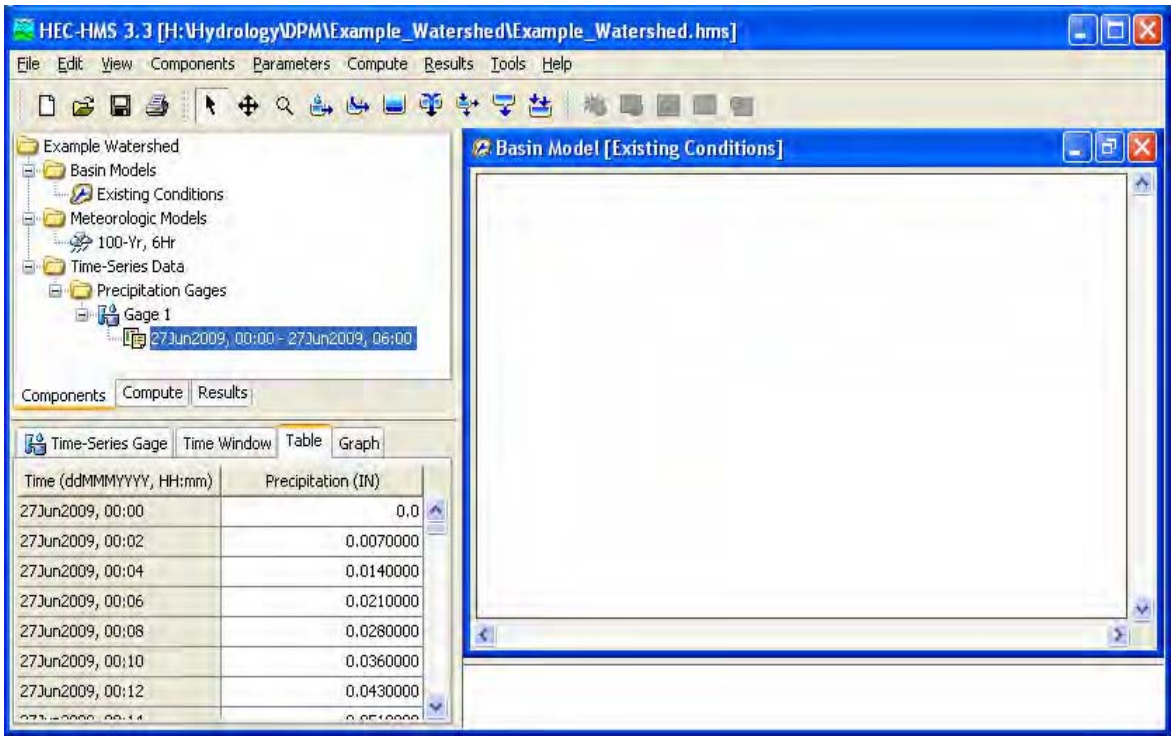
b. In the *Component Editor* for "Gage-1", set the following:
   i. Units = Cumulative Inches
   ii. Time Interval = 2 Minutes



c. Set the time duration of rainfall

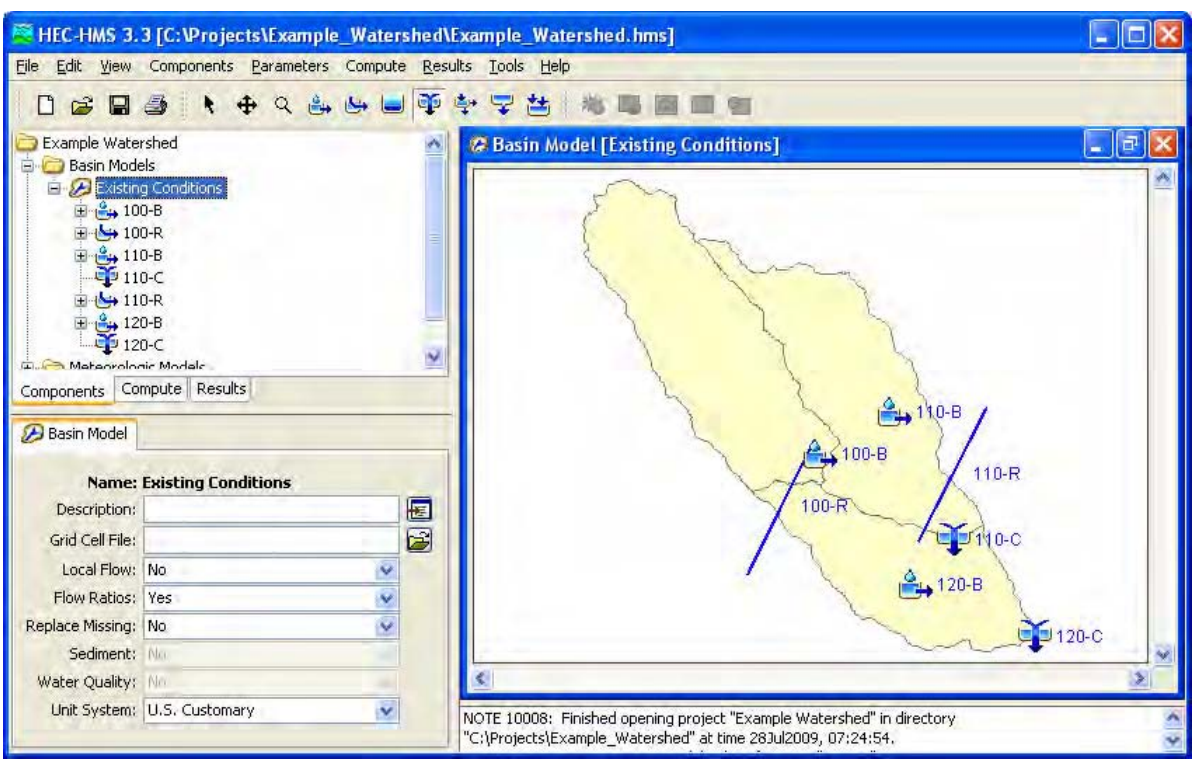


d. Cut and paste the cumulative rainfall data from Table F-8

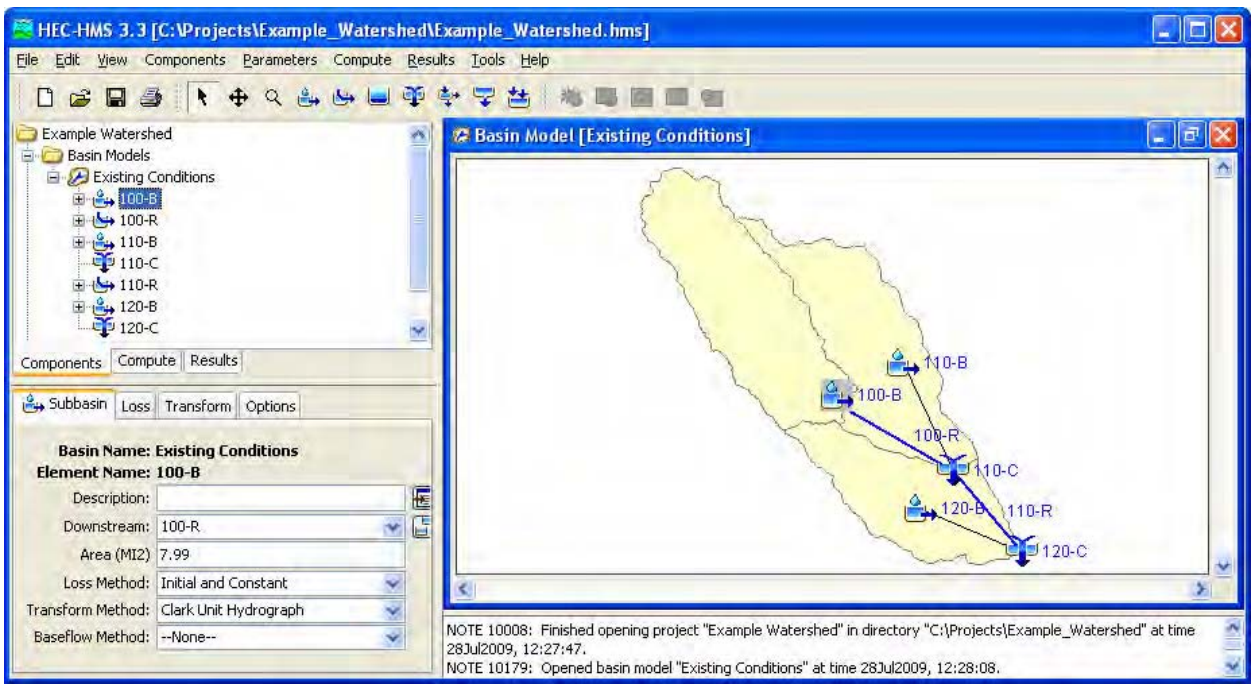


F.8.3 Basin Data

1. Build watershed schematic in the HEC-HMS Desktop using the watershed icons for each element.



2. Set the default methodologies for subbasin and channel routing elements
    a. From the *Parameters* pull down menu
        i. Select *Subbasin Methods*
            - Select *Loss* and set the Method to “Initial and Constant”
            - Select *Transform* and set the Method to “Clark Unit Hydrograph”
            - Select *Baseflow* and set the Method to “None”
        ii. Select *Reach Methods*
            - Select *Routing* and set the Method to “Muskingum-Cunge”
            - Select *Loss/Gain* and set the Method to “None”
3. Code in subbasin areas and set downstream connectivity



F.8.4 Rainfall Loss Parameters

Compute the subbasin average rainfall loss parameters and code the values into the HEC-HMS project for the watershed. Existing condition land use within the watershed is illustrated in Figure F-11. The areas for each unique land use type with each subbasin are listed in the following Table F-9.

FIGURE F-11. EXAMPLE WATERSHED LAND USE CONDITIONS

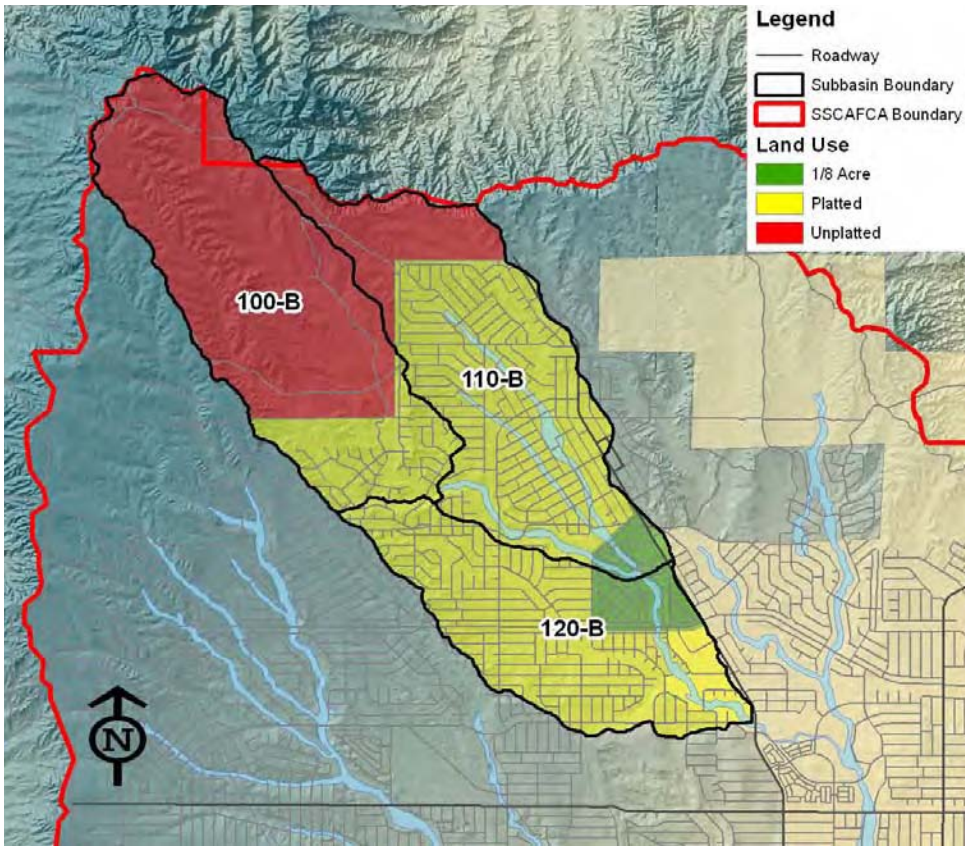


TABLE F-9 EXAMPLE WATERSHED LAND USE DATA

Parcel	**Area, in sq. miles**			**Total Area**
Description	**100-B**	**110-B**	**120-B**	**sq. miles**
1/8 Acre	---	0.38	0.62	1.00
Platted	1.60	4.58	5.69	11.87
Unplatted	6.39	1.24	---	7.63
Total	7.99	6.20	6.31	20.50

1. From Table F-3, percentage of Land Treatment Types for each parcel within the watershed are:

Parcel Description	Percent of Land Treatment Type			
	A	**B**	**C**	**D**
1/8 Acre	0	15	15	70
Platted	18.7	29.5	27.0	24.8
Unplatted	95	5	0	0

2. Calculate the area of each Land Treatment type within each subbasin by multiplying the area of each parcel type by the percent of Land Treatment type, for example:

   For subbasin 100-B, the area of each Land Treatment type is as follows:

   $Area_A = (0)(0\%) + (1.6)(18.7\%) + (6.39)(95\%) =$ 6.37 sq. miles

   $Area_B = (0)(15\%) + (1.6)(29.5\%) + (6.39)(5\%) =$ 0.79 sq. miles

   $Area_C = (0)(15\%) + (1.6)(27.0\%) + (6.39)(0\%) =$ 0.43 sq. miles

   $Area_D = (0)(70\%) + (1.6)(24.8\%) + (6.39)(0\%) =$ 0.40 sq. miles

   Total Area = 7.99 sq. miles

   Therefore, the area of each Land Treatment type for each subbasin is as follows:

