GEORGIA INSTITUTE OF TECHNOLOGY

OFFICE OF RESEARCH ADMINISTRATION

RESEARCH PROJECT INITIATION

Date: January 10, 1974

Project Title: Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project No: E-20-646

Principal Investigator Dr. A. M. Lumb

Sponsor: Board of Commissioners - DeKalb County

Agreement Period: From 12/17/73 Until 12/1/74

Type Agreement: Contract dated 12/5/73

Amount: $78,901 DeKalb Funds (E-20-646)
20,399 GIT Contrib. (E-20-323)
$99,300 TOTAL

Reports Required: Monthly Progress Reports, Final Report

Sponsor Contact Person (s):

Ms. Jean Ackerman (temporarily)
Contract Manager
DeKalb County Government
Decatur, Georgia 30030

Assigned to: School of Civil Engineering

COPIES TO:

Principal Investigator	Library
School Director	Rich Electronic Computer Center
Dean of the College	Photographic Laboratory
Director, Research Administration	Project File
Director, Financial Affairs (2)	
Security-Reports-Property Office	
Patent Coordinator	Other

RA-3 (6-71)

GEORGIA INSTITUTE OF TECHNOLOGY

OFFICE OF RESEARCH ADMINISTRATION

RESEARCH PROJECT TERMINATION

Date: July 9, 1975

Project Title: Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project No: E-20-646

Principal Investigator: Dr. A. M. Lumb

Sponsor: Board of Commissioners - DeKalb County

Effective Termination Date: May 30, 1975

Clearance of Accounting Charges: June 30, 1975

Grant/Contract Closeout Actions Remaining:

Assigned to School of Civil Engineering

COPIES TO:

- Principal Investigator
- School Director
- Dean of the College
- Director of Research Administration
- Office of Financial Affairs (2)
- Security - Reports - Property Office
- Patent and Inventions Coordinator
- Library, Technical Reports Section
- Computer Sciences
- Photographic Laboratory
- Terminated Project File No. ______
- Other ______

PROGRESS REPORT

For the Period January 1 to January 31, 1973

Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project Number: E-20-646

Project Director: Alan M. Lumb

During January 1974, progress continued with the collection, coding and keypunching of the needed hydrologic data. Coding of the streamflow data measured on the small drainage area near Clairmont and LaVista Roads was completed and the coding of precipitation at that site began. The data is 5-minute values covering a period from July, 1971, to present. Detailed data for three floods per year was ordered from the U.S. Geological Survey for Camp Creek, North Fork Camp Creek, South Utoy Creek, and South River @ East Point. Ten years of daily precipitation at Alpharetta and Norcross were coded and punched. Additional information on the specific soils in DeKalb County was obtained from Grover Thomas, SCS.

Seven sites in the County were examined as possible locations for streamflow and precipitation measurement. A summary report prepared by Carvel Deese is included as Appendix I. Instruments required for the five sites were ordered.

Site visits by Drs. James, Wallace and Lumb were made during January to five of the twelve priority drainage basins not visited in December. Since none of the sites really fall in the small watershed category as envisioned in the proposal, the concept of two catagories, small and large watersheds, will not be used and the priorties will need to be reassigned to all drainage basins as one group. Priorities will need to be assigned by the end of March.

Several meetings took place during January. Dr. Lumb and a graduate assistant met Brenda Day of the Planning Department and Charles Hill of the Data Center concerning the parcel file and computer capabilities. Drs. Lumb, Wallace and James meet with Larry Lyons to discuss the Corps of Engineers' study of the Metropolitan area. Dr. Sanders was present at a meeting with Commissioner Williams for residents of the north part of the County. Dr. Lumb gave a presentation to the DeKalb Chapter of the Georgia Conservancy on flooding problems and the role of the project in solving these problems.

During January the definition of tasks was refined. The current working document on tasks is included as Appendix II.

APPENDIX I

SITE PROPOSALS FOR INSTALLATION OF GAGING STATIONS FOR DEKALB

Records which are currently being collected in the Metropolitan area by the U.S. Geological Survey will be available for calibrating the Hydrologic Simulation Model for Dekalb County. It appears, however, that additional gaging sites are needed. Currently there is only one U.S.G.S. gage located on a small Watershed (less than 200 acres) in DeKlab County. Also, there are no gages in the southeast portion of the county, where exposed rock formations may cause unique hydrologic conditions. This document, then, discusses possible sites for the location of precipitation and streamflow gaging stations to supplement existing data sources.

In the course of developing sufficient facts on which to base proposals for gaging sites, numerous forms of data were collected, analyzed, and stored for future use. The factors generally considered were as follows:

a. Drainage area should be small, preferably less than 200 acres.
b. Some coverage is needed in the eastern portion of the county.
c. Future development in the watershed should be minimal
d. Sites must be accessible. (This provides an additional benefit of furnishing a control structure at culverts.)
e. New soil types should be included to broaden the base of calibration.

Steps which were taken to consider these factors included:

a. Composite map showing U.S.G.S. gages watersheds, DeKalb County study watersheds, and an indices for all map sources was developed.
b. A composite map of the coverage of all Corps flood plain information reports was developed. None of the proposed basins are covered by F. P. I. reports.
c. Meetings were held with county planning officials, the county drainage engineer, and the local S.C.S. and U.S.G.S. offices. The Corps was contacted by telephone.
d. A General Soil Map of the county was interpreted for appropriate data
e. Watershed areas and lengths were measured, 10-85% slopes computed, and land use percentages estimated.
f. Upon preliminary site selection, DeKalb officials were then consulted on the possibilities of future development in the watersheds. Opinions concerning development were based on interpretations of soil maps, sewer maps, and composite land use maps.
g. Field inspection was made of the proposed sites.

The proposed sites are discussed below.

a. Wesley Branch at Hightower Trail, a tributary to Yellow River. Drainage area is 180 acres. The outlet structure is a 36" CMP with about 3' for freeboard. Entrance conditions of the culvert need to be improved. Trees and foliage are very heavy at the culvert, so the raingage may have to located elsewhere. Soil is mostly type 8, with some type 9. Neither of these types are found in watersheds presently being gaged. There are no sewers in the watershed now, both the master plan proposes a truck line in the area eventually. Soil conditions are not conducive for development.

b. Pine Mountain Creek at Bruce Road, a tributary to Yellow River. Drainage area is 240 acres. The outlet structure is a double 5' X 5' box culvert with 3' of freeboard. Foliage at the culvert is very heavy. Soil is mostly type 9 with some type 18, neither of which are found wn Watersheds presently being gaged. Located in the town of Lithonia, the watershed is fairly well developed with residential and light commercial uses. There is a new sewer main now being constructed between Pine Mountain and the creek. Also, an industrial park zone lines just north of the watershed. Locating a gage in the town might have some public relations value. A county park is located on the right bank near the culvert, and would provide a good site for the raingage. This gage in conjunction with either site a or site c would permit comparison of natural and developed watersheds in the eastern part of the county where rock is so prevalent.

c. Crooked Creek at South Goddard Road, a tributary to the South River. This site has the largest drainage area, 470 acres. Three 36" C.M. pipes provide the outlet structure. There is about 4' of freeboard. Some foliage at the culvert entrance would need to be removed for gaging. Soil is about evenly divided between types 8 and 18, with one side of Arabia Mountain lying in the watershed. The soil conditions would pose moderate to severe problems for future development. There is no sewer in the area now; however, a trunk line is planned within the watershed for sometime in the future (around 1980). Watershed should be stable in the near future. Pastureland at the culvert provides an excellent raingage site.

d. East branch of Warren Creek above I-285 a tributary to the North Fork of Peachtree Creek. Drainage area is 270 acres. Outlet structure is a box culvert (size unknown) with unlimited freeboard. Extensive development is occurring in the watershed. An apartment complex is partially completed and restrictions placed on the construction site make access to the culvert difficult. Considerable modification to the creek channel has also occurred. Soils are divided about equally between types 5, 12, and 19, all of which occur in other watersheds now being gaged by U.S.G.S. However, soil type 19 does not comprise as large a percentage of the drainage area anywhere else. This site is obviously not stable; measured flows may not be consistent, and sediment, loads should be extremely heavy. This site might have value for comparison purposes or for filling this gap in a complete hydrologic data field. The watershed is one of the small basins listed by DeKalb County. Flood damage has occurred below I-285, and it may be worsened by present construction.

e. Unnamed tributary of Peavine Creek at Dickey Drive on the Emory campus. Drainage area is 53 acres. The culvert is a single 36" or 48" R.C. pipe with unlimited freeboard. Location of the raingage at the site may be difficult because of trees. The soil is all type 10, which is found in mnay of the gaged watersheds. The basin is heavily developed and contains a number of large buildings along with residential areas. It therefore should provide data for mixed-use, basins having commerical or light-industrial zones. The area should remain stable, with only limited growth.

f. Unnamed tributary to Peavine Creek in Fernbank Science Center. The site has a drainage area of 82 acres and is equally comprised of soil types 5 and 10. No details of the Culvert were obtained; several feet of freeboard appeared available. There should be no changes to this watershed in the foreseeable future, and it was hoped that this site would provide data on a completely natural watershed. The Science Center has a full-time metorologist on its staff, and good public relations might alos be part of the results. There are several small retarding structures on the creek from which supplemental data could be gathered. A rain gage currently exists at the center. Adverse features would be that the edge of the basin contains residental property, which tends to slightly decrease the completely natural aspect. Two paved roads meet at the proposed gage site; however, they are near the crest of the drainage basin and should not intercept too much flow. Despite these latter facts, this basin is the closest to a natural setting that was found in the western portion of the County.

g. Wildwood Creek tributary to North Fork Peachtree Creek. Drainage area is 320 acres. The soil is evenly divided between types 5 and 12. The watershed is a well established residential neighborhood with commerical zones beginning to encroach along the edges (Lavista Road and I-85). Also, widening of Lavista Road in this area may occur. The culvert is a large (approximately 8'x 8') single box culvert. There is significant foliage in the stream channel which approach the culvert at about a 45° angle. There has been flood damage on this watershed in the past, and a retaining structure has recently been constructed by the county at the intersection of Wild Creek trail and Broodforest Road. It might be interesting to determine the effectiveness of this structure, and direct gage data would be valuable. The Watershed is one of the small basins listed by DeKalb County for our study.

APPENDIX II

OUTLINE OF TASKS

A. Instrument watersheds

(1) Identify criteria and needs for selection of sites (eg. soil type, slopes, existing gages, land-use, stability of land-use, accessibility)
(2) Order recorders and rain guages
(3) Select potential sites which meet criteria
(4) Field trip to sites
(5) Select 5 sites, write memo on where and why, distribute to DeKalb County and USGS for comment
(6) Order additional supplies for gage installations
(7) Get approval from land owners
(8) Install gages
(9) Service gages
(10) Code data from gages

B. Select method for computer storage and retrieval of land-use and stream channel information

(1) Meet with Brenda Day for information on parcel file
(2) Determine computer capabilities of DeKalb County
(3) Interview agency persons in Atlanta, EPA, Ga. Department of Natural Resources, etc, for other systems
(4) Consider other possibilities
(5) Evaluate and select system or systems

C. Conduct sensitivity studies to determine how best to subdivide channels (Figure 4) (Use storms on Clairmont watershed, North Fork Camp Creek and Camp Creek)

(1) Select 3 levels of subdividing channels and calculate flow. Then compare discrepancy in simulated and measured flow with ratio of average reach flow time to total watershed flow time.
(2) If physical situations can be found, check need for splitting flow when flow gets above a certain discharge.
(3) Develop regression equations for parameters a and m in rating, $Q = aA^m$, of channel, then use regression equation instead of field data for parameters a and m
(4) Study effect of considering or ignoring man-made channel constrictions, debris blockages, sensitivity of culvert rating on downstream flood hydrographs
(5) Study sensitivity of number of subareas and time distributions of direct runoff from subareas to develop rules for subdividing drainage area.
(6) Select method for time distribution of direct runoff based on above sensitivity and check with measured hydrographs.

D. Develop routing methods that approximate backwater situations (Figure 4)

(1) Select a variety of channel constrictions creating backwater (various channel slopes, flood plain sizes, types of constriction)
(2) Measure critical dimensions
(3) Estimate Q_{2-yr}, Q_{10-yr} and Q_{100-yr} (from USGS frequency relation for this region) and compute backwater (or collect high water-mark mark data of past flood)
(4) Determine input requirements (channel lengths, Q vs A relation) for kinematic routing that best approximates the backwater profile

E. Collect and code hydrologic data

(1) Order daily streamflow data on computer cards from USGS for North Fork Camp Creek, South River @ East Point, South Utoy Creek, Wildcat Creek, Pew Creek, Shetley Creek, Garner Creek, and Yellow River.
(2) Order stage hydrographs for 2 to 4 storm periods per year and rating tables from USGS for Camp Creek, North Fork Camp Creek, South Utoy Creek and South River at East Point
(3) From data in item (2) above calculate discharge hydrographs
(4) Stream cross-section data from Corps of Engineers
(5) Obtain data from USGS for recently installed stream gages in DeKalb County
(6) Code rainfall and discharge data for Clairmont watershed

F. Map watersheds (1st-watersheds for sensitivity studies, 2nd-USGS watersheds in DeKalb County and 3rd-priority watersheds for study)

(1) On 1" = 200' maps determine drainage divide
(2) Divide drainage network into channel segments (3 levels-gross, moderate and detailed for watersheds for sensitivity studies)
(3) Number channel segments (from upstream to downstream) and measure lengths.
(4) Divide drainage area into subwatersheds (3 levels-gross, moderate and detailed for watersheds for sensitibity studies).
(5) Number subwatersheds, determine the channel segment into which each flows, measure area of subwatersheds and estimate fraction of each subwatersheds in forest, opne, each soil type, and impervious surfaces.

G. Take field trips to:

(1) Sites of USGS stream gaging stations in DeKalb County
(2) Priority watersheds selected by DeKalb County
(3) Locate potential sites for additional gaging stations
(4) Watersheds used in sensitivity studies for
  (a) location of major inlets and outfalls
  (b) channel and flood plain cross-sections, roughness and slope at outlet of each reach and
  (c) dimensions of constrictions
(5) USGS gaged watersheds in DeKalb County for information in (4) above
(6) DeKalb priority watersheds for information in (4) above.

H. Select model to calculate the volume of runoff from rainfall (Figure 2)

(1) Simulate Camp Creek, North Fork Camp Creek, South River @ East Point, South Utoy Creek and Clairmont watershed with the Georgia Tech, Kentucky and National Weather Service versions of SWM
(2) Select best model for DeKalb County based on accuracy of predicted flood volumes.
(3) Determine needed sets of basic hydrologic parameters.

I. Simulate to develop runoff file (Figure 3)

(1) Simulate for Clairmont watershed (15 acres) South Utoy (480 acres) and North Fork Camp Creek (3170 acres) with 80-years of Atlanta Airport precipitation data
(2) Determine for 80-years of simulated flood peaks which years and/or hypothetical storms are required to adequately define flood frequency
(3) For each set of parameters including one for impervious areas, store runoff, without routing, on computer file.

J. Prepare manual on use of system

K. Meet with DeKalb County officials

(1) Make final selection of priority watersheds
(2) Make presentation to describe capabilities of simulation and discuss and list measures to be included in simulation studies

L. Apply simulation model (basically routing model using runoff from file) to priority watersheds selected by DeKalb County one at a time (Figure 5).

M. Final Report.

Figure 1. DeKalb Drainage Project

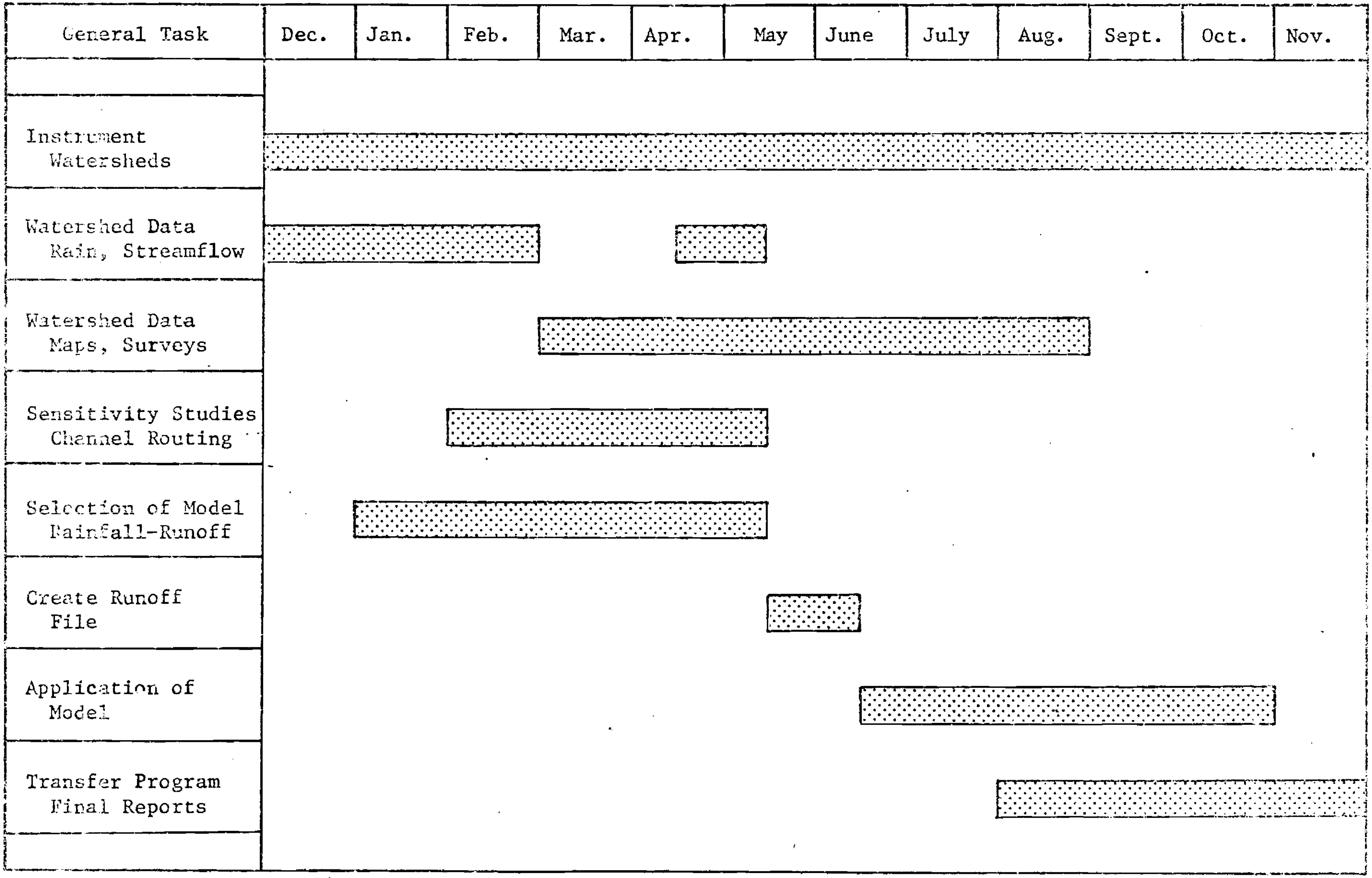


Figure 2. Selection of Rainfall-Runoff Model

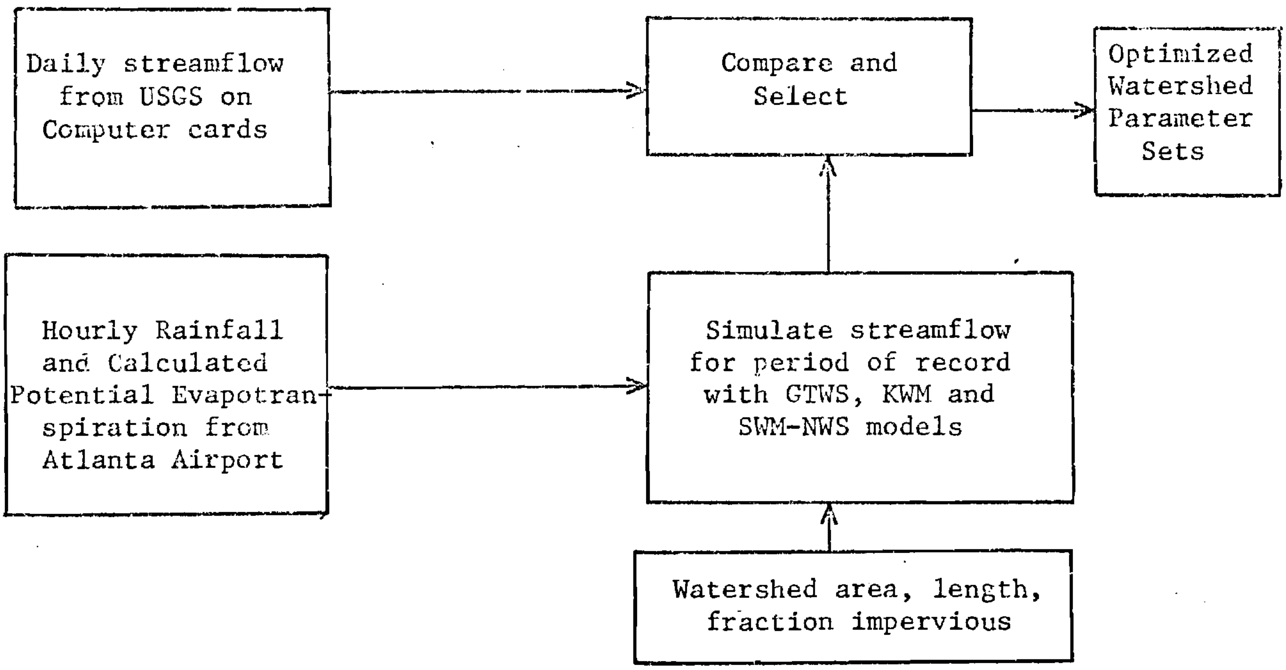


Figure 3. Creation of the Runoff File

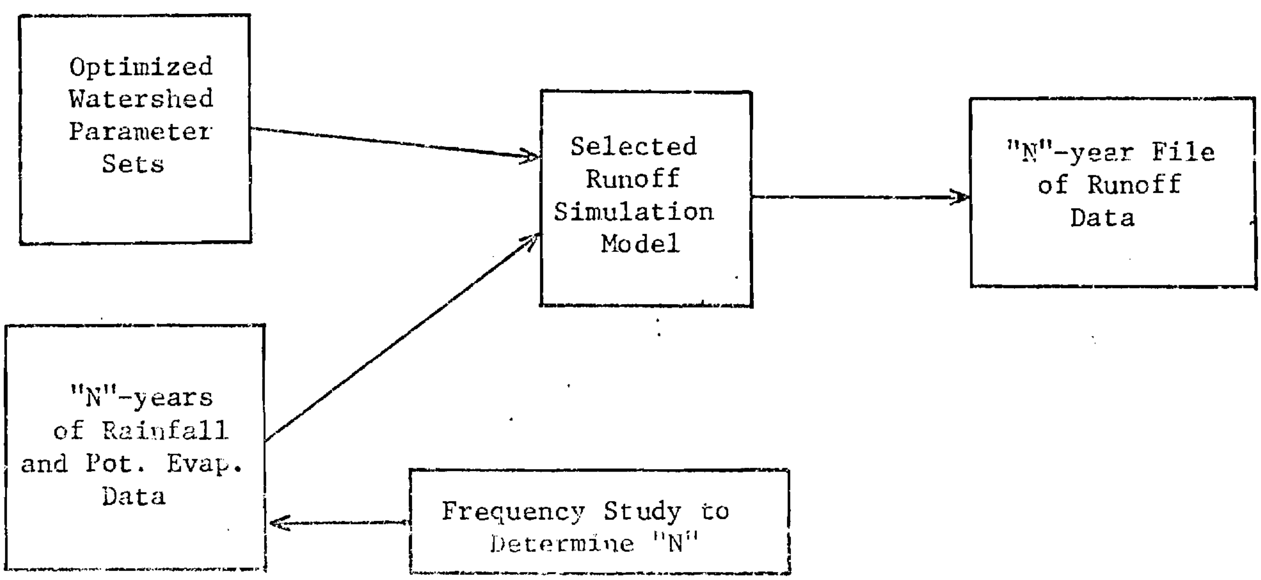


Figure 4. Sensitivity Studies for Channel Routing

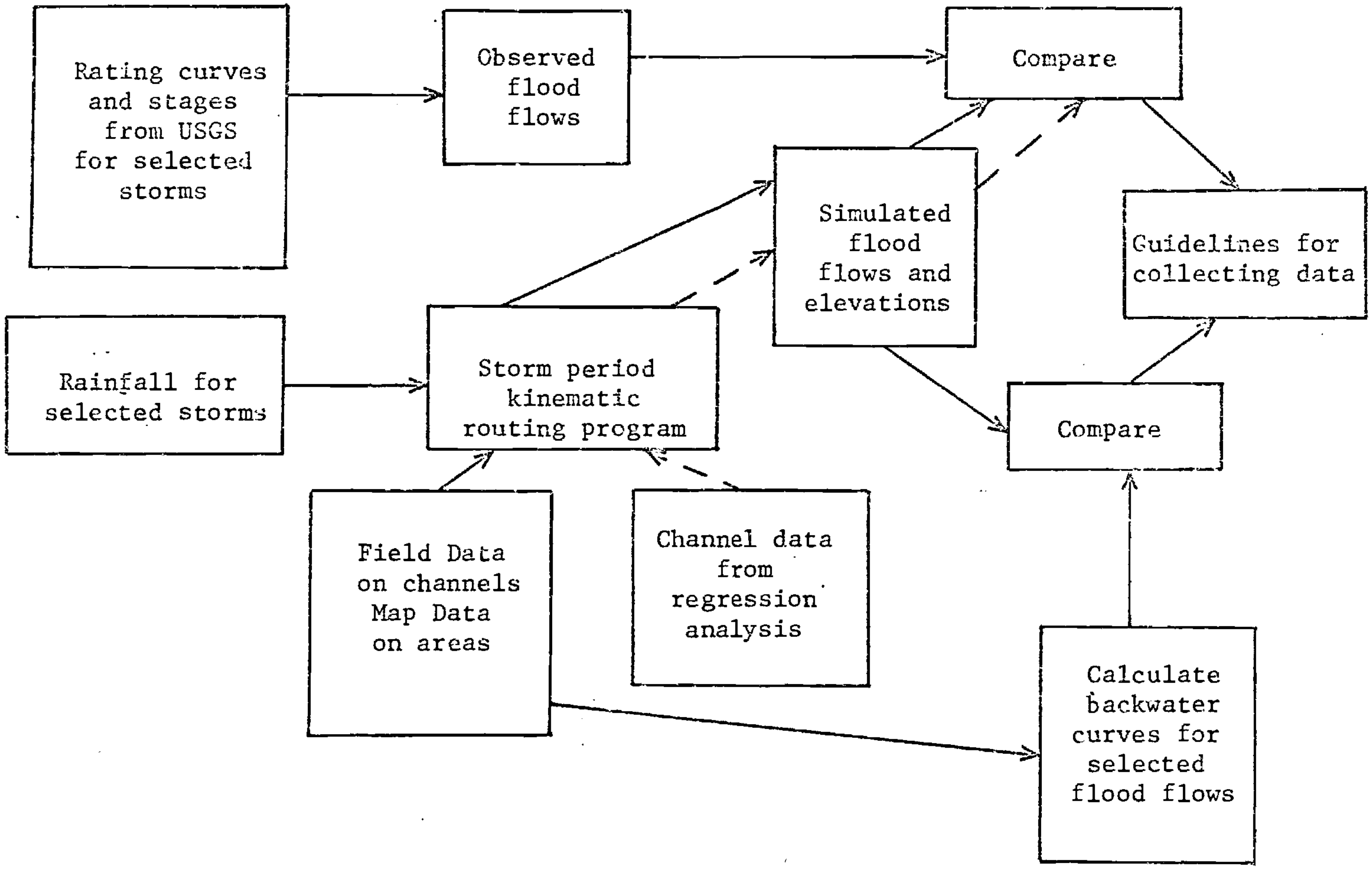
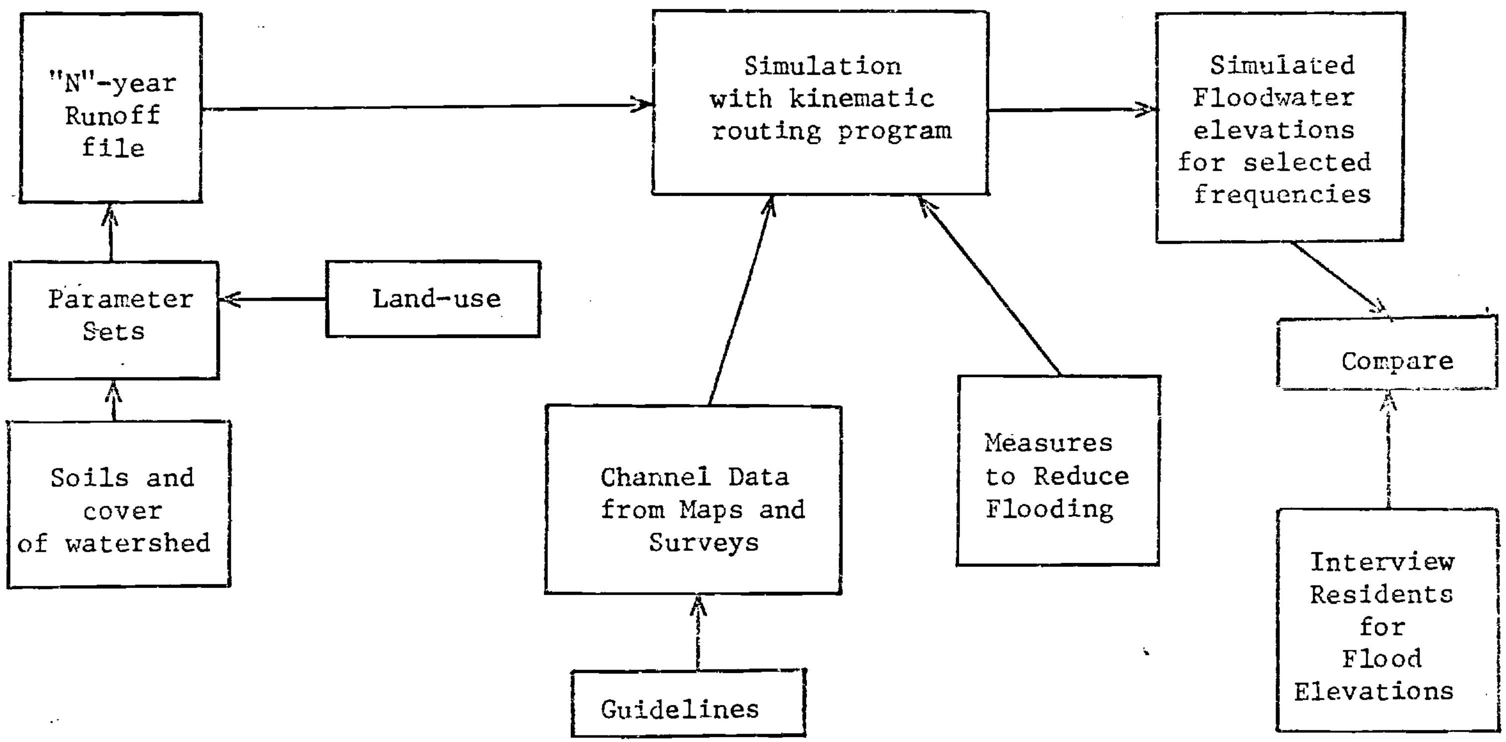


Figure 5. Model Application to Priority Watersheds



PROGRESS REPORT

For the Period December 17 to December 31, 1973

Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project Number: E-20-646

Project Director: Alan M. Lumb

During December, 1973, space for the project was made available in the lab room on the 4th floor of the Civil Engineering Building. Furniture was rearranged to meet the needs of the project and additional files and shelves moved into the room for storage of data publications, computer cards, and maps. Site visits were made to priority drainage basins and locations of USGS stream gages. The assessment and acquisition of information and data for the project began.

Specific accomplishments include the following:

(1) Collection of maps, aerial photographs, and other information of DeKalb County,
- a) Soil map of DeKalb and adjacent Counties (from ARC)
- b) Flood Insurance Study, City of Decatur, by Corps of Engineers
- c) Flood Insurance Study, DeKalb County, by Corps of Engineers
- d) Flood Plain Information Reports of Corps of Engineers for creeks of DeKalb County
- e) Aerial photographs, Peachtree Creek Basin, 1"=400'
- f) Inventory of Structures on Main Streams, DeKalb County, report prepared June, 1972, by C. N. Crocker, Drainage Division
- g) Stream cross-sections on Peavine Creek, Henderson Creek and Burnt Fork Creek from Corps of Engineers
- h) U.S. Geological Survey maps (1:24,000) of DeKalb County and surrounding areas
- i) Maps of DeKalb County entitled "DeKalb Drainage and Flood Plains" and "Road System", from Department of Planning
- j) Maps of DeKalb County (1"=200')

(2) Ordering and receiving daily streamflow data on computer cards for 8 stream gages in the Atlanta area from the U.S. Geological Survey, District Office, Atlanta
   North Fork Camp Creek @ Atlanta (1964-1969)
   South Utoy Creek @ East Point (1964-1969)
   South River @ East Point (1964-1969)
   Wildcat Creek near Lawrenceville (1954-1971)
   Pew Creek near Lawrenceville (1954-1963)
   Shetley Creek near Norcross (1954-1963)
   Yellow River near Snellville (1954-1971)
   Garner Creek near Snellville (1954-1963)

(Data for Peachtree Creek and Camp Creek were already available at Georgia Tech on computer cards.)

(3) Color coding the following information on the Department of Planning map entitled "DeKalb Drainage and Flood Plains"
   a) Stream channel reaches for which Corps of Engineers flood plain information is available,
   b) Location of stream gages operated by the U.S. Geological Survey,
   c) Priority drainage basins selected by DeKalb County officials

(4) Site visits by A. M. Lumb, L. D. James, J. R. Wallace, Carvel Deese and John Clerici to
   a) 11 of the 13 stream gages in DeKalb County operated by the U.S. Geological Survey under cooperative agreement of the USGS, DeKalb County and Corps of Engineers, and
   b) 7 of the 12 priority drainage basins including Henderson Creek, Peavine Creek, Warren Creek, North Fork of Peachtree, Nancy Creek, Cobbs Creek and Wommack Creek.

PROGRESS REPORT

For the Period February 1 to February 28, 1974

Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project Number: E-20-646

Project Director: Alan M. Lumb

During February 1974, precipitation data was coded, four watersheds were calibrated with a simulation model, and soil permeability categories were mapped for DeKalb and adjacent counties.

Five-minute precipitation data were coded from the strip charts for the 15-acre residential watershed near Clairmont and LaVista Roads for a period from July, 1971, through July, 1972. The remainder of the record will be coded in March. The 80-year record of 15-minute precipitation for Atlanta on magnetic tape was sent to Georgia State to be placed on another tape which would be compatible with the Georgia Tech computer system.

Calibration with one of the three simulation models to be used to generate the runoff file was completed for the five years of data on South River at East Point, North Fork Camp Creek, and South Utoy Creek, and for the eight years of data on Camp Creek near Fayetteville. This work will continue with calibration of the above drainage basins with the other two models; then the best of the three will be calibrated on three Gwinnett drainage basins for which data has been obtained.

Additional soils information was obtained through Grover Thomas of the Soil Conservation Service. This information has been summarized as Appendix I. From this information, the soil associations of the five county region were placed in five soil permeability categories and these categories were color coded on the soils maps of the five counties.

All the student assistants for the project have been selected and their percent time in each of the four quarters determined. A listing of the students with a brief comment is found in Appendix II. The expenditure of funds for student assistants by quarters is 20% first quarter, 30% second quarter, 35% third quarter and 15% last quarter. These percentages correspond to the work load as outlined in Appendix II of the January progress report.

APPENDIX I

SUMMARY OF SOIL PERMEABILITY

The characteristic of the soil that enables it to transmit water is called soil permeability. Seven classes of soil permeability are used by the Soil Conservation Service and are listed in the following table:

Permeability class	Numerical Range (inches per hour)	Average Rate (inches per hour)
Very slow	Less than 0.06	-
Slow	0.06 - 0.2	0.13
Moderately slow	0.2 - 0.6	0.40
Moderate	0.6 - 2.0	1.30
Moderately rapid	2.0 - 6.0	4.00
Rapid	6.0 - 0.0	13.00
Very Rapid	More than 20	-

Another classification of soils can be achieved with the four hydrologic soil groups of the Soil Conservation Service indicating their runoff potential. These are listed in the following table.

Hydrologic Soil Group	Description
A	Soil with lowest runoff potential. Deep sands with very little silt and clay.
B	Mostly sandy soils less deep than A.
C	Shallow soils and soils containing considerable clay.
D	Soil with highest runoff potential. Clay soils and shallow soils with impermeable subhorizons.

Twenty-one soils are found in the five-county area and are listed below with their average permeability and hydrologic soil group.

Soil	Permeability (inches/hour)	Hydrologic Soil Group
Altavista	2.0	C
Appling	1.8	B
Alluvial	4.0	B
Cecil	1.8	B
Chewaela	1.3	C
Congaree	1.9	B
Davidson	1.8	B
Gwinnett	1.3	B
Iredell	0.2	D
Linker	2.2	B
Louisa	2.2	B
Louisburg	13.0	B

Soil	Permeability (inches/hour)	Hydrologic Soil Group
Madison	1.3	B
Mecklenburg	0.2	C
Musella	2.2	B
Pacolet	1.8	B
Red Bay	1.3	B
Wedowee	2.2	B
Wehadkee	1.8	D
Wickham	1.8	B
Wilkes	0.6	C

These twenty-one soils are grouped into seventeen soil associations for the five county region and are listed in Table I-1. Table I-2 lists the weighted permeabilities and hydrologic group for each of the soil associations. From the permeability information on Table I-2 the soil associations were placed in one of five permeability catagories as indicated below.

Permeability	Soil Associations	Color Code for Map
Very Slow (VS)	18	Red
Moderately Slow (MS)	2	Yellow
Moderate (M)	1A,3,4,5,6,7,10,13,15,16	None
Moderately Rapid (MR)	1,9,11,12,14	Green
Rapid (R)	8,17	Blue

Table I-1.

SOIL#	SOIL ASSOCIATION	SOIL#	%COMPONENTS 1st	2nd	3rd
1	ALLUVIAL LAND-CHEWACLA-WEHADKEE	1	60	20	10
1A	CONGAREE-CHEWACLA-WEHADKEE	1A	50	20	15
2	WILKES-IREDELL-MECKLENBURG	2	50	20	20
3	MADISON-LOUISA-PACOLET	4	45	25	15
4	APPLING-CECIL-MADISON	5	40	25	20
5	MADISON-PACOLET-MUSELLA	6	45	25	15
6	GWINNETT-DAVIDSON-MUSELLA	7	45	25	15
7	GWINNETT-DAVIDSON-MUSELLA	8	40	30	20
8	LOUISBURG-WEDOWEE-PACOLET	9	50	25	15
9	APPLING-LOUISBURG-PACOLET	10	40	30	20
10	MADISON-PACOLET-GWINNETT	11	60	15	10
11	LINKER-LOUISBURG-MISELLA	12	40	35	10
12	PACOLET-GWINNETT-LOUISBURG	13	65	20	10
13	WICKHAM-ALTAVISTA-RED-BAY	14	60	20	10
14	APPLING-PACOLET-LOUISBURG	15	50	30	10
15	WILKES-GWINNETT-MUSELLA	16	40	35	10
16	APPLING-PACOLET-GWINNETT	17	37	35	21
17	LOUISBURG-PACOLET-WEDOWEE	18	-	-	-
18	ROCK OUTCROP	19	-	-	-
19	MADE LAND	20	-	-	-
20	UNCLASSIFIED				

Table I-2.

SOIL#	SOIL ASSOCIATION	%SLOPE	PERMEABILITY				HYDROLOGIC GROUP			
			1st	2nd	3rd	wt ave	1st	2nd	3rd	wt ave
1	ALLUVIAL LAND-CHEWACLA-WEHDKEE	0-2	MR	M	M	3.16	B	C	D	B
1A	CONGAREE-CHEWACLA-WEHADKEE	0-2	M	M	M	1.74	B	C	D	B
2	WILKES-IREDELL-MECKLENBURG	2-10	MS	S	S	.42	C	D	C	C
3	MADISON-LOUISA-PACOLET	10-45	M	MR	M	1.65	B	B	B	B
4	APPLING-CECIL-MADISON	2-10	M	M	M	1.74	B	B	B	B
5	MADISON-PACOLET-GWINNETT	2-10	M	M	M	1.45	B	B	B	B
6	GWINNETT-PACOLET-MUSELLA	10-45	M	M	MR	1.61	B	B	B	B
7	GWINNETT-DAVIDSON-MUSELLA	2-10	M	M	MR	1.61	B	B	B	B
8	LOUISBURG-WEDOWEE-PACOLET	10-45	R	M	M	6.91	B	B	B	B
9	APPLING-LOUISBURG-PACOLET	2-10	M	R	M	4.91	B	B	B	B
10	MADISON-PACOLET-GWINNETT	10-25	M	M	M	1.47	B	B	B	B
11	LINKER-LOUISBURG-MUSELLA	10-60	MR	R	MR	4.11	B	B	B	B
12	PACOLET-GWINNETT-LOUISBURG	10-45	M	M	R	2.91	B	B	B	B
13	WICKHAM-ALTAVISTA-RED BAY	2-10	M	M	M	1.79	B	C	B	B
14	APPLING-PACOLET-LOUISBURG	2-10	M	M	R	3.04	B	B	B	B
15	WILKES-GWINNETT-MUSELLA	10-45	MS	M	MR	1.01	C	B	B	C
16	APPLING-PACOLET-GWINNETT	2-10	M	M	M	1.84	B	B	B	B
17	LOUISBURG-PACOLET-WEDOWEE	10-45	R	M	M	6.35	B	B	B	B
18	ROCK OUTCROP	NOT GIVEN	VS			0.0	D			D
19	MADE LAND	NOT GIVEN								
20	UNCLASSIFIED									

Maps for five counties were developed showing the area in each of the five permeability catagories.

Each of the drainage basins gauged by the U.S. Geological Survey, studied for gauging by the Geological Survey and studied for gauging by personnel on this project were traced on the soils maps and weighted permeabilities calculated. These results are shown on Table I-3. The table and map indicate a need for gauge sites in the southwest portion of DeKalb where soils have moderately slow permeabilities and along the eastern boundary where soils of rapid permeability are mixed with rock outcrop. These two areas of DeKalb County will be studied for additional gauge sites in March.

Table I-3.

Name or USGS Site #	Soils Associations and Percent in Drainage Basin				Weighted Permeability (inches/hour)
Potential					
10	1-6%	10-38%	5-56%		1.57
55	1-14%	10-35%	5-47%	20-4%	1.70
28	12-46%	5-54%			2.12
31	1-5%	12-42%	5-53%		2.16
17	1-14%	10-42%	5-44%		1.70
13	1-16%	10-13%	5-14%	20-57%	2.11
5	1-12%	12-30%	5-58%		2.10
19	1-13%	12-46%	5-41%		2.34
Crooked Creek	1-3%	8-43%	18-54%		3.06
Pine Mtn. Ck.	9-94%	18-6%			4.60
Wesley Creek	8-34%	9-57%	18-9%		5.14
Existing					
1	1-9%	10-38%	5-53%		1.61
7	1-11%	12-25%	10-12%	5-52%	2.01
8	1-11%	12-33%	5-56%		2.12
9	1-9%	12-15%	10-26%	5-50%	1.83
12	1-12%	10-35%	5-31%	20-22%	1.72
16	1-14%	10-39%	5-47%		1.69
21	1-10%	12-47%	5-43%		2.31
26	1-7%	10-9%	5-15%	20-69%	1.82
27	1-9%	10-32%	5-59%		1.61
29	12-44%	5-59%			2.09
32	1-8%	12-43%	5-49%		2.21
57	1-14%	10-41%	5-45%		1.70
2-3371	1-10%	12-51%	5-39%		2.37
2-3367	1-4%	10-44%	5-52%		1.53
2-2036	20-100%				-

APPENDIX II

John Clerici - Undergraduate student in Civil Engineering who will attend graduate school at Georgia Tech starting September. He will be with the project the entire year.

Tim Hassett - Graduate student in the water resources program in Civil Engineering. He will be with the project summer quarter.

Jack Kittle - Undergraduate student in Civil Engineering. He will be with the project the entire year. Jack has lived in DeKalb County for 12 years and his familiarity with the County will be quite helpful.

Paul Nowak - Graduate student in Information and Computer Science. His expertise will be very helpful. He will be with the project through August.

Adnan Saad - Graduate student in the Ph.D. program in hydrology and water resources. Adnan brings to the project his practical experience in hydrology gained over the past several years working for an Atlanta consulting firm. He will be with the project starting in April.

Ed Sing - Graduate student in the water resources program in Civil Engineering. He will be with the project through August.

PROGRESS REPORT

For the Period of April 1 to April 30, 1974

Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project Number: E20-646

Project Director: Alan M. Lumb

During April, 1974 progress continued with calibration of the watershed models, GTWS, KWM and SWM, which will be used for the creation of the runoff file and the sensitivity studies for subdividing the drainage system into reaches.

The two undergraduate students on the project, John Clerici and Jack Kittle, gave a talk at the May meeting of the Georgia Section of the American Society of Civil Engineers. Both gave a very fine presentation.

The instruments needed for the rain and stream gage stations were finally delivered May 10th. The following six locations for the stations have been selected from over a dozen which were studied. Joe Inman of the U.S. Geological Survey has visited each of the sites and found no major complications in gaging the streams at these locations. Installation will begin when permission to locate the gages on private property or right-of-way can be obtained. The locations are:

Gage 1. Tributary to South River at I-285 between Moreland Avenue, and Forest Park Rd., LL17, 15th District.

Gage 2. Tributary to Jackson Creek, LL259, 15th District.

Gage 3. Wesley Branch at private dirt road, on boundary of LL193 & 194, 16th District

Gage 4. Honey Creek at Turner Hill Rd. near corner of LL170, 171, 182, & 183, 16th District

Gage 5. Tributary to Womack Creek at Leeds Way, LL361, 18th District.

Gage 6. Tributary to Snapfinger Creek at Arbington Drive, LL93, 18th District.

Two meetings were held on April 19 and May 3 with staff of the Planning and the Roads and Drainage Departments to make preliminary assignments of priorities for studies on selected drainage basins. The following watersheds and priorities were selected.

(1) Womack Creek (tributary to Nancy Creek)
Drainage Area: 420 acres, 0.44 sq. mi.
Soil Permeabilities: 43% moderately rapid
57% moderate
Flooding residential area around Cambridge Road.

(2) Nancy Creek
Drainage Area: 4880 acres, 7.63 sq. mi.
Soil Permeabilities: 53% moderately rapid
43% moderate
5% heavily graded
Flooding in area of Gainesboro Drive and Tilly Mill Road. Sites for retention storage available.

(3) Cobbs Creek
Drainage Area: 2330 acres, 3.64 sq. mi.
Soil Permeabilities: 35% moderately rapid
65% moderate
Residential flooding and undersized culverts at Misty Valley Road, Bobby Lane, Beach Drive and Brookfield Drive. Site available for retention storage.

(4) Honey Creek
Drainage Area: 2690 acres, 4.2 sq. mi.
Soil Permeability: 20% rapid
77% moderately rapid
3% very slow
Undeveloped area in the southeast near I-20 and Lithonia which will be developing in the near future.

(5) Snapfinger Creek (west branch)
Drainage Area: 1240 acres, 1.94 sq. mi.
Soil Permeability: 13% moderately rapid
87% moderate
Flooding of commercial establishments at Memorial Drive. Potential site for retention storage.

(6) Snapfinger Creek
Drainage Area: 4860 acres, 7.59 sq. mi.
Soil Permeabilities: 14% moderately rapid
83% moderate
3% heavily graded
Residential flooding at Indian Lake Circle and Susan Creek Court.

(7) Wild Creek
Drainage Area: 299 acres, 0.47 sq. mi.
Soil Permeabilities: 51% moderately rapid
49% moderate
Residential flooding between I-85 and Sherdan Road. Site for retention structure available.

(8) Warren Creek and adjacent tributaries
Drainage Area: 1140 acres, 1.78 sq. mi.
Soil Permeabilities: 42% moderately rapid
36% moderate
22% heavily graded
Residential flooding at Santa Fe Trail and flooding industrial area on Marian Road. Undersized culvert at McClove Drive. Potential sites for retention structures.

(9) North Fork Peachtree Creek
Drainage Area: 6098 acres, 9.53 sq. mi.
Soil Permeabilities: 21% rapid
27% moderately rapid
37% moderate
15% heavily graded
Flooding problems along Barkside Court in addition to the problems on Warren Creek. Some channel maintenance work has been done.

(10) Blue Creek

(11) South Fork Peachtree Creek and Crocker Creek

(12) Peavine Creek

(13) Burnt Fork Creek

(14) Henderson Mill Creek

PROGRESS REPORT

For the Period of May 1 to August 31, 1974

Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project Number: E20-646

Project Director: Alan M. Lumb

Several meetings were held between Georgia Tech and DeKalb County during this summer period. May 3rd was a meeting to describe the project to new personnel and discuss the location of the stream gage stations. Several subsequent meetings concerned the necessary right-of-way required for the installation of the gages. In July, a meeting was held with staff members of the Planning Department and Roads and Drainage Department. Discussed at this meeting was the hydrologic simulation process, data requirements for hydrologic simulation, and a preliminary outline of the problems and potential solutions for each of the nine drainage areas selected for analysis. Written response to this outline was received in August from the Development, Planning, and Roads and Drainage Departments.

Equipment for gage installation has been received and the right-of-way secured for three of the six sites. Dr. Wallace has the responsibility for the installation of these gages.

Calibration of the Georgia Tech variation of the Stanford Watershed Model (SWM) has been completed. This model will be used to create the runoff file. Problems have been encountered in developing the 72 years of precipitation to use with SME to create the runoff. Most of this problem and delay came from the need to change personnel assigned to this task.

The bugs were eventually worked out of the computer program needed to perform this specific task. This two month delay has not held up the entire project but did require rescheduling several tasks. Seventy-two years of runoff has been simulated and is currently being analyzed to determine the appropriate number of years and the time period to use to generate the runoff file. Subsequent to this analysis, all tasks can proceed.

All nine drainage basins have been (1) mapped, (2) subwatersheds and channel reaches delineated and (3) photo mosaics and overlays for impervious area prepared. Much of the channel and culvert data remains to be collected.

Twice in July students ran programs after midnight on the DeKalb computer. Several problems were encountered and the reprogramming to overcome the problems has been completed and awaits another trial.

Rough drafts of portions of the manual and report have been written.

A few remaining sensitivity studies on the channel routing are needed. All other sensitivity studies are complete.

PROGRESS REPORT

For the Period of September 1 to September 30, 1974

Utilization of a Computer Model to Determine the Impact of
Urban Development on Flooding in DeKalb County

Project Number: E20-646

Project Director: Alan M. Lumb

One of the six watersheds selected for rainfall and streamflow measurement has been dropped from the list due to lack of property - owner cooperation. An additional site will be selected to replace the one dropped from the list.

All problems in generating the runoff file have been solved and the seventy-two years of simulated runoff has been analyzed. A study has been made of all possible 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60 and 65 year periods within the 72-year period and it was found that the 25-year period from 1918 to 1942 was most representative of the 72-year period. Thus, runoff files for this 25-year period were created and are ready for use.

Forty percent of the channel, culvert and detention structure data was collected for the nine drainage areas to be studied. The remaining sixty percent will be collected in October.

Features were added to the computer program to make the input data preparation much easier for water detention structures and culverts. By giving the basic dimensions the program can handle circular, elliptical and rectangular culverts, broad-crested weirs as roadways or spillway, and drop inlets. The description of the input requirements that we are currently using is attached as an Appendix to this report. A much more elaborate description will be put in the user's manual.

URBAN WATERSHED FLOOD HYDROGRAPH MODEL (UROS4)

PROGRAMMED BY ALAN M. LUMB , ALLEN F. JOHNSON,
L.D. JAMES AND J.L. KITTLE, JR.

***** INPUT REQUIREMENTS *****

VARIABLE ON CARD	FORMAT	COMMENT
INFC(1), INFC( ), RUNCPT	I6,12A6, I2	NUMBER OF RUNOFF FILES TO BE USED (IF LEFT BLANK, 2 ASSUMED), GENERAL INFORMATION ON RUN (NAME OF WATERSHED, PURPOSE OF RUN), RUNOPT=1 USE RUNOFF FILE RUNOPT=0 SINGLE STORM SIMULATION
NELMTS,TOTALT, DELT,TUNITS,AU, PK,CFCHK,STCHK	I4,2F4.0, 2A4,4F8.0	NUMBER OF HIGHEST SEGMENT(MAX.99), TOTAL TIME, TIME INCREMENT, ((TOTAL TIME)/(TIME INCREMENT)MUST BE LESS THAN 121), TIME UNITS(SEC=1 OR MIN=60 OR HRS=3600), AREA UNITS(ACRE=1 OR SQMI=640) (IF TUNITS AND AU LEFT BLANK, MINUTES AND ACRES ASSUMED), COEFFICIENT FOR LINEAR STORAGE ROUTING OF EXCESS FROM AREA SEGMENTS (LEAVE BLANK FOR DEFAULT VALUE OF 1.0), ACCURACY OBJECTIVE FOR ITERATIVE APPROXIMATIONS FOR OVERLAND FLOW AND STORAGE SIMULATION (LEAVE BLANK). NOTE: WHEN USING RUNOFF FILE DELT MUST BE 5 OR 15 MINUTES DEPENDING ON FILE USED.

*** INPUT PHYSICAL DISCRIPTORS OF EACH ELEMENT (LOOP NELMTS TIMES)

VARIABLE ON CARD	FORMAT	COMMENT
N,TYPE( ), INFC( )	3I4	ELEMENT NUMBER (SEQUENCE OF NUMBERS FROM 1 TO 99, SMALLER NUMBERS MUST ALWAYS DRAIN TO A LARGER NUMBER),ELEMENT TYPE (AREA=11,CHANNEL=22,STORAGE=33, DUMMY = 55, QUIT=99) , ELEMENT NUMBER INTO WHICH THIS ELEMENT DISCHARGES. NOTE: NUMBERS MAY BE SKIPPED- EITHER IGNORE OR SET TYPE = 55 NOTE: AT END OF DISCRIPTOR CARDS

```
                                   PUT ELEMENT WITH N =
                                   100 AND TYPE( ) = 99

* IF TYPE( ) = AREA (11) AND RUNOPT = 0 INPUT THE FOLLOWING

EA( ),LENGTH( ),      9F8.0       AREA OF ELEMENT,LENGTH,SLOPE,
CPE( ),FF( ),                     MANNING'S N,INITIAL LOSS IN
( ),RTL( ),ER( ),                 INCHES/DELT(ANYTHING FROM 0.0 TO
TKR( ),U( )                       10.0 DEPENDING UPON ANTECEDENT
                                  RAINFALL), SLOPE OF LOSS
                                  FUNCTION (RATIO OF LOSS
                                  TO THAT AFTER 10 INCHES OF RECHARGE
                                  -USUALLY 3.0), RAIN EXPONENT
                                  (USUALLY 0.7 OR 1.0  FOR
                                  RATIONAL FORMULA TYPE LOSS, 0.001
                                  FOR INFILTRATION FCN TYPE LOSS),
                                  SURFACE RETENTION VOLUME(INCHES)
                                  (ANYTHING FROM 0.0 TO 2.0 DEPENDING
                                  UPON ANTECEDENT RAINFALL),
                                  FRACTION IMPERVIOUS AREA.

                                  NOTE: IF AREA OVER
                                      TEN ACRES OR LINEAR
                                      STORAGE DESIRED FOR ROUTING
                                      TO OUTLET LEAVE VALUE FOR
                                      FF( ) BLANK

* IF TYPE( ) = AREA(11) AND RUNOPT=1 INPUT THE FOLLOWING

EA( ),LENGTH( ),      4F8.0,8X,   AREA,LENGTH,SLOPE AND FF SAME AS
CPE( ),FF( ),         4F8.0       ABOVE, SOIL1 THRU SOIL4 ARE
IL1( ),SOIL2( ),                  FRACTION OF AREA WITH RAPID,
IL3( ),SOIL4( )                   MODERATE,SLOW OR ZERO SOIL
                                  PERMEABILITIES,RESPECTIVELY
                                  (ZERO PERMEABILITIES FOR IMPERVIOUS
                                  SURFACES)

                                   NOTE: SUM(SOIL1+SOIL2+SOIL3+SOIL4)
                                         MUST=1.0 FOR EACH SUBWATERSHED

* IF TYPE( ) = CHAN (22) INPUT THE FOLLOWING INFORMATION

ND( ),LENGTH( ),      I8,9F8.0    KIND OF CHANNEL(0,1,2,3 OR 4 AS
CPE( ),FF( ),                     DEFINED BELOW), LENGTH OF CHANNEL
ARM( ),MPARM( ),                  (FEET), SLOPE OF CHANNEL(FT/FT),
                                  MANNING'S 'N' FOR CHANNEL,
                                    KIND OF CHANNEL OPTIONS:
                                  0=IRREGULAR, LEAVE SLOPE AND FF
                                    BLANK, FLOOD PLAIN AND CHANNEL
                                    FLOW SIMILAR, INPUT VALUES FOR
                                    APARM AND MPARM
                                  1=RECTANGULAR, LEAVE MPARM BLANK
                                    AND SET APARM EQUAL TO CHANNEL
                                    WIDTH
                                  2=TRIANGULAR, LEAVE MPARM BLANK
                                    AND SET APARM EQUAL TO WIDTH
                                    AT ONE FOOT DEPTH
```

3=CIRCULAR, LEAVE MPARM BLANK AND
   SET APARM EQUAL TO PIPE DIAMETER
   (SEGMENT SHOULD BE PRECEEDED
   BY A STORAGE SEGMENT WITH THE
   OUTLET OF THE STORAGE SEGMENT THE
   SAME AS INLET TO THE PIPE)
4=IRREGULAR, FLOOD PLAIN AND
   CHANNEL FLOW DIFFERENT, APARM IS
   FOR CHANNEL CROSS-SECTION AREA,
   MPARM IS FOR WETTED PERIMETER
   OF THE CHANNEL

** IF TYPE( ) = CHAN(22) AND KIND( ) = 4 INPUT THE FOLLOWING

RN,ART,WPR, NL,ALT,WPL,ACX	7F8.0	MANNING'S N FOR RIGHT FLOOD PLAIN, CROSS-SECTION AREA RIGHT FLOOD PLAIN, WETTED PERIMETER RIGHT FLOOD PLAIN, MANNING'S N LEFT FLOOD PLAIN, CROSS-SECTION AREA LEFT FLOOD PLAIN, WETTED PERIMETER LEFT FLOOD PLAIN, EXTRA CROSS-SECTION AREA ABOVE CHANNEL WHEN FLOW IN FLOOD PLAIN.

NOTE: RIGHT AND LEFT LOOKING
      DOWNSTREAM

NOTE: FOR CROSS-SECTION, PICK
      ANY FLOOD LEVEL SUCH THAT
      FLOW EQUALS ABOUT THE
      50-YEAR FLOOD OR DEPTH
      IN FLOOD PLAIN AT LEAST
      SEVERAL FEET OR CROSS-
      SECTION AREA OF FLOOD
      PLAINS AT LEAST SEVERAL
      TIMES CROSS-SECTION AREA
      OF CHANNEL.

```
XX                                                   XX
 XX-------------------------------------------------XX
  XX         I                            I        XX
   XX  ALT   I             ACX            I  ART  XX
    XX       I                            I      XX
     XXXXXXXXX----------------------------XXXXXXXXX
        WPL   XX                        XX WPR
        FNL    XX     AC(APARM)        XX  FNR
                XX                    XX
                 XXXXXXXXXXXXXXXXXXXXXX
                      WPC(MPARM)
                      FNC(FF( ))
```

** IF TYPE( ) = STOR (33) INPUT THE FOLLOWING INFORMATION

U,VU, K( ),SX( )	2I4, 2F8.0	TIME UNITS FOR SK(SEC=1, MIN=60, HOURS=3600), VOLUME AND AREA UNITS (CUBIC FEET AND SQUARE FEET = 1, ACRE-FEET AND ACRES = 2),

```
                                              IF TU OR VU LEFT BLANK MINUTES
                                              OR ACRES ASSUMED,
                                              MUSKINGUM K AND X FOR CHANNEL
                                              ROUTING(AVAILABLE FOR USE BUT
                                              NOT RECOMMENDED)

**IF TYPE( ) = STOR(33) AND SK AND SX ARE BLANK INPUT THE FOLLOWING

D,KE                          2I4             KD=0, INPUT RESERVOIR VOLUMES
                                              KD=1, INPUT RESERVOIR SURFACE
                                                    AREAS AND ELEVATIONS
                                              KE=0, INPUT RESERVOIR OUTFLOWS
                                              KE=1, INPUT DISCRIPTION OF
                                                    OUTLET WORKS

** IF TYPE( ) = STOR(33) AND EITHER KD OR KE = 1 INPUT FOLLOWING

,SELEV( )                     I8/             NUMBER OF VALUES(MAX.20),
                              (10F8.2)        WATER SURFACE ELEVATIONS(FEET),

** IF TYPE( ) = STOR(33) AND KD = 0 INPUT THE FOLLOWING

,VOL( )                       I8/             NUMBER OF VALUES(MAX.20),
                              (10F8.2)        RESERVOIR VOLUMES (FIRST
                                              VALUE MUST BE ZERO)

** IF TYPE( ) = STOR(33) AND KD = 1 INPUT THE FOLLOWING

,AREA( )                      I8/             NUMBER OF VALUES(MAX.20),
                              (10F8.2)        RESERVOIR SURFACE AREAS

** IF TYPE( ) = STOR(33) AND KE = 0 INPUT THE FOLLOWING

,DIST( )                      I8/             NUMBER OF VALUES(MAX.20),
                              (10F8.2)        RESERVOIR OUTFLOWS

                                              NOTE: K,N AND L FROM ABOVE CARDS
                                                    MUST BE THE SAME

** IF TYPE( ) = STOR(33) AND KE = 1 INPUT THE FOLLOWING

A                               I4                NUMBER OF OUTLET STRUCTURES

** INPUT FOR 1 TO JA OUTLET STRUCTURES

T,JEL,JS1,JS2                 2I4,            TYPE OF STRUCTURE
                              2F8.3            JT=1 FOR CIRCULAR PIPE
                                               JT=2 FOR BOX CULVERT
                                               JT=3 ELLIPTICAL PIPE
                                               JT=4 RECTANGULAR BROAD-CRESTED WEIR
                                               JT=5 TRAPEZOIDAL BROAD-CRESTED WEIR
                                               JT=6 CIRCULAR DROP INLET
                                              ELEVATION OF BASE OF STRUCTURE,
                                              FOR JT = 1, 2 OR 3
                                               JS1 = VERTICAL DIMENSION
                                               JS2 = HORIZONTAL DIMENSION
                                              FOR  JT = 4
```

```
                                            JS1 = WIDTH OF WEIR
                                            JS2 = (LEAVE BLANK)
                                          FOR  JT = 5
                                            JS1 = WIDTH AT CREST
                                            JS2 = ANGLE OF ONE SIDE FROM VERTICAL
                                                  IN DEGREES
                                          FOR  JT = 6
                                            JS1 = DIAMETER OF PIPE
                                            JS2 = (LEAVE BLANK)

*  END ELEMENT LOOP WITH TYPE( ) = QUIT (99)

PT( )                          20I4       OUTPUT/INPUT OPTIONS
                                          NOTE: IF RUNOPT EQUALS 1 ONLY 9,16
                                                AND/OR 17 SHOULD BE SELECTED
                                                OR TONS OF OUTPUT WILL BE
                                                PRODUCED
                                            1 = PLOT HYDROGRAPHS ON PRINTER
                                            2 = PRINT DETAILED HYDROGRAPHS
                                            3 = PRINT MAX,FLOWS, ALL POINTS
                                            4 = PLOT HYDROGRAPHS ON CALCOMP
                                            5 = OUTPUT RAIN, RUNOFF VOLUMES
                                            6 = OUTPUT MAXIMUM STORAGE
                                            8 = CHANGE DLTKR AND STRKR
                                            9 = PRINT PEAKS, SELECTED POINTS
                                           16 = LIST ALL PEAKS FROM RUNOFF FILE
                                           17 = PRINT STAGE-DISCHARGE TABLES
                                           18 = COMPARE DISCH AT TWO POINTS
                                           19 = OUTPUT DISCHARGE AT LAST PT
                                           20 = INPUT FROM PREVIOUS SIM.

UPT( )                         20I4       ELEMENT NUMBERS OUTPUT DESIRED
                                          (UP TO 20 MAY BE SPECIFIED,IF
                                          MORE DESIRED PUNCH 9999 IN
                                          FIRST THREE COLUMNS AND GET ALL).

** IF OUTPUT/INPUT OPTION 4 SELECTED, INPUT THE FOLLOWING CARD

ACT,LDEV,                      F4.0,I4,   SCALE FACTOR(USUALLY 1.0 FOR A
IMX,ISPEC,                     F4.0,I4    10 INCH PLOT),LOGICAL UNIT NUMBER
FACT                                      (ANY NUMBER 1 TO 100),MAXIMUM
                                          PLOT TIME IN MINUTES, SPECIAL
                                          PEN OR PAPER OPTIONS(LEAVE BLANK
                                          OR REFER TO CALCOMP PLOTTER
                                          MANUAL),SCALE FACTOR ON TIME AXIS
                                          (IF BLANK,1.0 ASSUMED).

** IF OUTPUT/INPUT OPTION 8 SELECTED, INPUT THE FOLLOWING CARD

LTKR,SL                        4F8.0      REPLACEMENT VALUES FOR PERVIOUS
PARM,MPARM                                AREA HEC-1 PARAMETERS DLTKR AND
                                          SL IF NON-ZERO, REPLACEMENT
```

```
                                                VALUES FOR APARM AND MPARM IF
                                                NON-ZERO

* INPUT PRECIPITATION DATA IF RUNOPT = 0

MTS,INC,ISTRT                 3I4               FORMAT CODE NUMBER(1 OR 2),(SET
                                                IFMTS=3 FOR DEFAULT 10-YEAR STORM
                                                OR IFMTS=4 FOR DEFAULT 100-YEAR
                                                STORM), TIME STEP(SAME UNITS AS
                                                DELT AND MUST DIVIDE EVENLY INTO
                                                DELT),
                                                STARTING TIME INCREMENT(2-12) ON
                                                FIRST PRECIP CARD IF OTHER THAN
                                                FIRST VALUE OTHERWISE LEAVE BLANK

** IF IFMTS = 1 INPUT FOLLOWING CARD

STA,YR,MO,                    I6,5I2,           STATION NUMBER,YEAR,MONTH,DAY,
Y,HR,MIN,INC,                 F4.1,             HOUR,MINUTE,TIME INCREMENT(MIN),
RA( )                         12F5.2            12 PRECIPITATION INCREMENTS.
                                                NOTE: LAST CARD MUST HAVE 99 IN
                                                        COLUMNS 9+10 AND NO DATA.

** IF IFMTS = 2 INPUT FOLLOWING CARD

CSTA,TI,TY,YR,                A6,2I2,           STATION IDENTIFICATION, TIME
O,DAY,YR,ARRA( )              2X,4I2,           INCREMENT,CODE TYPE(1=P,2=SF),
                              12F5.2            YEAR,MONTH,DAY,HOUR,12 RAINFALL
                                                VALUES.

** IF IMPTS = 3, 10-YEAR STORM ASSUMMED

** IF IMPTS = 4, 100-YEAR STORM ASSUMMED

** INPUT STREAMFLOW DATA IF RUNOPT = 0

M,IFMTS,INC,                  20I4              NUMBER OF FLOWPOINTS(ELEMENTS)
TRT,NSD( )                                      WITH MEASURED DISCHARGE(MAX.=5),
                                                FORMAT CODE NUMBER, TIME STEP
                                                FOR DATA(SAME UNITS AS DELT AND
                                                MUST DIVIDE EVENLY INTO DELT),
                                                STARTING TIME INCREMENT(2-12) ON
                                                FIRST STREAMFLOW CARD IF OTHER
                                                THAN FIRST VALUE OTHERWISE LEAVE
                                                BLANK,
                                                FLOWPOINT(ELEMENT) NUMBERS.
                                                NOTE - IF NO MEASURED FLOW THIS
                                                CARD SHOULD BE BLANK

** IF NUM GT 0 AND IFMTS = 1, INPUT MEASURED DISCHARGE

STA,SF( , )                                     STATION NUMBER, DISCHARGE DATA
                                                FOR EACH STATION FOR TOTAL TIME
                                                AND TIME INCREMENT SPECIFIED IN
                                                INPUT CARD NUMBER 2.

** IF NUM GT 0 AND IFMTS = 2, INPUT MEASURED DISCHARGE
```

NCSTA,TI,TY,YR, MO,DAY,HR,ARRA( )	A6,2I2, 2X,4I2, 12F5.2	STATION IDENTIFICATION, TIME INCREMENT,CODE TYPE(1=P,2=SF), YEAR,MONTH,DAY,HOUR, 12 STREAMFLOW VALUES.

*** IF OUTPUT/INPUT OPTION 20 SELECTED, INPUT FOLLOWING CARDS.

,NU,NE( )	20I4	I/O FILE NUMBER, NUMBER OF INFLOW ARRAYS, NUMBERS OF THE FLOWPOINTS (ELEMENTS) FOR COMPARISON
SF(,)	20X 12F5.0	NU ARRAYS OF INFLOW DATA FOR TIME DETERMINED FROM CARD 2.

*** IF OUTPUT/INPUT OPTION 18 SELECTED, INPUT FOLLOWING CARDS.

NCMP, (N1( ),N2())	20I4	NUMBER OF PAIRS OF ELEMENTS TO BE PLOTTED ON SAME PLOT, ELEMENT NUMBERS. FIRST ELEMENT PLOTTED WITH *, SECOND WITH O.

**** IF OPTIONS 20 AND 4 SELECTED, INPUT THE FOLLOWING CARD ****

INFCA( ), INFCB( )	6A6,4X, 6A6	INFORMATION FOR CALCOMP ON CURREN SIMULATION, INFORMATION ON PREVIOUS SIMULATION

*** CONTROL RETURNS TO INPUT CARD NUMBER 1. TO END RUN PUNCH END

PROGRESS REPORT

For the Period October 1 to December 1, 1974

Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County

Project Number: E20-646

During the last two months of the contract period it became apparent that the project would not be complete by December 1 and all the funds for the project would not be spent. Arrangements were made for an extension of the project though final approval would be after December 1.

Gage installation was ninety percent complete by December 1 and would be completed by the first of the year when DeKalb County would take responsibility for the operation of the gages.

Only seventy-five percent of the computer runs had been completed by December 1 and some needed to be repeated because of input data errors. Drafts of portions of the final reports had been completed.

During November, several afternoon seminars were held at DeKalb County to explain the hydrologic simulation model and the stream gage operation procedures. The computer program and data files were placed on the DeKalb County computer and modified to fit the computer size and software.

The project is to be extended through March 31st to complete the simulation runs for the eight study watersheds, write reports, and complete the stream gage installation.

E-20-646

UROS04: URBAN FLOOD SIMULATION MODEL

Part 1. Documentation and Users Manual

for
Board of Commissioners
DeKalb County, Georgia

by
Alan M. Lumb, Project Director
School of Civil Engineering

March, 1975

GEORGIA INSTITUTE OF TECHNOLOGY
Atlanta, Georgia

ACKNOWLEDGMENTS

This report is one of a series of three reports presenting the work done by Georgia Institute of Technology for DeKalb County, Georgia on a project entitled "Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County." Work on the project began December, 1973, and was completed March, 1975. Alan M. Lumb, with the assistance of J. R. Wallace and L. D. James, directed the project. Dr. James assisted with the development and application of UROS4, while Dr. Wallace directed most of the work in gage site selection and installation. Paul Sanders and W. M. Sangster provided invaluable administrative assistance.

Eleven students worked on the project, and three made major contributions. John Clerici worked on the project for the entire period and took major responsibility in the collection of the field data. Jack Kittle worked on the programming of UROS4, the creation of the runoff files and program applications. D. Rao assisted with the statistical analysis and was very productive in program applications. Two graduate students supported by the Corps of Engineers, Carvel Deese and Roy Powell, worked on the project as part of their studies and their efforts are gratefully appreciated. Although only working with the project one quarter, Tom Debo developed the procedures for and performed most of the work on the determination of the fraction of impervious area. The services of six other research assistants, Ed Sing, Paul Nowak, Adnan Saad, Tim Hassett, Jaime Nino-Pinto and Steve Todd, aregratefully acknowledged.

Special gratitude is due to Ms. Brenda Manley for her efforts in typing drafts and the final manuscript and Dr. James for his extensive review and revision of drafts of this report.

ABSTRACT

Urban development has occurred so rapidly in the Atlanta Metropolitan Area that the citizens and their governments have not been able to deal adequately with the associated flood and drainage problems. As the idealistic approach of locating everyone and everything on higher ground is costly if not impossible, the welfare of DeKalb County will best be served by a combination of 1) tributary area land use planning, 2) flood plain management including land use planning and regulation of flood plain building practices, and 3) structural measures involving detention storage and drainage system improvements. Selection of a successful combination requires information on how land surfaces and drainageways respond to a variety of precipitation patterns. Since watershed configurations and precipitation patterns are so complex and varied, hydrologic simulation is the only method powerful enough to determine fully the effects of land use and channel changes on flood elevations.

In order to provide a working simulation model for use by DeKalb County, the Urban Flood Simulation Model was developed. Rainfall, streamflow, and soils data in DeKalb and similar adjacent areas were analyzed with a watershed model to develop an historic data file of rainfall excess for the range of land surface conditions found in DeKalb County. The Urban Flood Simulation Model simulates floods given the data file and prescribed physical characteristics of as many as 100 area, channel, and storage segments in a selected drainage area (Snapfinger Creek for example). The Model will calculate flood elevations and associated probabilities for all critical points specified in the input data. Though collecting, coding and checking the data on the physical characteristics may take a man-month or more depending on the size and resolution, once the coding is complete it is relatively easy to explore the effects of changing land-use, altering the drainage system, or adding detention storage. The procedures used in developing the file of runoff data, the computational framework, the computer programming, and the recommended procedures for collecting and coding data on drainage characteristics are all described in detail. Several case study applications illustrate how the Model can be used in hydrologic studies.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

LIST OF FIGURES

LIST OF FIGURES (Cont'd)

LIST OF TABLES

LIST OF TABLES (Cont'd)

SECTION I

Urban Drainage Problems

Mass construction of houses, apartments, commercial and industrial units, streets, and parking lots has decreased the infiltration of rainwater into the soil and, hence, increased storm runoff and complaints from flooded property owners. The problem is compounded by 1) constrictions in natural stream channels caused by debris, land fills, and roads with culverts sized to handle runoff with rural conditions; 2) reduction in storm water storage on flood plains due to land fills and buildings; 3) increased velocity of flow on flood plains from removal of vegetation, 4) channels clogged with soil eroded from construction sites, and 5) paved gutters and channels which quickly deliver water from roofs, streets, and parking areas to nearby channels.

In an effort to reduce complaints, culverts are enlarged, channels cleaned, and detention storage areas constructed. Though these measures almost always reduce the immediate problem, they can and have compounded problems by increasing flows downstream. Even detention dams on lower tributaries can, in some cases, increase flood levels in the main channel by delaying the tributary flow so that it peaks closer to the crest of the flood wave in the main channel.

The major storm drainage problem that exists in DeKalb County has thus been created by a combination of 1) manmade changes to land surfaces and drainageways that have increased and accelerated runoff 2) capital investment in land development in low-lying areas, and 3) remedial measures that have not always been successful because they were designed from an incomplete or erroneous understanding of the total hydrologic system. As the idealistic approach of moving everyone and everything to higher ground is costly if not impossible, the economic and environmental welfare of the county will best be served by a combination of 1) land use planning for flood plain and tributary areas, 2) control of building practices, and 3) detention storage and drainage system improvements. Selection of a successful combination requires information on how land surfaces and drainageways will interact in responding to a variety of precipitation patterns. Drainage system configurations are so complex and the precipitation patterns are so varied that hydrologic simulation is the only method that can be used to determine fully the effects of various actions on flood flows and elevations.

The hydrologic information required to deal with the current storm drainage problem in DeKalb County and to evaluate proposed plans of action are:

1) flood flows and stages and associated probabilities for all streams in DeKalb County for current land-use and channel conditions;
2) expected changes in flood flows and stages and associated probabilities for projected land-use patterns, for the addition or removal of detention storage facilities, or for alteration of channels and floodways; and
3) effects of channel constrictions which are presently altering both upstream and downstream flooding.

These facts can only be obtained through hydrologic simulation of the runoff, channel storage and routing processes on a digital computer. Once the computer is programmed, only the existing or proposed physical characteristics of the drainage area, channels, and storages are needed for generating the required hydrologic information.

SECTION II

Development of the Urban Flood Simulation Model (UROS4)

Introduction

Hydrologic simulation is the programming and operation of a digital computer model to describe the behavior of a drainage system over extended periods of real time. The value of a simulation model depends on the degree to which it matches the response of the actual system. Thus, establishment of a reliable model requires calibration and verification with measured data in the location it is to be applied. A calibrated model can be used to predict the consequences of actions not yet taken or of events which have not yet occurred by running the model with data representing those actions or events.

Hydrologic simulation uses mathematical expressions to represent the physical processes through which water moves through a watershed. The primary processes include precipitation, infiltration of water into the soil, overland flow of the water that does not infiltrate, passage of water through the soil to the channel, and movement and storage of stormwater in channels and associated floodways as well as in lakes and other ponded areas. The goal of the hydrologic simulation model, then, is to combine the mathematical expressions so as to represent the processes in a way that can be used to provide the information needs listed in the previous section. More specifically, it is desired that the model:

1) generate stormwater runoff from rainfall for many storms for many years so as to cover the range of conditions needed to determine the probabilities associated with various flood flows,

2) route the stormwater runoff over the land-surface, through channels, floodways and lakes,

3) statistically analyze the annual series of peak floods to assign probabilities to floods flows, and

4) transform flood flows at specified probabilities to flood elevations.

Figure 1 is a flow chart of the steps used in this study to build and apply a computer model to fulfill these objectives. The overall strategy involved 1) developing a runoff file of runoff volumes for various land surfaces at several time increments for a sequence of historical storms and 2) programming a flow routing model given the acronym UROS4. The strategy in developing the runoff file was to calibrate a runoff simulation model patterned after the Stanford Watershed Model on several gaged watersheds in the general area of DeKalb County, use the calibrated model to simulate 72 years of hourly streamflows from the Atlanta rainfall record, select a representative portion of this total period so that subsequent simulations would be less expensive, and use the model and the selected years of precipitation to simulate runoff. A 25-year period was selected. The simulation of runoff was repeated four times, once for each of the three broad soil permeability groups found in DeKalb County and once for impervious surfaces. Each simulation produced sums of direct runoff, interflow, and baseflow for three to six major storm periods per year at one, five and fifteen-minute time increments. The results were stored in "runoff files" so that the runoff process would not have to be resimulated in model applications.

Hydrologic analysis of a specific watershed with UROS4 thus starts with the appropriate runoff file as input to the routing model. Three types of routing are used: subarea routing, channel routing, and storage routing.