Subbasin ID	Land Treatment Type Area, in sq. miles				Total Area sq. miles
	A	**B**	**C**	**D**	
100-B	6.37	0.79	0.43	0.40	7.99
110-B	2.03	1.47	1.30	1.40	6.20
120-B	1.06	1.77	1.63	1.85	6.31
Total	9.46	4.03	3.36	3.65	20.50

3. Using values of IA from Table F-4, calculate the weighted value of IA for each subbasin, for example:

   For subbasin 100-B, the area weighted IA is calculated as follows:

$$IA = \frac{(6.37)(0.65)+(0.79)(0.50)+(0.43)(0.35)}{6.37+0.79+0.43} = 0.62$$

   Therefore, the area weigthed IA for each subbasin is as follows:

Subbasin ID	IA inches
100-B	0.62
110-B	0.52
120-B	0.48

4. Using values of INF from Table F-5, calculate the weighted value of INF for each subbasin, for example:

   For subbasin 100-B, the area weighted INF is calculated as follows:

$$IA = \frac{(6.37)(1.67)+(0.79)(1.25)+(0.43)(0.83)}{6.37+0.79+0.43} = 1.58$$

   Therefore, the area weigthed INF for each subbasin is as follows:

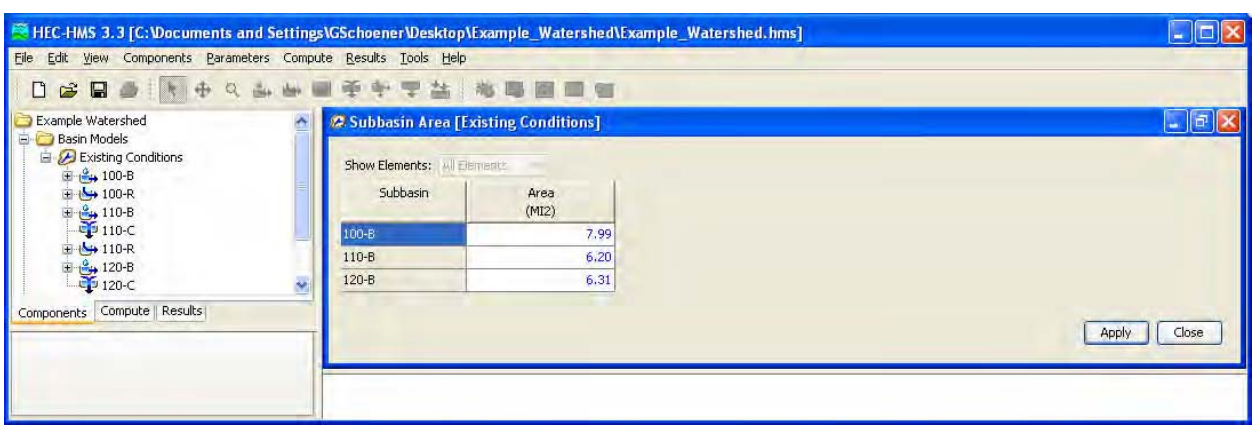
Subbasin ID	INF in/hr
100-B	1.58
110-B	1.32
120-B	1.20

5. Using the area of Land Treatment Type D, compute the impervious area percentage for each subbasin

Subbasin ID	Impervious Area %
100-B	5.0
110-B	22.6
120-B	29.3

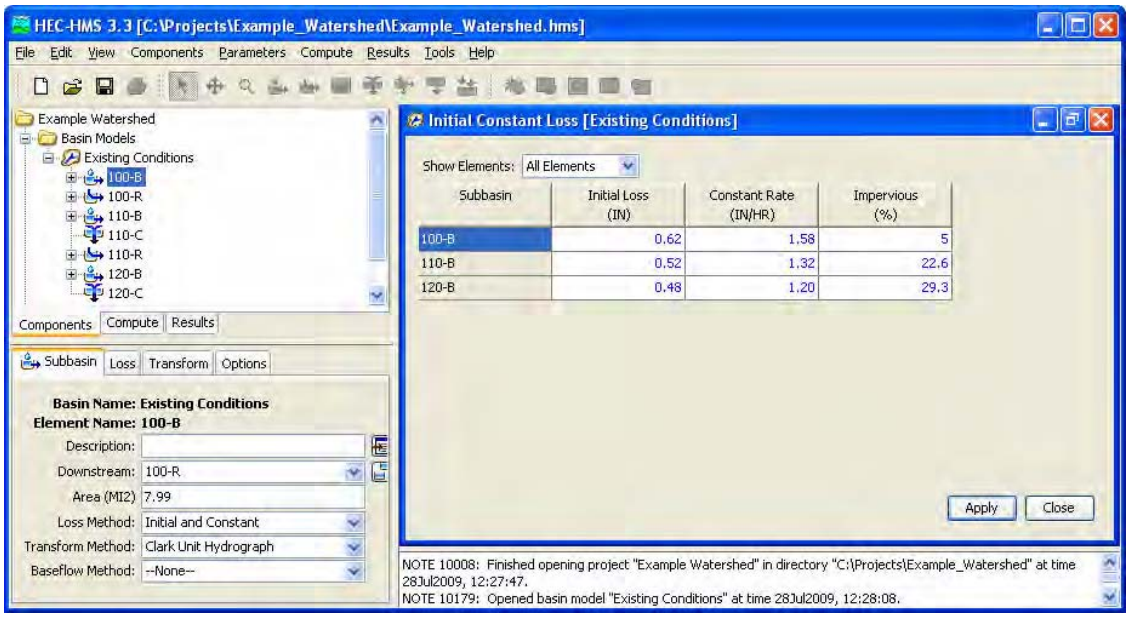
6. Code subbasin areas in HEC-HMS: from the Parameters pull down menu

   a. Select Subbasin Area
   b. Code area for each subbasin



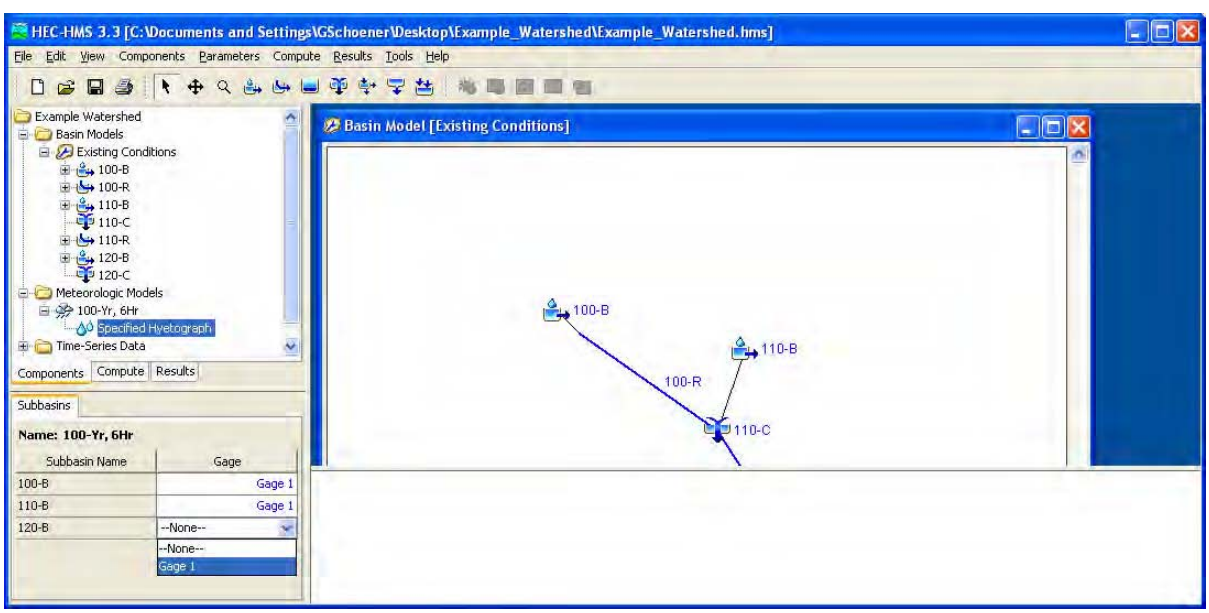
7. Code the rainfall loss parameters in HEC-HMS: from the Parameters pull down menu

    a. Select *Loss* and then *Initial and Constant*
    b. Select "All Elements"
    c. Code in the rainfall loss parameters for each subbasin



8. Assign rainfall data to each subbasin.

    a. Select *Specified Hyetograph* under *Meteorologic Models*.
    b. In the *Component Editor* for the *Specified Hyetograph*, select *Gage 1* for each of the three subbasins.



F.8.5 Unit Hydrograph Parameters

Compute the Clark unit hydrograph parameters and code the values into the HEC-HMS project for the watershed. Runoff from each subbasin should account for sediment bulking. Time of Concentration (Tc) flow paths, subbasin centroid locations and Lca flow paths for each subbasin are illustrated in Figure F-12. The physical data for calculation of the Clark unit hydrograph parameters for each subbasin are listed in Table F-9.

FIGURE F-12. EXAMPLE WATERSHED FLOW PATHS

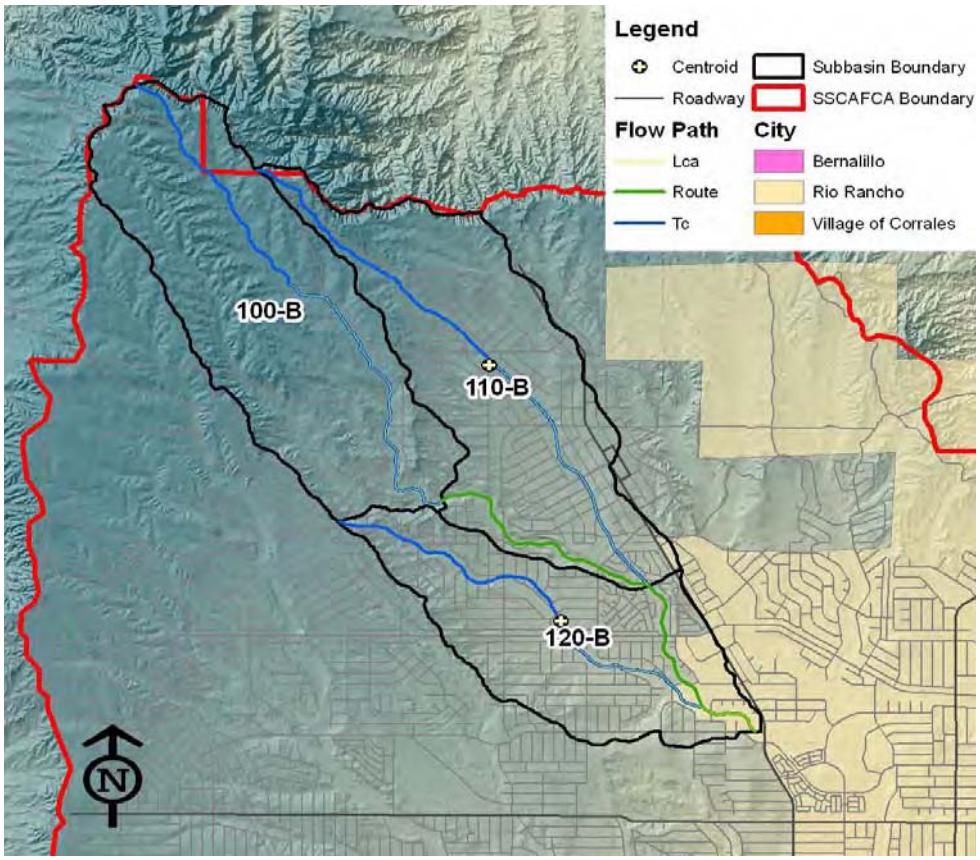


TABLE F-10 EXAMPLE WATERSHED FLOW PATH DATA

Subbasin ID	Flow Path Length L miles	Flow Path Length L_{ca} miles	Slope ft/ft
100-B	5.15	2.71	0.0185
110-B	5.81	2.68	0.0171
120-B	4.92	2.35	0.0165

1. Calculate the T_c for each subbasin

   The Tc flow path length is greater than 12,000 feet for all subbasins, therefore use Equation F-8 and select a value of K_n from Table F-6.

   The majority of each subbaisn is undeveloped land, either platted or unplatted, therefore assume a value of K_n of 0.033 for all subbasins.

   Using Equation F-8, T_c for subbasin 100-B is:

$$T_c = \frac{8}{9} * 26(0.033) * \left(\frac{27{,}192 * 14{,}309}{5280^2 * \sqrt{5280 * 0.0185}} \right)^{0.33} = 0.855 \text{ hrs}$$

   Using Equation F-8, T_c for subbasin 110-B is:

$$T_c = \frac{8}{9} * 26(0.033) * \left(\frac{30{,}677 * 14{,}150}{5280^2 * \sqrt{5280 * 0.0171}} \right)^{0.33} = 0.899 \text{ hrs}$$

   Using Equation F-8, T_c for subbasin 120-B is:

$$T_c = \frac{8}{9} * 26(0.033) * \left(\frac{25{,}998 * 12{,}408}{5280^2 * \sqrt{5280 * 0.0165}} \right)^{0.33} = 0.819 \text{ hrs}$$

2. Calculate the Storage Coefficient (R) for each subbasin using Equation F-13 and the results from Example Problem No. 2

   Using Equation F-13, R for subbasin 100-B is:

$$R = 1.165 * 0.855 * \left(1.58^{0.45} - 0.62^{1.4} \left(\frac{5}{100} \right)^{0.40} \right) \qquad = 1.070 \text{ hrs}$$

   Using Equation F-13, R for subbasin 110-B is:

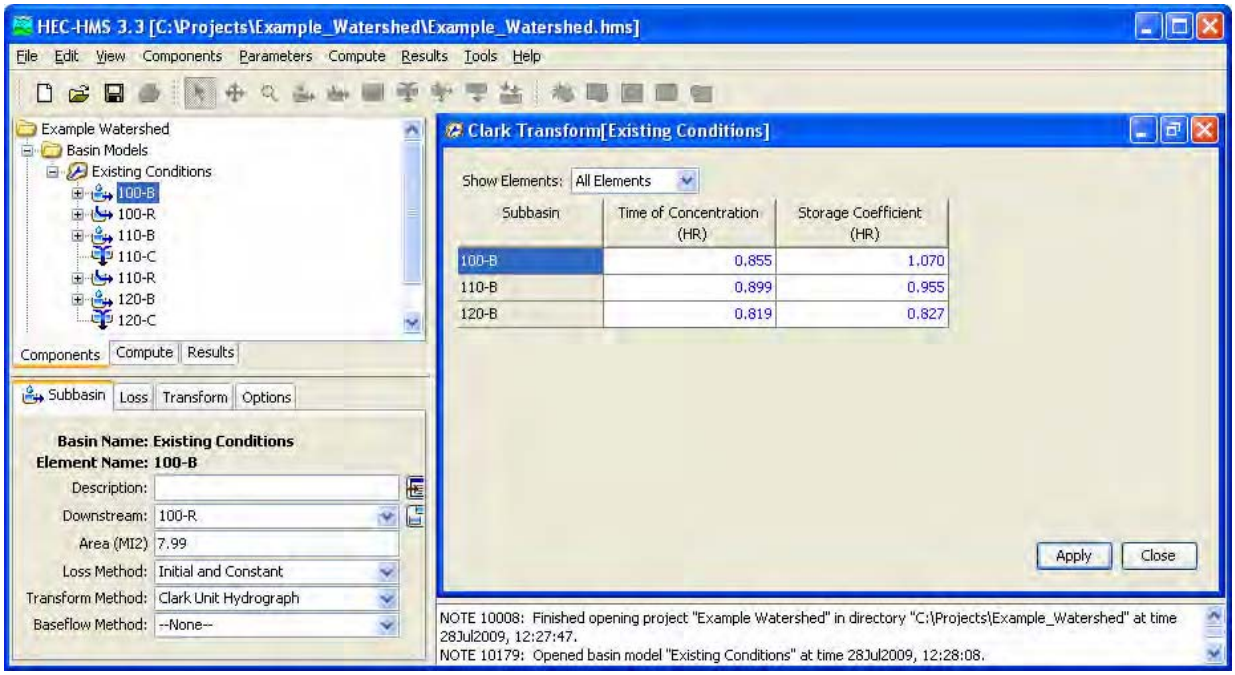
$$R = 1.165 * 0.899 * \left(1.32^{0.45} - 0.52^{1.4} \left(\frac{22.6}{100} \right)^{0.40} \right) \qquad = 0.955 \text{ hrs}$$

   Using Equation F-13, R for subbasin 120-B is:

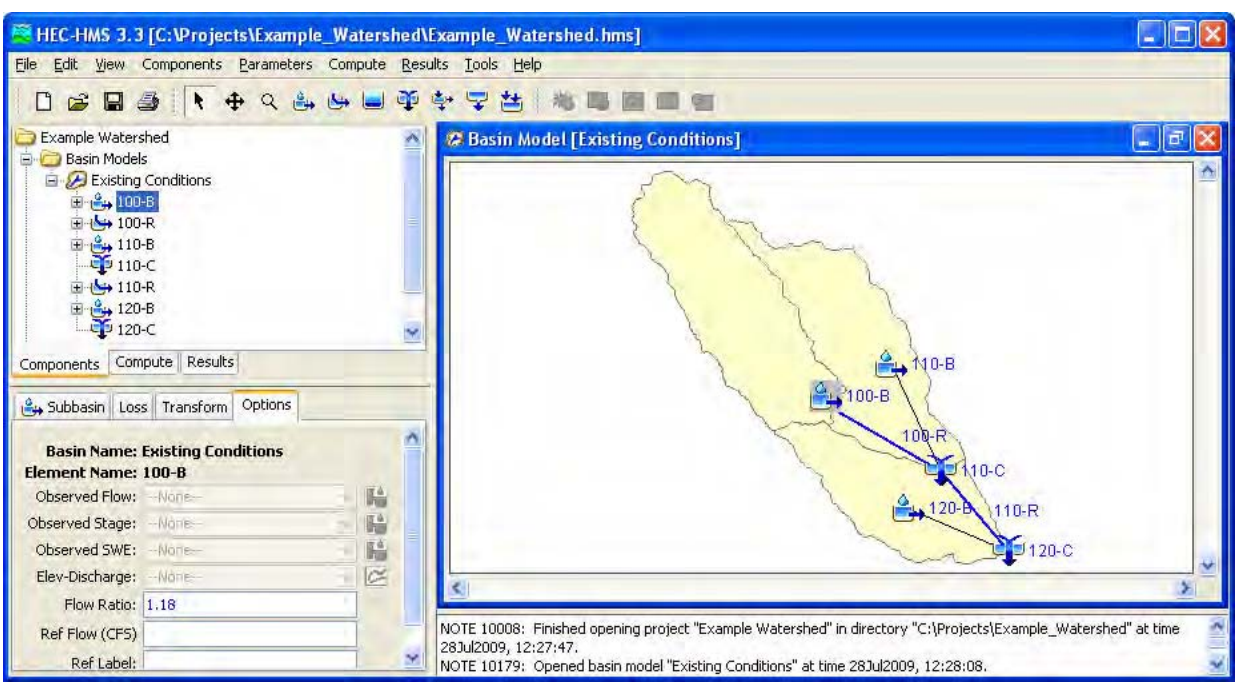
$$R = 1.165 * 0.819 * \left(1.20^{0.45} - 0.48^{1.4} \left(\frac{29.3}{100} \right)^{0.40} \right) \qquad = 0.827 \text{ hrs}$$

3. Assign sediment bulking factors for each subbasin based on the guidance in Section F.6. Since the majority of all three subbasins are undeveloped, but platted lands, use a sediment bulking factor of 18% for all subbasins.

4. Code the Clark unit hydrograph parameters in HEC-HMS: from the *Parameters* pull down menu
    a. Select *Transform* and then Clark Unit Hydrograph
    b. Select "All Elements"
    c. Code in the rainfall loss parameters for each subbasin



5. Code in the sediment bulking ratio: on the *Options* tab in the *Component Editor* for each subbasin, code in 1.18 as the flow ratio



F.8.6 Channel Routing Parameters

Develop the Muskingum-Cunge channel routing data and code that data into the HEC-HMS project for the watershed, execute the model and summarize the results. Routing reaches for the watershed are illustrated in Figure F-13. The physical data for routing reach is listed in Table F-11. Cross sections typical of the geometry for each reach are shown in Figures F-14 and F-15 for Routing Reach 100-R and 110-R, respectively.

FIGURE F-13. EXAMPLE WATERSHED ROUTING REACHES

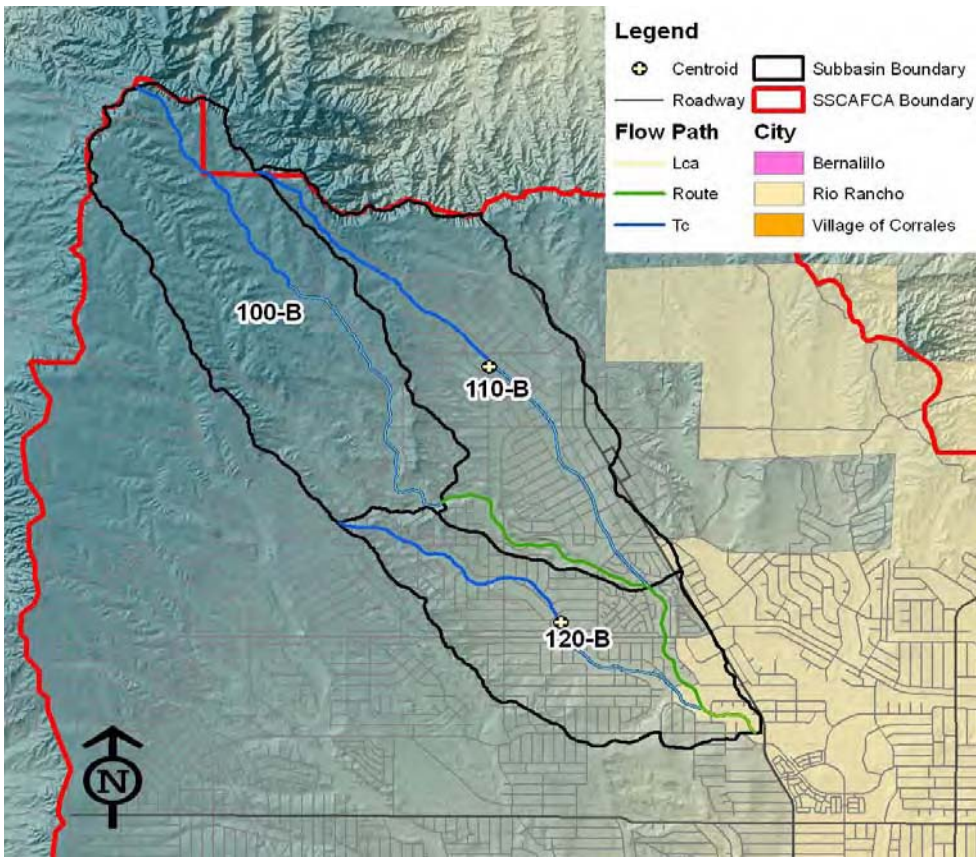


TABLE F-11 EXAMPLE WATERSHED CHANNEL ROUTING DATA

Reach ID	Reach from Subbasin	Length feet	Slope ft/ft
100-R	100-B	11,263	0.0165
110-R	120-B	9,685	0.0158

FIGURE F-14. REACH 100-R CROSS SECTIONAL GEOMETRY

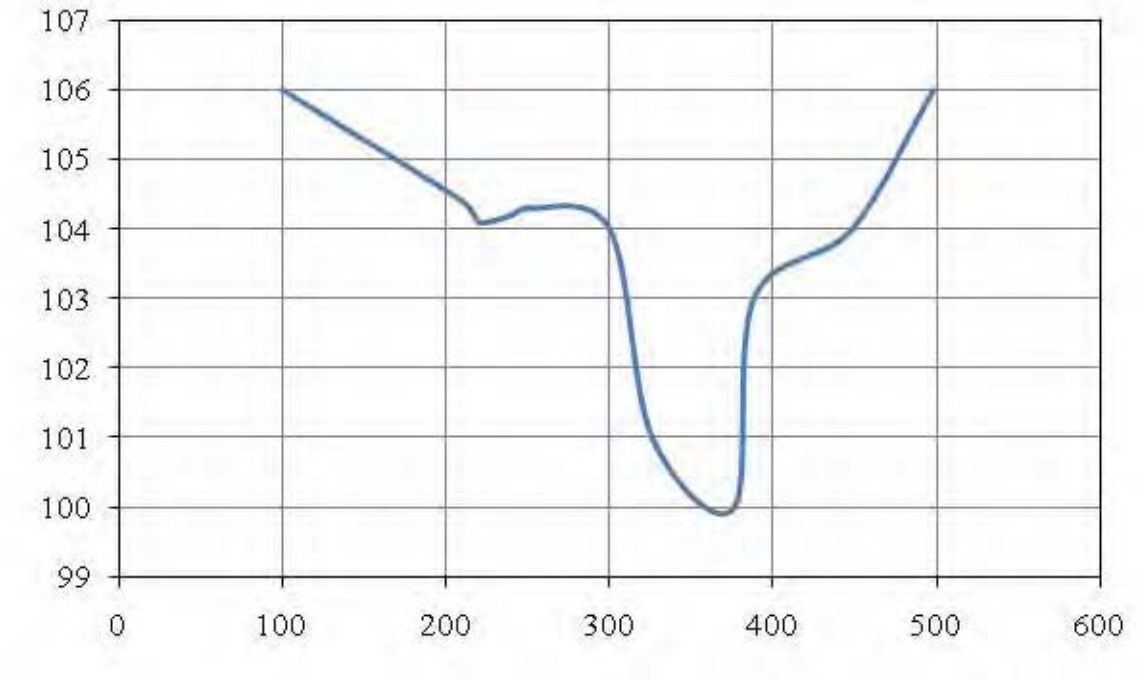
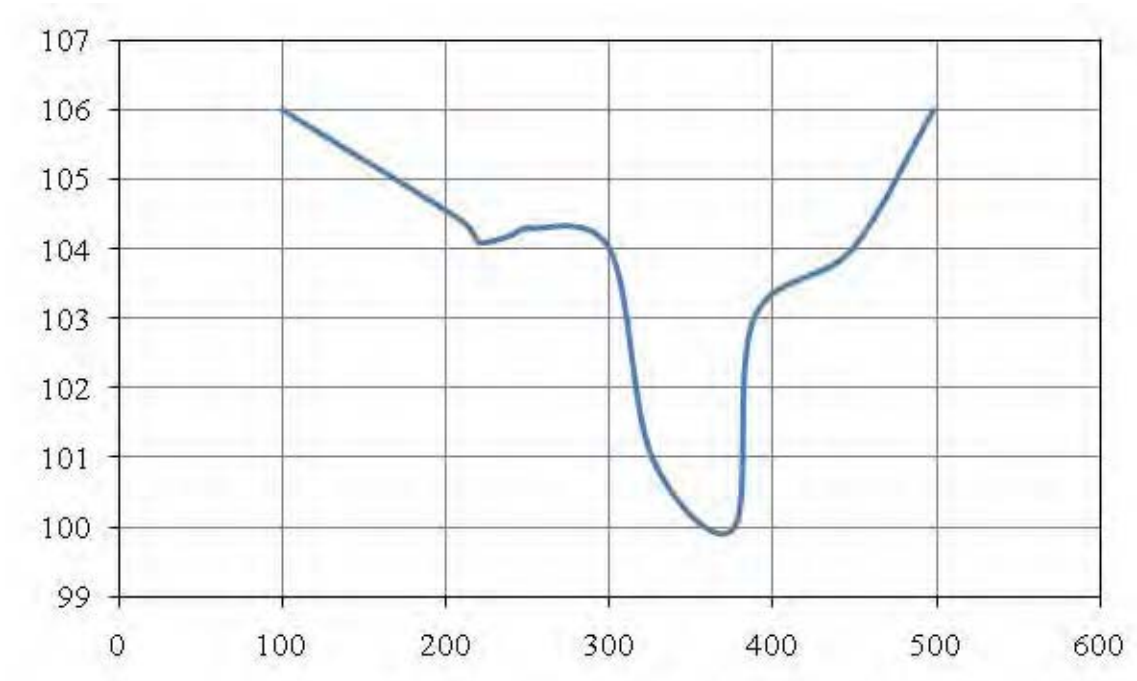