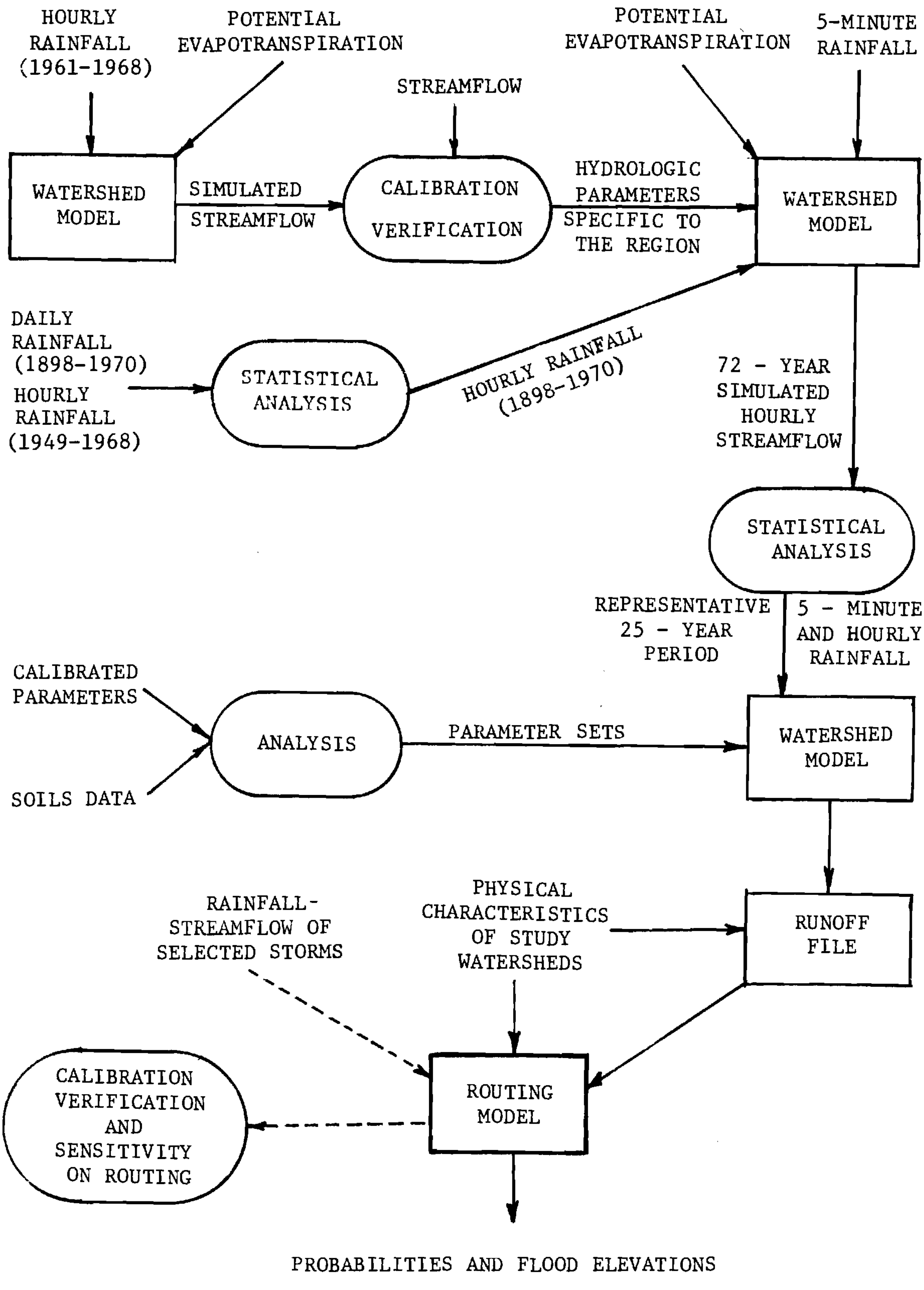


Figure 1. Flowchart for Model Development

Subarea routing accounts for the time lags and surface storage effects as the runoff originating over the subarea moves to its low point. Channel routing uses kinematic wave theory to represent the effect on the flood hydrograph of traveling a length of channel. Storage routing represents the effect of ponding behind culverts and in other detention areas. The required inputs to the routing model for a particular watershed are the distribution of subarea surfaces among the four soil permeability types and the information needed to physically describe the land areas, channels, and storage locations for the routing.

This section will first define subarea, channel, and storage segments, their relevant quantitative characteristics, and the mathematics involved in generating runoff hydrographs. Second, the procedure of using a watershed model and input data from a long meteorological record to develop a file of storm runoff data is discussed. Last, frequency analysis of the simulated peak flows and conversions of flows to associated flood elevations is discussed.

Watershed Segments

Simulation of runoff from a watershed of much more than a few acres is best accomplished by dividing the total drainage area into a number of discrete and relatively homogeneous subareas in order to account for 1) drainage area shape and the configuration of the stream channel system, 2) spatial variation in intensity of land development, and 3) time lags and storage effects present in channels, lakes and other ponding areas. Representation of the physical characteristics of all small homogeneous areas, the geometry of every possible flow path, and the storage in every depression or behind every constriction would make a model needlessly costly. Thus, it is necessary to lump areas and group flow paths into larger segments to make an urban runoff model manageable. Segment size, then, must balance the cost of analysis with the desired spatial detail of information and reliability of the simulated flows. Segments too large would produce more approximate results while segments too small would make the cost of analysis excessive.

The geometric description of the land surface is accomplished by dividing the total drainage area into smaller relatively homogenous units called subareas. UROS4 provides options for division at either a micro or a macro level. For micro division into areas of no more than a few acres, the subareas are conceived as rectangles whose runoff empties perpendicularly into an open channel at the downstream edge of the flow plane. Areas greater than several acres are not sufficiently homogeneous to treat as rectangles with uniform slope and cover; and thus when larger subareas are used, they are treated at a macro level instead of the micro level. The drainage network is described by subdivision into channel segments and storage areas. Channels may be rectangular, triangular, circular or of an irregular natural shape. Storage areas are defined by a storage-discharge relationship.

In arranging the segments for UROS4, any type of segment may discharge into any other type segment, although discharge of channels or storages to areas are rare. One possibility on the micro-scale would be downspouts from roofs draining onto lawns. In such cases the discharge onto a source area is assumed to be spread uniformly across that area and is added to any incoming precipitation.

Hydrologic Processes Modeled in UROS4

Infiltration. Two options are available in UROS4 for determining runoff rates. When the runoff files are used, infiltration of storm water into the soil has already been deducted and the runoff has been estimated with the watershed model. However, an alternative has been provided in UROS4 so that rainfalls may be used for storm events not covered by the runoff file. In such cases, losses from infiltration into the soil and filling depressions on the land surface must be calculated. Other losses such as interception by vegetation and evapotranspiration, are relatively small during the short and very intense rainfalls that generate runoff peaks from small urban watersheds and are not estimated by the loss function described below. They are, however, included in the generation of the runoff files by the watershed model.

The loss function used in the Corps of Engineers' HEC-1 Flood Hydrograph Package was selected for use, as an alternate to the runoff file because of its flexibility.

The loss rate function used by HEC-1 is

$$ALOSS = (AK + DLTK)\ P^{ERAIN} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where DLTK and AK are defined by the following two equations:

$$DLTK = 0.2\ DLTKR\ (1-CUML/DLTKR)^2 \quad \text{when } CUML < DLTKR \quad \ldots\ldots\ldots \quad (2)$$

$$DLTK = 0.0 \qquad \text{when } CUML \geq DLTKR$$

$$AK = STRKR/RTIOL^{(0.1\ CUML)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

The various terms are represented on Figure 2 and defined as follows:

P = rainfall intensity (inches per hour) during the time interval

ALOSS = loss rate for particular time interval (inches per hour)

AK = loss rate coefficient at beginning of time interval

DLTK = incremental increase in loss rate coefficient during the time interval

CUML = accumulated loss (inches) up to current time interval

ERAIN = exponent of precipitation for the loss rate function

DLTKR = additional rain loss at the beginning of the storm

STRKR = value of the loss coefficient at the beginning of the storm

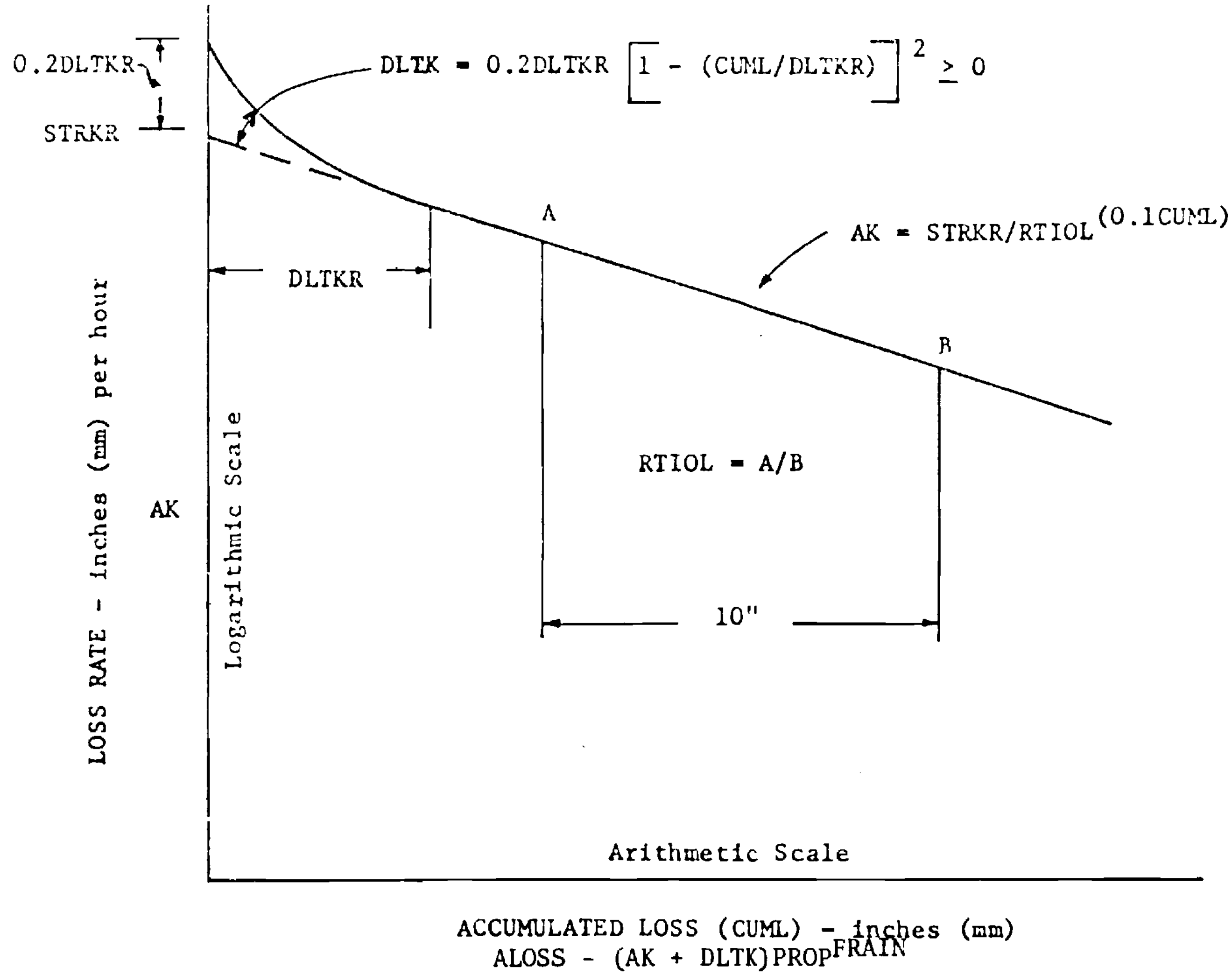


Figure 2. Loss Rate Function Used by HEC Model

RTIOL = ratio of rain loss coefficient to that corresponding to 10 inches more of accumulated loss.

Of the nine terms listed above, P comes from input precipitation data; AK, ALOSS, and DLTK are calculated from the above expressions; CUML is the accumulation of calculated values of ALOSS up to the time interval at hand; and ERAIN, DLTKR, STRKR and RTIOL are the parameters that need to be calibrated for a given storm on a given watershed, where ERAIN and RTIOL are relatively less and DLTKR and STRKR are relatively more sensitive to antecedent moisture conditions. During the summer following several weeks with little rain, DLTKR and STRKR would be fairly high, whereas during the winter following several weeks of low intensity rainfall, DLTKR and STRKR would be quite low.

The HEC-1 program contains a routine for selecting optimum values for the four parameters (ERAIN, DLTKR, STRKR, and RTIOL) appearing in the loss rate function from rainfall and streamflow data from a measured storm. In a study by O.C. White (1973) at Georgia Institute of Technology, ten storms on Camp Creek, Clayton County, were used to find values for ERAIN, DLTKR, STRKR, and RTIOL. A detailed description of the Camp Creek drainage area is found in a subsequent section of this report. Characteristics of the ten storms are listed in Table 1.

The four parameters were optimized in a manner recommended in HEC-1 (1970) in the following order: 1) ERAIN, 2) RTIOL, 3) STRKR and 4) DLTKR. After the parameters were optimized for each storm, an average value was computed for ERAIN; and that parameter was then fixed at that value for subsequent optimizations. From the next set of optimized values, RTIOL was averaged, and the process continued through DLTKR. Values obtained for each of the four parameters from the first optimization are enumerated in Table 2. The average values are from the optimization

Table 1. Characteristics of Storms Used To Calibrate HEC-1 for Camp Creek

Flood Date[a]	Duration of Rainfall (hours)	Maximum Hourly Rate of Rainfall (inches/hr.)	Total Rainfall (inches)	Total Excess (inches)	Peak Recorded Discharge Rate (c.f.s.)
Feb. 25, 1961	18	1.23	5.67	4.25	4,000
Jan. 15, 1966	38	.28	2.12	.94	684
Feb. 12, 1966	26	.61	3.95	2.43	1,390
March 4, 1966	28	.28	2.33	1.63	850
March 10, 1967	12	.98	2.02	.84	761
Aug. 24, 1967	38	.56	4.16	1.46	868
March 12, 1968	24	.56	2.72	1.52	1,100
April 5, 1968	20	.37	2.68	.91	746
April 18, 1969	14	.74	2.62	1.27	1,020
May 9, 1969	12	2.30	4.34	.73	768

[a]Floods are identified by the date on which the peak streamflow rate occurred.

Table 2. Results of Loss Rate Parameter Optimization

Flood Date	ERAIN	RTIOL	STRKR	DLTKR
2/25/61 [a]	.49	3.13	.03	1.86
1/15/66	.49	2.35	.15	.32
2/12/66	.54	1.34	.18	1.13
3/4/66	.46	10.12	.05	-
3/10/67	.29	3.72	.21	1.76
8/24/67	.57	3.66	.24	2.92
3/12/68	.54	1.51	.19	1.03
4/5/68	.46	5.06	.15	2.10
4/18/69	.58	4.24	.12	1.45
5/9/69	1.06	3.38	.66	5.34
Average [b]	.49	3.13	.18	1.53

[a] Data not initially available and not included in the average. Values for ERAIN and RTIOL were not optimized while the remaining parameters were optimized simultaneously.

[b] Average is for only eight storms. The 3/4/66 and 5/9/69 storms were dropped from the analysis because of unrealistic values.

just described. The storms on March 4, 1966, and May 9, 1969, were not included in the average because unreasonable values were estimated for some parameters; a situation that often occurs when a gaged record poorly represents the true precipitation patterns.

The three principal reasons for variation from storm to storm in parameter values are 1) differences between the true time and spatial pattern of rainfall over the basin and rainfall measured at the gage, 2) differences in antecedent moisture conditions at the beginning of the storm, and 3) errors in streamflow measurements. An average parameter for the HEC-1 model, thus, represents an average precipitation pattern with average antecedent moisture and is not directly applicable for flood frequency studies. The range of values from storm to storm illustrates the uncertainty involved in selecting parameter values for use when trying to use a ten-year rainfall to predict a ten-year flood flow.

Routing Flow from Subarea Segments. UROS4 overland flow simulation uses storm runoff volumes generated by the HEC-1 infiltration model or from the runoff file. The simulation at the micro scale is discussed first, is taken from Crawford and Linsley (1966), and is based on the continuity equation

$$D_t = D_{t-\Delta t} + (\bar{P} - \bar{I} - \bar{Q})\ \Delta t \quad \text{.................} (4)$$

where: D = surface detention storage

t = time

Δt = time increment

$\bar{P}$ = average rainfall less interception during Δt

$\bar{I}$ = average infiltration during Δt

$\bar{Q}$ = average outflow during Δt

Average outflow equals $(Q_t + Q_{t-\Delta t})/2$ where Q at any time t is a function of outflow depth which in turn is a function of current and equilibrium detention storage for a given rainfall intensity. An empirical equation is used to relate the two detention storages to outflow depth, and Manning's equation is used to determine both detention storage at equilibrium and the relation between outflow depth and outflow. The resulting equations are

$$Q_t = \frac{1.486}{n} S^{1/2} \left(\frac{D_t}{L}\right)^{5/3} \left(1.0 + 0.6 \left(\frac{D_t}{D_e}\right)^3\right)^{5/3} \quad \ldots\ldots\ldots\ldots \quad (5)$$

$$D_e = \frac{0.000818}{S^{0.3}} \; i^{0.6} n^{0.6} L^{1.6} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

where n = Manning's n

L = length of flow surface in feet

S = slope of flow surface

i = intensity of rainfall in inches per hour

D_e = detention storage at equilibrium in average depth in inches over the area

An implicit solution for Q_t and D_t with the Newton-Raphson scheme is used. Instabilities were found for very steep slopes and short flow lengths such as rooftops. In these cases outflow for each time increment is arbitrarily calculated as 75% of the sum of the existing water depth, $D_{t-\Delta t}$, plus the rainfall excess for the current time step.

Published data from the Johns Hopkins Storm Drainage Research Project (Schaake, 1965) and from Izzard (1946) were used to test the overland flow model for completely impervious watersheds. The data from the Johns Hopkins' project was for a catchment comprising a 0.39-acre paved surface which was subdivided into six source areas and three channels. Simulation of this area is greatly simplified by its small size and lack of pervious areas. One storm event was modeled, and the simulated hydrograph was found to correspond very closely with the observed runoff (Figure 3). In addition, two events from Izzard's work were simulated with the same good reconstitution of the hydrographs as shown in Crawford and Linsley (1966) and Figure 4.

The Johns Hopkins' parking lot was also modeled as a single source area rather than a collection of six areas and three channels. The simulated hydrograph still corresponded quite well with observed data (Figure 5). In all cases only measured physical characteristics and selected roughness coefficients were needed. No parameter calibration was required.

Overland flow simulation at the macro scale is needed for areas over several acres. For these areas, land surface and flow pattern characteristics are too complex to be adequately represented by a uniform flow plane, and a different approach is needed to represent the combined effects of overland flow and collector channels.

In a Purdue study (Sarma, 1969) and other studies (Willeke, 1966) of urban watersheds, it was found that a single linear reservoir model is often adequate to represent the time distribution of runoff for urban watersheds less than a few square miles in area. The storage-discharge relationship for a linear reservoir is given by

$$S = KQ \qquad (7)$$

where S = water stored in the reservoir (on the watershed)

Q = outflow from the reservoir (watershed)

K = storage coefficient

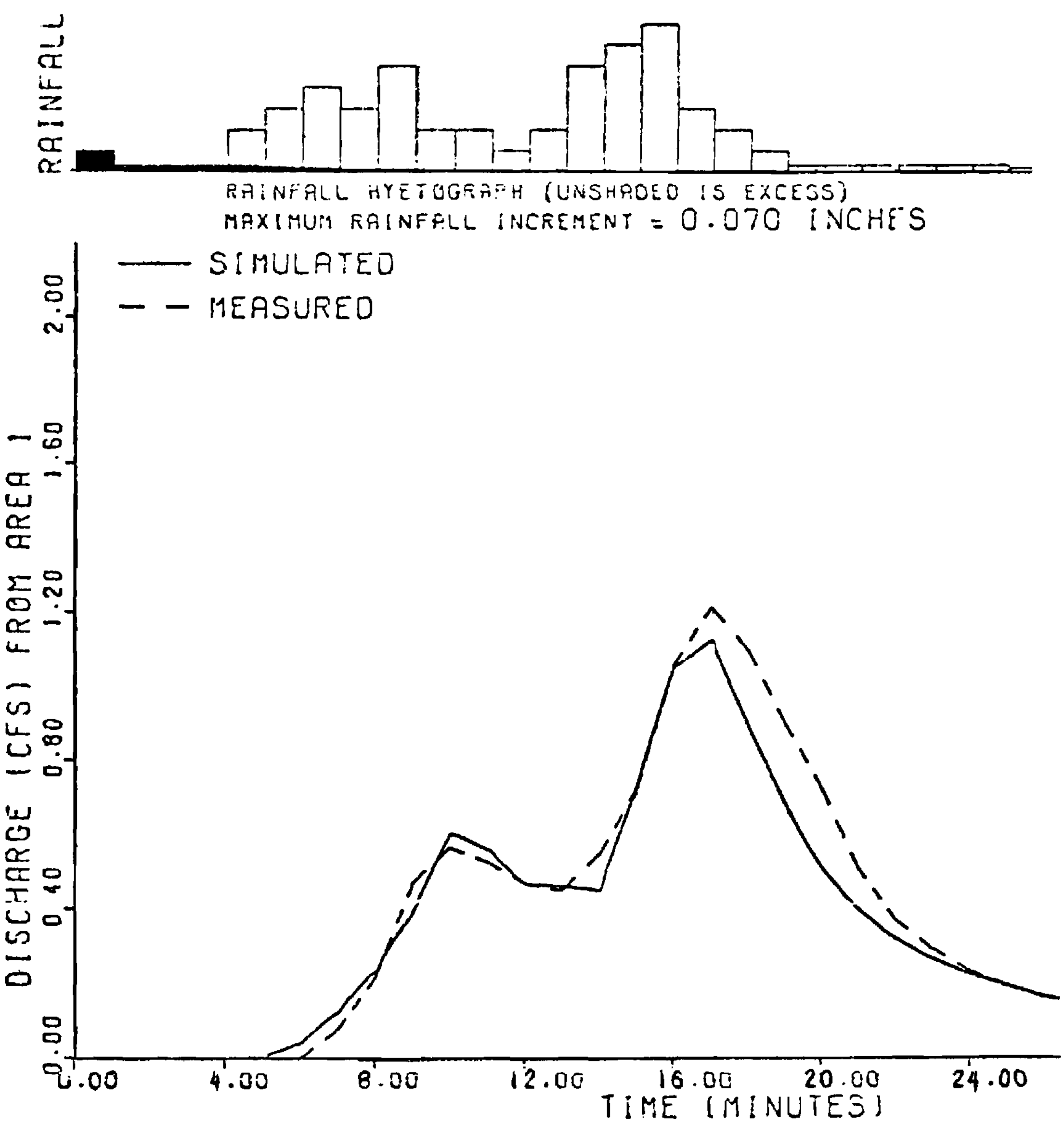


Figure 3. Comparison of Simulated with Observed Hydrograph Based on Johns Hopkins Data on South Parking Lot No. 1, With 6 Area Segments

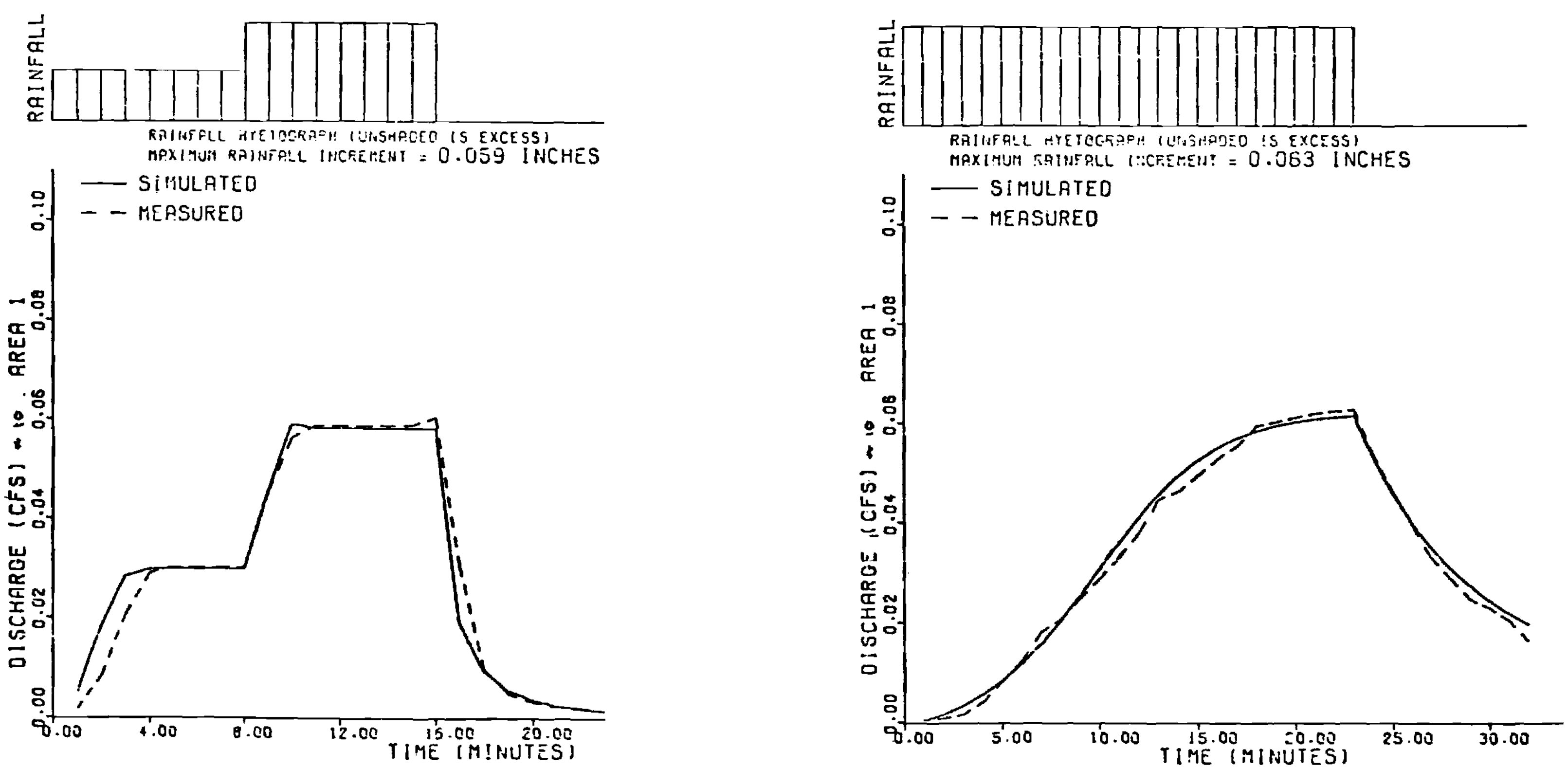


Figure 4. Comparisons of Simulated with Observed Hydrographs Based on Izzard's Data

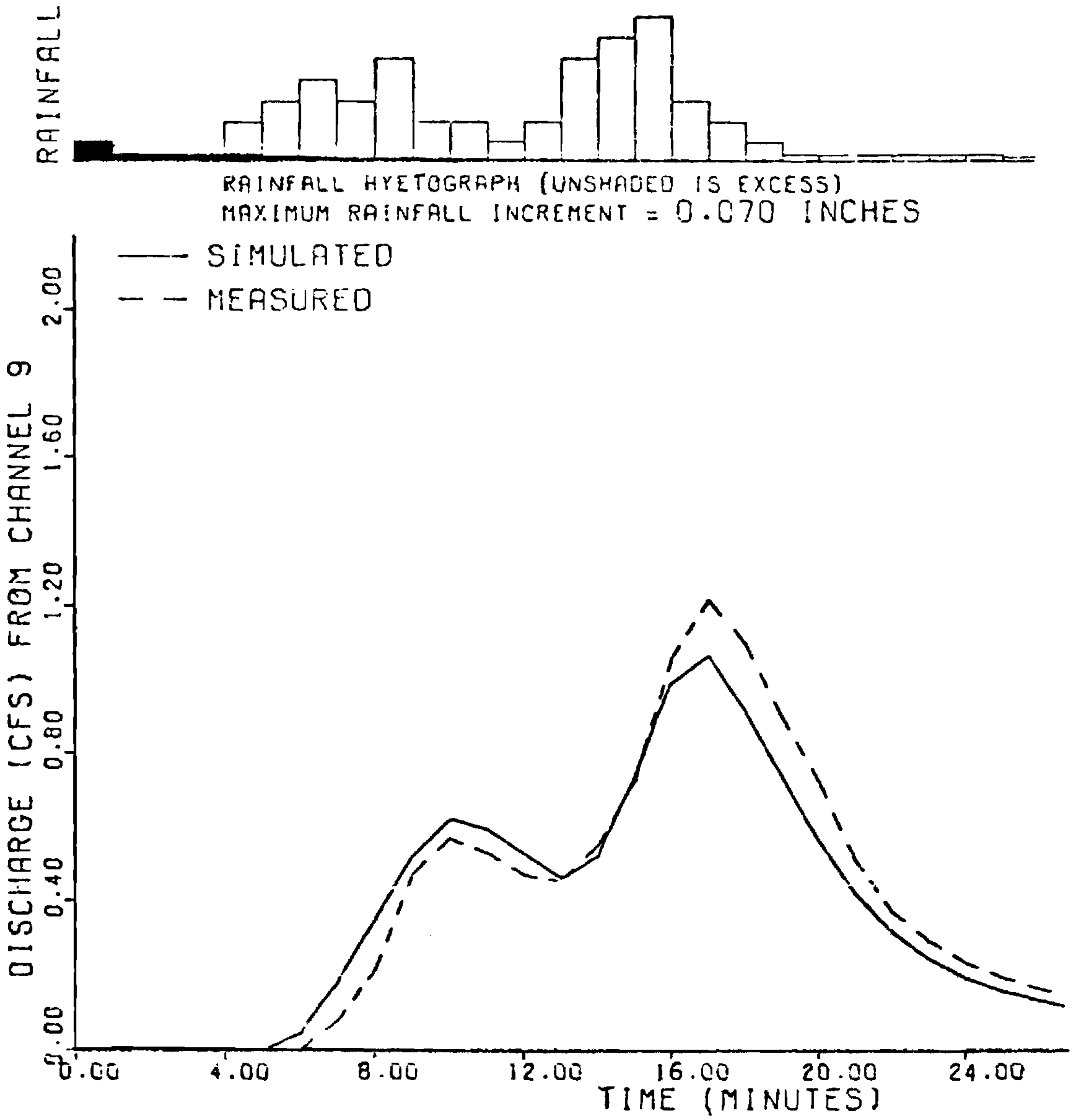


Figure 5. Simulation of Runoff from Johns Hopkins Parking Lot Using One Area Segment

The value of the storage coefficient depends on the physiographic characteristics of the watershed including the percent impervious area and the characteristics of the storm which causes the runoff. The relationship found in the Purdue study between these characteristics and the storage coefficient is

$$K = 0.887A^{0.49}\ (1 + U)^{-1.683} P_E^{\ -0.24} T_R^{\ 0.294} \ \ldots\ldots\ldots\ldots \ (8)$$

where A is the drainage area (square miles), U is the decimal fraction expressing the ratio of impervious area to total watershed area, P_E is the precipitation excess (inches), and T_R is the duration of rain (hours). The exponents for the terms P_E and T_R are small relative to the other two terms in Equation 8. Thus, the effects of the duration and amount of rainfall are small and can be dropped from the equation with little loss in accuracy. Also, the exponent on the term for drainage area was so close to 0.5 that it was changed to that value. With these modifications, the coefficient for the equation, 0.887, would no longer be valid. Equation 8 with modifications becomes

$$K = PK*A^{0.5}\ (1 + U)^{-1.68} \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots \ (9)$$

where,

K = storage coefficient for routing (hours)

PK = coefficient calibrated for the Atlanta region

A = drainage area (square miles)

U = decimal fraction expressing the ratio of impervious area to total watershed area

To determine the appropriate value for PK for the Atlanta region, storms on five watersheds ranging in area from 11 acres to 17 square miles were analyzed. The drainage areas are the Clairmont Watershed near Black Fox Drive (11 acres), South River at East Point (1.49 square miles), a tributary to South Utoy Creek (0.75 square miles), North Fork Camp Creek (5.2 square miles), and Camp Creek near Fayetteville (17 square miles). Table 3 lists physical characteristics of the five watersheds. Estimates using data from the Purdue study (Rao, 1972) indicated a value of 1.0 for PK would be a good initial estimate. This value was used and several storms on each watershed were simulated. The resulting storm hydrographs for the Clairmont Watershed are shown in Figures 6a through 6g. Table 4 lists the infiltration parameters and storm characteristics that were used for the other four watersheds. Other values of PK, 0.5 to 2.2, were also tried. The results were not found to be very sensitive to the value of PK (Table 5), and a value of 1.0 was selected as most appropriate. The results also showed the method was only applicable to areas under two square miles.

Equation 7 can be combined with the continuity equation written as Equation 36 to obtain a routing equation

$$Q_2 = C_0 I_2 + C_1 I_1 + C_2 Q_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

where I_i = inflow (usually precipitation) to the watershed at time i

Q_i = outflow from the watershed at time i

Δt = interval of time used in routing

$$C_0 = (0.5\ \Delta t)/(K + 0.5\Delta t) \quad \ldots\ldots\ldots\ldots \quad (11)$$

$$C_1 = (0.5\ \Delta t)/(K + 0.5\Delta t) \quad \ldots\ldots\ldots\ldots \quad (12)$$

$$C_2 = (K - 0.5\Delta t)/(K + 0.5\Delta t) \quad \ldots\ldots\ldots\ldots \quad (13)$$

Since $C_0 = C_1$, Equation 10 can be written as

Table 3. Watershed Characteristics

Watershed	Drainage Area	Impervious Area*	Number of Subareas	Average Size Subarea (acres)	Average Reach Length (feet)
Clairmont	10.8 acres	0.25 **	-	-	-
South Utoy Creek Tributary	0.75 sq. mi.	0.12	6	80	1467
South River at East Point	1.50 sq. mi.	0.228	6	159	1000
North Fork Camp Creek	5.20 sq. mi.	0.104	20	168	1618
Camp Creek at Fayetteville	17.0 sq. mi.	0.027	38	286	2088

* Determined from computer simulation for the larger 4 watersheds by optimization of impervious area perameter. Measured for Clairmont Watershed.

** 0.33 if impervious area that drains onto land surfaces rather than into the creek is included.

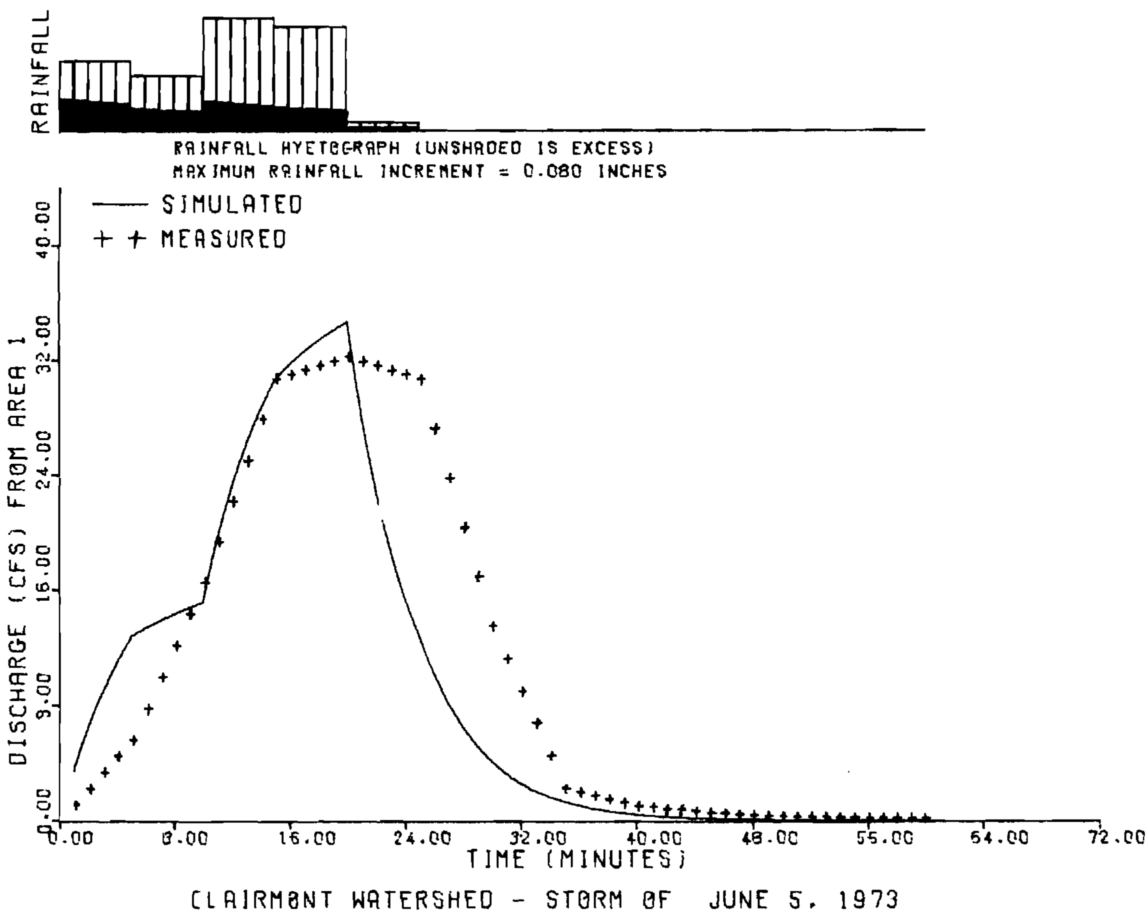


Figure 6a. Comparison of Simulated and Observed

Hydrographs, Linear Storage Model

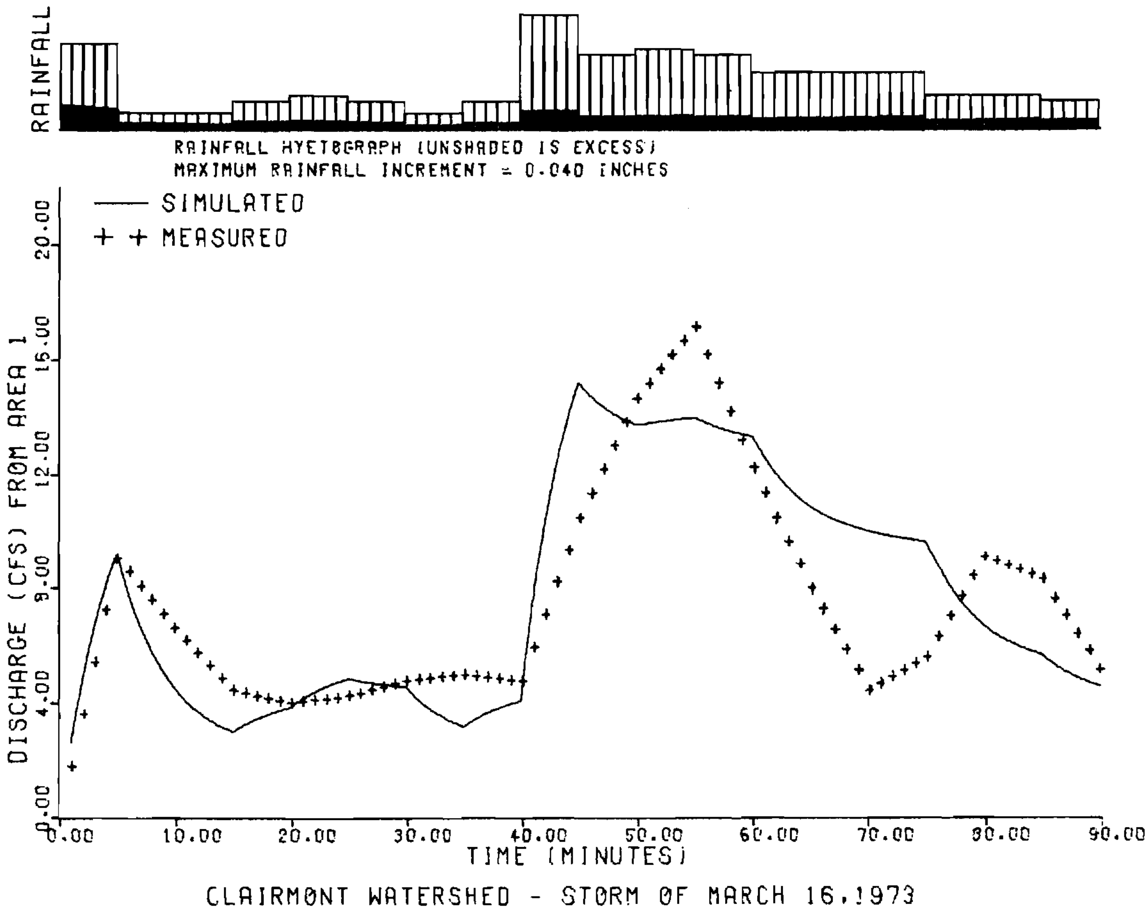


Figure 6b. Comparison of Simulated and Observed

Hydrographs, Linear Storage Model

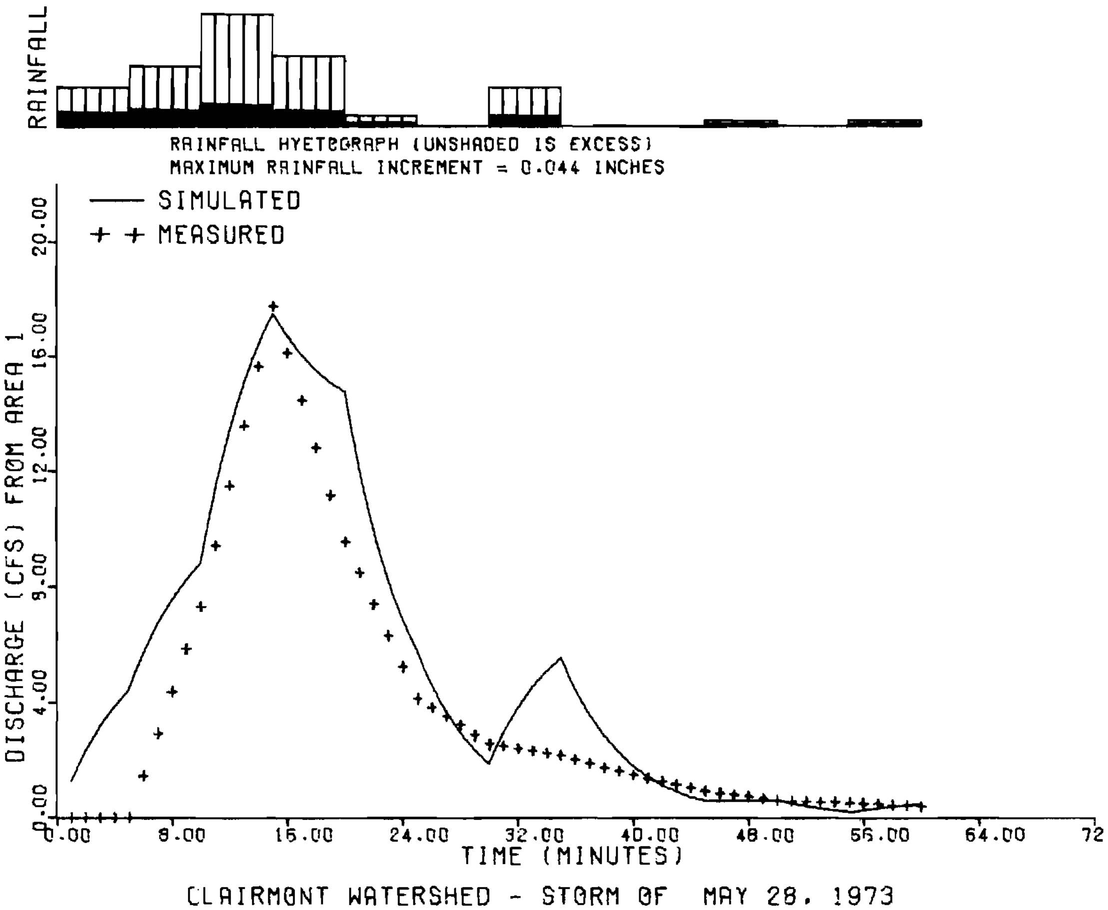


Figure 6c. Comparison of Simulated and Observed

Hydrographs, Linear Storage Model

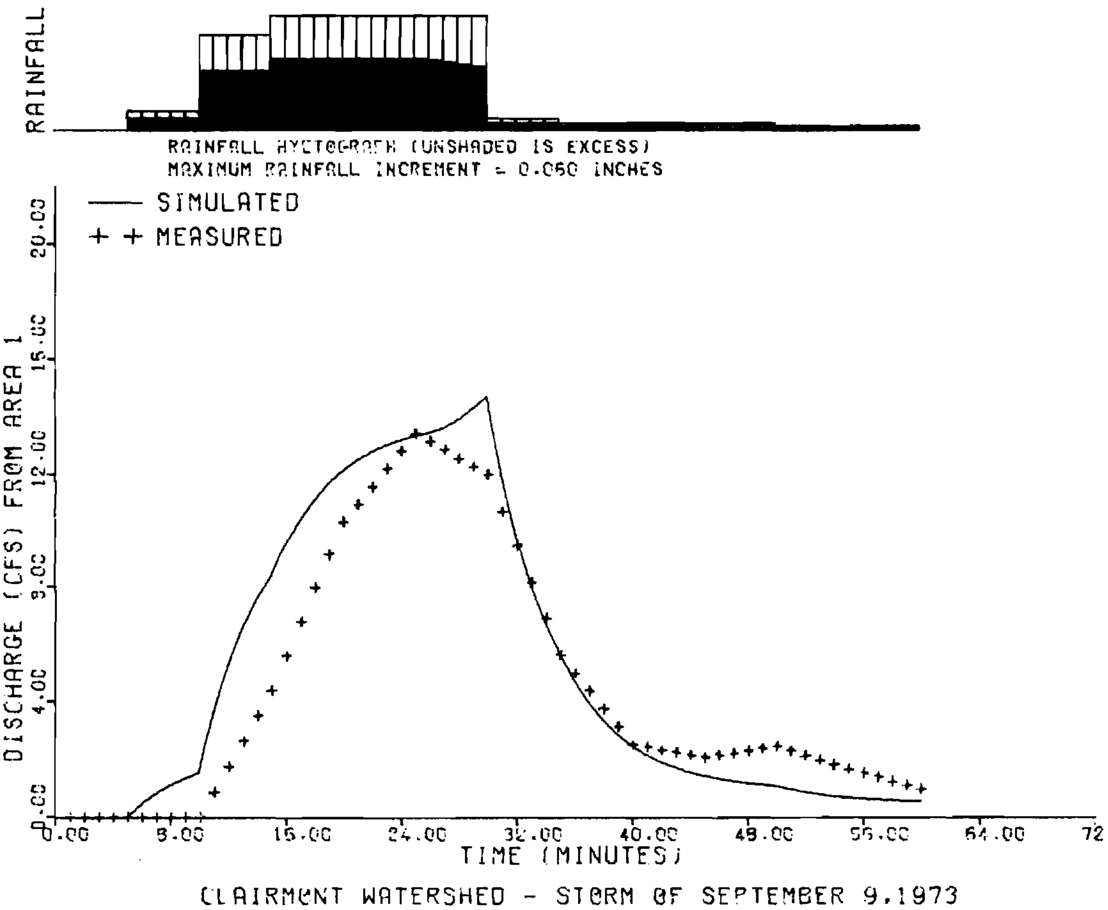


Figure 6d. Comparison of Simulated and Observed Hydrographs, Linear Storage Model

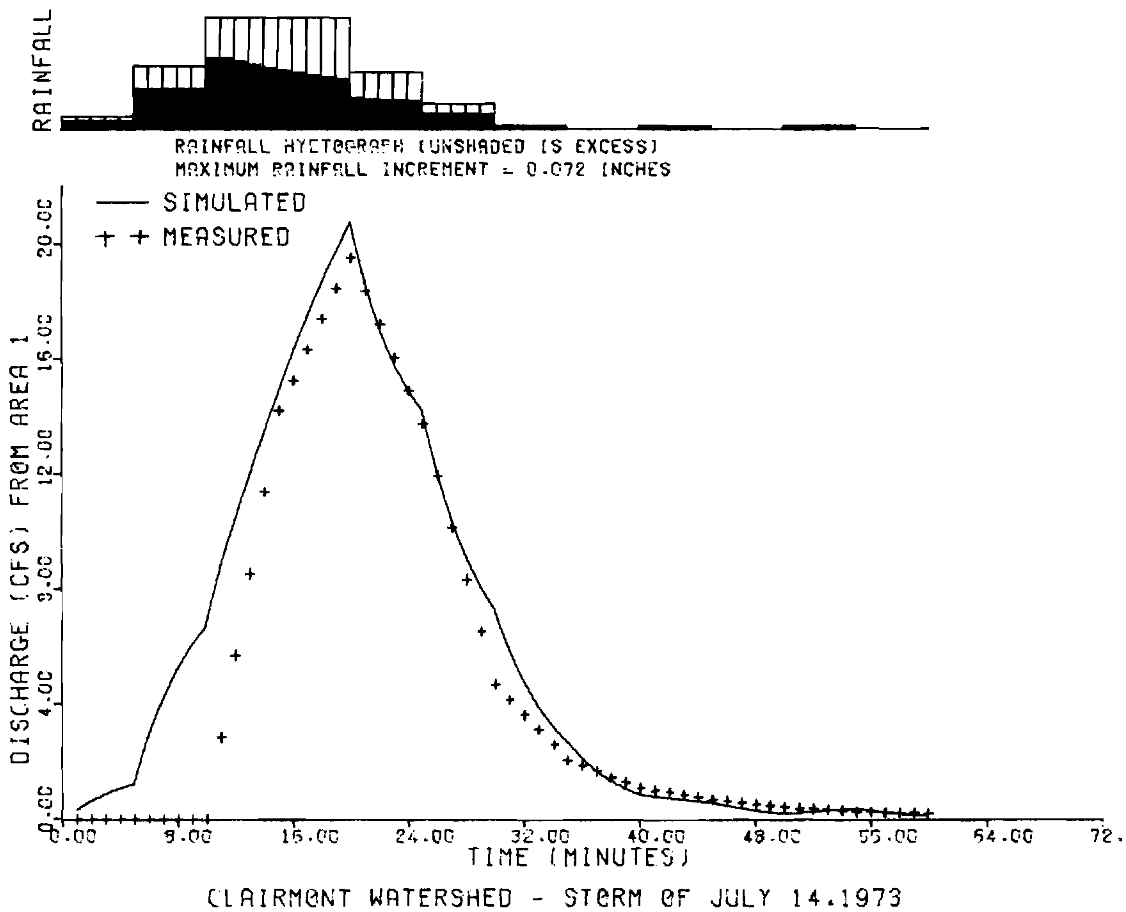


Figure 6e. Comparison of Simulated and Observed Hydrographs, Linear Storage Model

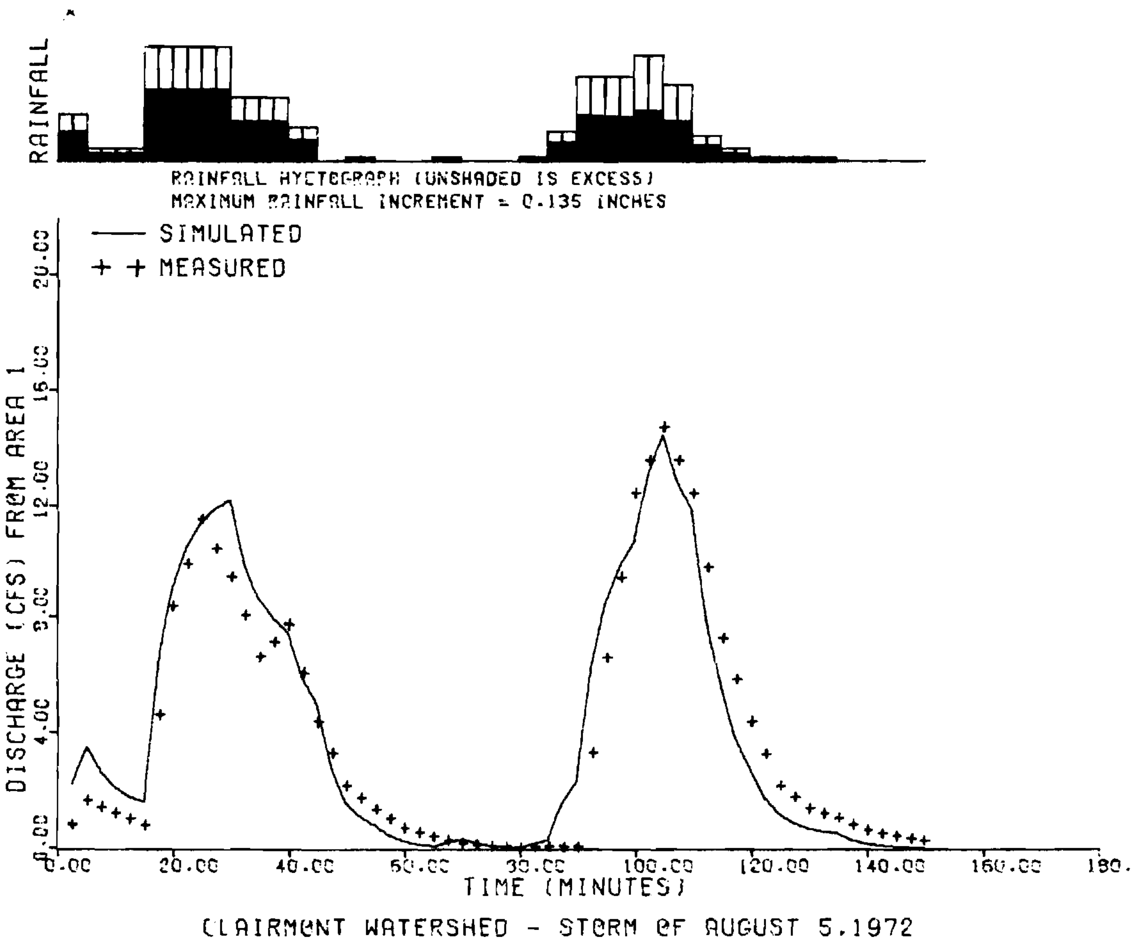


Figure 6f. Comparison of Simulated and Observed Hydrographs, Linear Storage Model

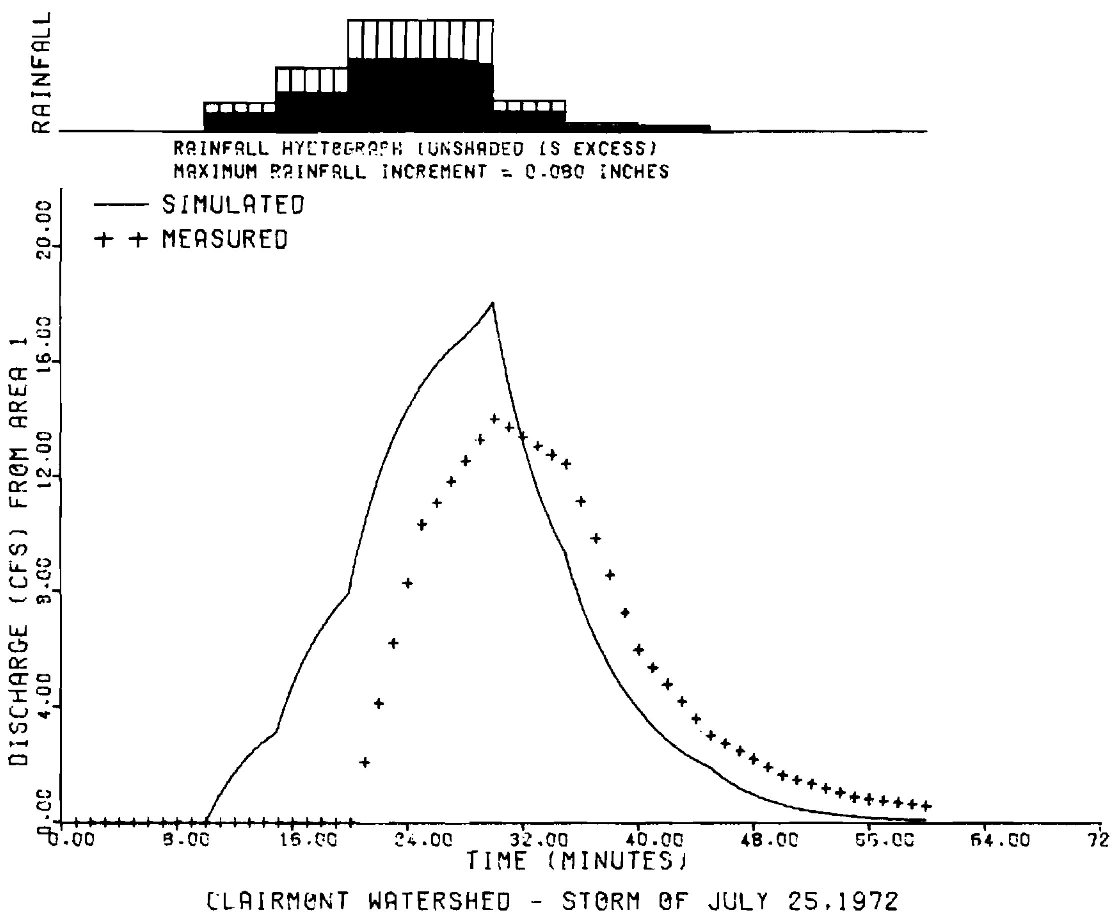


Figure 6g. Comparison of Simulated and Observed

Hydrographs, Linear Storage Model

Table 4. Infiltration Parameters

Watershed	Storm	STRKR	DLTKR	Rainfall (inches)	Runoff Simulated (inches)	Runoff Measured (inches)
S. Utoy Creek	4/6/64	0.192	1.2	3.26	1.85	1.85
	12/4/64	0.215	1.1	1.50	0.56	0.56
	2/12/66	0.226	1.2	3.78	1.84	1.84
	11/24/67	0.260	1.2	1.76	0.74	0.74
South River	4/6/64	0.115	1.2	3.26	2.31	2.31
	8/23/67	0.235	1.2	2.81	1.45	1.45
	3/12/68	0.248	1.2	2.18	1.07	1.07
N.F. Camp Creek	10/16/64	0.235	1.0	2.75	1.25	1.25
	2/12/66	0.170	1.0	3.78	2.35	2.35
Camp Creek	2/25/61	0.0425	0.25	5.67	4.94	4.93
	4/6/64	0.230	1.0	3.26	1.33	1.34
	4/27/64	0.065	0.25	2.12	1.74	1.74
	7/12/64	0.140	1.0	0.84	0.38	0.38
	10/15/64	0.300	1.0	2.75	0.73	0.73

Table 5. Sensitivity of Parameter PK

Watershed	Storm	PK	Peak Flow Simulated cfs	Peak Flow Measured cfs	Time to Peak Simulated min	Time to Peak Measured min
South Utoy	4/6/64	1.5	149	222	600	600
		1.0	156	222	600	600
		0.5	163	222	600	600
		1.5	76	97	360	360
South Utoy	12/4/64	1.0	89	97	360	360
		0.8	95	97	360	360
		0.5	106	97	360	360
South Utoy	2/12/66	1.50	165	172	960	1080
		1.30	171	172	960	1080
		1.00	179	193	360	1080
South Utoy	11/24/67	1.50	165	193	360	480
		1.00	170	193	360	480
		0.5	188	193	360	480
South River	4/6/64	1.50	333	557	600	660
		1.00	353	557	600	660
		0.50	373	557	600	660
South River	8/23/67	1.50	304	517	480	480
		1.00	330	517	480	480
		0.50	357	517	480	480
South River	3/12/68	1.50	253	325	480	480
		1.00	279	325	480	480
		0.50	310	325	480	480
N.F. Camp Creek	4/6/64	1.0	974	777	600	900
		1.50	85k	777	600	900
		1.70	808	777	600	900
		2.00	750	777	600	900
Camp Creek	4/6/64	1.00	1912	1212	600	1050
		1.75	1369	1212	600	1050
		1.90	1294	1212	600	1050
		2.20	1166	1212	600	1050

$$Q_2 = 2C_0 \left(\frac{I_1 + I_2}{2}\right) + C_2Q_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

or

$$Q_2 = 2C_0\ \bar{I} + C_2Q_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

where $\bar{I}$ = average rate of precipitation excess during the interval Δt.

Values of C_2 from Equation 13 will be negative when the time step Δt is less than twice the value of K. Thus, if a 5-minute time step is used and the drainage area is so small and the fraction of impervious area is so large that the value of K from Equation 9 is less than 2.5 minutes, then the procedure breaks down. In such cases, a smaller time step must be used or the drainage area must be combined with an adjacent area. Since the standard time steps in the routing model are 1, 5 or 15 minutes, Table 6 was developed to determine the minimum subarea sizes that can be used for the different time steps at different levels of impervious areas.

Channel Routing. The movement of flood waves is largely kinematic in all but very large flat rivers. Rastogi (1971) and Kellerhals (1970) have demonstrated that the kinematic wave equations are applicable in regions as diverse as Illinois and British Columbia. Solutions to these equations have also been programmed into the Hydrocomp Hydrologic Simulation Program (1969) and the MIT Catchment Model (1970), and each has reproduced measured hydrographs quite well.

Thus, channel routing in UROS4 is based on the solution of the kinematic wave equations

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial X} = q \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

$$Q = aA^m \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

TABLE 6. Minimum Drainage Areas

(area in acres)

Impervious Area (fraction)	Time Step 1-minute	5-minute	15 minute
0.0	.04	1.11	10.0
0.2	.08	2.05	18.5
0.4	.14	3.44	31.0
0.6	.22	5.39	48.5
0.8	.32	8.01	72.1
1.0	.46	14.41	102.7

in which A is the cross-sectional area of flow, Q is the rate of flow, q is the rate of lateral inflow, t is time, x is distance along the channel reach in the downstream direction, and the parameters a and m are determined by channel slope, roughness, and size. Substituting Equation 17 into Equation 16 gives

$$\frac{\partial A}{\partial t} + amA^{m-1}\frac{\partial A}{\partial x} = q \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Thus, A is a function of x, t, and q, and can be substituted into Equation 17 to determine Q.

The solution of Equation 18 involves finite approximations to the following derivatives

$$\frac{\partial A}{\partial t} = \frac{A(x + \Delta x,\ t + \Delta t) - A(x + \Delta x, t)}{\Delta t} \quad \ldots\ldots\ldots\ldots (19)$$

$$\frac{\partial A}{\partial x} = \frac{A(X + \Delta x,\ t + \Delta t) - A(x,\ t + \Delta t)}{\Delta x} \quad \ldots\ldots (20)$$

and the following substitutions for the remaining terms.

$$amA^{m-1} = am\left(\frac{A(x,\ t + \Delta t) + A(x + \Delta x,\ t)}{2}\right)^{m-1} \quad \ldots\ldots (21)$$

$$q = \frac{q(x + \Delta x, t) + q(x + \Delta x,\ t + \Delta t)}{2} \quad \ldots\ldots\ldots\ldots (22)$$

Substituting equations 19-22 into Equation 18 and solving for $A(x + \Delta x,\ t + \Delta t)$ gives

$$A(x + \Delta x,\ t + \Delta t) = \frac{U + V + W}{Z} \quad \ldots\ldots\ldots\ldots (23)$$

where: $$U = \frac{A(x + \Delta x,\ t)}{\Delta t} \quad \ldots\ldots\ldots\ldots\ldots\ldots \ (24)$$

$$V = ma \left(\frac{A(x,\ t + \Delta t) + A(x + \Delta x,\ t)}{2\Delta x}\right)^{m-1} A(x,\ t + \Delta t) \ .(25)$$

$$W = \frac{q(x + \Delta x,\ t) + q(x + \Delta x,\ t + \Delta t)}{2} (\Delta x) \ \ldots\ldots\ (26)$$

$$Z = \frac{1}{\Delta t} + ma \left(\frac{A(x,\ t + \Delta t) + A(x + \Delta x,\ t)}{2}\right)^{m-1} \ \ldots\ldots\ (27)$$

Discharge at the downstream point then becomes

$$Q(x + \Delta x,\ t + \Delta t) = aA(x + \Delta x,\ t + \Delta t)^m \ \ldots\ldots\ldots\ldots (28)$$

For the computer code and the remainder of the report, the parameters a and m will be listed APARM and MPARM, respectively. Values for APARM and MPARM for different geometric channel shapes are given in Table 7.

Sensitivity of the routed flow to the incremental channel reach length, Δx, used in the routing was examined by comparing the routed discharge from one 1000-foot reach with that at the downstream end of a series of ten 100-foot reaches. Although the hydrographs are similar (Figure 7), minor differences can be noted. Besides the different estimates of flood peaks, a difference exists in the way in which the flood wave is transmitted. For the single long reach, changes in inflow (due to changing precipitation) are quickly transmitted downstream to change outflow. These results suggest that the selection of reach length is important but not critical. A reach length equal to the time step divided by the average velocity would make the finite approximation to Equation 14 a little more accurate than longer or shorter reaches.

Table 7. Values of Kinematic Routing Coefficients for Three Channel Shapes

Channel Type	APARM	MPARM	L
Rectangular	$\frac{1.49\ S_o^{0.5}}{n\ L^{0.67}}$	1.67	Width of Channel
Circular	$\frac{1.49\ S_o^{0.5}}{n(0.25L)}$	1.00	Diameter
Triangular	$\frac{1.49\ S_o^{0.5}}{n\ L^{0.33}}$	1.33	Width at 1.0 ft. Depth

S_o = Longitudinal Slope of Channel Bottom

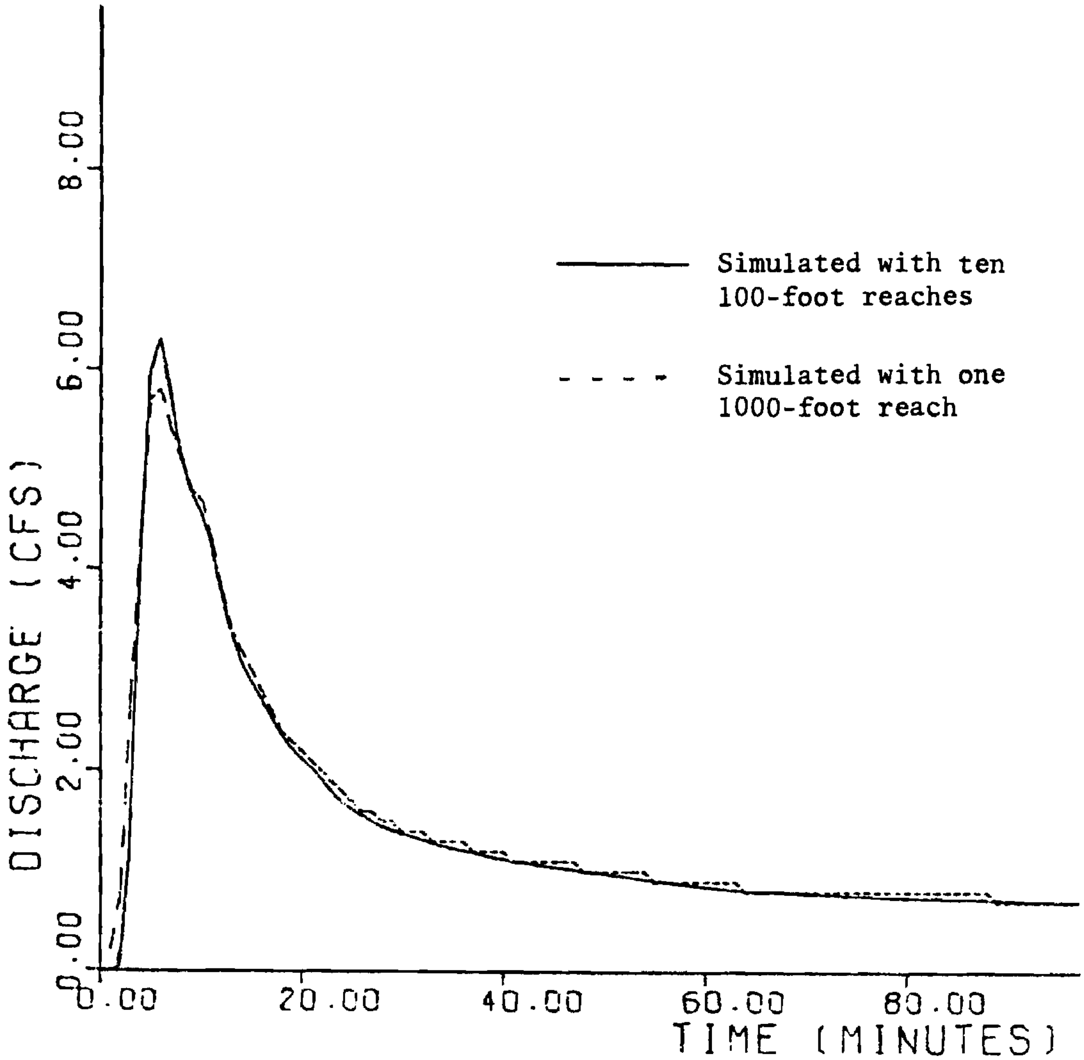


Figure 7. Effects of Number of Channel Increments on Kinematic Routing of a Flood Hydrograph

In order to apply the kinematic routing equations, it is necessary to develop an appropriate method for estimating APARM and MPARM. Lumb (1973) studied travel times in Georgia streams and found a correlation between Q and A of the form of Equation 17. Values of APARM and MPARM were determined from U.S. Geological Survey data for over 140 streamgage stations in Georgia. APARM and MPARM for the six watersheds in the metropolitan Atlanta area were plotted and a curve sketched between the points. Two points were selected from the curve for APARM and MPARM, and flow from selected storms on the four drainage areas were simulated. The results are shown on Table 8. The simulation is fairly good for South Utoy where the channels remain in their natural state. Simulated peak flows for South River are too low because the channels have been improved and channel storage reduced. Simulated peak flows for North Fork Camp Creek and Camp Creek are too high because the swampy lower reaches provide more storage than does the average Georgia stream. The results of Table 9 thus indicate that statewide average values for APARM and MPARM estimated from the U.S. Geological Survey discharge measurements are not adequate for kinematic routing in DeKalb streams and that it is necessary to measure the physical characteristics of the channel reaches from maps and field surveys.