FIGURE F-15. REACH 110-R CROSS SECTIONAL GEOMETRY



1. Simplify the channel geometry for each reach into an 8-point irregular section.

FIGURE F-16. 8-POINT GEOMETRY FOR REACH 100-R

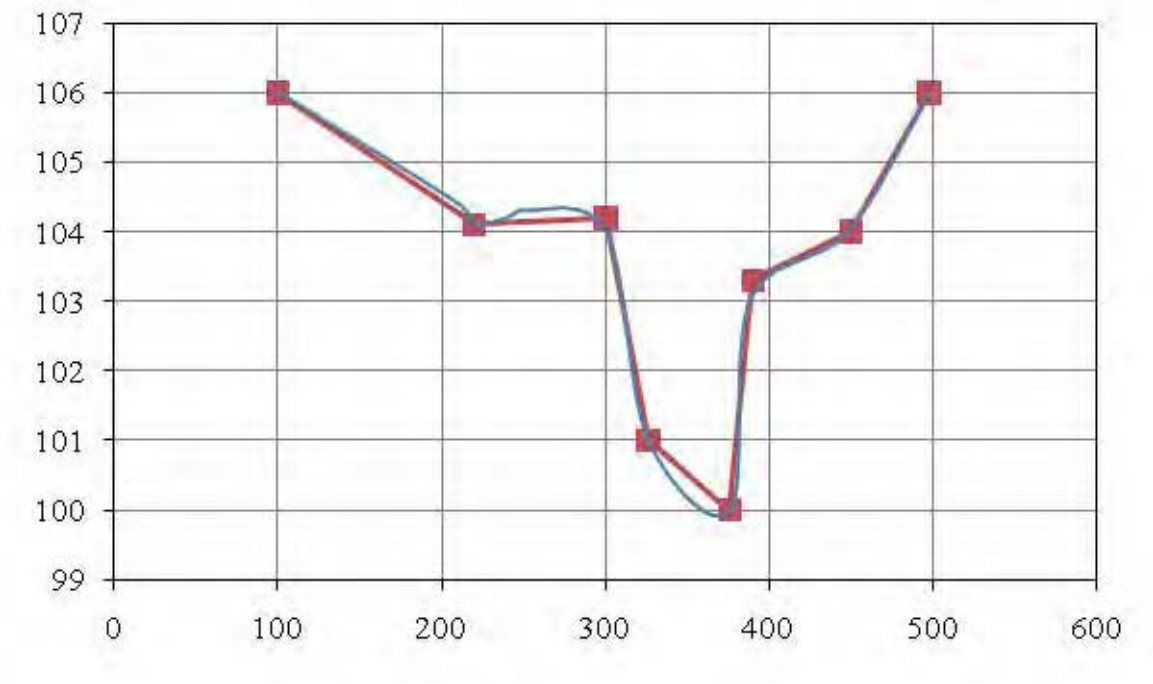
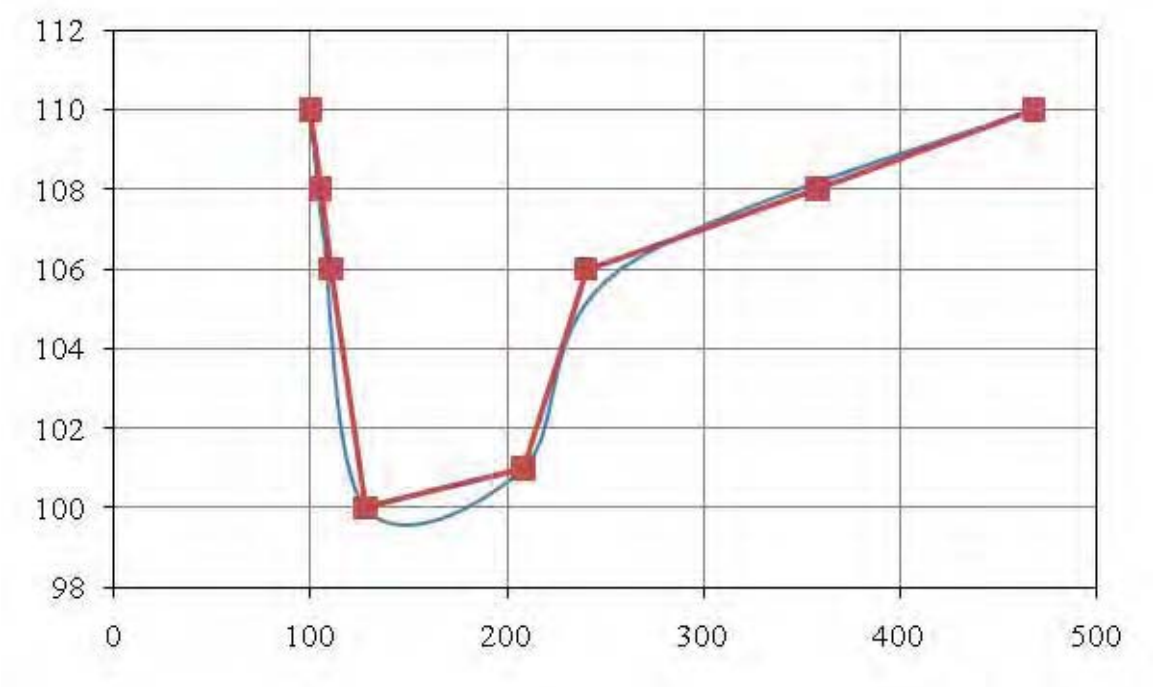


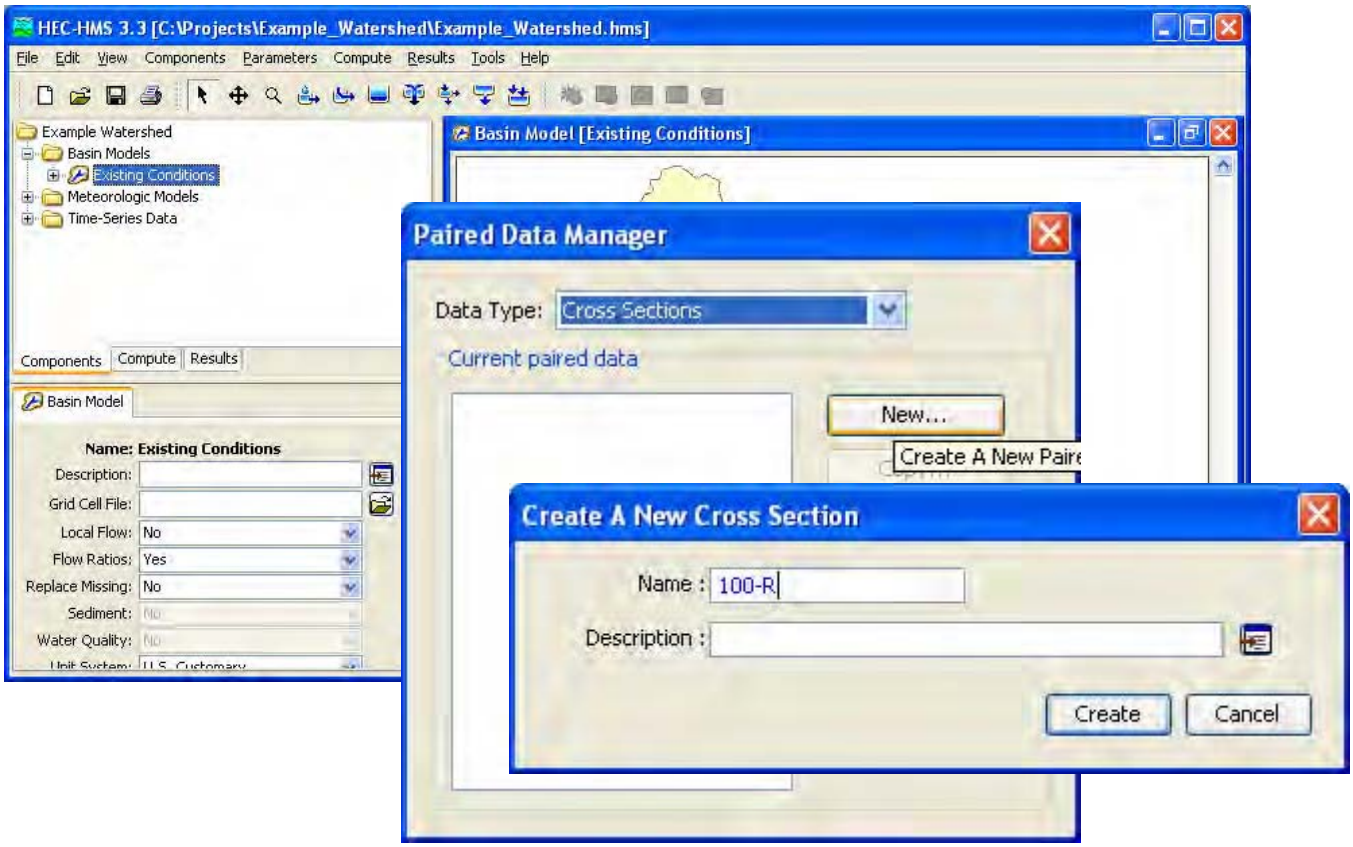
FIGURE F-17. 8-POINT GEOMETRY FOR REACH 110-R



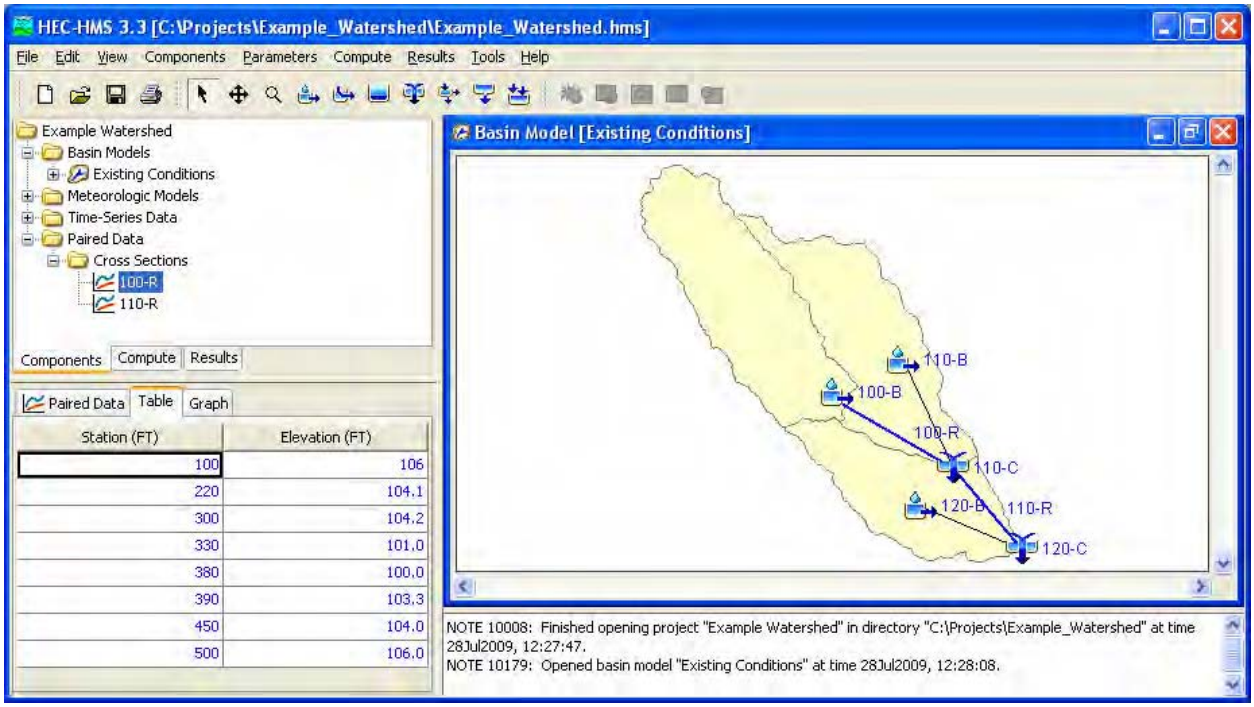
2. From Table F-7, select the appropriate Manning's n-value(s) for each reach.

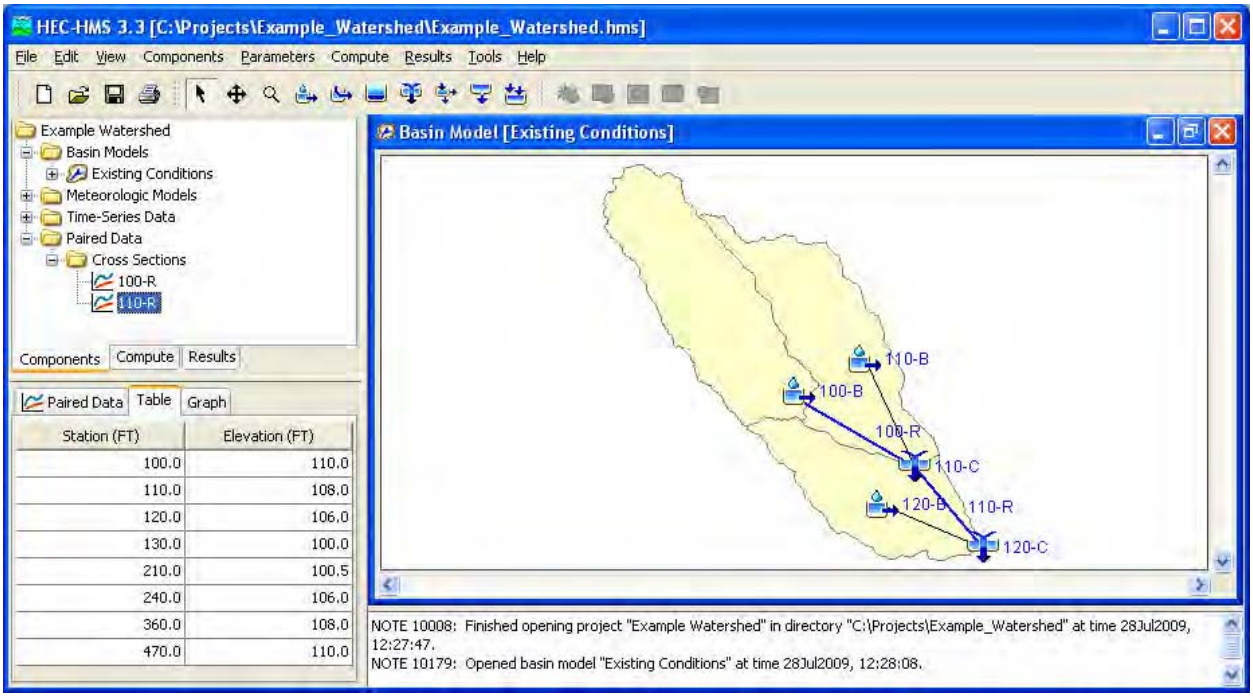
   Both routing reaches are natural, sand bed arroyos. From Table F-7 use a Manning's n-value of 0.05 for the entire section.

3. Code the routing data into HEC-HMS
   a. From the *Components* pull down menu, select *Paired Data Manager*
   b. Select "Cross Sections" as the *Data Type*
   c. Select *New* and enter a name for the first cross section (e.g. 100-R)
   d. Repeat Step 3.c for the second cross section

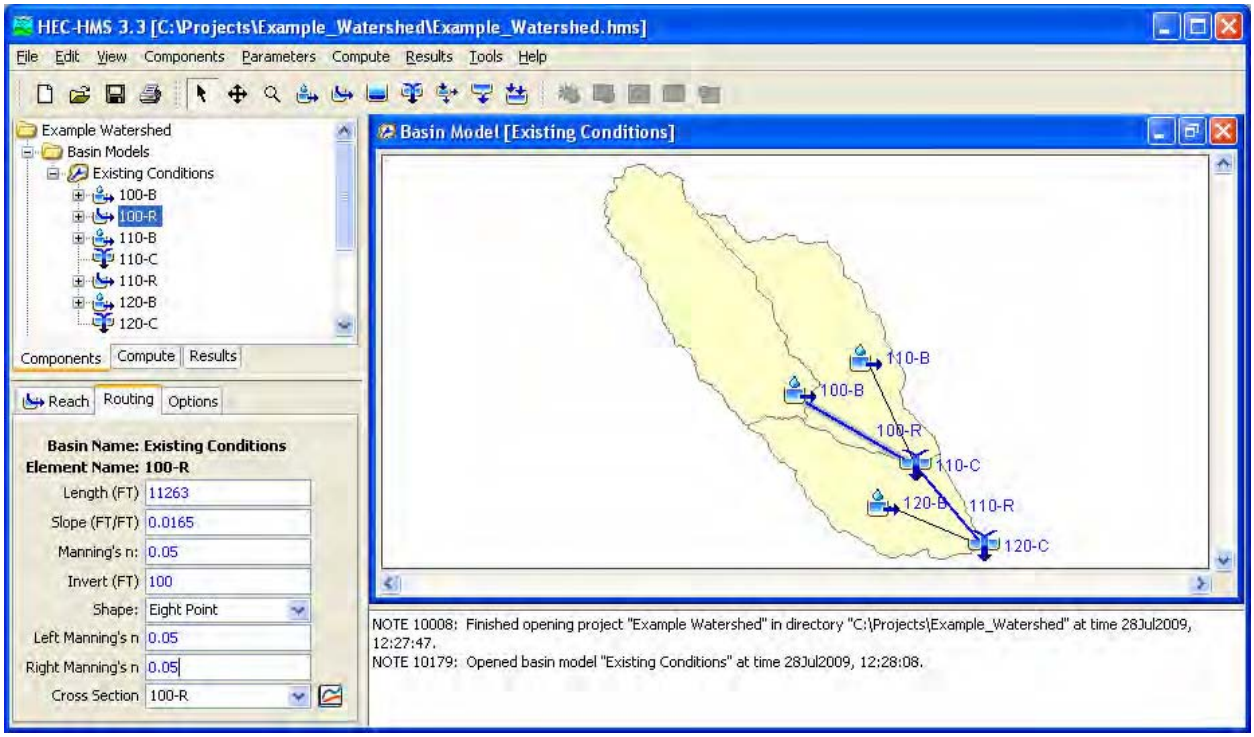


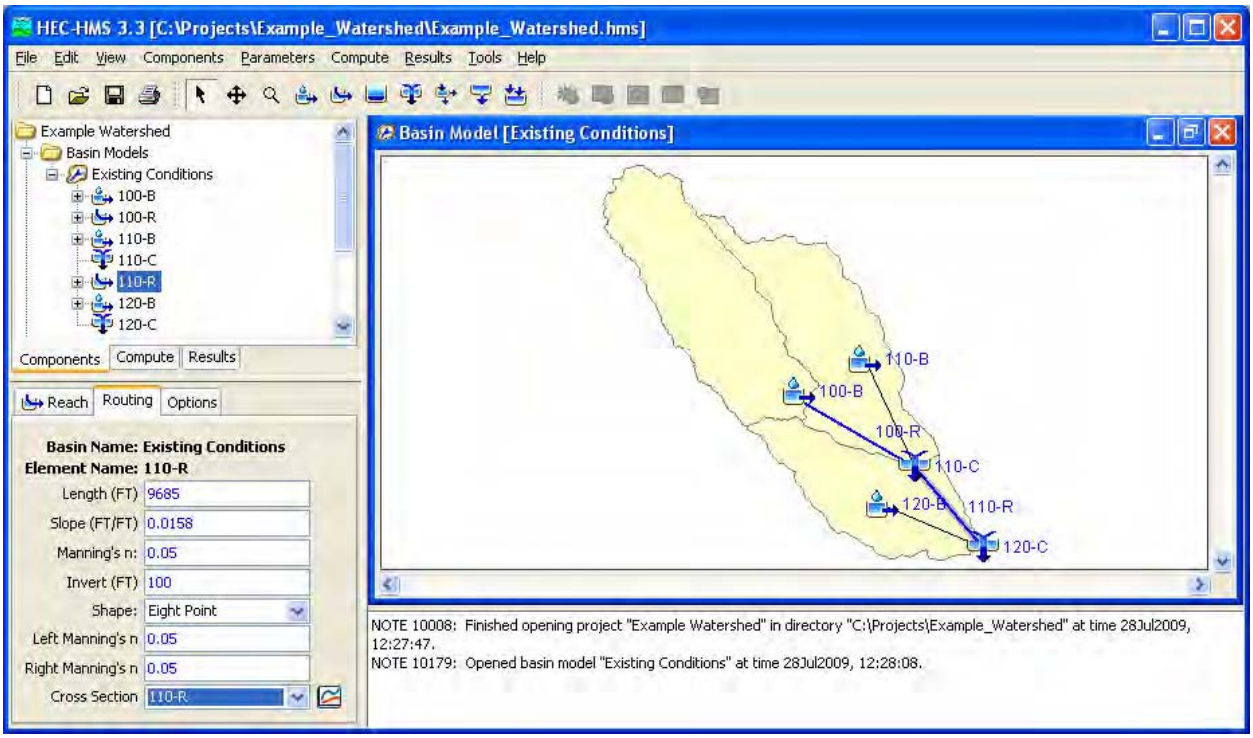
4. On the Table tab in the Component Editor of the Cross Section data, code in the 8-point geometry for each cross section using Figures F-11 and F-12





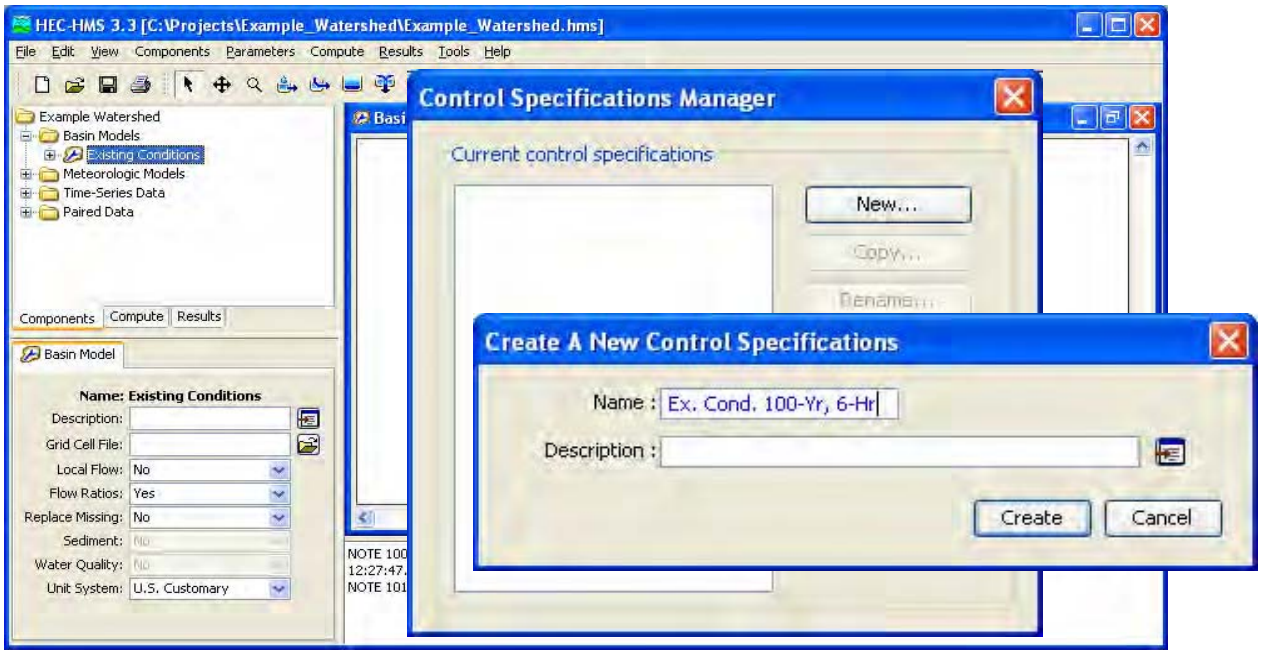
5. On the Routing tab of the Component Editor for each routing reach, code in the physical routing parameters