A second study was conducted on APARM and MPARM to determine if the same values could be used for both flows within the stream channel and flows extending on the flood plain. Six cross-sections on North Fork Camp Creek were selected from data available from the Corps of Engineers. Discharge was calculated with Manning's equation for each cross-section at several water surface elevations below and above the bankfull level. Plots of the calculated discharge versus cross-sectional area on Figures 8 through 13 show a break point exists at bankfull channel capacity. Straight lines

Table 8. Peak Flows for USGS based APARM and MPARM

				Peak Flow		Volume Runoff		Time to Peak	
Watershed	Storm	APARM	MPARM	Simulated	Measured	Simulated	Measured	Simulated	Measured
S. Utoy	4/6/64	0.186	2.0	165	222	1.83	1.85	600	600
		0.678	1.25	158					
	12/4/64	0.186	2.0	106	97	0.56	0.56	360	360
		0.678	1.25	95					
	2/12/66	0.186	2.0	192	172	1.85	1.84	975	1080
		0.678	1.25	186					
	11/24/67	0.186	2.0	231	193	0.75	0.74	360	480
		0.678	1.25	204					
South R.	4/6/64	0.186	2.0	375	557	2.24	2.31	600	660
		0.678	1.25	355					
	8/23/67	0.186	2.0	353	517	1.46	1.45	480	480
		0.678	1.25	336					
	3/12/68	0.186	2.0	283	325	1.07	1.07	360	480
		0.678	1.25	253					
N.F. Camp Ck.	10/16/64	0.186	2.0	469	536	1.08	1.25	720	1500
		0.678	1.25	399					
	2/12/66	0.186	2.0	1215	823	2.02	2.35	940	1140
		0.678	1.25	1044					
Camp Ck.	2/25/61	0.186	2.0	9680	4000	4.96	4.94	780	1080
		0.678	1.25	6330					
	4/6/64	0.186	2.0	2890	1212	1.46	1.34	600	1050
		0.678	1.25	2390					
	4/27/64	0.186	2.0	4440	1488	1.76	1.74	540	780
		0.678	1.25	3050					
	7/12/64	0.186	2.0	1800	463	0.39	0.38	540	840
		0.678	1.25	924					

Table 9. Calculated APARM and MPARM from Channel Sections on North Fork Camp Creek

x-section number	Channel Flow APARM	Channel Flow MPARM	Overbank Flow APARM	Overbank Flow MPARM
3	2.53	1.01	1.96	0.834
5	0.411	1.50	10.64	0.875
6	0.295	1.56	43.17	0.735
7	1.04	1.37	20.87	0.696
9	1.91	1.28	19.40	0.772
10	1.74	1.33	17.50	0.807

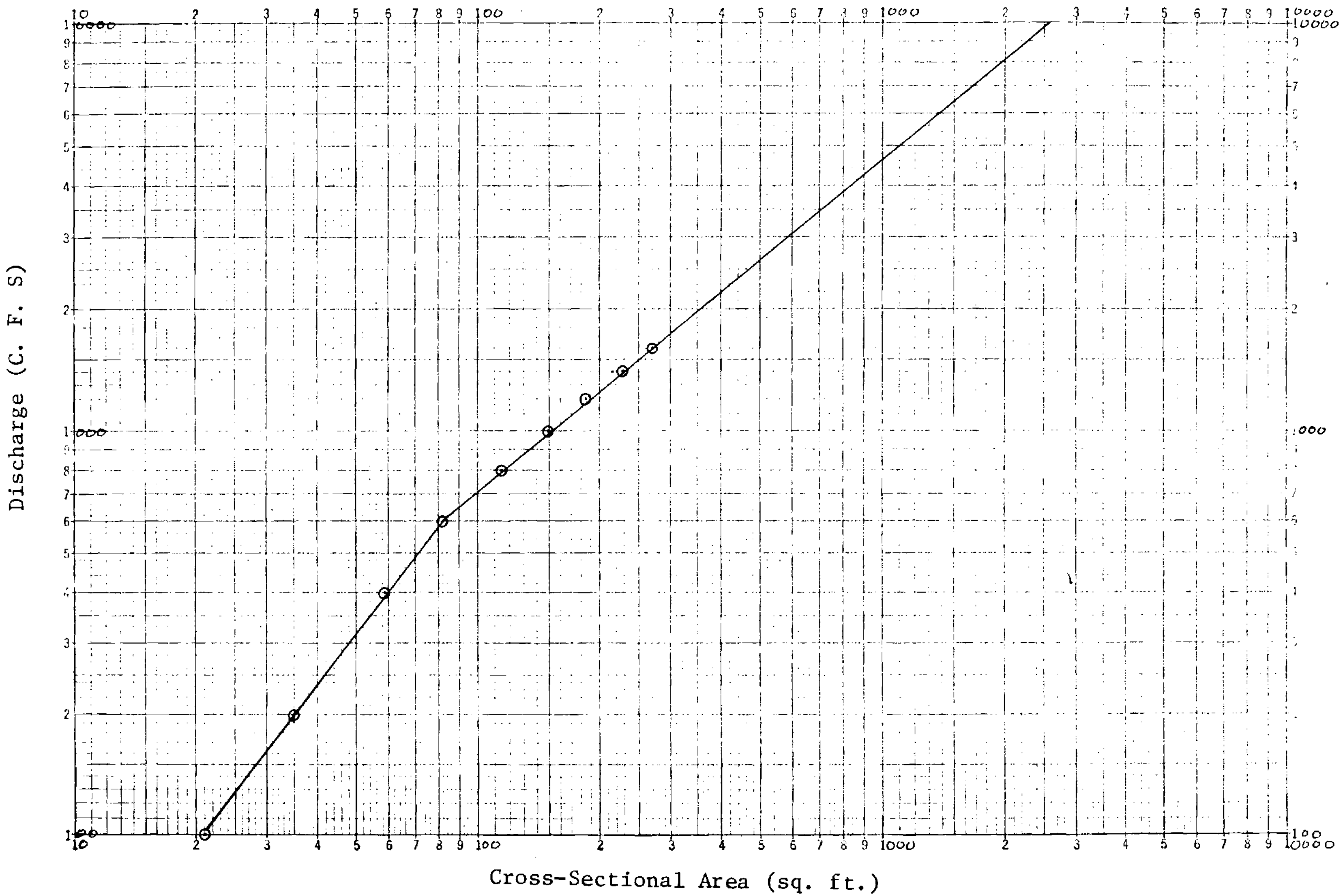


Figure 8. Discharge vs. Cross-Sectional Area, Section 10, North Fork Camp Creek

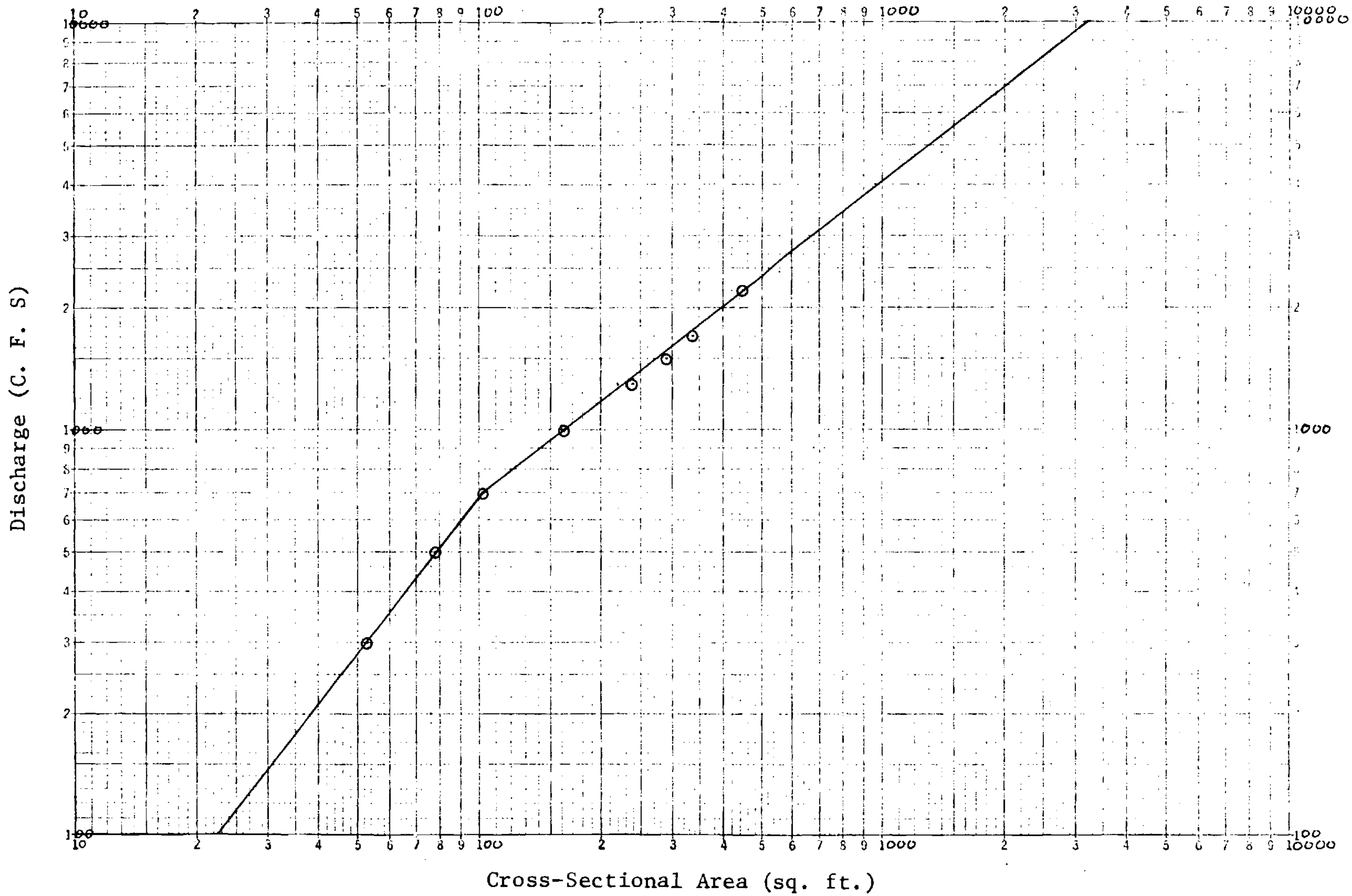


Figure 9. Discharge vs. Cross-Sectional Area, Section 9, North Fork Camp Creek

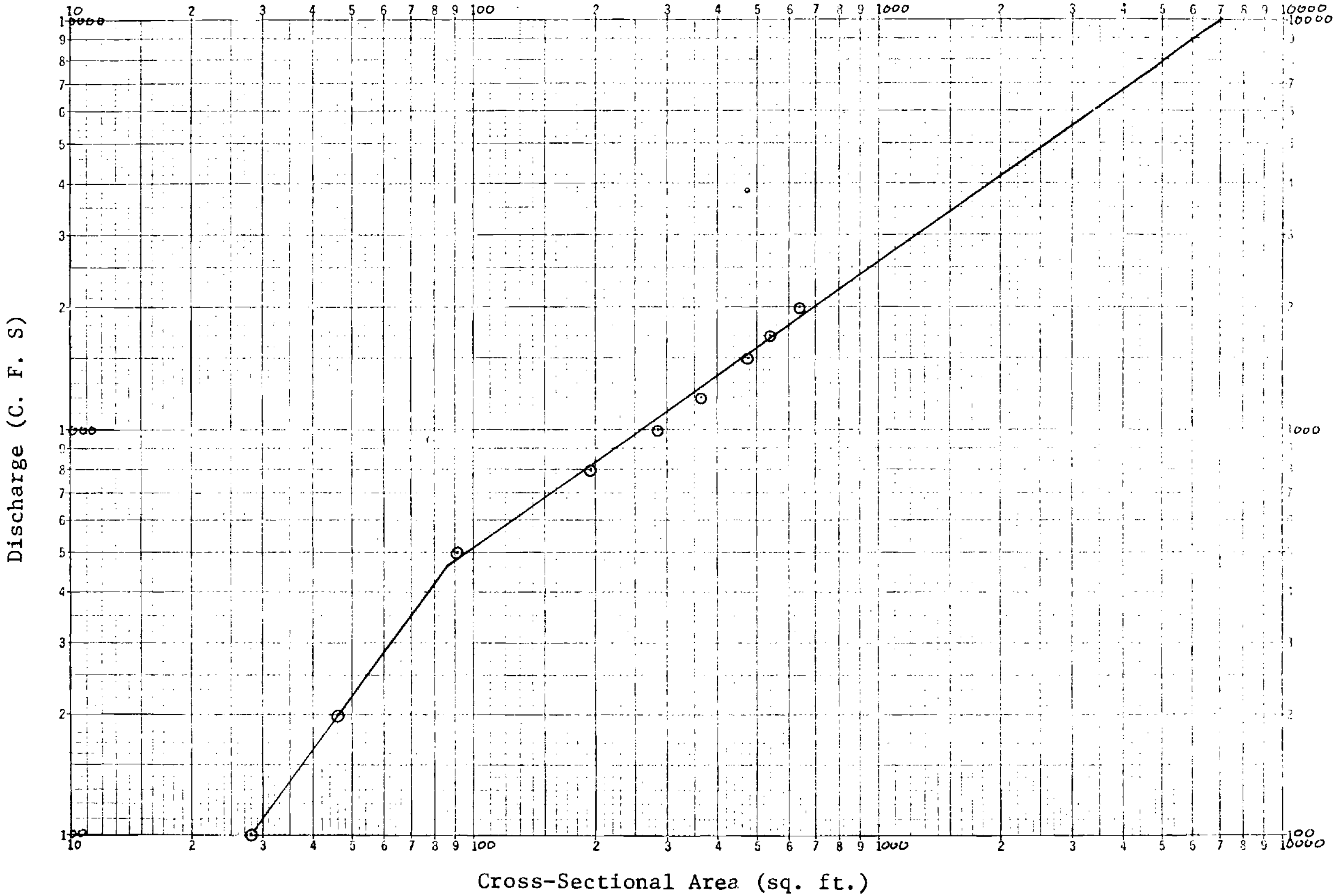


Figure 10. Discharge vs. Cross-Sectional Area, Section 7, North Fork Camp Creek

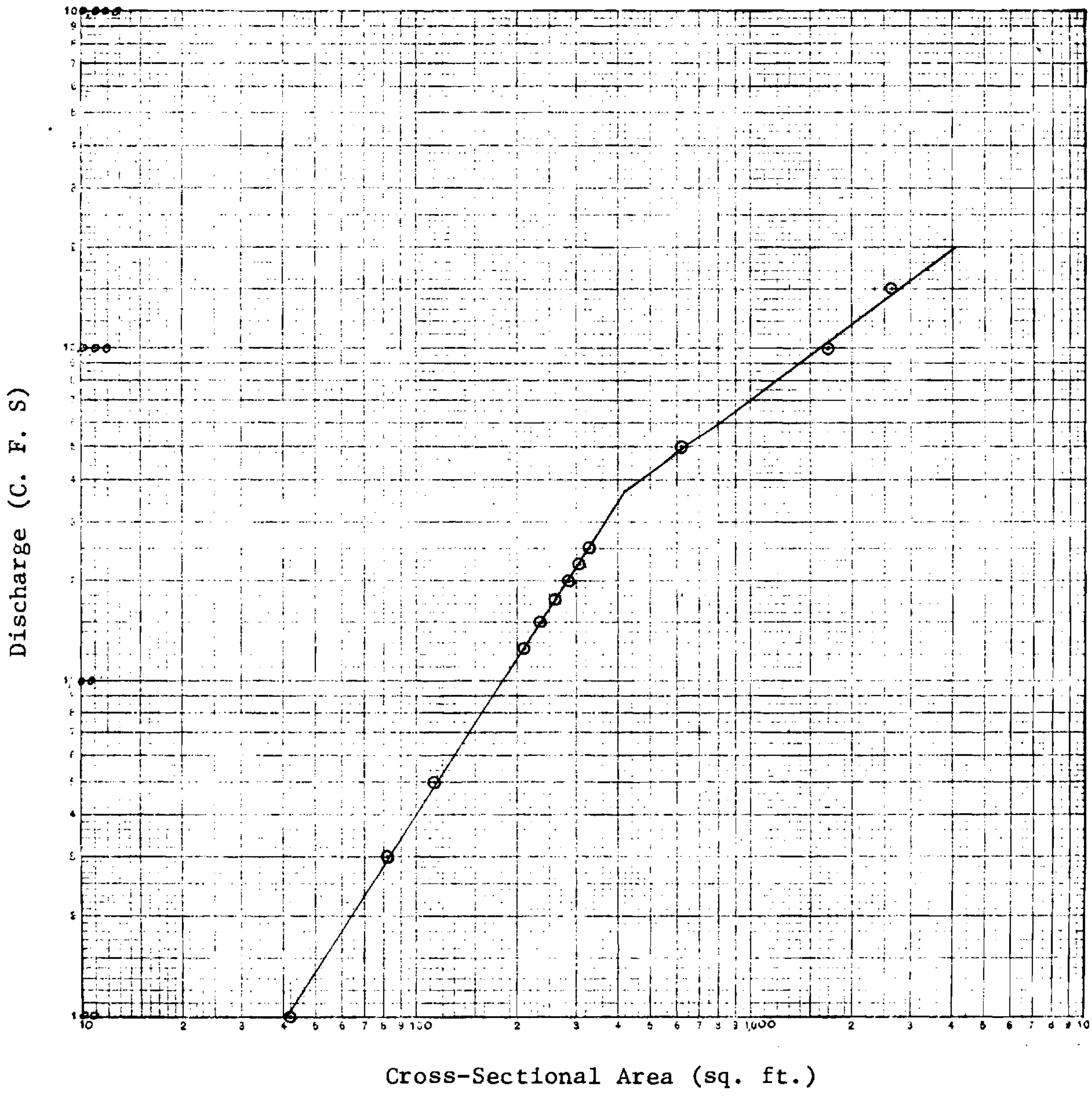


Figure 11. Discharge vs. Cross-Sectional Area, Section 6 , North Fork Camp Creek

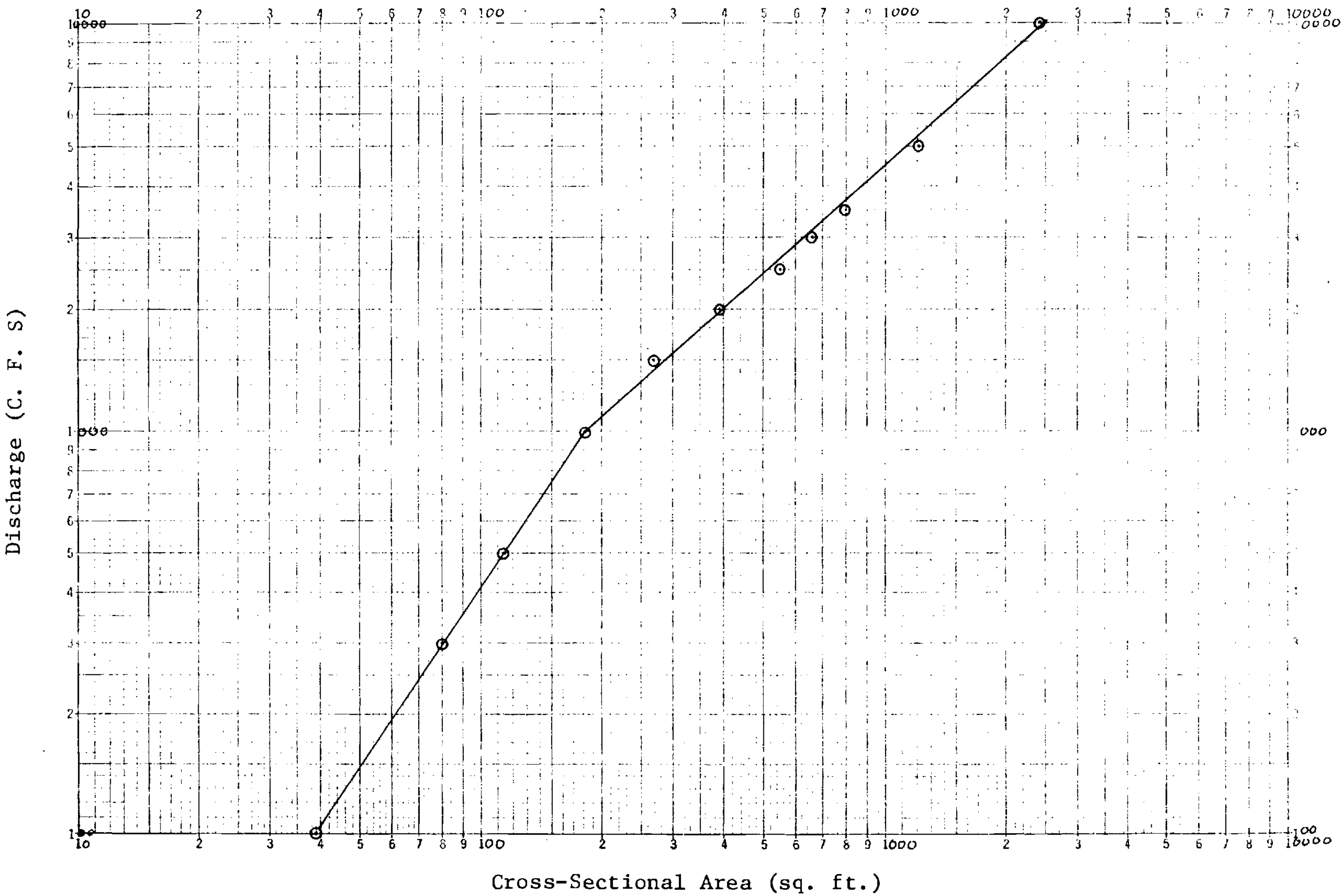


Figure 12. Discharge vs. Cross-Sectional Area, Section 5, North Fork Camp Creek

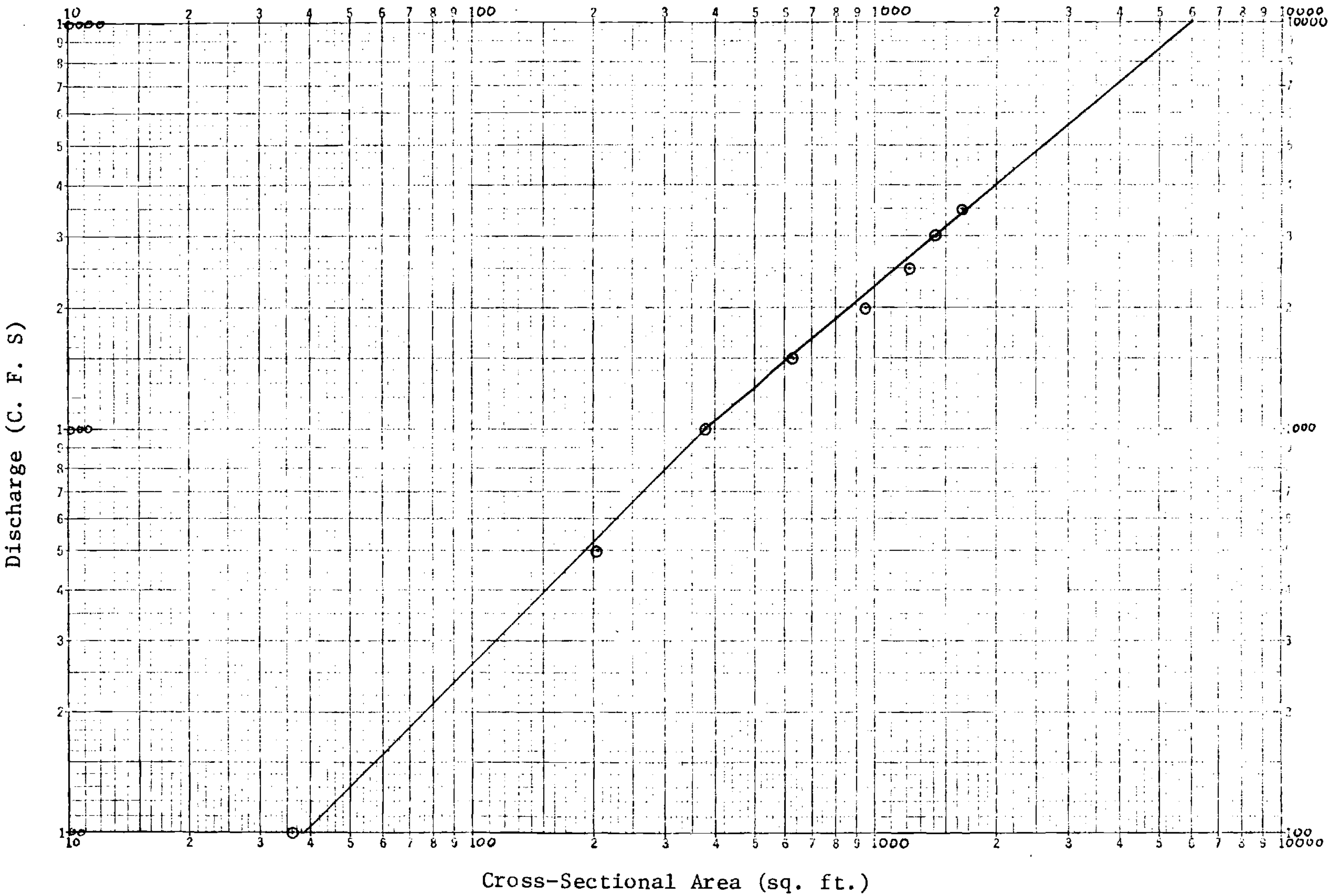


Figure 13. Discharge vs. Cross-Sectional Area, Section 3, North Fork Camp Creek

were drawn through the points and values of APARM and MPARM calculated. The results are shown in Table 9 and Figure 14. Channel slopes ranged from 0.0034 at a downstream reach to 0.0097 at an upstream reach. Manning's n for the flood plain was assumed to be 0.12, and a value of 0.04 was used for the channel.

Since the above results dictated the use of channel routing parameters estimated from specific field data, the model was programmed to accept physical characteristics of the channel and flood plain. These characteristics for a water surface elevation as shown on Figure 15 are:

AC = cross-sectional area of the channel

ACX = cross-sectional area projected above the channel

ALT = cross-sectional area of the left flood plain

ART = cross-sectional area of the right flood plain

WPC = wetted perimeter of the channel

WPL = wetted perimeter of the left flood plain

WPR = wetted perimeter of the right flood plain

FNC = Manning's n of the channel

FNL = Manning's n of the left flood plain

FNR = Manning's n of the right flood plain

SLOPE= slope of the hydraulic grade line as approximated by that of the invert of the channel

Given 1) the cross-sectional area of the channel, 2)wetted perimeter, 3) Manning's n and 4) slope of the invert of the channel; the associated discharge at channel capacity can be calculated from Manning's equation

$$QC = \frac{1.49}{FNC}\left(\frac{AC}{WPC}\right)^{2/3}(AC)(SLOPE)^{1/2} \qquad (29)$$

Assuming an MPARM of 1.5 for channel flow, a number less that that for an

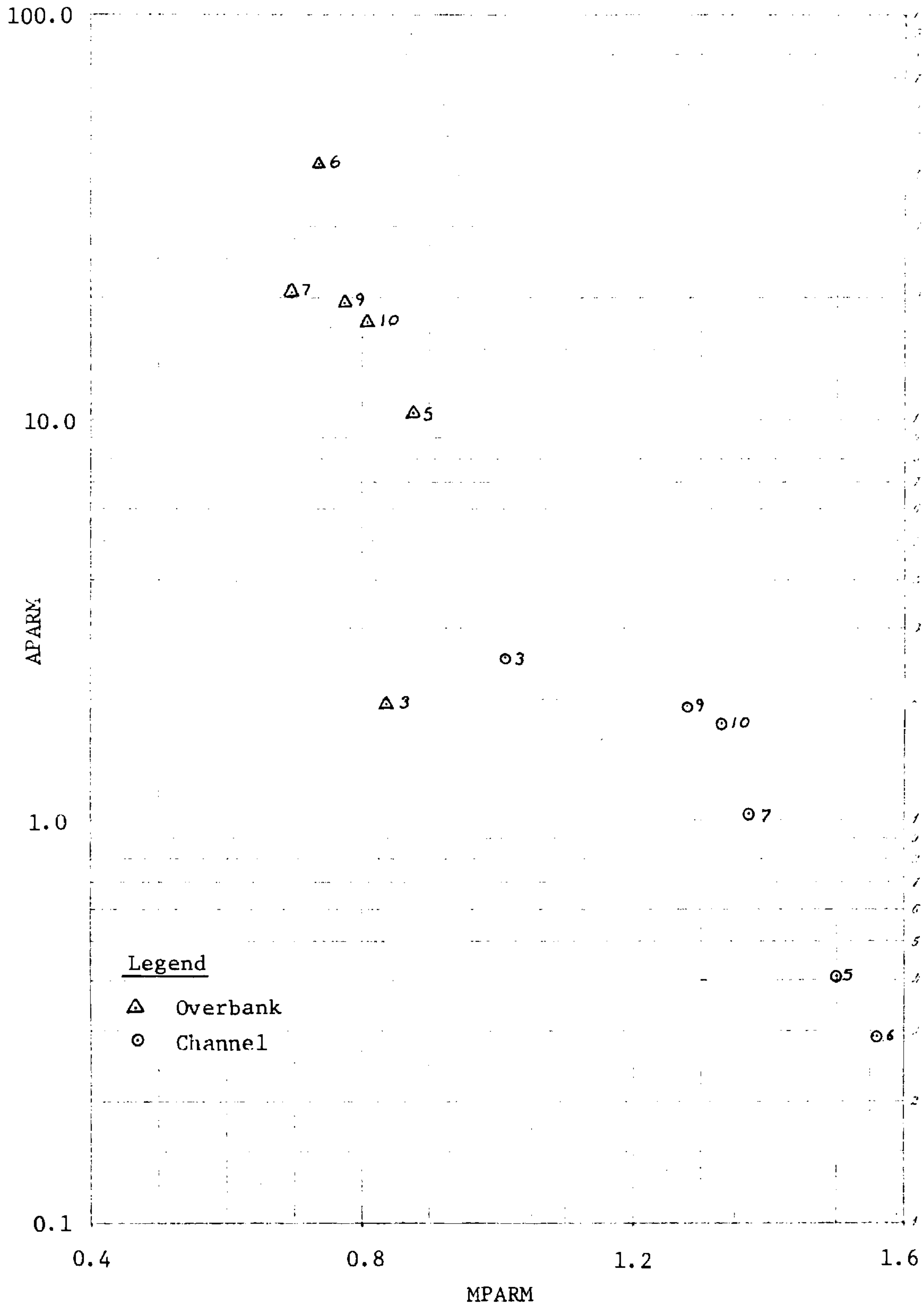
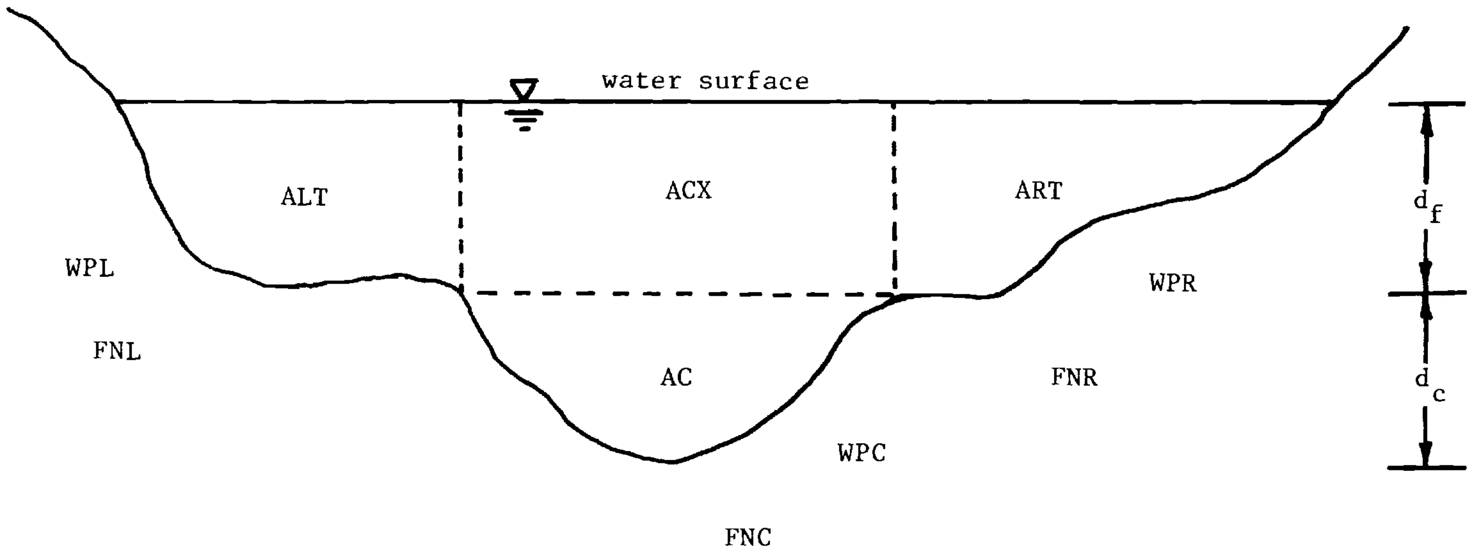


Figure 14. Plot of APARM vs. MPARM



$$0.3 < \frac{d_f}{d_c} < 0.7$$

Figure 15. Channel Cross-section Definition Sketch

infinitely wide channel and near the average for North Fork Camp Creek, APARM for the channel can be calculated by

$$APARM = e^{(\log_e QC - MPARM*\log_e AC)} \quad \text{................} \quad (30)$$

MPARM for flows exceeding channel capacity is calculated by the following equations.

$$MPARM_{fp} = \log_e(QS/QC)/\log_e(AS/AC) \quad \text{.............} \quad (31)$$

where

$$AS = AC + ACX + ART + ALT \quad \text{................} \quad (32)$$

$$QS = 1.49(SLOPE)^{1/2}(ZQ) \quad \text{................} \quad (33)$$

$$ZQ = \frac{(AC + ACX)}{FNC} \left(\frac{AC + ACX}{WPC}\right)^{2/3} + \frac{ART}{FNR} \left(\frac{ART}{WPR}\right)^{2/3} + \frac{ALT}{FNL} \left(\frac{ALT}{WPL}\right)^{2/3} \quad .(34)$$

APARM for flows exceeding channel capacity is

$$APARM_{fp} = e^{(\log_e QC - MPARM_{fp} \log_e AC)} \quad \text{..............} \quad (35)$$

For simulation of channel flows above the value of QC, $APARM_{fp}$ and $MPARM_{fp}$ are substituted.

The sensitivity of simulated flows to the values used for parameters APARM and MPARM was studied for the Camp Creek drainage area. Table 10 shows the results from using several values of APARM. Overall values of APARM equal to 0.15 and MPARM equal to 1.25 matched peaks. Little weight, however, should be placed on these values because

Table 10. APARM Sensitivity on Camp Creek with MPARM = 1.25

Storm	APARM	Peak Flow cfs Simulated	Measured
2/25/61	0.0007	39	4000
	0.007	326	4000
	0.070	2569	4000
	0.150	3830	4000
	0.200	4203	4000
4/6/64	0.0007	9	1212
	0.007	74	1212
	0.070	632	1212
	0.150	1250	1212
	0.200	1512	1212
4/27/64	0.0007	11	1488
	0.007	91	1488
	0.070	761	1488
	0.150	1556	1488
	0.200	1927	1488
7/12/64	0.0007	2	463
	0.007	16	463
	0.070	129	463
	0.150	256	463
	0.200	334	463
10/16/64	0.0007	4	548
	0.007	32	548
	0.070	262	548
	0.150	517	548
	0.200	633	548

storage in small reservoirs and behind roadways were not considered and the same values were used for all reaches. Reaches in the lower portion of Camp Creek are flat and swampy while the upper tributaries have steeper channels.

The kinematic channel routing model was also verified on the Clairmont Watershed, a 10.8-acre residential watershed. The watershed is located in the vicinity of Clairmont Road and Black Fox Drive (Figure 16). Runoff collects in a wooded ravine about 15 feet deep and eventually discharges into the North Fork of Peachtree Creek. The watershed area contains all or part of 19 residential lots and is approximately equally divided among impervious, lawn, and wooded areas.

Most of the area drains into gutters along one of the streets and thence into the ravine at the nearest drainage inlet. Approximately three quarters of the impervious area drains directly into a gutter while the remaining quarter (mostly roofs) drains onto lawns or wooded areas. The flow at the gage rises rapidly to a peak within about 15 minutes after a heavy rain and becomes dry again within a few hours after a storm ends. Because of the relatively large impervious area, runoff occurs during all but the very lightest rain.

Precipitation and streamflow have been recorded since July, 1971. Both a recording and a storage precipitation gage are located just outside the watershed near the downstream end. Streamflow is measured by a sharp crested rectangular weir 3-feet wide and 2-feet deep. Heads greater than 2 feet cause water to flow over the sidewalls. A stage recorder is used for continuous measurements of head and is located about three feet upstream from the crest. The weir capacity is 31.1 cfs, and two storms have been large enough to cause some overtopping of the sidewalls. The flood of record is approximately 32.5 cfs.

Several storms were selected for calibration of the infiltration

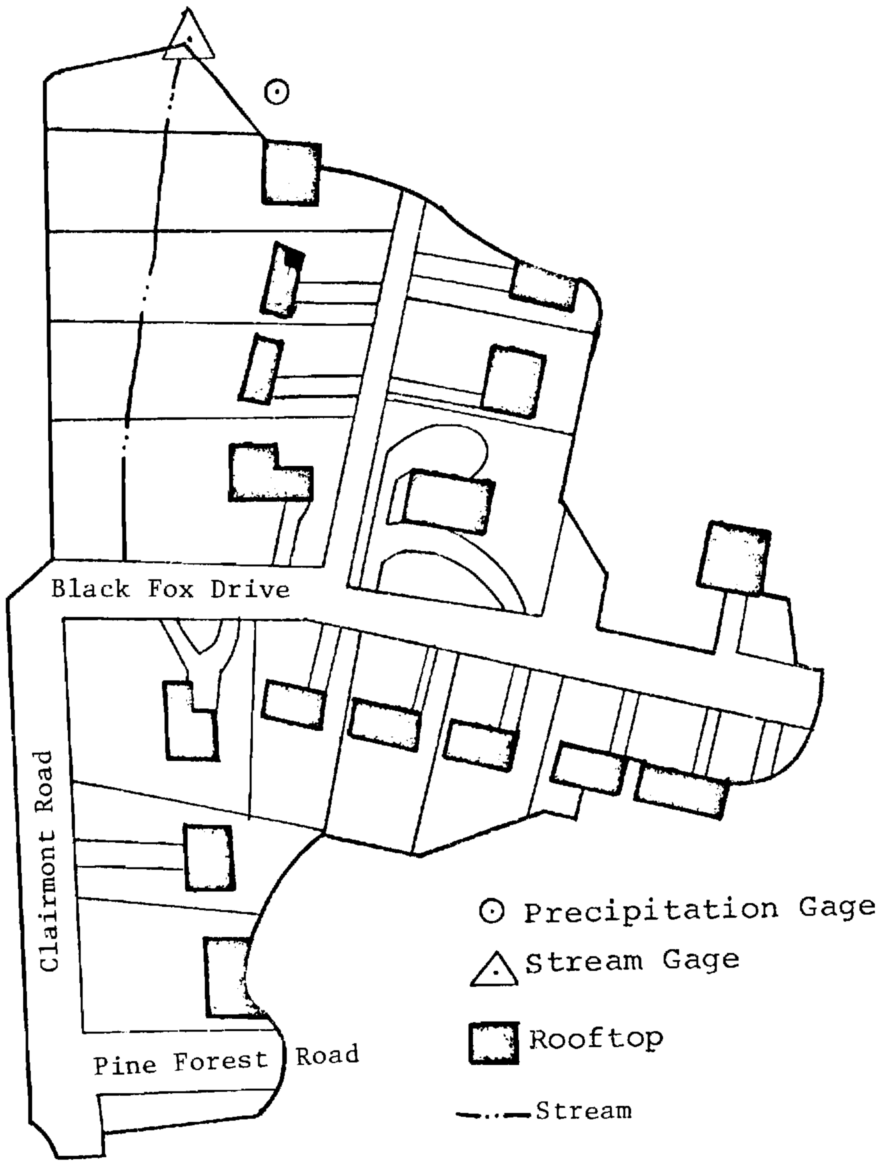


Figure 16. Geomentry of Clairmont Watershed

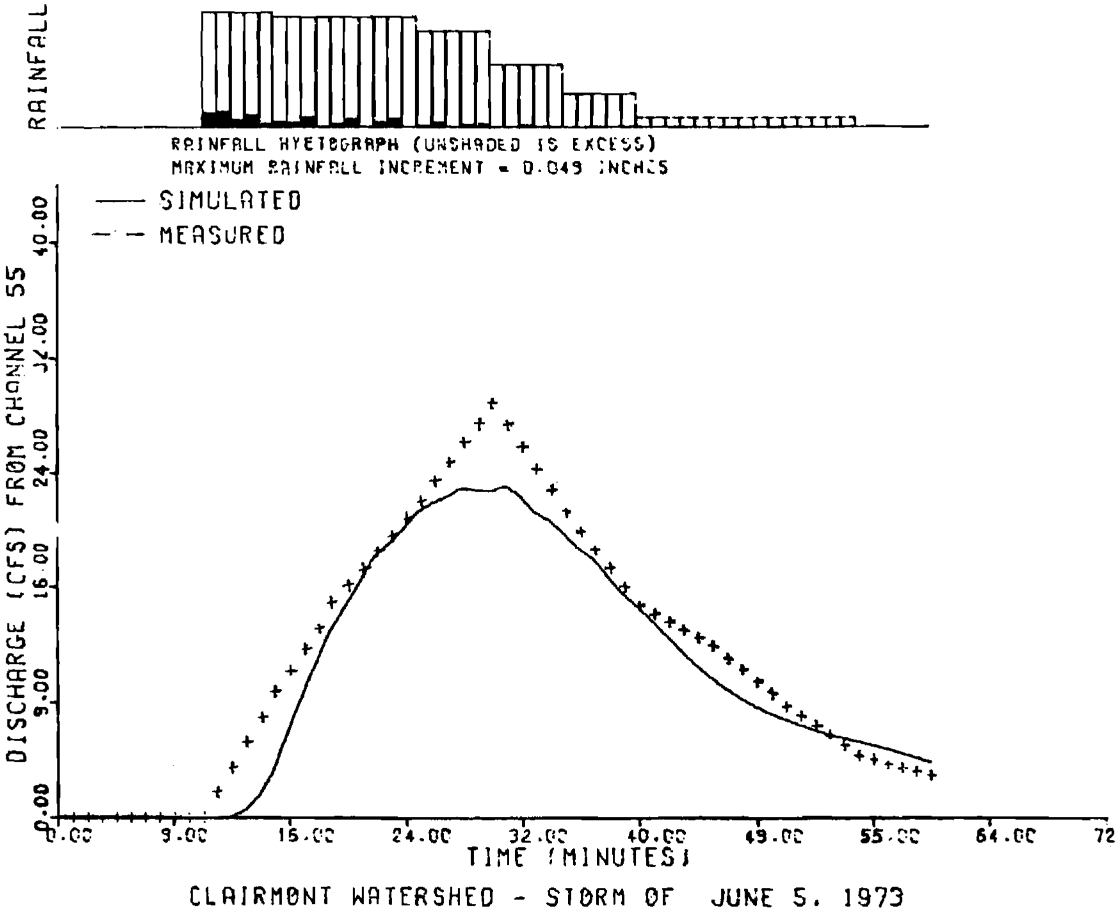


Figure 17. Comparison of Simulated with Observed Hydrograph for Storm of June 5, 1973, on Clairmont Watershed

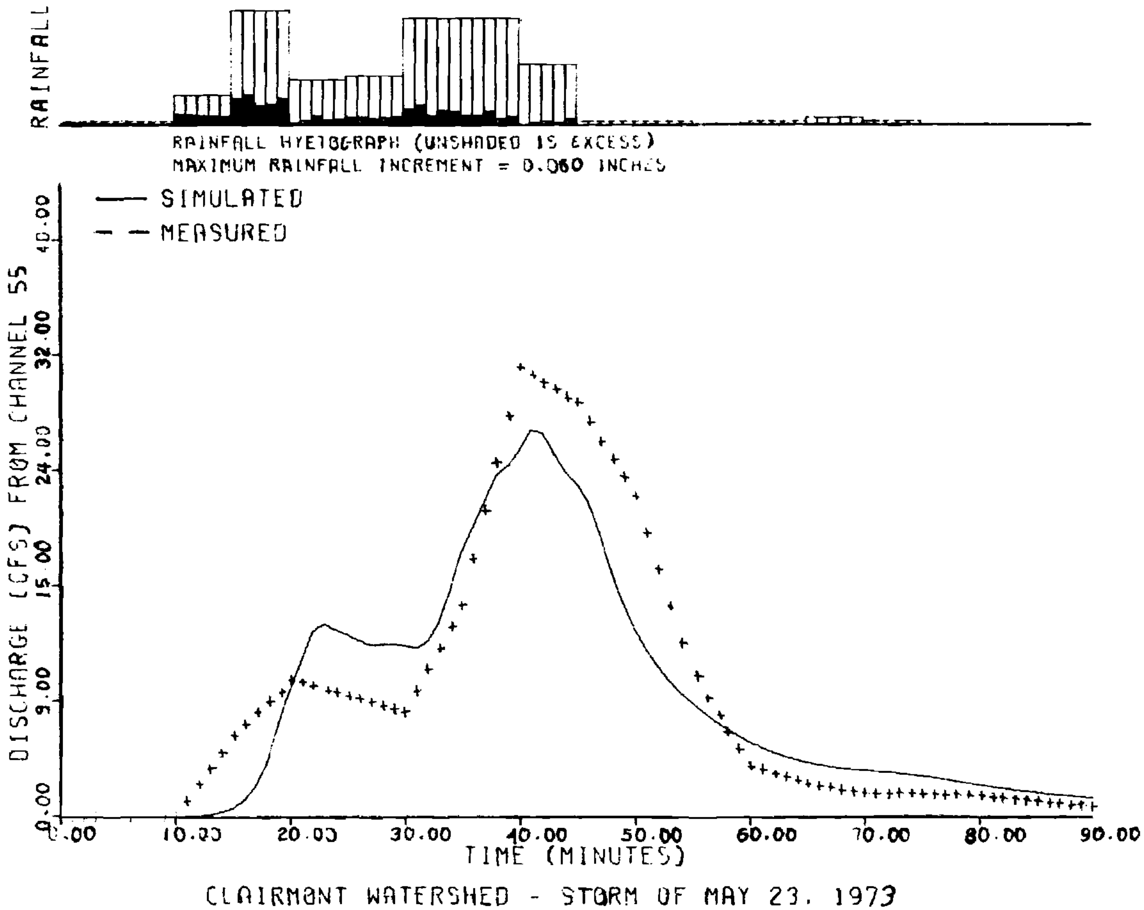


Figure 18. Comparison of Simulated with Observed Hydrograph for Storm of May 23, 1973, on Clairmont Watershed

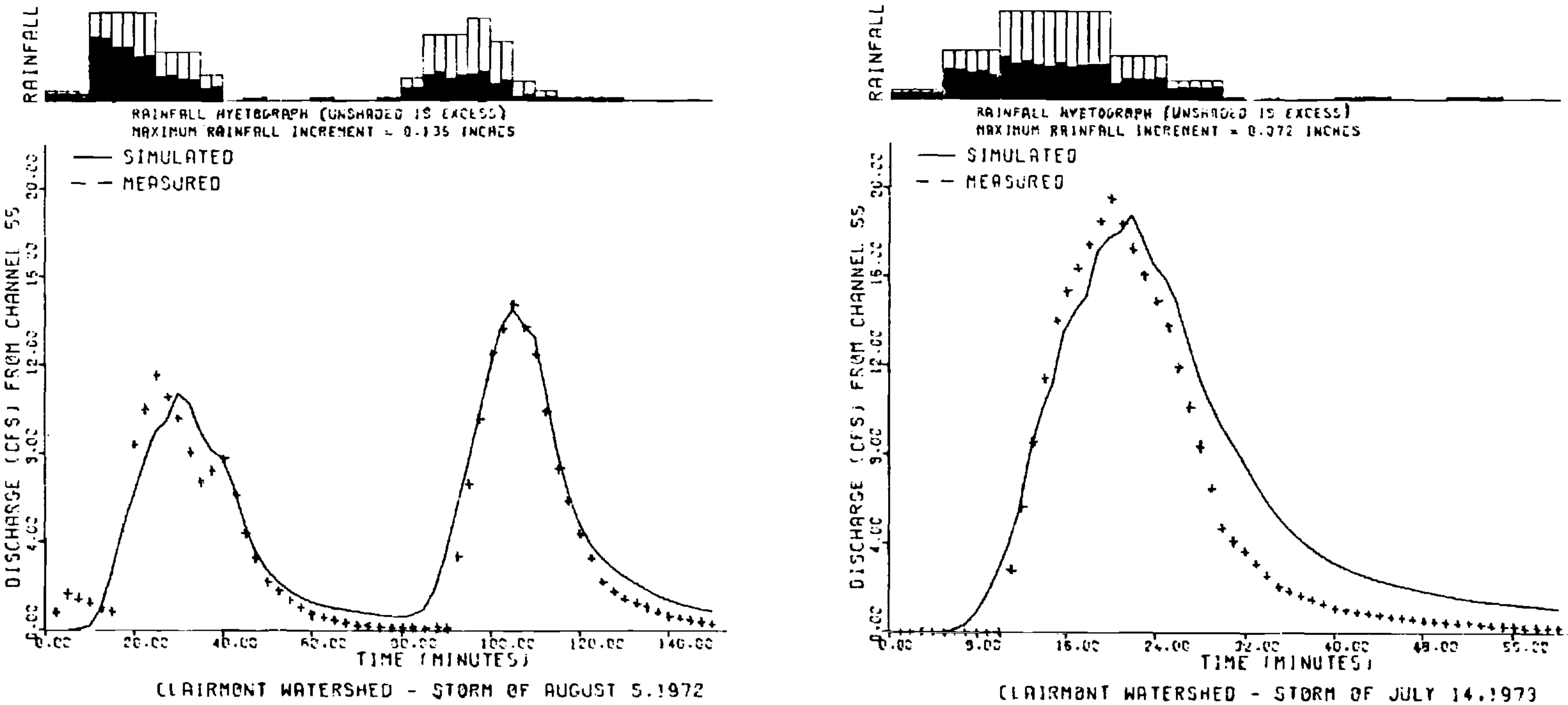


Figure 19. Comparison of Simulated with Observed Hydrograph for Two Smaller Storms on Clairmont Watershed

parameters for the Clairmont watershed and verification of the timing of the runoff. Infiltration parameters RTIOL and ERAIN were given values of 3.0 and 0.7, respectively, based on application of the HEC-1 program to other watersheds in the Atlanta area. The other two parameters, DLTKR and STRKR, were determined by trial and error for each storm. With the computed rainfall excess adjusted with DLTKR and STRKR to match the measured rainfall excess, the computed peak flows for the three largest events were within 8 percent of the measured peak flows (Figure 17-19). This agreement in peak flows was accomplished by dividing the Clairmont watershed into 30 source areas and 11 channel reaches. Characteristics of the channels and source areas were measured on the watershed and no other parameters except DLTKR and STRKR required calibration.

One problem with this verification of the model on the Clairmont Watershed was the accuracy of the data used for comparison. The finest time increment in which rainfall and runoff amounts could be read from the recorders was 5 minutes. Consequently, hydrograph shapes were not as easily verified as were the peak flows. Comparison of the timing of runoff with respect to streamflow was made more difficult because spring and pendulum clocks cannot be synchronized within one minute. Even without considering the data problems, the model simulates the storm hydrograph quite well.

Routing Through Storage Segments

The procedure used for routing streamflows through storage requires application of the continuity equation

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{\Delta t} \quad \ldots\ldots\ldots\ldots\ldots(36)$$

in which I is inflow, O is outflow, and S is storage. The subscripts refer to time one and two and the difference in the two times is Δt. In Equation 36 the unknowns are O_2 and S_2. The additional equation needed for a solution is a relation between outflow and storage and can be developed from analysis of the hydraulics of the geometry of the outflow section.

A trial-and-error procedure is used in the model for the simultaneous solution of the two equations. First, O_2 is assumed to be 0.0 and S_2 calculated. Then with the value for S_2 and the storage-discharge relation, O_2 is estimated. These values establish the bounds, and by trial and error the set of values for O_2 and S_2 that satisfy both equations are found.

One minor difficulty was encountered in routing flows through the storage behind roadways when a large drop in the water level can occur during the time step Δt. With inflow zero,

$$S_2 = S_1 - \frac{\Delta t}{2} (O_1 + O_2) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

Cases occur when the storage at the start of the time step, S_1, is less than $\Delta t(O_1/2)$ with O_2 equal to zero. Instead of forcing S_1 to zero, continuity was "preserved" and a small negative storage can occur at the tail of the hydrograph. This should have no effect on the flood peak.

The storage-discharge relation used by the model in routing is an array of 20 points. The values 1) can be input to the model or 2) a physical description of the storage and outlet works can be input and the model will calculate the 20 points needed for the storage-discharge relation.

Storage volumes or surface areas at selected elevations may be given. If surface areas are given, volumes will be calculated by the following equations:

$$V_1 = 0.0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (38)$$

$$V_2 = 0.5\ (E_2 - E_1)\ (A_1 + A_2) \quad \ldots\ldots\ldots\ldots\ldots \quad (39)$$

$$V_i = 0.1667\ (E_i - E_{i-2})(A_i + 4A_{i-1} + A_{i-2}) + V_{i-2} \quad \ldots\ldots\ldots \quad (40)$$

where:

V = volume in cubic feet or acre-feet

E = elevation in feet

A = surface area in square feet or acres

Flows through four types of outlet works have been coded into the model, trapezoidal broad-crested weir, circular or elliptical pipes, box culverts, or drop inlets. The equation for a trapezoidal broad-crested weir is

$$Q = 4.8\ H^{1.5}(0.67\ W + 0.533\ H\ \mathrm{Tan}(0.01744\ \theta)) \quad \ldots\ldots \quad (41)$$

where:

Q = discharge in cubic feet per second

H = head on weir in feet

W = width at the base in feet

θ = angle from the vertical in degrees

The angle must be equal to or less than 90 degrees. The trapezoidal weir can be used for roadways or emergency spillways. A triangular weir may used by setting W equal to zero and a rectangular weir by setting θ equal to zero.

The equation for circular and elliptical pipes with inlet control is

$$Q = C\ 3.14 \frac{1}{4}\ D_v D_h (64.4\ H)^{0.5} \quad \text{......} \quad (42)$$

$$\text{for } H \geq D_v$$

$$Q = C\ 3.14\ \frac{1}{4} D_v D_H \left(\frac{H}{D}\right)_v\ (64.4\ H)^{0.5} \quad \text{......} \quad (43)$$

$$\text{for } H < D_v$$

where,

D_v = vertical diameter of pipe in feet,

D_h = horizontal diameter of pipe in feet,

H = head above invert at inlet in feet,

C = loss coefficient.

The equation for C was taken from the US Geological Survey publication on culvert ratings (Bodhaine, 1968) and is

$C = 0.5$	$H/D_v < 1.5$	(44)
$C = 2.75+0.15H/D_v$	$1.5 < H/D_v < 2.0$	(45)
$C = 0.49+0.04H/D_v$	$2.0 < H/D_v < 4.0$	(46)
$C = 0.61+0.01H/D_v$	$4.0 < H/D_v < 14.0$	(47)
$C = 0.75$	$H/D_v > 14.0$	(48)

The equation for a box culvert is

$$Q = C\ D_v D_h (64.4\ H)^{0.5} \quad \text{......} \quad (49)$$

where,

D_v is the height in feet,

D_h is the width in feet,

and Q, C and H are defined above.

The equation for a drop inlet is

$$Q = C\ 3.14\ D\ H^{1.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (50)$$

where,

D = diameter of the pipe in feet,

C = loss coefficient,

H = water surface height above pipe inlet in feet.

The loss coefficient is a function of the ratio of H to D and values are interpolated from the Table 11.

UROS4 is coded to accept any number of any of the outlets discussed above. Each outlet can be at any elevation. The outlets act in parallel and not in series. Thus, a situation with a V-notched weir followed by a closed conduit could not be simulated if the control changed from one to the other for increasing water elevations. One of the two must be selected or a storage-discharge relation provided instead of the physical dimensions of the outlet.

Creation of Runoff Files

Introduction. A widespread procedure for estimating flood frequency in urban watersheds is to calculate flood peaks with the rational formula or related equations for storms of selected rainfall frequency. With this method, however, a major problem exists in estimating appropriate values for the parameters indexing infiltration or retention of precipitation in the watershed during and immediately following the storm. On any given watershed, the values of these parameters vary widely over time because infiltration rates and infiltration capacity change continually with the soil moisture content. Only in the special case of the completely impervious watershed

Table 11. Loss Coefficient for Drop Inlet

H/D	loss coefficient*
0.1	4.2
0.2	3.89
0.3	3.57
0.4	3.10
0.5	2.46
0.6	2.02
0.7	1.71
0.8	1.47
0.9	1.28
1.0	1.14
1.1	1.02
1.2	1.0
>1.2	1.0

*From Design of Small Dams, Bureau of Reclamation.

are these parameters not critical.

One approach would be to begin with the storms of selected precipitation frequency, analyze the probabilities of various combinations of precipitation and antecedent moisture, and select appropriate parameter values from the results. The difficulty is that there are so many patterns in the sequences of meteorological events and combinations of channel and land-use characteristics that the joint probability of various combinations of events is very difficult to establish. A thorough study with this approach has never been successfully performed.

The approach used in this study was to use a watershed model to continuously simulate antecedent moisture and runoff volumes from a long historic record of precipitation. From that simulation, runoff from major storms was saved for use in UROS4. Creation of such storm runoff files meant that all the combinations of influences that actually occurred during the period of record would be directly incorporated in the runoff information. Subsequent use of the resulting runoff files implicitly accounts for the critical combinations of antecedent moisture and storm intensity for any drainage area without explicit analysis of joint probability.

Continuous simulation of storm runoff, however, is not a simple process. The long term meteorological record had to be obtained and prepared in suitable form. The watershed model had to be calibrated. A great deal of expertise in model application and much computer time were required. The steps in the process are shown in Figure 1 and discussed in the sections that follow.

Selection of a Watershed Model

Simulation of streamflow from a watershed over long periods of time with short time steps has a relatively recent history. The first major effort was completed by Crawford and Linsley in 1962 with the Stanford Watershed Model. Since that time Crawford has continually made improvements while others have made modifications to the Stanford Model or created new models. In a study by Lumb (1975), a quantitative comparison was made with five watershed models on three watersheds in Georgia. For the Piedmont region, the versions of the Stanford Watershed Model responded the best. Of these, the one which closely resembles that published by Crawford and Linsley in 1966, Stanford Watershed Model IV, was selected for use in creating the runoff file because the simulated streamflow hydrographs most closely matched the measured hydrographs and it had the least number of parameters which needed calibration. Also, it was part of the Georgia Tech Watershed Simulation Model (Lumb, 1975) which has the greatest capability for data management, parameter optimization, and program printout and plotting.

Continuous streamflow simulation requires 1) a continuous record of precipitation-usually hourly increments, 2) a continuous record of pan evaporation or sufficient meterological data to determine potential evapotranspiration-usually daily values, and 3) parameters characterizing the selected drainage basin. With this information, the watershed model maintains a budget of the movement of the water through the hydrologic cycle to streamflow, evaporation, transpirstion or deep seepage to groundwater aquifers. The budgeting includes water intercepted by the vegetation, ponded at the land surface, percolated through or held by the soil, and water drained from the land and through the stream channel system. A schematic of the process as coded in the Stanford Watershed Model is given in Figure 20. The parameters in the Stanford Watershed Model are EPXM to reflect the amount

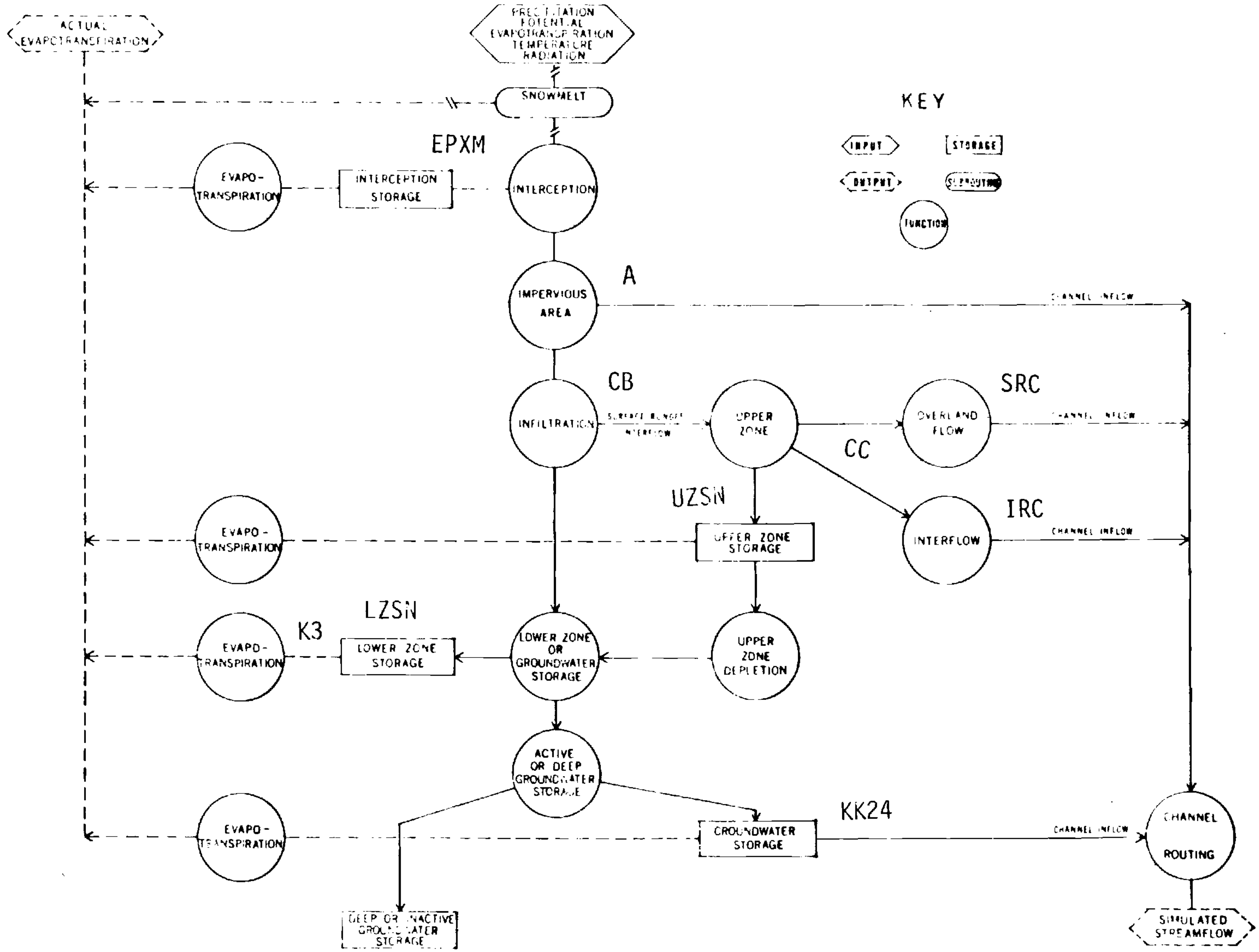


Figure 20. Flowchart Stanford Watershed Model

of water intercepted by vegetation, A to reflect the amount of rain falling on impervious surfaces and draining directly to the channel system, UZSN to reflect the amount of water ponded on the surface or absorbed immediately by the forest litter and upper inch of soil, LZSN to reflect the capacity of the watershed to store water in the soil, CB to reflect the ease with which water drains into the soil, K3 to reflect the rate vegetation transpires water to the atmosphere, and SRC, CC, IRC and KK24 to reflect rates that overland flow, interflow the baseflow drain from the watershed to the channel system. Each of these parameters is listed on Figure 20.

Details of the mathematics and logic of the Stanford Watershed Model and similar models, and streamflow simulation philosophy and purpose can be found in other reports. Though there are many publications on the subject, the following are recommended. For a good overview of watershed models, parameter estimation, and applications read:

L.D. James (1972), "Hydrologic Modeling, Parameter Estimation, and Watershed Characteristics", Journal of Hydrology, Volume 17, pp. 283-307.

For a detailed description of the Stanford Watershed Model, parameter estimation, and additional applications read:

Norman H. Crawford and Ray K. Linsley (1966), "Digital Simulation in Hydrology: Stanford Watershed Model IV", Technical Report No. 39, Dept. of Civil Engineering, Stanford University.

or

Hydrocomp International, Inc. (1972), "Hydrocomp Simulation Programming Operations Manual", Palo Alto, California.

For a detailed description of the use of the version of the Stanford Watershed Model in the Georgia Tech Watershed Simulation Program, parameter optimization, and other applications read:

Alan M. Lumb, et al. (1975), "GTWS: Georgia Tech Watershed Simulation Model", Environmental Resources Center Report No. ERC-0175, Georgia Institute of Technology, Atlanta, Georgia.

For a quantitative comparison of five watershed models on drainage areas in Georgia read:

Alan M. Lumb and Timothy D. Hassett (1975), "Comparison of Georgia Tech, Kansas, Kentucky, Stanford and TVA Watershed Models in Georgia", Environmental Resources Center Report No. ERC-0275, Georgia Institute of Technology, Atlanta, Georgia.

Parameter Calibration

Three time series of data are required for calibration of a watershed model; streamflow, hourly rainfall, and pan evaporation. Locations of streamflow and precipitation data in the general area of DeKalb County are shown on Figure 21. From this information, four watersheds were selected: 1) Camp Creek near Fayetteville, U.S. Geological Survey Gaging Station No. 2-3443, 2) North Fork Camp Creek, Station No. 2-3371, 3) a tributary to South Utoy Creek, Station No. 2-3367, and 4) South River at East Point, Station No. 2-2036. Each of the gaging stations are near the Atlanta Airport where a National Weather Service first order station is located. This station has a continuous recording raingage and also measures wind speed, humidity, solar radiation and temperature. No streamflow data were available for streams in DeKalb County having a period of record as long as two years, the minimum required for calibration.

Four stream gaging stations with sufficiently long records existed in Gwinnett County. These watersheds, however, are rural,

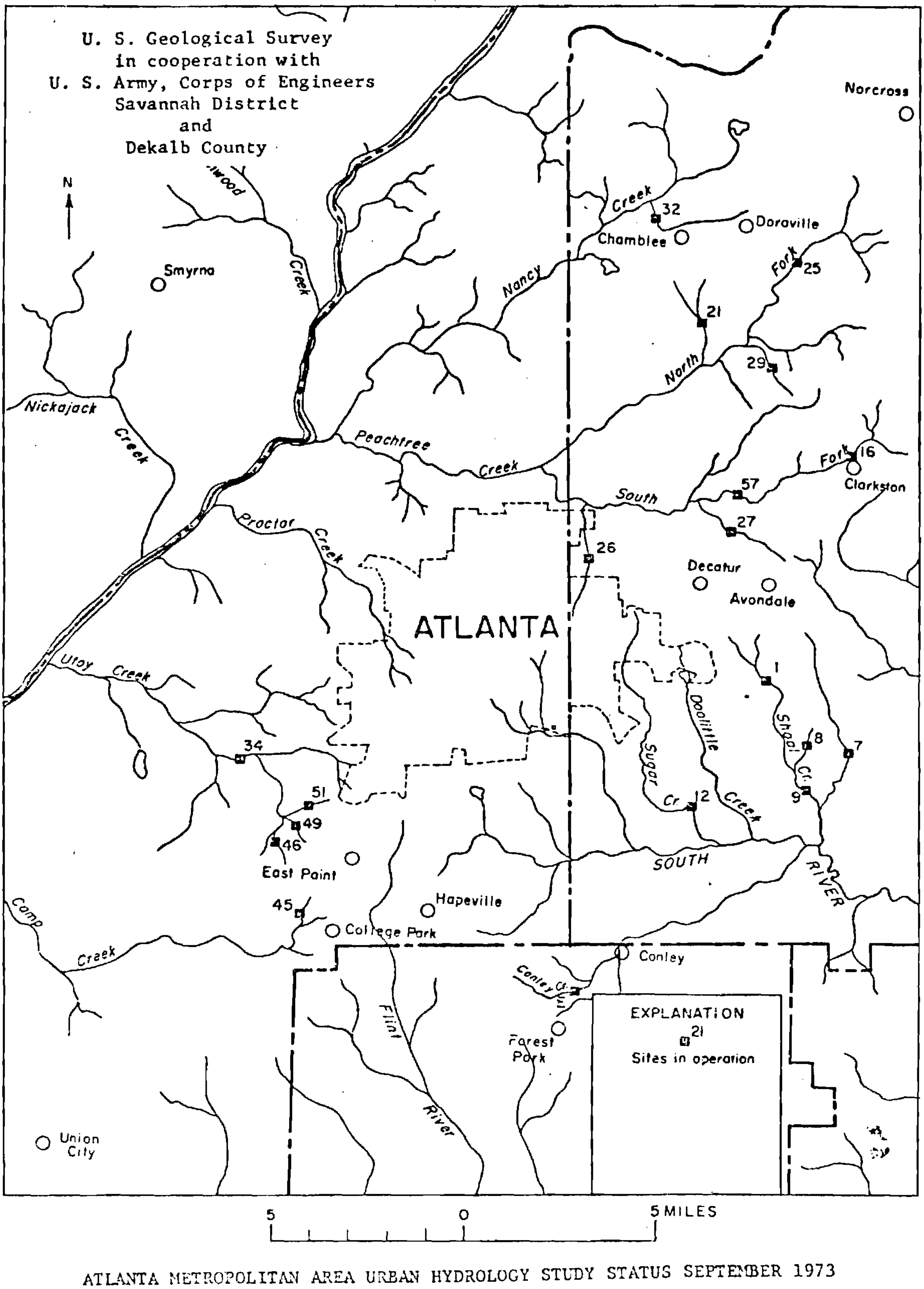


Figure 21. Map Showing Location of Hydrologic Data in and Around DeKalb County

do not have good rainfall data, and are of the same general soils as the four watersheds around the Atlanta Airport. Thus, the information gained from calibration of these watersheds would not justify the added expense.

The data files established for the calibration process are listed in Table 12. The values for daily potential evapotranspiration were computed from daily data on wind speed, temperature, humidity, and solar radiation. All were measured directly or determined for the National Weather Service first order station at the Atlanta Airport.

Calibration is a process of adjusting the values of the parameters in the model to represent the characteristics of the local watershed and thus best match the computed streamflow with the measured streamflow. Calibration can be done with a series of trials followed by checks and adjustments based on the judgment of the modeller or with a computerized parameter optimization routine. Both methods were used for the calibration of the Stanford Watershed Model parameters for the four watersheds. Initial calibration was conducted as a class project and graduate student research. Parameters from these efforts were refined with computerized optimization runs for each watershed. From these results, the less sensitive parameters were averaged over all years and all watersheds and these average values were then used for further calibration of the more significant parameters. The less sensitive parameters and the average values are:

EPXM = 0.05	IRC = 0.025
SRC = 0.8	KK24 = 0.995
CC = 0.75	K3 = 0.05

Table 12. Data Base for Watershed Model Calibration

Data Type	Source	Gage No.	Period of Record
Hourly Precipitation (Atlanta AP)	National Weather Service	0451	Oct. 1, 1960- Sept. 30, 1968
Calculated Potential Evapotranspiration (Atlanta AP)	National Weather Service	0451	Oct. 1, 1960- Sept. 30, 1968
Daily Streamflow (Camp Creek)	U.S. Geological Survey	2-3443	Oct. 1, 1960- Sept. 30, 1968
Daily Streamflow (North Fork Camp Creek)	U.S. Geological Survey	2-3371	Oct. 1, 1963- Sept. 30, 1968
Daily Streamflow (tributary to South Utoy Creek)	U.S. Geological Survey	2-3367	Oct. 1, 1963- Sept. 30, 1968
Daily Streamflow (South River)	U.S. Geological Survey	2-2036	Oct. 1, 1963- Sept. 30, 1968

Four of the more sensitive parameters, A, UZSN, LZSN and CB required additional study. These four parameters were optimized for each of several years on each watershed, and the estimates of UZSN and LZSN were averaged. Next, CB and A were optimized for each year, and the results were averaged as shown on Table 13. It was extremely encouraging to see the model indirectly estimate fractions of impervious area from rainfall data from a gage outside the watershed and from streamflow records which were only rated fair. These indirect estimates are about as accurate as could be measured from aerial photographs. This is very strong substantiation that the model is properly indicating the increase in the volume of runoff from urban development.

The values of CB for the South River on Table 13 were high because diversions into that watershed made the baseflow higher than normal. To compensate, the computerized optimization increased the value of CB to simulate more baseflow. Differences in CB among the other watersheds were not judged to be significant. Thus, the value for the South River was excluded and CB was averaged over the other watersheds.

The entire set of values was checked with a final calibration involving trial runs on all watersheds with several values of UZSN and LZSN. For these trial runs, the ratio found for UZSN to LZSN was held constant. The final values for the three parameters are

$$UZSN = 0.6$$

$$LZSN = 6.0$$

$$CB = 0.16$$

These values were used together with the six values for the less sensitive parameters tabulated above to simulate flows based on the soils in DeKalb County as described below. In developing the

Table 13. Parameter Values

Watershed	Water Year	A	CB
South River @ East Point	1964	.267	.217
	1965	.297	.377
	1966	.239	.331
	1967	.267	.490
	1968	.245	.266
	Average	.263	.336
North Fork Camp Creek	1964	.112	.152
	1965	.158	.173
	1966	.171	.216
	1967	.136	.124
	1968	.170	.113
	Average	.149	.156
South Utoy	1964	.108	.173
	1965	.156	.173
	1966	.138	.140
	1967	.119	.115
	1968	.089	.124
	Average	.122	.145
Camp Creek	1961	.039	.154
	1962	.012	.195
	1963	.017	.179
	1968	.040	.171
	Average	.027	.175

runoff file, A was set to zero for simulating runoff from a given soil and set to unity for impervious area.

Soils Data and Parameter Values

The next step was an analysis of the soils data for DeKalb County 1) to see how well the range in soil conditions found in the calibrated watersheds represent the soil conditions found in the county and 2) to estimate parameter values for soils for which no data were available for direct calibration. The only two parameters for which a close enough relationship with soil characteristics has been established for this task are CB with permeability and LZSN with moisture storage capacity (James, 1972).

Analyzing and mapping and soils data in DeKalb County is currently underway by the Soil Conservation Service. Preliminary data and a generalized soils map for the County is available and was used for this study.

The characteristic of the soil that enables it to transmit water is called permeability. The seven classes of soil permeability used by the Soil Conservation Service are listed in Table 14. Another method of classification is into the four hydrologic soil groups that the Soil Conservation Service uses to indicate runoff potential. These are listed in Table 15. Twenty-one soils found in DeKalb County are listed in Table 16 with their average permeability, available water capacity, and hydrologic soil group. The available water capacity is the volume of water that can be stored in the hydrologically active upper soil profile.

These 21 soils are grouped into 18 soil associations and three other catagories listed in Table 17. Table 18 lists the weighted permeabilities and hydrologic group for each soil association, and

Table 14. Classes of Soil Permeability

Permeability class	Numerical Range (inches per hour)	Average Rate (inches per hour)
Very slow	Less than 0.06	-
Slow	0.06 - 0.2	0.13
Moderately slow	0.2 - 0.6	0.40
Moderate	0.6 - 2.0	1.30
Moderately rapid	2.0 - 6.0	4.00
Rapid	6.0 -20.0	13.00
Very rapid	More than 20	-

Table 15. Hydrologic Soil Group

Hydrologic Soil Group	Description
A	Soil with lowest runoff potential. Deep sands with very little silt and clay.
B	Mostly sandy soils less deep than A.
C	Shallow soils and soils containing considerable clay.
D	Soil with highest runoff potential. Clay soils and shallow soils with impermeable subhorizons.

Table 16. Soils in DeKalb County

Soil	Permeability (inches/hour)	Available Water Capacity (inches)	Hydrologic Soil Group
Alluvial	4.0	5.85	B
Altavista	2.0	8.13	C
Appling	1.8	7.24	B
Cecil	1.8	8.06	B
Chewaela	1.3	8.20	C
Congaree	1.9	7.64	B
Davidson	1.8	5.20	B
Gwinnett	1.3	7.90	B
Iredell	0.2	4.20	D
Linker	2.2	4.68	B
Louisa	2.2	2.64	B
Louisburg	13.0	3.16	B
Madison	1.3	5.10	B
Mecklenburg	0.2	5.18	C
Musella	2.2	4.05	B
Pacolet	1.8	6.25	B
Red Bay	1.3	6.49	B
Wedowee	2.2	5.66	B
Wehadkee	1.8	4.44	D
Wickham	1.8	6.83	B
Wilkes	0.6	3.41	C

TABLE 17. SOIL ASSOCIATIONS

SOIL #	SOIL ASSOCIATION	SOIL #	% COMPONENTS 1st	2nd	3rd
1	ALLUVIAL LAND-CHEWACLA-WEHADKEE	1	60	20	10
1A	CONGAREE-CHEWACLA-WEHADKEE	1A	50	20	15
2	WILKES-IREDELL-MECKLENBURG	2	50	20	20
3	MADISON-LOUISA-PACOLET	3	45	25	15
4	APPLING-CECIL-MADISON	4	45	30	10
5	MADISON-PACOLET-MUSELLA	5	40	25	20
6	GWINNETT-DAVIDSON-MUSELLA	6	45	25	15
7	GWINNETT-DAVIDSON-MUSELLA	7	45	25	15
8	LOUISBURG-WEDOWEE-PACOLET	8	40	30	20
9	APPLING-LOUISBURG-PACOLET	9	50	25	15
10	MADISON-PACOLET-GWINNETT	10	40	30	20
11	LINKER-LOUISBURG-MISELLA	11	60	15	10
12	PACOLET-GWINNETT-LOUISBURG	12	40	35	10
13	WICKHAM-ALTAVISTA-RED-BAY	13	65	20	10
14	APPLING-PACOLET-LOUISBURG	14	60	20	10
15	WILKES-GWINNETT-MUSELLA	15	50	30	10
16	APPLING-PACOLET-GWINNETT	16	40	35	10
17	LOUISBURG-PACOLET-SEDOWEE	17	37	35	21
18	ROCK OUTCROP	18	-	-	-
19	MADE LAND	19	-	-	-
20	UNCLASSIFIED	20	-	-	-

TABLE 18. PERMEABILITY AND HYDROLOGIC GROUP OF THE SOIL ASSOCIATIONS

SOIL #	SOIL ASSOCIATION	% SLOPE	PERMEABILITY 1st	PERMEABILITY 2nd	PERMEABILITY 3rd	PERMEABILITY wt ave	HYDROLOGIC GROUP 1st	HYDROLOGIC GROUP 2nd	HYDROLOGIC GROUP 3rd	HYDROLOGIC GROUP wt ave
1	ALLUVIAL LAND-CHEWACLA-WEHADKEE	0-2	MR	M	M	3.16	B	C	D	B
1A	CONGAREE-CHEWACLA-WEHADKEE	0-2	M	M	M	1.74	B	C	D	B
2	WILKES-IREDELL-MECKLENBURG	2-10	MS	S	S	.42	C	D	C	C
3	MADISON-LOUISA-PACOLET	10-45	M	MR	M	1.65	B	B	B	B
4	APPLING-CECIL-MADISON	2-10	M	M	M	1.74	B	B	B	B
5	MADISON-PACOLET-GWINNETT	2-10	M	M	M	1.45	B	B	B	B
6	GWINNETT-PACOLET-MUSELLA	10-45	M	M	MR	1.61	B	B	B	B
7	GWINNETT-DAVIDSON-MUSELLA	2-10	M	M	MR	1.61	B	B	B	B
8	LOUISBURG-WEDOWEE-PACOLET	10-45	R	M	M	6.91	B	B	B	B
9	APPLING-LOUISBURG-PACOLET	2-10	M	R	M	4.91	B	B	B	B
10	MADISON-PACOLET-GWINNETT	10-25	M	M	M	1.47	B	B	B	B
11	LINKER-LOUISBURG-MUSELLA	10-60	MR	R	MR	4.11	B	B	B	B
12	PACOLET-GWINNETT-LOUISBURG	10-45	M	M	R	2.91	B	B	B	B
13	WICKHAM-ALTAVISTA-RED BAY	2-10	M	M	M	1.79	B	C	B	B
14	APPLING-PACOLET-LOUISBURG	2-10	M	M	R	3.04	B	B	B	B
15	WILKES-GWINNETT-MUSELLA	10-45	MS	M	MR	1.01	C	B	B	C
16	APPLING-PACOLET-GWINNETT	2-10	M	M	M	1.84	B	B	B	B
17	LOUISBURG-PACOLET-WEDOWEE	10-45	R	M	M	6.35	B	B	B	B
18	ROCK OUTCROP	NOT GIVEN	VS			0.0	D			D
19	MADE LAND	NOT GIVEN								
20	UNCLASSIFIED									

Table 19 lists the available water capacities. From the permeability information on Table 18, the soil associations were placed in one of four permeability catagories as indicated in Table 20. Category 18 (rock outcrop) is treated the same as man-made impervious surfaces.

To determine the variability of basin average permeability (related to CB) and available soil water capacity (related to LZSN) for drainage areas in DeKalb County, values were estimated from the soils data for 1) the drainage areas above the recently installed U.S. Geological Survey gages in DeKalb County and 2) a tributary to South Utoy Creek and North Fork Camp Creek in Fulton County. A map showing the location of each gage by number is given in Figure 22. Table 21 lists the values for permeability and available water capacity. Values for South River could not be calculated because the soils were not classified on the soils maps. The range in the calculated basin average permeabilities was quite small, and the watersheds used for model calibration encompass that range. Since the extremes in the permeability range are from the two calibrated watersheds in Fulton County and the calibration established the same parameter estimates for the two, the calibrated values for the watershed model parameters were judged as also valid for the gaged watersheds in DeKalb County. The available water capacities vary over an even smaller range adding validity to the extension of the values to DeKalb County.

The next step was to attempt to estimate watershed model parameters for soils not represented by the calibrated watersheds. Since the soils in the calibrated watershed represent 90 per cent of DeKalb County in having moderate to moderately rapid permeabilities and in having similar moisture storage capacities, the calibrated values for the parameters can be used directly for most County soils. For the small number of soils with substantially different characteristics in the southwest and

TABLE 19. AVAILABLE WATER CAPACIT

SOIL#	SOIL ASSOCIATION			CAPACITY rd	Weighted Average
1	ALLUVIAL LAND-CHEWACLA-WEHDKEE			.66	6.36
1A	CONGAREE-CHEWACLA-WEHADKEE			.66	7.42
2	WILKES-IREDELL-MECKLENBURG			.18	3.98
3	MADISON-LOUISA-PACOLET			.25	4.58
4	APPLING-CECIL-MADISON			10	7.28
5	MADISON-PACOLET-GWINNETT			90	6.09
6	GWINNETT-PACOLET-MUSELLA			05	6.74
7	GWINNETT-DAVIDSON-MUSELLA			05	6.42
8	LOUISBURG-WEDOWEE-PACOLET			25	6.68
9	APPLING-LOUISBURG-PACOLET			.25	5.94
10	MADISON-PACOLET-GWINNETT	5.10	6.25	7.90	6.11
11	LINKER-LOUISBURG-MUSELLA	4.68	3.16	4.05	4.34
12	PACOLET-GWINNETT-LOUISBURG	6.25	7.90	3.16	6.56
13	WICKHAM-ALTAVISTA-RED BAY	6.83	8.13	6.49	7.06
14	APPLING-PACOLET-LOUISBURG	7.24	6.25	3.16	6.57
15	WILKES-GWINNETT-MUSELLA	3.41	7.90	4.05	4.98
16	APPLING-PACOLET-GWINNETT	7.24	6.25	7.90	6.91
17	LOUISBURG-PACOLET-WEDOWEE	3.16	6.25	5.66	4.89
18	ROCK OUTCROP	-	-	-	-
19	MADE LAND	-	-	-	-
20	UNCLASSIFIED	-	-	-	-

Table 20. Four Soil Permeability Categories

Category	Permeability	Soil Associations
1	Rapid (R)	8,17
2	Moderate (M) to Moderately Rapid (MR)	1A, 3, 4, 5, 6, 7, 10, ,3, ,5, 16 1, 9, 11, 12, 14
3	Moderately Slow (MS)	2
4	Very Slow (VS) and Impermeable	18

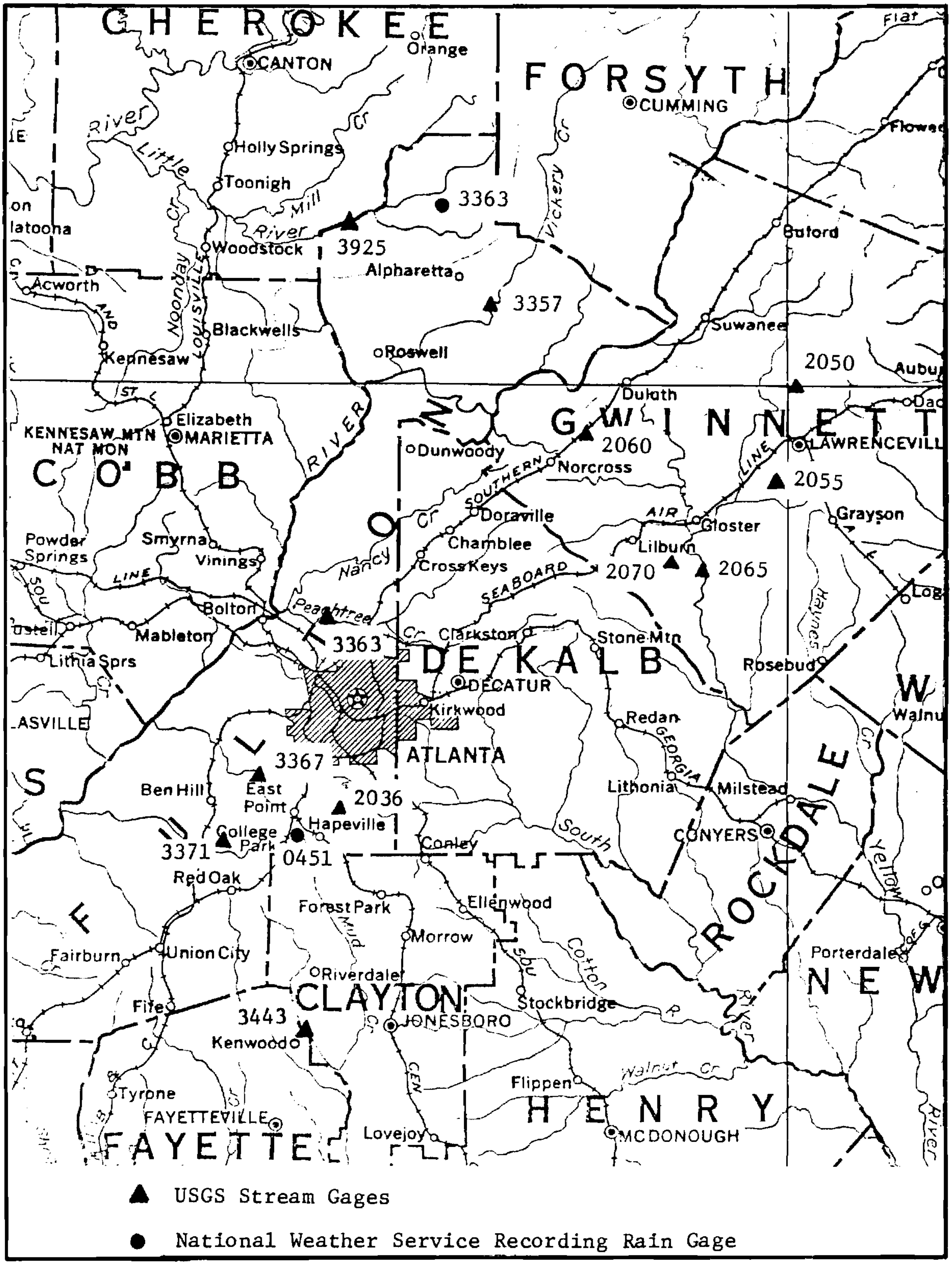


Figure 22. Map Showing Location of US Geological Survey Gages

TABLE 21. Permeability and Available Water Capacity for Selected Drainage Areas

DRAINAGE AREA* SITE NUMBER	PERMEABILITY (inches/hour)	AVAILABLE WATER Capacity (inches)
1	1.61	6.12
7	2.01	6.24
8	2.12	6.27
9	1.83	6.19
12	1.72	6.14
16	1.69	6.14
21	2.31	6.34
26	1.82	6.16
27	1.61	5.99
29	2.09	6.30
32	2.21	6.31
57	1.70	6.14
2-3371	2.37	6.36
2-3367	1.53	6.11

*Maps on Figures 21 and 22 show locations.

eastern portions of the county, adjustments to parameters CB, LZSN and UZSN were needed. The adjustments were made by multiplying CB by ratios of the soil permeabilities and LZSN by the ratio of available water capacities for both soil categories. UZSN was maintained as one tenth of LZSN. The result is shown in Table 22.

Precipitation Data Analysis. The next step as shown on Figure 1 was the development of a file of 72 years of hourly precipitation data needed for the watershed model to create the runoff files. Atlanta rainfall data are available in three forms: 1) 5-minute values of rainfall for the largest storms each year for 72 years are on a magnetic tape created by the U.S. Geological Survey, 2) hourly values of rainfall for 20 years are on data cards from the National Weather Record Center in Asheville, North Carolina, and 3) daily values of rainfall for the 72-year period from 1898 to 1970 are on magnetic tape created by the U.S. Geological Survey. In all cases the data were measured at the first order weather station in Atlanta.

Simulation with the watershed model can use either hourly or 5-minute data. When 5-minute data is not available, the watershed model divides the hourly values into twelve equal parts for each 5-minute time step used in simulation. This division reduces simulated peaks from small watersheds but has little effect on the simulated antecedent moisture needed for events simulated with 5-minute rainfall. Division of daily precipitation into 24 equal hourly amounts does, however, cause significant upward bias in antecedent moisture estimates. The resulting long duration, low intensity rainfall generates too much infiltration and too high a soil moisture whereas the actual unevenly distributed rain results in more runoff and less soil moisture. Thus, a statistical analysis was made on the available 20 years of hourly

TABLE 22. Final Parameter Values

Parameter	Soils of Moderately Slow Permeabilities	Soils of Moderate to Moderately Rapid Permeability	Soils of Rapid Permeability
UZSN	0.4	0.6	0.5
LZSN	4.0	6.0	5.0
CB	0.05	0.16	0.48
EPXM	0.05	0.05	0.05
CC	0.75	0.75	0.75
SRC	0.8	0.8	0.8
IRC	0.025	0.025	0.025
KK24	0.995	0.995	0.995
K3	0.05	0.05	0.05

data to determine how best to divide daily rainfall for the entire 72 years into hourly amounts. The following information was tablulated from the 20 years of hourly rainfall data for each month of the year and for five ranges of daily rainfall:

1) total number of rainfall days,
2) number of hours with rain for each rainfall day, and
3) average ratio of 2nd, 3rd, 4th etc. highest to the highest hourly amount for each duration.

The five ranges selected were 0.01 to 0.25 inches, 0.26 to 0.50 inches, 0.51 to 1.00 inches, 1.01 to 2.00 inches and greater than 2.00 inches. The tabulated statistics on duration of storms and distribution of amount were stored on a file to be used to synthesize hourly rainfall from the 72 years of daily values, as follows:

1) Given the month and a daily rainfall amount, determine the range and locate the appropriate file for the probability histogram of rainfall duration.
2) With a random number and a probability histogram select a rainfall duration and use the associated average distribution of hourly values.
3) With the duration and distribution of hourly values, divide the daily rainfall into hourly values and order those hours in the predetermined pattern (Table 23).

Generation of hourly values from daily values was tested by comparison with the 20-year period for which the actual hourly values were available. Following the synthesis of a 20-year hourly precipitation record, both records were used with the watershed model to simulate 20 years of streamflow for a watershed with 0.8 square miles and no impervious area. Table 24 lists the maximum hourly flow for each year from

TABLE 23. Hourly Pattern for Distribution of Daily Rainfall

Storm Duration (hours)	Pattern of Ranked Hourly Rainfalls (1 represents highest hourly amount)
1	1
2	1, 2
3	3, 1, 2
4	3, 1, 2, 4
5	5, 3, 1, 2, 4
6	5, 3, 1, 2, 4, 6
7-12	7-12, 5, 3, 1, 2, 4, 6
13-24	7-12, 5, 3, 1, 2, 4, 6, 13-14

Table 24. Annual Maximum Hourly Flow

for Actual and Synthesized Hourly Rainfall

	Actual Hourly Rainfall		Synthesized Hourly Rainfall	
Water Year	Maximum Mean Hourly Flow (cfs)	Date	Maximum Mean Hourly Flow (cfs)	Date
1949	178	11-28-48	178	11-28-48
1950	3	7-28-50	3	3-13-50 *
1951	24	2-20-51	22	2-20-51
1952	102	3-10-52	121	3-23-52 *
1953	106	3-4-53 *	61	5-6-53 *
1954	84	1-21-54	92	1-21-54
1955	73	2-6-55	95	2-6-55
1956	100	3-16-56	72	3-16-56
1957	121	4-5-57	119	4-5-57
1958	82	2-27-58 *	70	2-6-58
1959	34	3-11-59 *	64	3-11-59 *
1960	125	4-3-60	141	4-3-60
1961	335	2-25-61	333	2-25-61
1962	128	2-22-62	131	2-22-62
1963	98	3-12-63 *	141	3-12-63 *
1964	37	1-25-64 *	143	1-24-64 *
1965	124	1-23-65	126	1-23-65
1966	134	2-13-66	162	2-13-66
1967	172	3-10-67	170	3-10-67
1968	179	12-9-67 *	119	3-12-68 *
1969	121	5-8-69	132	4-18-69
mean	117.2		118.9	

* Storms when 5-minute data was not available on file.

the two rainfall records. The means of the two data series were within 2%. As indicated by the asterisks, about one-third of the maximum flows were from storms which were not included on the 5-minute data file. To check on the effect of simulating an annual flood series without using these storms, another 20-year simulation was run just to get flood peaks on all storms with 5-minute data. From this data series the maximum mean hourly flows each year were selected and are shown on Table 25. All three maximum hourly flow series are plotted on log-probability paper in Figure 23. The discrepancies are mostly at the lower frequencies and too small to warrant further adjustment. Thus 1) the use of hourly data generated from daily data to simulate antecedent moisture conditions was not found to bias the flood peaks simulated from storms with 5-minute data, and 2) the exclusion of simulated flood peaks from storms for which 5-minute data were not available was also not found to cause significant bias in the results.

Selection of a Representative Period. From this tested and calibrated method for synthesis of hourly rainfall from daily rainfall, a 72-year precipitation record was created and stored on file for use by the watershed simulation model. While 72 years of precipitation data are now available for flow simulation and flood frequency analysis, it would be costly to have to simulate such a long record repeatedly. Thus the next step was to determine if a shorter period could be found that would give approximately equivalent results. The search was for some subset of the total record having a mean and variance of the annual flood peaks about the same as those for the entire record.

Because the mean and variance of the annual flood peaks depend on watershed characteristics, the search was based on a hypothetical watershed fairly typical of those to which the model would be applied

Table 25. Annual Maximum Hourly Flow for Actual Hourly Rainfall for Days with 5-Minute Data

Water Year	Maximum Mean Hourly Flow (cfs)	Date
1949	178	11-28-48
1950	3	7-28-50
1951	24	2-20-51
1952	102	3-10-52
1953	43	4-12-53
1954	84	1-21-54
1955	73	2-6-55
1956	100	3-16-56
1957	121	4-5-57
1958	70	2-6-58
1959	22	3-5-59
1960	125	4-3-60
1961	335	2-5-61
1962	128	2-22-62
1963	87	7-24-63
1964	132	4-6-64
1965	124	1-23-65
1966	134	2-13-66
1967	172	3-10-67
1968	109	3-12-68
1969	121	5-8-69
Mean	108.9	

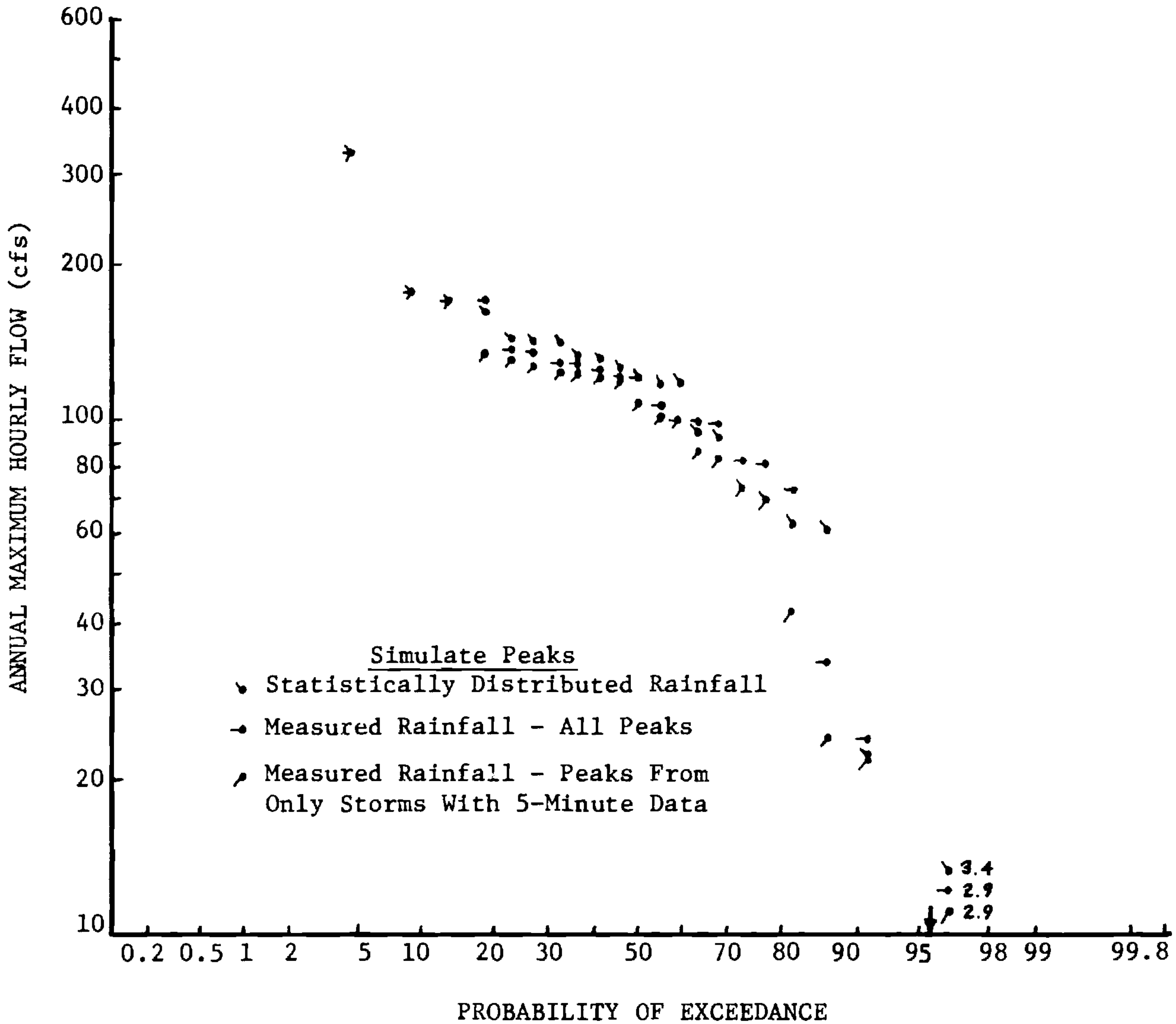


FIGURE 23. LOG-PROBABILITY PLOT ANNUAL MAXIMUM MEAN HOURLY FLOW

in practice. The hypothetical watershed had 1.5 square miles, 25 percent impervious area, and the optimized parameter values for the calibrated watersheds. The 72-year file of precipitation data was used to simulate 72 years of annual flood peaks, and the mean and standard deviation of this series were calculated.

In the search for a shorter representative period, the mean and standard deviation were also calculated for running periods of 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, and 65 years beginning with every one of the 72 years. When the length of a running period exceeded the remaining years in the 72-year record, values at the beginning of the period of record were used to complete the required number. For example, the 30-year running period beginning in 1953 included 1953 through 1970 and 1899 through 1910. In a like manner, the mean and the standard deviation of the annual series of 72 maximum 5-minute rainfalls were also calculated to represent impervious conditions. The mean and the standard deviations of the flood peaks and the means and the standard deviations of the largest 5-minute rainfalls in each year were calculated for each running period. The former provides results typical of a urban watershed, and the latter provides results typical of a small completely impervious area.

For the 72 running periods of a given length (e.g., 5 years) the standard deviations varied between a minimum and a maximum value. The ranges for the various lengths of running period are plotted on Figures 24 and 25. From these graphs, the range is seen to decrease rapidly with the length of the running period until a 25-year period is reached and then to decrease more slowly as one goes to longer periods. Thus 25 years was selected as an appropriate length for the representative period.

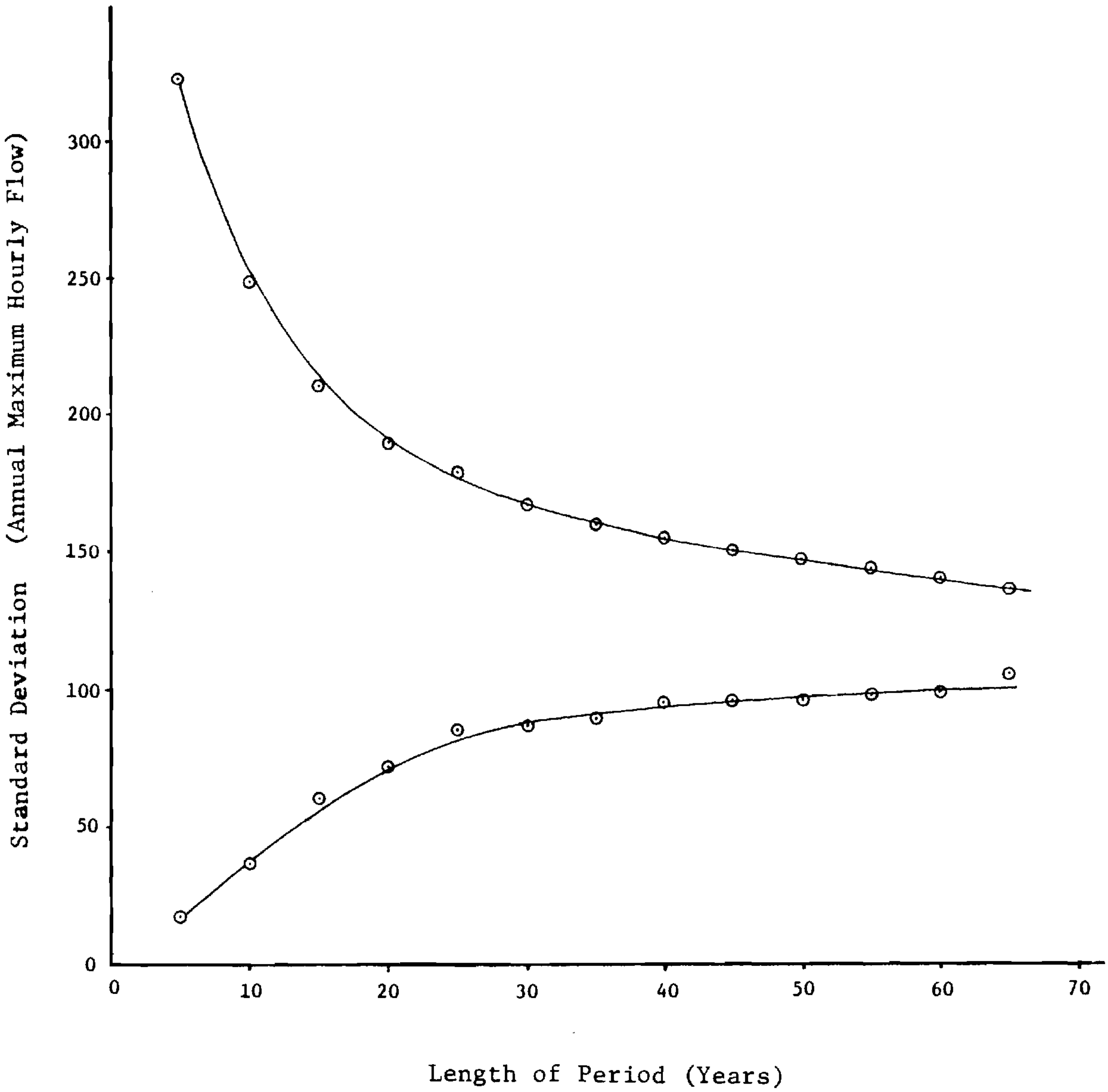


Figure 24. Plot of Range of Standard Deviation in Maximum Hourly Flow

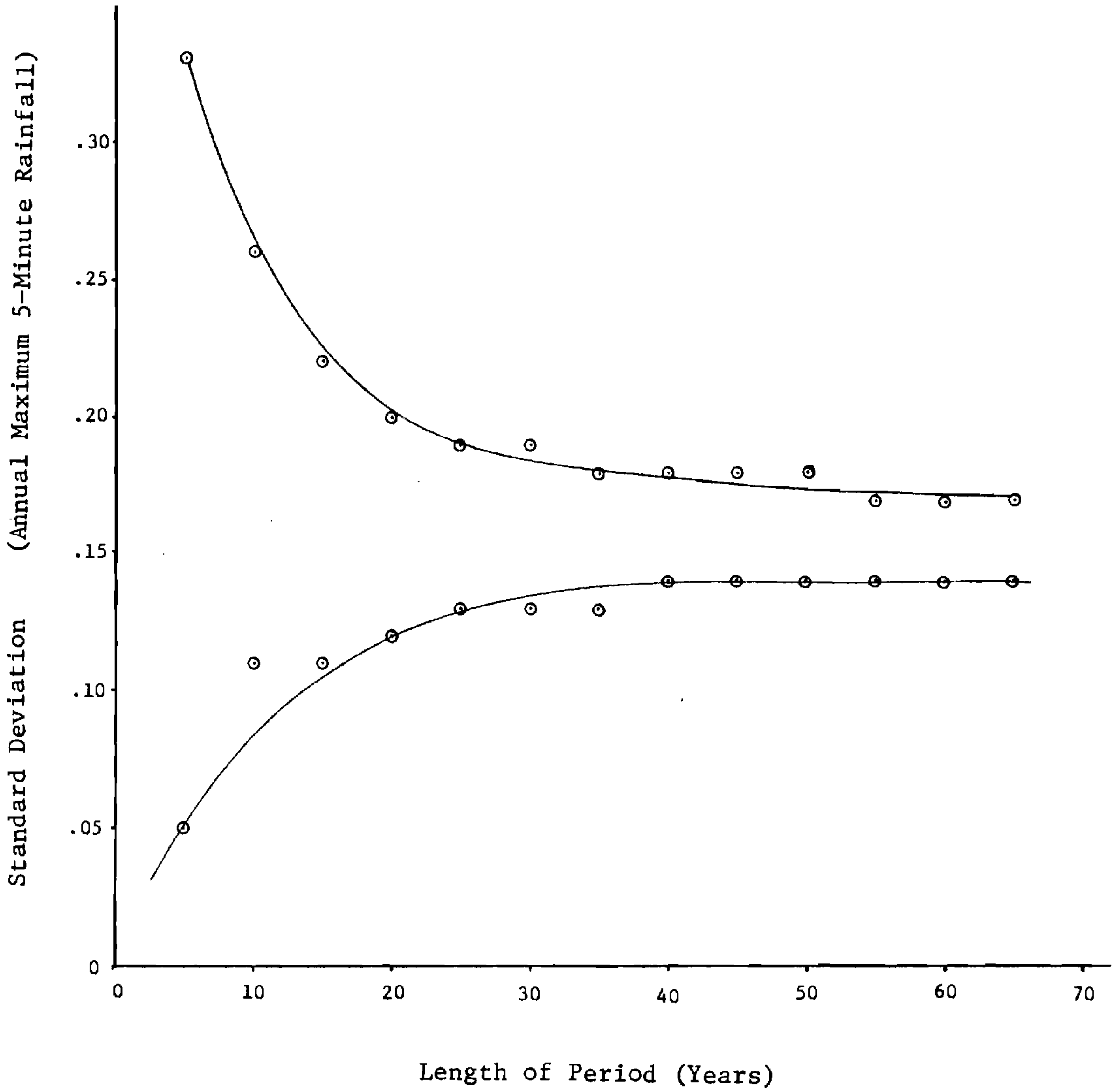


Figure 25. Plot of Range of Standard Deviations in Maximum 5-Minute Rainfall

The next step was to find the specific 25 years that best represent the 72-year total period. The period from 1918 through 1942 was selected as having a mean and standard deviation closest to those for the entire record. Table 26 lists the statistics for the two periods. To find a more representative period, many additional years were required. The tradeoffs in the period selected were between added accuracy in estimation of flow frequency statistics and added computer time and cost for simulation. As advances in computer technology reduce costs, a longer period could be justified. The values from which the 1918 through 1942 period was selected are given in Tables 27 through 30. The 72-year annual series of hourly flow and 5-minute rain are listed in Table 31.

Generation of Runoff Files. With the 25-year period from 1918 through 1942 selected and the parameter sets for the four soil types, a simulation run was made with the watershed model to generate 5-minute runoff values. The 5-minute runoff values for the days that 5-minute rainfall was available were written on a file for further analysis. Each value included all direct runoff plus one-twelfth of the interflow and baseflow that were computed for the hour.

Since the routing model can only route storms for 120 time steps, the critical 120-step period for each storm on the runoff file was determined. In doing this, 1-minute and 15-minute time steps were also considered. One minute runoff values were determined by dividing the 5-minute values by 5, and the 15-minute values were estimated by adding 3 consecutive 5-minute values. A computer program was written to put the 1-, 5-and 15-minute runoff values in the format needed by the routing model. The critical period for a given storm was selected as the 120-time-step period which had the maximum total runoff. The results of the selection were checked with the original runoff file

Table 26. Statistics for Total Period and Representative Period

Data Base	Period	Mean	Standard Deviation
Hourly Flow	1899-1970	248.	130.
Hourly Flow	1918-1942	249.	122.
5-Minute Rain	1899-1970	0.37	0.16
5-Minute Rain	1918-1942	0.40	0.16

Table 27. Means - Maximum

Hourly Flows

Year Began	Number of Years for Statistic 5	10	15	20	25	30	35	40	45	50	55	60	65
1899	37.50	79.36	191.58	170.54	169.43	159.80	153.54	149.67	144.04	142.39	138.30	134.68	134.65
1900	61.97	81.43	191.60	172.98	171.04	159.80	155.42	149.88	144.10	142.38	138.66	136.20	134.63
1901	77.83	86.56	192.23	187.89	171.34	162.35	157.87	149.87	144.06	143.61	139.25	136.15	134.72
1902	75.08	05.78	192.53	187.77	171.19	165.47	157.70	150.23	143.90	143.74	140.15	139.97	134.59
1903	78.70	237.00	192.88	187.74	171.65	165.92	158.14	152.70	144.19	144.,2	140.23	139.99	135.21
1904	100.19	237.51	197.63	189.53	174.89	165.67	159.78	152.52	149.82	144. 9	140.37	139.88	135.15
1905	104.99	233.45	198.55	189.22	172.80	166.43	158.87	151.51	148.71	144.31	141.30	139.19	135.17
1906	101.54	226.91	210.40	185.08	172.35	166.93	156.58	149.41	148.10	143.35	139.92	138.00	134.18
1907	104.34	227.07	210.39	185.09	176.62	166.85	157.23	149.32	148.39	144.,0	143.91	137.91	134.11
1908	319.02	220.54	205.24	182.07	174.52	165.02	157.61	147.64	147.56	143.24	142.70	137.32	132.98
1909	316.48	230.89	206.96	185.40	174.36	166.55	157.52	153.52	148.10	143.37	142.51	137.22	132.94
1910	320.87	236.21	210.40	185.49	177.03	166.84	157.56	153.48	148.51	145.86	142.39	137.80	132.86
1911	307.43	248.22	204.81	185.56	178.07	164.48	155.46	153.13	147.73	143. 7	141.31	136.95	131.83
1912	277.81	230.33	198.02	186.92	175.17	162.79	153.01	151.35	147.43	146._3	139.90	135.58	131.29
1913	61.54	134.84	119.83	124.14	119.71	118.00	109.69	115.07	112.33	115.14	110.87	107.65	106.08
1914	84.46	139.36	127.13	122.66	121.96	116.96	118.72	115.07	111.76	114.37	110.22	107.23	105.60
1915	69.65	144.58	127.02	126.42	122.32	117.07	119.72	115.63	114.05	114.37	111.31	107.29	106.92
1916	195.85	143.75	132.67	131.01	122.07	116.73	120.44	116.37	113.97	114. 0	111.56	107.22	107.01
1917	194.19	142.85	140.06	130.09	122.21	116.10	120.43	117.45	119.99	113.98	111.19	107.67	105.82
1918	192.60	143.70	140.68	131.16	127.16	116.49	121.36	117.45	112.90	114.91	111.16	109.16	107.04
1919	166.74	137.78	133.10	129.95	122.47	122.91	118.84	115.11	117.49	112.67	109.17	107.38	107.49
1920	157.37	123.27	133.63	126.71	119.36	119.76	117.32	116.12	115.91	112. 4	107.73	107.53	133.01
1921	88.00	101.46	109.90	103.44	100.23	108.23	104.02	103.19	105.11	102.76	98.49	98.05	127.25
1922	86.28	117.25	109.63	104.29	99.96	108.64	106.57	111.14	104.96	102.65	99.29	99.03	127.16
1923	96.71	118.31	111.20	111.82	100.68	109.97	106.42	110.96	106.09	102.60	100.06	100.24	127.22
1924	113.42	106.22	110.93	106.26	112.21	107.44	103.19	108.07	103.55	100.61	98.96	99.50	126.17
1925	91.70	109.85	109.06	104.31	110.11	106.77	105.17	107.22	104.22	99. 5	99.71	129.82	125.68
1926	121.98	112.86	107.40	102.93	112.49	107.09	104.29	106.68	104.11	99.37	99.60	130.06	126.96
1927	143.67	113.53	108.00	102.86	112.95	108.96	113.84	106.67	104.11	100. 7	99.92	130.02	127.83
1928	144.05	119.03	118.92	103.69	114.00	108.82	113.64	108.01	104.01	101.98	101.07	129.98	134.12
1929	85.47	109.12	105.12	114.30	107.45	101.70	107.86	102.91	99.06	97.04	98.47	127.76	131.96
1930	75.21	107.39	104.59	114.26	107.88	104.38	107.77	104.72	99.78	99.31	132.38	127.81	132.02
1931	79.48	104.07	99.90	113.33	106.24	102.65	105.96	103.25	98.05	98.36	131.79	128.30	132.22
1932	67.75	94.54	90.99	107.21	104.22	110.90	102.85	100.25	96.28	96.23	129.89	127.60	131.15
1933	105.42	111.05	91.15	108.49	103.87	110.50	104.22	99.97	98.22	97.60	129.77	134.26	131.13
1934	132.72	110.98	116.34	110.14	104.15	110.37	104.18	100.28	98.43	99. 0	130.07	134.32	131.13
1935	117.11	101.06	108.00	106.89	105.50	107.85	103.63	98.00	98.44	134.05	128.05	133.18	130.18
1936	69.91	81.12	103.08	102.31	101.12	104.10	100.57	94.69	96.08	132.,7	128.85	132.59	130.50
1937	69.54	81.18	105.57	106.39	113.71	104.00	100.50	96.13	96.25	132.,3	129.98	133.27	130.48
1938	112.91	81.11	109.53	105.40	112.04	105.00	99.93	98.20	97.57	132.32	136.87	133.25	131.53
1939	98.81	110.75	106.64	99.93	108.56	101.46	97.22	95.52	96.73	131.11	135.52	132.06	132.11
1940	95.92	107.14	107.62	104.85	108.31	103.39	96.93	97.43	136.98	131.05	135.51	132.11	132.21
1941	96.16	119.56	110.26	104.61	108.41	104.07	97.09	97.90	137.56	133.06	136.84	134.31	132.20
1942	82.28	117.13	113.22	120.31	107.64	103.22	97.84	97.65	137.00	133.83	137.08	133.83	132.92
1943	36.23	111.85	101.30	112.49	103.47	97.91	95.68	95.15	134.69	139. 1	135.37	133.35	132.62
1944	126.06	115.30	101.79	112.53	103.53	98.46	96.15	97.23	135.07	139.51	135.37	135.13	132.69
1945	120.45	117.31	107.53	112.60	105.30	98.40	98.39	142.09	135.14	139. 2	135.55	135.34	132.96
1946	148.03	120.71	107.94	112.85	107.16	98.67	97.15	142.79	137.32	140.99	137.93	135.36	134.17
1947	155.14	125.18	130.64	112.91	107.21	100.28	99.56	142.77	138.54	141. 3	137.93	136.49	134.29
1948	163.79	120.58	128.44	114.58	105.33	102.26	100.70	142.54	146.69	141.72	139.03	137.75	134.06
1949	49.52	40.20	96.42	90.31	89.08	85.89	87.08	135.29	140.26	135.82	135.39	132.50	129.44
1950	25.65	37.18	96.27	95.42	87.90	87.97	143.11	135.36	140.37	136.01	135.60	132.86	131.38
1951	17.44	47.58	93.93	93.45	85.50	87.62	142.79	136.39	141.00	137.70	134.70	133.44	130.55
1952	21.03	114.98	95.07	93.69	87.12	88.38	142.56	137.53	141.55	137.48	135.72	133.41	130.43
1953	32.13	113.97	99.81	92.53	89.69	89.90	141.92	146.70	141.17	138.24	136.89	132.93	129.93
1954	33.20	113.51	98.46	92.12	89.64	91.90	141.74	146.23	140.59	139.80	136.56	132.91	129.54
1955	48.39	113.11	102.58	90.89	92.18	151.24	141.36	145.03	140.40	139.71	136.59	134.51	129.48
1956	69.17	110.89	101.39	88.89	91.99	150.98	143.18	146.60	142.27	139.03	137.35	133.92	133.50
1957	154.33	104.63	95.62	87.46	89.52	148.87	143.21	145.96	140.67	139.19	136.36	132.87	132.40
1958	153.42	109.05	94.92	93.19	93.02	148.96	153.10	145.97	142.28	141.05	136.25	132.74	133.27
1959	148.98	106.93	93.11	92.71	96.13	148.63	152.60	145.42	144.32	140.78	136.41	132.43	133.18
1960	114.19	91.29	79.79	90.56	157.72	145.30	149.58	143.06	142.39	139.04	136.34	130.78	132.08
1961	122.48	97.94	81.83	91.44	158.31	149.03	151.10	145.69	142.39	140.35	136.37	135.46	132.58
1962	27.45	60.07	59.81	71.40	150.93	144.08	147.02	140.89	139.31	136.18	132.37	131.87	129.36
1963	55.78	59.29	70.87	76.96	151.25	155.64	147.15	142.86	141.48	136.15	132.32	132.02	129.90
1964	59.21	60.15	72.10	82.05	151.63	155.69	147.13	145.67	141.55	136.68	132.32	133.16	130.91
1965	74.22	60.98	79.41	168.50	152.01	155.73	147.60	146.12	141.89	138.79	132.58	133.81	131.01
1966	80.75	60.14	80.87	168.63	155.56	157.04	150.18	145.81	143.15	138.56	137.46	134.10	131.10
1967	80.72	71.76	81.85	168.67	157.90	158.53	150.18	147.60	143.38	138.63	137.46	134.47	132.59
1968	69.06	77.17	82.33	168.02	169.54	157.97	151.80	149.12	142.65	137.95	138.09	134.46	131.94
1969	66.59	79.58	87.93	168.22	169.42	157.75	154.68	149.12	143.11	137.87	138.25	135.45	135.65
1970	48.43	74.85	191.80	167.09	168.26	157.24	154.14	148.47	144.51	137.33	138.20	134.75	134.96

Table 28. Standard Deviations -

Maximum Hourly Flows

Year Began	5	10	15	20	25	30	35	40	45	50	55	60	65
	Number of Years for Statistic												
1899	254.92	221.88	263.29	250.25	266.11	267.16	254.85	252.46	253.18	259.32	251.98	244.91	246.27
1900	229.20	224.45	242.07	242.74	261.32	266.99	250.37	250.72	252.12	253.06	250.29	241.87	246.01
1901	194.22	208.19	257.13	257.59	262.88	261.52	244.94	250.65	251.44	256.09	248.07	241.62	244.81
1902	192.54	188.97	258.26	256.97	261.69	254.29	244.36	247.39	250.67	254.01	244.82	244.97	244.33
1903	168.44	254.27	256.62	259.41	260.12	252.72	246.76	252.14	252.51	252.44	244.38	244.85	246.16
1904	168.84	267.47	248.69	268.91	269.60	254.84	252.11	252.96	259.81	251.'8	244.00	245.55	246.39
1905	219.70	278.50	247.25	269.35	274.55	253.89	253.00	254.98	262.37	252.39	243.02	247.41	249.74
1906	222.16	288.59	278.71	280.04	274.98	253.40	259.71	258.59	262.96	253.45	245.93	249.03	250.89
1907	185.40	291.12	278.45	278.97	266.64	253.00	255.23	257.93	260.84	250.05	249.74	248.65	250.57
1908	340.10	300.71	289.74	283.04	269.58	259.82	264.10	263.01	261.78	251.97	251.80	252.63	253.07
1909	346.10	278.62	295.60	289.79	268.04	262.66	262.12	268.68	258.66	249.,2	250.71	251.18	252.96
1910	337.30	261.03	295.91	288.26	260.73	259.48	260.02	267.71	256.02	245. 5	249.93	252.24	252.00
1911	355.02	306.99	299.34	288.19	259.64	264.80	263.80	268.06	256.93	248.31	251.47	253.28	253.41
1912	396.84	324.97	310.17	286.95	266.52	266.87	269.30	270.27	257.23	256.17	254.40	256.00	253.94
1913	261.32	264.56	264.02	251.95	243.76	251.44	252.00	251.99	242.18	242.97	244.68	245.82	241.85
1914	211.14	270.35	271.03	248.52	245.97	248.12	257.62	247.73	238.79	241.17	242.55	245.20	241.15
1915	164.76	260.21	271.91	241.59	243.92	247.14	257.77	245.86	235.13	241.19	244.51	244.89	239.01
1916	258.96	271.50	265.91	235.80	246.76	248.60	255.64	244.66	236.45	241.11	244.03	244.94	240.62
1917	252.10	266.83	250.33	233.94	240.80	246.87	252.19	239.78	240.54	240.16	243.19	242.03	240.31
1918	267.80	265.37	248.83	239.37	249.46	250.45	259.65	239.79	240.93	243.02	244.41	240.23	238.94
1919	329.56	300.97	260.99	254.68	255.52	265.36	252.96	242.24	244.51	245.'9	248.29	243.65	239.17
1920	335.66	315.49	260.53	258.70	259.62	269.93	254.59	241.43	247.46	250.49	250.35	243.53	250.96
1921	284.04	269.39	228.08	243.70	246.52	255.08	242.62	233.64	239.13	242.'4	243.67	239.10	247.33
1922	260.56	248.94	227.55	237.82	245.63	252.04	237.88	238.97	238.72	242.20	241.02	239.24	246.79
1923	262.94	239.34	229.89	244.87	246.98	247.80	235.79	237.57	240.26	242.07	237.72	236.42	245.55
1924	272.38	226.70	229.72	237.00	252.52	240.20	229.77	233.87	236.38	240.17	235.84	231.63	244.34
1925	295.32	222.97	233.05	240.60	256.79	241.08	227.97	236.44	241.02	241. 2	235.15	243.90	245.24
1926	254.74	200.10	230.26	237.14	249.29	235.72	225.44	233.52	237.93	239.,3	235.01	244.27	241.72
1927	217.32	201.05	223.57	236.89	246.33	230.77	233.03	233.49	237.94	237.07	235.49	243.97	239.42
1928	215.74	213.37	230.85	242.99	244.77	231.26	233.95	237.43	239.75	235.20	234.01	244.10	245.24
1929	181.02	208.39	225.21	247.56	233.76	222.67	228.37	231.07	236.59	232.18	227.93	242.01	242.86
1930	150.62	201.92	222.37	247.15	230.23	216.74	229.03	234.23	235.88	229.13	239.22	241.06	243.12
1931	145.46	218.02	231.28	247.93	231.92	221.72	230.49	235.83	237.95	233.04	243.32	240.63	247.48
1932	184.78	226.79	243.42	253.58	233.46	235.65	235.80	240.52	239.26	237.30	246.39	241.26	249.52
1933	211.00	250.41	252.07	252.02	234.37	236.98	240.53	242.75	237.36	235. 4	246.67	247.70	250.49
1934	235.76	247.31	269.74	246.94	231.00	236.27	239.14	243.53	237.87	232. 2	247.55	249.01	250.82
1935	253.22	250.24	279.33	250.13	229.97	240.93	246.18	246.53	237.86	248.08	249.29	250.83	251.76
1936	290.58	274.19	282.09	253.53	236.98	244.66	248.74	249.51	242.77	253.10	249.29	255.98	257.24
1937	260.62	272.74	276.52	245.62	245.82	244.30	249.48	246.07	243.14	252.56	246.39	253.83	257.02
1938	269.82	272.61	265.70	240.21	242.18	245.45	247.29	240.65	238.60	250.24	251.03	253.78	253.96
1939	250.86	286.73	250.67	229.80	236.37	239.70	244.65	238.13	232.27	248.73	249.13	251.75	249.10
1940	263.26	292.37	249.11	224.15	238.47	245.01	245.58	235.94	247.51	248. 9	250.61	251.64	248.98
1941	257.80	277.04	241.18	223.57	235.47	241.76	243.64	236.79	248.94	245.16	252.83	254.46	248.81
1942	276.86	280.47	237.96	240.12	239.44	245.12	242.85	239.95	250.77	244.17	252.49	256.06	247.95
1943	255.40	253.64	223.67	230.27	236.57	240.20	233.63	232.20	245.84	247.16	250.51	250.86	242.75
1944	314.60	246.58	220.12	230.74	235.87	242.28	235.17	228.95	247.60	248.15	251.10	249.28	243.40
1945	321.52	242.03	211.12	232.27	241.36	242.63	232.03	245.54	247.30	249.35	250.58	247.68	244.86
1946	297.08	232.87	212.17	229.89	238.56	241.28	233.79	247.83	243.75	252.33	254.16	249.06	247.12
1947	284.08	218.51	227.88	230.08	239.78	237.18	234.68	247.51	240.54	250.05	254.17	245.54	246.13
1948	251.88	227.81	221.89	231.86	237.16	230.00	229.88	244.65	246.24	250.02	250.45	241.70	245.46
1949	178.56	172.09	222.79	216.19	227.81	221.93	216.71	239.23	240.77	244.75	242.26	237.47	239.86
1950	162.54	155.92	222.53	221.31	226.86	217.12	234.69	238.02	241.33	243.49	240.97	239.47	242.42
1951	167.86	169.31	227.23	223.72	229.96	223.11	240.68	236.09	247.27	249.78	243.53	242.88	244.11
1952	152.74	199.78	212.08	227.45	227.80	226.45	242.29	235.09	246.27	251.17	242.03	242.96	244.53
1953	163.74	216.90	225.19	233.48	225.63	225.05	243.62	245.53	249.81	250.31	240.77	244.92	245.92
1954	167.20	214.91	228.73	240.12	230.61	223.07	247.90	248.55	252.10	248.63	242.82	244.97	247.42
1955	149.30	222.52	240.91	242.93	228.04	246.71	249.80	251.17	252.48	248. 1	245.37	249.07	249.56
1956	170.76	226.91	242.35	245.49	234.16	252.82	246.06	257.20	258.89	251.10	249.71	250.47	255.40
1957	246.62	241.65	252.29	246.51	241.15	257.18	246.83	257.93	262.09	250.94	251.15	252.16	257.49
1958	250.06	255.92	256.73	241.10	237.31	256.93	257.22	260.57	259.92	248.48	252.31	252.76	256.23
1959	262.62	259.50	264.43	246.46	234.25	261.35	260.17	262.72	257.67	250. 8	252.04	254.11	256.42
1960	295.74	266.71	274.15	247.72	266.20	265.38	265.73	265.38	259.86	254.98	258.15	257.92	257.45
1951	263.06	278.14	270.40	250.01	269.23	259.55	269.55	269.90	260.03	257.'0	257.71	262.45	255.00
1962	236.68	255.12	246.48	239.78	259.29	246.87	259.55	264.02	251.42	251. 0	252.66	258.40	251.03
1963	261.78	260.06	238.11	234.12	258.30	258.41	262.07	261.16	248.30	252. 3	253.01	256.75	249.91
1964	256.38	265.34	241.07	227.15	261.09	259.76	262.73	257.05	249.02	250.98	253.34	255.90	247.98
1965	277.68	263.35	231.71	258.81	259.31	260.73	261.04	255.38	250.45	254. 9	254.48	254.26	247.30
1966	273.22	264.07	238.99	265.77	254.84	267.30	268.02	257.15	254.77	255.18	260.58	253.75	247.09
1967	273.56	251.38	249.82	264.94	248.90	263.36	267.93	253.26	253.26	254.26	260.37	252.22	244.30
1968	258.34	226.29	224.91	257.43	257.74	262.12	261.07	246.62	251.50	252.13	256.99	248.92	242.91
1969	274.30	233.42	217.41	262.27	260.44	263.79	257.15	248.10	250.38	253.03	255.85	247.28	247.23
1970	249.02	208.73	252.52	254.72	257.34	258.27	252.19	247.04	251.80	252.16	252.13	244.77	245.17

Table 29. Means - Maximum

5-Minute Rainfall

Year Began	5	10	15	20	25	30	35	40	45	50	55	60	65
	Number of Years for Statistic												
1899	.39	.35	.42	.40	.40	.40	.39	.39	.40	.39	.37	.39	.38
1900	.40	.41	.41	.39	.39	.40	.40	.40	.40	.39	.39	.39	.38
1901	.38	.41	.41	.38	.39	.39	.40	.40	.39	.39	.39	.38	.38
1902	.39	.42	.41	.40	.40	.39	.40	.40	.40	.39	.39	.38	.38
1903	.33	.39	.40	.39	.39	.38	.39	.40	.39	.39	.39	.38	.37
1904	.32	.43	.40	.40	.40	.39	.39	.40	.40	.39	.39	.38	.37
1905	.42	.41	.37	.39	.40	.40	.40	.39	.39	.39	.38	.38	.37
1906	.43	.42	.38	.39	.39	.40	.40	.39	.39	.39	.38	.38	.37
1907	.45	.43	.41	.40	.40	.40	.41	.40	.39	.39	.38	.38	.37
1908	.46	.43	.42	.41	.40	.40	.41	.40	.40	.39	.38	.38	.37
1909	.54	.44	.43	.42	.40	.41	.41	.40	.40	.40	.39	.38	.37
1910	.41	.35	.38	.39	.39	.39	.39	.39	.39	.38	.38	.37	.37
1911	.41	.35	.37	.38	.40	.39	.38	.38	.39	.38	.37	.36	.37
1912	.40	.38	.38	.38	.39	.40	.39	.38	.39	.38	.37	.36	.37
1913	.40	.39	.40	.38	.39	.40	.39	.39	.38	.38	.37	.37	.37
1914	.34	.37	.38	.37	.38	.39	.39	.39	.38	.38	.36	.36	.36
1915	.30	.37	.39	.39	.39	.39	.39	.39	.38	.37	.36	.37	.36
1916	.30	.35	.37	.40	.39	.38	.38	.39	.38	.37	.36	.36	.36
1917	.36	.37	.37	.39	.40	.39	.38	.38	.37	.37	.36	.36	.37
1918	.39	.39	.37	.39	.40	.39	.39	.38	.37	.37	.36	.36	.37
1919	.41	.40	.38	.39	.40	.39	.39	.39	.38	.36	.36	.36	.37
1920	.45	.43	.42	.41	.41	.40	.40	.39	.38	.37	.37	.37	.38
1921	.41	.41	.43	.41	.40	.39	.40	.39	.38	.37	.37	.36	.38
1922	.36	.38	.40	.40	.39	.38	.39	.37	.37	.36	.37	.37	.37
1923	.40	.37	.39	.40	.39	.39	.38	.37	.36	.36	.36	.37	.37
1924	.38	.36	.38	.39	.39	.39	.38	.38	.36	.36	.36	.37	.37
1925	.42	.41	.40	.40	.39	.39	.38	.37	.36	.36	.36	.37	.37
1926	.41	.44	.41	.39	.39	.40	.38	.37	.36	.36	.36	.37	.37
1927	.38	.41	.41	.39	.38	.39	.37	.37	.36	.36	.37	.37	.37
1928	.33	.38	.40	.39	.39	.38	.37	.36	.36	.36	.37	.37	.36
1929	.34	.38	.40	.39	.39	.38	.37	.35	.35	.36	.37	.37	.37
1930	.40	.39	.39	.38	.39	.37	.37	.35	.36	.36	.36	.37	.37
1931	.47	.41	.39	.38	.40	.38	.37	.35	.36	.35	.37	.37	.37
1932	.44	.43	.40	.38	.39	.37	.36	.36	.36	.37	.37	.36	.37
1933	.43	.43	.41	.40	.39	.37	.36	.36	.36	.37	.37	.37	.37
1934	.43	.43	.41	.40	.39	.38	.36	.36	.36	.37	.37	.37	.37
1935	.38	.38	.38	.38	.37	.36	.35	.35	.35	.36	.36	.37	.37
1936	.36	.35	.36	.38	.36	.35	.34	.34	.34	.36	.36	.36	.37
1937	.42	.38	.36	.38	.36	.35	.35	.35	.36	.36	.36	.37	.37
1938	.43	.40	.39	.38	.36	.35	.35	.35	.36	.37	.36	.37	.37
1939	.42	.40	.39	.38	.37	.34	.35	.35	.36	.37	.37	.37	.37
1940	.39	.38	.38	.36	.36	.34	.35	.35	.36	.36	.36	.37	.37
1941	.34	.35	.38	.36	.35	.33	.34	.34	.36	.36	.36	.37	.36
1942	.34	.33	.37	.34	.34	.33	.34	.35	.36	.35	.35	.37	.37
1943	.38	.37	.36	.35	.34	.34	.34	.35	.36	.35	.36	.36	.37
1944	.38	.38	.37	.36	.33	.33	.34	.36	.36	.36	.36	.36	.37
1945	.37	.38	.35	.35	.33	.34	.34	.35	.36	.36	.37	.36	.37
1946	.37	.41	.37	.35	.33	.34	.34	.37	.36	.37	.37	.37	.37
1947	.33	.38	.35	.34	.33	.34	.35	.36	.35	.36	.37	.37	.37
1948	.36	.35	.34	.32	.33	.34	.35	.36	.35	.36	.36	.37	.37
1949	.39	.36	.35	.32	.33	.34	.35	.36	.36	.36	.36	.37	.37
1950	.40	.35	.34	.32	.34	.34	.35	.36	.36	.37	.36	.37	.37
1951	.44	.36	.35	.32	.34	.33	.36	.36	.37	.37	.37	.37	.37
1952	.43	.35	.34	.33	.35	.36	.37	.36	.37	.37	.38	.38	.38
1953	.34	.33	.31	.32	.33	.35	.36	.35	.36	.36	.37	.37	.37
1954	.34	.33	.29	.31	.33	.35	.35	.36	.36	.36	.37	.37	.37
1955	.30	.32	.30	.32	.32	.35	.35	.36	.36	.36	.37	.37	.37
1956	.29	.30	.29	.31	.31	.35	.35	.36	.36	.36	.36	.37	.37
1957	.28	.30	.30	.33	.34	.35	.35	.36	.37	.37	.37	.37	.37
1958	.32	.30	.31	.33	.35	.36	.35	.36	.37	.38	.37	.37	.37
1959	.33	.27	.30	.32	.35	.36	.36	.36	.36	.37	.37	.37	.37
1960	.33	.30	.33	.33	.35	.36	.37	.37	.37	.37	.38	.38	.38
1961	.31	.28	.32	.32	.36	.36	.37	.37	.37	.37	.38	.38	.38
1962	.32	.31	.34	.36	.37	.36	.37	.38	.38	.38	.38	.38	.38
1963	.29	.31	.34	.36	.37	.36	.37	.37	.38	.38	.38	.38	.38
1964	.21	.28	.32	.35	.36	.36	.37	.37	.38	.38	.38	.37	.38
1965	.26	.33	.33	.36	.37	.37	.38	.37	.38	.38	.38	.38	.38
1966	.26	.32	.32	.38	.37	.38	.38	.37	.38	.38	.38	.38	.38
1967	.31	.35	.38	.38	.37	.38	.39	.39	.39	.39	.38	.39	.38
1968	.34	.36	.39	.39	.37	.38	.38	.39	.39	.38	.38	.39	.38
1969	.36	.37	.40	.40	.39	.39	.39	.40	.40	.39	.39	.39	.38
1970	.40	.36	.39	.40	.39	.39	.39	.39	.40	.39	.39	.39	.38

Table 30. Standard Deviations

Maximum 5-Minute Rainfall

Year Began	5	10	15	20	25	30	35	40	45	50	55	60	65
	Number of Years for Statistic												
1899	.18	.16	.21	.19	.19	.18	.17	.18	.17	.17	.17	.16	.16
1900	.19	.26	.22	.20	.19	.18	.18	.18	.17	.17	.17	.17	.16
1901	.19	.26	.22	.20	.19	.18	.18	.18	.17	.17	.17	.17	.16
1902	.19	.25	.21	.20	.19	.18	.18	.18	.17	.17	.17	.17	.16
1903	.13	.24	.20	.20	.18	.17	.18	.17	.17	.17	.17	.16	.16
1904	.14	.24	.20	.19	.18	.17	.18	.17	.17	.17	.16	.16	.16
1905	.33	.24	.21	.19	.18	.18	.18	.17	.17	.17	.16	.16	.16
1906	.33	.24	.21	.19	.18	.19	.18	.17	.17	.17	.16	.16	.16
1907	.32	.23	.22	.19	.18	.19	.18	.17	.17	.17	.17	.16	.16
1908	.32	.23	.22	.19	.18	.19	.18	.17	.17	.17	.17	.16	.16
1909	.27	.22	.20	.19	.17	.18	.17	.17	.17	.17	.16	.16	.16
1910	.12	.13	.13	.13	.15	.15	.14	.14	.14	.14	.14	.14	.14
1911	.12	.13	.13	.13	.15	.15	.14	.14	.14	.14	.14	.14	.15
1912	.13	.16	.14	.13	.16	.15	.15	.14	.15	.15	.15	.14	.15
1913	.12	.16	.14	.13	.16	.15	.15	.15	.15	.15	.15	.14	.15
1914	.07	.14	.13	.12	.15	.15	.14	.15	.14	.14	.15	.14	.14
1915	.13	.14	.13	.16	.16	.14	.14	.15	.15	.15	.15	.15	.14
1916	.13	.14	.14	.16	.16	.15	.15	.15	.15	.15	.15	.15	.15
1917	.20	.15	.14	.17	.16	.15	.15	.15	.15	.15	.15	.15	.17
1918	.20	.16	.14	.17	.16	.15	.16	.15	.15	.15	.15	.15	.17
1919	.19	.15	.14	.17	.16	.15	.16	.15	.15	.15	.15	.15	.17
1920	.12	.12	.16	.16	.14	.14	.15	.15	.15	.15	.15	.15	.16
1921	.14	.14	.16	.16	.15	.15	.15	.15	.15	.15	.15	.15	.16
1922	.09	.11	.16	.15	.14	.14	.14	.15	.14	.14	.15	.17	.16
1923	.12	.11	.16	.15	.15	.15	.15	.14	.14	.14	.15	.17	.16
1924	.12	.11	.16	.15	.15	.15	.15	.15	.15	.14	.15	.17	.16
1925	.12	.18	.17	.15	.15	.15	.15	.15	.15	.15	.15	.16	.16
1926	.14	.17	.16	.15	.15	.15	.15	.15	.15	.15	.15	.17	.16
1927	.14	.19	.16	.16	.15	.15	.15	.15	.15	.15	.17	.17	.16
1928	.10	.19	.16	.15	.16	.15	.15	.15	.15	.15	.17	.17	.16
1929	.10	.19	.16	.16	.16	.15	.15	.15	.15	.15	.17	.17	.17
1930	.24	.19	.16	.15	.16	.16	.15	.15	.15	.15	.17	.16	.17
1931	.22	.18	.16	.16	.15	.15	.15	.15	.15	.15	.17	.16	.17
1932	.24	.18	.17	.16	.16	.16	.15	.15	.15	.15	.17	.17	.17
1933	.25	.18	.17	.17	.16	.16	.15	.15	.15	.18	.17	.17	.17
1934	.25	.18	.17	.17	.16	.16	.16	.15	.15	.18	.17	.17	.17
1935	.16	.12	.13	.14	.14	.14	.14	.14	.14	.16	.16	.16	.16
1936	.14	.12	.13	.14	.13	.14	.13	.14	.14	.16	.16	.16	.16
1937	.11	.12	.12	.13	.14	.13	.13	.14	.17	.16	.16	.16	.16
1938	.10	.13	.15	.14	.14	.13	.13	.14	.17	.16	.16	.16	.16
1939	.11	.13	.15	.14	.14	.14	.14	.14	.17	.16	.16	.16	.16
1940	.08	.12	.14	.14	.14	.14	.14	.14	.16	.16	.16	.16	.16
1941	.12	.13	.14	.14	.14	.14	.14	.14	.17	.16	.16	.16	.16
1942	.12	.12	.14	.14	.14	.13	.14	.17	.17	.16	.16	.16	.16
1943	.15	.17	.15	.14	.14	.14	.14	.17	.17	.16	.16	.16	.17
1944	.15	.16	.15	.14	.14	.14	.14	.17	.17	.17	.16	.16	.17
1945	.15	.16	.15	.15	.14	.15	.14	.17	.17	.17	.16	.16	.17
1946	.14	.15	.14	.14	.14	.15	.14	.17	.17	.17	.16	.16	.16
1947	.13	.15	.15	.14	.14	.15	.18	.17	.17	.17	.16	.17	.17
1948	.20	.15	.14	.14	.13	.14	.18	.17	.17	.16	.16	.17	.16
1949	.19	.15	.15	.14	.14	.14	.18	.17	.17	.17	.16	.17	.17
1950	.19	.16	.15	.14	.15	.14	.18	.17	.17	.17	.16	.17	.17
1951	.16	.15	.15	.14	.15	.15	.18	.17	.17	.17	.16	.17	.16
1952	.17	.16	.15	.14	.15	.19	.18	.17	.17	.17	.17	.17	.16
1953	.11	.11	.11	.12	.13	.18	.17	.17	.16	.16	.17	.16	.16
1954	.11	.12	.12	.12	.13	.18	.17	.17	.16	.16	.17	.16	.16
1955	.13	.13	.12	.14	.13	.18	.17	.17	.17	.16	.17	.16	.16
1956	.11	.12	.12	.14	.14	.18	.17	.17	.17	.16	.17	.16	.16
1957	.12	.12	.12	.14	.19	.18	.17	.17	.17	.17	.17	.16	.16
1958	.13	.12	.12	.14	.19	.18	.17	.17	.17	.17	.17	.16	.16
1959	.15	.12	.12	.14	.19	.18	.18	.17	.17	.17	.17	.16	.16
1960	.14	.13	.15	.14	.19	.18	.18	.17	.16	.17	.17	.16	.17
1961	.15	.13	.15	.14	.19	.18	.18	.17	.16	.17	.17	.16	.17
1962	.14	.13	.15	.20	.19	.18	.17	.17	.17	.17	.16	.16	.16
1963	.12	.13	.15	.20	.19	.18	.18	.17	.18	.17	.17	.16	.16
1964	.05	.11	.14	.20	.19	.19	.17	.17	.18	.17	.17	.16	.16
1965	.11	.15	.14	.20	.18	.18	.18	.17	.18	.17	.17	.17	.16
1966	.11	.16	.15	.20	.18	.18	.17	.17	.17	.17	.17	.17	.16
1967	.13	.16	.22	.20	.19	.18	.17	.18	.17	.17	.16	.17	.16
1968	.14	.16	.22	.20	.19	.18	.17	.18	.17	.17	.17	.16	.16
1969	.12	.15	.21	.19	.19	.18	.17	.18	.17	.17	.16	.16	.16
1970	.17	.15	.21	.19	.19	.18	.17	.18	.17	.17	.17	.16	.16

Table 31. Annual Series Maximum Hourly Flow and Five-Minute Rainfall

Water Year	Annual Maximum Hourly Flow (cfs)	Annual Maximum Five-Minute Rainfall (in
1899	268.5	.44
1900	273.9	.23
1901	297.8	.64
1902	231.5	.19
1903	203.0	.52
1904	139.9	.35
1905	98.9	.24
1906	289.4	.34
1907	111.0	.16
1908	305.0	1.00
1909	294.2	.42
1910	111.2	.35
1911	105.6	.35
1912	884.5	.60
1913	335.0	.31
1914	250.2	.45
1915	199.8	.30
1916	314.7	.36
1917	206.9	.27
1918	162.3	.10
1919	234.8	.45
1920	570.8	.63
1921	285.4	.48
1922	280.4	.38
1923	392.9	.31
1924	764.6	.27
1925	312.7	.47
1926	268.0	.56
1927	192.3	.31
1928	440.1	.48
1929	263.5	.21
1930	109.8	.33
1931	80.9	.34
1932	184.4	.32
1933	266.5	.82
1934	132.4	.53
1935	88.9	.19
1936	277.5	.29
1937	315.5	.33
1938	390.3	.56
1939	198.8	.41
1940	270.8	.50
1941	167.7	.34
1942	421.5	.29

Table 31. (Cont'd)

1943	235.5	.38
1944	220.8	.17
1945	296.1	.50
1946	263.0	.54
1947	314.2	.29
1948	531.0	.33
1949	255.4	.20
1950	125.3	.27
1951	194.0	.70
1952	194.4	.43
1953	164.9	.38
1954	175.3	.44
1955	151.9	.17
1956	119.4	.22
1957	207.2	.23
1958	192.8	.44
1959	144.8	.18
1960	259.2	.36
1961	498.7	.17
1962	224.4	.43
1963	247.7	.50
1964	251.4	.20
1965	195.8	.23
1966	266.8	.22
1967	349.9	.28
1968	218.0	.14
1969	357.9	.44
1970	173.5	.24

and found satisfactory.

The end product is twelve runoff files; a combination of 3 time steps (1-minute, 5-minute and 15-minute) and 4 soils (very rapid permeability, moderate to moderately rapid permeability, moderately slow permeability, and impermeable). These runoff files have been given the following acronyms:

ROF1SOIL1	ROF5SOIL3
ROF1SOIL2	ROF5SOIL4
ROF1SOIL3	ROF15SOIL1
ROF1SOIL4	ROF15SOIL2
ROF5SOIL1	ROF15SOIL3
ROF5SOIL2	ROF15SOIL4

The first number in the acronym refers to the time step in minutes and last number has the following code:

1-soils of very rapid permeability

2-soils of moderate to moderately rapid permeability

3-soils of moderately slow permeability

4-impermeable surfaces such as granite outcrop, roads, roofs, and parking lots.

Flood Frequency Analysis

Introduction. Ninety-nine storms are on the runoff file for the 25-year period from 1918 through 1942. For each storm the routing model calculates the flow at the downstream end of each segment specified in the system. To translate this information to a form needed for planning and the decision process, two manipulations of the data are needed. First, the flood peaks for pre-selected probabilities must be estimated from the 25 years of simulated flows. Second, the water surface elevations must be estimated for the flood peaks. Flood frequency analysis is used for the first task, and a rating table is used for the second. The final product, then, is flood elevations for selected probabilities.

Flood Frequency Distributions. Several distributions have been used for flood frequency analysis over the past decades. The more popular ones are the extreme-value distribution, the log-normal distribution, and the Log-Pearson Type III distribution. In addition, there are three methods for estimating the parameter values for the distribution functions. They are the methods of moments, maximum likelihood, and least squares. In 1967, the Water Resources Council of the federal government recommended the Log-Pearson Type III distribution. The Log-normal distribution, which is the Log-Pearson Type III distribution with a skew of 0.0, fit by the method of moments, was initially programmed for the frequency analysis in UROS4. After the model was used on several watersheds, it became apparent that this distribution did not fit the data properly. The situation was analyzed by fitting all three distributions to the 25-year annual flood series for Womack Creek with all three methods. Results showed that the method initially selected was estimating the 100-year flood flow at least twenty percent higher than the other methods. Upon closer examination, it appeared that the method of parameter estimation gave too much weight to the lower peaks which are under-estimated by the routing model since the runoff file can miss the critical peak in years when no major storms occur. The effect is shown on Figure 23. The frequency distribution and parameter estimation method which most closely fit the data was the extreme-value distribution with the method of moments for parameter estimation.

The extreme value distribution as defined by Gumbel is

$$P = 1 - e^{-e^{-a(X - b)}} \quad \text{.....................} \quad (51)$$

where,

$$a = \frac{1.28}{\sigma_x} \quad \text{........................} \quad (52)$$

$$b = \bar{X} - 0.45\sigma_x \quad \text{........................} \quad (53)$$

$$\bar{X} = \frac{1}{N} \sum_{i=1}^{N} X_i \quad \text{..........................} \quad (54)$$

$$\sigma_x^2 = \frac{1}{N-1} \sum_{i=1}^{N} (X_i - \bar{X})^2 \quad \text{....................} \quad (55)$$

P = probability in any one year that a flood greater than the value of X will occur

X_i = annual flood series (largest flood flow each year for N years)

N = number of years of record (25 years for the runoff file)

$\bar{X}$ = mean of the annual flood series

σ_x^2 = variance of the annual flood series

X = flow associated with probability P

e = base of natural logarithms (2.718)

The return period is related to the probability by

$$T_r = 1.0/P \quad \text{.........................} \quad (56)$$

Thus, the 100-year (T_r) flood flow is a flow with only a 1% chance (P) of being exceeded in any one year. The computations with Equation 51 can be simplified by expressing the relationship in the form

$$X = \bar{X} + K\sigma_x \quad \text{.......................} \quad (57)$$

where

$$K = \frac{a(X - b) - 0.571}{1.28} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (58)$$

and is evaluated by solving for a(X - b) in Equation 51 given values of P. Values of K for the seven probability levels programmed into UROS4 are given in Table 32. From the annual series, $\bar{X}$ is calculated with Equation 54, σ_x with Equation 55, and the flood flows for seven probabilities with Equation 57.

Rating Table. With the seven discharges, only the stage (flood elevation) remains to be estimated. Any procedure beyond the application of a rating table provided in the input data was too costly for inclusion in UROS4. Backwater computations, an example of a more refined method, could, however, be used to develop the rating table or to determine the water surface from the peak flows calculated with the routing model.

The rating table is a set of monotonically increasing water surface elevations (E) paired with associated peak flows (Q). For each flow calculated from Equation 57 a water surface elevation is interpolated from the table. For example, given a ten-year peak flow of 570 cfs, UROS4 searches the rating table and finds the first discharge value greater than 570 cfs. If this value is given the subscript i, the elevation associated with 570 cfs would be

$$E_{570} = E_{i-1} + (E_i - E_{i-1}) \left(\frac{Q_i - 570}{Q_i - Q_{i-1}}\right)^{0.9} \quad \ldots\ldots\ldots\ldots \quad (59)$$

where,

E = elevation above any datum,

Q = flow in cfs.

The datum used for E, may be mean sea level or some other base such as

TABLE 32. Coefficients for Extreme-Value Distribution

Probability of Exceedance	Return Period (years)	K
0.5	2	-0.16
0.2	5	0.72
0.1	10	1.31
0.04	25	2.05
0.02	50	2.60
0.01	100	3.14
0.005	200	3.68

the bottom of the channel. The ratio of the discharges in Equation 59 will always have a value between zero and one. The exponent, 0.9, is used to represent typical rating curves non-linearity. If the value of a given discharge, such as 570 cfs, is greater than the highest discharge given in the rating table, the routing model will not extrapolate but will set the elevation to zero. In such cases, additional points should be read in the input data.

Rating tables can be developed by assuming uniform flow and applying Manning's equation, running backwater curves, computed data from the routing model for storage components, or rating curves from the U.S. Geological Survey gaging stations. Another possibility might be generalized rating curves based on channel or drainage area characteristics. Although a possibility and used by some federal agencies in determining 100-year flood plains, generalized rating curves should be used with caution.

SECTION 3

Procedures for Simulation Studies

Introduction

Section 2 has described the development of UROS4, its components, functions, and verification. This section discusses the procedures in using the Model to analyze the hydrology of a particular drainage problem and evaluate the hydrologic effects of various methods for dealing with the problem. These steps will be discussed in the order that they should be followed in a drainage study. Section 5 will illustrate the procedure with an example study on Wild Creek.

Identification of Existing or Potential Drainage Problems

Explicit definition of existing or potential drainage problems should be taken very seriously. Citizen complaints should be recorded reviewed, and analyzed. Problem locations should be plotted on maps and studied for patterns. Field inspections and photographs are helpful. Such examinations may point to obvious solutions such as removing debris from a culvert, channel maintenance for short reaches, or storage of runoff from a new development. However, it may be that simulation for analysis of hydrologic effects may show that what was thought to be an obvious solution may not help or may even create new problems.

Preparation of Input Data

Identification of Target Areas. Target areas are locations of existing or potential problems or locations where actions could be taken to reduce the problems. Such target areas must be located on the map of the drainage area under study. The following should be considered in locating all target areas on the map:

1) complaints from flooded property owners

2) culverts that appear to be too small

3) roadways that could act as detention dams

4) open land along the stream that could be used for temporary storage of storm water

5) existing dams and lakes

6) channel reaches clogged with debris

7) property in the flood plain that might be damaged in a flood

Segmenting the Drainage Area. Following the location of the target areas, the drainage divide above the most downstream target must be drawn on a contour map of the area. In most cases the map will be the DeKalb County maps drawn to the scale 1" equals 200' with 5-foot contour lines. For areas over ten square miles the U.S. Geological Survey 7 1/2 minute maps could be used. With the entire drainage area located, the next step is segmenting that area into subareas, channel lengths, and storm water storage areas. The following guidelines must be followed in segmenting the drainage area.

1) No subarea segment should be greater than about 1000 acres.

2) All runoff from a subarea must leave the subarea at only one location, its outlet.

3) The outlet must begin a channel segment or storage segment.

4) The intersection of major tributaries or a major tributary with the main channel should begin a channel segment.

5) A storage segment may follow an area segment, channel segment, or another storage segment.

6) The travel time through a channel segment should approximate the time step specified for the simulation run.

7) When the runoff file is not used any segment may discharge to an area, though this situation rarely occurs.

8) Target areas should determine the downstream end of an area segment, channel segment or storage segment.

9) Channel and area segments should be constructed so that potential alterations may be easily added for subsequent simulation runs.

10) Undersized culverts should be considered as creating storage segments.

11) Closed conduits are channel segments and should only receive water from a storage segment.

12) Short closed conduits under roadways can be left out since flow times are very short.

13) Channel segments should be relatively homogeneous.

14) The maximum number of all type segments is 99.

15) The maximum number of storage segments of 25.

16) The maximum number of area segments is 50.

17) The maximum number of channel segments is 65.

18) The maximum number of channel plus storage segments is 65.

When the drainage area has been segmented to subareas, channels and storages, each segment must be given a number from 1 to 99. Only one rule must be followed in the numbering: segments with smaller numbers must always discharge to segments with larger numbers. Numbers may be skipped and should be if it is anticipated that a new segment may be added later. It is sometimes helpful to number all subarea segments first,then the channel and storage segments. With all segments numbered, map data and field data can be collected and referenced to the numbering system.

Map Data. The following information should be determined from topographic or soils maps and aerial photographs.

1) Size of each subarea segment,
2) Fraction of the surface area for each area segment that is impervious and that is in each of the other 3 soil types,
3) Length and slope of each channel,
4) Water surface elevations and associated water surface areas for each storage segment when sufficient contours are available to determine this information.

The size of each subarea segment can be measured from the map with a planimeter or an overlay grid. Since soil 2, Table 19 and Figure 26, covers 90% of the County, most subareas will only require the determination of the fraction of impervious area. If the drainage area is in the southwest or eastern part of the county, the area covered by soil 1, 3, or 4 should be marked on the map and the area of each measured. The fraction of area that is impervious is more difficult to determine. One method that works quite well is to color on an overlay of an aerial photo the areas that are uniformly natural, rock outcrops, completely impervious, residential, multi-family, commercial, or office and industrial. Estimates of the impervious fraction for each of the land-use classes can be made from 1) a grid overlay on the aerial photograph, 2) field measurements at sample points, or 3) a table of average values. Sometime in the future the digitized imagery from ERTS may become cost-effective. Listed below are impervious area fractions typical for DeKalb County. Measurements from aerial photographs, however, would enable one to get a more accurate estimate than average values from a table because such a wide range is found for each type of development, especially the higher density uses.

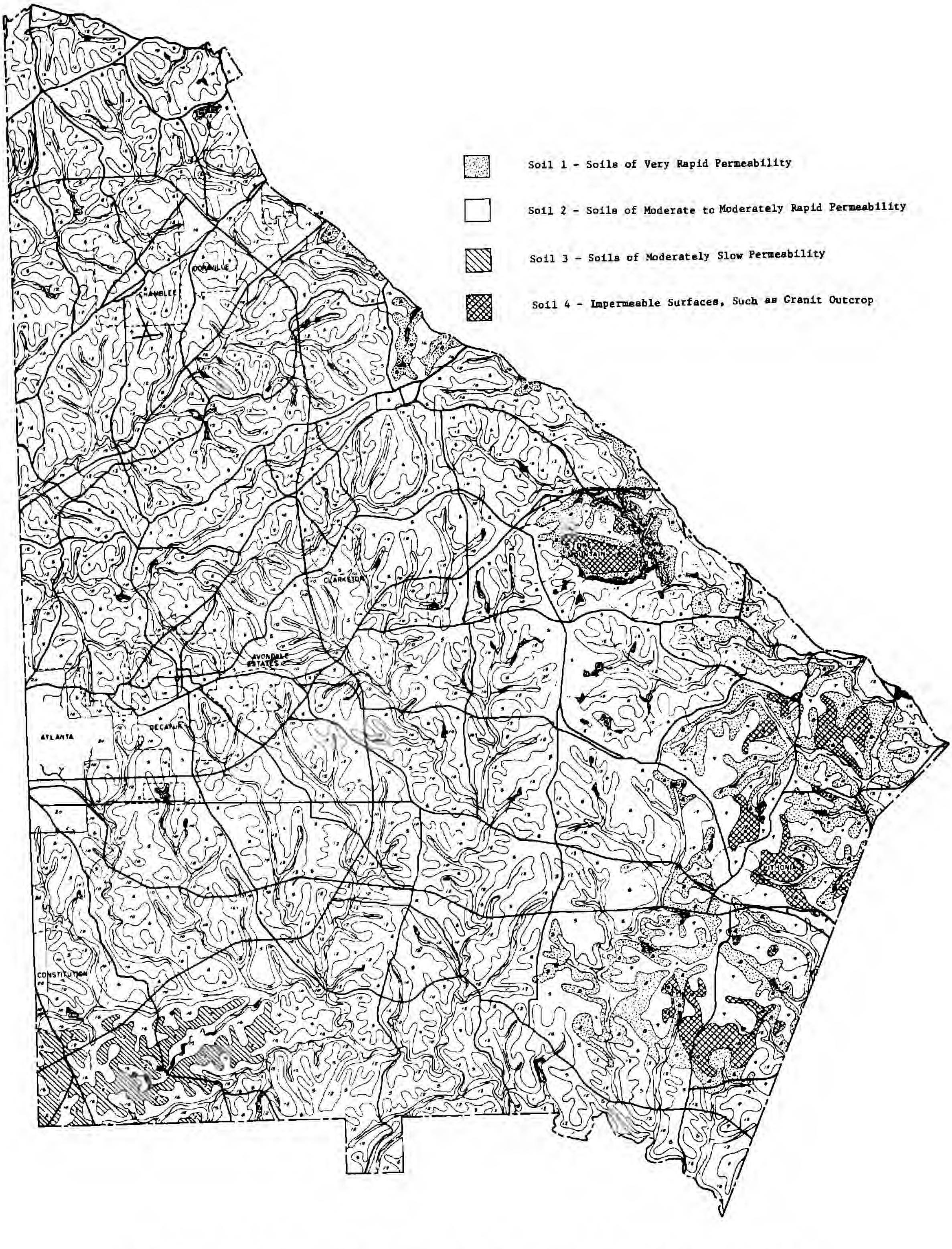


Figure 26. Map of Four Soil Groups in DeKalb County

Impervious Area Fractions

Land Use	Impervious Fraction
Natural-rural	0.00-0.02
Residential	0.15-0.30
Multi-family	0.4-0.8
Office and Industrial	0.5-1.0
Commercial	0.7-1.0
Rock outcrops	1.0

Field Data. Field data is needed to increase the reliability of the simulation. Although a simulation could be made by using "typical values" from other simulations and field data from "similar areas", such an approach is discouraged because each urban drainage system is unique.

The following field information should be measured or noted.

1) Drainage divides which cannot be determined from the maps. It is often difficult to determine whether a specific area drains along a roadway into one drainage area or across the roadway into another. Sometimes the contour interval is too large to accurately determine the drainage divide. Generally a quick inspection of the area in question is sufficient.

2) Culverts.

   Information required includes:

   a) vertical and horizontal dimension of each conduit,

   b) elevation of the invert of the channel and each conduit and the low point of the roadway.

   Information that would be helpful includes:

a) Upstream channel invert slope and channel cross-section for the elevation-area or elevation-volume relation needed for storage routing,

b) roadway profile to estimate dimensions for the broad-crested trapezoidal weir used to represent the flow when the road is overtopped,

c) inlet conditions such as beveled ends, wing walls, protruding conduit, and debris at inlet.

3) Outlet works on existing dams.

The information required is the size, type, and invert elevation of each of the outlets for the dam. These could include drop inlets, horizontal pipes, and weirs.

4) Channel and flood plain cross-sections and roughness characteristics.

Cross-section data should include the location and elevation of the top and bottom of the banks of the channel and break points across the flood plain to a point with an elevation approximately half the channel depth above the top of channel bank. In addition, either a description should be written, photograph taken, or Manning's n estimated for the main channel and the flood plain on each side of the channel.

5) Impervious area.

Field measurements of impervious surfaces can be made. Such measurements might include lengths and widths of driveways, streets, roofs or parking lots. More often, however, the needed areas can be estimated from aerial photographs. On occasion it may be difficult to determine the nature of a surface from the aerial photograph. In such cases, field verification is needed only for the type of surface. Any land development since the aerial photo was taken should be noted on the field trip.

Simulation Runs

The first step in making a simulation run is to code the map and field data. This must be done in accordance with the specifications of Section 5. Examples are shown in Section 4. With the data coded and keypunched, the card deck should be marked on the top with a magic marker and catalogued. Though any number of cataloging schemes could be equally effective, they should include the name of the drainage basin and the location of the downstream point (outlet) for each deck of cards. Additional information such as date coded, land and channel conditions used, streets in the drainage system, and purpose of the simulation run could also be catalogued.

The first task in simulation is to debug the collected data by setting RUNOPT equal to 2 and running with only one selected storm. The storms available on the runoff file are listed in Table 33. Although any storm on the runoff file could be used, selection of a storm from the first three years minimizes computer time. For this reason, a storm dated May 6, 1919, March 12, 1920, or August 15, 1920 should be selected. Code for the routing model was written to check the input data for errors and write messages identifying the errors. The various messages are discussed at the end of Section 7. The simulation runs with RUNOPT equal to 2 should be repeated until all model identified errors have been eliminated.

In the debugging, option 1 should be selected so that hydrographs at several points in the drainage systems will be plotted on the printout. Inspection of these hydrographs can reveal additional errors such as an improper time step or improper values describing the channel, area or storage segments. If the hydrographs rise and fall too fast, a shorter time step may be required. The peak flow should be no more than 50% greater than the second highest plotted value, and 25% would be even better. The selection of the time interval must balance two factors. First, too long a time interval reduces accuracy. Second, too short a time interval may reduce the total storm simulation period to less than the interval needed for the rising limb of the hydrograph to reach a peak flow. As a general rule, the time interval should approximate the average of the travel times through the channel segments and should be less than the storage constants listed for the subarea segments. Both the travel times and storage constants are calculated by the routing model and listed in the summaries on the printout.

The second set of simulation runs should set RUNOPT equal to 1 to utilize the entire runoff file and estimate flood frequencies for existing watershed and channel conditions. Outputoptions 16 and 17 should be selected to get some additional output. These two options may be omitted in subsequent runs. Output from the second set of simulation runs may include warning messages that the hydrograph was still rising at a given location for a specified storm at the end of the storm simulation period. Data from these warning messages should be checked with the output given from output option 16 to see whether any of the listed storms may have been critical for the given water year. In most cases they are not. If they include a critical storm for a given water year, a larger time step with the associated runoff file must be used.

A third set of simulation runs are made when the hydrologic effects of projected land use or alternative management schemes are to be studied. In such cases it is most efficient to pick a range of storm events from the results of the second set of simulation runs. Typically this would include a small, medium and large storm event. With RUNOPT equal to 2 and with 3 to 5 storms specified, management schemes can be checked with much less computer time. Also, with only 3 to 5 storms selected, more output on each storm can be requested.

When the more effective management schemes have been identified from the third set of simulation runs, a fourth set of simulation runs can be made for a final evaluation of those schemes. This final run would use the entire runoff file.

In no way is the user of the routing model required to take the above approach. With experience, simulation set 1 and 3 might be eliminated. Also, the routing model has the capability of accepting precipitation values and loss coefficients. This capability could be used for verification runs where streamflow data is available for duplicating specific historical storms or for simulation of runoff from a maximum possible precipitation event.

SECTION 4

Specifications for Input Data

Introduction

This section describes the format for coding the input data to UROS4. Listed at the end of this section are the required variables for each data card, the format and a brief explanation. After a user becomes familiar with the routing model, the listing will be the most frequently referenced section of the report. An additional listing of the definition of variables used in the report and in the code of the routing model can be found in Section 6.

Explanation of Input Requirements

The input specifications at the end of this section have three columns. Column one lists the variables to be read on each card. Column two gives the format for each variable, and column three has a brief definition. The variable names, with a couple of minor exceptions, are the ones used in the Fortran code for the routing model. Format specifications are in standard Fortran notation. Three letters are used in the format and are defined as follows:

A - Any alphanumeric character found on the keypunch

I - Integer number (whole number) with no decimal point and right justified

F - Floating point number (decimal number) with a decimal point.

Numbers to the right of the letter specify field size which is the number of columns on the card for the datum. Numbers to the left of the letter specify a multiplier when the field specification is to be repeated. Commas separate field specifications. If no number appears to the left of a letter, then a value of one is assumed. To the right of the letter F are two numbers separated by a decimal point. The first of these two numbers is the field size and the second is the number of digits to the right of the decimal point. The second number is not used when the decimal point is punched in the input data, and this practice is highly recommended. Thus, all data with the F format should have a punched decimal point while all data with the I format must not have a decimal point. All numbers with the I format must have the last digit coded in the right most column of the field. All numbers with the F format and the decimal point may be coded and punched in any of the columns within the specified field.

The specifications for each data card are listed sequentially. In some cases a statement identified by asterisks precedes the specifications for a card. This may be a general information statement on the purpose for the card or a conditional statement indicating when the card is needed. The conditional deck statements are based on values read for options on the preceding cards. Thus, the input data must be developed from top to bottom with each conditional statement checked before data for the next card is coded. A more detailed explanation of each card follows.