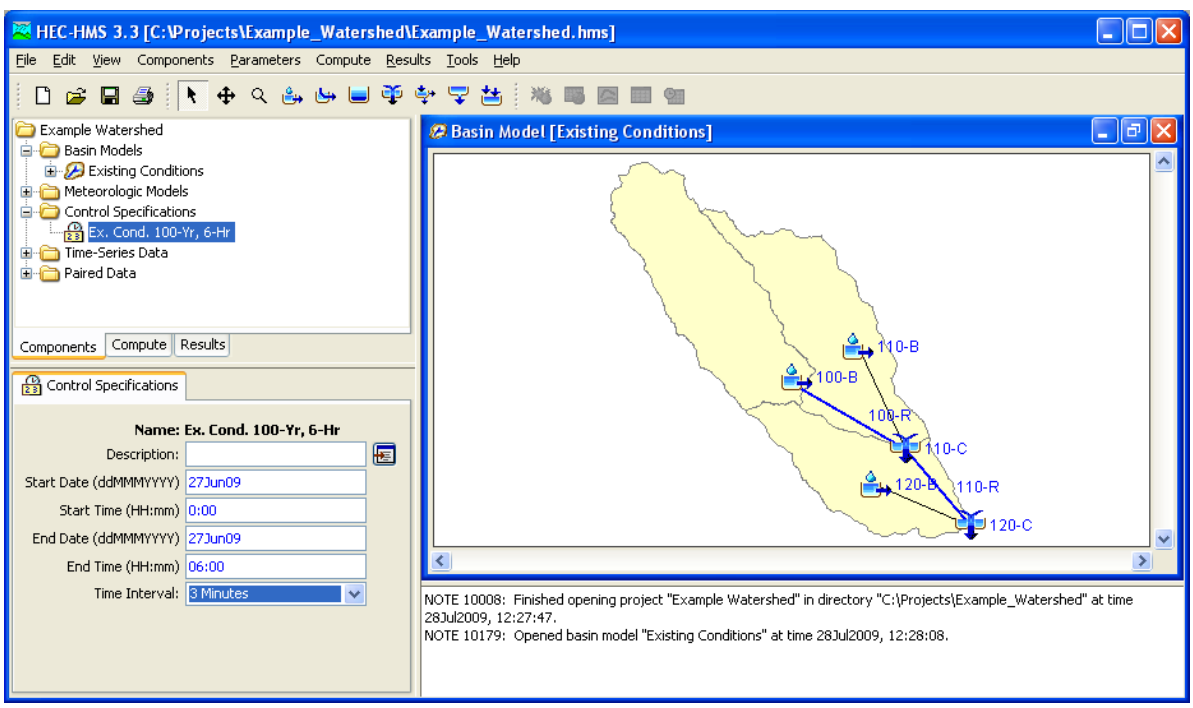


F.8.7 Model Execution

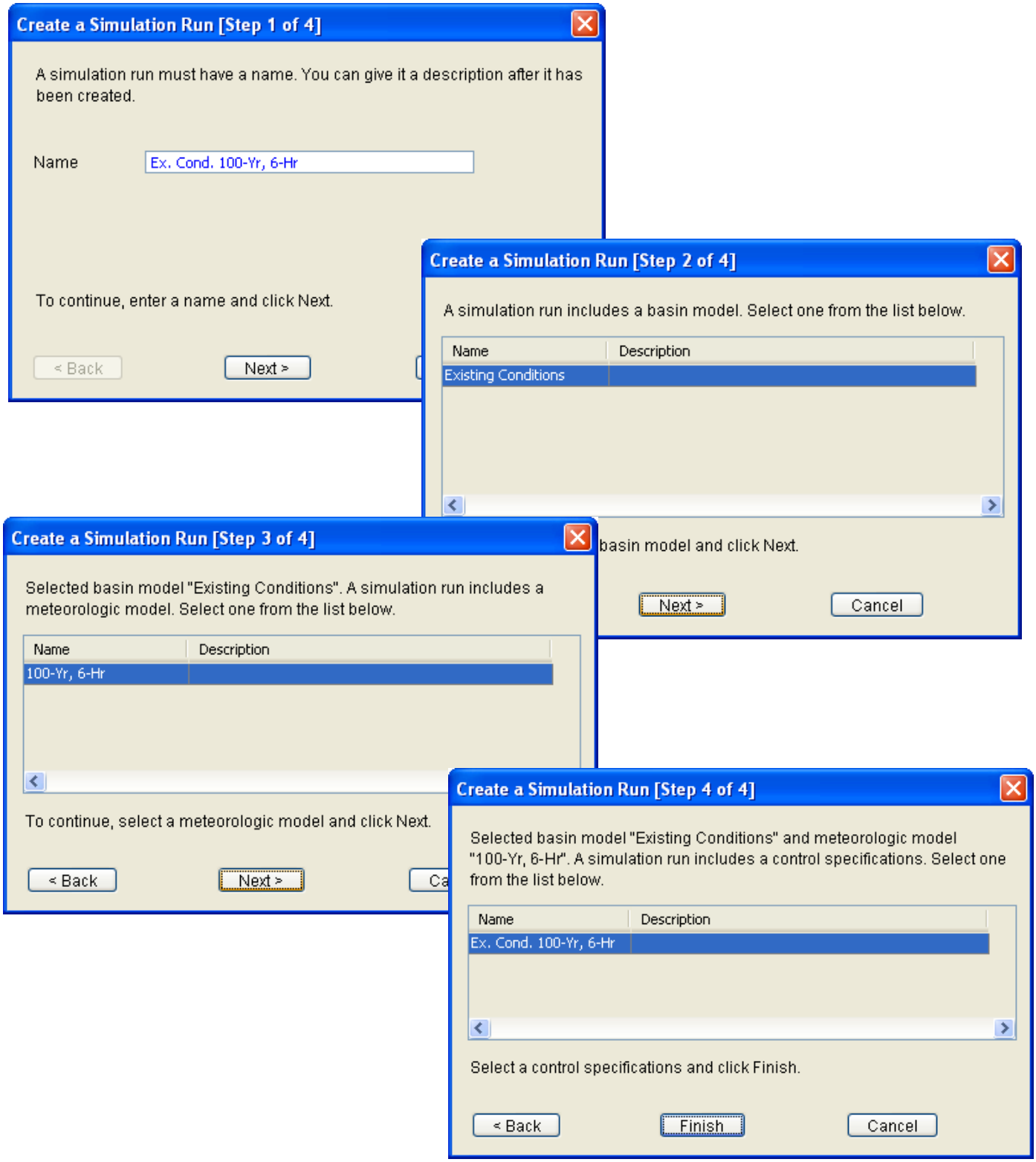
1. Create the Control Mata for model execution: From the *Components* pull down menu, select *Control Specifications Manager*
   a. Select *New*
   b. Enter a name for the control model (e.g. Ex. Cond. 100-Yr, 6-Hr)



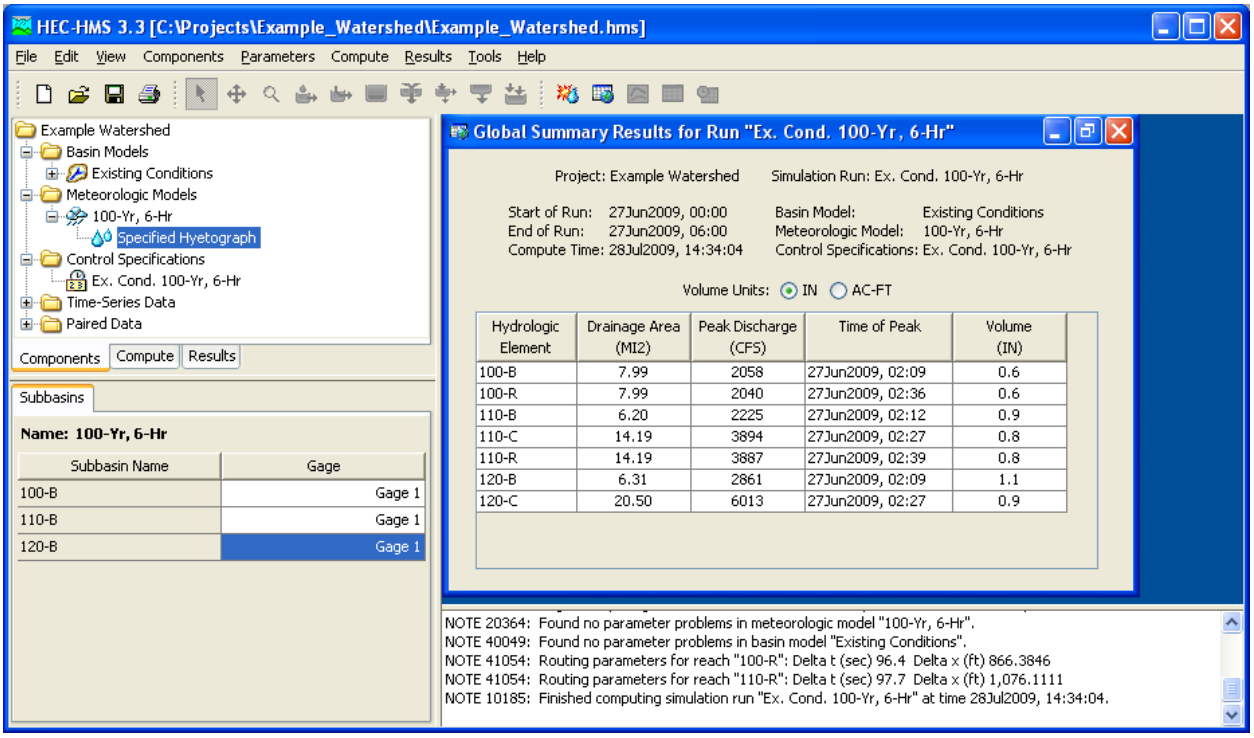
2. In the Component Editor for the Control Specifications input the model simulation time and the computational interval



3. From the *Compute* pull down menu, select *Create Simulation Run*
    a. Input a run name (e.g. Ex. Cond. 100-Yr, 6-Hr)
    b. Select the Basin Model, Meteorologic Model and Component Model



4. From the Compute pull down menu, select Compute Run and view the global summary results.



G. Procedure for Probable Maximum Flood

Computation of the Probable Maximum Flood (PMF), or one-half Probable Maximum Flood (½ PMF), is typically required for design of dam spillways in high hazard areas. For flood control dams, the PMF is typically used for design of the emergency spillway. The Office of the State Engineer (OSE) should be contacted regarding specific requirements on the use of the PMF.

G.1 JURISDICTION OF THE OFFICE OF THE STATE ENGINEER (OSE)

NOTE: FACILITIES THAT COME UNDER THE OSE MUST BE COORDINATED WITH THAT JURISDICTION.

The OSE has jurisdiction over the design and construction of non-federal dams. OSE authority for the safety of dams is contained primarily within Chapter 72, NMSA 1978. All dams must conform to the OSE criteria as demonstrated by correspondence issued by the OSE and provided to the City Engineer/SSCAFCA. Before proceeding to design any project requiring a permit for a dam, the Office of the State Engineer should be contacted to obtain guidance on applicable regulations and design criteria. City/SSCAFCA review must occur before submittal to OSE to obtain concurrence on determination of PMP. This includes dams intended for sediment, erosion and flood control.

Copies of the Manual of Rules and Regulations Governing the Appropriation and Use of the Surface Waters of the State of New Mexico and the Summary of New Mexico State Engineer Office Procedure on Design Criteria and Safety of Dams are available from the OSE, Santa Fe, New Mexico. Included in the summary is information on the classification of dams, hydrologic evaluation guidelines, probable maximum precipitation (PMP) criteria, and the "Engineering Review Project Check List". Special engineering requirements are required for project design and construction supervision.

The procedures for determination of the PMF must be consistent with the OSE's rules, regulations, procedures and design criteria. The OSE shall make the final determination on the design criteria, safety requirements, alternate specifications/procedures and/or additional requirements.

H. 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 SSCAFCA and other reviewing agencies. The use of alternate procedures should be reviewed with the SSCAFCA agency early in the project to establish that such alternate procedures are acceptable and to establish specific parameters.

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.

H.1 PROGRAMS FOR ALTERNATE PROCEDURE ACCEPTANCE

Some computer programs which have had previous use in the community and will be considered for alternate procedure acceptance include:

1) **SWMM -** Stormwater Management Model. Version 5 by the U.S. Environmental Protection Agency. This is an extremely complex model with an extensive range of capabilities. The program was developed for urban areas with storm sewer systems. Of special interest is the capability to model stormwater quality in addition to water quantity. The EXTRAN module of the SWMM model has been used locally to model flow in irrigation canals and drains because its dynamic flow routing capability can compute backwater profiles in open channels and closed conduits under unsteady flow conditions. Hydrograph input for the EXTRAN application can use hydrographs generated by the HYMO computer program. Specific parameters to calibrate SWMM parameters for local conditions have not been established.

2) **TR-20 -** Computer Program Project Formulation, Hydrology by the U.S.D.A. Soil Conservation Service. This SCS computer program is widely used throughout the U.S. It is available through independent licensed software vendors and from the National Technical Information Service. The program was initially developed for rural areas with relatively large sub-basins. The "TYPE-II" (24-hour) rainfall distribution commonly used with TR-20 is not applicable for the Albuquerque area. In New Mexico, a TYPE II-a (24-hour) distribution should be used with TR-20. The "a" used in the TYPE II-a distribution refers to the percentage of the one-hour precipitation (P_{60}) to the 24-hour precipitation (P1440) or, a = 100 * P60 / P1440. The value of "a" is rounded to the nearest five percent (i.e.: 60, 65, 70 and 75). Tables of TYPE II-60, 11-65, II-70, and II-75 distributions, with a 0.25 hour incremental time, are available from the SCS. SCS CNs should be consistent with TR-55, Chapter 2 procedures; but should not be less than the values in TABLE E-1, or as computed by equation E-7.

3) **TR-48 -** Computer Program for Project Formulation - Structure Site Analysis by the U.S.D.A. Soil Conservation Service. This program has particular application to the analysis and design of dams and therefore may have special application to this area. The program normally uses the sites' storage-discharge capacities to floodroute inflow hydrographs through a potential reservoir. Inflow hydrographs may be input from other models or developed from a storm rainfall distribution. The program will compute runoff by the standard SCS CN procedure or by the initial abstraction-average infiltration

method. The program also has limited routing capability for analysis of multiple structures and channels.