Card 1 is always the first card of a data deck. The first six columns are reserved for specification of the number of runoff files to be used. These columns will usually be left blank since a blank defaults to the typical value of 2 representing one soil type plus impervious area. However, if only one runoff file is needed, such as a watershed with all impervious or no impervious area and only one soil type, a 1 must be coded in column six. Also, if 3 or all 4 runoff files are needed, a 3 or 4 must be coded in column 6. The last two columns (79 and 80) are reserved for the run option which was discussed in the previous section. The columns in between (7-78) are available for any information desired on the headings of tables on the printout. This information generally includes the name of the watershed and some notation on the purpose of the run to distinguish it from the printout from all the other runs. Care and effort in specifying this information is well spent.

Card 2 is always the second card of a data deck. It gives the overall specifications for the run. NELMTS comes from the highest number used from numbering all the segments of the drainage area under study. With RUNOPT = 0 for verification runs, any total time may be used as long as it can be divided evenly by the time increment. With RUNOPT = 1 or 2, only 3 time increments are acceptable, 1-minute, 5-minute, and 15-minute. The total time in all cases must be some multiple of the time increment and the multiple must be less than 121. Some computer time can be saved when using RUNOPT = 1 if the total time is less than 120 times the time increment. However, one should be cautious in using a smaller number because the chances are greater for terminating the calculations for a particular storm before the peak flow has been reached. Numbers for the remaining variables on the card should not be coded unless they are completely understood and a reason exists for a value other than the default value. Because the time and area units have default values, only the first three variables usually are needed. With RUNOPT = 1 or 2, the time unit must be coded with a value of 60. Area units may be coded 1 or 640 for acres or square miles, respectively.

Card 3 is always required and is the first card of a series of cards specifying the characteristics of each segment. It always begins a group of cards needed for each segment and has the same format for each type of segment. Information on this card links the flows between segments and associates the segment number with the type of segment.

Cards 4 and 5 are preceeded by conditional statements and follow card 3 only when card 3 has the segment coded as a subarea. Either card 4 or card 5 is used, but not both. If a verification run or maximum possible precipitation run is being made with RUNOPT = 0 then card 4 is used. Values for the parameters may be estimated by a series of trials when streamflow data is available or be approximated from the values given in tables in Section 2. Information required on card 5 was discussed in Section 2. If the area specified is greater than 10 acres, values for LENGTH, SLOPE and FF should not be coded. If the area is less than 10 acres, LENGTH, SLOPE and FF may be specified for more accurate overland flow routing. The accuracy is gained at the expense of more computer time. When they are not specified or the area exceeds 10 acres, the linear storage model is used. The last four variables on card 5 specify the fractions of each subarea for each runoff file and they must sum to 1.0. If only 2 runoff files are used (usually SOIL 2 and SOIL 4), then only two fractions should be coded.

Cards 6 and 7 are preceded by conditional statements and follow card 3 only when card 3 has the segment coded as a channel. Card 7 is the additional card needed for an irregular cross-section with information on the channel and flood plain. Each listed characteristic is discussed in Section 2.

Cards 8 through 15 are preceded by conditional statements and follow card 3 only when card 3 has the segment coded as a storage. Since several options are available for describing storage segments, several cards are listed.

Card 8 is always required for a storage segment, but if default values are desired and optional output is not, the card can be left blank. Values for NU, SK and SX are needed only for Muskingum channel routing, which is not recommended for use and has not been thoroughly checked. If values are coded for SX and SK, cards 9 through 15 are not required. A value of 1 can be coded in column 28 if additional output on each type outlet structure is desired. This option should be selected on the first set of simulation runs to help check for input errors. It may be left blank in subsequent runs. A value of 1 or 2 should be coded in column 4, depending on the units desired for the data in cards 10, 11 and 12.

Card 9 has two variables for options that can be coded in any combination. Usually KD and KE will both be set to zero for a user calculated storage-discharge relation or both be set to one to minimize calculations required by the user.

Cards 10, 11, 12, 13, 14 and 15 depend on the KD and KE values of card 9 and in no case would values on all six cards be coded. The possible combinations are:

1) Cards 10, 11 and 13
2) Cards 10, 12 and 13
3) Cards 12 and 13
4) Cards 10, 11, 14 and 15
5) Cards 10, 12, 14 and 15.

Note that in only one combination is card 10 not required. Also the slash, /, in the format specifications means the following specifications are for a second card. Thus, cards represented by the numbers 9, 10, 11 and 12 refer to paired cards. For each storage segment, the values for K, N and L must be the same and that value must be greater than one and less than 21. The first value for surface water elevations or reservoir surface areas may be non-zero but the first value for reservoir volume or outflow must be zero.

Card 15 describes the outlet structures, and the number of structures is specified on card 14. Any number of structures may be specified with the usual being 1 to 5. Although a rectangular broad-crested weir is listed as an optional structure, the trapezoidal weir option includes a rectangular or triangular weir.

Card 16 is always required though it may be left blank. It follows the number 3 card which has a value equal to NELMTS + 1 for N, a 99 for TYPE, and a blank for INTO. Proper selection of options is important so that enough output is received without wasting paper. With all options selected and RUNOPT equal to 1, thousands of pages of output could be generated. For the four sets of simulation runs discussed in Section 4 the following options are suggested.

Simulation Set	Recommended Options
1	1,3,14
2	16,17
3	1,3,14
4	16,17

Option 4 is not available for the DeKalb County computer. Option 5 and 18 can be used when rainfall and infiltration parameters are provided. Option 8 can be used to make calibration and verification studies easier. Options 19 and 20 are used with RUNOPT = 0 to compare two simulation runs.

Card 17 is always required and is used to adjust the amount of output since information for all segments usually is not needed. Before a 9999 is coded in the first 4 columns, an estimate of the pages of output should be made.

Cards 18 and 29 cannot be used on computer systems that do not have Calcomp plotters and associated software.

Card 19 is optional and can be used in calibration and verification studies. The four values are used to replace the values read on cards 4 and 6. If values for any of the four variables are left blank, the original values on cards 4 and 6 will remain valid.

Cards 20 and 21 or 22 are needed only when RUNOPT = 0. Note that a card with 9's in the proper columns is needed to tell the programs to quit reading precipitation data cards.

Card 23 is only required for RUNOPT = 0 and cards 24 and 25 are conditional depending upon the values of variables on card 23. As with the precipitation data, the streamflow data has two different formats that can be selected.

Cards 26 and 27 are conditional. They only apply when RUNOPT = 0, option 20 is selected, and a previous simulation has been made with option 19.

Card 28 is optional and should be coded only when RUNOPT = 0 and output option 18 selected. Its use is only for comparing flows at an upstream point with a downstream point. It may work with RUNOPT = 2, a combination which has not been tested.

Cards 30 and 31 are required when RUNOPT = 1. There is a slash in the format so a total of 4 cards are required for each segment listed on card 17. Values for J and K on cards 30 and 31 must be the same, respectively. If flood elevation data is not desired, the four cards are still required, but values for K, DISCH and STAGE may be left blank. Segment numbers, J, are always required. Values for stage can be elevations above mean sea level or elevations above the invert of the channel.

Card 32 is preceded by a conditional statement and is only used when RUNOPT = 2. Any number of cards up to 99 may be coded with storm dates from the runoff file which are listed in Table 33 in Section 3. If program control is to return to card 1 for another simulation run, values of 99 for I1 and I2 must be coded. If not, the date of the last storm desired should be the last card coded.

Table 33. Dates of Storms on the 1-Minute Runoff File

Year	Month	Day
1918	1	11
1918	4	7
1918	11	28
1918	12	21
1919	5	7
1919	7	9
1919	12	9
1920	2	3
1920	3	12
1920	8	15
1920	11	15
1921	2	7
1921	8	24
1921	11	16
1922	2	14
1922	3	10
1922	5	3
1922	6	1
1922	7	19
1922	10	6
1923	2	13
1923	3	12
1923	5	15
1923	12	4
1924	4	4
1924	4	18
1924	5	27
1924	7	3
1924	12	8
1925	1	11
1925	1	17
1925	11	1
1926	1	4
1926	1	17
1926	2	18
1926	3	30
1926	8	11
1926	11	15
1927	2	13
1927	2	23
1927	12	2
1928	5	20
1928	5	23
1928	6	13
1928	7	10
1928	8	15
1929	3	4
1929	3	14
1929	3	23
1929	9	25
1929	10	1
1930	1	28
1930	3	7
1930	5	17
1930	11	16
1931	7	28
1931	8	10
1931	12	4
1931	12	7
1932	2	21
1932	6	18
1932	10	16
1932	12	17
1932	12	27
1933	3	19
1933	6	10
1934	2	25
1934	3	3
1934	7	19
1934	10	6
1935	3	5
1935	9	10
1935	11	12
1936	1	2
1936	2	3
1936	4	6
1936	9	29
1937	4	29
1937	6	17
1937	10	18
1938	4	1
1938	4	6
1938	4	8
1938	6	25
1938	9	27
1938	11	16
1939	2	18
1939	6	22
1939	8	17
1940	7	8
1940	8	12
1940	9	10
1941	6	23
1941	8	13
1941	12	23
1942	2	16
1942	3	2
1942	3	20
1942	9	26

Summary of Requirements

```
C
C
C                    URBAN FLOOD SIMUALTION MODEL (URUS4)
C
C                    DEVELOPED FOR DEKALB COUNTY, GEORGIA
C
C                 PROGRAMMED BY ALAN M. LUMB , ALLEN E. JOHNSON,
C                      L.D. JAMES AND J.L. KITTLE, JR.
C
C                    ***** INPUT REQUIREMENTS *****
C                    -----------------------------
C
C NUM  VARIABLE ON CARD        FORMAT          COMMENT
C -----------------------    ---------   --------------------------------
C
C(1) INFO(1),                 I6,12A6,  NUMBER OF RUNOFF FILES TO BE USED
C     INFO( ),                I2        (IF LEFT BLANK, 2 ASSUMED),
C     RUNOPT                            GENERAL INFORMATION ON RUN
C                                       (NAME OF WATERSHED, PURPOSE OF RUN),
C                                       RUNOPT=0 SINGLE STORM SIMULATION
C                                       RUNOPT=1 USE RUNOFF FILE
C                                       RUNOPT=2 SELECTED STORMS FROM
C                                                RUNOFF FILE
C
C(2) NELMTS,TOTALT,           I4,2F4.0, NUMBER OF LAST REAL SEGMENT(MAX.99),
C     DELT,TUNITS,AU,         2I4,4F8.0 TOTAL TIME, TIME INCREMENT,
C     PK,OFCHK,STCHK                    ((TOTAL TIME)/(TIME INCREMENT)MUST
C                                       BE LESS THAN 121),
C                                       TIME UNITS(SEC=1 OR MIN=60 OR
C                                       HRS=3600), AREA UNITS(ACRE=1 OR
C                                       SQMI=640) (IF TUNITS AND AU LEFT
C                                       BLANK, MINUTES AND ACRES ASSUMED),
C                                       COEFFICIENT FOR LINEAR STORAGE
C                                       ROUTING OF EXCESS FROM AREA SEGMENTS
C                                       (LEAVE BLANK FOR DEFAULT VALUE OF 1.0),
C                                       ACCURACY OBJECTIVE FOR ITERATIVE
C                                       APPROXIMATIONS FOR OVERLAND FLOW
C                                       AND STORAGE SIMULATION (LEAVE BLANK).
C
C                                       NOTE: WHEN USING RUNOFF FILE
C                                             DELT MUST BE 1,5 OR 15 MINUTES
C                                             DEPENDING ON FILE USED.
C
C
C******* INPUT PHYSICAL DISCRIPTORS OF EACH SEGMENT (LOOP NELMTS TIMES)
C
C(3) N,TYPE( ),               3I4       SEGMENT NUMBER (SEQUENCE OF
C     INTO( )                           NUMBERS FROM 1 TO 99, SMALLER
C                                       NUMBERS MUST ALWAYS DRAIN TO A
C                                       LARGER NUMBER),SEGMENT TYPE
C                                       (AREA=11,CHANNEL=22,STORAGE=33,
C                                       DUMMY = 55, QUIT=99) , SEGMENT
C                                       NUMBER INTO WHICH THIS SEGMENT
C                                       DISCHARGES.
C
```

```
C                                         NOTF: NUMBERS MAY BE SKIPPED-
C                                               FITHER IGNORE OR SET TYPE = 55
C                                         NOTE: AT END OF DISCRIPTOR CARDS
C                                               PUT SEGMENT WITH N = NELMTS+1
C                                               AND TYPE( ) = 99
C                                         NOTE: LAST REAL SEGMENT MUST
C                                               DISCHARGE INTO SEGMENT
C                                               NELMTS+1.
C
C     *** IF TYPE( ) = AREA (11) AND RUNOPT = 0 INPUT THE FOLLOWING
C
C(4)  AREA( ),LENGTH( ),        9F8.0     DRAINAGE AREA,LENGTH(FT),SLOPE
C     SLOPE( ),FF( ),                     (FT/FT),MANNING'S N,INITIAL LOSS IN
C     STRKR( ),RTIOL( ),                  INCHES/DELT(ANYTHING FROM 0.0 TO
C     ERAIN( ),DLTKR( ),U( )              10.0 DEPENDING UPON ANTECEDENT
C                                         RAINFALL), SLOPE OF LOSS
C                                         FUNCTION (RATIO OF LOSS
C                                         TO THAT AFTER 10 INCHES OF RECHARGE
C                                         -USUALLY 3.0), RAIN EXPONENT
C                                         (0.7 FOR TYPICAL VALUE,
C                                         OR 1.0 FOR RATIONAL FORMULA
C                                         TYPE PERCENTAGE LOSS, OR 0.001
C                                         FOR INFILTRATION FCN TYPE LOSS),
C                                         SURFACE RETENTION VOLUME(INCHES)
C                                         (ANYTHING FROM 0.0 TO 2.0 DEPENDING
C                                         UPON ANTECEDENT RAINFALL),
C                                         FRACTION IMPERVIOUS AREA.
C
C                                         NOTE: IF AREA OVER
C                                             TEN ACRES OR LINEAR
C                                             STORAGE DESIRED FOR ROUTING
C                                             TO OUTLET, LEAVE LENGTH, SLOPE,
C                                             AND FF( ) BLANK.
C
C     *** IF TYPE( ) = AREA(11) AND RUNOPT=1 OR 2 INPUT THE FOLLOWING
C
C(5)  AREA( ),LENGTH( ),        4F8.0,8X, AREA,LENGTH,SLOPE AND FF SAME AS
C     SLOPE( ),FF( ),           4F8.0     ABOVE, SOIL1 THRU SOIL4 ARE
C     SOIL1( ),SOIL2( ),                  FRACTION OF AREA WITH RAPID,
C     SOIL3( ),SOIL4( )                   MODERATE,SLOW OR ZERO SOIL
C                                         PERMEABILITIES,RESPECTIVELY.
C                                         (ZERO PERMEABILITIES FOR IMPERVIOUS
C                                         SURFACES)
C
C                                          NOTE: SUM(SOIL1+SOIL2+SOIL3+SOIL4)
C                                                MUST=1.0 FOR EACH SUBWATERSHED
C
C     *** IF TYPE( ) = CHAN (22) INPUT THE FOLLOWING INFORMATION
C
C(6)  KIND( ),LENGTH( ),        I8,9F8.0  KIND OF CHANNEL(0,1,2,3 OR 4 AS
C     SLOPE( ),FF( ),                     DEFINED BELOW), LENGTH OF CHANNEL
C     APARM( ),MPARM( )                   (FEET), SLOPE OF CHANNEL(FT/FT),
C                                         MANNING'S 'N' FOR CHANNEL,
C                                           KIND OF CHANNEL OPTIONS:
C                                         0=IRREGULAR, LEAVE SLOPE AND FF
C                                           BLANK, FLOOD PLAIN AND CHANNEL
C                                           FLOW SIMILAR, INPUT VALUES FOR
```

```
C                                              APARM AND MPARM
C                                            1=RECTANGULAR, LEAVE MPARM BLANK
C                                              AND SET APARM EQUAL TO CHANNEL
C                                              WIDTH
C                                            2=TRIANGULAR, LEAVE MPARM BLANK
C                                              AND SET APARM EQUAL TO WIDTH
C                                              AT ONE FOOT DEPTH
C                                            3=CIRCULAR, LEAVE MPARM BLANK AND
C                                              SET APARM EQUAL TO PIPE DIAMETER
C                                              (SEGMENT SHOULD BE PRECEEDED
C                                              BY A STORAGE SEGMENT WITH THE
C                                              OUTLET OF THE STORAGE SEGMENT THE
C                                              SAME AS INLET TO THE PIPE.
C                                              CHANNEL SEGMENT NOT NEEDED FOR
C                                              SHORT PIPES AS UNDER ROADS.)
C                                            4=IRREGULAR, FLOOD PLAIN AND
C                                              CHANNEL FLOW DIFFERENT, APARM IS
C                                              FOR CHANNEL CROSS-SECTION AREA,
C                                              MPARM IS FOR WETTED PERIMETER
C                                              OF THE CHANNEL
C                                          (ALL UNITS IN FEET)
C
C     *** IF TYPE( ) = CHAN(22) AND KIND( ) = 4 INPUT THE FOLLOWING
C
C(7)  FRN,ART,WPR,              7F8.0      MANNING'S N FOR RIGHT FLOOD PLAIN,
C     FNL,ALT,WPL,ACX                      CROSS-SECTION AREA RIGHT FLOOD PLAIN,
C                                          WETTED PERIMETER RIGHT FLOOD PLAIN,
C                                          MANNING'S N LEFT FLOOD PLAIN,
C                                          CROSS-SECTION AREA LEFT FLOOD PLAIN,
C                                          WETTED PERIMETER LEFT FLOOD PLAIN,
C                                          EXTRA CROSS-SECTION AREA ABOVE
C                                          CHANNEL WHEN FLOW IN FLOOD PLAIN.
C                                          (ALL UNITS IN FEET OR SQUARE FEET)
C
C                                           NOTE: RIGHT AND LEFT LOOKING
C                                                 DOWNSTREAM
C
C                                           NOTE: FOR CROSS-SECTION, PICK
C                                                 ANY FLOOD LEVEL SUCH THAT
C                                                 FLOW EQUALS ABOUT THE
C                                                 50-YEAR FLOOD OR DEPTH
C                                                 IN FLOOD PLAIN AT LEAST
C                                                 SEVERAL FEET OR ONE-HALF
C                                                 THE VERTICAL DISTANCE FROM
C                                                 CHANNEL BOTTOM TO THE TOP
C                                                 OF THE BANK.
C
C                    XX                                                  XX
C                     XX--------------------------------------------------XX
C                      XX        I                          I          XX
C                       XX  ALT  I            ACX           I  ART    XX
C                        XX      I                          I        XX
C                         XXXXXXXXX---------------------------XXXXXXXXX
C                            WPL  XX                       XX WPR
C                            FNL   XX    AC(APARM)        XX  FNR
C                                   XX                   XX
C                                    XXXXXXXXXXXXXXXXXXXXX
```

```
C                                                 WPC(MPARM)
C                                                 FNC(FF( ))
C
C
C     *** IF TYPE( ) = STOR (33) INPUT THE FOLLOWING INFORMATION
C
C(8) VU,TU,                    2I4,         VOLUME AND AREA UNITS
C     SK( ),SX( ),I            2F8.0,       (CUBIC FEET AND SQUARE FEET = 1,
C                              I4           ACRE-FEET AND ACRES = 2),
C                                           TIME UNITS FOR SK(SEC=1,
C                                           MIN=60, HOURS=3600),
C                                           (IF TU OR VU LEFT BLANK MINUTES
C                                           AND ACRES ASSUMED),
C                                           MUSKINGUM K AND X FOR CHANNEL
C                                           ROUTING(AVAILABLE FOR USE BUT
C                                           NOT RECOMMENDED, THUS LEAVE CARD
C                                           BLANK FROM COLUMN 5 TO 25),
C                                           SET I = 1 FOR EXTRA OUTPUT ON
C                                           STORAGE SEGMENT(IF NOT WANTED
C                                           LEAVE  COLUMN 28 BLANK).
C
C     ***IF TYPE( ) = STOR(33) AND SK AND SX ARE BLANK INPUT THE FOLLOWING
C
C(9) KD,KL                     2I4          KD=0, INPUT RESERVOIR VOLUMES
C                                           KD=1, INPUT RESERVOIR SURFACE
C                                                 AREAS AND ELEVATIONS
C                                           KE=0, INPUT RESERVOIR OUTFLOWS
C                                           KE=1, INPUT DISCRIPTION OF
C                                                 OUTLET WORKS
C
C     *** IF TYPE( ) = STOR(33) AND EITHER KD OR KE = 1 INPUT FOLLOWING
C
(10) K,SELEV( )                I8/          NUMBER OF VALUES(MAX.20),
C                              (10F8.2)     WATER SURFACE ELEVATIONS(FEET).
C
C     *** IF TYPE( ) = STOR(33) AND KD = 0 INPUT THE FOLLOWING
C
(11) N,VOL( )                  I8/          NUMBER OF VALUES(MAX.20),
C                              (10F8.2)     RESERVOIR VOLUMES (FIRST
C                                           VALUE MUST BE ZERO)
C
C     *** IF TYPE( ) = STOR(33) AND KD = 1 INPUT THE FOLLOWING
C
(12) N,AREA( )                 I8/          NUMBER OF VALUES(MAX.20),
C                              (10F8.2)     RESERVOIR SURFACE AREAS
C
C     *** IF TYPE( ) = STOR(33) AND KE = 0 INPUT THE FOLLOWING
C
(13) L,OTS( )                I8/          NUMBER OF VALUES(MAX.20),
C                              (10F8.2)     RESERVOIR OUTFLOWS(CFS).
C
C                                           NOTE: K,N AND L FROM ABOVE CARDS
C                                                 MUST BE THE SAME
C
C     *** IF TYPE( ) = STOR(33) AND KE = 1 INPUT THE FOLLOWING
C
(14) JA                           I4            NUMBER OF OUTLET STRUCTURES
```

```
C
C     *** INPUT FOR 1 TO JA OUTLET STRUCTURES
C
(15) JT,JEL,JS1,JS2              I4,      TYPE OF OUTLET
C                                3F8.3     JT=1 FOR CIRCULAR PIPE
C                                          JT=2 FOR BOX CULVERT
C                                          JT=3 ELLIPTICAL PIPE
C                                          JT=4 RECTANGULAR BROAD-CRESTED WEIR
C                                          JT=5 TRAPEZOIDAL BROAD-CRESTED WEIR
C                                          JT=6 CIRCULAR DROP INLET
C                                         ELEVATION OF BASE OF OUTLET(FEET),
C                                         FOR JT = 1, 2 OR 3
C                                          JS1 = VERTICAL DIMENSION(FEET)
C                                          JS2 = HORIZONTAL DIMENSION(FEET)
C                                         FOR  JT = 4
C                                          JS1 = WIDTH OF WEIR(FEET)
C                                          JS2 = (LEAVE BLANK)
C                                         FOR  JT = 5
C                                          JS1 = WIDTH AT CREST(FEET)
C                                          JS2 = ANGLE OF ONE SIDE FROM VERTICAL
C                                                IN DEGREES
C                                          (JS1 = 0 FOR TRIANGULAR)
C                                          (JS2 = 0 FOR RECTANGULAR)
C                                         FOR  JT = 6
C                                          JS1 = DIAMETER OF PIPE(FEET)
C                                          JS2 = (LEAVE BLANK)
C
C
C******  END SEGMENT LOOP WITH TYPE( ) = QUIT (99)
C
C
(16) OPT( )                      20I4     OUTPUT/INPUT OPTIONS
C                                         NOTE: IF RUNOPT EQUALS 1 ONLY 16
C                                               AND/OR 17 SHOULD BE SELECTED
C                                               OR TONS OF OUTPUT WILL BE
C                                               PRODUCED
C                                           1 = PLOT HYDROGRAPHS ON PRINTER
C                                               (FOR RUNOPT = 0 OR 2
C                                               FOR SEGMENTS IN OUPT LIST)
C                                           2 = PRINT DETAILED HYDROGRAPHS
C                                               (NOT NEEDED IF 1 SELECTED)
C                                           3 = PRINT MAX.FLOWS, ALL POINTS
C                                               (FOR RUNOPT = 0 OR 2)
C                                           4 = PLOT HYDROGRAPHS ON CALCOMP
C                                           5 = OUTPUT RAIN, RUNOFF VOLUMES
C                                               (FOR RUNOPT = 0 ONLY AND
C                                               FOR SEGMENTS IN OUPT LIST)
C                                           6 = OUTPUT MAXIMUM STORAGE
C                                               (FOR RUNOPT = 0 OR 2)
C                                           8 = CHANGE DELTKR, STRKR, APARM AND
C                                               MPARM(FOR RUNOPT = 0 ONLY)
C                                           9 = PRINT PEAKS, SELECTED POINTS
C                                               (FOR RUNOPT = 0 OR 2
C                                               FOR SEGMENTS IN OUPT LIST,
C                                               NOT NEEDED IF 3 SELECTED)
C                                          14 = INFLOW,OUTFLOW,STORAGE TABLES
C                                               (FOR RUNOPT = 0 OR 2
```

```
C                                                 FOR SEGMENTS IN OUPT LIST)
C                                             16 = LIST ALL PEAKS FROM RUNOFF FILE
C                                                  (FOR RUNOPT = 1 ONLY)
C                                             17 = PRINT STAGE-DISCHARGE TABLES
C                                                  (FOR RUNOPT = 1 ONLY)
C                                             18 = COMPARE DISCH AT TWO POINTS
C                                                  (FOR RUNOPT = 0 ONLY)
C                                             19 = OUTPUT DISCHARGE AT LAST PT
C                                                  (FOR RUNOPT = 0 ONLY)
C                                                  (WRITTEN ON FILE 10)
C                                             20 = INPUT FROM PREVIOUS SIM.
C                                                  (FOR RUNOPT = 0 ONLY)
C
C
(17) OUPT( )                  20I4            SEGMENT NUMBERS OUTPUT DESIRED
C                                             (IF MORE THAN 20 DESIRED,PUT 5555
C                                             IN COLUMNS 77 THRU 80 AND THE
C                                             ADDITIONAL SEGMENTS NUMBERS ON THE
C                                             NEXT CARD. IF OUTPUT IS DESIRED ON
C                                             ALL SEGMENTS CODE 9999 IN 1 THRU 4.)
C
C     *** IF OUTPUT/INPUT OPTION 4 SELECTED, INPUT THE FOLLOWING CARD
C
(18) FACT,LDEV,               F4.0,I4,        SCALE FACTOR(USUALLY 1.0 FOR A
C    TIMX,ISPEC,              F4.0,I4         10 INCH PLOT),LOGICAL UNIT NUMBER
C    TFACT                                    (ANY NUMBER 1 TO 100),MAXIMUM
C                                             PLOT TIME IN MINUTES, SPECIAL
C                                             PEN OR PAPER OPTIONS(LEAVE BLANK
C                                             OR REFER TO CALCOMP PLOTTER
C                                             MANUAL),SCALE FACTOR ON TIME AXIS
C                                             (IF BLANK,1.0 ASSUMED).
C
C     *** IF OUTPUT/INPUT OPTION 8 SELECTED, INPUT THE FOLLOWING CARD
C
(19) DLTKR,STRKR,             4F8.0           REPLACEMENT VALUES FOR PERVIOUS
C    APARM,MPARM                              AREA HFC-1 PARAMETERS DLTKR AND
C                                             STRKR IF NON-ZERO, REPLACEMENT
C                                             VALUES FOR APARM AND MPARM IF
C                                             NON-ZERO
C
C     *** INPUT PRECIPITATION DATA IF RUNOPT = 0
C
(20) IFMTS,INC,ISTRT          3I4             FORMAT CODE NUMBER(1 OR 2),(SET
C                                             IFMTS=3 FOR DEFAULT 10-YEAR STORM
C                                             OR IFMTS=4 FOR DEFAULT 100-YEAR
C                                             STORM), TIME STEP(SAME UNITS AS
C                                             DELT AND MUST DIVIDE EVENLY INTO
C                                             DELT),
C                                             STARTING TIME INCREMENT(2-12) ON
C                                             FIRST PRECIP CARD IF OTHER THAN
C                                             FIRST VALUE OTHERWISE LEAVE BLANK
C
C      *** IF IFMTS = 1 INPUT FOLLOWING CARD
C
(21) NOSTA,YR,MO,             I6,5I2,         STATION NUMBER,YEAR,MONTH,DAY,
C    DAY,HR,MIN,INC,          F4.1,           HOUR,MINUTE,TIME INCREMENT(MIN),
C    APRA( )                  12F5.2          12 PRECIPITATION INCREMENTS.
```

```
C                                          NOTE: LAST CARD MUST HAVE 99 IN
C                                               COLUMNS 9+10 AND NO DATA.
C
C      *** IF IFMTS = 2 INPUT FOLLOWING CARD
C
(22)   NOSTA,TI,TY,YR,           A6,2I2,   STATION IDENTIFICATION, TIME
C      MO,DAY,YR,ARRA( )         2X,4I2,   INCREMENT,CODE TYPE(1=P,2=SF),
C                                12F5.2    YEAR,MONTH,DAY,HOUR,12 RAINFALL
C                                          VALUES.
C                                          NOTE: LAST CARD MUST HAVE 99 IN
C                                               COLUMNS 15+16 AND NO DATA.
C
C      *** IF IFMTS = 3, 10-YEAR STORM ASSUMMED (NOT AVAILABLE)
C
C      *** IF IFMTS = 4, 100-YEAR STORM ASSUMMED (NOT AVAILABLE)
C
C      *** INPUT STREAMFLOW DATA IF RUNOPT = 0
C
(23) NUM,IFMTS,INC,              20I4      NUMBER OF FLOWPOINTS(SEGMENTS)
C    ISTRT,NSD( )                          WITH MEASURED DISCHARGE(MAX.=5),
C                                          FORMAT CODE NUMBER, TIME STEP
C                                          FOR DATA(SAME UNITS AS DELT AND
C                                          MUST DIVIDE EVENLY INTO DELT),
C                                          STARTING TIME INCREMENT(2-12) ON
C                                          FIRST STREAMFLOW CARD IF OTHER
C                                          THAN FIRST VALUE OTHERWISE LEAVE
C                                          BLANK,
C                                          FLOWPOINT(SEGMENT) NUMBERS.
C
C                                          NOTE: IF NO MEASURED FLOW THIS
C                                          CARD SHOULD BE BLANK
C
C      *** IF NUM GT 0 AND IFMTS = 1, INPUT MEASURED DISCHARGE
C
(24) NOSTA,SF( , )               I8,       STATION NUMBER, DISCHARGE DATA
C                                12F6.0    FOR EACH STATION FOR TOTAL TIME
C                                          AND TIME INCREMENT SPECIFIED IN
C                                          INPUT CARD NUMBER 2.
C
C      *** IF NUM GT 0 AND IFMTS = 2, INPUT MEASURED DISCHARGE
C
(25)   NOSTA,TI,TY,YR,           A6,2I2,   STATION IDENTIFICATION, TIME
C      MO,DAY,HR,ARRA( )         2X,4I2,   INCREMENT,CODE TYPE(1=P,2=SF),
C                                12F5.2    YEAR,MONTH,DAY,HOUR,
C                                          12 STREAMFLOW VALUES.
C
C
C    *** IF OUTPUT/INPUT OPTION 20 SELECTED, INPUT FOLLOWING CARDS.
C
(26) V,NU,NE( )                  20I4      I/O FILE NUMBER, NUMBER OF
C                                          INFLOW ARRAYS, NUMBERS OF
C                                          THE FLOWPOINTS (SEGMENTS)
C                                          FOR COMPARISON
C
(27) SF( , )                     20X       NU ARRAYS OF INFLOW DATA FOR
C                                12F5.0    TIME DETERMINED FROM CARD 2.
C
```

```
C
C     *** IF OUTPUT/INPUT OPTION 18 SELECTED, INPUT FOLLOWING CARDS.
C
(28)  NCMP, (N1( ),N2( ))        2014        NUMBER OF PAIRS OF SEGMENTS
C                                              TO BE PLOTTED ON SAME PLOT,
C                                              SEGMENT NUMBERS.  FIRST SEGMENT
C                                              PLOTTED WITH *, SECOND WITH O.
C
C
C     **** IF OPTIONS 20 AND 4 SELECTED, INPUT THE FOLLOWING CARD ****
C
(29)  INFOA( ),                  6A6,4X,     INFORMATION FOR CALCOMP ON CURREN
C     INFOB( )                   6A6         SIMULATION, INFORMATION ON
C                                            PREVIOUS SIMULATION
C
C     *** IF RUNOPT = 0 THEN
C
C         CONTROL RETURNS TO FIRST INPUT CARD TO  START ANOTHER
C         SIMULATION.  TO END RUN PUNCH 999999 IN COLUMNS 1 THRU 6.
C
C     *** IF RUNOPT = 1 THEN INPUT FOLLOWING FOR EACH SEGMENT IN OUPT() LIST
C
(30)  J,K,                       2I4/        SEGMENT NUMBER, NUMBER OF VALUES,
C     DISCH( )                   10F8.0      DISCHARGE VALUES(CFS)
C
(31)  J,K,                       2I4/        SEGMENT NUMBER, NUMBER OF VALUES,
C     STAGE( )                   10F8.0      STAGE(ELEVATION) VALUES(FEET).
C
C     *** IF RUNOPT = 2 THEN INPUT THE FOLLOWING
C
(32)  I1,I2,I3                   3I4         YEAR, MONTH, DAY OF STORM FROM
C                                            RUNOFF FILE TO BE SIMULATED.
C                                            NOTE: ANY NUMBER OF STORMS
C                                                  (ONE PER CARD). NO ADDITIONAL
C                                                  CARDS ENDS SIMULATION
C                                                  OR I1 AND I2 = 99 RETURNS
C                                                  CONTROL TO FIRST CARD.
C
C
C
C
C
C
C
C
```

Computer System Control Cards

The DeKalb Runoff Model has been used on the Univac 1108 with Exec 8 control language and on an IBM 360/40 - DOS. The necessary control cards for each system are described below.

Univac 1108

To make a simulation with RUNOPT equal to 0, the following cards are required:

```
@RUN IIIIII,NNNNNNNN,AAAAA
@PWRD XXXXXX
@ASG,AX CE*DEKALB
@XQT CE*DEKALB.UROS4
  (data card input)
@FIN
```

where,

IIIIII = run identification (bin number and/or initials)
NNNNNNNN = project number
AAAAA = user name
XXXXXX = user passwork

To make a simulation with RUNOPT equal to 1 with all four runoff files, the following cards are required.

```
@RUN IIIIII, NNNNNNNN,AAAAA,TT,PPP
@PWRD XXXXXX
@ASG,AX CE*DEKALB
@ASG,AX CE*ROFQSOIL1
@ASG,AX CE*ROFQSOIL2
@ASG,AX CE*ROFQSOIL3
@ASG,AX CE*ROFQSOIL4
@USE 15,CE*ROFQSOIL1
@USE 16,CE*ROFQSOIL2
@USE 17,CE*ROFQSOIL3
@USE 18,CE*ROFQSOIL4
@ASG,T TEMP,F
@USE 9,TEMP
@XQT CE*DEKALB.UROS4
  (input data)
@FIN
```

where

TT = maximum time allowed for the run
PPP = maximum pages allowed for the run
Q = time increment for run (<u>1</u> min., <u>5</u> min, or <u>15</u> min)

To make a simulation with RUNOPT equal to 2, eliminate the following two cards from the above sequence:

```
@ASG,T TEMP,F
@USE 9, TEMP
```

To make a simulation with less than four runoff files, eliminate the @ASG and @USE for the files not needed. However, the numbering on the @USE cards must begin with 15 and be sequential. Thus, to make a simulation with RUNOPT equal to 2 with two 5-minute runoff files for soil 2 and soil 4, the following cards are required,

```
@RUN IIIIII,NNNNNNNN,AAAAA,TT.PPP
@PWRD XXXXXX
@ASG,AX CE*DEKALB
@ASG,AX CE*ROF5SOIL2
@ASG,AX CE*ROF5SOIL4
@USE,15,CE*ROF5SOIL2
@USE,16,CE*ROF5SOIL4
@XQT CE*DEKALB.UROS4

  (data card input)

@FIN
```

IBM 360-40

To make a simulation with RUNOPT equal to 0, the following cards are required:

```
//JOB RBRBRBRB 1 HYDROLOGICAL SIMULATION
// EXEC HYDROL

  (data card input)

/*
/&
```

To make a simulation with RUNOPT equal to 1 with all four runoff files, the following cards are required:

```
// JOB RBRBRBRB 1 HYDROLOGICAL SIMULATION
// ASSGN SYS008,X'180'
// ASSGN SYS009,X'181'
// ASSGN SYS010,X'182'
// ASSGN SYS011,X'183'
```

```
// LBLTYP TAPE
// TLBL IJSYS08
// TLBL IJSYS09
// TLBL IJSYS10
// TLBL IJSYS11
// ASSGN SYS006,X'184'
// TLBL IJSYS06
// EXEC HYDROL

   (data card input)

/*
/&
```

To make a simulation with RUNOPT equal to 2, eliminate the following two cards from the above sequence:

```
// ASSGN SYS006,X'184'
// TLBL IJSYS06
```

To make a simulation with less than four runoff files, eliminate the // ASSGN and // TLBL for the files not needed. However, the numbering on the ASSGN cards must begin with SYS008 and be sequential. Thus, to make a simulation with RUNOPT equal to 2 with two runoff files the following cards are required,

```
// JOB RBRBRBRB 1 HYDROLOGICAL SIMULATION
// ASSGN SYS008,X'180'
// ASSGN SYS009,X'181'
// LBLTYP TAPE
// TLBL IJSY08
// TLBL IJSYS09
// ASSGN SYS006,X'182'
// TLBL IJSYS06
// EXEC HYDROL

   (data card input)

/*
/&
```

With the job control cards for the IBM, the names of the runoff files must be listed on a note to the operator so that he will mount the proper tapes.

SECTION 5

Sample Simulations

Introduction. As discussed at the end of the Section 3 several types of simulation runs can be made. This section will illustrate the input and output of four separate runs on Wild Creek. The first example uses the existing conditions of the watershed and four storms from the runoff file and the second example uses the entire runoff file for a frequency analysis. The third example also uses the entire runoff file for a frequency analysis except that a detention storage structure has been added. The fourth example illustrates the simulation of a storm event with precipitation and infiltration parameters as added input.

Description of Wild Creek. The examples were taken from a study of Wild Creek using UROS4. Wild Creek is in the west central portion of DeKalb County within an area bounded by LaVista, Briarcliff and North Druid Hills Roads and I-85. Wild Creek flows generally to the west and empties into North Fork Peachtree Creek near the I-85 interchange with Cheshire Bridge Road.

The downstream point of the study was approximately 1000 feet from the outlet to North Fork Peachtree Creek at the driveway leading to Lanier Electronics off Chantilly Road. The area of the watershed above the driveway is 240 acres and almost entirely residential. Slopes of the upstream channels and hillside are quite steep. Numerous roads and driveways cross the stream channels.

Figure 27 shows an outline of the Wild Creek drainage area and each subarea and channel segment. The schematic diagram on Figure 28 shows the linkage of the segments including all the storage segments that were used in the example 3 simulation run. Storage segment 21, however, was not included in example runs 1, 2 and 4.

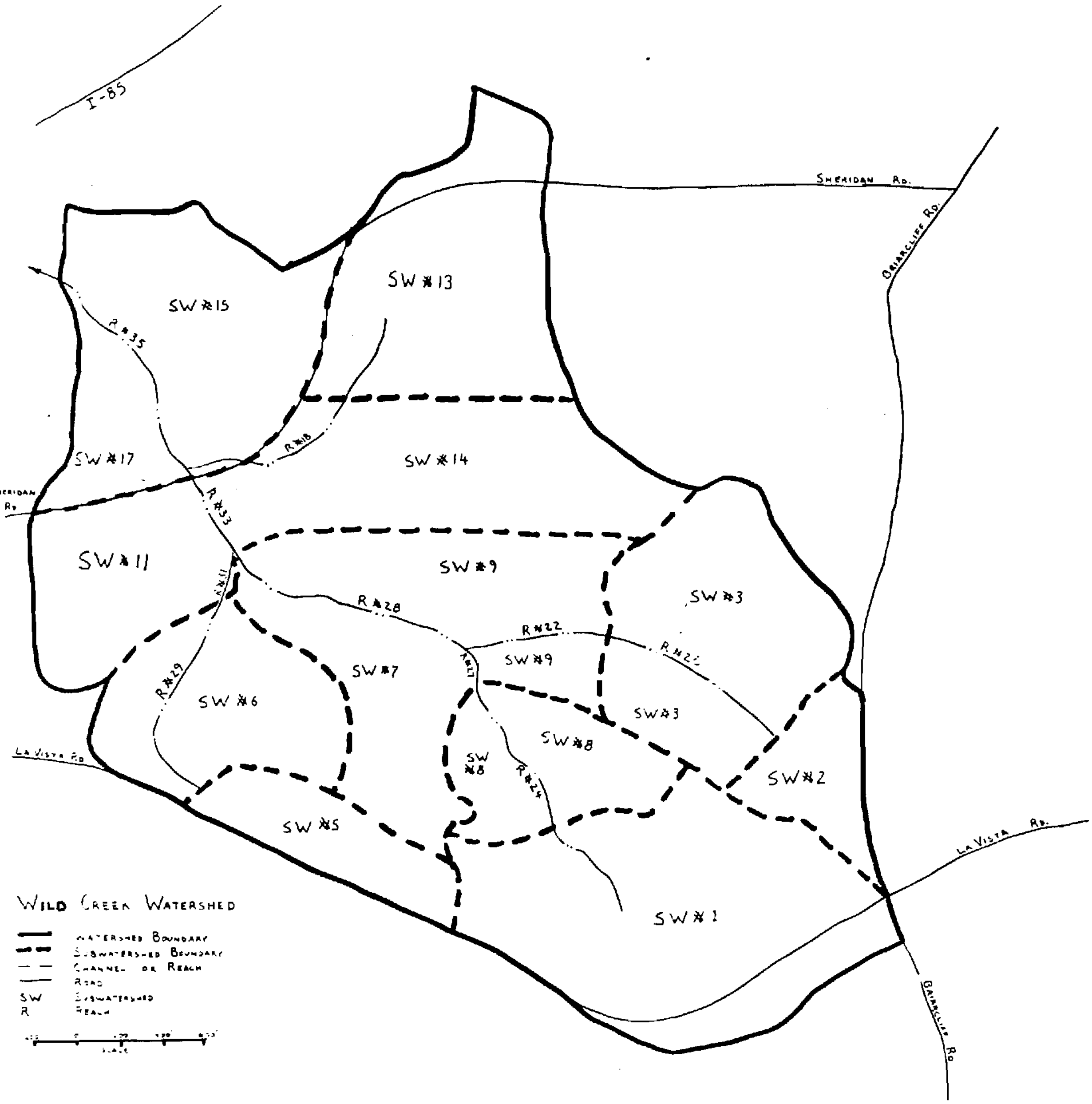


Figure 27. Map of Wild Creek Watershed

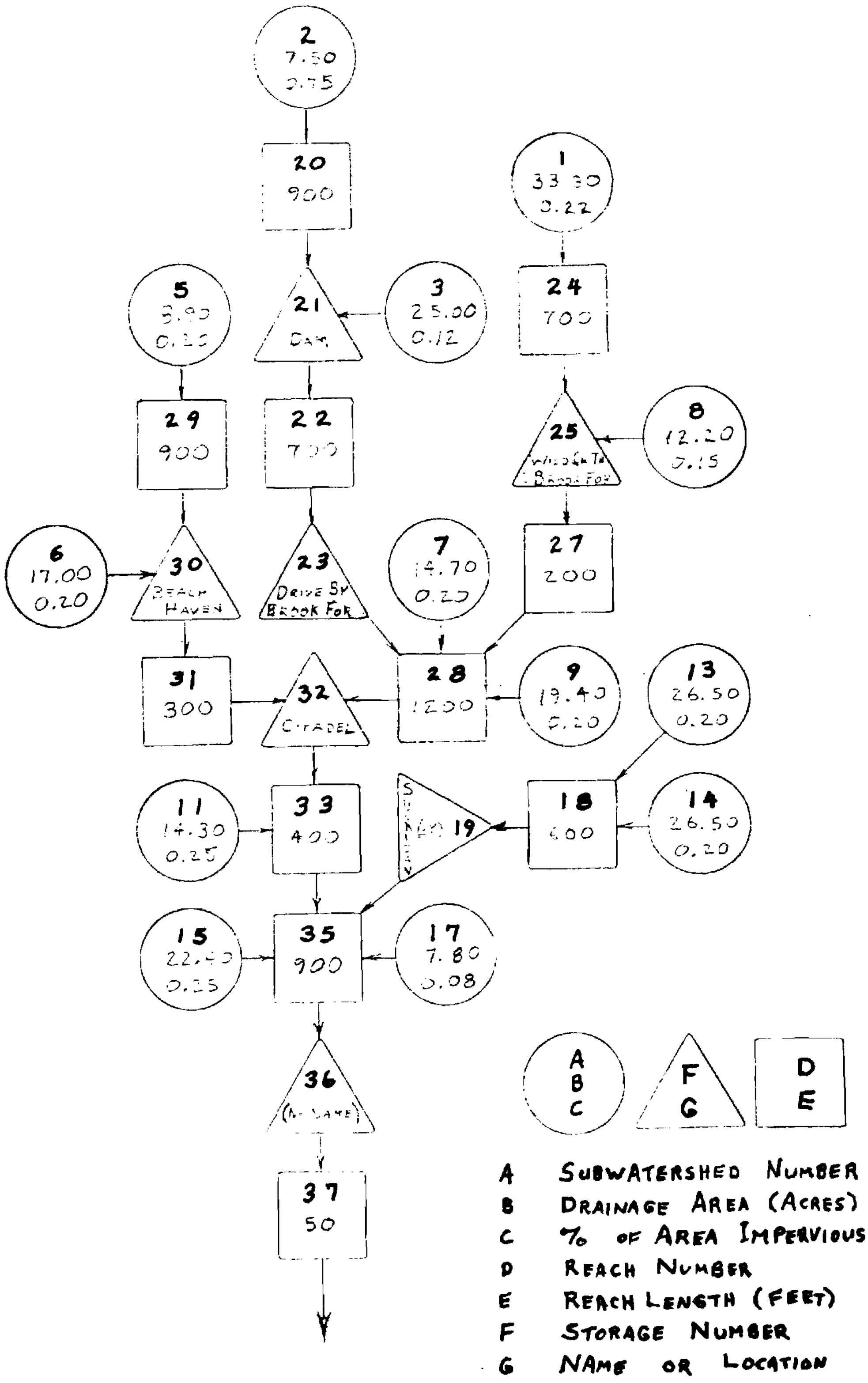


Figure 28. Schematic Diagram of Wild Creek

Input Data. Input data for the four simulation runs are listed in Tables 34 through 37. The only change from the first run to the second is with the first line and the lines following the line for QUIT segment 38. In going from the second to the third run, the only change was the addition of storage segment 21. Input data for the fourth run is similar to the first run except the first card and the last several cards following the output option card.

Output. The number of pages of output generated from the input data listed on Tables 34 to 37 is:

Example Run 1 - 22 pages

Example Run 2 - 66 pages

Example Run 3 - 68 pages

Example Run 4 - 14 pages

The first six pages of output is almost identical for all four example runs and is a summary of the input data describing the segments of the watershed. Tables 38 through 40 show the output from example run 3. Table 41 shows the peak flow table from example run 1 for all segments for the March 12, 1920, storm on the runoff file. Output option 3 gives the peak flow table. Figure 29 shows the flood hydrograph plotted on the printout for segment 35 from example run 4.

When the entire runoff file is used for a frequency analysis, example runs 2 and 3, additional output is received. Table 42 lists some of the warning messages received when the maximum flow was on the rising limb of the hydrograph at the last time step for the storm. The message can be ignored if the storm for the given date was not close to the maximum for the water year. This can be checked with the results from output option 16 as shown on Table 43. Table 44 is a sample of the output from option 17 for example run 4. Tables 45 and 46 show the flood frequency tables for

Table 34. Input Data for Example Run 1

```
   WILD CREEK - STORAGE DEVICE AT 25                                                             2.
36300. 5.0
 1  11  24
  38.3                                                            .78                .22
 2  11  20
   7.5                                                            .25                .75
 3  11  22
  25.0                                                            .88                .12
 5  11  29
   8.9                                                            .80                .20
 6  11  30
  17.0                                                            .80                .20
 7  11  28
  14.7                                                            .80                .20
 8  11  25
  12.2                                                            .85                .15
 9  11  28
  19.4                                                            .80                .20
11  11  33
  14.3                                                            .75                .25
13  11  18
  26.5                                                            .80                .20
14  11  18
  26.5                                                            .80                .20
15  11  35
  22.4                                                            .75                .25
17  11  35
   7.8                                                            .92                .08
18  22  19
     4    600.0    .0285     .050      30.     14.83
   .08      90.      60.      .08      72.       48.       36.
19  33  35
 2
 1   1
     8
 828.0    829.0    830.0    831.0    832.0    833.0    834.0    835.0
     8
   0.0     .074     .294     .413     .574     .792    1.004    1.240
 2
 1   828.0      3.0      3.0
 5   834.0     30.0
20  22  22
     4    900.0    .0611      .05    11.81     10.1
   .08     88.6    26.06      .08     143.    38.55    53.63
22  22  23
     4    700.0    .0143      .05    11.81     10.1
   .08     88.6    26.06      .08     143.    38.55    53.63
23  33  28
 2
 1   1
     8
 845.0    846.0    847.0    848.0    849.0    850.0    851.0    852.0
     8
   0.0     .009     .021     .037     .057     .083     .113     .186
 2
```

Table 34. (Cont'd)

```
  3   845.0     2.0     3.0
  5   850.0    30.0
 24  22  25
      4   700.0   .0807    .050     44.8    19.48
    .08     27.   18.25     .08      36.    24.08      48.
 25  33  27
  2                          1
  1
     13
 836.0   837.0   838.0   839.0   840.0   841.0   842.0   843.0   844.0   845.0
 846.0   847.0   848.0
     13
   0.0    .006    .028    .050    .077    .107    .141    .222    .342    .434
  .550    .650    .750
     13
   0.0     5.8    10.0    14.0    16.9    19.8    22.6    24.6   109.1   207.6
 223.6   239.6   345.5
 27  22  28
      3   200.0   .0500    .025      4.5
 28  22  32
      4  1200.0   .0167    .050      30.    14.83
    .08     90.     60.     .08      72.      48.      36.
 29  22  30
      4   900.0   .0500     .05    11.81     10.1
    .08    88.6   26.06     .08     143.    38.55    53.63
 30  33  31
  2
  1   1
     10
833.0   834.0   835.0   836.0   837.0   838.0   839.0   840.0   841.0   842.0
     10
   0.0    .014    .041    .083    .126    .179    .249    .331    .434    .551
  2
  1   833.0     3.0     3.0
  5   841.0    30.0
 31  22  32
      4   300.0   .0500    .050      33.      17.
    .08     90.     60.     .08     34.5    23.19      42.
 32  33  33
  2
  1   1
     11
 816.0   817.0   818.0   819.0   820.0   821.0   822.0   823.0   824.0   825.0
 826.0
     11
   0.0    .012    .028    .057    .092    .207    .321    .436    .551    .689
  .861
  2
  2   816.0     3.0     8.0
  5   825.0    20.0
 33  22  35
      4   400.0   .0039    .050      33.      17.
    .08     90.     60.     .08     34.5    23.19      42.
 35  22  36
      4   900.0   .0039    .050      33.      17.
    .08     90.     60.     .08     34.5    23.19      42.
 36  33  37
```

Table 34. (Cont'd)

```
  2
  1    1
      14
  810.0    811.0    812.0    813.0    814.0    815.0    816.0    817.0    818.0    819.0
  820.0    821.0    822.0    823.0
      14
    0.0     .022     .055     .105     .198     .285     .386     .514     .882    1.322
  1.837    2.479    3.214    3.788
  4
  3    810.0      4.0      3.0
  3    810.0      4.0      3.0
  3    810.0      4.0      3.0
  5    822.0    50.0
 37   22   38
       4     50.0    .0039     .050      33.      17.
     .08      90.      60.      .08     34.5    23.19      42.
 36   99
  1    3    6
 25   28   35
 20    3   12
 23    7   10
 33    6   10
 99   99
99999999
```

Table 35. Input Data for Example Run 2

```
     WILD CREEK - STORAGE DEVICE AT 25                                                         1.
36300. 5.0
 1  11   24
  38.3                                                          .78                .22
 2  11   20
   7.5                                                          .25                .75
 3  11   22
  25.0                                                          .88                .12
 5  11   29
   8.9                                                          .80                .20
 6  11   30
  17.0                                                          .80                .20
 7  11   28
  14.7                                                          .80                .20
 8  11   25
  12.2                                                          .85                .15
 9  11   28
  19.4                                                          .80                .20
11  11   33
  14.3                                                          .75                .25
13  11   18
  26.5                                                          .80                .20
14  11   18
  26.5                                                          .80                .20
15  11   35
  22.4                                                          .75                .25
17  11   35
   7.8                                                          .92                .08
18  22   19
     4    600.0     .0285     .050      30.     14.83
  .08       90.       60.      .08      72.       48.      36.
19  33   35
 2
 1   1
     8
 828.0    829.0     830.0     831.0    832.0    833.0    834.0    835.0
     8
  0.0      .074      .294      .413     .574     .792    1.004    1.240
 2
 1    828.0       3.0       3.0
 5    834.0     30.0
20   22   22
     4    900.0     .0611      .05     11.81     10.1
  .08      88.6     26.06      .08      143.     38.55    53.63
22   22   23
     4    700.0     .0143      .05     11.81     10.1
  .08      88.6     26.06      .08      143.     38.55    53.63
23   33   28
 2
 1   1
     8
 845.0    846.0     847.0     848.0    849.0    850.0    851.0    852.0
     8
  0.0      .009      .021      .037     .057     .083     .113     .186
 2
 3    845.0       2.0       3.0
 5    850.0     30.0
24   22   25
     4    700.0     .0807     .050      44.8     19.48
  .08       27.     18.25      .08       36.     24.08      48.
```

Table 35. (Cont'd)

```
25  33  27
 2                        1
 1
    13
 836.0   837.0   838.0   839.0   840.0   841.0   842.0   843.0   844.0   845.0
 846.0   847.0   848.0
    13
   0.0    .006    .028    .050    .077    .107    .141    .222    .342    .434
  .550    .650    .750
    13
   0.0     5.8    10.0    14.0    16.9    19.8    22.6    24.6   109.1   207.6
 223.6   239.6   345.5
27  22  28
     3   200.0   .0500    .025     4.5
28  22  32
     4  1200.0   .0167    .050     30.   14.83
   .08     90.     60.     .08     72.     48.     36.
29  22  30
     4   900.0   .0500     .05   11.81    10.1
   .08    88.6   26.06     .08    143.   38.55   53.63
30  33  31
 2
 1   1
    10
833.0   834.0   835.0   836.0   837.0   838.0   839.0   840.0   841.0   842.0
    10
   0.0    .014    .041    .083    .126    .179    .249    .331    .434    .551
 2
 1   833.0     3.0     3.0
 5   841.0    30.0
31  22  32
     4   300.0   .0500    .050     33.     17.
   .08     90.     60.     .08    34.5   23.19     42.
32  33  33
 2
 1   1
    11
 816.0   817.0   818.0   819.0   820.0   821.0   822.0   823.0   824.0   825.0
 826.0
    11
   0.0    .012    .028    .057    .092    .207    .321    .436    .551    .689
  .861
 2
 2   816.0     3.0     8.0
 5   825.0    20.0
33  22  35
     4   400.0   .0039    .050     33.     17.
   .08     90.     60.     .08    34.5   23.19     42.
35  22  36
     4   900.0   .0039    .050     33.     17.
   .08     90.     60.     .08    34.5   23.19     42.
36  33  37
 2
 1   1
    14
 810.0   811.0   812.0   813.0   814.0   815.0   816.0   817.0   818.0   819.0
 820.0   821.0   822.0   823.0
    14
   0.0    .022    .055    .105    .198    .285    .386    .514    .882   1.322
 1.837   2.479   3.214   3.788
 4
 3   810.0     4.0     3.0
 3   810.0     4.0     3.0
 3   810.0     4.0     3.0
 5   822.0    50.0
37  22  38
     4    50.0   .0039    .050     33.     17.
   .08     90.     60.     .08    34.5   23.19     42.
38  99
```

Table 35. (Cont'd)

16 17
18 19 20 22 23 24 25 27 28 29 30 31 32 33 35 36 37

18	3							
	0.0	1.0	2.0					
18	3							
	0.0	1.0	2.0					
19	5							
	0.0	32.8	56.7	79.2	184.0			
19	5							
	828.0	830.0	832.0	834.0	835.0			
20	3							
	0.0	1.0	2.0					
20	3							
	0.0	1.0	2.0					
22	3							
	0.0	1.0	2.0					
22	3							
	0.0	1.0	2.0					
23	8							
	0.0	13.4	26.7	32.8	43.1	49.9	153.0	335.9
23	8							
	845.0	846.0	847.0	848.0	849.0	850.0	851.0	852.0
24	3							
	0.0	1.0	2.0					
24	3							
	0.0	1.0	2.0					
25	7							
	0.0	10.0	16.9	22.6	24.6	109.1	223.6	345.5
25	7							
	836.0	838.0	840.0	842.0	843.0	844.0	846.0	848.0
27	3							
	0.0	1.0	2.0					
27	3							
	0.0	1.0	2.0					
28	8							
	0.0	25.0	50.0	100.0	200.0	300.0	400.0	500.0
28	8							
	0.0	0.9	1.3	1.9	2.8	3.5	4.0	4.3
29	3							
	0.0	1.0	2.0					
29	3							
	0.0	1.0	2.0					
30	4							
	0.0	49.1	79.2	200.3				
30	4							
	833.0	836.0	839.0	842.0				
31	3							
	0.0	1.0	2.0					
31	3							
	0.0	1.0	2.0					
32	6							
	0.0	111.2	192.6	268.9	325.0	444.0		
32	6							
	816.0	818.0	820.0	822.0	824.0	826.0		
33	3							
	0.0	1.0	2.0					
33	3							
	0.0	1.0	2.0					
35	8							
	0.0	25.0	50.0	100.0	200.0	400.0	600.0	800.0
35	8							
	0.0	3.4	4.5	5.6	6.5	7.4	7.9	8.4
36	7							
	0.0	113.5	226.9	277.9	365.9	423.3	479.5	
36	7							
	810.0	812.0	814.0	816.0	818.0	820.0	822.0	
37	3							
	0.0	1.0	2.0					
37	3							
	0.0	1.0	2.0					

Table 36. Input Data for Example Run 3

```
   WILD CREEK - STORAGE DEVICE AT 21 AND 25                                    1.
38300. 5.0
1  11   24
 38.3                                                   .78            .22
2  11   20
  7.5                                                   .25            .75
3  11   21
 25.0                                                   .88            .12
5  11   29
```

. . .

(Same as in Table 35)

. . .

```
   7.8                                                  .92            .08
18  22   19
     4    600.0    .0285     .050      30.     14.83
   .08      90.      60.      .08      72.      48.      36.
19  33   35
 2
 1   1
     8
 828.0    829.0    830.0    831.0    832.0    833.0    834.0    835.0
     8
   0.0     .074     .294     .413     .574     .792    1.004    1.240
 2
 1   828.0     3.0      3.0
 5   834.0    30.0
20  22   21
     4    900.0    .0611      .05    11.81     10.1
   .08     88.6    26.06      .08     143.    38.55    53.63
21  33   22
 2                          1
 1   1
     9
 853.0    854.0    855.0    856.0    857.0    858.0    859.0    860.0    861.0
     9
   0.0     .020     .080     .157     .257     .382     .530     .717     .932
 3
 1   853.0     1.5      1.5
 4   859.0    20.0
 4   860.0   250.0
22  22   23
     4    700.0    .0143      .05    11.81     10.1
   .08     88.6    26.06      .08     143.    38.55    53.63
23  33   28
 2
```

. . .

(Same as in Table 35)

. . .

Table 37. Input Data for Example Run 4

```
WILD CREEK - STORAGE DEVICE AT 25                                                                  0.
38300. 5.0
1  1 1  24
  38.3                                      0.18     3.0     0.5     1.5     .22
2  1 1  20
   7.5                                      0.18     3.0     0.5     1.5     .75
3  1 1  22
  25.0                                      0.18     3.0     0.5     1.5     .12
5  1 1  29
   8.9                                      0.18     3.0     0.5     1.5     .20
6  1 1  30
  17.0                                      0.18     3.0     0.5     1.5     .20
7  1 1  28
  14.7                                      0.18     3.0     0.5     1.5     .20
8  1 1  25
  12.2                                      0.18     3.0     0.5     1.5     .15
9  1 1  28
  19.4                                      0.18     3.0     0.5     1.5     .20
11 1 1  33
  14.3                                      0.18     3.0     0.5     1.5     .25
13 1 1  18
  26.5                                      0.18     3.0     0.5     1.5     .20
14 1 1  18
  26.5                                      0.18     3.0     0.5     1.5     .20
15 1 1  35
  22.4                                      0.18     3.0     0.5     1.5     .25
17 1 1  35
   7.8                                      0.18     3.0     0.5     1.5     .08
18  22  19
     4   600.0     .0285     .050      30.      14.83
   .08     90.       60.      .08      72.      48.      36.
19  33  35
2
1    1
     8
828.0    829.0    830.0    831.0    832.0    833.0    834.0    835.0
     8
  0.0     .074     .294     .413     .574     .792    1.004    1.240
2
1    828.0      3.0      3.0
5    834.0     30.0
20  22  22
     4   900.0     .0611     .05      11.81    10.1
   .08     88.6    26.06     .08      143.     38.55    53.63
22  22  23
     4   700.0     .0143     .05      11.81    10.1
   .08     88.6    26.06     .08      143.     38.55    53.63
23  33  28
2
1    1
     8
845.0    846.0    847.0    848.0    849.0    850.0    851.0    852.0
     8
  0.0     .009     .021     .037     .057     .083     .113     .186
2
3    845.0      2.0      3.0
5    850.0     30.0
24  22  25
     4   700.0     .0807     .050     44.8     19.48
   .08     27.     18.25     .08      36.      24.08    48.
25  33  27
2                            1
1
    13
836.0    837.0    838.0    839.0    840.0    841.0    842.0    843.0    844.0    845.0
846.0    847.0    848.0
    13
  0.0     .006     .028     .050     .077     .107     .141     .222     .342     .434
 .550     .650     .750
    13
  0.0      5.8     10.0     14.0     16.9     19.8     22.6     24.6    109.1    207.6
223.6    239.6    345.5
27  22  28
     3   200.0     .0500     .025     4.5
28  22  32
     4  1200.0     .0167     .050     30.      14.83
   .08     90.       60.      .08      72.      48.      36.
29  22  30
     4   900.0     .0500     .05      11.81    10.1
   .08     88.6    26.06     .08      143.     38.55    53.63
30  33  31
2
1    1
    10
```

Table 37. (Cont'd)

```
833.0    834.0    835.0    836.0    837.0    838.0    839.0    840.0    841.0    842.0
   10
  0.0     .014     .041     .083     .126     .179     .249     .331     .434     .551
2
1    833.0      3.0        3.0
5    841.0     30.0
31   22   32
      4   300.0     .0500     .050      33.      17.
  .08       90.       60.      .08    34.5    23.19      42.
32  33   33
2
1    1
    11
816.0    817.0    818.0    819.0    820.0    821.0    822.0    823.0    824.0    825.0
826.0
   11
  0.0     .012     .028     .057     .092     .207     .321     .436     .551     .689
 .861
2
2    816.0      3.0        8.0
5    825.0     20.0
33   22   35
      4   400.0     .0039     .050      33.      17.
  .08       90.       60.      .08    34.5    23.19      42.
35  22   36
      4   900.0     .0039     .050      33.      17.
  .08       90.       60.      .08    34.5    23.19      42.
36  33   37
2
1    1
    14
810.0    811.0    812.0    813.0    814.0    815.0    816.0    817.0    818.0    819.0
820.0    821.0    822.0    823.0
   14
  0.0     .022     .055     .105     .198     .285     .386     .514     .882    1.372
1.837    2.479    3.214    3.788
4
3    810.0      4.0        3.0
3    810.0      4.0        3.0
3    810.0      4.0        3.0
5    822.0     50.0
37   22   38
      4    50.0     .0039     .050      33.      17.
  .08       90.       60.      .08    34.5    23.19      42.
38  99
1    3    5    6
25   28   35
2    5
413874 5  1   28 71014  .00  .00  .00  .00  .06  .24  .48  .32  .21  .18  .36  .15
413874 5  1   28 71015  .09  .05  .03  .02  .01  .02  .01  .01  .01
99999999999999999999
  1    2    5
```

Table 38. First Page of Printout for Example Run 3

```
URBAN STORM RUNOFF MODEL FOR SMALL WATERSHEDS
---------------------------------------------

     RUN INFORMATION - WILD CREEK - STORAGE DEVICE AT 21 AND 25

                          FLOOD FREQUENCY SIMULATION USING RUNOFF FILE
THIS SIMULATION RUN HAS A TOTAL OF  38 SOURCE AREAS, PONDING AREAS, AND/OR CHANNEL SEGMENTS

COMPUTATIONS WILL BE MADE ON A 5.0 MINUTE TIME INCREMENT FOR A TOTAL OF 300.0 MINUTES

COEFFICIENT FOR STORAGE CONSTANT FOR LINEAR ROUTING FROM SOURCE AREAS IS  1.00

OUTPUT OPTIONS ARE

     16, PRINT STAGE-DISCHARGE TABLES
     17, LIST ALL PEAKS FROM RUNOFF FILE


AREA SEGMENT      1 DISCHARGES TO CHAN SEGMENT    24
AREA SEGMENT      2 DISCHARGES TO CHAN SEGMENT    20
AREA SEGMENT      3 DISCHARGES TO STOR SEGMENT    21
SKIP SEGMENT      4
AREA SEGMENT      5 DISCHARGES TO CHAN SEGMENT    29
AREA SEGMENT      6 DISCHARGES TO STOR SEGMENT    30
AREA SEGMENT      7 DISCHARGES TO CHAN SEGMENT    28
AREA SEGMENT      8 DISCHARGES TO STOR SEGMENT    25
AREA SEGMENT      9 DISCHARGES TO CHAN SEGMENT    28
SKIP SEGMENT     10
AREA SEGMENT     11 DISCHARGES TO CHAN SEGMENT    33
SKIP SEGMENT     12
AREA SEGMENT     13 DISCHARGES TO CHAN SEGMENT    18
AREA SEGMENT     14 DISCHARGES TO CHAN SEGMENT    18
AREA SEGMENT     15 DISCHARGES TO CHAN SEGMENT    35
SKIP SEGMENT     16
AREA SEGMENT     17 DISCHARGES TO CHAN SEGMENT    35
CHAN SEGMENT     18 DISCHARGES TO STOR SEGMENT    19 AND FLOWS WILL BE PRINTED
STOR SEGMENT     19 DISCHARGES TO CHAN SEGMENT    35 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     20 DISCHARGES TO STOR SEGMENT    21 AND FLOWS WILL BE PRINTED
STOR SEGMENT     21 DISCHARGES TO CHAN SEGMENT    22
CHAN SEGMENT     22 DISCHARGES TO STOR SEGMENT    23 AND FLOWS WILL BE PRINTED
STOR SEGMENT     23 DISCHARGES TO CHAN SEGMENT    28 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     24 DISCHARGES TO STOR SEGMENT    25 AND FLOWS WILL BE PRINTED
STOR SEGMENT     25 DISCHARGES TO CHAN SEGMENT    27 AND FLOWS WILL BE PRINTED
SKIP SEGMENT     26
CHAN SEGMENT     27 DISCHARGES TO CHAN SEGMENT    28 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     28 DISCHARGES TO STOR SEGMENT    32 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     29 DISCHARGES TO STOR SEGMENT    30 AND FLOWS WILL BE PRINTED
STOR SEGMENT     30 DISCHARGES TO CHAN SEGMENT    31 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     31 DISCHARGES TO STOR SEGMENT    32 AND FLOWS WILL BE PRINTED
STOR SEGMENT     32 DISCHARGES TO CHAN SEGMENT    33 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     33 DISCHARGES TO CHAN SEGMENT    35 AND FLOWS WILL BE PRINTED
SKIP SEGMENT     34
CHAN SEGMENT     35 DISCHARGES TO STOR SEGMENT    36 AND FLOWS WILL BE PRINTED
STOR SEGMENT     36 DISCHARGES TO CHAN SEGMENT    37 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     37 DISCHARGES TO QUIT SEGMENT    38 AND FLOWS WILL BE PRINTED
QUIT SEGMENT     38
SKIP SEGMENT     34
CHAN SEGMENT     35 DISCHARGES TO STOR SEGMENT    36 AND FLOWS WILL BE PRINTED
STOR SEGMENT     36 DISCHARGES TO CHAN SEGMENT    37 AND FLOWS WILL BE PRINTED
CHAN SEGMENT     37 DISCHARGES TO QUIT SEGMENT    38 AND FLOWS WILL BE PRINTED
QUIT SEGMENT     38
```

Table 39. Printout for Area and Channel Segments

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE: RAPID	MODERATE	SLOW	IMPERVIOUS	STORAGE CONSTANT	LENGTH-FT	SLOPE	ROUGHNESS
1	38.300	.000	.780	.000	.220	11.	.0000	.0000	.0000
2	7.500	.000	.250	.000	.750	3.	.0000	.0000	.0000
3	25.000	.000	.880	.000	.120	10.	.0000	.0000	.0000
5	8.900	.000	.800	.000	.200	5.	.0000	.0000	.0000
6	17.000	.000	.800	.000	.200	7.	.0000	.0000	.0000
7	14.700	.000	.800	.000	.200	7.	.0000	.0000	.0000
8	12.200	.000	.850	.000	.150	7.	.0000	.0000	.0000
9	19.400	.000	.800	.000	.200	8.	.0000	.0000	.0000
11	14.300	.000	.750	.000	.250	6.	.0000	.0000	.0000
13	26.500	.000	.800	.000	.200	9.	.0000	.0000	.0000
14	26.500	.000	.800	.000	.200	9.	.0000	.0000	.0000
15	22.400	.000	.750	.000	.250	8.	.0000	.0000	.0000
17	7.800	.000	.920	.000	.080	6.	.0000	.0000	.0000

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS	CHANNEL APARM	CHANNEL MPARM	FLOOD PLAIN APARM	FLOOD PLAIN MPARM	AVERAGE TRAVEL TIME(SEC)
18	4	600.00	.02850	.050	1.469	1.500	10.497	.922	74.5
20	4	900.00	.06110	.050	2.379	1.500	5.398	1.168	110.1
22	4	700.00	.01430	.050	1.151	1.500	2.611	1.168	177.0
24	4	700.00	.08070	.050	2.204	1.500	9.451	1.117	47.4
27	3	200.00	.05000	.025	91.622	1.000			
28	4	1200.00	.01670	.050	1.125	1.500	8.035	.922	194.8
29	4	900.00	.05000	.050	2.152	1.500	4.863	1.168	121.7
31	4	300.00	.05000	.050	1.805	1.500	10.831	.988	28.9
33	4	400.00	.00390	.050	.504	1.500	3.025	.988	138.1
35	4	900.00	.00390	.050	.504	1.500	3.025	.988	310.7
37	4	50.00	.00390	.050	.504	1.500	3.025	.988	17.3

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

Table 40. Printout for a Storage Segment

SPECIFICATIONS FOR STORAGE SEGMENT 36

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	810.000	.000	.000
.011	56.725	811.000	1.000	.022
.048	113.450	812.000	2.000	.055
.127	170.175	813.000	3.000	.105
.272	226.900	814.000	4.000	.198
.521	253.682	815.000	5.000	.285
.847	277.895	816.000	6.000	.386
1.302	322.673	817.000	7.000	.514
1.955	365.809	818.000	8.000	.882
3.090	394.806	819.000	9.000	1.322
4.625	423.338	820.000	10.000	1.837
6.807	451.526	821.000	11.000	2.479
9.615	479.463	822.000	12.000	3.214
13.183	668.022	823.000	13.000	3.788
16.751	856.581	.000	.000	.000
20.320	1045.140	.000	.000	.000
23.888	1233.699	.000	.000	.000
27.456	1422.258	.000	.000	.000
31.025	1610.817	.000	.000	.000
34.593	1799.376	.000	.000	.000

Table 41. Peak Flow Table Printout for Selected Storms

PEAK FLOWS FOR WILD CREEK - STORAGE DEVICE AT 25
STORM OF 3 12 20

ELEMENT NUMBER	ELEMENT TYPE	PEAK FLOW(CFS)
1	AREA	103.72
2	AREA	36.79
3	AREA	66.15
5	AREA	29.43
6	AREA	50.81
7	AREA	44.79
8	AREA	37.02
9	AREA	56.07
11	AREA	44.82
13	AREA	73.73
14	AREA	73.73
15	AREA	66.25
17	AREA	23.99
18	CHAN	137.51
19	STOR	66.49
20	CHAN	28.95
22	CHAN	81.01
23	STOR	84.26
24	CHAN	93.30
25	STOR	120.40
27	CHAN	123.38
28	CHAN	262.25
29	CHAN	27.10
30	STOR	62.70
31	CHAN	62.16
32	STOR	294.54
33	CHAN	313.75
35	CHAN	396.89
36	STOR	363.11
37	CHAN	362.35
38	OUTF	.00

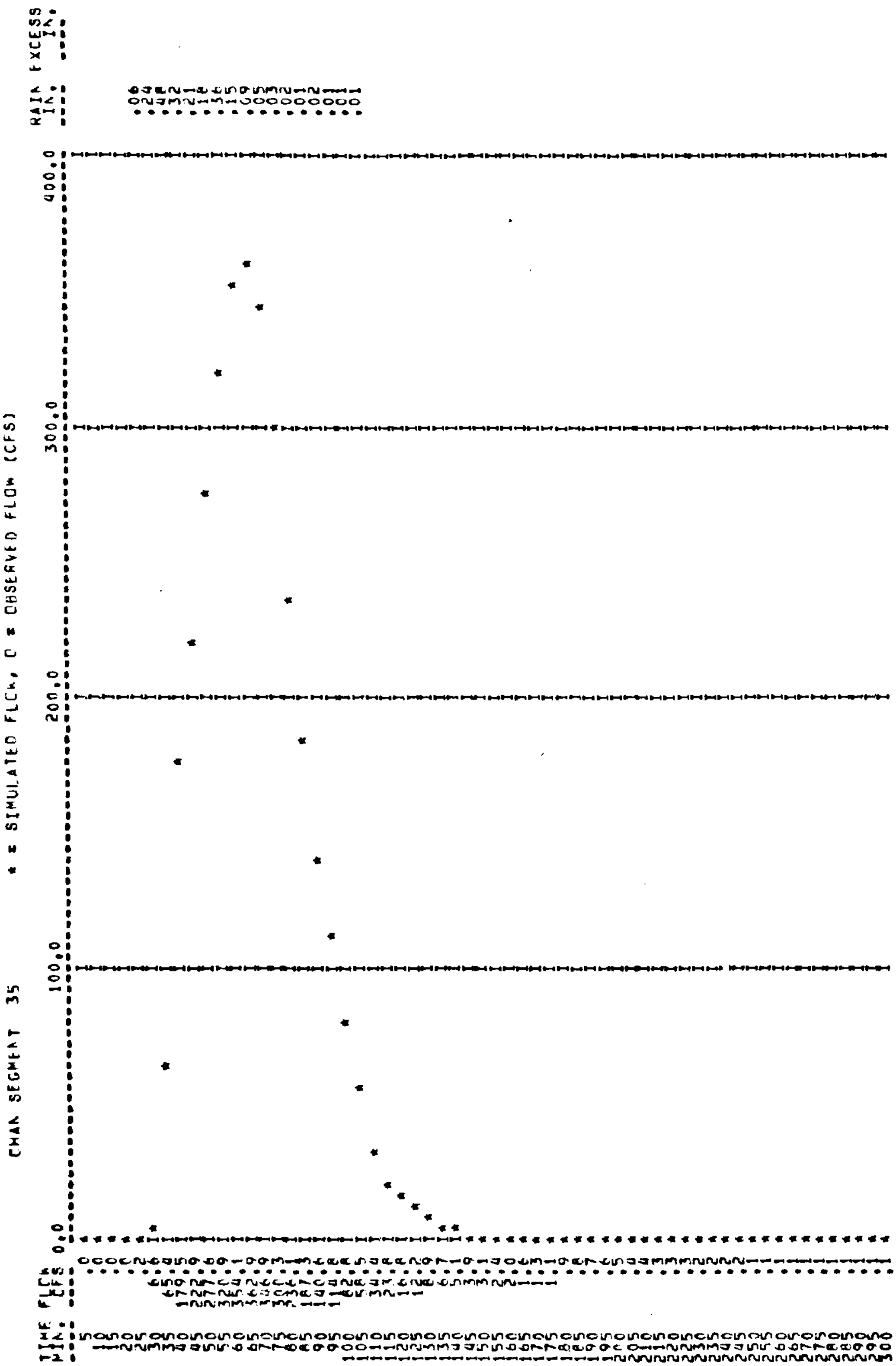


Figure 29. Computer Plotted Hydrograph for Wild Creek

Table 42. Printout of Potential Problem Error Messages

```
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 36 AND STORM(YEAR,MONTH,DAY) 1926  3 30 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 37 AND STORM(YEAR,MONTH,DAY) 1926  3 30 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT [illegible] AND STORM(YEAR,MONTH,DAY) [illegible]  [illegible] 28 [illegible]
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 31 AND STORM(YEAR,MONTH,DAY) 1929  2 28 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 22 AND STORM(YEAR,MONTH,DAY) 1936  9 29 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 23 AND STORM(YEAR,MONTH,DAY) 1936  9 29 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 36 AND STORM(YEAR,MONTH,DAY) 1936  9 29 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 37 AND STORM(YEAR,MONTH,DAY) [illegible]  9 [illegible] OCCURRED [illegible] STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 18 AND STORM(YEAR,MONTH,DAY) 1940  8 12 [illegible]
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 19 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 22 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 23 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 24 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 25 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 27 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 28 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 29 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 30 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
***** POTENTIAL PROBLEM *****
      MAXIMUM FLOW FROM SEGMENT 31 AND STORM(YEAR,MONTH,DAY) 1940  8 12 OCCURRED DURING LAST TIME STEP.
      A LARGER TIME STEP MAY BE NEEDED.
```

Table 43. Printout of Floods at Selected Segments for All Storms

SEQ=	23	DATE=	26	1	17	PEAK=	6.318
SEQ=	24	DATE=	26	1	17	PEAK=	7.269
SEQ=	25	DATE=	26	1	17	PEAK=	9.639
SEQ=	27	DATE=	26	1	17	PEAK=	9.633
SEQ=	28	DATE=	26	1	17	PEAK=	21.427
SEQ=	29	DATE=	26	1	17	PEAK=	1.738
SEQ=	30	DATE=	26	1	17	PEAK=	5.615
SEQ=	31	DATE=	26	1	17	PEAK=	5.281
SEQ=	32	DATE=	26	1	17	PEAK=	26.436
SEQ=	33	DATE=	26	1	17	PEAK=	28.685
SEQ=	35	DATE=	26	1	17	PEAK=	42.867
SEQ=	36	DATE=	26	1	17	PEAK=	42.827
SEQ=	37	DATE=	26	1	17	PEAK=	42.791
SEQ=	18	DATE=	26	2	18	PEAK=	17.916
SEQ=	19	DATE=	26	2	18	PEAK=	16.923
SEQ=	20	DATE=	26	2	18	PEAK=	4.024
SEQ=	22	DATE=	26	2	18	PEAK=	9.194
SEQ=	23	DATE=	26	2	18	PEAK=	9.215
SEQ=	24	DATE=	26	2	18	PEAK=	12.312
SEQ=	25	DATE=	26	2	18	PEAK=	13.838
SEQ=	27	DATE=	26	2	18	PEAK=	13.837
SEQ=	28	DATE=	26	2	18	PEAK=	30.325
SEQ=	29	DATE=	26	2	18	PEAK=	3.245
SEQ=	30	DATE=	26	2	18	PEAK=	10.063
SEQ=	31	DATE=	26	2	18	PEAK=	9.135
SEQ=	32	DATE=	26	2	18	PEAK=	37.657
SEQ=	33	DATE=	26	2	18	PEAK=	39.674
SEQ=	35	DATE=	26	2	18	PEAK=	57.734
SEQ=	36	DATE=	26	2	18	PEAK=	57.869
SEQ=	37	DATE=	26	2	18	PEAK=	57.468
SEQ=	18	DATE=	26	3	30	PEAK=	5.414
SEQ=	19	DATE=	26	3	30	PEAK=	5.322
SEQ=	20	DATE=	26	3	30	PEAK=	2.267
SEQ=	22	DATE=	26	3	30	PEAK=	4.057
SEQ=	23	DATE=	26	3	30	PEAK=	4.042
SEQ=	24	DATE=	26	3	30	PEAK=	4.097
SEQ=	25	DATE=	26	3	30	PEAK=	5.277
SEQ=	27	DATE=	26	3	30	PEAK=	5.275
SEQ=	28	DATE=	26	3	30	PEAK=	12.215
SEQ=	29	DATE=	26	3	30	PEAK=	.917
SEQ=	30	DATE=	26	3	30	PEAK=	2.869
SEQ=	31	DATE=	26	3	30	PEAK=	2.813
SEQ=	32	DATE=	26	3	30	PEAK=	15.027
SEQ=	33	DATE=	26	3	30	PEAK=	16.266
SEQ=	35	DATE=	26	3	30	PEAK=	23.139
SEQ=	36	DATE=	26	3	30	PEAK=	23.091
SEQ=	37	DATE=	26	3	30	PEAK=	22.983
SEQ=	18	DATE=	26	8	11	PEAK=	129.009
SEQ=	19	DATE=	26	8	11	PEAK=	72.328
SEQ=	20	DATE=	26	8	11	PEAK=	32.814
SEQ=	22	DATE=	26	8	11	PEAK=	64.656
SEQ=	23	DATE=	26	8	11	PEAK=	67.358
SEQ=	24	DATE=	26	8	11	PEAK=	90.930
SEQ=	25	DATE=	26	8	11	PEAK=	116.742
SEQ=	27	DATE=	26	8	11	PEAK=	116.658
SEQ=	28	DATE=	26	8	11	PEAK=	240.930
SEQ=	29	DATE=	26	8	11	PEAK=	22.901

Table 44. Printout of Stage-Discharge Tables

ELEVATION(STAGE) - DISCHARGE RELATION FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

SEGMENT NO.	18	Q(CFS) =	.000	1.000	2.000					
SEGMENT NO.	18	ELEV(FT)=	.000	1.000	2.000					
SEGMENT NO.	19	Q(CFS) =	.000	32.800	56.700	79.200	184.000			
SEGMENT NO.	19	ELEV(FT)=	828.000	830.000	832.000	834.000	835.000			
SEGMENT NO.	20	Q(CFS) =	.000	1.000	2.000					
SEGMENT NO.	20	ELEV(FT)=	.000	1.000	2.000					
SEGMENT NO.	22	Q(CFS) =	.000	1.000	2.000					
SEGMENT NO.	22	ELEV(FT)=	.000	1.000	2.000					
SEGMENT NO.	23	Q(CFS) =	.000	13.400	26.700	32.800	43.100	49.900	153.000	335.900
SEGMENT NO.	23	ELEV(FT)=	845.000	846.000	847.000	848.000	849.000	850.000	851.000	852.000
SEGMENT NO.	24	Q(CFS) =	.000	1.000	2.000					
SEGMENT NO.	24	ELEV(FT)=	.000	1.000	2.000					
SEGMENT NO.	25	Q(CFS) =	.000	10.000	16.900	22.600	24.600	109.100	223.600	
SEGMENT NO.	25	ELEV(FT)=	836.000	838.000	840.000	842.000	843.000	844.000	846.000	
SEGMENT NO.	27	Q(CFS) =	.000	1.000	2.000					
SEGMENT NO.	27	ELEV(FT)=	.000	1.000	2.000					
SEGMENT NO.	28	Q(CFS) =	.000	25.000	50.000	100.000	200.000	300.000	400.000	500.000
SEGMENT NO.	28	ELEV(FT)=	.000	.900	1.300	1.900	2.800	3.500	4.000	4.300
SEGMENT NO.	29	Q(CFS) =	.000	1.000	2.000					
SEGMENT NO.	29	ELEV(FT)=	.000	1.000	2.000					

Table 45. Printout Flood Frequency Table for Segment 32

ANNUAL PEAK FLOWS FOR SEGMENT NO. 32 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	34.4	1918	APR.	7
1919	108.6	1919	MAY	6
1920	270.6	1920	MAR.	12
1921	282.9	1921	AUG.	24
1922	111.0	1922	MAR.	10
1923	76.8	1923	MAR.	12
1924	87.6	1924	MAY	27
1925	73.6	1924	DEC.	8
1926	280.2	1926	AUG.	11
1927	76.8	1927	FEB.	23
1928	309.9	1928	JULY	10
1929	119.8	1929	MAR.	14
1930	51.4	1930	JAN.	28
1931	54.8	1931	JULY	28
1932	131.9	1932	JUNE	18
1933	162.3	1933	JUNE	10
1934	119.1	1934	JULY	19
1935	36.8	1935	MAR.	5
1936	125.7	1936	FEB.	3
1937	159.6	1937	JUNE	17
1938	239.4	1938	JUNE	25
1939	109.1	1939	JUNE	22
1940	220.1	1940	SEPT	10
1941	111.5	1941	AUG.	13
1942	145.7	1942	MAR.	20

MEAN = 140.0 STANDARD DEVIATION = 81.6

FLOOD FREQUENCY FOR FLOWPOINT 32 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

RETURN PERIOD	PROBABILITY *	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	126.6	818.4
5-YEAR	20.0	198.7	820.2
10-YEAR	10.0	246.5	821.5
25-YEAR	4.0	306.8	823.4
50-YEAR	2.0	351.6	824.5
100-YEAR	1.0	396.1	825.3
200-YEAR	0.5	440.3	825.9

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABL

Table 46. Printout Flood Frequency Table for Segment 35

ANNUAL PEAK FLOWS FOR SEGMENT NO. 35 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	54.5	1918	APR.	7
1919	168.3	1919	MAY	6
1920	381.2	1920	AUG.	15
1921	401.2	1921	AUG.	24
1922	175.8	1922	MAR.	10
1923	122.6	1923	MAR.	12
1924	138.1	1924	MAY	27
1925	118.5	1924	DEC.	8
1926	414.1	1926	AUG.	11
1927	124.0	1927	FEB.	23
1928	462.1	1928	JULY	10
1929	191.8	1929	MAR.	14
1930	80.5	1930	JAN.	28
1931	83.8	1931	JULY	28
1932	207.2	1932	JUNE	18
1933	225.6	1933	JUNE	10
1934	179.1	1934	JULY	19
1935	58.6	1935	MAR.	5
1936	191.9	1936	FEB.	3
1937	221.3	1937	JUNE	17
1938	329.9	1938	JUNE	25
1939	174.2	1939	JUNE	22
1940	309.2	1940	SEPT	10
1941	174.0	1941	AUG.	13
1942	226.1	1942	MAR.	20

MEAN = 208.5 STANDARD DEVIATION = 113.9

FLOOD FREQUENCY FOR FLOWPOINT 35 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	189.8	6.4
5-YEAR	20.0	290.5	6.9
10-YEAR	10.0	357.1	7.2
25-YEAR	4.0	441.3	7.5
50-YEAR	2.0	503.8	7.7
100-YEAR	1.0	565.8	7.8
200-YEAR	0.5	627.6	8.0

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE

segments 32 and 35 showing the water surface elevations in feet above mean sea level and above the channel bottom, respectively. Flood frequency tables are created for each segment listed on the option card for output and for RUNOPT equal to 1.

SECTION 6

Recommendations for Futher Development

UROS4 was developed for the specific purpose of providing DeKalb County a tool to study drainage problems and the hydrologic effects of land use and channel changes and detention storage. The model was field tested on several watersheds in order to define their specific problems and examine the hydrologic effects of potential solutions. These applications also provided the opportunity to check the adequacy of the model in a variety of situations and to provide future users with illustration of such applications. As a result, to improve the model by programming additional capabilities became apparent.

One of these would deal with situations in which flow divides from one path into two separate paths. This quite often happens at street inlets where one portion of the flow enters and the other portion by-passes the inlet. Usually this situation has little significance; but in some cases the dynamic change in the drainage boundary may add substantially to runoff in a small stream. For example, when the flow rate is low, all the water drains to the inlet and thence to watershed A; but as the flow rate gets higher, a greater portion of the water by-passes the inlet to watershed B. Another field situation the model cannot handle relates to a series of reservoirs. In one case two reservoirs were in series. Outlets from the upstream reservoir directed water to either the downstream reservoir through a drop inlet or to a channel by-passing the downstream reservoir through a broad-crested weir. Separation of flow also occurs in combined sewers. Design of floodways to divert excess flood water would also benefit from a separation procedure in UROS4.

Another part of the model that could be expanded is the routing of rainfall excess from the area segments. The current linear storage model, which limits the size of subareas to about 1.5 square miles, was used because data was not sufficient for a more flexible model. In a year or two, the data from the six DeKalb County stream gages and the recently installed gage network of the U.S. Geological Survey can be used to expand and improve this portion of the model. A model balancing cost with accuracy would involve the routing of a time-area diagram. This would allow subareas to exceed the 1.5 square mile limitation.

A third area for expansion of program capability is the creation, retrieval and updating of channel and land-use information of the watersheds. The current scheme involved coding, keypunching, and filing computer data cards. After some experience has been gained with UROS4, a more efficient system could be developed for better utilization of available computer hardware and software. Such a system would be quite beneficial on a regional scale.

Several other developments could be made to expand UROS4 capabilities. Additional error messages and program checks could always be added. Forty error messages are already programmed in UROS4. A printer plot of the flood frequency curve and data would also be very helpful. Though the programming might be extensive, a printout of a schematic diagram showing each segment and the connections would be very helpful to a user. Some additional programming could be added to help determine stage-discharge tables from channel cross-section data and storage-discharge tables for additional types of outlet works.

Runoff File. The runoff file was developed for the soils and sizes of drainage basins found in DeKalb County. For applications at other locations, even others within the Atlanta region, both additional types of soils are found and larger watersheds exist. In either case, the applicability of the runoff files must be evaluated. This is particularly true of the larger drainage basins for which the assumption of spacial uniformity of the rainfall is no longer valid. In such cases, a second or third series of runoff files or adjustment factors for the existing files are needed to represent the spacial variation of precipitation.

Experiences gained from application of UROS4 and the runoff files showed that several storms on the runoff file could never be critical for the given water year. With well-designed criteria, such storms could be removed from the runoff file with no loss in accuracy or capability, but with a reduction in computer time in the order of 20 to 50 percent.

Better definition of probability could be achieved with a longer runoff file to include more combinations of critical periods. However, from the analysis described in Section 2, such a file must exceed 50 years before a noticeable improvement could be made. This would more than double computer run time. On the other hand, if features are added for the effects of aerial variation of rainfall, then the expansion of the runoff file to more than 25 years might be needed for a good definition of probability.

A further area of study would be an examination of the 72-year rainfall record for additional storms. Though additional storms are not needed in years with major events, they would improve the definition of probability of the small events. For most of the 72-year record, the critical storms were selected from values of daily rainfall. The selection of critical periods based on hourly records is now more feasible than when the U.S. Geological Survey originally developed the 72-year, 5-minute, major storm rainfall data.

One further programming effort could size detention storage structures within UROS4 to meet specific design criteria. If the peak rates of flow after land development is not to exceed the peaks before development the needed storage volume and sizes of outlet structures could be calculated. By changing pipe sizes or slopes or channel designs a smaller storage might also meet the criteria. With such simulations, a least cost drainage system could be designed.

SECTION 7

UROS4 Fortran Program

Introduction. This section lists the Fortran IV computer code for UROS4. It has a fairly long main program with two subroutines called STODIS and FREQ. STODIS calculates the storage volume and discharge arrays needed for storage routing. FREQ picks the largest peak flow in each water year, makes the frequency analysis for flood flows, and estimates water surface elevations. A general flowchart of the main program is found on Figure 30.

Names of variables for equations in the text of the report is, in most cases, the same as the Fortran variables in UROS4. Table 47 lists these variables. Table 48 lists some additional variables found in UROS4 but not included in the text of the report.

Forty different error messages can be printed by UROS4. Each message is numbered for easy reference. In most cases the message is self-explanatory. For those that are not, error message descriptions are included in this section.

Figure 30. Flowchart of UROS4

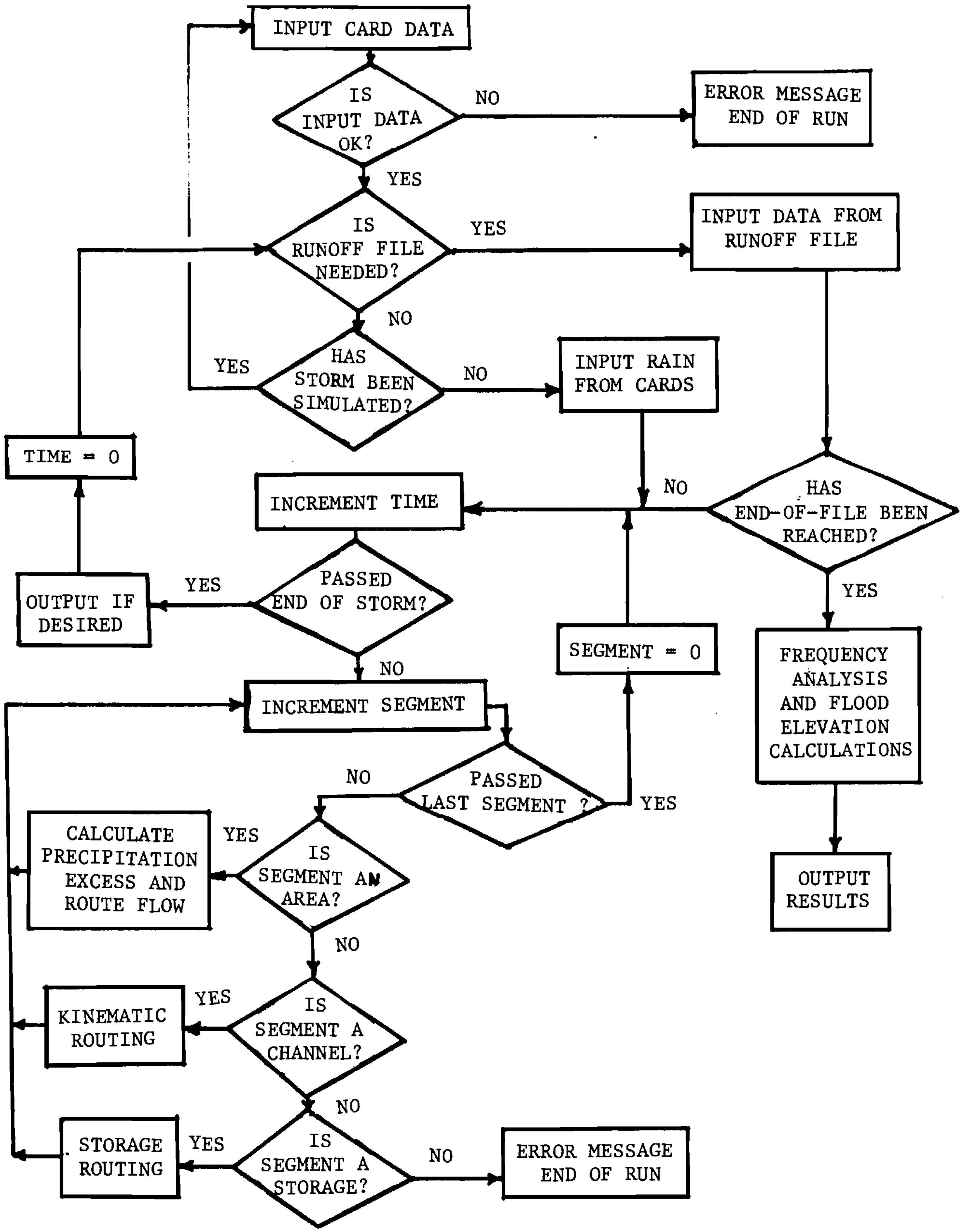


Table 47.

Notation Used in Text of Report

A	Fraction impervious area in watershed model
A	Drainage area in square miles
A_i	Water surface area of detention storage
A	Cross-sectional area of flow in square feet
AC	Cross-sectional area of the channel
ACX	Cross-sectional area projected above the channel
AK	Loss rate coefficient at beginning of time interval
ALOSS	Loss rate for particular time interval (inches per hour)
ALT	Cross-sectional area of the left flood plain
ART	Cross-sectional area of the right flood plain
a	A parameter of channel slope, roughness, and size
APARM	Same as "a"
CUML	Accumulated loss (inches) up to current time interval
C	Loss coefficient for outlet works
C_0	Muskingum routing parameter
C_1	Muskingum routing parameter
C_2	Muskingum routing parameter
CB	Infiltration parameter in watershed model
CC	Interflow volume parameter in watershed model
D	Length of channel in miles
D	Detention storage on the overland flow surface in inches
D_e	Detention storage at equilibrium in average depth in inches over the overland flow surface
D_h	Horizontal dimension of conduit
DLTK	Incremental increase in loss rate coefficient during the time interval

Table 47. (Cont'd)

DLTKR	Additional rain loss at the beginning of the storm
D_V	Vertical dimension of conduit
E_i	Water surface elevation of detention storage
EPXM	Interception storage parameter in watershed model
ERAIN	Exponent of precipitation for the loss rate function
FNC	Manning's n of the channel
FNL	Manning's n of the left flood plain
FNR	Manning's n of the right flood plain
H	Water surface elevation above invert of outlet structure
I	Inflow to a storage site in cfs
$\bar{I}$	Average rate of precipitation excess
$\dot{\bar{I}}$	Average infiltration during Δt in inches
i	Intensity of rainfall in inches per hour
IRC	Interflow drainage parameter in watershed model
K	Storage constant for a linear reservoir model
K3	Evaporation parameter in watershed model
KK24	Baseflow drainage parameter in watershed model
K	Multiplier of standard deviation for probability equation
L	Length of overland flow surface in feet
L	Length of a channel segment
LZSN	Nominal lower zone storage capacity in watershed model
m	A parameter of channel slope, roughness, and size
MPARM	Same as "m"
N	Number of days from peak discharge to end of direct runoff
n	Manning's n
n	Number of years of the annual series of peak flows

Table 47. (Cont'd)

p	Rainfall intensity (inches per hour) during the time interval
p	Probability in any one year that a given flood will be exceeded
p	Wetted perimeter in feet
$\bar{p}$	Average rainfall less interception during Δt in inches
P_E	Storm precipitation excess in inches
PK	Coefficient in equation for calculating storage constant
Q	Flow rate in cubic feet per second
Q	Outflow from a channel reach in cfs
$\bar{Q}$	Average outflow during Δt in inches
QRSCN	Streamflow at the time after a storm when only base flow remains expressed as a percentage of the flood peak
q	Rate of lateral inflow to a channel in cfs
R	Basin storage constant used in routing through the linear reservoir in HEC-1
R	Hydraulic radius in feet
RTIOL	Ratio of rain loss coefficient to that corresponding to 10 inches more of accumulated loss
RTIOR	Ratio of flow at a point during the recession to that ten time periods later
S	Reservoir storage in cubic feet
S	Slope of overland flow surface
S	Slope of a channel segment
SLOPE	Same as "S"
STRKR	Value of the loss coefficient at the beginning of the storm
SRC	Overland flow drainage parameter in watershed model

Table 47. (Cont'd)

STRTQ	Streamflow at time storm begins
T_R	Duration of storm precipitation in hours
t	A point in time
Δt	Time increment
U	Fraction of watershed in impervious area
UZSN	Nominal upper zone storage capacity in watershed model
V_i	Volume in cubic feet or acre-feet of a storage at a given elevation
V	Stream flow velocity in feet per second
W	Width at base of broad-crested weir
WPC	Wetted perimeter of the channel
WPL	Wetted perimeter of the left flood plain
WPR	Wetted perimeter of the right flood plain
x	Distance measure along a channel in the downstream direction in feet
Δx	Length of a channel segment
X_i	Peak flow in annual flood series
$\bar{X}$	Mean of the annual flood series
θ	Angle from vertical in degrees of sides of a broadcrested weir
σ_x	Standard deviation of the annual flood series
σ_x^2	Variance of the annual flood series

Table 48.

Additional Variables Not Included in Previous Sections

Variable	Type	Definition
AA	R	Peak storage (cu. ft.)
ADD	R	Flow to be spread over an area segment
B	R	Peak storage (acre-feet)
C	R	Peak storage (inches)
CFSIN	R	Conversion factor (cfs to inches)
CUMA	R	Drainage area at flowpoint
CUML	R	Accumulative loss (inches)
C1	R	Coefficient for Muskingum routing
C2	R	Coefficient for Muskingum routing
C3	R	Coefficient for Muskingum routing
DE	R	Detention storage at equilibrium
DLTK	R	Incremental increase in loss rate coefficient
DNEW	R	Surface detention storage at end of time step
DOLD	R	Surface detention storage at beginning of time step
DT	R	Time step
DTEN	R	Increment on streamflow axis
EXC	R	Rainfall excess
EXCESS	R	Rainfall excess
IAN	I	Array of area segment numbers
IBLK	I	Plotting symbol
IBUF	I	Array required for Calcomp plot
ICHAR	I	Array of symbols for plotting
ICI	I	Plotting symbol
ICO	I	Plotting symbol

Variable	Type	Definition
ICX	I	Plotting symbol
IFPT	I	Index array for flow segments to be plotted
IO	I	Option indicator
IPARM	R	Reciprocal of MPARM
IT	I	Total number of time steps
ITYPE	I	Element type (O>AREA, O=CHAN, O<STOR)
IYAREA	I	Indicator for existance of area segments
IYCHAN	I	Indicator for existance of channel segments
JT	I	Time on plotting axis
LOSS	R	Rainfall minus excess
MAXFLO	R	Peak flow
MAXSTR	R	Maximum reservoir storage
NG	I	Array of numbers of channel segments for comparison
NOP	I	Number trial and error iterations before printout
NSC	I	Array of numbers of channel segments for comparison
NST	I	Array of storage segment numbers
NSTOR	I	Index array for storage segment numbers
ODEC	R	Calculated constant for overland flow
OF	R	Overland flow
OFTMI	R	Overland flow at previous time step
OUT	R	Flow at downstream end of channel reach
PC	R	Time step in hours
PCOR	R	Rainfall adjustment factor
PCT	R	Excess as a percent of rainfall
PREC	R	Rainfall
Q	R	Lateral inflow to channel reach

Variable	Type	Definition
QT	R	Interpolated values of observed stream flow
RAIN	R	Rainfall plus cascaded flow
RC	R	Runoff as a percent of rainfall
SNEW	R	New approximation to reservoir storage
SOLD	R	Previous approximation to reservoir storage
STR	R	Final approximation to reservoir storage
SUMP	R	Total storm precipitation
SUMQ	R	Total storm runoff
VOL1	R	Total storm runoff (inches)
VOL2	R	Total storm runoff (acre-feet)
XDLTKR	R	DLTKR value for all pervious area segments
XM	R	Size of axis for streamflow
XSTRKR	R	SL value for all pervious area segments
X1	R	Tic mark on streamflow axis
X2	R	Tic mark on streamflow axis
X3	R	Tic mark on streamflow axis

ERROR MESSAGES

1) I CANNOT ACCEPT DATA WITH___ AS A TIME UNIT.
TIME UNITS MUST BE 1 (SEC) OR 60 (MIN) OR 3600 (HRS).

Self explanatory. Card 2 in error.

2) I CANNOT ACCEPT DATA WITH___ AS AN AREA UNIT.
AREA UNITS MUST BE 1 (ACRES) OR 640 (SQ. MILES).

Self explanatory. Card 2 in error.

3) A TOTAL TIME OF____ WITH AN INCREMENT OF___ GIVES___ STEPS WHICH EXCEEDS THE 120 LIMIT.

REDUCE TOTALT OR INCREASE DELT.

Self explanatory. Card 2 in error.

4) NUMBER OF SEGMENTS SPECIFIED (__) EXCEEDS LIMIT OF 100.

Self-explanatory. Card 3 in error.

5) SEGMENT NUMBER___ IS GREATER THAN MAXIMUM ALLOWED WHICH IS 100.

Self-explanatory. Card 3 in error.

6) SEGMENT___ DESCRIPTION CARD IS OUT OF ORDER

Discriptor cards for each segments must be sequenced in numerical order. Program has encountered a card specifying a segment number smaller than the previous one. Card 3 in error.

7) ___ SEGMENT CANNOT BE LIKE___ WHICH IS LARGER.

An option not otherwise discussed in this report allows the user to specify just the number of a previous segment if the current segment is identical. The option applies only for subarea segments and channel segments. In this case a subsequent instead of a previous segment was used which is not allowed. Also segment types must match. Card 3 in error.

8) __ SEGMENT TYPE___ CANNOT BE LIKE ANOTHER

See number 7 above. Card 3 in error.

9) __ELEMENT NUMBER EXCEEDS MAXIMUM SPECIFIED___

Self-explanatory. Card 3 in error.

10) SORRY BUT AREA SEGMENT__IS TOO SMALL FOR TIME STEP OF __
STORAGE CONSTANT EQUALS__YOU WILL NEED TO REDUCE TIME STEP OR GROUP AREA SEGMENTS.

The value of K must exceed one-half the time step since

$$K = \frac{1.0(\text{area})}{640}^{0.5} \qquad (1 + U)^{-1.68}$$

either area must increase or impervious area decrease. A table in Section 2 gives the minimum areas for different time steps and impervious fractions. Card 4 in error.

11) SUM OF FRACTION OF SOIL TYPES (___) DOES NOT EQUAL 1.0 FOR SEGMENT___

With runoff file, all land surface must be counted so the fractions sum to 1.0. Card 5 in error.

12) __ IS NOT RECOGNIZED AS A KIND OF CHANNEL REACH.

Specification for KIND not equal to one of the five options which are 0,1,2,3,and 4. Card 6 in error.

13) PARAMETERS FOR ELEMENT__ IMPROPER

The ten values listed are AREA, LENGTH, SLOPE, FF, STRKR, RTIOL, ERAIN, APARM, MPARM and U. One or more of the above is unreasonable; LENGTH less than 1.0 or greater then 9000, SLOPE greater than 0.9, FF greater 0.5, APARM less than 0.00001, MPARM less than 0.1, channel width for rectangular channel less than 0.9 or greater than 900.0, pipe diameter less then 0.04 or greater than 50.0, width on triangular channel greater than 900.0 or less than 0.08. Card 6 is in error.

14) __ IS NOT ACCEPTABLE FOR TU ACCEPTABLE VALUES - 1(SEC), 60(MIN), 3600 (HOUR)

Time unit on storage description not one of the three available options. Card 8 is in error.

15) *** ERROR-VALUES OF INDEX VARIABLES KE AND KD___ FOR STORAGE SEGMENT GREATER THAN 1.

CARDS POSSIBLY OUT OF ORDER.

Self-explanatory. Card 9 in error.

16) __ IS TOO MANY ELEVATIONS FOR STORAGE SEGMENT, MAX = 21

Self-explanatory. Card 10 is in error.

17) __ IS TOO MANY VOLUMNS FOR STORAGE SEGMENT MAX. = 20

Self-explanatory. Card 11 is in error

18) NO. AREAS =__ NO. ELEV =__ MUST BE EQUAL AND LESS THAN 21

Self-explanatory. Card 12 is in error.

19) ___IS TOO MANY DISCHARGE VALUES FOR STORAGE SEGMENT, MAX. = 20

Self-explanatory. Card 13 is in error.

20) ******************
WARNING MESSAGE   ARE YOU SURE YOU WANT ___
OUTLET STRUCTURES ON ONE OF THE STORAGE SEGMENTS.
****

Self-explanatory. Card 14 is in error.

21) JT FOR STORAGE SEGMENT CANNOT EQUAL ___ .

Options for JT are 1,2,3,4,5,or 6 only. Card 15 is in error.

22) **
WARNING MESSAGE
NO DISCHARGE FROM ONE OF THE OUTLET WORKS
OUTLET TYPE ____
ELEVATION OF BOTTOM OF STORAGE_______
ELEVATION AT BOTTOM OF THIS OUTLET STRUCTURE
HIGHEST ELEVATION REACHED_______
****

More elevations need to be put on elevation card 10 to exceed invert on one of the outlet works. Note: when extra elevations given, an equal number of extra areas or volumes must also be given.

23) I DO NOT KNOW WHAT TO DO WITH ___ AS A SEGMENT TYPE.

TYPE not one of the available options which are 11,22,33,55, and 99. Card 3 in error.

24) SEGMENT ___ DISCHARGES TO SEGMENT ___ WHICH IS A NONO

Segment numbers cannot exceed 100. Card 3 in error.

25) SEGMENT ___ DISCHARGES TO DUMMY SEGMENT _ WHICH IS MISTOOK

A real segment is discharging water to a dummy segment where it would get lost. Card 3 in error.

26) ___ SEGMENT ___ HAS NOTHING FLOWING INTO IT.
PLEASE CORRECT BY ELIMINATING THE SEGMENT OR PUTTING SOMETHING INTO IT.

Self-explanatory. Card 3 in error.

27) OUTPUT SPECIFICATIONS IMPROPER
OPT = ____ ___ ___ ___
OUPT = ____ ___ ___ ___

A value for a program a value for a program option exceeds 20 or a value for an output option exceeds 100.

28) YOU ARE GIVING ME MORE PRECIPITATION CARDS THAN I CAN HOLD. 120 VALUES IS THE LIMIT.

Self-explanatory. Card 21 or 22 in error.

29) I CAN ONLY ACCEPT 5 LOCATIONS FOR MEASURED STREAMFLOW, YOU ARE GIVING ME _______

Self-explanatory. Card 23 is in error.

30) **
SORRY, BIG TROUBLE
TIME STEP ON TAPE_____
TIME STEP SPECIFIED FOR THIS RUN_______
THEY MUST BE THE SAME
****

Change to make them the same. It could be that the wrong runoff files were specified on the system control cards or if files on tape, the wrong tape mounted. Card 2 or system control card is in error.

31) SOIL NUMBER (1,2,3, or 4) FROM RUNOFF FILE IMPROPER
VALUE READ WAS______

Try again. If error persists then a problem reading data from runoff file exists. Get assistance from computer programmer.

32) YOU HAVE SPECIFIED SOIL___FOR ONE OF THE AREAS BUT HAVE NOT GIVEN THE PROGRAM THE RUNOFF FILE FOR THAT SOIL.

Either correct the error on card 5 if there is one or add system control card to mount the needed runoff file.

33) IN TWENTY TRIALS COULD NOT GET SOLUTION FOR ELEMENT__ AT TIME___

If program won't finish then one of the subareas probably has some unreasonable descriptor values.

34) STORAGE ON SEGMENT__ HAS EXCEEDED THE EXTRAPOLATED MAXIMUM.

This is a warning message. When it occurs, the inflow is set equal to the outflow and no additional storage effects are computed. One should add more elevations and areas or volumes to cards 10 and 11 or 12.

35) ***** POTENTIAL PROBLEM *****
MAXIMUM FLOW FROM SEGMENT__ AND STORM (YEAR, MONTH, DAY) 19____ __ __
OCCURRED DURING LAST TIME STEP. A LARGER TIME STEP MAY BE NEEDED.

This is a warning message only. The output of all peaks should be checked to see that this storm is not likely to be the largest for the water year. Usually it is not. If it is, a longer time step must be used or up to 120 time steps should be given on card 2 if 120 is not already being used.

36) *** POTENTIAL PROBLEM - NUMBER OF STAGE VALUES (__) AND DISCHARGE VALUES (__)
ARE NOT THE SAME FOR SEGMENT NUMBER ____

The values for K on cards 30 and 31 must be the same.

37) *** POTENTIAL PROBLEM - PROGRAM WANTS RATING TABLE FOR SEGMENT ___
AND DATA CARD SAYS _____

A rating table (cards 30 and 31 must be given for each segment listed for output options on card 17. The rating table cards must also be sequenced in numerical order.

38) ERROR REACHED END OF FILE ___ ___

Problems have been encountered with the temporary file. Consult computer programmer. First number given is the segment and the second number given is the year problem encountered.

39) YEAR OR SEGMENT NO. FROM FILE IMPROPER ___ ___

Same comments as 38 above apply.

40) PRECIP INCREMENT OF___MINUTES NOT COMPATIBLE WITH SIMULATION INCREMENT OF___ MINUTES.