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PART G - HYMO INPUT AND OUTPUT

G.1 HYMO INPUT FILE

```
*S FILE:TESTDPM.DAT
START       TIME=0.0    NPU=O       PRINT LINE=O
*S****COMPUTE HYDROGAPHS FOR SECTION 22.2, HYDROLOGY, DPM

******************
*S EXAMPLE C-2  **
******************

****PERVIOUS PORTION *****

* TREATMENT A, B,  C  - 100 YEAR STORM
COMPUTE HYD    ID=1  HYD N0=101.1 DT=.033333 HRS   DA=1.2500   SQ MI
               IA=-0.515  INF=1.292     K= -0.263600     TP=-0.292000 RAIN=
      .0000     .0017     .0035     .0053     .0071     .0090    .0109
      .0128     .0148     .0169     .0190     .0212     .0234    .0257
      .0280     .0304     .0329     .0355     .0381     .0409    .0437
      .0467     .0497     .0529     .0563     .0597     .0633    .0672
      .0712     .0754     .0798     .0850     .0906     .0965    .1093
      .1379     .1819     .2450     .3311     .4444     .5887    .7685
      .9878    1.1907    1.2756    1.3473    1.4111    1.4691   1.5226
      1.5722   1.6185    1.6620    1.7029    1.7414    1.7779   1.8124
      1.8450   1.8760    1.9054    1.9333    1.9598    1.9660   1.9719
      1.9774   1.9827    1.9877    1.9926    1.9972    2.0017   2.0060
      2.0102   2.0143    2.0182    2.0220    2.0257    2.0292   2.0327
      2.0361   2.0395    2.0427    2.0459    2.0490    2.0520   2.0550
      2.0579   2.0607    2.0635    2.0663    2.0690    2.0716   2.0742
      2.0767   2.0793    2.0817    2.0842    2.0865    2.0889   2.0912
      2.0935   2.0958    2.0980    2.1002    2.1023    2.1045   2.1066
      2.1087   2.1107    2.1127    2.1147    2.1167    2.1187   2.1206
      2.1225   2.1244    2.1263    2.1281    2.1299    2.1317   2.1335
      2.1353   2.1371    2.1388    2.1405    2.1422    2.1439   2.1456
      2.1472   2.1489    2.1505    2.1521    2.1537    2.1553   2.1568
      2.1584   2.1599    2.1615    2.1630    2.1645    2.1660   2.1675
      2.1689   2.1704    2.1718    2.1733    2.1747    2.1761   2.1775
      2.1789   2.1803    2.1816    2.1830    2.1844    2.1857   2.1870
      2.1884   2.1897    2.1910    2.1923    2.1936    2 1948   2.1961
      2.1974   2.1986    2.1999    2.2011    2.2024    2.2036   2.2048
      2.2060   2.2072    2.2084    2.2096    2.2108    2.2120   2.2131
      2.2143   2.2154    2.2166    2.2177    2.2189    2.2200
PRINT HYD      ID=1      CODE=1
**** IMPERVIOUS PORTION **** TREATMENT D
COMPUTE HYD    ID=2      HYD NO=101.2        DT=.033333 HRS DA=0.5000 SQ MI
               IA=-0.10  INF=0.04  K=-0.168200 TP= -0.292000      RAIN=-1
PRINT HYD      ID=2      CODE=1
**** COMBINED HYDROGRAPH ****
ADD HYD        ID=2      HYD NO=101.3 ID=1 ID=2
PRINT HYD      ID=2      CODE=1

********************
*S  EXAMPLE C-3   **
********************

*** PERVIOUS PORTION ***** TREATMENT A, B & C

COMPUTE HYD    ID=1 HYD NO=101.1   DT=.033333 HRS       DA=0.1250 SQ MI
               IA=0.515  INF= -1.292 K= -0.156500 TP= -0.16200O    RAIN=-1
PRINT HYD      ID=1      CODE=1
*** IMPERVIOUS PORTION ***** TREATMENT D
COMPUTE HYD    ID=2 HYD N0=101.2   DT=.033333 HRS       DA=0.0500 SQ MI
```

```
                IA=-0.10  INF=0.04  K= -0.090600 TP= -0.162000     RAIN=-1
PRINT HYD       ID=2      CODE=1
***** COMBINED HYDROGRAPH *****
ADD HYD         ID=2 HYD N0=101.3 ID=1 ID=2
PRINT HYD       ID=2 CODE=l

*******************
*S EXAMPLE C 4   **
*******************

RAINFALL        TYPE=1    RAIN QUARTER=0.0       RAIN ONE=1. 88
                RAIN SIX=2 22 RAIN DAY=2.68 DT=.033333
COMPUTE NM HYD  ID= 2     HYD NO= 101.3          DA=0.175 SQ MI
                PER A=21.43 PER B=35.71 PER C=14.29 PER D=28.57
                TP= -0.162 MASSRAIN=-1
PRINT HYD       ID=2 CODE=1

*******************
*S EXAMPLE D-3   **
*******************

***** PERVIOUS PORTION ****** TREATMENT A, B & C

COMPUTE HYD     ID=1 HYD NO=101.1 DT=.033333 HRS DA=1.2500 SQ MI
                IA=-0.515   INF=-1.292    K=-0.173400    TP=-0.292000 RAIN=
      .0000     .0070     .0142     .0217       .0294    .0375     .0459
      .0547     .0638     .0733     .0832       .0936    .1044     .1156
      .1272     .1394     .1520     .1650       .1786    .1927     .2072
      .2223     .2379     .2540     .2707       .2879    .3056     .3239
      .3428     .3622     .3822     .4028       .4240    .4457     .4681
      .4911     .5146     .5388     .5636       .5890    .6150     .6417
      .6690     .6969     .7255     .7548       .7847    .8152     .8464
      .8783     .9109     .9441     .9780      1.0126   1.0479    1.0838
      1.1205    1.1578    1.1959    1.2347      1.2741   1.7377    2.6322
      3.9411    5.4901    7.0155    8.1916      8.6459   9.0766    9.4890
      9.8382    10.1365   10.3938   10.6181     10.8157  10.9918   11.1507
      11 2957   11.4294   11.5541   11.6715     11.7830  11.8898   11.9928
      12.0928   12.1902   12.2857   12.3795     12.4721  12.5636   12.6542
      12.8014   12.9384   13.0662   13.1855     13.2969  13.4009   13.4983
      13.5894   13.6748   13.7549   13.8301     13.9008  13.9673   14.0300
      14.0892   14.1451   14.1979   14.2480     14.2954  14.3405   14.3834
      14.4243   14.4633   14.5006   14.5363     14.5705  14.6033   14.6349
      14.6654   14.6948   14.7231   14.7506     14.7773  14.8031   14.8283
      14.8527   14.8766   14.8999   14.9227     14.9450  14.9669   14.9883
      15.0094   15.0301   15.0505   15.0706     15.0905  15.1101   15.1294
      15.1485   15.1675   15.1862   15.2048     15.2232  15.2415   15.2596
      15.2777   15.2956   15.3134   15.3311     15.3487  15.3662   15.3837
      15.4011   15.4184   15.4356   15.4528     15.4700  15.4871   15.5042
      15.5212   15.5382   15.5551   15.5720     15.5889  15.6058   15.6226
      15.6395   15.6562   15.6730   15.6898     15.7065  15.7232   15.7400
      15.7567   15.7733   15.7900   15.8067     15.8233  15.8400
PRINT HYD       ID=1      CODE=1
***** IMPERVIOUS PORTION***** TREATMENT D
COMPUTE HYD     ID=2      HYD NO.=101.2     DT=.033333 HRS   DA=0.5000 SQ MI
                IA=-0.10  INF=0.04   K=-0.159700 TP=-0.292000 RAIN=-1
PRINT HYD       ID=2      CODE=1
***** COMBINED HYDROGRAPH*****
ADD HYD         ID=2      HYD NO.=101.3          ID=1     ID=2
PRINT HYD       ID=2      CODE=1

*******************
* S EXAMPLE D-4  **
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```
RAINFALL          TYPE=3    RAIN QUARTER=7. 58     RAIN ONE=11.38
                  RAIN SIX=15.84  RAIN DAY=0.0  DT=.033333
COMPUTE NM HYD    ID=2   HYD NO= 101.3  DA=1.750 SQ MI
                  PER A=240   PER B=400   PER C=160     PER D=320  TP=-0.292
                  MASSRAIN=1
PRINT HYD         ID=2      CODE=1
FINISH
G.2 HYMO OUTPUT FILE
      AHYMO PROGRAM (AHYM0392) AMAFCA VERSION OF HYMO - MARCH, 1992
      RUN DATE (MON/DAY/YR) = 01/18/1993
      START TIME (HR:MIN:SEC) = 18:32:27         USER NO. - AMAFCA01.491
      INPUT FILE = TESTDPM.DAT
*TEST OF THE DPM EXAMPLES - JANUARY 1993
*S FILE:TESTDPM.DAT
START             TIME=0.0  NPU=0     PRINT LINE=0
*S*******COMPUTE HYDROGRAPHS FOR SECTION 22.2, HYDROLOGY, DPM

******************
*S EXAMPLE C-2  **
******************

***** PERVIOUS PORTION *****

* TREATMENT A, B & C - 100 YEAR STORM
COMPUTE HYD       ID=1      HYD NO=101.1   DT=.033333 HR    DA=1.2500 SQ MI
                  IA=-0.515   INF=-1.292   K=-0.263600   TOP=-0.292000RAIN=
      .0000     .0017     .0035     .0053       .0071    .0090     .0109
      .0128     .0148     .0169     .0190       .0212    .0234     .0257
      .0280     .0304     .0329     .0355       .0381    .0409     .0437
      .0467     .0497     .0529     .0563       .0597    .0633     .0672
      .0712     .0754     .0798     .0850       .0906    .0965     .1093
      .1379     .1819     .2450     .3311       .4444    .5887     .7685
      .9878     l.1907    l.2756    1.3473      l.4111   l.4691    l.5226
      1.5722    1.6185    1.6620    1.7029      1.7414   1.7779    1.8124
      1.8450    1.8760    1.9054    1.9333      1.9598   1.9660    1.9719
      l.9774    l.9827    1.9877    l.9926      l.9972   2.0017    2.0060
      2.0102    2.0143    2.0182    2.0220      2.0257   2.0292    2.0327
      2.0361    2.0395    2.0427    2.0459      2.0490   2.0520    2.0550
      2.0579    2.0607    2.0635    2.0663      2.0690   2.0716    2.0742
      2.0767    2.0793    2.0817    2.0842      2.0865   2.0889    2.0912
      2.0935    2.0958    2.0980    2.1002      2.1023   2.1045    2.1066
      2.1087    2.1107    2.1127    2.1147      2.1167   2.1187    2.1206
      2.1225    2.1244    2.1263    2.1281      2.1299   2.1317    2.1335
      2.1353    2.1371    2.1388    2.1405      2.1422   2.1439    2.1456
      2.1472    2.1489    2.1505    2.1521      2.1537   2.1553    2.1568
      2.1584    2.1599    2.1615    2.1630      2.1645   2.1660    2.1675
      2.1689    2.1704    2.1718    2.1733      2.1747   2.1761    2.1775
      2.1789    2.1803    2.1816    2.1830      2.1844   2.1857    2.1870
      2.1884    2.1897    2.1910    2.1923      2.1936   2.1948    2.1961
      2.1974    2.1986    2.1999    2.2011      2.2024   2.2036    2.2048
      2.2060    2.2072    2.2084    2.2096      2.2108   2.2120    2.2131
      2.2143    2.2154    2.2166    2.2177      2.2189   2.2200
      K = .263600HR       TP =  29200H      SHAPE CONSTANT, N = 3.92515
      UNIT PEAK = 1498.9  CFS        UNIT VOLUME = 1.000  B = 350.15
RUNOFF COMPUTED BY INITIAL ABSTRACTION - INFILTRATION METHOD - DT =.033333
PRINT HYD         ID=1      CODE=1

                                     PARTIAL HYDROGRAPH   101.10

RUNOFF VOLUME = .65128 INCHES        = 43.4181 ACRE-FEET
PEAK DISCHARGE RATE = 905.66 CFS  AT 1.700 HOURS BASIN AREA = 1.2500 SQ. MI

****IMPERVIOUS PORTION **** TREATMENT D
```

```
COMPUTE HYD      ID=2       HYD NO=101.2     DT=.033333 HRS   DA=0.5000 SQ MI
                 IA=-0.10   INF=0.04   K=-0.168200 TP=-0.292000       RAIN=-1
      K= .168200HR         TP = .292000HR         SHAPE CONSTANT, N = 6.62354
      UNIT PEAK = 861.53   CFS UNIT VOLUME = 1.000         B = 503.13
      RUNOFF COMPUTED BY INITIAL ABSTRACTION - INFILTRATION METHOD - DT =
.033333
PRINT HYD        ID=2       CODE=1
                                      PARTIAL HYDROGRHAPH  101.20
RUNOFF VOLUME = 1.98503 INCHES        =            52.9338 ACRE-FEET
PEAK DISCHARGE RATE = 923.75 CFS AT 1.667 HOURS   BASIN AREA = .5000 SQ. MI.

*****COMBINED HYDROGRAPH*****

ADD HYD          ID=2       HYD NO=101.3           IN=1     ID=2
PRINT HYD        ID=2       CODE=1
PARTIAL HYDROGRAPH 101.30
      RUNOFF VOLUME = 1.03235 INCHES = 96.3518 ACRE-FEET
      PEAK DISCHARGE RATE = 1827.79 CFS AT 1.667 HOURS BASIN AREA = 1.7500 SQ.

******************
*S EXAMPLE C-3  **
******************

**** PERVIOUS PORTION **** TREATMENT A, B & C
COMPUTE HYD      ID=1   HYD NO=101.1  DT=.033333  HRS DA=0.1250 SQ.MI
                 IA =-0.515 INF=-1.292 K=-0.156500  TP=-0.162000 RAIN=-1
      K = .156500 HR       TP = .162000HR         SHAPE CONSTANT, N = 3.65682
      UNIT PEAK = 255.86 CFS         UNIT VOLUME = 1.000   B = 331.60
      RUNOFF COMPUTED BY INITIAL ABSTRACTION - INFILTRATION METHOD - DT = .033333

      PRINT HYD            ID=1       CODE=1

                                      PARTIAL HYDROGRAPH  101.10

      RUNOFF VOLUME = .65128 INCHES  = 4.3418 ACRE-FEET
      PEAK DISCHARGE RATE = 139.88 CFS AT 1.533 HOURS    BASIN AREA = .1250 SQ. MI.