Change value on card 2 or card 20 to make time increments compatible. The value on card 20 must divide evenly by the value on card 2.

```
C   URBAN WATERSHED FLOOD HYDROGRAPH MODEL (URPS4)
C              PROGRAMMED BY ALAN M. LUMB , ALLEN E. JOHNSON,
C                     L.D. JAMES AND J.L. KITTLE,JR.
C
      REAL  MAXFLO,AREA(100),LENGTH(100),SLOPE(100),FF(100),CUML(100),
     1      IN,F(15,20),STOR(15,20),SUMQ(100),QT(120),MAXSTR(100),
     2      APARM(200),MPARM(200),APRA(12),A(100),QDEC(100),QFCT(100),
C
     3      CFSIN(100),STR(15,120),OLDOUT(25)
     4      ,OLDIN(25),SL(100),RTL(100),ER(100),DNEW(100),DOLD(100)
     5      ,EXC(50,120),Q(100,2),SK(100),SX(100),C1(50),C2(50),C3(50)
     7      ,FD(25),DLTKR(100),LOSS,RAIN(120),INC,ADD(100)
     8     ,U(100),CHANA(100)
      REAL SOIL1(100),SOIL2(100),SOIL3(100),SOIL4(100)
      EQUIVALENCE (SOIL1,RTL),(SOIL2,ER),(SOIL3,DLTKR),(SOIL4,U)
      INTEGER T,NSTOR(100),OPT(20),OUPT(100),ICHAR(101),V,
     1    NE(10), IFPT(100),TUNITS,INTO(100),KIND(100)
     2      ,TU,VU,AU,NS(100),NST(25),N1(25),N2(25)
      INTEGER YR,DAY,HR,TI,TY,TYPE,ISEQ(9),ISOIL(4),N10(100)
      COMMON USR(99),SF(5,120),PREC(120),IN(65,120),OUT(65,120),FACT,
     1        CUMA(100),MAXFLO(100),INFO(20),TRUF(1120),LDEV,TIMX,ISPEC,
     2    IT,NUM,TYPE(100),TFACT,ITYPE(100),TAN(100),QF(50,120)
     3      ,PC,NG(100),NSC(100),IO(20)
      COMMON /STOD/ VU,IERROR,SLLEV(25),ELEV(25),SAREA(25),VOL(25),DIS(2
     15)
      DATA ICI/'I'/
      DATA ICO/'O'/
      DATA ICX/'*'/
      DATA IBLK/' '/
      DATA FD/0.8,0.7,0.6,0.55,0.5,0.5,0.5,0.5,0.5,0.5,0.5,0.5,0.5,
     1      0.5,0.5,0.5,0.5,0.5,0.5,0.5,0.5,0.5,0.5,0.5,0.5/
      DATA ISEQ/'AREA','CHAN','STOR','    ','SKIP','DUMB',
     1      '    ','    ','QUIT'/
  100 FORMAT(I6,18A4,F2.0)
  101 FORMAT(20I4)
  102 FORMAT(I4,4X,I4,2I4)
  103 FORMAT(10F8.0)
  104 FORMAT(I4,2F4.0,2I4,4F8.0,I4)
  105 FORMAT(I6,5I2,F4.1,12F5.2)
  106 FORMAT(20X,12F5.1)
  107 FORMAT(I8,12F6.0)
  108 FORMAT(I8,9F8.0)
  109 FORMAT(2I4,2F8.0,I4)
  110 FORMAT(F4.0,I4,F4.0,I4,F4.0,I4,F4.0)
  111 FORMAT()
  112 FORMAT(13A6)
  113 FORMAT(A6,I2,I2,2X,4I2,12F5.2)
  114 FORMAT(2I4,F4.0,17I4)
  115 FORMAT(I4,F4.0,I4)
C
C   INPUT GENERAL INFORMATION
C
      INDEX = 0
 1000 INDEX = INDEX + 1
```

```
      READ(1,100)  (INFO(I),I=1,19),RUNOPT
      IF(INFO(1).GE.900000) CALL EXIT
      WRITE(3,900) (INFO(I),I=2,19)
  900 FORMAT(1H1//5X,'URBAN STORM RUNOFF MODEL FOR SMALL WATERSHEDS '/5X
     1,9('------')// 10X,'RUN INFORMATION - ',18A4)
      IF(RUNOPT.GT.2.01) RUNOPT = 0.0
      IF(RUNOPT.LT.0.5) WRITE(3,922)
      IF(RUNOPT.GT.0.5.AND.RUNOPT.LT.1.5) WRITE(3,923)
      IF(RUNOPT.GT.1.5) WRITE(3,924)
  924 FORMAT(/28X,'STORM PERIOD SIMULATION USING SELECTED STORM FROM RUN
     1OFF FILE')
  922 FORMAT(/28X,'STORM PERIOD SIMULATION, PRECIPITATION DATA REQUIRED'
     1 )
  923 FORMAT(/28X,'FLOOD FREQUENCY SIMULATION USING RUNOFF FILE' )
C
C  SET INITIAL VALUES
C
      IERROR=0
      ISOIL(1) = 0
      ISOIL(2) = 0
      ISOIL(3) = 0
      ISOIL(4) = 0
      DO 1004 N = 1,100
      PREC(N)=0.0
      POLD(N) = 0.0
      IFPT(N) = 0
      CUML(N) = 0.0
      Q(N,2)=0.0
      AOD(N)=0.0
      SUMQ(N)=0.0
      SUMP=0.0
      DO 1003 M = 1,120
      OUT(N,M) = 0.0
 1003 IN(N,M) = 0.0
 1004 A(N) = 0.0
      DO 1019 N = 1,20
      PREC(N+100)=0.0
 1019 IO(N) = 0
      IF(RUNOPT.GT.0.5) IO(10) = 1
      DO 1005 N = 1,25
      OLDIN(N) = 0.0
      OLDOUT(N) = 0.0
      DO 1005 M = 1,20
      F(N,M) = 0.0
 1005 STOR(N,M) = 0.0
      IYCHAN = 0
      IYAREA = 0
      GO TO 1002
C
C INPUT NUMBER OF SEGMENTS, TOTAL TIME, TIME INCREMENT, TIME UNITS
C
 1002 CONTINUE
      READ(1,104)NELMTS,TOTALT,DELT,TUNITS,AU,PK,OLCHK,STCHK
      IF(AU.EQ.0)  AU=1
      IF(TUNITS.EQ.0) TUNITS=60
      IOCHK = 1
      IF(OLCHK+STCHK.LT.1.0E-5) IOCHK = 0
```

```
      IF(OLCHK.LT.1.0E-6)  OLCHK = 0.001
      IF(STCHK.LT.1.0E-6)  STCHK = 0.01
      IF(PK.LT.0.01.OR.PK.GT.10.0)  PK = 1.0
      NCT = 0
      IT = TOTALT/DELT + 0.001
      IF(TUNITS.NE.1) GO TO 1101
      PC =DELT/3600.0
      GO TO 1104
 1101 IF(TUNITS.NE.60) GO TO 1102
      PC = DELT/60.0
      GO TO 1104
 1102 IF(TUNITS.NE.3600) GO TO 1103
      PC=DELT
      GO TO 1104
 1103 WRITE(3,995) TUNITS
  995 FORMAT(///5X,'1)   I CANNOT ACCEPT DATA WITH ',I4,' AS A TIME UNIT
     1.'/7X,'TIME UNITS MUST BE 1(SEC) OR 60(MIN) OR 3600(HR).'/)
      IERROR=IERROR+1
 1104 CONTINUE
      PCS = PC*3600.
      IF(AU.EQ.1.OR.AU.EQ.640)  GO TO 1105
      WRITE(3,994)  AU
  994 FORMAT(///5X,'1)  I CANNOT ACCEPT DATA WITH ',I4,' AS AN AREA UNIT
     1.'/7X'AREA UNITS MUST BE 1(ACRE) OR 640(SQ MILE).'/)
      IERROR=IERROR+1
 1105 CONTINUE
      IF(IT.LE.120)  GO TO 1007
      WRITE(3,997) TOTALT,DELT,IT
  997 FORMAT(///5X,'3)  A TOTAL TIME OF ',T4,' WITH AN INCREMENT OF ',I4
     1,' GIVES ',I4,' STEPS WHICH EXCEEDS THE 120 LIMIT.'/7X,' REDUCE TO
     2TALT OR INCREASE DELT.')
      IERROR=IERROR+1
 1007 IF(NELMTS.LT.100)  GO TO 1008
      WRITE(3,996) NELMTS
  996 FORMAT(///5X,'4)  NUMBER OF SEGMENTS SPECIFIED (',I4,') EXCEEDS LI
     1MIT OF 100.')
      IERROR=IERROR+1
 1008 CONTINUE
C
C  INPUT  PHYSICAL DISCRIPTORS OF EACH SEGMENT.
C
      KNTIO = 0
      KOUNT = 0
      DO 1006 K = 1,100
      CUMA(K) = 0.0
 1006 IAN(K) = 0
      TYPE(100) = 99
      K = 0
C
C  RETURN HERE FOR EACH SEGMENT
C
 1500 K = K + 1
      READ(1,101)   N,TYPE(N),INTO(N),NS(N)
      IF(N.LE.100.AND.N.GT.0) GO TO 1606
      WRITE(3,1666) N
 1666 FORMAT(//10X,'5) SEGMENT NUMBER',I4,' IS GREATER THAN MAXIMUM ALLO
     1WED WHICH IS 100. ')
```

```
      IERROR=IERROR+1
 1606 IF(TYPE(N).EQ.99) GO TO 1551
      IF(N.GT.NELMTS) GO TO 1551
      IF(TYPE(N).EQ.55.AND.K.EQ.N) GO TO 1500
      IF(K.EQ.N) GO TO 1609
      IF(K.LT.N) GO TO 1607
      WRITE(3,1665) N
 1665 FORMAT(/10X,'6) SEGMENT',I3,' DISCRIPTION CARD IS OUT OF '
     1   ,'ORDER')
      IERROR=IERROR+1
 1607 NM1 = N - 1
      DO 1608 I=K,NM1
 1608 TYPE(I) = 55
      K = N
 1609 CONTINUE
      NSAME = NS(N)
      IF(NSAME.LE.0)  GO TO 1925
      IF(NSAME.LT.N)  GO TO 1022
      WRITE(3,978) N,NSAME
  978 FORMAT(/10X,I4,'7) SEGMENT CANNOT BE LIKE',I4,' WHICH IS LARGER.')
      IERROR=IERROR+1
 1022 CONTINUE
      IF(TYPE(N).NE.11) GO TO 1070
      ITYPE(N) = -2
      CUMA(N) = AREA(NSAME)
      ODEC(N) = ODEC(NSAME)
      OFCT(N) = OFCT(NSAME)
      CFSIN(N) = CFSIN(NSAME)
      KOUNT = KOUNT + 1
      IAN(N) = KOUNT
      AREA(N)=AREA(NSAME)
      LENGTH(N)=LENGTH(NSAME)
      SLOPE(N)=SLOPE(NSAME)
      FF(N)=FF(NSAME)
      SL(N)=SL(NSAME)
      RTL(N)=RTL(NSAME)
      ER(N)=ER(NSAME)
      DLTKR(N)=DLTKR(NSAME)
      GO TO 1050
 1070 IF(TYPE(N).NE.22)  GO TO 1071
      ITYPE(N) = 0
      KNTIO=KNTIO+1
      NIO(N)=KNTIO
      MPARM(N) = MPARM(NSAME)
      APARM(N) = APARM(NSAME)
      KIND(N)=KIND(NSAME)
      LENGTH(N)=LENGTH(NSAME)
      SLOPE(N)=SLOPE(NSAME)
      FF(N)=FF(NSAME)
      GO TO 1050
 1071 WRITE(3,979) N,TYPE(N),NSAME
  979 FORMAT(/10X,'8) ',I4,' SEGMENT TYPE ',I4,' CANNOT BE LIKE ANOTHER.
     1',I4)
      IERROR=IERROR+1
      GO TO 1050
 1925 CONTINUE
      IF(INTO(N).LT.1.OR.INTO(N).GT.NELMTS+1)  GO TO 1024
```

```
      IF(N.GT.0.AND.N.LE.NELMTS)   GO TO 1026
 1024 WRITE(3,989) N,NELMTS
  989 FORMAT(/2X,'9) ',I10,' SEGMENT NUMBER EXCEEDS MAXIMUM SPECIFIED,',
     1 I5)
      IERROR=IERROR+1
 1026 CONTINUE
C
C    AREA SEGMENT DISCRIPTION
C
      IF(TYPE(N).NE.11)  GO TO 1010
      IYAREA = 1
      ITYPE(N) = -1
      READ(1,103) AREA(N),LENGTH(N),SLOPE(N),FF(N),SL(N),RTL(N),LR(N)
     1       ,DLTKR(N),U(N)
      IF(AU.EQ.640)   AREA(N) = AREA(N)*640.0
      IF(AREA(N).LT.1.0E-5)   GO TO 1023
      IF(AREA(N).GT.3000.0) GO TO 1023
      IF(RTL(N).LT.1.0.AND.RUNOPT.LT.0.5)   GO TO 1023
      IF(U(N).GT.1.0) GO TO 1023
      CUMA(N) = AREA(N)
      IF(IO(10).NE.1) GO TO 1029
      X = ABS(1.0-SOIL1(N)-SOIL2(N)-SOIL3(N)-SOIL4(N))
      IF(SOIL1(N).GT.0.001) ISOIL(1) = 1
      IF(SOIL2(N).GT.0.001) ISOIL(2) = 1
      IF(SOIL3(N).GT.0.001) ISOIL(3) = 1
      IF(SOIL4(N).GT.0.001) ISOIL(4) = 1
      IF(X.LT.0.02) GO TO 1029
       WRITE(3,1028) SOIL1(N),SOIL2(N),SOIL3(N),SOIL4(N),N
 1028 FORMAT(//10X,'11) SUM OF FRACTION OF SOIL TYPES (',
     1 4F7.3,') DOES NOT EQUAL 1.0 FOR SEGMENT',I3)
      IERROR=IERROR+1
 1029 CONTINUE
      IF(AREA(N).GT.10.0) FF(N) = 0.0
      IF(FF(N).LT.1.0E-5) GO TO 1605
      ODEC(N) = FF(N)**0.6*LENGTH(N)**1.6*0.000818/SLOPE(N)**0.3
      OFCT(N) = 1.486*SQRT(SLOPE(N))*AREA(N)*43560.0/(FF(N)*LENGTH(N)
     1       **2.67)
 1605 CFSIN(N) = PC*3600.0/(AREA(N)*43560.0*12.0)
      IF(FF(N).GT.1.0E-5) GO TO 1610
      SK(N) = PK*(AREA(N)/640.0)**0.5*(1.0+U(N))**(-1.68)
      IF(TUNITS.EQ.1)   SK(N)=SK(N)*3600.
      IF(TUNITS.EQ.60)   SK(N)=SK(N)*60.
      IF(SK(N).GT.0.5*DELT)   GO TO 1509
      WRITE(3,1508) N,DELT,SK(N)
 1508 FORMAT(//5X,'10) SORRY BUT AREA SEGMENT ',I3,' IS TOO SMALL FOR TI
     1ME', ' STEP OF ',F6.2/10X,'STORAGE CONSTANT EQUALS',F8.3,
     1  10X,'YOU WILL NEED TO REDUCE TIME STEP OR GROUP AREA SEGMENTS.')
      IERROR=IERROR+1
 1509 CONTINUE
      SX(N) = DELT/(SK(N) + 0.5*DELT)
      SK(N) = 1.0 - SX(N)
C        SX(N) = MUSKINGUM COEF   C1 + C2
C          SK(N) = MUSKINGUM COFF  C3
 1610 KOUNT = KOUNT + 1
      IAN(N) = KOUNT
      IF(KOUNT.LE.50) GO TO 1050
 1608 I=TYPE(N)/10
```

```
      WRITE(3,1689) ISEQ(I),N
 1689 FORMAT(//10X,50('*')/15X,
     1      'SORRY BUT I CANNOT DIGEST MORE THAN 60',
     1 A4,'SEGMENTS'/15X,'SEGMENT',I3,' MADE ME BURP.'/10X,50('*'))
      IERROR=IERROR+1
C
C   CHANNEL SEGMENT DISCRIPTION
C
 1010 IF(TYPE(N).NE.22) GO TO 1020
      IYCHAN = 1
      KNTIO=KNTIO+1
      NIO(N)=KNTIO
      IF(KNTIO.GT.65) GO TO 1686
      ITYPE(N) = 0
      READ(1,108) KIND(N),LENGTH(N),SLOPE(N),FF(N),APARM(N),MPARM(N)
      IF(KIND(N).EQ.4) READ(1,103) FNR,ART,WPR,FNL,ALT,WPL,ACX
      IF(LENGTH(N).LT.1.0)  GO TO 1023
      IF(LENGTH(N).GT.9000.0) GO TO 1023
      IF(SLOPE(N).GT.0.9) GO TO 1023
      IF(FF(N).GT.0.5) GO TO 1023
      IF(KIND(N).NE.0)  GO TO 1015
      IF(APARM(N).LT.1.0E-6)  GO TO 1023
      IF(MPARM(N).LT.0.01 )  GO TO 1023
      APARM(N+100) = -1.0
      MPARM(N+100) = -1.0
      CHANA(N) = 1.0E10
      GO TO 1050
 1015 IF(KIND(N).NE.1)  GO TO 1011
      IF(APARM(N).GT.900.0.OR.APARM(N).LT.0.9) GO TO 1023
      APARM(N) = 1.49*SQRT(SLOPE(N))/(FF(N)*APARM(N)**0.667)
      MPARM(N) = 1.67
      APARM(N+100) = -1.0
      MPARM(N+100) = -1.0
      CHANA(N) = 1.0E10
      GO TO 1050
 1011 IF(KIND(N).NE.3) GO TO 1012
      IF(APARM(N).GT.50.0.OR.APARM(N).LT.0.04) GO TO 1023
      APARM(N)=1.49*(APARM(N)*4.)**0.667*SQRT(SLOPE(N))/FF(N)
      MPARM(N) = 1.0
      APARM(N+100) = -1.0
      MPARM(N+100) = -1.0
      CHANA(N) = 1.0E10
      GO TO 1050
 1012 IF(KIND(N).NE.2) GO TO 1013
      IF(APARM(N).GT.900.0.OR.APARM(N).LT.0.08) GO TO 1023
      APARM(N)=1.49*SQRT(SLOPE(N))/(FF(N)*APARM(N)**0.333)
      MPARM(N) = 1.33
      APARM(N+100) = -1.0
      MPARM(N+100) = -1.0
      CHANA(N) = 1.0E10
      GO TO 1050
 1013 CONTINUE
      IF(KIND(N).NE.4) GO TO 1014
      WPC = MPARM(N)
      AC = APARM(N)
      QC = 1.49*AC*(AC/WPC)**0.667*SQRT(SLOPE(N))/FF(N)
      ACX = ACX + AC
```

```
      QS = ACX*(ACX/WPC)**0.667/FF(N)
     1        + ART*(ART/WPR)**0.667/FNR
     2             + ALT*(ALT/WPL)**0.667/FNL
      QS = 1.49*SQRT(SLOPE(N))*QS
      AS = ART + ALT + ACX
      MPARM(N) = 1.5
      APARM(N) = EXP(ALOG(QC)-MPARM(N)*ALOG(AC))
      MPARM(N+100) = ALOG(QS/QC)/ALOG(AS/AC)
      APARM(N+100) = EXP(ALOG(QC)-MPARM(N+100)*ALOG(AC))
      CHANA(N) = AC
      GO TO 1050
 1014 CONTINUE
      WRITE(3,992) KIND(N)
  992 FORMAT(//5X,'12) ',I4,' IS NOT RECOGNIZED AS A KIND OF CHANNEL REA
     1CH.')
      IERROR=IERROR+1
      GO TO 1050
 1023 WRITE(3,988) N,AREA(N),LENGTH(N),SLOPE(N),FF(N),SL(N),RTL(N),ER(N)
     1   ,APARM(N),MPARM(N),U(N)
  988 FORMAT(/5X,'13) PARAMETER FOR SEGMENT',I3,' IMPROPER'/10X,10F12.5)
      IERROR=IERROR+1
      GO TO 1050
C
C STORAGE SEGMENT DISCRIPTION
C
 1020 IF(TYPE(N).NE.33) GO TO 1030
      ITYPE(N) = 1
      KNTIO=KNTIO+1
      NIO(N)=KNTIO
      READ(1,100) VU,TU,SK(N),SX(N),ICHK
      IF(NCT.EQ.0.AND.ICHK.GE.1) WRITE(3,898)
  898 FORMAT(//8X,'ADDITIONAL INFORMATION ON STORAGE SEGMENTS'/
     1  8X,42('-')/)
      NCT = NCT + 1
      IF(ICHK.GE.1) WRITE(3,897) N
  897 FORMAT(/10X,'STORAGE SEGMENT ',I3/10X,10('-')/)
      NSTOR(N) = NCT
      NST(NCT) = N
      IF(VU.LE.0) VU = 2
      IF(VU.GT.2) VU = 2
      IF(TU.LE.0) TU = 60
      IF(SK(N).LT.1.0E-8)  GO TO 1027
      IF(TU.EQ.TUNITS)  GO TO 1035
      IF(TU.NE.1)  GO TO 1031
      SK(NCK) = SK(NCK)/60.0
      IF(TUNITS.EQ.3600) SK(NCK) = SK(NCK)/60.0
      GO TO 1035
 1031 IF(TU.NE.60)  GO TO 1032
      IF(TUNITS.EQ.1)  SK(NCK) = SK(NCK)*60.0
      IF(TUNITS.EQ.3600)  SK(NCK) = SK(NCK)/60.0
      GO TO 1035
 1032 IF(TU.NE.3600)  GO TO 1033
      SK(NCK) = SK(NCK)*60.0
      IF(TUNITS.EQ.1)  SK(NCK) = SK(NCK)*60.0
      GO TO 1035
 1033 WRITE(3,985) TU
  985 FORMAT(15X,'14) ',I4,'IS NOT ACCEPTABLE FOR TU'/' ACCEPTABLE VALUE
```

```
     1S-1(SEC),60(MIN),3600(HOUR)')
      IERROR=IERROR+1
 1035 CONTINUE
      DDD = 1.0/(SK(N)-SK(N)*SX(N) + 0.5*DELT)
      C1(NCT) = (0.5*DELT - SK(N)*SX(N))*DDD
      C2(NCT) = (0.5*DELT + SK(N)*SX(N))*DDD
      C3(NCT) = (SK(N) - SK(N)*SX(N) - 0.5*DELT)*DDD
      GO TO 1050
 1027 CONTINUE
      CALL STODIS(ICHK)
      DO 1025 I = 1,20
      OUT(NCT,I) = SELEV(I)
      OUT(NCT,I+20) = ELEV(I)
      OUT(NCT,I+40) = SAREA(I)
      F(NCT,I) = DIS(I)
 1025 STOR(NCT,I) = VOL(I)
      GO TO 1050
 1030 WRITE(3,999) TYPE(N)
  999 FORMAT(//5X,'23)  I DO NOT NO WHAT TO DO WITH, ',I4,' AS AN SEGMEN
     1T TYPE'/28X,'------'/28X,'------')
      IERROR=IERROR+1
 1050 GO TO 1500
 1551 CONTINUE
      DO 1555 I = 1,NELMTS
      K = INTO(I)
      IF(K.GT.100) GO TO 1552
 6954 IF(K.LT.1) K = 100
      IF(TYPE(K).EQ.55) GO TO 1554
      IF(.NOT.(TYPE(I).EQ.22.OR.TYPE(I).EQ.33)) GO TO 1555
      DO 1553 J = 1,100
 1553 IF(INTO(J).EQ.I) GO TO 1555
      WRITE(3,1561) TYPE(I),I
 1561 FORMAT(//10X,'26) ',A4,' SEGMENT ',I4,' HAS NOTHING '
     1 'FLOWING INTO IT.' / 10X,'PLEASE CORRECT BY ELIMINATING'
     2 ' THE SEGMENT OR PUTTING SOMETHING INTO IT.')
      IERROR=IERROR+1
      GO TO 1555
 1552 WRITE(3,1549) I,K
 1549 FORMAT(//10X,'24) SEGMENT',I3,' DISCHARGES TO SEGMENT',
     1 I3,' WHICH IS A NONO')
      IERROR=IERROR+1
      GO TO 6954
 1554 WRITE(3,1556) I,K
 1556 FORMAT(/10X,'25) SEGMENT',I3,' DISCHARGES TO DUMMY SEGMENT'
     1  ,I3,' WHICH IS A MISTOOK.')
      IERROR=IERROR+1
 1555 CONTINUE
C
C  INPUT THE OUTPUT OPTIONS(OPT) AND SEGMENT NUMBER OUTPUT DESIRED(OUPT)
C
      NOP = 51
      READ(1,101) (OPT(I),I=1,20)
      DO 1054 K = 1,20
      I = OPT(K)
      IF(I.GT.20)  GO TO 1056
 1054 IF(I.GT.0) IO(I) = 1
      IF(IO(12).EQ.1)  NOP = 10
```

```
      IF(IO(11).EQ.1)  NOP = 0
 1055 READ(1,101) (OUPT(I),I=1,20)
      IF(OUPT(1).EQ.9999) GO TO 1052
      DO 1057 K = 1,20
      I = OUPT(K)
      IF(I.EQ.5555) GO TO 1055
      IF(I.GT.100)  GO TO 1056
 1057 IF(I.GT.0) IFPT(I) = 1
      GO TO 1058
 1052 DO 1051 K = 1,100
 1051 IFPT(K) = 1
      GO TO 1058
 1056 WRITE(3,990) (OPT(I),I=1,20),(OUPT(I),I=1,20)
  990 FORMAT(5X,'27) OUTPUT SPECIFICATIONS IMPROPER'/10X,'OPT = ',20I5/1
     110X,'OUPT =',20I5)
      IERROR=IERROR+1
 1058 CONTINUE
C
C  INPUT IF INPUT OPTION 8 SELECTED
C
      IF(IO(8).NE.1) GO TO 1559
      READ(1,103)XDLTKR,XSTRKR,XAPRM,XMPRM
      DO 1558 I = 1,NELMTS
       IF(TYPE(I).NE.11)  GO TO 1558
      IF(SL(I).LT.0.0001) GO TO 1558
      SL(I) = XSTRKR
      DLTKR(I) = XDLTKR
 1558 CONTINUE
      DO 1557 I=1,NELMTS
      IF(TYPE(I).NE.22)GO TO 1557
      IF(KIND(I).NE.0)GO TO 1557
      IF(APARM(I).LT.0.000001)GO TO 1557
      APARM(I)=XAPRM
      MPARM(I)=XMPRM
 1557 CONTINUE
 1559 CONTINUE
C
C PRINT OUT INPUT SPECIFICATIONS
C
      IF(ICHK.LE.0) GO TO 899
      WRITE(3,900) (INFO(I),I=2,19)
      IF(RUNOPT.LT.0.5) WRITE(3,922)
      IF(RUNOPT.GT.0.5.AND.RUNOPT.LT.1.5) WRITE(3,923)
      IF(RUNOPT.GT.1.5) WRITE(3,924)
  899 CONTINUE
      WRITE(3,901) NELMTS
  901 FORMAT( /10X,'THIS SIMULATION RUN HAS A TOTAL OF',I4,
     1  ' SOURCE AREAS, PONDING AREAS, AND/OR CHANNEL SEGMENTS'/)
      IF(TUNITS.NE.1) GO TO 8201
      WRITE(3,8202) DELT,TOTALT
      GO TO 8210
 8201 IF(TUNITS.NE.60) GO TO 8203
      WRITE(3,8204) DELT,TOTALT
      GO TO 8210
 8203 WRITE(3,8204) DELT,TOTALT
 8210 CONTINUE
 8202 FORMAT(10X,'COMPUTATIONS WILL BE MADE ON A',F4.1,1X,'SECOND TIME 1
```

```
     1NCREMENT FOR A TOTAL OF',F6.1,' SECONDS'/)
8204 FORMAT(10X,'COMPUTATIONS WILL BE MADE ON A',F4.1,1X,'MINUTE TIME I
     1NCREMENT FOR A TOTAL OF',F6.1,' MINUTES'/)
8206 FORMAT(10X,'COMPUTATIONS WILL BE MADE ON A',F4.1,1X,'HOUR TIME INC
     1REMENT FOR A TOTAL OF',F6.1,' HOURS'/)
      IF(IOCHK.EQ.1) WRITE(3,904) OLCHK,STCHK
 904 FORMAT(10X,'ACCURACY OBJECTIVES FOR OVERLAND FLOW'
     1,' AND STORAGE ITERATIONS ARE ',3F9.5/)
      WRITE(3,906) PK
 906 FORMAT(10X,'COEFFICIENT FOR STORAGE CONSTANT FOR LINEAR ROUTING ',
     1        'FROM SOURCE AREAS IS',F6.2,1X,A4)
      WRITE(3,921)
 921 FORMAT(/10X,'OUTPUT OPTIONS ARE '/)
      IF(IO(1).EQ.1)   WRITE(3,1081)
      IF(IO(2).EQ.1)   WRITE(3,1082)
      IF(IO(3).EQ.1)   WRITE(3,1083)
      IF(IO(5).EQ.1)   WRITE(3,1085)
      IF(IO(6).EQ.1)   WRITE(3,1086)
      IF(IO(8).EQ.1) WRITE(3,1093)
      IF(IO(9).EQ.1) WRITE(3,1094)
      IF(IO(14).EQ.1) WRITE(3,1097)
      IF(IO(16).EQ.1) WRITE(3,1095)
      IF(IO(17).EQ.1) WRITE(3,1096)
      IF(IO(18).EQ.1)  WRITE(3,1088)
      IF(IO(19).EQ.1)  WRITE(3,1089)
      IF(IO(20).EQ.1)  WRITE(3,1091)
1081 FORMAT(15X,'1.  PLOT HYDROGRAPHS ON PRINTER')
1082 FORMAT(15X,'2,  PRINT DETAILED HYDROGRAPHS')
1083 FORMAT(15X,'3,  PRINT MAXIMUM FLOWS AT ALL POINTS')
1085 FORMAT(15X,'5,  PRINT RAINFALL AND RUNOFF VOLUMES')
1086 FORMAT(15X,'6,  PRINT MAXIMUM VOLUMES FOR STORAGE SEGMENTS')
1088 FORMAT(15X,'18, COMPARE DISCHARGE AT TWO POINTS ON PLOT')
1089 FORMAT(15X,'19, WRITE DISCHARGE AT LAST FLOW POINT ON FILE 10')
1091 FORMAT(15X,'20, READ INPUT FROM PREVIOUS SIMULATION')
1093 FORMAT(15X,'8,  CHANGE OLFKR AND STRKR FOR PERVIOUS AREAS')
1094 FORMAT(15X,'9,  OUTPUT MAX FLOWS AT SELECTED POINTS')
1095 FORMAT(15X,'16, PRINT STAGE-DISCHARGE TABLES')
1096 FORMAT(15X,'17, LIST ALL PEAKS FROM RUNOFF FILE')
1097 FORMAT(15X,'14, PRINT INFLOW-OUTFLOW-STORAGE TABLES')
      WRITE(3,1092)
1092 FORMAT(//)
      DO 1060 N = 1,NELMTS
      I = INTO(N)
      IF(TYPE(N).EQ.55) TYPE(I) = 66
      KKN=TYPE(N)/10
      KKI=TYPE(I)/10
      IF(INTO(N).GT.0) GO TO 1061
      WRITE(3,1903) ISEG(KKN),N
1903 FORMAT(10X,A4,' SEGMENT',I5)
      GO TO 1060
1061 WRITE(3,903)  ISEG(KKN),N,ISEG(KKI),INTO(N)
1060 IF(IFPT(N).EQ.1)  WRITE(3,902)
 902 FORMAT('+',59X,'AND FLOWS WILL BE PRINTED')
 903 FORMAT(10X,A4,' SEGMENT',I5,' DISCHARGES TO ',A4,' SEGMENT',I5)
      IF(IYAREA.EQ.0)  GO TO 1160
      IF(IO(10).EQ.1) GO TO 1360
      WRITE(3,905)
```

```
 905 FORMAT(1H1///5X,'SPECIFICATIONS FOR SOURCE AREAS'/5X,31('-')//
    1 65X,'LOSS',20X,'RAIN',6X,'INITIAL',2X,'IMPERVIOUS',4X,'STORAGE'/
    1    5X,'NUMBER', 1X,
    1 'AREA-ACRE',3X,'LENGTH-FT',6X,'SLOPE',5X,'ROUGHNESS',7X,
    1 'RATE',7X,'RATIO',4X,'EXPONENT',8X,'LOSS',8X,'AREA', 2X,
    1 2X,'CONSTANT' /5X,125('-')/)
     DO 1062 N = 1,NELMTS
     Z = DELT/SX(N) - 0.5*DELT
1062 IF(TYPE(N).EQ.11) WRITE(3,907)N,AREA(N),LENGTH(N),SLOPE(N),
    1                        FF(N),SL(N),RTL(N),ER(N),DLTKR(N)
    1                       ,U(N),Z
 907 FORMAT(6X,I4,F11.3,F12.2,F12.5,F12.4,6F12.3)
     GO TO 1370
1360 WRITE(3,940)
 940 FORMAT(1H1//5X,'SPECIFICATIONS FOR SOURCE AREAS'/5X,31('-')//
    1 30X,'FRACTION OF AREA WITH EACH SOIL TYPE'/
    1   28X,41('-'),5X,'STORAGE'/
    2 5X,'NUMBER',1X,'AREA-ACRE',7X,'RAPID',4X,'MODERATE',8X,'SLOW',
    3 2X,'IMPERVIOUS',4X,'CONSTANT',3X,'LENGTH-FT',7X,'SLOPE',5X,
    4 'ROUGHNESS' /         5X,112('-')/)
     DO 1362 N = 1,NELMTS
     IF(TYPE(N).EQ.11) Z = DELT/SX(N) - 0.5*DELT
1362 IF(TYPE(N).EQ.11) WRITE(3,941)N,AREA(N),SOIL1(N),SOIL2(N),
    1 SOIL3(N),SOIL4(N),Z,LENGTH(N),SLOPE(N),FF(N)
 941 FORMAT(6X,I4,F11.3,4F12.3,F12.0,3F12.4)
1370 CONTINUE
     IF(NELMTS.GT.25)  WRITE(3,919)
     IF(NELMTS.LE.25)  WRITE(3,920)
 919 FORMAT(1H1)
 920 FORMAT(////)
1100 IF(IYCHAN.EQ.0)  GO TO 1164
     WRITE(3,909)
 909 FORMAT(,5X,'SPECIFICATIONS FOR CHANNEL SEGMENTS'/5X,35('-')//
    1 69X,'CHANNEL',12X,'FLOOD PLAIN       AVERAGE'/64X,17('-'),5X
    1    ,15('-'),5X,'TRAVEL'/10X,'NUMBER ',1X,
    1   'TYPE',3X,'LENGTH-FT',6X,'SLOPE',5X,'ROUGHNESS',5X,'APARM',
    2   7X,'MPARM',5X,'APARM',5X,'MPARM',2X,'TIME(SEC)'/10X,102('-')/)
     DO 1064 N = 1,NELMTS
     IF(TYPE(N).NE.22) GO TO 1064
     WRITE(3,911)N,KIND(N),LENGTH(N),SLOPE(N),FF(N),APARM(N),MPARM(N)
 911 FORMAT(10X,I4,3X,I4,F12.2,F12.5,3F12.3)
     IF(APARM(N+100).GT.0.0)ZZ= LENGTH(N)/(APARM(N)*CHANA(N)**MPARM(N)
    1                  /CHANA(N))
     IF(APARM(N+100).GT.0.0) WRITE(3,912) APARM(N+100),MPARM(N+100),ZZ
 912 FORMAT('<',80X,2F10.3,F10.1)
1064 CONTINUE
     WRITE(3,8901)
8901 FORMAT(//6X,'DEFINITION OF NUMERIC CHANNEL TYPES'//10X,'1 RECTANGU
    1LAR'/10X,'2 TRIANGULAR'/10X,'3 CIRCULAR'/10X,
    1 '0 IRREGULAR WITH NO FLOOD PLAIN'/10X,
    1 '4 IRREGULAR WITH FLOOD PLAIN')
1164 CONTINUE
     IF(NCT.LT.1) GO TO 1068
     DO 1065 I = 1,NCT
     IF(MOD(I,2).EQ.1) WRITE(3,914)
 914 FORMAT(1H1)
     NZ=NST(I)
```

```
      WRITE(3,913) NZ
  913 FORMAT(////10X,'SPECIFICATIONS FOR STORAGE SEGMENT',I3/10X,37('-')
     1 /)
      WRITE(3,915)
  915 FORMAT(/10X,'STORAGE',3X,'DISCHARGE',3X,'ELEVATION',6X,'HEAD',
     1  2X,'SURFACE AREA'/6X,'(ACRE-FEET)',5X,'(CFS)',5X,
     2  '(FEET-MSL)',4X,'(FEET)',4X,'(ACRES)'/6X,5(1X,11('-'))/)
      DO 1066 N = 1,20
 1066 WRITE(3,917) STOR(I,N),F(I,N),OUT(I,N),OUT(I,N+20),
     1  OUT(I,N+40)
  917 FORMAT(5X,5F12.3)
      DO 1065 N = 1,60
      OUT(I,N) = 0.0
 1065 CONTINUE
C  CONVERT STORAGE UNITS TO CFS
      DO 1067 I=1,NCT
      DO 1067 N=1,20
 1067 STOR(I,N)=STOR(I,N)*43560.0
 1068 CONTINUE
      IF (IO(10).EQ.1) GO TO 2051
C
C  INPUT PRECIPITATION DATA
C
      NK = -1
      DO 1090 K = 1,120
 1090 PREC(K) = 0.0
      READ(1,115) IFMTP,INC,ISTRT
      IF(ISTRT.LT.1) ISTRT = 1
      NNN = 0
 2000 IF(IFMTP.EQ.1) READ(1,105)NOSTA,YR,MO,DAY,HR,MIN,INC,
     1                          (ARRA(I),I=1,12)
      IF(IFMTP.EQ.2) READ(1,113)NOSTA,TI,TY,YR,MO,DAY,HR,
     1                          (ARRA(I),I=1,12)
      IF(MO.GT.12) GO TO 2020
      TYR=YR
      TMO=MO
      TDAY=DAY
      NK = NK + 1
      IF(NK.LT.10) GO TO 2005
      WRITE(3,998)
  998 FORMAT(///'28)  YOU ARE GIVING ME MORE PRECIPITATION CARDS THAN I
     1CAN HOLD, 120 VALUES IS THE LIMIT.')
      CALL EXIT
 2005 CONTINUE
      NKI = 13 - ISTRT
      DO 2010 I = 1,NKI
      N = I + ISTRT - 1
      IF(12*NK+I.GT.120) GO TO 2000
      NNN = NNN + 1
 2010 PREC(NNN) = ARRA(N)
      ISTRT = 1
      GO TO 2000
 2020 PCOR=ARRA(1)
      IF(PCOR.LT.0.01) PCOR=1.0
      DIST=INC/DELT
      NDIST=DIST
      DISTR=NDIST
```

```
      IF(NDIST.LT.1) GO TO 931
      IF(ABS(DIST-DISTP).LT.0.001) GO TO 2021
  931 WRITE(3,932) INC,DELT
  932 FORMAT(10X,'40) PRECIP INCREMENT OF',F5.2, ' MINUTES NOT COMPATIBL
     1E WITH SIMULATION INCREMENT OF',F5.2,' MINUTES')
      IERROR=IERROR+1
 2021 DO 2022 K=1,IT
      JD=(K-1)/NDIST+1
      RAIN(K)=PREC(JD)/DIST
 2022 CONTINUE
      DO 2023 K=1,IT
      PREC(K)=RAIN(K)*PCOR
 2023 CONTINUE
 2026 CONTINUE
C
C   INPUT STREAMFLOW DATA
C
      READ(1,114) NUM,IFMTS,INC,ISTRT,(NSD(I),I=1,NUM)
      IF(NUM.LE.0)  GO TO 2030
      IF(ISTRT.LT.1) ISTRT = 1
      DIST = INC/DELT
      NDIST = DIST
      DISTR = NDIST
      IF (NDIST.LT.1) GO TO 931
      IF (ABS(DIST-DISTR).GT.0.001) GO TO 931
      IF(NUM.GT.0.AND.NUM.LE.5)  GO TO 2024
      WRITE(3,991) NUM
  991 FORMAT(/5X,'29) I CAN ONLY ACCEPT 5 LOCATIONS FOR MEASURED STREAMF
     1LOW, YOU ARE GIVING ME',I5)
      CALL EXIT
 2024 K2 = 0
      DO 2029 I=1,NUM
      N = NSD(I)
      NSD(I) = 0
      NSD(N) = I
      N = I
      DO 2025 K = 1,IT
      SF(N,K)=0.0
 2025 CONTINUE
 2028 K2 = K2 + 12
      K1 = K2 - 11
      IF(K2.GT.IT/NDIST)  K2 = IT/NDIST
      IF(IFMTS.EQ.1)READ(1,107)NOSTA,(SF(N,K),K=K1,K2)
      IF(IFMTS.EQ.2)READ(1,113)NOSTA,TT,TY,YR,MO,DAY,HR,
     1                          (SF(N,K),K=K1,K2)
      IF(ISTRT.EQ.1) GO TO 2031
      K2 = 13 - ISTRT
      DO 2035 IK2=1,K2
      NKI = IK2 - 1 + ISTRT
 2035 SF(N,IK2) = SF(N,NKI)
      ISTRT = 1
 2031 CONTINUE
      IF(K2.LT.(IT/NDIST)) GO TO 2028
      IF(NDIST.EQ.1) GO TO 2029
      J=1
      DO 2027 K=1,IT
 2033 JK=K-(J-1)*NDIST
```

```
      DJK=JK
      IF(JK.LE.NDIST) GO TO 2032
      J=J+1
      GO TO 2033
 2032 IF(J.EQ.1) GO TO 2034
      QT(K)=SF(N,J-1)+(SF(N,J)-SF(N,J-1))*DJK/DIST
      GO TO 2027
 2034 QT(K)=SF(N,J)*DJK/DIST
 2027 CONTINUE
      DO 2037 K=1,IT
      SF(N,K) = QT(K)
 2037 CONTINUE
 2029 CONTINUE
 2030 CONTINUE
C
C   INPUT DATA FROM PREVIOUS SIMULATION
C
      IF(IO(20).NE.1) GO TO 2045
      READ(1,101) V,NM,(NE(I),I=1,NM)
      INM = NUM + 1
      NUM = NUM + NM
      DO 2040 K = INM,NUM
      N = NE(K-INM+1)
      NSD(N) = K
      N = K
 2040 READ(V,106) (SF(N,I),I=1,IT)
 2045 CONTINUE
C
C   COMPARE OUTPUT AT PAIRS OF SELECTED LOCATIONS
C
      IF(IO(18).NE.1) GO TO 2051
      IO(1) = 1
      READ(1,101) NCMP,(N1(I),N2(I),I=1,NCMP)
      DO 2046 I = 1,NCMP
      N = N1(I)
      NSC(N) = N2(I)
      NF = N2(I)
      NG(NF) = I + NUM
      IFPT(N) = 1
      IFPT(NF) = 1
 2046 CONTINUE
      NUM = NUM + NCMP
 2051 IF(RUNOPT.GT.1.5) READ(1,101,END=1000) I1,I2,I3
      IF(I1.EQ.99.AND.I2.EQ.99) GO TO 1000
 2055 CONTINUE
C
C    INPUT EXCESS RAINFALL FROM RUNOFF FILE
C
      IF(IO(10).NE.1) GO TO 2999
      IZN = INFO(1)
      IF(INFO(1).GT.4) IZN = 4
      IF(INFO(1).LT.1) IZN = 2
      I=DELT+0.01
      INF=11
      DO 2997 K=1,IZN
      READ(INF,101) IZ
      IF(IZ.LT.1.OR.IZ.GT.4) GO TO 2989
```

```
 6953 ISOIL(IZ) = 2
      READ(INF,2998,END=6000) IYR,IMO,IDAY,IHRS,NSTEP,(EXC(IZ,IW),
     1                  IW=1,120)
      IF(NSTEP.EQ.I) GO TO 2997
      WRITE(3,977) NSTEP,DELT
  977 FORMAT(///10X,50('*')//10X,'30) SORRY, BIG TROUBLE.'/15X,
     1  'TIME STEP ON TAPE',I5/15X,'TIME STEP SPECIFIED FOR THIS RUN',
     3  I5/10X,'THEY MUST BE THE SAME'//10X,50('*')/)
      IERROR=IERROR+1
      GO TO 2997
 2998 FORMAT (5I5/(12F6.3))
 2969 WRITE(3,2988) IZ
 2988 FORMAT(5X,'31) SOIL NUMBER(1,2,3 OR 4) FROM RUNOFF FILE'
     1  ,' IMPROPER'/ 5X,'  VALUE READ WAS',I8 )
      IERROR=IERROR+1
      GO TO 6953
 2997 INF=INF+1
      DO 2987 K = 1,4
      IF(ISOIL(K).NE.1) GO TO 2987
      WRITE(3,2986) K
 2986 FORMAT(//5X,'32) YOU HAVE SPECIFIED SOIL',I2,' FOR ONE',
     1 ' OF THE AREAS BUT HAVE NOT GIVEN THE PROGRAM THE',
     2 ' RUNOFF FILE FOR THAT SOIL')
      IERROR=IERROR+1
 2987 CONTINUE
      IF(IMO.GT.12) GO TO 6000
      IF(RUNOPT.LT.1.5) GO TO 2052
      IF(I1.NE.IYR) GO TO 2055
      IF(I2.NE.IMO) GO TO 2055
      IF(I3.NE.IDAY) GO TO 2055
 2052 CONTINUE
C
C    RESET ARRAYS TO ZERO
C
      DO 2996 T=1,120
      DO 2995 N=1,NELMTS
      NZ=NIO(N)
      IN(NZ,T) = 0.0
 2995 OUT(NZ,T) = 0.0
      DO 2994 N = 1,50
 2994 OF(N,T) = 0.0
 2996 CONTINUE
 2999 CONTINUE
      IF(IERROR.EQ.0) GO TO 6569
      WRITE(3,6570) IERROR
 6570 FORMAT(///10X,I5,' ERRORS IN INPUT CARDS - BETTER LUCK NEXT TIME')
      IF(IERROR.GT.25) WRITE(3,6571)
 6571 FORMAT(/10X,'DROP BACK 30 YARDS AND PUNT')
      CALL EXIT
 6569 CONTINUE
      WRITE(3,6572)
 6572 FORMAT(/////10X,'NO FATAL ERRORS IN INPUT CARDS DETECTED-BEGIN SIM
     1ULATION',1H1)
C
C BEGIN SIMULATION OF RUNOFF
C
C             T = TIME
```

```
C            N = SEGMENT NUMBER FOR OUTFLOW ARRAY
C            K = SEGMENT NUMBER FOR INFLOW ARRAY
      T = 0
 3000 T = T + 1
      N = 0
      IF(T.GT.IT) GO TO 5000
      DO 3002 K = 1,100
      Q(K,1) = Q(K,2)
 3002 Q(K,2) = 0.0
      DO 3003 K = 1,100
 3003 ADD(K) = 0.0
 3010 N = N + 1
      IF(N.GT.NELMTS) GO TO 3000
      IF(TYPE(N).EQ.11) GO TO 3100
      IF(TYPE(N).EQ.22) GO TO 3200
      IF(TYPE(N).EQ.33) GO TO 3300
      IF(TYPE(N).EQ.55) GO TO 3010
      IF(TYPE(N).EQ.99) GO TO 3000
      WRITE(3,999) TYPE(N)
      CALL EXIT
C
C  SEGMENT SIMULATION FOR SOURCE AREA
C
 3100 CONTINUE
      IF(IO(10).EQ.1) GO TO 3115
C
CALCULATION OF RAINFALL EXCESS VIA HEC-1,HYDROLOGIC ENGINEERING CENTER,
C
      IF(ITYPE(N).GE.-1)  GO TO 3108
      NSAME = NS(N)
      K = IAN(N)
      J = IAN(NSAME)
      EXC(K,T) = EXC(J,T)
      K = INTO(N)
      J = INTO(NSAME)
      Q(K,2)=Q(J,2)
      ADD(K)=ADD(J)
      GO TO 3010
 3108 CONTINUE
      X = 1.0
      RAIN(T)=PREC(T)+ADD(N)
      EXCESS = 0.0
      LOSS = 0.0
      IF(U(N).GT.0.999) GO TO 3109
      IF(CUML(N).LT.0.001) GO TO 3107
      X = RTL(N)**(0.1*CUML(N))
 3107 CONTINUE
      DLTK=0.0
      IF(DLTKR(N).GT.1.0E-6) DLTK=0.2*DLTKR(N)*(1.0-CUML(N)/DLTKR(N))**2
      IF(DLTK.LT.0.0)  DLTK = 0.0
      LOSS = DLTK + SL(N)/X
      IF(ER(N).GT.0.01) LOSS = LOSS*RAIN(T)**ER(N)
      EXCESS = (1.0-U(N))*(RAIN(T)-LOSS)
      IF(EXCESS.GT.0.0)  GO TO 3110
      EXCESS = 0.0
      LOSS = RAIN(T)
 3110 CUML(N) = CUML(N) + LOSS
```

```
 3109 EXCESS = EXCESS + U(N)*RAIN(T)
      K = IAN(N)
      JJ = INTO(N)
      IF(TYPE(JJ).NE.11) EXC(K,T) = EXCESS
      GO TO 3118
C
C    CALCULATION OF RAINFALL EXCESS FROM RUNOFF FILE
C
 3115 EXCESS = SOIL1(N)*EXC(1,T) + SOIL2(N)*EXC(2,T)
     1          + SOIL3(N)*EXC(3,T) + SOIL4(N)*EXC(4,T)
C
CALCULATION OF OVERLAND FLOW VIA CRAWFORD, STANFORD TECH.REPT.39
C
 3118 IF(FF(N).LT.0.1E-5) GO TO 3160
      DE = ODEC(N)*(EXCESS/PC)**0.6
      OFTM1 = 0.0
      IF(T.GT.1)  OFTM1 = OF(K,T-1)
      OFCHK = OFTM1
      NCK = 0
      Z = PC*LENGTH(N)/(23.8*AREA(N))
      Z2=Z*2.0
C         23.8=2.0*12.0*0.9917 : CFS-HR/ACRE TO FT
      EXCESS = EXCESS*LENGTH(N)/12.0
      IF(OLCHK.LT.0.99) GO TO 3120
      DNEW(N)=DOLD(N) + EXCESS
      ISKIP=0
      GO TO 3122
 3120 DNEW(N)=DOLD(N)+EXCESS-Z*(OFTM1+OFCHK)
      IF(DNEW(N).GT.1.0E-12) GO TO 3122
      DNEW(N)=DOLD(N)+EXCESS
      ISKIP=1
 3122 X = 1.0
      IF(DE.LT.1.0E-12)  GO TO 3124
      IF(DE.GT.DNEW(N))  X = DNEW(N)/DE
 3124 X = DNEW(N)*(1.0 + 0.6*X**3)
      IF(X.GT.1.0E-12)  X = X**1.667
      OF(K,T) = OFCT(N)*X
      IF(OLCHK.LT.0.99.AND.ISKIP.EQ.0) GO TO 3126
      IF(Z2*OF(K,T).GT.0.75*DNEW(N)) OF(K,T)=0.75*DNEW(N)/Z2
      DNEW(N)=DOLD(N)+EXCESS-Z2*OF(K,T)
      GO TO 3151
 3126 CONTINUE
      DIFF = OFCHK + OF(K,T)
      IF(DIFF.LT.1.0E-12)  GO TO 3150
      CHK = ABS(OFCHK-OF(K,T))/(0.5*DIFF)
      IF(CHK.LE.OLCHK)  GO TO 3150
      IF(NCK.EQ.NOP) WRITE(3,981) ODEC(N),DE,OFCT(N)
  981 FORMAT(2X,'ODEC=',E10.4,' DE=',E10.4,' OFCT=',E10.4)
      NCK = NCK + 1
      IF(NCK.GT.NOP)
     1        WRITE(3,982) T,N,NCK,OFCHK,OF(K,T),DOLD(N),DNEW(N),EXCESS
  982 FORMAT(2X,'TIME=',I3,' SEGMENT=',I3,' ITERATION=',I3,
     1  ' OLD-NEW FLOW='2E10.4,' OLD-NEW DEPTH=',2E10.4,' EXCESS=',E9.
      FD1 = FD(NCK)
      FD2 = 1.0 - FD1
      OFCHK = FD1*OF(K,T) + FD2*OFCHK
      IF(NCK.LT.21)  GO TO 3120
```

```
      WRITE(3,903) N,T
  903 FORMAT(//10X,'33) IN TWENTY TRIALS COULD NOT GET SOLUTION FOR SEGM
     1ENT',I4,'   AT TIME ',I5//)
      IF(CHK.GT.0.2)  GO TO 1000
      WRITE(3,964) OF(K,T),OFCHK
  964 FORMAT(10X,'HOWEVER, SINCE ESTIMATES DIFFER BY LESS THAT 20 PER CE
     1NT USED AVERAGE OF ',2E10.4/)
      OF(K,T) = 0.5*(OF(K,T) + OFCHK)
 3150 DNEW(N) = DOLD(N) + EXCESS - Z*(OFTM1 + OF(K,T))
 3151 DOLD(N)=DNEW(N)
      GO TO 3170
C
C     ROUTE EXCESS FROM SUBWATERSHED WITH LINEAR STORAGE MODEL
C
 3160 J = INTO(N)
      EXCESS = EXCESS/PC*AREA(N)*0.9917
      K = IAN(N)
      OF(K,T) = SX(N)*EXCESS
      IF(T.GT.1) OF(K,T) = OF(K,T) + SK(N)*OF(K,T-1)
C
C     CONTINUE ROUTING EXCESS WITH EITHER MODEL
C
 3170 CONTINUE
      J = INTO(N)
      JZ=NIO(J)
      Q(J,2) = Q(J,2) + OF(K,T)
      SUMQ(N)=SUMQ(N)+OF(K,T)
      IF(TYPE(J).EQ.11) ADD(J)=Q(J,2)*PC/AREA(J)
      IF(TYPE(J).EQ.33) IN(JZ,T)=IN(JZ,T)+OF(K,T)
      GO TO 3910
 3200 CONTINUE
C
C  SEGMENT SIMULATION FOR CHANNEL REACH VIA KINEMATIC ROUTING--EXPLICIT
C
      DT=PC*3600.
      DX=LENGTH(N)
      CD=A(N)
      NX = N
      IF(CHANA(N).LT.A(N)) NX = N + 100
      NZ=NIO(N)
      CC=IN(NZ,T)/APARM(NX)
      IF(CC.LT.1.0E-6) CC=1.0E-6
      CC=CC**(1.0/MPARM(NX))
      CB=CC+CD
      CB=(CB/2.)**(MPARM(NX)-1.)
      CB=CB*APARM(NX)*MPARM(NX)
      CA=Q(N,2)+Q(N,1)
      CA=CA/(2.*DX)
      A2=CA*DX*DT+CD*DX+CB*CC*DT
      A2=A2/(DX+DT*CB)
      A(N)=A2
      OUT(NZ,T)=0.0
      IF(A2.GT.1.0 E-10) OUT(NZ,T)=(A2**MPARM(NX))*APARM(NX)
      K=INTO(N)
      KZ=NIO(K)
      IN(KZ,T)=IN(KZ,T)+OUT(NZ,T)
      SUMQ(N)=SUMQ(N)+OUT(NZ,T)
```

```
      IF(TYPE(K).EQ.11) ADD(K) = IN(KZ,T)*PC/AREA(K)
      GO TO 3010
 3300 CONTINUE
C
C   SEGMENT SIMULATION FOR STORAGE EFFECTS
C
      K = NSTOR(N)
      J = INTO(N)
      NZ=NIO(N)
      JZ=NIO(J)
       IF(SK(N).LT.1.0E-5)  GO TO 3320
      OUT(NZ,T) = C1(K)*(IN(NZ,T))
      IF(T.GT.1) OUT(NZ,T) = OUT(NZ,T) + C2(K)*(IN(NZ,T-1))
     1  + C3(K)*OUT(NZ,T-1)
      GO TO 3010
 3320 STR(K,T) = (IN(NZ,T))*0.5 * PCS
      IF(T.GT.1)STR(K,T)=STR(K,T-1)+STR(K,T)+0.5*((IN(NZ,T-1))
     1  -OUT(NZ,T-1))*PCS
      IF(STR(K,T).GT.-0.00001) GO TO 3323
      OUT(NZ,T) = IN(NZ,T)
      STR(K,T) = STR(K,T) - 0.5*PCS*OUT(NZ,T)
      GO TO 3390
 3323 SOLD = STR(K,T)
      NCK = 0
      KS = 0
      IF(STR(K,T).GT.1.0E-9)  GO TO 3324
      STR(K,T) = 0.0
      OUT(NZ,T) = 0.0
      GO TO 3390
 3324 CONTINUE
      IF(STR(K,T).LE.STOR(K,20)) GO TO 3325
      WRITE(3,993) N
  993 FORMAT(2X,'34) STORAGE ON SEGMENT',I3,' HAS EXCEEDED THE EXTRAPOLA
     1TED MAXIMUM')
      OUT(NZ,T) = (STR(K,T) - STOR(K,20))/PCS
      STR(K,T) = STOR(K,20)
      GO TO 3390
 3325 CONTINUE
      I = 1
 3330 I = I + 1
      IF(SOLD.GT.STOR(K,I)) GO TO 3330
      OCH = OUT(NZ,T)
 3360 OUT(NZ,T) = F(K,I-1) + (F(K,I) - F(K,I-1))*(SOLD - STOR(K,I-1))
     1                              /(STOR(K,I)-STOR(K,I-1))
      ZZ = 0.0
      IF(KS.GT.0) GO TO 3365
      SOLD = SOLD - 0.5*OUT(NZ,T)  *PCS
      KS = 1
      IF(IO(13).EQ.1) WRITE(3,3395) N,T,IN(NZ,T),OCH,OUT(NZ,T),STR(K,T)
     1 ,SOLD
      IF(SOLD.GT.0.0) GO TO 3325
      Z = 1.0
      DO 3361 II = 1,10
      SOLD = SOLD + 0.5*(OUT(NZ,T)-Z*IN(NZ,T))*PCS
      OUT(NZ,T) = IN(NZ,T)*Z
      IF(SOLD.GT.0.0) GO TO 3325
 3361 Z = Z - 0.1
```

```
      OUT(NZ,T) = IN(NZ,T)
      STR(K,T) = STR(K,I) - 0.5*PCS*OUT(NZ,T)
      GO TO 3390
 3365 SNEW = SOLD + (OCH-OUT(NZ,T))*0.5 * PCS
      IF(IO(13).EQ.1)WRITE(3,3395)N,T,IN(NZ,T),OCH,OUT(NZ,T),SOLD,SNEW,ZZ
 3395 FORMAT(5X,'SEG',I5,'   TIME',I4,3X,'   IN-BA*OUT-BA*STOR',6F10.2)
      CHK = 1.0E20
      DO 3368 II = 1,11
      Z = II - 1
      Z = Z*0.1
      CC = (1.0-Z)*OCH + Z*OUT(NZ,T)
      SNEW = SOLD + (OCH-CC)*0.5*PCS
      IF(SNEW.LT.0.0) GO TO 3368
      DO 3366 I = 1,20
      KS = I
 3366 IF(SNEW.LT.STOR(K,I)) GO TO 3367
 3367 I = KS
      IF(I.LT.2) I = 2
      CB = F(K,I-1) + (F(K,I) - F(K,I-1))*(SNEW - STOR(K,I-1))
     1                              /(STOR(K,I)-STOR(K,I-1))
      CD = ABS(CC-CB)
      IF(CD.GT.CHK) GO TO 3368
      CHK = CD
      CA = CB
      AA = SNEW
      ZZ = Z
      C = CC
 3368 CONTINUE
      OUT(NZ,T) = CA
      OCH = C
      SOLD = AA
      CHK = ABS((OCH-OUT(NZ,T))/OUT(NZ,T))
      IF(CHK.LT.STCHK)  GO TO 3340
      NCK = NCK + 1
      IF(NCK.GT.5)  GO TO 3340
      GO TO 3365
 3340 CONTINUE
      IF(IO(13).EQ.1)WRITE(3,3395)N,T,IN(NZ,T),OCH,OUT(NZ,T),SOLD,SNEW,ZZ
      STR(K,T) = SOLD
      OUT(NZ,T) = OCH
 3390 SUMQ(N) = SUMQ(N) + OUT(NZ,T)
      IN(JZ,T) = IN(JZ,T) + OUT(NZ,T)
      GO TO 3010
 5000 CONTINUE
C
C  OUTPUT DESIRED HYDROGRAPHS, AND OTHER INFORMATION.
C
C
C
C
C  FIND MAXIMUM FLOW AT FLOWPOINTS
C
      DO 5100 N = 1,NFLMTS
      NZ=NIO(N)
      IF(TYPE(N).EQ.55) GO TO 5100
      IF(IO(3).EQ.1) GO TO 5008
      IF(IO(9).EQ.1.AND.IFPT(N).EQ.1) GO TO 5008
      IF(IFPT(N).EQ.1) GO TO 5008
      GO TO 5100
```

```
 5008 CONTINUE
      MAXFLO(N)  = 0.0
      IF(ITYPE(N)) 5009,5019,5019
 5009 DO 5010 T = 1,IT
      K = IAN(N)
      IF(MAXFLO(N).GE.OF(K,T)) GO TO 5010
      MAXFLO(N) = OF(K,T)
      IF(T.EQ.IT) WRITE(3,5199) N,IYR,IMO,IDAY
 5010 CONTINUE
      GO TO 5050
 5019 DO 5020 T = 1,IT
      IF(MAXFLO(N).GE.OUT(NZ,T)) GO TO 5020
      MAXFLO(N) = OUT(NZ,T)
      IF(T.EQ.IT) WRITE(3,5199) N,IYR,IMO,IDAY
 5199 FORMAT(3X,'35) ***** POTENTIAL PROBLEM *****'/9X,
     1  'MAXIMUM FLOW FROM SEGMENT',I3,' AND STORM(YEAR,MONTH,DAY)'
     2  ' 19',I2,2I3,' OCCURRED DURING LAST TIME STEP.'/ 9X,
     3  'A LARGER TIME STEP MAY BE NEEDED.')
 5020 CONTINUE
 5050 CONTINUE
 5100 CONTINUE
C
C OUTPUT OF STORAGE,INFLOW,OUTFLOW
C
      IF(IO(14).NE.1) GO TO 5552
      DO 5551 N=1,NELMTS
      NZ=NIO(N)
      IF(TYPE(N).NE.33) GO TO 5551
      K=NSTOR(N)
      WRITE(3,968) (INFO(I),I=2,19),N
  968 FORMAT(1H1,10X,'STORAGE FOR ',18A4,//10X,
     1'STORAGE SEGMENT',I4,//15X,'TIME',
     210X,'INFLOW',9X,'OUTFLOW',9X,'STORAGE',/
     3 15X,'----',3(10X,'-------'),//)
      DO 5553 T = 1,IT
      JT = FLOAT(T) *DELT + 0.01
 5553 WRITE(3,969) JT,IN(NZ,T),OUT(NZ,T),STR(K,T)
  969 FORMAT(15X,I4,7X,F9.1,7X,F9.1,7X,F9.1)
 5551 CONTINUE
 5552 IF(IO(3).NE.1) GO TO 5175
C
C   OUTPUT MAX. FLOWS AT ALL POINTS
C
      WRITE(3,950) (INFO(K),K=2,19),IMO,IDAY,IYR
      DO 5176 N=1,NELMTS
      IF(TYPE(N).EQ.55) GO TO 5176
      KKN=TYPE(N)/10
      WRITE(3,951) N,ISEG(KKN),MAXFLO(N)
 5176 CONTINUE
 5175 IF(IO(9).NE.1) GO TO 5200
C
C  OUTPUT MAXIMUM FLOWS AT DESIRED FLOWPOINTS
C
      WRITE(3,950) (INFO(K),K=2,19),IMO,IDAY,IYR
  950 FORMAT(////10X,'PEAK FLOWS FOR ',18A4/
     1 10X,'STORM OF ',3I5//10X,'SEGMENT',5X,'SEGMENT',
     1    8X,'PEAK' /10X,'NUMBER',8X,'TYPE',6X,'FLOW(CFS)'/10X,'-------',
```

```
     2  5X,'-------',6X,'-------'/)
      DO 5150 N = 1,NELMTS
      KKN=TYPE(N)/10
 5150 IF(IFPT(N).EQ.1) WRITE(3,951) N,TSEQ(KKN),MAXFLO(N)
  951 FORMAT(1X,I14,9X,A4,F13.2)
 5200 IF(IO(2).NE.1)  GO TO 5400
C
C  PRINTOUT DETAILED HYDROGRAPHS
C
      WRITE(3,952) (INFO(N),N=2,19)
  952 FORMAT(1H1,10X,'HYDROGRAPH DATA FOR ',20A4//)
      DO 5240 N = 1,NELMTS
      NZ=NIO(N)
      IF(TYPE(N).EQ.55) GO TO 5240
      IF(IFPT(N).NE.1)  GO TO 5240
      IF(ITYPE(N)) 5210,5220,5220
 5210 K = IAN(N)
      WRITE(3,953) N,TYPE(N),(OF(K,T),T=1,IT)
      GO TO 5240
 5220 WRITE(3,953) N,TYPE(N),(T,I=1,12),(OUT(NZ,T),T=1,IT)
  953 FORMAT(2X,'FLOWPOINT',I3,' DRAINS FROM ',I4,', (CFS)'/
     1  4X,12(7X,I3)/5X,12('  --------')/(5X,12F10.3))
 5240 CONTINUE
C
C  PLOT HYDROGRAPHS ON PRINTOUT
C
 5400 CONTINUE
      IF(IO(1).NE.1) GO TO 5600
      WRITE(3,954) (INFO(N),N=2,19)
  954 FORMAT(1H1//10X,'PLOTTED HYDROGRAPHS FOR ',18A4)
      DO 5405 N = 1,NELMTS
      NZ=NIO(N)
      IF(TYPE(N).EQ.55) GO TO 5405
      K = INTO(N)
      CUMA(K) = CUMA(KZ) + CUMA(NZ)
C     J = IAN(N)
C     IF(J.GT.0)  GO TO 5403
C     DO 5402 T = 1,IT
C5402 IN(NZ,T) = 0.0
C     GO TO 5405
C5403 DO 5404 T = 1,IT
C5404 IN(NZ,T) = EXC(J,I)*AREA(N)
 5405 CONTINUE
C     DO 5410 N = 1,NELMTS
C     NZ=NIO(N)
C     IF(TYPE(N).EQ.55) GO TO 5410
C     J = INTO(N)
C     JZ=NIO(J)
C     DO 5409 T = 1,IT
C5409 IN(JZ,T)=IN(JZ,T)+IN(NZ,T)
C5410 CONTINUE
      DO 5500 N = 1,NELMTS
      IF(TYPE(N).EQ.55) GO TO 5500
C     DO 5415 T = 1,IT
C5415 IN(N,T) = IN(N,T)/CUMA(N)
      IF(IFPT(N).NE.1)  GO TO 5500
      IF(IO(1).NE.1)  GO TO 5500
```

```
      NSF = 0
      IF(NSC(N).LE.0) GO TO 5418
      NKT = NSC(N)
      NSF = NG(NKT)
 5418 IF(NSD(N).GT.0) NSF = NSD(N)
      IF(NSF.LE.0) GO TO 5419
      DO 5417 T = 1,IT
      IF(IO(18)) 5416,5416,5414
 5414 NKT = NIO(NKT)
      IF(OUT(NKT,T).GT.MAXFLO(N)) MAXFLO(N) = OUT(NKT,T)
      GO TO 5417
 5416 IF(SF(NSF,T).GT.MAXFLO(N)) MAXFLO(N) = SF(NSF,T)
 5417 CONTINUE
 5419 CONTINUE
      DTEN = 0.00001
 5420 DTEN = DTEN*10.0
      IF(MAXFLO(N).GT.DTEN)  GO TO 5420
      J = MAXFLO(N)*5.0/DTEN + 1.0
      X = J*2
      XM = 0.1*X*DTEN
      X1 = XM*0.25
      X2 = XM*0.5
      X3 = XM*0.75
      KKN=TYPE(N)/10
      WRITE(3,957)  ISEG(KKN),N
  957 FORMAT(///20X,A4,' SEGMENT',I4,10X,'* = SIMULATED FLOW, O = OBSERV
     1ED FLOW (CFS)'/)
      WRITE(3,8000)
 8000 FORMAT(' TIME FLOW',105X,'RAIN EXCESS   %')
      IF(XM.GT.9.0) GO TO 8001
      IF(TUNITS.NE.1) GO TO 8011
      WRITE(3,8110) X1,X2,X3,XM
      GO TO 8190
 8011 IF(TUNITS.NE.60) GO  TO 8012
      WRITE(3,8111) X1,X2,X3,XM
      GO TO 8190
 8012 WRITE(3,8112) X1,X2,X3,XM
      GO TO 8190
 8001 IF(TUNITS.NE.1)GO TO 8021
      WRITE(3,8120) X1,X2,X3,XM
      GO TO 8190
 8021 IF(TUNITS.NE.60) GO TO 8022
      WRITE(3,8121) X1,X2,X3,XM
      GO TO 8190
 8022 WRITE(3,8122) X1,X2,X3,XM
      GO TO 8190
 8110 FORMAT(' SEC.  CFS 0.0',20X,F6.4,19X,F6.4,19X,F6.4,17X,F6.4,3X,'IN
    1.     IN.'/' ---- ----  -',20('-----'),3('  ----'))
 8111 FORMAT(' MIN.  CFS 0.0',20X,F6.4,19X,F6.4,19X,F6.4,17X,F6.4,3X,'IN
    1.     IN.'/' ---- ----  -',20('-----'),3('  ----'))
 8112 FORMAT(' HRS.  CFS 0.0',20X,F6.4,19X,F6.4,19X,F6.4,17X,F6.4,3X,'IN
    1.     IN.'/' ---- ----  -',20('-----'),3('  ----'))
 8120 FORMAT(' SEC.  CFS 0.0',20X,F6.1,19X,F6.1,19X,F6.1,17X,F6.1,3X,'IN
    1.     IN.'/' ---- ----  -',20('-----'),3('  ----'))
 8121 FORMAT(' MIN.  CFS 0.0',20X,F6.1,19X,F6.1,19X,F6.1,17X,F6.1,3X,'IN
    1.     IN.'/' ---- ----  -',20('-----'),3('  ----'))
 8122 FORMAT(' HRS.  CFS 0.0',20X,F6.1,19X,F6.1,19X,F6.1,17X,F6.1,3X,'IN
```

```
     1.     IN.'/' ---- ----  -',20('-----'),3('  ----'))
 8190 CONTINUE
      NSF = 0
      K = N
      IF(NSD(K).GT. 0) GO TO 5427
      IF(NSC(K) .LE. 0) GO TO 5428
      NKT = NSC(N)
      NSF = NG(NKT)
      GO TO 5430
 5427 NSF = NSD(K)
      GO TO 5430
 5428 CONTINUE
 5430 CONTINUE
      DO 5450 T = 1,IT
      DO 5434 K = 1,101
 5434 ICHAR(K) = IBLK
      ICHAR(1) = ICI
      ICHAR(26) = ICI
      ICHAR(51) = ICI
      ICHAR(76) = ICI
      ICHAR(101) = ICI
      IF(ITYPE(N)) 5436,5438,5438
 5436 K = IAN(N)
      I = OF(K,T)/XM*100.0 + 1.0
      Z = OF(K,T)
      GO TO 5439
 5438 NZ = NTO(N)
      I = OUT(NZ,T)/XM*100.0 + 1.0
      Z = OUT(NZ,T)
 5439 CONTINUE
      IF(I.GT.101) I = 101
      ICHAR(I) = ICX
      IF(NSF.LT.1)  GO TO 5441
      IF(IO(18) .EQ.1) SF(NSF,T) = OUT(NKT,T)
      I = SF(NSF,T)/XM*100.0 + 1.0
      IF(I.GT.101)  I = 101
      ICHAR(I) = ICO
 5441 CONTINUE
      JT = FLOAT(T) * DELT + 0.01
      IF(PREC(T).LT.0.001)  GO TO 5442
      IF(IN(N,T).LT.0.001)  GO TO 5444
C     PCT = 100.0*IN(N,T)/PREC(T)
C5443 WRITE(3,955) JT,Z,(ICHAR(K),K=1,101),PREC(T),IN(N,T),PCT
C     GO TO 5446
 5442 WRITE(3,955) JT,Z,(ICHAR(K),K=1,101)
      GO TO 5446
 5444 WRITE(3,955) JT,Z,(ICHAR(K),K=1,101),PREC(T)
  955 FORMAT(1X,I3,F7.1,1X,101A1,2F6.2,F6.1)
 5446 CONTINUE
 5450 CONTINUE
 5500 CONTINUE
C
C  OUTPUT OF MAXIMUM STORAGE
C
      IF(IO(6).NE.1) GO TO 5550
      WRITE(3,965) (INFO(I),I=2,19)
  965 FORMAT(1H1,10X,'PEAK STORAGE VOLUMES FOR',18A4,//15X,'SEGMENT',
```

```
     110X,'PEAK STORAGE--CU FT      AC FT       IN '/15X,'-------',10X,'---
     2---------  -----     -----      --'//)
      DO 5540 N=1,NELMTS
      IF(TYPE(N).NE.33) GO TO 5540
      MAXSTR(N) = 0.0
      K = NSTOR(N)
      DO 5535 T = 1,IT
      IF(MAXSTR(N).LT.STR(K,T)) MAXSTR(N) = STR(K,T)
 5505 CONTINUE
      AA=MAXSTR(N)
      B=AA/43560.
      C=B*12./CUMA(N)
      WRITE(3,966) N,AA,B,C
  966 FORMAT(I10,F30.1,F12.2,F8.2)
 5540 CONTINUE
C
C  OUTPUT OF RAINFALL AND RUNOFF VOLUMES
C
 5550 IF(IO(5).NE.1) GO TO 5600
C     WRITE(3,960) (INFO(I),I=2,19)
C     PC = 0.0
C     DO 5810 T = 1,IT
C5810 SUMP=SUMP+PREC(T)
C     DO 5800 N=1,NELMTS
C     IF(IFPT(N).NE.1) GO TO 5800
C     IF(TYPE(N).EQ.55) GO TO 5800
C     VOL1=SUMO(N)*PC*3600./43560.
C     VOL2=VOL1*12./CUMA(N)
C     IF(RUNOPT.LT.0.9) RC=VOL2*100./SUMP
C     WRITE(3,961) N,SUMP,VOL2,VOL1,PC
C 960 FORMAT(1H1,10X,'RAINFALL AND RUNOFF VOLUMES FOR',18A4,//15X,'ELEME
C    1NT',5X,'RAIN',5X,'RUNOFF',5X,'RUNOFF',5X,'PER'/28X,'IN.',7X,'IN.',6
C    2X,'AC-FT.',5X,'CENT'/15X,'-------',5X,'----',5X,'------',5X,'-----
C    3-',5X,'----'//)
C 961 FORMAT(I10,F12.2,F11.2,F11.3,F9.1)
 5800 CONTINUE
C
C  OUTPUT DISCHARGE ON CARDS OR FILE OF DISCHARGE AT LAST POINT.
C
 5600 IF(IO(19).NE.1) GO TO 5700
      NEL = NIO(NELMTS)
      IF(INDEX.NE.1)GO TO 5890
      WRITE(10,106)(OUT(NEL,T),T=1,IT)
 5890 IF(INDEX.NE.2) GO TO 5900
      IF(IO(10).EQ.1) GO TO 5700
      WRITE(11,106)(OUT(NEL,T),I=1,IT)
 5900 IF(INDEX.NE.3) GO TO 5910
      WRITE(12,106)(OUT(NEL,T),T=1,IT)
 5910 IF(INDEX.NE.4) GO TO 5920
      WRITE(13,106)(OUT(NEL,T),T=1,IT)
5920  IF(INDEX.NE.5) GO TO 5930
      WRITE(14,106)(OUT(NEL,T),T=1,IT)
 5930 IF(INDEX.NE.6) GO TO 5940
      WRITE(15,106)(OUT(NEL,T),T=1,IT)
 5940 IF(INDEX.NE.7) GO TO 5950
      WRITE(16,106)(OUT(NEL,T),T=1,IT)
 5950 IF(INDEX.NE.8) GO TO 5960
```

```
      WRITE(17,106)(OUT(NEL,T),T=1,IT)
 5960 IF(INDEX.NE.9) GO TO 5970
      WRITE(18,106)(OUT(NEL,T),T=1,IT)
 5970 CONTINUE
 5700 CONTINUE
C
C  WRITE PEAK FLOWS FOR FREQUENCY ANALYSIS
C
      IF(IO(10).NE.1) GO TO 1000
      IF(RUNOPT.GT.1.5) GO TO 2051
      DO 6050 N = 1,NELMTS
      IF(IFPT(N).NE.1) GO TO 6050
      WRITE(9,6001) N,IYR,IMO,IDAY,MAXFLO(N)
 6001 FORMAT(4I5,F12.2)
 6050 CONTINUE
      GO TO 2055
 6000 N = 999
      X = 0.0
      WRITE(9) N,IYR,IMO,IDAY,X
      REWIND 9
      CALL FREQ
      END
```

```
      SUBROUTINE STODIS(ICHK)
      REAL JS1,JS2
      REAL CR(12)
      DATA CR/4.2,3.89,3.57,3.10,2.46,2.02,1.71,1.47,1.28,1.14,1.02,1.0/
      INTEGER VU
      REAL CDIS(25),JEL,JELE
      COMMON /STOD/ VU,IERROR,SELEV(25),ELEV(25),AREA(25),VOL(25),DIS(25
     1)
      DO 200 IN=1,25
      SELEV(IN)=0.0
      ELEV(IN)=0.0
      AREA(IN)=0.0
      VOL(IN)=0.0
      DIS(IN)=0.0
  200 CONTINUE
      READ(1,10) KD,KE
   10 FORMAT(2I4)
      IF(KD.LE.1.AND.KE.LE.1) GO TO 5
      WRITE(3,95) KD,KE
   95 FORMAT(//10X,'15) *** ERROR-VALUES OF INDEX VARIABLES KE AND KD'
     1 ,2I5,' FOR STORAGE SEGMENT GREATER THAN 1.'//
     2   10X,'CARDS POSSIBLY OUT OF ORDER')
      IERROR=IERROR+1
    5 IF(KD.NE.1.AND.KE.NE.1) GO TO 40
      READ(1,11) K,(SELEV(I),I=1,K)
      DO 45 I=1,K
   45 ELEV(I)=SELEV(I)-SELEV(1)
      IF(K.LE.20) GO TO 40
      WRITE(3,31) K
   31 FORMAT(///10X,'16) ',I4,' IS TOO MANY ELEVATIONS FOR STORAGE SEGME
     1NT,MAX=21')
      IERROR=IERROR+1
   40 CONTINUE
      IF(KD.EQ.1) GO TO 50
      READ(1,11) N,(VOL(I),I=1,N)
   11 FORMAT(I8/(10F8.2))
      IF(N.LE.20) GO TO 42
      WRITE(3,94) N
   94 FORMAT(/10X,'17) ',I4,'IS TOO MANY VOLUMES FOR STORAGE SEGMENT,'
     1    ,' MAX.= 20')
      IERROR=IERROR+1
   42 IF(VU.EQ.2) GO TO 60
      DO 46 I = 1,N
   46 VOL(I) = VOL(I)/43560.
      GO TO 60
   50 READ(1,11) N,(AREA(I),I=1,N)
      IF(VU.EQ.2) GO TO 52
      DO 51 I = 1,N
   51 AREA(I) = AREA(I)/43560.
      IF(N.GT.20) GO TO 53
   52 IF(N.EQ.K) GO TO 55
   53 WRITE(3,30) N,K
   30 FORMAT(///10X,'18) NO. AREAS=',I4,' NO. ELEV=',I4,' MUST BE EQUAL'
     1,   ' AND LESS THAN 21')
      IERROR=IERROR+1
```

```
55 CONTINUE
   VOL(1)=0.0
   VOL(2)=(ELEV(2)-ELEV(1))*.5*(AREA(1)+AREA(2))
   DO 58 I=3,N
   I2=I-2
   I1=I-1
58 VOL(I)=(ELEV(I)-ELEV(I2))*.1667*(AREA(I)+4*AREA(I1)+AREA(I2))
  1        +VOL(I2)
60 CONTINUE
   IF(KE.EQ.1) GO TO 70
   READ(1,11) L,(DIS(I),I=1,L)
   IF(L.LE.20) GO TO 80
   WRITE(3,93) L
93 FORMAT(10X,'19) ',I4,' IS TOO MANY DISCHARGE VALUES FOR STORAGE',
  1  ' SEGMENT, MAX.= 20')
   IERROR=IERROR+1
70 READ(1,13) JA
13 FORMAT(I4)
   IF(JA.GT.6) WRITE(3,91) JA
91 FORMAT(//5X,50('*')//10X,'20) WARNING MESSAGE -- ARE YOU SURE',
  1 ' YOU WANT',I5/5X,'OUTLET STRUCTURES ON ONE OF THE STORAGE'
  2  ' SEGMENTS.'//5X,50('*')//)
   DO 72 I=1,25
72 DIS(I) = 0.0
   DO 78 IX=1,JA
   READ(1,12) JT,JEL,JS1,JS2
12 FORMAT(I4,5F8.3)
   IF(JT.GE.1.AND.JT.LE.6) GO TO 73
   WRITE(3,32)JT
32 FORMAT(///10X,'21) JT FOR STORAGE SEGMENT CANNOT EQUAL',I4)
   IERROR=IERROR+1
73 CONTINUE
   JELE = JEL-SELEV(1)
   IF(ICHK.GE.1) WRITE(3,36) JT,SELEV(1),JEL,JS1,JS2
36 FORMAT(//12X,'    OUTLET TYPE      ELEVATIONS           DIMENSIONS',
  1 /12X,52('-')/20X,I4,4F10.2)
   IF(JT.EQ.2) CSAREA=JS1*JS2
   IF(JT.EQ.1) CSAREA=3.14159*(JS1/2)**2
   IF(JT.EQ.3) CSAREA=3.14159*(JS1/2)*(JS2/2)
   IF(ICHK.GE.1) WRITE(3,99)
99 FORMAT(/11X,'  TOTAL Q    THIS Q    VOLUME    HEIGHT  BASE'
  1 ' HEIGHT   HEAD'/10X,60('-'))
   DO 77 I=1,K
   IF(JELE.LT.ELEV(I)) GO TO 74
   CDIS(I)=0.0
   B = 0.0
   BX = 0.0
   IF(I.EQ.K) WRITE(3,98) JT,SELEV(1),JEL,SELEV(I)
98 FORMAT(//5X,50('*')//10X,'22) WARNING MESSAGE'/10X,
  1 'NO DISCHARGE FROM ONE OF THE OUTLET WORKS'/10X,
  2 'OUTLET TYPE ',I6/10X,'ELEVATION OF BOTTOM OF STORAGE ',
  3 F10.2/10X,'ELEVATION AT BOTTOM OF THIS OUTLET STRUCTURE'
  4 ,F10.2/10X,'HIGHEST ELEVATION REACHED ',F10.2//
  5 5X,50('*')//)
   GO TO 76
74 CONTINUE
   B = ELEV(I) - JELE
```

```
      GO TO (700,700,700,710,720,730),JT
  700 BX = B
      B = B/JS1
      IF(B.LT.2) C=0.275+0.15*B
      IF(B.GE.2.AND.B.LT.4) C=0.49+0.04*B
      IF(B.GE.4) C=0.61+0.01*B
      IF(C.GE..75) C=0.75
      IF(C.LT..50) C=0.50
      IF(B.GE.1.0) CDIS(I)=C*CSAREA*SQRT(64.4*BX)
      IF(B.LT.1.0) CDIS(I)=C*CSAREA*SQRT(64.4*BX)*B
      GO TO 76
  710 CDIS(I) = 3.0*JS1*B**1.5
      GO TO 76
  730 RTO = 2.0*B/JS1
      IR = RTO/0.2
      IR = IR + 2
      IRM1 = IR - 1
      IF(IR.LE.12) GO TO 115
      C = CB(12)
      GO TO 120
  115 FCT = (RTO - FLOAT(IR-2)*0.2)/0.2
      C = CB(IRM1) + (CB(IR)-CB(IRM1))*FCT
  120 CDIS(I) = C*3.14*JS1*B**1.5
      GO TO 76
  720 CDIS(I)=4.8*B**1.5*(0.67*JS1+0.533*B*TAN(JS2*0.01744))
   76 CONTINUE
      DIS(I)=DIS(I)+CDIS(I)
      IF(JT.LT.4) B = BX
      IF(ICHK.GE.1) WRITE(3,39) DIS(I),CDIS(I),VOL(I),ELEV(I),JELE,B
   39 FORMAT(10X,6F10.2)
   77 CONTINUE
   78 CONTINUE
   80 CONTINUE
      DS = VOL(N)-VOL(N-1)
      DQ = DIS(N)-DIS(N-1)
      DO 85 IA=N,20
      VOL(IA+1) = VOL(IA)+DS
      DIS(IA+1) = DIS(IA)+DQ
      IF(DIS(IA+1).GT.999999.0) DIS(IA+1)=999999.0
      IF(VOL(IA+1).GT.9.999E+7) VOL(IA+1)=9.999E+7
   85 CONTINUE
   90 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE FREQ
      INTEGER TYPE
      REAL IN,MAXFLO
      COMMON NSP(99), SF(5,120), PREC(120),IN(30,120),FLOW(100,120),
     1  FACT, CUMA(100), MAXFLO(100), INFO(20), IBUF(1120), LDEV, TIMX,
     1  ISPEC, IT, NUM, TYPE(100), IFACT, IFPT(100), IAN(100),
     1  OF(100,30,2), PC, NE(100), NSC(100),IO(20)
      INTEGER ICHAR(12)
      DATA ICHAR/'JAN.','FEB.','MAR.','APR.','MAY ','JUNE','JULY',
     1             'AUG.','SEPT','OCT.','NOV.','DEC.'/
      DO 10 I = 1,100
      IFPT(I)=0
      NE(I) = 0
      IAN(I)=0
      DO 10 J = 1,100
   10 FLOW(I,J) = 0.0
      MSQ = 0
      NOLD = 0
      ICOUNT = 0
      IF(IO(17).EQ.1) WRITE(3,333) (INFO(I),I=2,19)
  333 FORMAT(1H1,///5X,'PEAK FLOWS FOR ALL STORMS FOR ',12A4/
     1  5X,100('-'))
   20 READ(9,996,ERR=200,END=200)NSEQ,NYR,NMO,NDAY,PEAK
  996 FORMAT(4I5,F12.2)
      IF(IO(17).EQ.1)WRITE(3,7)NSEQ,NYR,NMO,NDAY,PEAK
    7 FORMAT(10X,'SEQ= ',I4,' DATE=',3I5,' PEAK= ',F10.3)
      NYZ = NYR
      Z = 0.0005*CUMA(NSEQ)/PC
      IF(FLOW(NSEQ,NYZ).LT.Z) FLOW(NSEQ,NYZ) = Z
      IF(NMO.GE.10) NYZ = NYR + 1
      IF(NSEQ.GT.100) GO TO 33
      IF(NYZ.LE.0.OR.NYZ.GE.100) GO TO 203
      IF(NSEQ.LE.0.OR.NSEQ.GT.100) GO TO 203
      IF(NYZ  .EQ. 0) NYZ = 100
      IF(NSEQ.LT.NOLD) NOLD = 999
      IF(NOLD.GT.100) GO TO 17
      ICOUNT = ICOUNT + 1
      IF(ICOUNT.GT.40) ICOUNT = 40
      NE(NSEQ) = ICOUNT
      NOLD = NSEQ
   17 CONTINUE
      IF(PEAK .LT. FLOW(NSEQ,NYZ)) GO TO 27
      FLOW(NSEQ,NYZ) = PEAK
      J = (NE(NSEQ) - 1 )*3 + 1
      IN(NYZ-16,J) = NYR + 0.001
      IN(NYZ-16,J+1) = NMO + 0.001
      IN(NYZ-16,J+2) = NDAY + 0.001
   27 CONTINUE
      IFPT(NSEQ) = 1
      IF(NSEQ.GT.MSQ) MSQ = NSEQ
      GO TO 20
   33 CONTINUE
      DO 30 I = 1,100
      DO 30 J = 1,100
      IF(FLOW(I,J) .LE. 0.0) FLOW(I,J) = -1000.
```

```
      IF(FLOW(I,J) .GT. 0.0) FLOW(I,J) = ALOG(FLOW(1,J))
   30 CONTINUE
      IF(IO(16).EQ.1) WRITE(3,444) (INFO(I),I=2,19)
  444 FORMAT(1H1//10X,'ELEVATION(STAGE) - DISCHARGE RELATION FOR'
     1 ,1X,12A4/10X,100('-')/)
      DO 5 I=1,MSQ
      IF(IFPT(I).NE.1) GO TO 5
C READ DISCHARGE
      READ(1,2) J,K,(QF(I,NQ,1),NQ=1,K)
      KOS = K
      IF(IO(16).EQ.1) WRITE(3,8)J,(QF(I,NQ,1),NQ=1,K)
    8 FORMAT(//2X,'SEGMENT NO. ',I4,3X,'Q(CFS) = ',10F10.3/
     1  30X,10F10.3/30X,10F10.3)
C READ STAGE
      READ(1,2) J,K,(QF(I,NQ,2),NQ=1,K)
      IF(KOS.NE.K) WRITE(3,890) K,KOS,I
  390 FORMAT(/10X,'*** POTENTIAL PROBLEM - NUMBER OF STAGE VALUES'
     1  '(',I3,') AND DISCHARG  VALUES(',I3,')'/10X,
     2   'ARE NOT THE SAME FOR SEGMENT NUMBER',I4//)
      IF(IO(16).EQ.1) WRITE(3,9)J,(QF(I,NQ,2),NQ=1,K)
    9 FORMAT(/2X,'SEGMENT NO. ',I4,3X,'ELEV(FT)=',10F10.3/
     1    30X,10F10.3/30X,10F10.3)
      IF(J.NE.I) WRITE(3,899) I,J
  899 FORMAT(//10X,'*** POTENTIAL PROBLEM - PROGRAM WANTS RATING TABLE',
     1 ' FOR SEGMENT ',I3,' AND DATA CARD SAYS',I3)
      KOS1 = KOS+1
      DO 15 K = KOS1,30
      QF(I,K,1) = 0.0
   15 QF(I,K,2) = 0.0
    5 CONTINUE
    2 FORMAT(2I4/(10F8.0))
      DO 100 NSQ = 1,MSQ
      IF(IFPT(NSQ).NE.1) GO TO 100
      SPK = 0.0
      ZSPK = 0.0
      SSQ = 0.0
      ZSSQ = 0.0
      KYR = 0
      WRITE(3,555)NSQ,(INFO(I),I=2,19)
  555 FORMAT(1H1/5X,'ANNUAL PEAK FLOWS FOR SEGMENT NO.',I3,' FOR ',
     1  12A4/ 5X,100('-')
     1   //10X,'WATER YEAR',5X,'PEAK FLOW(CFS)'
     1 ,5X,'YEAR MONTH DAY'    /10X,'----------',5X,
     2  '--------------',5X,'---- ----- ---'/)
      DO 40  NYR = 1,100
      IF (FLOW(NSQ,NYR) .LT. -999.) GO TO 40
      KYR = KYR + 1
      SPK = SPK + FLOW(NSQ, NYR)
      XX=EXP(FLOW(NSQ,NYR))
      ZSPK = ZSPK + XX
      J = (NF(NSQ)-1)*3 + 1
      I1 = IN(NYR-16,J)
      I2 = IN(NYR-16,J+1)
      I3 = IN(NYR-16,J+2)
      WRITE(3,666) NYR,XX,I1,ICHAR(I2),I3
  666 FORMAT(15X,'19',I2,9X,F8.1,8X,'19',I2,2X,A4,I4)
   40 CONTINUE
```

```
      IF (KYR .EQ. 0) GO TO 100
      FYR = KYR
      APK = SPK/FYR
      ZAPK = ZSPK/FYR
      DO 50 NYR = 1,100
      IF(FLOW(NSQ,NYR).LT.-999.) GO TO 50
      SSQ=SSQ+(FLOW(NSQ,NYR) -APK)**2
      ZSSQ=ZSSQ+(EXP(FLOW(NSQ,NYR)) -ZAPK)**2
   50 CONTINUE
      SDPK = (SSQ/(FYR - 1.0))**0.5
      ZDPK = (ZSSQ/(FYR-1.0))**0.5
      WRITE(3,808)ZAPK,ZDPK
  808 FORMAT(29X,'--------'//22X,'MEAN =',F8.1,8X,'STANDARD DEVIATION =',
     1 F8.1)
      QL5   = APK + 0.842*SDPK
      QL10  = APK + 1.282*SDPK
      QL25  = APK + 1.751*SDPK
      QL50  = APK + 2.054*SDPK
      QL100= APK + 2.326*SDPK
      QL200= APK + 2.576*SDPK
      Q2    = EXP(APK)
      Q5    = EXP(QL5)
      Q10   = EXP(QL10)
      Q25   = EXP(QL25)
      Q50   = EXP(QL50)
      Q100  = EXP(QL100)
      Q200  = EXP(QL200)
      L = 0
      Q = Q2
   60 L = L + 1
      NQ = 1
   62 NQ = NQ + 1
      IF(OF(NSQ,NQ,1).GT.1.0E-5) GO TO 61
   65 H = 0.0
      GO TO 64
   61 IF(NQ.GE.30) GO TO 65
      IF(OF(NSQ,NQ,1).LT.Q) GO TO 62
   63 LQ = NQ - 1
      C = (OF(NSQ,NQ,2) -OF(NSQ,LQ, 2))/((OF(NSQ,NQ,1)-OF(NSQ, LQ, 1))
     1   **0.9)
      H = OF(NSQ, LQ, 2) + C*((Q-OF(NSQ, LQ,1))**0.9)
   64 IF(L .EQ. 1) H2    = H
      IF(L .EQ. 2) H5    = H
      IF(L .EQ. 3) H10   = H
      IF(L .EQ. 4) H25   = H
      IF(L .EQ. 5) H50   = H
      IF(L .EQ. 6) H100  = H
      IF(L .EQ. 7) H200  = H
      IF(L .EQ. 1) Q = Q5
      IF(L .EQ. 2) Q = Q10
      IF(L .EQ. 3) Q = Q25
      IF(L .EQ. 4) Q = Q50
      IF(L .EQ. 5) Q = Q100
      IF(L .EQ. 6) Q = Q200
      IF(L .EQ. 7) GO TO 70
      GO TO 60
   70 WRITE(3,3) NSQ,(INFO(K), K=2,19)
```

```
  3 FORMAT(//5X,'FLOOD FREQUENCY FOR FLOWPOINT',I3,' FOR ',
   1 12A4/5X,100('-')/37X,'*'/10X, 'RETURN '
   1 'PERIOD',3X,'PROBABILITY',5X,'FLOW IN CFS',5X,'W S ELEV IN FT'/
   1 10X,'-------------',3X,'-----------',5X,'-----------',
   1 5X,'--------------'/)
     WRITE(3,4) Q2, H2, Q5, H5, Q10, H10, Q25, H25, Q50, H50, Q100,
   1  H100, Q200, H200
  4 FORMAT(15X,'2-YEAR',10X,'50.0',10X,F7.1,10X,F7.1/
   1          15X,'5-YEAR',10X,'20.0',10X,F7.1,10X,F7.1/
   1         14X,'10-YEAR',10X,'10.0',10X,F7.1,10X,F7.1/
   1         14X,'25-YEAR',10X,' 4.0',10X,F7.1,10X,F7.1/
   1         14X,'50-YEAR',10X,' 2.0',10X,F7.1,10X,F7.1/
   1        13X,'100-YEAR',10X,' 1.0',10X,F7.1,10X,F7.1/
   1        13X,'200-YEAR',10X,' 0.5',10X,F7.1,10X,F7.1)
     WRITE(3,777)
777 FORMAT(///10X,'*'/11X,'PERCENT CHANCE IN ANY YEAR OF GETTING',
   1 ' A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.',
   1  /20X,'---',25X,'-------')
100 CONTINUE
     GO TO 202
200 WRITE(3,201) NSEQ,NYR
201 FORMAT(10X,'ERROR REACHED END OF FILE',2I5)
     CALL EXIT
203 WRITE(3,204)NYZ,NSEQ
204 FORMAT(10X,'YEAR OR SEQ. NO. FROM FILE IMPROPER',
   1    2I5)
     CALL EXIT
202 RETURN
     END
```

REFERENCES

Bodhaine, G. L. (1968), "Measurement of Peak Discharge at Culverts by Indirect Methods", Techniques of Water-Resources Investigations of the U.S. Geological Survey, Washington, D.C.

Crawford, Norman H. and Ray K. Linsley (1966), "Digital Simulation in Hydrology: the Stanford Watershed Model IV", Tech. Report 39, Dept. of Civil Engineering, Stanford University.

Harley, Brendon F., Frank E. Perkins and Peter S. Eagleson (1970), "A Modular Distributed Model of Catchment Dynamics", Report No. 133, Dept. of Civil Engineering, Massachusetts Institute of Technology, 1970.

HEC-1 Flood Hydrograph Package Users Manual (1973), U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, California.

Hydrocomp International, Inc. (1969 and 1972), "Hydrocomp Simulation Programming Operations Manual", Palo Alto, California.

Izzard, C. F. (1946), "Hydraulics of Runoff from Developed Surfaces", Proc. Highway Research Board, Twenty-Sixth Annual Meeting.

James, L. D. (1972), "Hydrologic Modeling, Parameter Estimation, and Watershed Characteristics", Journal of Hydrology, Vol. 17, pp. 283-307.

Kellerhals, Rulf (1970), "Runoff Routing Through Steep Natural Channels", J. of the Hydraulics Division, ASCE, Vol. 96, HY 11.

Lumb, Alan M. (1973), "Travel Time in Georgia Streams", Rept. No. ERC-1273, Environmental Resources Center, Georgia Institute of Technology, Atlanta, Georgia.

Lumb, Alan M. et. al. (1975), "GTWS: Georgia Tech Watershed Simulation Model", Report No. ERC-0175, Environmental Resources Center, Georgia Institute of Technology, Atlanta, Georgia.

Lumb, Alan M. and Timothy D. Hassett (1975), "Comparison of Georgia Tech, Kansas, Kentucky, Stanford and TVA Watershed Models in Georgia", Report No. ERC-0275, Environmental Resources Center, Georgia Institute of Technology, Atlanta, Georgia.

Rao, Ramachandra A., Jacques W. Delleur, Bhaskara Seshadri P. Sarma (1972), "Conceptual Hydrologic Models for Urbanizing Basins", J. of the Hydralics Division, ASCE, Vol. 98, No. HY7, pp. 1205-1220.

Rostogi, Ram A. and Benjamin A. Jones (1971), "Non-linear Response of a Small Drainage Basin Model", Journal of Hydrology , Vol. 14.

Sarma, P.B.S., J. W. Delleur, and A. R. Rao (1969), "A Program in Urban Hydrology, Part II. An Evaluation of Rainfall - Runoff Models for Small Urbanized Watersheds and the Effect of Urbanization on Runoff", Tech. Report No. 9, Purdue University Water Resources Center, Lafayette, Indiana.

Schaake, John C. (1965), "Synthesis of the Inlet Hydrograph," Tech. Report No. 3, Storm Drainage Research Project, The Johns Hopkins University, Baltimore.

White, O. C. (1973), "An Investigation of Downstream Effects of Limiting Peak Outflow from Subwatershed Development," M. S. Special Project, School of Civil Engineering, Georgia Institute of Technology, Atlanta.

Willeke, Gene E. (1966), "Time in Urban Hydrology", J. of the Hydraulics Division, ASCE, Vol. 92, No. HY1, pp. 13-30.

UROS4: URBAN FLOOD SIMULATION MODEL

Part 2. Applications to Selected DeKalb County Watersheds

for
Board of Commissioners
DeKalb County, Georgia

by

L. Douglas James and Alan M. Lumb
School of Civil Engineering

May, 1975

School of Civil Engineering
GEORGIA INSTITUTE OF TECHNOLOGY
Atlanta, Georgia

UROS4: URBAN FLOOD SIMULATION MODEL

Part 2. Applications to Selected DeKalb County Watersheds

for

Board of Commissioners

DeKalb County, Georgia

by

L. Douglas James and Alan M. Lumb

School of Civil Engineering

Georgia Institute of Technology

Atlanta, Georgia

May, 1975

ACKNOWLEDGMENTS

This report is one of a series of three reports presenting the work at Georgia Institute of Technology for DeKalb County, Georgia on a project entitled "Utilization of a Computer Model to Determine the Impact of Urban Development of Flooding in DeKalb County." Work on the project began December, 1973, and was completed March, 1975. Alan M. Lumb, with the assistance of J. R. Wallace and L. D. James, directed the project. Dr. James assisted with the development and application of UROS4, while Dr. Wallace directed most of the work in gage site selection and installation. Paul Sanders and W. M. Sangster provided invaluable administrative assistance.

Eleven students worked on the project, and three made major contributions. John Clerici worked on the project for the entire period and took major responsibility in the collection of the field data. Jack Kittle worked on the programming of UROS4, the creation of the runoff files and program applications. D. Rao assisted with the statistical analysis and was very productive in program applications. Three graduate students supported by the Corps of Engineers, Carvel Deese, Brian Tarras, and Roy Powell, worked on the project as part of their studies and their efforts are gratefully appreciated. Although only working with the project one quarter, Tom Debo developed the procedures for and performed most of the work on the determination of the fraction of impervious area. The services of six other research assistants, Ed Sing, Paul Nowak, Adnan Saad, Tim Hassett, Jaime Nino-Pinto and Steve Todd, are gratefully acknowledged.

ABSTRACT

Urban development has occurred so rapidly in the Atlanta Metropolitan Area that the citizens and their governments have not been able to deal adequately with the associated flood and drainage problems. As the idealistic approach of locating everyone and everything on higher ground is costly if not impossible, the welfare of DeKalb County can best be served by a combination of 1) tributary area land use planning, 2) flood plain management including land use planning and regulation of flood plain building practices, and 3) structural measures involving detention storage and drainage system improvements. Selection of a successful combination requires information on how land surfaces and drainageways respond to a variety of precipitation patterns. Since watershed configurations and precipitation patterns are so complex and varied, hydrologic simulation is the only method powerful enough to determine fully the effects of land use and channel changes on flood elevations.

In order to provide a working simulation model for use by DeKalb County, the Urban Flood Simulation Model was developed. Rainfall, streamflow, and soils data were analyzed with the Stanford Watershed Model to develop and historic data file of rainfall excess for the range of land surface conditions found in Dekalb County. The Urban Flood Simulation Model simulates floods given the data file and prescribed physical characteristics of as many as 100 area, channel, and storage segments in a selected drainage area (Snapfinger Creek for example). The Model will calculate flood elevations and associated probabilities for critical points specified in the input data. Though collecting, coding, and checking the data on the physical characteristics may take a man-month or more, depending on watershed size and resolution, once the coding is complete it is relatively easy to explore the effects of changing land-use, altering the drainage system, or adding detention storage.

The procedures used in developing the file of runoff data, the computational framework, the computer programming, and the recommended procedures for collecting and coding data on drainage characteristics are all described in a companion report, "Part 1. Documentation and Users Manual." This report, Part 2, illustrates use of the Model in hydrologic studies. Eight DeKalb County watersheds were studied in varying degrees of detail, and preliminary assessments were made of the hydrologic aspects of the problems and potential solutions.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

LIST OF FIGURES

LIST OF FIGURES (cont'd)

LIST OF TABLES

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

LIST OF TABLES (cont'd)

EXECUTIVE SUMMARY

Three reports present the study conducted at Georgia Institute of Technology for DeKalb County on a project entitled "Utilization of a Computer Model to Determine the Impact of Urban Development on Flooding in DeKalb County". Titles of these reports are:

1) UROS4: URBAN FLOOD SIMULATION MODEL
Part 1. Documentation and Users Manual
Appendix 1. Hydrographs from Watershed Model Calibration

2) UROS4: URBAN FLOOD SIMULATION MODEL
Part 2. Application to Selected DeKalb County Watersheds

3) UROS4: URBAN FLOOD SIMULATION MODEL
Part 3. A Gaging System for Flood Measurements in DeKalb County

Drainage problems become readily apparent following large rain storms, and the media coverage provides vivid descriptions. Property gets damaged, transportation routes get blocked, and lives can be lost. The problem has been made worse because quantitative information on flood elevations and the effects of urban development on storm runoff have not been available for use by home-buyers, elected officials, land-use planners, and administrators of the drainage program. Thus, land-use planning and flood plain management were not effectively used to minimize the devastating effects of large storms. Today solutions are so costly that good hydrologic information is essential in choosing among them. Such solutions are combinations of: 1) tributary area land use planning, 2) flood plain management including both land use planning and regulation of flood plain building practices, and 3) structural measures involving detention storage and drainage system improvements. Selection of a successful combination requires information on how land surfaces and drainageways respond to a variety of precipitation patterns. Since combinations of land surface and drainageway characteristics with precipitation patterns are so complex and varied throughout the County, hydrologic simulation is the only method powerful enough to determine fully the effects of land use and drainage modifications on flood elevations. The contract between the County and Georgia Institute of Technology provided the necessary hydrologic simulation model and illustrated its use through analyses of selected watersheds in the County.

The analyses given in the Part 2 report are not final plans nor engineering designs, but rather:

1) Examples of the types of analyses that can be made with the hydrologic simulation model,
2) Preliminary screening, to eliminate those that would not be hydrologically effective, of potential alternatives to reduce the damages from flood waters.
3) Estimations of the effects of land development, changes to drainage structures, and channel improvements of downstream flooding.

Economic analysis is essential before selecting from among the alternatives but was not part of the contract. In large part, the studies were done by students with supervision by faculty members. The application of the simulation model by several students also provided information on needed refinements to assist future model users. Many of the refinements have been added to the model.

Headwater portions of eight watersheds were studied as part of the project: Warren, Wild, Honey, Womack, Susan, Snapfinger, Cobbs, and Nancy Creeks. Several general conclusions are made from these studies.

1) It is very difficult to find sites suitable for flood retardation dams and the sites that were found generally did not reduce downstream flood elevations substantially. This was especially true for drainage areas exceeding several square miles. On smaller watersheds, a potential exists for detention structures which may be economical.

2) Replacing pervious soil surfaces with such impervious surfaces as asphalt, concrete, and roofs has sizably increased flood flows, especially the more frequent floods. Peak flows were found in some cases to double and triple with a change from no urban development to a combination of commercial, industrial, multi-family and residential land use.

3) Channel improvements and culvert enlargements can usually be made to protect the adjacent property but worsen conditions downstream. Downstream flood peak increases depend on the length of channel improved and the total drainage area. Improvements on an entire channel network of a 1.5-square mile drainage area increased peak flows at the downstream point 20 to 30 percent. Channel improvements on larger streams appear to have a relatively larger adverse effect downstream. In a few cases, channel improvement may be justified as it may be on one segment of Wild Creek, which is a small tributary that flows to North Fork Peachtree Creek.

4) Regular removal of debris is essential to an effective drainage program, but the effects and the effectiveness of more extensive channel maintenance are generally difficult to predict. Erosion from land surfaces washed into the stream increases dramatically during construction periods. Runoff is increased by added impervious area. Both processes accelerate the erosion of channel banks and sediment deposition in the channel. Efforts to stabilize the channels have not proved particularly effective.

5) From the hydrologic studies, it appears that monies spent in channel improvements, culvert enlargements, and flood retardation dams would not generally be as beneficial as flood proofing and conversion of frequently inundated flood plain land to uses more compatible with the hazard. Procedures for implementing and financing appropriate actions remain to be resolved. Hydrologic analysis can be made very effectively, economic analysis of alternatives can follow, but if the best solution can not be implemented, benefits can not be realized.

SECTION I

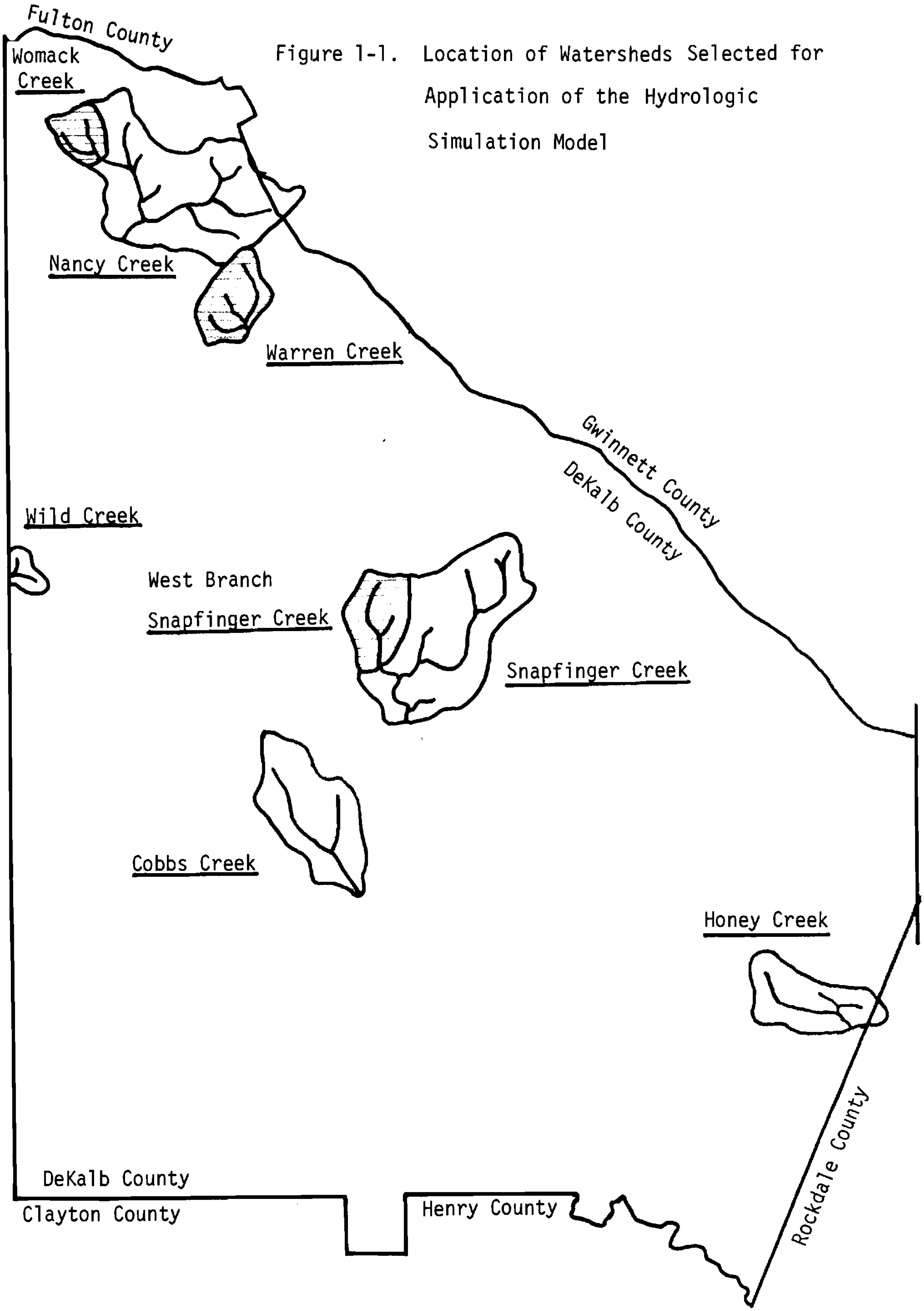
Introduction

As part of the study to provide DeKalb County with a hydrologic simulation model, eight watersheds were selected for application of the model (see Figure 1-1). The applications provide 1) examples of how the model can be used, 2) preliminary analysis of the drainage problems in the selected watersheds, 3) estimates of the effects of channelization and urban development on downstream flooding and 4) an opportunity to check and refine the simulation model. No engineering designs nor economic analyses were included in the scope of work.

In large part, the studies were done by students at Georgia Institute of Technology with supervision by faculty members. The field data was collected by one group of students and data from maps and aerial photographs by another. Each study watershed initially had assigned a single student and faculty member. All final work on the eight watersheds was done by three students, John Clerici, Jack Kittle and D. Rao, and two faculty members, L. D. James and A. M. Lumb.

The section in this report on each watershed has basically four parts. First is the description of the watershed, its physical attributes and location, and any previous studies. Second is a description of the drainage problems. Third is a description of the simulation runs, alternatives studied and the results. Last is the summary and conclusions.

Figure 1-1. Location of Watersheds Selected for Application of the Hydrologic Simulation Model



SECTION 2

Warren Creek

I. Physical Attributes of Watershed

a. Location

Warren Creek flows southward into the North Fork of Peachtree Creek just west of the I-85 interchange at I-285. Two branches of approximately equal size join 1000 feet upstream from where the Creek drains under I-85. The north branch drains a highly developed area upstream from I-285 and then flows between a wooded hillside and a row of houses along Sante Fe Drive. The west branch drains the largely residential areas around Sequoyah High School.

b. Size

The watershed has a total area of 1058 acres with about 39 percent impervious. The north branch drains 413 acres with 48 percent impervious, and the west branch drains 573 acres with 35 percent impervious. Eleven roads cross the Creek or its major tributaries. Table 2-1 shows the culvert size, capacities and tributary drainage areas at these crossings. Capacities were estimated for headwater elevations at the low point in the roadway.

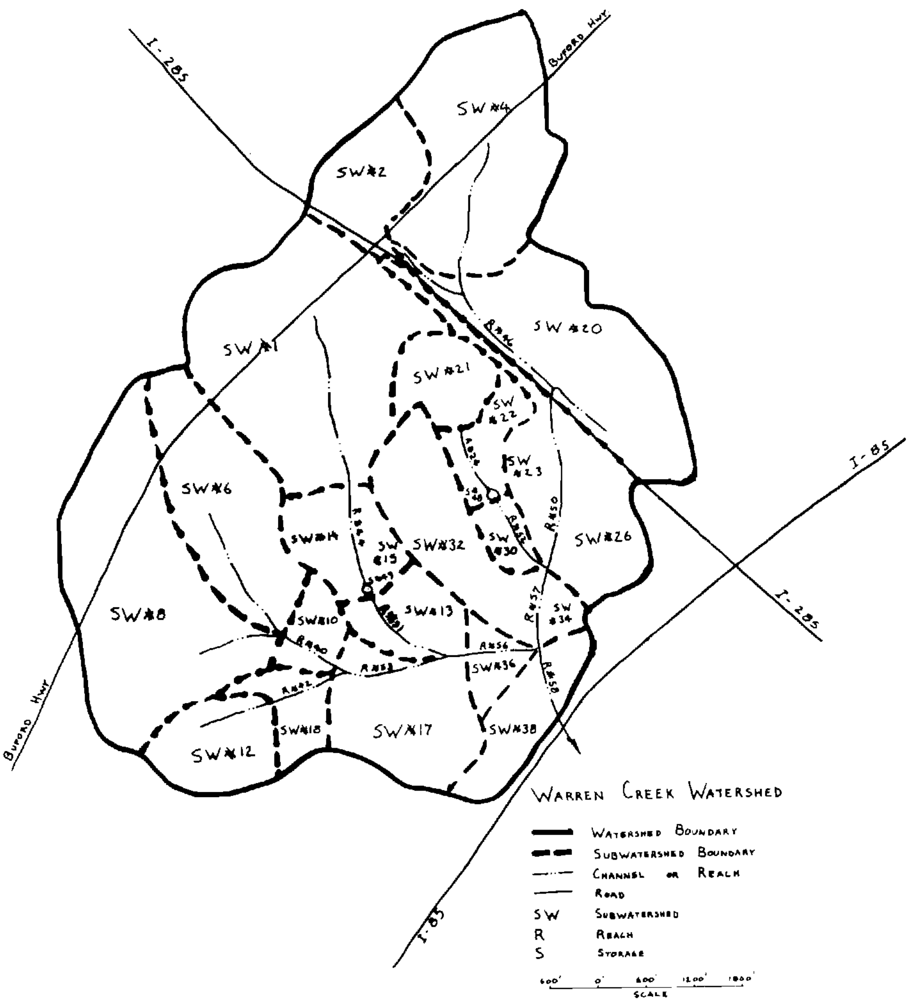
c. General Drainage Patterns

The subarea and channel configurations are shown on Figure 2-1. The north branch drains long, narrow basin that extends 5500 feet upstream from the junction with the west branch to North DeKalb Avenue. Intensive development (63% impervious) upstream from I-285 has been widely attributed as a cause of downstream flooding. The west branch extends 3200 feet upstream from the junction to Pineland Avenue and drains

Table 2-1

Drainage Areas at Culverts

Road	Drainage Area Acres	Culvert Description	Capacity cfs
Pineland	151	60"	377
North DeKalb	105	2-72"	944
McClave	75	42"	109
Brook Park	36	36"	145
Wheeler (from McClave)	234	2-72"	815
Wheeler (from Brook Park)	57	42"	122
Chesnut	291	2-72"	572
I-285	264	8'x7'	1214
Poplar	34	42"	100
Aztec	567	2-7'x6'	1366
I-85	1058	2-12'x8'	3250



I-285
Buford Hwy
SW #4
SW #2
SW #1
SW #20
SW #21
SW #22
SW #23
SW #6
SW #14
SW #15
SW #32
SW #30
SW #26
SW #8
SW #10
SW #13
SW #34
SW #36
SW #12
SW #18
SW #17
SW #38
I-85
I-285
Buford Hwy
Warren Creek Watershed
Watershed Boundary
Subwatershed Boundary
Channel or Reach
Road
SW Subwatershed
R Reach
S Storage
Scale
I-85

Figure 2-1

a round shaped basin containing several major tributaries. The floodplains are heavily wooded except localized areas where lawns or other landscaping has extended to the stream bank.

d. Other Studies

The U. S. Army Corps of Engineers completed a special Flood Hazard Information Report including the Peachtree Creek Basin in October, 1971, but the coverage did not extend to Warren Creek. A Study funded by the Office of Water Research and Technology and conducted through the Environmental Resources at Georgia Tech evaluated alternative measures for dealing with the flooding along the north branch of Warren Creek as a case study in "Synthesizing a Procedure for Formulating Urban Flood Control Programs" (Report ERC-0375).

e. Current Land Use

Most of the watershed has already been developed. Industrial and commercial property is concentrated near the interstates, and a number of apartment complexes are nearby. Single family homes on quarter to half acre lots and in generally good condition occupy the middle portion of the watershed. The only significant natural area left is around 70 acres of woodlands near the junction of the two branches and extending up the hillside on the left bank of the north branch to I-285 (areas 26, 32, and 34 on Figure 2-1).

f. Projected Land Use

Some of the less intensively developed areas are becoming more intensively developed. For example, older single-family homes along Buford Highway and Chamblee Road are being converted to commercial use, and this trend is expected to continue and increase the fraction of impervious area. The undeveloped wooded tract is presently zones R-3. Recent zoning hearings have considered petitions for rezoning to office and commercial uses, but the conflicts between the more intensive use and the adjacent single-family residential area is creating strong pressures within the community against rezoning.

II. Map of Watershed

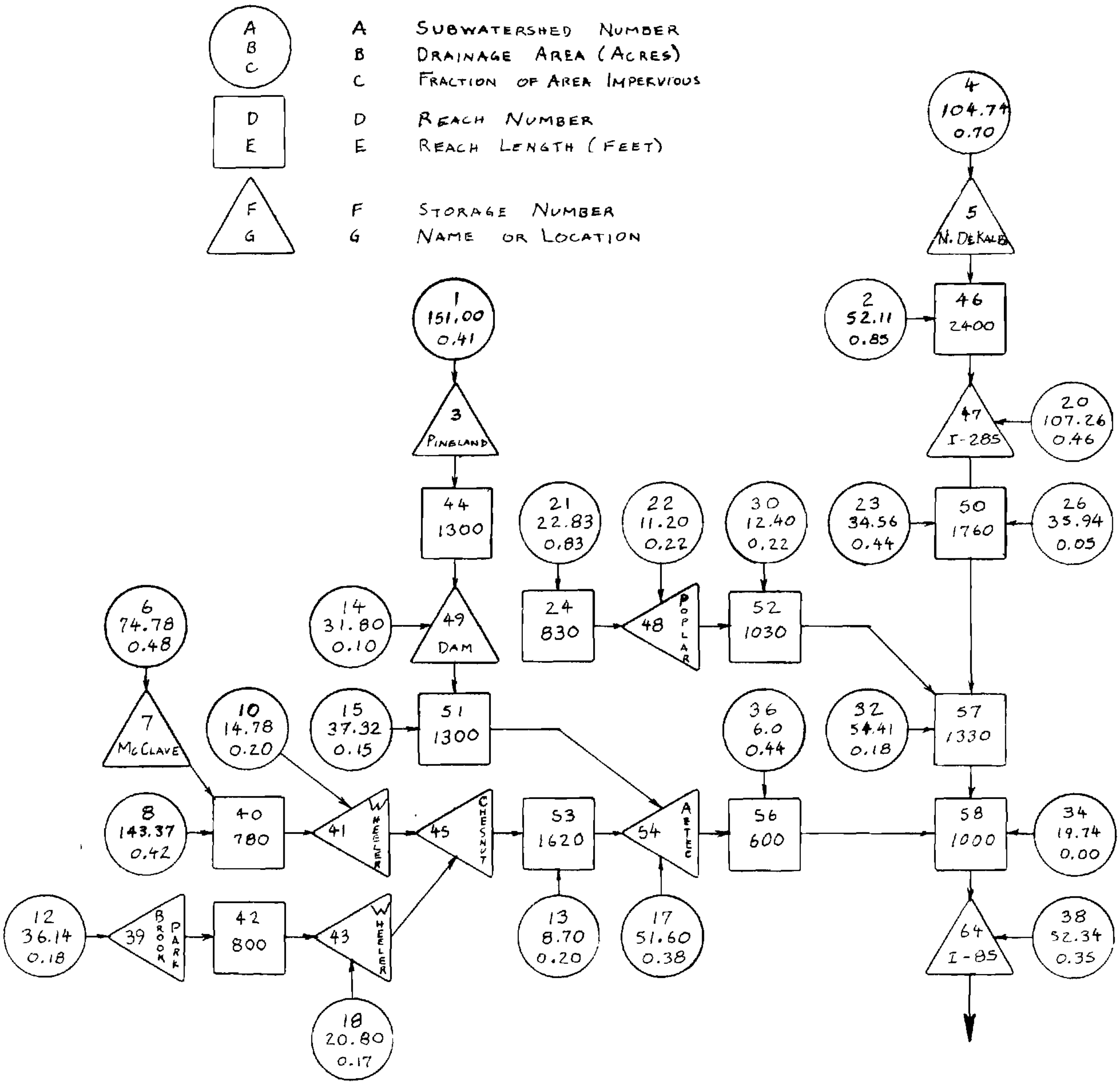
Figure 2-1 outlines the total watershed and the subareas into which it was divided. Major drainage ways are shown and divided into channel segments. As all 11 road crossings listed in Table 2-1 involve culverts small enough to have a significant backwater effect, a storage segment was added for each. A schematic diagram of the segments of all three types is shown on Figure 2-2.

III. Description of Drainage Problems

The analysis of the Warren Creek watershed investigated the general problem of quantifying the effects of drainageway and land-use changes on downstream flood peaks in DeKalb County as well as the specific flooding problems along Warren Creek.

The aspects of the general problem that were studied are

1. Changes to the land surface brought by urbanization increase runoff volumes and accelerate runoff rates. The resulting higher flood peaks aggravate downstream flooding. Because of the frequent complaints received from people living downstream from newly developed areas, information on the magnitude of the effect and how that magnitude varies with other watershed characteristics is important in resolving resulting controversies.



WARREN CREEK SCHEMATIC DIAGRAM

FIGURE 2-2

It is also significant for various land use planning decisions.

2. Changes to the drainageway brought by regular cleaning and maintenance or by channelization also accelerate runoff rates and increase flood peaks. Since this issue is also frequently raised by people living downstream from proposed channel changes, better information is needed on the magnitude of the effect and on how that magnitude varies with watershed characteristics.
3. Urban drainage facilities are frequently sized from a given design storm, but the fact is that the time pattern of precipitation that is critical in terms of producing the maximum flood peak from a given rainfall on one watershed may be quite different from that which is critical for another watershed. The 25-year runoff file of 99 historical storms was employed to estimate flood peaks at a number of points in the Warren Creek Basin and thereby establish how critical storm characteristics relate to watershed characteristics and the magnitude of the error in estimated flood peak caused by using a single design storm.

The specific Warren Creek problems studied are

4. Overtopping of various culverts within the watershed.
5. Flooding of seven houses facing Santa Fe Drive and backing onto Warren Creek in reach 50.
6. Drainage problems on some open land just upstream from Wheeler Drive at the Addison Drive intersection and at other locations.

7. The potential adverse hydrologic effect of development of the wooded area across the creek from the houses on Santa Fe Drive.
8. Potential aggravation of flooding and drainage problems in the watershed by additional urban development throughout the watershed.

IV. Descriptions of Simulation Runs of the General Problems

The general analysis to quantify the effects of land use and channel change on flood flows and the variation of critical storm characteristics with watershed characteristics was based on nine simulations. In order to reduce computer time, these simulations used the Warren Creek watershed without the storage effects of culverts. These simulations were also made with measurement errors in the lengths of channel segments 44 and 51 and with area segment 23 omitted. Although the errors were corrected for the runs on the specific flooding problems on Warren Creek, they were left in the runs for the general analysis which was not necessarily intended to represent a specific watershed.