****IMPERVIOUS PORTION****TREATMENT D
COMPUTE HYD      ID=2       HYD NO=101.2     DT=.033333 HRS DA=0.0500 SQ. MI.
                 IA=-0.10   INF=0.04   K=-0.090600 TP=-0.162000       RAIN=-1
      K = .090600 HR       TP = .162000HR         SHAPE CONSTANT, N = 6.87595
      UNIT PEAK = 159.06 CFS         UNIT VOLUME = .9999   B = 515.35
      RUNOFF COMPUTED BY INITIAL ABSTRACTION - INFILTRATION METHOD - DT = .033333

PRINT HYD        ID=2       CODE=1
      PARTIAL HYDROGRAPH 101.20

      RUNOFF VOLUME = 1.98503 INCHES = 5.2934 ACRE-FEET
      PEAK DISCHARGE RATE = 127.85 CFS AT 1.533 HOURS BASIN AREA = .0500 SQ. MI.

*****COMBINED HYDROGRAPH*****

ADD HYD          ID=2 HYD NO=1-1.3    ID=1         ID=2
PRINT HYD        ID=2       CODE=1

                                      PARTIAL HYDROGRAPH     101.30

      RUNOFF VOLUME =      1.03235 INCHES         =         9.6352 ACRE-FEET
      PEAK DISCHARGE RATE =     267.72 CFS AT   1.533 HOURS     BASIN AREA = .1750 SQ. MI.

******************
*S EXAMPLE C-4  **
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```
RAINFALL          TYPE=1     RAIN QUARTER=0.0       RAIN ONE-1.88
                  RAIN SIX=2.22         RAIN DAY=2.68         DT=.033333
      COMPUTED 6-HOUR RAINFALL DISTRIBUTION BASED ON NOAA ATLAS 2 - PEAK AT 1.40 HR.
      DT=         .033333 HOURS         END TIME =   5.999940 HOURS
      .0000       .0017      .0035      .0053        .0071     .0090      .0109
      .0128       .0148      .0169      .0190        .0212     .0234      .0257
      .0280       .0304      .0329      .0355        .0381     .0409      .0437
      .0467       .0497      .0529      .0563        .0597     .0633      .0672
      .0712       .0754      .0798      .0850        .0906     .0965      .1093
      .1379       .1819      .2450      .3311        .4444     .5887      .7685
      .9878      1.1907     1.2756     1.3473       1.4111    1.4691     1.5226
      1.5722     1.6185     1.6620     1.7029       1.7414    1.7779     1.8124
      1.8450     1.8760     1.9054     1.9333       l.9598    1.9660     1.9719
      1.9774     1.9822     1.9877     1.9926       l.9972    2.0017     2.0060
      2.0102     2.0143     2.0182     2.0220       2.0257    2.0292     2.0327
      2.0361     2.0395     2.0427     2.0459       2.0490    2.0520     2.0550
      2.0579     2.0607     2.0635     2.0663       2.0690    2.0716     2.0742
      2.0767     2.0793     2.0817     2.0842       2.0865    2.0889     2.0912
      2.0935     2.0958     2.0980     2.1002       2.1023    2.1045     2.1066
      2.1087     2.1107     2.1127     2.1147       2.1167    2.1187     2.1206
      2.1225     2.1244     2.1263     2.1281       2.1299    2.1317     2.1335
      2.1353     2.1371     2.1388     2.1405       2.1422    2.1439     2.1456
      2.1472     2.1489     2.1505     2.1521       2.1537    2.1553     2.1568
      2.1584     2.1599     2.1615     2.1630       2.1645    2.1660     2.1675
      2.1689     2.1704     2.1718     2.1733       2.1747    2.1761     2.1775
      2.1789     2.1803     2.1816     2.1830       2.1844    2.1857     2.1870
      2.1884     2.1897     2.1910     2.1923       2.1936    2.1948     2.1961
      2.1974     2.1986     2.1999     2.2011       2.2024    2.2036     2.2048
      2.2060     2.2072     2.2084     2.2096       2.2108    2.2120     2.2131
      2.2143     2.2154     2.2166     2.2177       2.2189    2.2200

COMPUTE NM HYD    ID=2       HYD NO.= 101.3         DA=0.175 SQ. MI
                  PER A=21.43           PER B=35.71 PERC=14.29     PER D=28.57
                  TP= -0.1162           MASSRAIN =-1

      K = .090554HR     TP = .162000HR    K/TP RATIO = .558978 SHAPE CONSTANT, N = 6.880332
      UNIT PEAK = 159.11 CFS   UNIT VOLUME = .9999      B = 515.56     P60 = 1.8800
      AREA = .049998 SQ. MI.  IA - .1000 INCHES   INF = .04000 INCHES PER HOUR
      RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD  - DT = .033333

      K = .156460HR     TP = .162000HR    K/TP RATIO = .965805 SHAPE CONSTANT, N = 3.657761
      UNIT PEAK = 255.92 CFS         UNIT VOLUME = 1.000      B = 331.67      P60 = 1.8800
      AREA = .125003  SQ. MI.        IA - .51499 INCHES      INF = 1.29198 INCHES PER HOUR
      RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD  - DT = .033333

PRINT HYD         ID=2       CODE=1

                                         PARTIAL HYDROGRAPH 101.30

      RUNOFF VOLUME = 1.03234 INCHES = 9.6351 ACRE-FEET
      PEAK DISCHARGE RATE = 267.77 CFS AT 1.533 HOURS     BASIN AREA = 1750 SQ. MI

******************
*S EXAMPLE D-3  **
******************

***** PERVIOUS PORTION ***** TREATMENT A, B & C

COMPUTE HYD       ID=1      HYD NO=101.1      DT=.033333 HRS   DA=1.2500 SQ. MI
                  IA=-0.515  INF=-1.292 K=-0.173400  TP=-0.292000 RAIN =
      .0000       .0070      .0142      .0217        .0294     .0375      .0459
      .0547       .0638      .0733      .0832        .0936     .1044      .1156
      .1272       .1394      .1520      .1650        .1786     .1927      .2072
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```
      .2223     .2379     .2540     .2707       .2879    .3056     .3239
      .3428     .3622     .3822     .4028       .4240    .4457     .4681
      .4911     .5146     .5388     .5636       .5890    .6150     .6417
      .6690     .6969     .7255     .7548       .7847    .8152     .8464
      .8783     .9109     .9441     .9780      1.0126   1.0479    1.0838
     1.1205    1.1578    1.1959    1.2347      1.2741   1.7377    2.6322
     3.9411    5.4901    7.0155    8.1916      8.6459   9.0766    9.4890
     9 8382   10.1365   10.3938   10.6181     10.8157  10.9918   11.1507
    11.2957   11.4294   11.5541   11.6715     11.7830  11.8898   11.9928
    12.0928   12.1902   12.2857   12.3795     12.4721  12.5636   12.6542
    12.8014   12.9384   13.0662   13.1855     13.2969  13.4009   13.4983
    13.5894   13.6748   13.7549   13.8301     13.9008  13.9673   14.0300
    14.0892   14.1451   14.1979   14.2480     14.2954  14.3405   14.3834
    14.4243   14.4633   14.5006   14.5363     14.5705  14.6033   14.6349
    14.6654   14.6948   14.7231   14.7506     14.7773  14.8031   14.8283
    14.8527   14.8766   14.8999   14.9227     14.9450  14.9669   14.9883
    15.0094   15.0301   15.0505   15.0706     15.0905  15.1101   15.1294
    15.1485   15.1675   15 1862   15.2048     15.2232  15.2415   15.2596
    15.2777   15.2956   15.3134   15.3311     15.3487  15.3662   15.3837
    15.4011   15.4184   15.4356   15.4528     15.4700  15.4871   15.5042
    15.5212   15.5382   15.5551   15.5720     15.5889  15.6058   15.6226
    15 6395   15.6562   15.6730   15 6898     15.7065  15.7232   15.7400
    15.7567   15.1733   15.7900   15.8067     15.8233  15.8400

K = .173400HR    TP = .292000HR       SHAPE CONSTANT, N =  6.37493
UNIT PEAK = 2101.2 CFS     UNIT VOLUME = .9999    B = 490.85
RUNOFF COMPUTED BY INITIAL ABSTRACTION - INFILTRATION METHOD - DT = .033333

PRINT HYD        ID=1       CODE=1
                                       PARTIAL HYDROGRAPH 101.10

RUNOFF VOLUME = 10.91309 INCHES      =            727.5348 ACRE-FEET
PEAK DISCHARGE RATE = 14586.49 CFS AT 2.433 HOURS             BASIN AREA = 1.2500 SQ. MI.

***** IMPERVIOUS PORTION ***** TREATMENT D

COMPUTE HYD      ID=2     HYD NO-101.2   DT=.033333 HRS     DA=0.50000   SQ MI
                 IA=-0.10  INF=0.04  K=-0.159700 TP=-0.292000       RAIN=-1

      K = 1.59700HR        TP = .292000HR         SHAPE CONSTANT, N =7.07453
      UNIT PEAK = 898.59 CFS         UNIT VOLUME = 1.000  B = 524.78
      RUNOFF COMPUTED BY INITIAL ABSTRACTION - INFILTRATION METHOD - DT = .033333
PRINT HYD        ID=1       CODE=1
                                       PARTIAL HYDROGRAPH 101.20

RUNOFF VOLUME = 15.57613 INCHES      =            415.3609  ACRE-FEET
PEAK DISCHARGE RATE = 6494.75 CFS AT 2.433 HOURS  BASIN AREA = .5000 SQ. MI.

***** COMBINED HYDROGRAPH *****

ADD HYD               ID=2     HYD NO-101.3     ID=1     ID=2
PRINT HYD               ID=2     CODE =1
PARTIAL HYDROGRAPH 101.3
RUNOFF VOLUME =            12.24539 INCHES        =        1142.8960 ACRE-FEET
PEAK DISCHARGE RATE = 21081.24 CFS AT 2.433 HOURS BASIN AREA = 1.7500 SQ. MI.

******************
*S EXAMPLE D-4  **
******************

RAINFALL         TYPE=3    RAIN QUARTER = 7.58    RAIN ONE=11.38
                 RAIN SIX=15.84       RAIN DAY=0.0          DT =.033333
      COMPUTED P.M.P.  6-HOUR RAINFALL DISTRIBUTION BASED ON H.M.R.-55a
      DT = .033333 HOURS             END TIME = 5.999940 HOURS
```

```
.0000     .0070     .0142     .0217       .0294    .0375     .0459
.0547     .0638     .0733     .0832       .0936    .1044     .1156
.1272     .1394     .1520     .1650       .1786    .1927     .2072
.2223     .2379     .2540     .2707       .2879    .3056     .3239
.3428     .3622     .3822     .4028       .4240    .4457     .4681
.4911     .5146     .5388     .5636       .5890    .6150     .6417
.6690     .6969     .7255     .7548       .7847    .8152     . 8464
.8783     .9109     .9441    1.9780      1.0126   1.0479    1.0838
1.1205    1.1578    1.1959    l.2347      1.2741   1.7377    2.6322
3.9411    5.4901    7.0155    8.1916      8.6459   9.0766    9.4890
9.8382    10.1365   10.3938   10.6181     10.8157  10.9918   11.1507
11.2957   11.4294   11.5541   11.6715     11.7830  11.8898   11.9928
12.0928   12.1902   12.2857   12.3795     12.4721  12.5636   12.6542
12.8014   12.9384   13.0662   13.1855     13.2969  13.4009   13.4983
13.5894   13.6748   13.7549   13.8301     13.9008  13.9673   14.0300
14.0892   14.1451   14.1979   14.2480     14.2954  14.3405   14.3834
14.4243   14.4633   14.5006   14.5363     14.5705  14.6033   14.6349
14.6654   14.6948   14.7231   14.7506     14.7773  14.8031   14.8283
14.8527   14.8766   14.8999   14.9227     14.9450  14.9669   14.9883
15.0094   15.0301   15.0505   15.0706     15.0905  15.1101   15.1294
15.1485   15.1675   15.1862   15.2048     15.2232  15.2415   15.2596
15.2777   15.2956   15.3134   15.3311     15.3487  15.3662   15.3837
15.4011   15.4184   15.4356   15.4528     15.4700  15.4871   15.5042
15.5212   15.5382   15.5551   15.5720     15.5889  15.6058   15.6226
15.6395   15.6562   15.6730   15.6898     15.7065  15.7232   15.7400
15.7567   15.7733   15.7900   15.8067     15.8233  15.8400

COMPUTE NM HYD   ID=2       HYD NO= 101.3          DA=1.750 SQ. MI
      PER A=240    PER B=400   PER C=160       PER D=320      TP=-0.292

K = .159697HR   TP = .292000HR   K/TP RATIO = .546909 SHAPE CONSTANT, N = 7.074674
UNIT PEAK - 898.60 CFS     UNIT VOLUME = 1.000    B = 524.78    P60 = 11.380
AREA = .500000 SQ MI       IA = .10000 INCHES     INF = .04000 INCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT = .033333

K = .173405HR   TP = .292000HR   K/TP RATIO = .593853   SHAPE CONSTANT, N = 6.374689
UNIT PEAK - 2101.2 CFS     UNIT VOLUME = .9999    B = 490.84    P60 = 11.380
AREA = 1.250000 SQ MI     IA = .51500 INCHES     INF = 1.29200 INCHES PER HOUR
RUNOFF COMPUTED BY INITIAL ABSTRACTION/INFILTRATION NUMBER METHOD - DT = .033333

PRINT HYD        ID=2       CODE=1
                                       PARTIAL HYDROGRAPH 101.30

      RUNOFF VOLUME = 12.24539 INCHES     =   1142.8960 ACRE-FEET
      PEAK DISCHARGE RATE = 21081.04 CFS AT    2.433 HOURS      BASIN AREA = 1.7500 SQ. MI

FINISH

      NORMAL PROGRAM FINISH          END TIME (HR:MIN:SEC) = 18:32:38
```