The nine simulations were selected to provide information on the hydrologic effects of channelization with three levels of impervious area (0.0, existing, and 0.85), and on the hydrologic effects of impervious area with three levels of channelization (none, downstream half of the channels channelized, all channels channelized). The value of 0.85 for impervious area represents the most intensively developed existing subarea. Full channelization was depicted as rectangular channels large enough to contain all simulated flows. The specific combinations of impervious area and channelization for the various runs are listed in Table 2-2. Each

Table 2-2

Summary of Computer Runs Used in General Studies

Run	Impervious Area	Channel Section
1	Existing	Existing
2	0.00	Existing
3	0.85	Existing
4	Existing	Rectangular, n = 0.025
5	0.00	Rectangular, n = 0.025
6	0.85	Rectangular, n = 0.025
7	Existing	Upper half-existing, Lower half-rectangular
8	0.00	Upper half-existing, Lower half-rectangular
9	0.85	Upper half-existing, Lower half-rectangular

of the nine runs simulated and printed peak flows and elevations for the largest event each year and for several preselected frequencies at the downstream end of each channel segment.

1. Relationship of Flood Peaks to Impervious Area

Flood frequency analyses of the 25 annual flood peaks simulated at the end of each channel reach produced the mean annual and 200-year peaks shown in Table 2-3. The top half of the table is based on natural channels and shows simulated flows in both cfs and as ratios to flow with no impervious area. The bottom half is based on rectangular channels. Each half of the Table is divided into three parts. The top part provides simulated flood peaks with no impervious area, the middle part provides peaks at the 12 flow points for the existing impervious areas which range from 18 to 63 percent, and the bottom part provides peaks with all areas 85 percent impervious. The ratios to flow with no impervious area are by definition 1.00 for all 12 points under the conditions of no impervious areas and natural channels. The ratios with natural channels and existing impervious area vary with the impervious percentage. The ratios with natural channels and 85 percent impervious area vary but not in any consistent pattern with drainage area. The average ratio are 3.09 for the mean annual event and 2.41 for the 200-year event. The ratios for zero impervious area, the various existing impervious areas, and the average values for 85 percent impervious area are plotted on Figure 2-3. Least squares regression lines are plotted for the 200-year and mean annual events.

The average increase in the mean annual flood peak as impervious area increases from 0.00 to 0.85 is by a factor of 3.09, and the average increase in the 200-year flood peak is by a factor of 2.41, both figured with natural channels. With rectangular channels, the

Table 2-3. Summary of Simulated Effects of Impervious Area

Reach	24	40	42	44	46	50	51	52	53	56	57	58
Trib Acres	34	218	57	151	264	335	217	46	350	573	413	1058
With Natural Channels												
I	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Mean Q = Q_m	35	152	54	100	183	194	152	42	235	377	248	660
200 yr Q = Q_b	116	503	178	333	625	658	510	146	800	1283	842	2254
I	0.63	0.44	0.18	0.41	0.63	0.55	0.32	0.22	0.37	0.35	0.48	0.39
Mean Q	80	310	71	202	473	434	258	81	421	660	498	1210
Q/Q_m	2.29	2.04	1.31	2.02	2.58	2.24	1.70	1.93	1.79	1.75	2.01	1.83
200 yr Q	210	839	208	545	1254	1154	722	223	1194	1885	1359	3413
Q/Q_b	1.81	1.67	1.17	1.64	2.01	1.75	1.42	1.53	1.49	1.47	1.61	1.51
I	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
Mean Q	101	509	160	353	612	588	484	117	723	1160	709	1963
Q/Q_m (ave. 3.09)	2.89	3.35	2.96	3.53	3.34	3.03	3.18	2.79	3.08	3.08	2.86	2.97
200 yr Q	269	1318	418	909	1626	1534	1247	314	1921	3102	1871	5224
Q/Q_b (ave. 2.41)	2.32	2.62	2.34	2.73	2.60	2.33	2.45	2.15	2.40	2.42	2.22	2.32
With Rectangular Channels												
I	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Mean Q	38	161	58	104	192	216	166	49	274	438	295	778
Q/Q_m	1.09	1.06	1.07	1.04	1.05	1.11	1.09	1.17	1.17	1.16	1.19	1.18
200 yr Q	122	527	187	339	656	746	551	164	927	1486	1030	2697
Q/Q_b	1.05	1.05	1.05	1.03	1.05	1.13	1.08	1.12	1.16	1.16	1.22	1.20
I	0.63	0.44	0.18	0.41	0.63	0.55	0.32	0.22	0.37	0.35	0.48	0.39
Mean Q	91	330	76	208	502	507	282	97	496	778	653	1484
Q/Q_m	2.39	2.05	1.31	2.00	2.61	2.35	1.70	1.98	1.81	1.78	2.21	1.91
200 yr Q	256	876	220	560	1318	1370	789	260	1382	2189	1770	4190
Q/Q_b	2.10	1.66	1.18	1.65	2.01	1.84	1.43	1.59	1.49	1.47	1.72	1.55
I	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
Mean Q	125	599	216	419	655	719	586	148	923	1474	956	2537
Q/Q_m	3.29	3.72	3.72	4.03	3.41	3.33	3.53	3.02	3.37	3.37	3.25	3.26
200 yr Q	412	1811	837	1410	1726	1929	1649	411	2533	3993	2586	6820
Q/Q_b	3.38	3.44	4.48	4.19	2.63	2.59	2.99	2.51	2.73	2.69	2.51	2.53

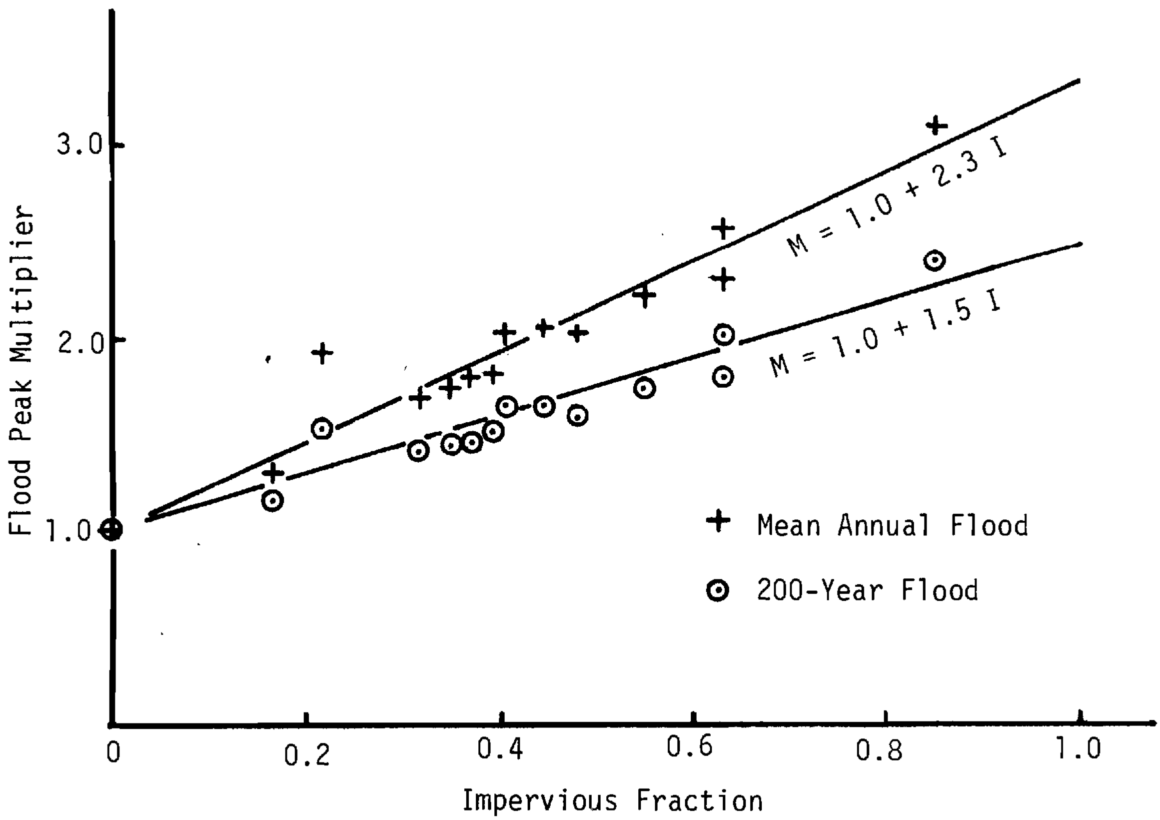


Figure 2-3. Ratios of Flood Peaks to Values With Zero Impervious Area for Simulation Based on Existing Warren Creek Channels

multipliers are 3.42 and 3.02 respectively. The effects of added impervious area are thus shown to be greater for smaller floods than for larger ones and greater with channelized drainageways than with natural ones. The effect seems to be relatively independent of drainage area for areas under 1100 acres. The regression lines on Figure 2-3 should provide a reasonable estimate of the magnitude of the increase in flood peak with increase in impervious area for DeKalb County basins in this size range and with largely natural channels. One would expect the effect of impervious area to be smaller in larger basins since channel storage increases geometrically with basin size, but this study did not address that point. One would also expect impervious area to have relatively greater effect for watersheds with more permeable soils and a relatively smaller effect for watersheds with less permeable soils.

2. Relationship of Flood Peaks to Channelization

The simulated flood peaks on Table 2-3 can also be used to quantify the effects of replacing small natural channels having frequently inundated flood plains with rectangular channel completely containing all flows. These results plus results from simulation with channelization on only the downstream half of the channel network are shown in Table 2-4. The effect of channelization is seen to be about the same for both the mean annual and the 200-year events and somewhat larger for a developed than for a natural watershed. Curves for estimating the effects of channelization are shown in Figure 2-4.

The trend of the simulated flows at the outlet of each subwatershed indicates that the effect of channelization increases with drainage area as can be seen from the numbers tabulated in Table 2-5 for natural watersheds. The regression equations for the two flood frequencies are given at the bottom of Table 2-5, and the 200-year values are plotted on Figure 2-5. The channelization effect for watersheds under 1100 acres is seen to be much less than the urbanization effect. However, the effect does increase with drainage area. While one should not extrapolate magnitudes from the Warren results, it is logical to expect the effect of channelization to be relatively larger with respect to that of impervious area for still larger watersheds. For larger basins, a greater fraction of the total time to the mouth is spent in channel flow and reducing channel travel time and associated channel storage can be expected to have a relatively greater effect.

Table 2-4

Summary of Simulated Effects of Channelization

	Existing Channels	Half Rectangular	All Rectangular
For 1058-acre natural watershed, I = 0.00			
Mean Q - flow	660	735	778
ratio	1.00	1.11	1.18
200-yr Q-flow	2254	2531	2697
ratio	1.00	1.12	1.20
For 1058-acre developed watershed, I = 0.85			
Mean Q - flow	1963	2298	2537
ratio	1.00	1.17	1.29
200-yr Q-flow	5224	6058	6820
ratio	1.00	1.16	1.31

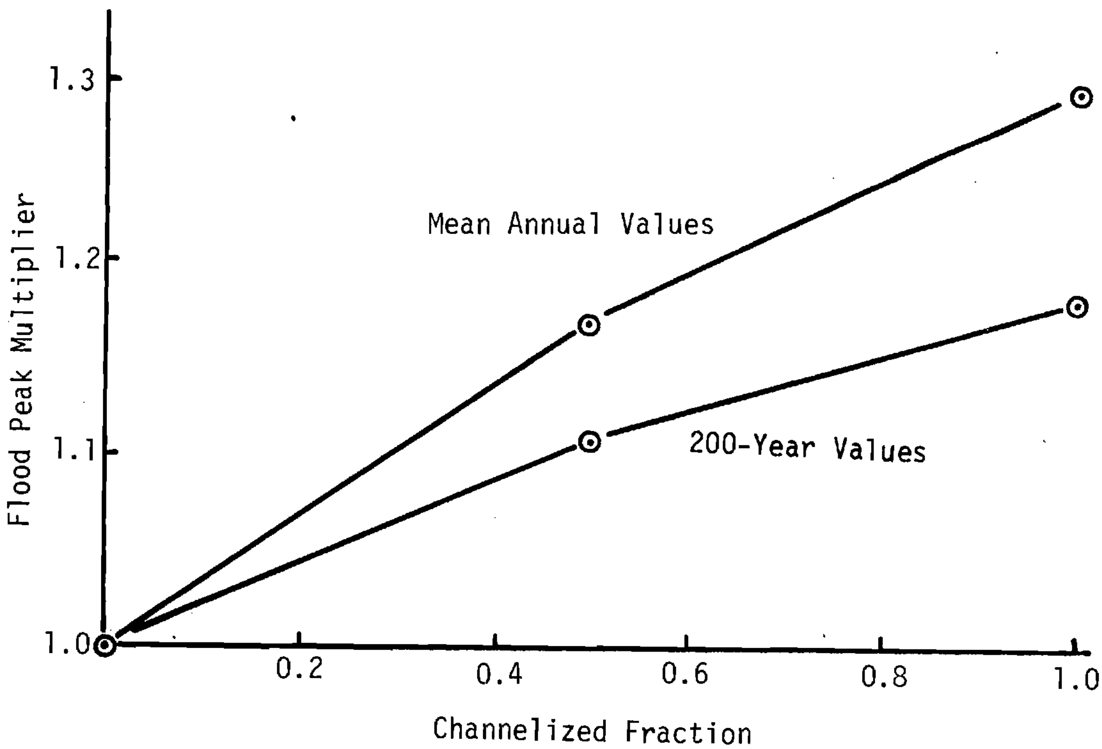


Figure 2-4. Ratios of Flood Peaks to Values with Natural Channels for Simulations with 1058-Acre Warren Watershed

Table 2-5

Summary of Relationship of Simulated Effects of

Channelization for Various Drainage Areas

Area acres A	Natural Watershed * Mean Q M	200-yr Q B
34	1.09	1.05
46	1.17	1.12
57	1.07	1.05
151	1.04	1.02
217	1.09	1.08
218	1.06	1.05
264	1.05	1.05
335	1.11	1.13
350	1.17	1.16
413	1.19	1.22
573	1.16	1.16
1058	1.18	1.20

* All values are ratios of peaks with rectangular channels to those with natural channels.

Regression results

Mean Q: $M = 1.082 + 0.000108\ A$ $\quad r = 0.55$

200-yr Q: $B = 1.056 + 0.000166\ A$ $\quad r = 0.71$

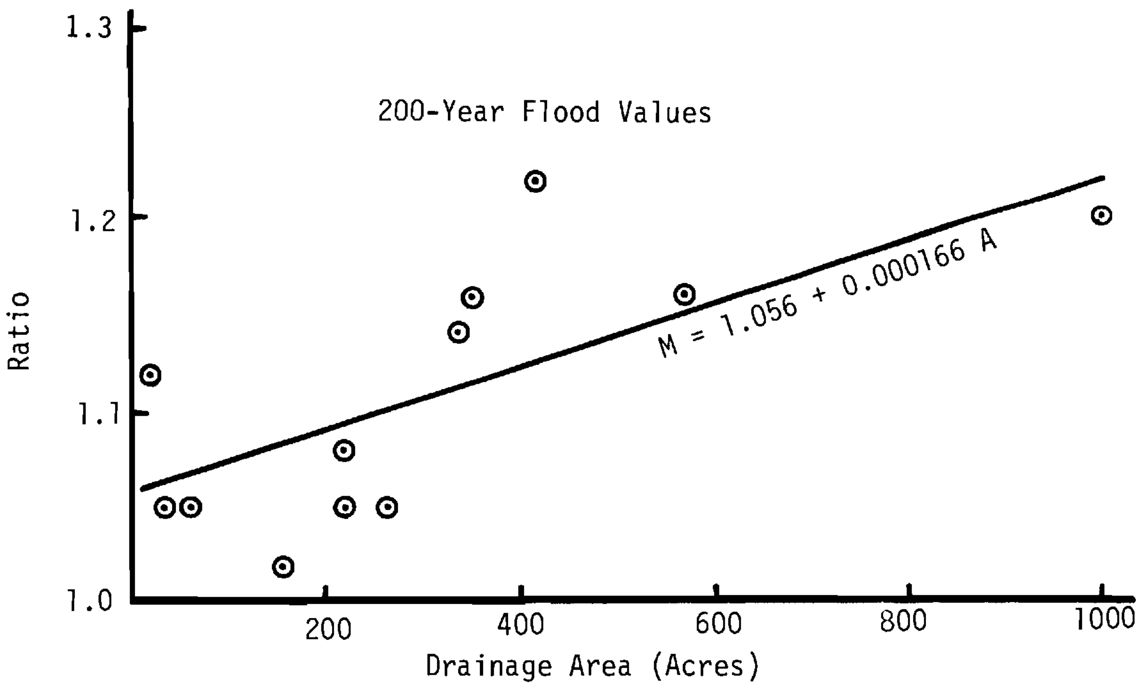


Figure 2-5. Ratio of Channelization Effect to Drainage Area

3. Variation in Critical Storm with Watershed and Channel Conditions

A storm that generates a rare flood for one combination of basin size, impervious area, and drainageway channelization may generate a relatively ordinary flood for some other combination. A larger basin size is associated with a shift in the critical storm from a short and very intense event to a prolonged period of rainfall. Greater impervious area reduces dependence on antecedent precipitation. Greater drainageway channelization increases sensitivity to intense events.

Both the spatial and the time distribution of the precipitation are important. For these simulations, a uniform spatial distribution was used in every case, but the time distributions varied over all the patterns found in the 99 storms occuring over the 25 year simulation period. Six different storms were found to be generating maximum floods for the combination of channelization, impervious fraction, and drainage area studied. The dates and selected characteristics of these six storms are shown in Table 2-6. The characteristics of the watersheds for which each storm was found to be critical are shown in Table 2-7. The storm of June 10, 1933, had the most intense 5-minute rainfall in the 25-year period; but the 60-minute rainfall was relatively low, and the soil was quite dry. Consequently, this storm only proved to be critical for small, impervious basins. The storm of August 15, 1920, was the second most intense but fell after much more antecedent moisture, and the intense rain falling on a wet watershed proved critical for small natural basins. The storm on June 25, 1938, had one of the most intense rainfalls in the 10 to 20 minute duration range and fell on a relatively dry watershed, and this combination proved critical for impervious basins slightly larger than those most responsive to the 1933 storm. The storm of August 24, 1921, had lower intensities than did the 1938 storm but fell on a wetter watershed and thus produced a greater

Table 2-6

Characteristics of Critical Storms

Date	8-15-1920	8-24-1921	8-11-1926	7-10-1928	6-10-1933	6-25-1938
Cumulative Rainfalls						
5 minute[a]	0.63	0.48	0.56	0.48	0.82	0.56
15 minute[a]	1.15	1.35	1.49	1.04	1.27	1.40
60 minute[a]	1.86	2.24	2.93	2.19	1.64	2.42
1 day[b]	2.07	3.21	2.97	2.59	2.52	2.48
4 day[b]	3.72	3.21	3.45	5.84	2.73	2.48
8 day[b]	7.33	3.25	3.85	6.25	2.73	2.73
Cumulative Runoffs						
5 minute[a]	0.25	0.30	0.25	0.32	0.15	0.27
15 minute[a]	0.65	0.66	0.63	0.78	0.29	0.54
60 minute[a]	1.12	0.88	1.13	1.66	0.31	0.64

[a]Maximum values summed from 5-minute files

[b]Over period preceding and including maximum values

Table 2-7

Watershed Characteristics for Which Various Storms Were Critical

Date	Channelization Fraction	Impervious Fraction	Area Acres
August 15, 1920	0.00	0.63	34
August 24, 1921	0.00	0.18 to 0.63	24 to 264
	1.00	0.18 to 0.55	24 to 1058
August 11, 1926	0.00	0.39 to 0.55	224 to 1058
	0.00	0.85	413
July 10, 1928	0.00 & 1.00	0.00	24 to 1058
	0.00	0.32 to 0.37	217 to 573
June 10, 1933	0.00	0.85	24 to 151
	1.00	0.85	24 to 1058
June 25, 1938	0.00	0.85	215 to 1058
	1.00	0.85	334 to 413
	0.00	0.63	34
	1.00	0.63	264

peak from watersheds with less impervious area. The storm of August 11, 1926, had higher intensities but a smaller daily rainfall total than did the 1921 storm and fell on a wetter watershed than did the 1938 storm, and this combination proved critical for an intermediate range of having more impervious area than those most sensitive to the 1921 storm but having less impervious area than those most sensitive to the 1938 storm. The storm of July 10, 1928, was less intense than others in the list but fell at the end of the wettest four-day period in the record and proved critical for all natural watersheds (I=0.00). All six of these events were summer thunderstorms; one would expect that for basins much larger than the 1058 acres used for this study that winter or spring cyclonic storms would begin to become critical because of their more uniform intensity over larger areas, their generally longer duration, and the almost completely saturated soil conditions that develop at the end of the long winter period of low evapotranspiration.

The August 11, 1926, storm is the one producing the largest runoff at the downstream end of the Warren Watershed under current land use and channel conditions. A common approach to hydrologic analysis would be to estimate flood peaks for all situations within a watershed from the storm determined to be critical. Table 2-8 shows the magnitude of the underestimate of peak flows one could expect for various conditions in the Warren watershed by following this practice. In many cases the error is quite significant. One of the major advantages of the approach of hydrologic simulation as used in this study is that it examines many historical combinations of storm rainfall patterns and antecedent moisture conditions to determine which combination is critical for a given watershed.

Table 2-8

Percentage Underestimates of 25-year Peak if Critical Storm Had Been Defined by Flood of Record for Existing Watershed

Channelization Fraction	Impervious Fraction	Area Acres	Percentage Underestimate
0.00	0.39	1058	0.0
0.00	0.55	355	0.0
0.00	0.35	573	2.6
0.00	0.37	350	2.3
0.00	0.63	264	11.0
0.00	0.44	218	6.9
0.00	0.32	217	5.5
0.00	0.41	151	4.5
0.00	0.48	57	10.9
0.00	0.63	34	11.6
0.00	0.22	24	16.8
0.00	0.00	1058	36.5
0.00	0.00	24	24.1
0.00	0.85	1058	2.3
0.00	0.85	24	12.6
1.00	0.39	1058	8.1
1.00	0.22	24	18.6
1.00	0.00	1058	29.9
1.00	0.00	24	16.5
1.00	0.85	1058	12.1
1.00	0.85	24	34.8

V. Description of Simulation Runs on Specific Warren Problems

The specific Warren Creek problems of periodic overtopping of various culverts, flooding of homes along Santa Fe Drive, ponded water on vacant lots near the intersection of Wheeler and Addison Drives, and potential adverse hydrologic effects from further development were analyzed through the sequence of computer runs listed in Table 2-9.

The overall objectives of these runs were to determine what effect storage behind existing culverts is now having on flood peaks, determine the hydrologic effects that would result from development of the hillside area above Santa Fe Drive, determine the hydrologic effects of channelization to protect the homes along Sante Fe Drive now subject to flooding, analyze possibilities for detention storage on the basin, and determine the degree to which one can expect the hazard to be aggravated by future intensification of urban development projected. Further development was based on information on current land use, number of vacant parcels, zoning, and land slope and flood hazard factors influencing probable development and resulted in the impervious areas shown in Table 2-10. The projected development is what one might reasonably expect over the next 10 to 20 years in response to reasonably likely development pressure.

Table 2-9

Summary of Computer Runs Used in Specific Warren Studies

Run	Description	Purpose
1	Existing land use and channels but no culverts	Establishment of a base for evaluating the effects of existing culverts on flood peaks
2	Existing land use, channels and culverts	Comparison with Run 1 to establish culvert effects and establishment of a base representing existing conditions for subsequent comparisons.
3	Existing channels and culverts but development (I=0.70) of wooded hillside areas behind houses on Santa Fe Drive	Comparison with Run 2 to establish the effects such development would have on downstream flood peaks.
4	Existing land use and culverts but channelization of Segment 50 to protect houses along Santa Fe Drive.	Comparison with Run 2 to establish the effects such channelization would have on downstream flood peaks.
5	Simulation with segments 12 and 39 only and alternative culvert sizes under Brook Park Road	Determination of a culvert size that would make maximum utilization of pondage behind the road embankment in reducing downstream floods based on the criterion of the road being overtopped by the 100-year flood.
6	Simulation with segments 1, 3, 14, 44, and 49 only and alternative outlet sizes for potential detention storage at a site behind Sequoyah High School.	Determination of an outlet size for a 100-year design with a ponded water surface elevation of 930.
7	Simulation with segments 2, 4, 5, 20, 46, and 47 only, and alternative inlet sizes to the culvert under I-285.	Determination of a size for an inlet to the existing culvert that would pond water to a 100-year design water surface elevation of 950 behind the freeway embankment.
8	Simulations for six storms (8-15-'20 8-24-'21, 8-11-'26, 7-10-'28, 6-10-'33, and 6-25-'38) and existing land use, channels, and culverts. a. Plus storage behind Brook Park Road (segment 39) b. Plus storage at Sequoyah site (segment 49) c. Plus storage behind I-285	Comparison with Run 2 to select hydrologically effective storage sites in terms of their effects on downstream flood peaks.

	(segment 47) d. Plus storage at all three sites	
9	Existing land use, channels, and culverts plus storage sites at segments 47 and 49.	Comparison with Run 2 to establish the effects such storage would have on downstream flood peaks.
10	Existing channels and culverts plus projected land use (see Table 10)	Comparison with Run 2 to establish how much reasonably expectable future development is likely to affect flood peaks.
11	Projected land use (see Table 10), existing channels, plus selected storage sites at segments 47 and 49.	Comparison with Run 9 to see how projected development will affect the storage sites and comparison with Run 10 to see how storage will affect projected flood peaks.

Table 2-10

Existing and Projected Impervious Fraction by Area Segment

Area Segment	Existing Impervious Fraction	Projected Impervious Fraction
1	0.41	0.60
2	0.85	0.85
4	0.70	0.70
6	0.48	0.55
8	0.42	0.50
10	0.20	0.30
12	0.18	0.40
13	0.20	0.30
14	0.10	0.30
15	0.15	0.30
17	0.38	0.50
18	0.17	0.50
20	0.46	0.60
21	0.83	0.83
22	0.22	0.30
26	0.05	0.70
30	0.22	0.30
32	0.18	0.40
34	0.00	0.70
36	0.44	0.44
38	0.35	0.60

4. Overtopping of Culverts

The initial simulation of 25 years of annual flood peaks (Run 1) was based on the existing watershed without culverts as represented by the data for area and channel segments in Table 2-11. The simulated annual peaks and the frequency analysis of the results for channel segments 53 (west branch at Aztec Avenue), 50 (north branch behind the houses on Santa Fe Drive), and 58 (at I-85) are shown in Tables 2-12 through 2-14.

The second simulation added a storage segment to account for each culvert (Table 2-1). Data for the storage segments are in Table 2-15. The simulated annual peaks and the results of the frequency analyses for the same three locations (53, 50, and 58) are shown in Tables 2-16 through 2-18.

Special care needs to be exercised in the frequency analysis of culvert affected flows. The 25 annual peaks at McClave Road (segment 7) are plotted in Figure 2-6. The flood peaks lie on two curves, one pertains for flows confined behind the road embankment, and the other pertains for flows that overtop the road (an event with a 3-year return period). The line of best fit that is calculated by the UROS4 Model, which assumes all the points to be from the same distribution, is also plotted and is obviously a very poor fit of the data. When applying the UROS4 Model and its frequency analysis at or shortly downstream from locations where flows overtop a road or other storage area, one should plot the annual flood peaks to make sure that the frequency analysis is reasonable. The break in the curve was found to occur at about the culvert capacity of 109 cfs.

The simulated flows at existing culverts are shown in Table 2-19, and estimates of the frequency at which each is overtopped are shown in the right column. This information should prove useful to culvert maintenance and enlargement programs.

Table 2-11 Specifications for source areas and channel segments for Runs 1 and 2

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE: RAPID	MODERATE	SLOW	IMPERVIOUS	STORAGE CONSTANT	LENGTH-FT	SLOPE	ROUGHNESS
1	151.000	.000	.590	.000	.410	16.	.0000	.0000	.0000
2	52.110	.000	.150	.000	.850	6.	.0000	.0000	.0000
4	104.740	.000	.300	.000	.700	10.	.0000	.0000	.0000
6	74.780	.000	.520	.000	.480	11.	.0000	.0000	.0000
8	143.370	.000	.580	.000	.420	16.	.0000	.0000	.0000
10	14.780	.000	.800	.000	.200	7.	.0000	.0000	.0000
12	36.140	.000	.820	.000	.180	11.	.0000	.0000	.0000
13	8.700	.000	.800	.000	.200	5.	.0000	.0000	.0000
14	31.800	.000	.900	.000	.100	11.	.0000	.0000	.0000
15	37.320	.000	.850	.000	.150	11.	.0000	.0000	.0000
17	51.600	.000	.620	.000	.380	10.	.0000	.0000	.0000
18	20.800	.000	.830	.000	.170	8.	.0000	.0000	.0000
20	107.260	.000	.540	.000	.460	13.	.0000	.0000	.0000
21	22.830	.000	.170	.000	.830	4.	.0000	.0000	.0000
22	11.200	.000	.780	.000	.220	6.	.0000	.0000	.0000
23	34.560	.000	.560	.000	.440	8.	.0000	.0000	.0000
26	35.940	.000	.950	.000	.050	13.	.0000	.0000	.0000
30	12.400	.000	.780	.000	.220	6.	.0000	.0000	.0000
32	54.410	.000	.820	.000	.180	13.	.0000	.0000	.0000
34	19.740	.000	1.000	.000	.000	11.	.0000	.0000	.0000
36	6.000	.000	.560	.000	.440	3.	.0000	.0000	.0000
38	52.340	.000	.650	.000	.350	10.	.0000	.0000	.0000

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS	CHANNEL: APARM	CHANNEL: MPARM	FLOOD PLAIN: APARM	FLOOD PLAIN: MPARM	AVERAGE TRAVEL TIME(SEC)
24	4	830.00	.03640	.100	.702	1.500	2.481	1.166	178.3
40	4	780.00	.01667	.080	.476	1.500	.998	1.295	269.3
42	4	800.00	.03500	.100	.784	1.500	1.390	1.306	234.1
44	4	1300.00	.01540	.080	.577	1.500	2.974	1.126	252.0
46	4	2400.00	.01208	.040	.755	1.500	8.212	1.067	202.5
50	4	1760.00	.01250	.070	.590	1.500	6.382	.930	369.9
51	4	1300.00	.00310	.080	.259	1.500	1.334	1.126	561.6
52	4	1030.00	.01942	.100	.513	1.500	1.812	1.166	302.9
53	4	1620.00	.00917	.070	.366	1.500	.913	1.266	626.5
56	4	600.00	.01000	.070	.322	1.500	1.045	1.241	192.3
57	4	1330.00	.00451	.070	.339	1.500	2.030	1.077	472.7
58	4	1000.00	.00900	.050	.565	1.500	1.169	1.360	132.4

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

Table 2-12 Run 1 Annual Peak Flows and Frequency Analysis at Aztec Ave. (Segment 53)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 53 FOR WARREN CREEK, RUN NO 1, EXISTING LAND USE WITH NO CULVERTS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	84.0	1918	APR.	7
1919	247.7	1919	MAY	6
1920	565.9	1920	AUG.	15
1921	657.8	1921	AUG.	24
1922	505.8	1921	NOV.	16
1923	377.2	1923	FEB.	13
1924	218.5	1924	MAY	27
1925	280.7	1925	JAN.	11
1926	660.8	1926	AUG.	11
1927	165.9	1927	FEB.	23
1928	675.3	1928	JULY	10
1929	253.1	1929	MAR.	14
1930	140.8	1930	MAY	17
1931	155.5	1931	JULY	28
1932	320.8	1932	JUNE	18
1933	405.5	1933	JUNE	10
1934	303.1	1934	JULY	19
1935	107.0	1934	OCT.	6
1936	319.4	1936	SEPT	29
1937	324.8	1937	JUNE	17
1938	556.6	1938	JUNE	25
1939	287.8	1939	JUNE	22
1940	515.2	1940	SEPT	10
1941	332.0	1941	AUG.	13
1942	300.6	1942	MAR.	20

MEAN = 350.5 STANDARD DEVIATION = 175.6

FLOOD FREQUENCY FOR FLOWPOINT 53 FOR WARREN CREEK, RUN NO 1, EXISTING LAND USE WITH NO CULVERTS

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	321.6	917.6
5-YEAR	20.0	476.8	919.3
10-YEAR	10.0	579.6	920.4
25-YEAR	4.0	709.4	921.7
50-YEAR	2.0	805.8	922.5
100-YEAR	1.0	901.4	922.7
200-YEAR	0.5	996.6	922.8

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-13 Run 1 Annual Peak Flows and Frequency Analysis behind Santa Fe Drive (Segment 50)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 50 FOR WARREN CREEK, RUN NO 1, EXISTING LAND USE WITH NO CULVERTS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	137.6	1918	APR.	7
1919	358.7	1919	MAY	6
1920	717.4	1920	AUG.	15
1921	874.0	1921	AUG.	24
1922	691.4	1921	NOV.	16
1923	455.6	1923	FEB.	13
1924	340.8	1924	MAY	27
1925	429.2	1924	DEC.	8
1926	867.7	1926	AUG.	11
1927	220.7	1927	FEB.	23
1928	819.2	1928	JULY	10
1929	332.3	1929	MAR.	14
1930	223.9	1930	MAY	17
1931	325.4	1931	JULY	28
1932	443.6	1932	JUNE	18
1933	662.2	1933	JUNE	10
1934	497.2	1934	JULY	19
1935	168.0	1934	OCT.	5
1936	472.0	1936	SEPT	29
1937	484.0	1937	JUNE	17
1938	227.5	1938	JUNE	25
1939	460.6	1939	JUNE	22
1940	768.7	1940	SEPT	10
1941	547.2	1941	AUG.	13
1942	372.5	1942	MAR.	20

MEAN = 499.9 STANDARD DEVIATION = 223.4

FLOOD FREQUENCY FOR FLOWPOINT 50 FOR WARREN CREEK, RUN NO 1, EXISTING LAND USE WITH NO CULVERTS

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	465.2	916.6
5-YEAR	20.0	660.6	917.3
10-YEAR	10.0	791.3	917.7
25-YEAR	4.0	956.5	918.2
50-YEAR	2.0	1079.0	918.6
100-YEAR	1.0	1200.6	918.7
200-YEAR	0.5	1321.7	918.9

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-14 Run 1 Annual Peak Flows and Frequency Analysis at I-85 (Segment 58)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 58 FOR WARREN CREEK, RUN NO 1, EXISTING LAND USE WITH NO CULVERTS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	313.3	1918	APR.	7
1919	861.4	1919	MAY	6
1920	1952.5	1920	AUG.	15
1921	2259.3	1921	AUG.	24
1922	1798.4	1921	NOV.	16
1923	1345.4	1923	FEB.	13
1924	812.4	1924	MAY	27
1925	989.6	1925	JAN.	11
1926	2361.7	1926	AUG.	11
1927	585.0	1927	FEB.	23
1928	2365.9	1928	JULY	10
1929	917.0	1929	MAR.	14
1930	462.1	1930	MAY	17
1931	558.0	1931	JULY	26
1932	1194.7	1932	JUNE	16
1933	1433.3	1933	JUNE	10
1934	1047.4	1934	JULY	19
1935	371.0	1934	OCT.	5
1936	1161.8	1936	SEPT	29
1937	1151.9	1937	JUNE	17
1938	1952.7	1938	JUNE	25
1939	1072.8	1939	JUNE	22
1940	1851.7	1940	SEPT	10
1941	1238.4	1941	AUG.	13
1942	1077.8	1942	MAR.	20

MEAN = 1245.4 STANDARD DEVIATION = 613.3

FLOOD FREQUENCY FOR FLOWPOINT 58 FOR WARREN CREEK, RUN NO 1, EXISTING LAND USE WITH NO CULVERTS

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	1144.7	903.0
5-YEAR	20.0	1686.7	904.5
10-YEAR	10.0	2045.6	905.1
25-YEAR	4.0	2499.0	905.7
50-YEAR	2.0	2835.3	906.1
100-YEAR	1.0	3169.2	906.5
200-YEAR	0.5	3501.9	906.9

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-15

Specifications for Storage Segments for Run 2

Storage Segment Number	Storage (acre-feet)	Discharge (cfs)	Surface Elevation (feet-msl)	Head (feet)	Surface Area (Acres)
3	.000	0	934.7	0.0	.00
	.477	181	940.0	5.3	.18
	1.872	289	945.0	10.3	.37
	4.828	377	950.0	15.3	.95
	12.340	3516	955.0	20.3	2.11
	25.399	12210	960.0	25.3	2.95
5	.000	0	957.0	0.0	.00
	.645	277	960.0	3.0	.43
	3.334	641	965.0	8.0	.78
	9.313	943	970.0	13.0	1.65
7	.000	0	932.5	0.0	.00
	.210	72	936.0	3.5	.12
	1.224	108	939.0	6.5	.65
	5.506	14349	945.0	12.5	.81
	12.337	54953	950.0	17.5	2.17
39	.000	0	940.0	0.0	.00
	.000	66	945.0	5.0	.00
	.385	111	950.0	10.0	.23
	3.418	144	955.0	15.0	1.13
	12.337	11440	960.0	20.0	2.42
41	.000	0	916.0	0.0	.00
	.810	522	922.0	6.0	.27
	2.897	815	927.0	11.0	.50
	4.677	5098	930.0	14.0	.63
	9.272	32304	935.0	19.0	1.76
43	.000	0	919.5	0.0	.00
	.087	72	923.0	3.5	.05
	.600	121	927.0	7.5	.28
	2.573	2982	930.0	10.5	.96
	8.548	20307	935.0	15.5	1.84
	21.060	58501	940.0	20.5	2.77
45	.000	0	914.0	0.0	.00
	.637	571	921.5	7.5	.17
	2.420	3345	925.0	11.0	.64
	8.955	17807	930.0	16.0	3.14
	34.077	49006	935.0	21.0	5.79

Table 2-15 continued

Storage Segment Number	Storage (acre-feet)	Discharge (cfs)	Surface Elevation (feet-msl)	Head (feet)	Surface Area (Acres)
47	.000	0	930.0	0.0	.00
	.350	424	935.0	5.0	.14
	2.150	710	940.0	10.0	.73
	9.118	1002	945.0	15.0	2.20
	26.505	1214	950.0	20.0	5.08
48	.000	0	926.5	0.0	.00
	.722	99	935.0	8.5	.17
	2.678	3357	940.0	13.5	.51
	6.440	12788	945.0	18.5	1.22
54	.000	0	899.5	0.0	.00
	1.800	825	905.5	6.0	.60
	13.336	1366	912.0	12.5	4.00
	38.620	2971	915.0	15.5	6.65

Table 2-16 Run 2 Annual Peak Flows and Frequency Analysis at Aztec Ave. (Segment 53)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 53 FOR WARREN CREEK, RUN NO 1A, EXIST LAND USE WITH CULVERTS, FLOWS AT REACHES

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	85.9	1918	APR.	7
1919	239.9	1919	MAY	6
1920	543.5	1920	AUG.	15
1921	643.1	1921	AUG.	24
1922	502.0	1921	NOV.	16
1923	371.0	1923	FEB.	13
1924	223.6	1924	MAY	27
1925	278.2	1925	JAN.	11
1926	655.7	1926	AUG.	11
1927	166.0	1927	FEB.	23
1928	669.8	1928	JULY	10
1929	257.9	1929	MAR.	14
1930	132.3	1930	MAY	17
1931	152.4	1931	JULY	28
1932	321.7	1932	JUNE	18
1933	383.8	1933	JUNE	10
1934	276.2	1934	JULY	19
1935	105.3	1934	OCT.	5
1936	306.8	1936	SEPT	29
1937	312.6	1937	JUNE	17
1938	538.7	1938	JUNE	25
1939	284.8	1939	JUNE	22
1940	508.5	1940	SEPT	10
1941	322.6	1941	AUG.	13
1942	301.7	1942	MAR.	20

MEAN = 343.4 STANDARD DEVIATION = 172.2

FLOOD FREQUENCY FOR FLOWPOINT 53 FOR WARREN CREEK, RUN NO 1A, EXIST LAND USE WITH CULVERTS, FLOWS AT REACHES

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	315.1	917.6
5-YEAR	20.0	467.3	919.2
10-YEAR	10.0	568.0	920.2
25-YEAR	4.0	695.3	921.5
50-YEAR	2.0	789.8	922.5
100-YEAR	1.0	883.6	922.6
200-YEAR	0.5	977.0	922.7

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-17 Run 2 Annual Peak Flows and Frequency Analysis behind Santa Fe Drive (Segment 50)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 50 FOR WARREN CREEK, RUN NO 1A, EXIST LAND USE WITH CULVERTS, FLOWS AT REACHES

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	138.4	1918	APR.	7
1919	353.7	1919	MAY	6
1920	710.9	1920	AUG.	15
1921	828.2	1921	AUG.	24
1922	681.1	1921	NOV.	16
1923	465.7	1923	FEB.	13
1924	335.5	1924	MAY	27
1925	425.6	1924	DEC.	8
1926	860.2	1926	AUG.	11
1927	220.8	1927	FEB.	23
1928	809.3	1928	JULY	10
1929	326.8	1929	MAR.	14
1930	2[illegible]0.9	1930	MAY	17
1931	305.3	1931	JULY	28
1932	457.3	1932	JUNE	18
1933	63[illegible].1	1933	JUNE	10
1934	49[illegible].8	1934	JULY	19
1935	16[illegible].7	1934	OCT.	5
1936	48[illegible].9	1936	SEPT	29
1937	46[illegible].9	1937	JUNE	17
1938	77[illegible].3	1938	JUNE	25
1939	47[illegible].2	1939	JUNE	22
1940	757.6	1940	SEPT	10
1941	558.0	1941	AUG.	13
1942	377.0	1942	MAR.	20

MEAN = 492.2 STANDARD DEVIATION = 216.1

FLOOD FREQUENCY FOR FLOWPOINT 50 FOR WARREN CREEK, RUN NO 1A, EXIST LAND USE WITH CULVERTS, FLOWS AT REACHES

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	456.7	916.6
5-YEAR	20.0	647.7	917.2
10-YEAR	10.0	774.2	917.7
25-YEAR	4.0	934.0	918.2
50-YEAR	2.0	1052.5	918.5
100-YEAR	1.0	1170.2	918.7
200-YEAR	0.5	1287.4	918.8

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-18 Run 2 Annual Peak Flows and Frequency Anaysis at I-85 (Segment 58)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 58 FOR WARREN CREEK, RUN NO 1A, EXIST LAND USE WITH CULVERTS, FLOWS AT REACHES

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	298.6	1918	APR.	7
1919	804.2	1919	MAY	6
1920	1691.6	1920	AUG.	15
1921	1910.4	1921	AUG.	24
1922	1621.9	1921	NOV.	16
1923	1270.5	1923	FEB.	13
1924	767.3	1924	MAY	27
1925	940.8	1925	JAN.	11
1926	2027.1	1926	AUG.	11
1927	557.4	1927	FEB.	23
1928	2032.1	1928	JULY	10
1929	873.6	1929	MAR.	14
1930	410.4	1930	MAY	17
1931	509.0	1931	JULY	28
1932	1139.5	1932	JUNE	18
1933	1303.0	1933	JUNE	10
1934	945.8	1934	JULY	19
1935	347.8	1934	OCT.	5
1936	1097.5	1936	SEPT	29
1937	1043.0	1937	JUNE	17
1938	1707.0	1938	JUNE	25
1939	1010.6	1939	JUNE	22
1940	1673.8	1940	SEPT	10
1941	1167.2	1941	AUG.	13
1942	1020.6	1942	MAR.	20

MEAN = 1126.8 STANDARD DEVIATION = 516.7

FLOOD FREQUENCY FOR FLOWPOINT 58 FOR WARREN CREEK, RUN NO 1A, EXIST LAND USE WITH CULVERTS, FLOWS AT REACHES

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	1042.0	902.4
5-YEAR	20.0	1498.6	904.3
10-YEAR	10.0	1800.9	904.7
25-YEAR	4.0	2182.8	905.2
50-YEAR	2.0	2466.2	905.6
100-YEAR	1.0	2747.5	906.0
200-YEAR	0.5	3027.7	906.3

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

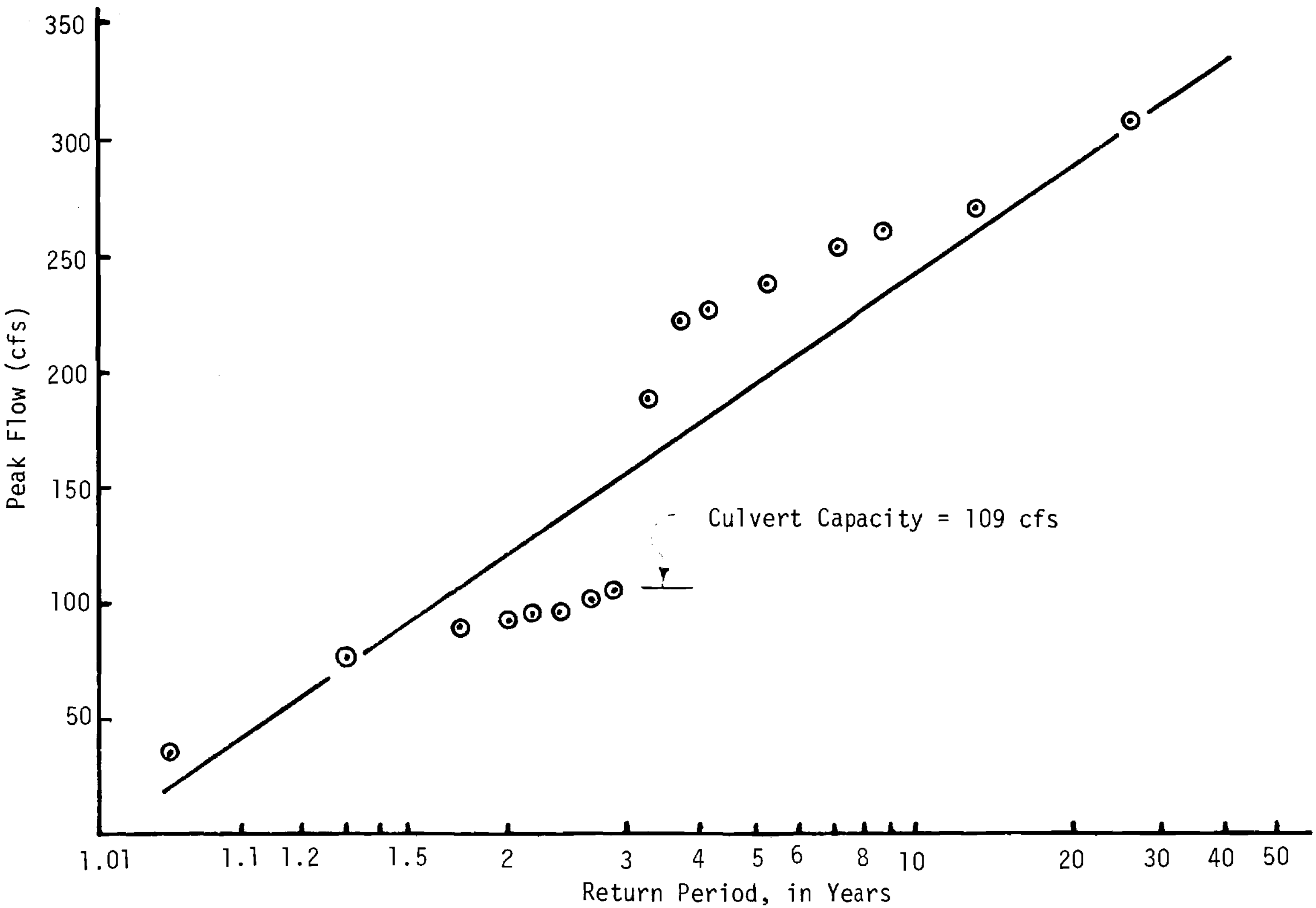


Figure 2-6. Effect of Upstream Culvert on the Flood Frequency in Channel Segment 7

Table 2-19

Simulated Flows at Existing Culverts

Road	Storage Segment	Capacity cfs	Max. Sim. Pk. cfs	Return Pd. Overtopped years
Pineland	3	377	317	40
North Dekalb	5	944	369	>200
McClave	7	109	309	3
Brook Park	39	145	91	200
Wheeler	41	815	580	100
Wheeler	43	122	142	15
Chesnut	45	572	712	15
I-285	47	1214	753	>200
Poplar	48	100	155	5
Aztec	54	1366	1065	100
I-85	64	3250	2086	200

The effects of storage behind existing culverts on three simulated flood peaks are summarized in Table 2-20. By comparing these results with Figure 2-2, one can see that the culverts are not affecting the flows on the north branch but are substantially reducing flows on the west branch. Closer study shows the culverts at Pineland (3) and Aztec (54) to be the ones having the primary downstream flood reduction effect. At least in this case, culverts are initially constructed larger in industrial than they are in residential areas,and this combines with a greater imperviousness to compound downstream flooding.

Table 2-20

Differences in Three Simulated Flood Peaks With and Without Storage Behind Existing Culverts

	1921 Peak			1926 Peak			1928 Peak		
Channel Reach	Without cfs	With cfs	Per. Red.	Without cfs	With cfs	Per. Red.	Without cfs	With cfs	Per. Red.
40	570	452	21	534	448	16	526	449	14
42	142	119	16	128	117	8	139	127	8
44	370	285	23	354	303	14	352	301	14
46	837	850	0	754	789	0	686	718	0
50	757	742	2	757	768	0	713	723	0
51	474	389	18	472	416	12	498	431	13
52	153	138	9	131	134	0	129	127	2
53	790	541	31	789	581	26	807	590	27
56	1235	727	41	1250	791	37	1283	828	35
57	876	863	1	923	928	0	883	885	0
58	2210	1528	31	2300	1662	28	2292	1673	27

5. Flooding of Houses Facing Santa Fe Drive

Seven houses facing Santa Fe Drive were found subject to inundation by the 100-year flood (from channel segment 50). Possible remedial measures include detention storage behind I-285, channelization of the stream in back of the houses, flood proofing the buildings, and purchase. As a hydrologic study, this analysis only dealt with the reduction in flood flows that could be achieved by detention storage and with the degree to which channelization to protect the houses might increase flood flows further downstream.

The design contemplated for storing water behind the I-285 embankment was to place an entrance structure on the existing 8' by 7' box culvert that would back the water up by restricting inflow. The design 100-year surface elevation for the ponded water was 950 feet or 10 feet below the roadway elevation of 960'. Run 7 was used to size an appropriate entrance. The data for the three area, one channel, and two storage segments found upstream from I-285 were the same as shown on Tables 2-11 and 2-15 except for the effect of the diminished culvert entrance. This effect was simulated by using values of JT = 6, JEL = 938, and several values of JS1 at half-foot increments to find an appropriate entrance size. A diameter of five feet was found to be the smallest with a 100-year water surface elevation less than 950'.

Run 8c then was based on the segment data on Tables 2-11 and 2-15 except for the modification to storage segment 47 to account for the optimum inlet diameter of five feet. The reductions in flows achieved behind the Santa Fe homes (Channel segment 50) are shown on Table 2-21. The storage is shown to be effective in flood peak reduction in this critical damage area.

Run 9 used the same data for the segments tributary to Reach 50 as did Run 8c but simulated the complete annual series required for the

Table 2-21

Reduction in Flood Flows Behind Santa Fe Drive from Detention Storage behind I-285 Embankment

Flood Year	Flow without Storage cfs	Flow with Storage cfs	Reduction Percent	Reduction Feet
1920	711	451	37	0.8
1921	828	533	36	1.0
1926	860	594	31	0.9
1928	809	542	33	0.8
1933	636	376	41	0.9
1938	775	488	37	1.0

frequency analysis. The estimated annual flood decreased from 457 to 314 cfs or by 0.6 feet. The 100-year flood decreased from 1170 to 758 cfs or by 1.1 feet. This will substantially reduce the flooding problem, but more precise channel sections and backwater studies are needed to evaluate whether this storage alone would provide adequate protection for the homes.

Run 4 examined the downstream hydrologic effects of channelization in segment 50. The data in Tables 2-11 and 2-15 were again used except for the modification to channel segment 50 to reflect a 50-foot bottom width channel with 2 on 1 side slopes and n = 0.025. The simulation showed that the channelization of the full 1760-foot length of channel segment 50 would increase both the mean annual and the 100-year flood peak by 10.6 percent at the downstream end of the reach, would increase them by 7.5% at the end of reach 57 and by 2.8% at the end of reach 58 at I-85. As very little flood damage is now occuring along Channel Segments 57 and 58, and the percentage increases are so small, it would be hard to argue against channelization in this case on the basis of adverse downstreams hydrologic effects. The effects would be smaller if only the 600 feet of channel immediately behind the homes are channelized and smaller yet if channelization takes the form of a small levee between the affected homes and the creek and located near the back of the lots. Such a levee and detention storage would seem to be the most promising alternatives and should be compared in seeking a least cost solution.

6. Other Drainage Problems

Other lesser drainage problems occur just upstream from Wheeler Drive near its intersection with Addison Drive and at other locations. Two storage sites were considered for reducing flows in the Wheeler Drive area. One was in the low area upstream from Brook Park Road, and the other was at a site behind Sequoyah High School. Run 5 was used to size an appropriate entrance to the culvert under Brook Park Road. A 2.5-foot culvert was found to be the smallest with a 100-year water surface elevation in storage segment 39 that did not overtop the road. Run 6 was used to size an outlet for the possible detention storage behind Sequoyah High School. A 4.5 foot outlet was found to be the smallest with a 100-year water surface elevation in storage segment 49 that did not exceed the design value of 930 feet.

Runs 8a and 8b were based on the segment data in Tables 2-11 and 2-15 except for the modifications to storage segments 39 and 49 respectively. The Brook Park storage site was judged ineffective from the results summarized in Table 2-22. The effects of the Sequoyah site are summarized in Table 2-23. That site is somewhat more effective in reducing downstream flood peaks since it controls a larger area. Even though this site does significantly reduce downstream flood peaks, it hardly seems justified at present because there is little if any damageable property in the affected area. Its most likely justification would be as compensation for future urbanization.

Run 9 included the Sequoyah but not the Brook Park storage. The simulated annual peaks and results of frequency analysis for channel segments 53, 50, and 58 are shown in Tables 2-24, 2-25 and 2-26, respectively. At I-85 (segment 58) the two storages are seen to reduce the mean-annual flood peak from 1012 to 872 cfs and the stage by 0.8 feet. The 100-year flood is reduced from 2748 to 2203 cfs, and its

Table 2-22

Reduction in Flood Flows from Storage at Brook Park Site

Segment	Flood Year	Flow without Storage cfs	Flow with Storage cfs	Reduction Percent
42	1920	78	72	8
	1921	83	77	7
	1926	79	75	5
	1928	89	81	9
	1933	42	44	0
	1938	68	62	9
53	1920	544	538	1
	1921	643	637	1
	1926	656	651	1
	1928	670	661	1
	1933	384	383	0
	1938	539	534	1
56	1920	888	884	0
	1921	968	964	0
	1926	1023	1017	1
	1928	1064	1047	2
	1933	641	637	1
	1938	863	860	0
58	1920	1692	1676	1
	1921	1910	1907	0
	1926	2027	2021	0
	1928	2032	1997	2
	1933	1303	1295	1
	1938	1707	1705	0

Table 2-23

Reduction in Flood Flows from Storage at Sequoyah Site

Segment	Flood Year	Flow without Storage cfs	Flow with Storage cfs	Reduction Percent
51	1920	355	212	40
	1921	388	224	42
	1926	411	241	41
	1928	432	251	42
	1933	236	151	36
	1938	332	197	41
56	1920	888	805	9
	1921	968	865	11
	1926	1023	905	12
	1928	1064	921	13
	1933	641	542	15
	1938	863	777	10
58	1920	1692	1607	5
	1921	1910	1814	5
	1926	2027	1926	5
	1928	2032	1892	7
	1933	1303	1203	8
	1938	1707	1636	4

Table 2-24 Run 9 Annual Peak Flows and Frequency Analysis at Aztec Ave. (Segment 53)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 53 FOR WARREN CREEK, RUN NC 9,RUN 2 + ACCEPTED STORAGE UNITS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	85.9	1918	APR.	7
1919	[illegible]	1919	MAY	6
1920	[illegible]	1920	AUG.	15
1921	[illegible]	1921	AUG.	24
1922	[illegible]	1921	NOV.	16
1923	371.0	1923	FEB.	13
1924	[illegible]	1924	MAY	27
1925	[illegible]	1925	JAN.	11
1926	[illegible]	1926	AUG.	11
1927	[illegible]	1927	FEB.	23
1928	[illegible]	1928	JULY	10
1929	[illegible]	1929	MAR.	14
1930	[illegible]	1930	MAY	17
1931	[illegible]	1931	JULY	28
1932	[illegible]	1932	JUNE	18
1933	[illegible]	1933	JUNE	10
1934	276.2	1934	JULY	19
1935	[illegible]	1934	OCT.	5
1936	306.8	1936	SEPT	29
1937	312.6	1937	JUNE	17
1938	[illegible]	1938	JUNE	25
1939	[illegible]	1939	JUNE	22
1940	[illegible]	1940	SEPT	10
1941	[illegible]	1941	AUG.	13
1942	[illegible]	1942	MAR.	20

MEAN = 343.4 STANDARD DEVIATION = 172.2

FLOOD FREQUENCY FOR FLOWPOINT 53 FOR WARREN CREEK, RUN NO 9,RUN 2 + ACCEPTED STORAGE UNITS

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	315.1	917.6
5-YEAR	20.0	467.3	919.2
10-YEAR	10.0	[illegible]	920.2
25-YEAR	4.0	695.3	921.5
50-YEAR	2.0	789.8	922.5
100-YEAR	1.0	883.6	922.6
200-YEAR	0.5	977.0	922.7

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-25 Run 9 Annual Peak Flows and Frequency Analysis behind Santa Fe Drive (Segment 50)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 50 FOR WARREN CREEK, RUN NC 9,RUN 2 + ACCEPTED STORAGE UNITS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	119.9	1918	APR.	7
1919	240.9	1919	MAY	8
1920	455.3	1920	AUG.	15
1921	533.1	1921	AUG.	24
1922	452.6	1921	NOV.	16
1923	359.0	1923	FEB.	13
1924	363.4	1924	MAY	27
1925	582.5	1925	JAN.	11
1926	594.3	1926	AUG.	11
1927	179.5	1927	FEB.	23
1928	549.6	1928	JULY	10
1929	247.6	1929	MAR.	14
1930	139.5	1930	MAY	17
1931	183.5	1931	JULY	28
1932	473.5	1932	JUNE	18
1933	376.7	1933	JUNE	10
1934	290.5	1934	JULY	18
1935	126.7	1934	OCT.	5
1936	465.6	1936	SEPT	26
1937	304.0	1937	JUNE	17
1938	487.4	1938	JUNE	25
1939	340.1	1939	JUNE	22
1940	477.8	1940	SEPT	10
1941	371.2	1941	AUG.	13
1942	289.0	1942	MAR.	20

MEAN = 336.3 STANDARD DEVIATION = 134.5

FLOOD FREQUENCY FOR FLOWPOINT 50 FOR WARREN CREEK, RUN NO 9,RUN 2 + ACCEPTED STORAGE UNITS

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	314.2	916.0
5-YEAR	20.0	453.1	916.5
10-YEAR	10.0	511.9	916.8
25-YEAR	4.0	611.3	917.1
50-YEAR	2.0	685.1	917.4
100-YEAR	1.0	758.4	917.6
200-YEAR	0.5	831.4	917.8

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-26 Run 9 Annual Peak Flows and Frequency Analysis at I-85 (Segment 58)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 58 FOR WARREN CREEK, RUN NO 9,RUN 2 + ACCEPTED STORAGE UNITS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	282.8	1918	APR.	7
1919	685.5	1919	MAY	6
1920	1337.9	1920	AUG.	15
1921	1552.8	1921	AUG.	24
1922	1314.6	1921	NOV.	16
1923	1087.8	1923	FEB.	13
1924	682.0	1924	MAY	27
1925	840.9	1925	JAN.	11
1926	1664.8	1926	AUG.	11
1927	525.7	1927	FEB.	23
1928	1665.0	1928	JULY	10
1929	745.6	1929	MAR.	14
1930	368.2	1930	MAY	17
1931	432.6	1931	JULY	28
1932	998.0	1932	JUNE	18
1933	902.0	1933	JUNE	10
1934	739.7	1934	JULY	19
1935	3[illegible]9.9	1934	OCT.	5
1936	9[illegible].5	1936	SEPT	29
1937	8[illegible].6	1937	JUNE	17
1938	1351.1	1938	JUNE	25
1939	916.3	1939	JUNE	22
1940	1310.9	1940	SEPT	10
1941	9[illegible].5	1941	AUG.	13
1942	803.2	1942	MAR.	20

MEAN = 938.4 STANDARD DEVIATION = 403.2

FLOOD FREQUENCY FOR FLOWPOINT 58 FOR WARREN CREEK, RUN NO 9,RUN 2 + ACCEPTED STORAGE UNITS

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	872.2	901.6
5-YEAR	20.0	1223.6	903.4
10-YEAR	10.0	14[illegible]4.5	904.2
25-YEAR	4.0	1762.6	904.7
50-YEAR	2.0	1983.8	905.0
100-YEAR	1.0	2203.3	905.3
200-YEAR	0.5	2422.0	905.6

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

stage drops 0.7 feet.

7. Development of Wooded Area

Run 3 examined the downstream hydrologic effects of development of the wooded hillsides in area segments 26 and 34. The data in Tables 2-11 and 2-15 were modified to provide impervious fractions of 0.70 in these two segments. The simulation showed that this urban development would increase the mean annual flood peak by 7.2 percent and the 100-year flood peak by 5.2 percent at the downstream end of Reach 50. The increases are 6.1 percent and 4.3 percent at the end of Reach 57 and 2.8 percent and 1.6 percent at the end of Reach 58. No stages are affected by more than 0.1 foot. Consequently, downstream flood control benefits do not present a very strong argument against development of this area.

8. Effects of Additional Urban Development on Flooding

Runs 10 and 11 were used to examine the degree to which one might expect flooding to be aggravated by the urban development projected in Table 2-10. Run 10 examined the effects of urban growth alone. Run 11 examined the effects of urban growth with compensating storage at storage segments 47 and 49. Although not simulated, smaller sites on individual developments are an alternate way to provide compensating storage. The simulated annual peaks and results of the frequency analysis from Run 10 for channel segments 53, 50, and 58 are shown in Tables 2-27, 2-28, and 2-29, respectively. The results from Run 11 for the same three channel segments are shown in Tables 2-30, 2-31, and 2-32.

At Reach 53, the additional urban development would increase the mean annual flood from 315 to 361 cfs for a 0.5 foot rise in stage. The 100-year flood would increase from 884 to 986 feet, a rise of 0.2 feet. Flows at this location are not affected by the two storage sites.

At Reach 50, the additional urban development would increase the 100-year flood from 1170 to 1273 cfs and add 0.1 feet to its stage. Storage would reduce the flow to 821 cfs or 0.9 feet below the current peak stage.

At Reach 58, the additional urban development would increase the 100-year flood from 2748 to 3005 cfs and add 0.3 feet to its stage. Storage would reduce the flow to 2410 cfs or 0.5 feet below the current peak stage.

Table 2-27 Run 10 Annual Peak Flows and Frequency Analysis at Aztec Ave. (Segment 53)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 53 FOR WARREN CREEK, RUN NO 10A, HILL SIDE DEV. FLOWS AT REACHES

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	108.3	1918	APR.	7
1919	265.4	1919	MAY	6
1920	604.4	1920	AUG.	15
1921	702.8	1921	AUG.	24
1922	557.7	1921	NOV.	16
1923	383.1	1923	FEB.	13
1924	256.3	1924	MAY	27
1925	296.8	1924	DEC.	8
1926	730.3	1926	AUG.	11
1927	179.0	1927	FEB.	23
1928	698.1	1928	JULY	10
1929	271.1	1929	MAR.	14
1930	154.8	1930	MAY	17
1931	206.1	1931	JULY	28
1932	368.3	1932	JUNE	18
1933	484.9	1933	JUNE	10
1934	348.2	1934	JULY	19
1935	121.9	1934	OCT.	5
1936	367.9	1936	SEPT	29
1937	360.7	1937	JUNE	17
1938	645.7	1938	JUNE	25
1939	349.5	1939	JUNE	22
1940	614.1	1940	SEPT	10
1941	417.2	1941	AUG.	13
1942	311.9	1942	MAR.	20

MEAN = 392.2 STANDARD DEVIATION = 189.3

FLOOD FREQUENCY FOR FLOWPOINT 53 FOR WARREN CREEK, RUN NO 10A, HILL SIDE DEV. FLOWS AT REACHES

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	361.1	918.1
5-YEAR	20.0	528.4	919.8
10-YEAR	10.0	639.2	921.0
25-YEAR	4.0	779.2	922.4
50-YEAR	2.0	883.0	922.6
100-YEAR	1.0	986.1	922.8
200-YEAR	0.5	1088.8	922.9

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-28 Run 10 Annual Peak Flows and Frequency Analysis behind Santa Fe Drive (Segment 50)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 50 FOR WARREN CREEK, RUN NO 10A, HILL SIDE DEV. FLOWS AT REACHES

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	167.9	1918	APR.	7
1919	406.9	1919	MAY	6
1920	755.1	1920	AUG.	15
1921	891.7	1921	AUG.	24
1922	737.6	1921	NOV.	16
1923	480.9	1923	FEB.	13
1924	372.4	1924	MAY	27
1925	518.6	1924	DEC.	8
1926	929.9	1926	AUG.	11
1927	241.0	1926	NOV.	15
1928	846.6	1928	JULY	10
1929	341.4	1929	MAR.	14
1930	245.0	1930	MAY	17
1931	376.0	1931	JULY	28
1932	511.4	1932	JUNE	18
1933	763.0	1933	JUNE	10
1934	568.2	1934	JULY	19
1935	204.4	1934	OCT.	5
1936	548.3	1936	SEPT	29
1937	527.8	1937	JUNE	17
1938	869.4	1938	JUNE	25
1939	548.6	1939	JUNE	22
1940	835.9	1940	SEPT	10
1941	652.3	1941	AUG.	13
1942	391.9	1942	MAR.	20

MEAN = 549.3 STANDARD DEVIATION = 230.8

FLOOD FREQUENCY FOR FLOWPOINT 50 FOR WARREN CREEK, RUN NO 10A, HILL SIDE DEV. FLOWS AT REACHES

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	511.4	916.8
5-YEAR	20.0	715.4	917.5
10-YEAR	10.0	850.4	917.9
25-YEAR	4.0	1021.1	918.4
50-YEAR	2.0	1147.7	918.7
100-YEAR	1.0	1273.4	918.8
200-YEAR	0.5	1398.6	919.0

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-29 Run 10 Annual Peak Flows and Frequency Analysis at I-85 (Segment 58)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 58 FOR WARREN CREEK, RUN NO 10A, HILL SIDE DEV. FLOWS AT REACHES

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	388.5	1918	APR.	7
1919	910.5	1919	MAY	6
1920	1799.7	1920	AUG.	15
1921	2076.8	1921	AUG.	24
1922	1768.0	1921	NOV.	16
1923	1317.5	1923	FEB.	13
1924	919.2	1924	MAY	27
1925	1075.3	1924	DEC.	8
1926	2248.0	1926	AUG.	11
1927	598.7	1927	FEB.	23
1928	2115.6	1928	JULY	10
1929	925.6	1929	MAR.	14
1930	496.4	1930	MAY	17
1931	719.6	1931	JULY	28
1932	1337.6	1932	JUNE	18
1933	1619.7	1933	JUNE	10
1934	1197.5	1934	JULY	19
1935	430.2	1934	OCT.	5
1936	1312.7	1936	SEPT	29
1937	1211.0	1937	JUNE	17
1938	1957.9	1938	JUNE	25
1939	1281.8	1939	JUNE	22
1940	1899.4	1940	SEPT	10
1941	1509.2	1941	AUG.	13
1942	1069.5	1942	MAR.	20

MEAN = 1287.4 STANDARD DEVIATION = 547.5

FLOOD FREQUENCY FOR FLOWPOINT 58 FOR WARREN CREEK, RUN NO 10A, HILL SIDE DEV. FLOWS AT REACHES

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	1197.5	903.2
5-YEAR	20.0	1681.3	904.5
10-YEAR	10.0	2001.7	905.0
25-YEAR	4.0	2406.4	905.5
50-YEAR	2.0	2706.6	905.9
100-YEAR	1.0	3004.7	906.3
200-YEAR	0.5	3301.6	906.7

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-30 Run 11 Annual Peak Flows and Frequency Analysis at Aztec Ave. (Segment 53)

NNUAL PEAK FLOWS FOR SEGMENT NO. 53 FOR WARREN CREEK, RUN NO 11, RUN 9 + PROJECTED IA'S

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	108.3	1918	APR.	7
1919	265.4	1919	MAY	6
1920	604.4	1920	AUG.	15
1921	702.8	1921	AUG.	24
1922	557.7	1921	NOV.	16
1923	383.1	1923	FEB.	13
1924	256.3	1924	MAY	27
1925	296.8	1924	DEC.	8
1926	730.3	1926	AUG.	11
1927	179.0	1927	FEB.	23
1928	698.1	1928	JULY	10
1929	271.1	1929	MAR.	14
1930	154.8	1930	MAY	17
1931	206.1	1931	JULY	28
1932	368.3	1932	JUNE	18
1933	484.9	1933	JUNE	10
1934	348.2	1934	JULY	19
1935	121.9	1934	OCT.	5
1936	367.9	1936	SEPT	29
1937	360.7	1937	JUNE	17
1938	645.7	1938	JUNE	25
1939	349.5	1939	JUNE	22
1940	614.1	1940	SEPT	10
1941	417.2	1941	AUG.	13
1942	311.9	1942	MAR.	20

MEAN = 392.2 STANDARD DEVIATION = 189.3

FLOOD FREQUENCY FOR FLOWPOINT 53 FOR WARREN CREEK, RUN NO 11, RUN 9 + PROJECTED IA'S

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	361.1	918.1
5-YEAR	20.0	528.4	919.8
10-YEAR	10.0	639.2	921.0
25-YEAR	4.0	779.2	922.4
50-YEAR	2.0	883.0	922.6
100-YEAR	1.0	986.1	922.8
200-YEAR	0.5	1088.8	922.9

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-31 Run 11 Annual Peak Flows and Frequency Analysis behind Santa Fe Drive (Segment 50)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 50 FOR WARREN CREEK, RUN NO 11, RUN 9 + PROJECTED IA'S

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	137.7	1918	APR.	7
1919	267.9	1919	MAY	6
1920	470.7	1920	AUG.	15
1921	580.8	1921	AUG.	24
1922	494.4	1921	NOV.	16
1923	365.1	1923	FEB.	13
1924	302.3	1924	MAY	27
1925	327.0	1924	DEC.	8
1926	643.3	1926	AUG.	11
1927	192.9	1927	FEB.	23
1928	563.9	1928	JULY	10
1929	255.4	1929	MAR.	14
1930	159.2	1930	MAY	17
1931	228.4	1931	JULY	28
1932	415.2	1932	JUNE	18
1933	444.9	1933	JUNE	10
1934	338.1	1934	JULY	19
1935	150.1	1934	OCT.	5
1936	418.8	1936	SEPT	29
1937	340.2	1937	JUNE	17
1938	547.2	1938	JUNE	25
1939	385.1	1939	JUNE	22
1940	535.5	1940	SEPT	10
1941	434.8	1941	AUG.	13
1942	295.7	1942	MAR.	20

MEAN = 371.8 STANDARD DEVIATION = 143.1

FLOOD FREQUENCY FOR FLOWPOINT 50 FOR WARREN CREEK, RUN NO 11, RUN 9 + PROJECTED IA'S

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	348.3	916.2
5-YEAR	20.0	474.7	916.7
10-YEAR	10.0	558.4	916.9
25-YEAR	4.0	664.2	917.3
50-YEAR	2.0	742.6	917.6
100-YEAR	1.0	820.5	917.8
200-YEAR	0.5	898.1	918.1

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 2-32 Run 11 Annual Peak Flow and Frequency Analysis at I-85 (Segment 58)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 58 FOR WARREN CREEK, RUN NO 11, RUN 9 + PROJECTED IA'S

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	361.7	1918	APR.	7
1919	765.9	1919	MAY	6
1920	1465.1	1920	AUG.	15
1921	1691.6	1921	AUG.	24
1922	1474.7	1921	NOV.	16
1923	1126.3	1923	FEB.	13
1924	813.7	1924	MAY	27
1925	879.6	1925	JAN.	11
1926	1822.0	1926	AUG.	11
1927	559.0	1927	FEB.	23
1928	1725.1	1928	JULY	10
1929	787.9	1929	MAR.	14
1930	434.5	1930	MAY	17
1931	576.4	1931	JULY	28
1932	1161.1	1932	JUNE	18
1933	1233.9	1933	JUNE	10
1934	912.4	1934	JULY	19
1935	390.3	1934	OCT.	5
1936	1141.2	1936	SEPT	29
1937	956.5	1937	JUNE	17
1938	1568.0	1938	JUNE	25
1939	1062.2	1939	JUNE	22
1940	1537.8	1940	SEPT	10
1941	1189.9	1941	AUG.	13
1942	911.8	1942	MAR.	20

MEAN = 1061.9   STANDARD DEVIATION = 429.8

FLOOD FREQUENCY FOR FLOWPOINT 58 FOR WARREN CREEK, RUN NO 11, RUN 9 + PROJECTED IA'S

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	991.4	902.2
5-YEAR	20.0	1371.2	904.0
10-YEAR	10.0	1622.6	904.5
25-YEAR	4.0	1940.3	904.9
50-YEAR	2.0	2176.1	905.2
100-YEAR	1.0	2410.0	905.5
200-YEAR	0.5	2643.1	905.9

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

V. Summary and Conclusions

The flows and stages for the mean annual and 100-year flood peaks for the seven principal runs of Warren Creek are shown in Table 2-33. The culverts on Pineland and Aztec were found to be ponding sufficient water behind them to have a significant effect in reducing downstream flooding. Locations above I-285 and behind Sequoyah High School are the most promising sites for additional storage. Damage to homes on Santa Fe Drive can be reduced by storage upstream from the I-285 embankment, by a levee behind the homes, or by flood proofing.

Curves were developed for general use in DeKalb County for estimating the separate effects of added impervious area and channelization on downstream flood peaks. Information showing how critical storms vary with watershed characteristics is also provided.

Table 2-33

Summary of Warren Creek Simulation Results

Run	1	2	3*	4*	9	10	11
Reach 24							
Mean Q	59	59			59	59	59
Mean Stage**	934.3	934.3			934.3	934.3	934.3
100-yr Q	155	155			155	155	155
100-yr Stage	936.1	936.1			936.1	936.1	936.1
Reach 40							
Mean Q	287	284			284	316	316
Mean Stage	924.7	924.7			924.7	924.7	924.7
100-yr Q	761	776			776	858	858
100-yr Stage	925.6	925.7			925.7	925.8	925.8
Reach 42							
Mean Q	40	40			40	54	54
Mean Stage	923.6	923.0			923.0	923.3	923.3
100-yr Q	117	117			117	139	139
100-yr Stage	923.7	923.7			923.7	923.8	923.8
Reach 44							
Mean Q	178	170			170	209	209
Mean Stage	927.7	927.7			927.7	928.1	928.1
100-yr Q	484	431			431	496	496
100-yr Stage	931.6	930.6			930.6	931.2	931.2
Reach 46							
Mean Q	295	294			294	294	294
Mean Stage	932.8	932.8			932.8	932.8	932.8
100-yr Q	771	767			767	767	767
100-yr Stage	934.7	934.7			934.7	934.7	934.7

* Runs 3 and 4 only pertain to Reaches 50,57, and 58 as the other reaches are on Tributaries unaffected by the development and channelization respectively.

** All stages are in feet above mean sea level as read from the topographic maps.

Reach 50

Mean Q	463	457	490	505	314	511	348
Mean Stage	916.6	916.6	916.7	912.7	916.0	916.8	916.2
100-yr Q	1200	1170	1231	1296	758	1273	821
100-yr Stage	918.7	918.7	918.8	913.6	917.6	918.8	917.8

Reach 51

Mean Q	210	207			145	256	168
Mean Stage	917.8	917.8			916.2	918.2	916.8
100-yr Q	593	559			319	645	345
100-yr Stage	919.5	919.4			918.5	919.7	918.6

Reach 52

Mean Q	76	72			72	75	75
Mean Stage	915.2	915.1			915.1	915.2	915.2
100-yr Q	206	199			199	210	210
100-yr Stage	917.4	917.3			917.3	917.4	917.4

Reach 53

Mean Q	322	315			315	361	361
Mean Stage	917.6	917.6			917.6	918.1	918.1
100-yr Q	1640	1407			1234	1548	1339
100-yr Stage	914.6	913.8			913.8	914.3	913.5

Reach 57

Mean Q	514	512	543	548	388	581	438
Mean Stage	912.0	912.0	912.1	912.1	911.4	912.3	911.7
100-yr Q	1379	1363	1422	1472	959	1496	1069
100-yr Stage	915.3	915.3	915.5	915.6	913.8	915.6	914.2

Reach 58

Mean Q	1145	1042	1071	1068	872	1198	991
Mean Stage	903.0	902.4	902.6	902.6	901.6	903.2	902.2
100-yr Q	3169	2748	2792	2830	2203	3005	2410
100-yr Stage	906.5	906.0	906.0	906.1	905.3	906.3	905.5

SECTION 3

Wild Creek

I. Physical Attributes of Watersheds

a. Location

Approximately 1000 feet below the limit of this study, Wild Creek flows into the North Fork of Peachtree Creek. The portion of Wild Creek Watershed included in this study is the area upstream from the private driveway to Lanier Electronics off Chantilly Road. The stream flows generally to the north west for most of its length, and the study watershed lies entirely within DeKalb County.

b. Size

The watershed studied has a total area of 240 acres. Numerous streets and private driveways cross the creek within the study limits. Selected crossings, drainage areas, and culvert sizes are shown in Table 3-1.

c. General Drainage Patterns

The upper reaches of Wild Creek and its tributaries are steep, with slopes in the order of 0.05 feet/foot. Flow to these reaches in many cases travels over even steeper slopes. In contrast, channels in the lower portion of the Watershed have slopes as flat as .0039 feet/foot.

d. Other Studies

The U. S. Army Corps of Engineers completed a Flood Plan Information Report on North Fork Peachtree Creek in October, 1964. Although this report does not cover the Wild Creek Basin, it shows the flood elevations of North

Table 3-1. Drainage Areas at Selected Road Crossings

Crossing	Drainage Area (acres)	Culvert Size
Private driveway off Chantilly Drive	240.5	3,5'x4' Concrete elliptical pipes
Citadel Drive	143.0	1,3.2'x7.5' Concrete arch pipe
Beach Haven Road	25.9	1,3'x3' Concrete circular pipe
Private Driveway 1178 Wild Creek Trail	32.5	1,3'x2' Concrete elliptical pipe
Wild Creek Trail at Brook Forest Drive	50.5	1,3.3'x6' Concrete arch pipe with retention

Fork Peachtree Creek at the intersection of Wild Creek to be 817 feet above mean sea level for a 25-year flood and 818 feet for a 50-year flood. According to the topographic maps obtained from DeKalb County, floods of these magnitudes would cause flooding in the lower reaches of the Wild Creek watershed defined for this study. The Flood Plain Map in the Corps' report, however, does not show the flood plain of North Fork Peachtree Creek to include any part of the Wild Creek basin. A field check indicated a sizeable elevation drop between the lowest study reaches of Wild Creek and North Fork Peachtree Creek. It was concluded that a difference in datum exists between the maps used in the Corps' study and those used for this study. As additional evidence against backwater flooding in the Wild Creek basin, the flood crest reached during the March, 1975, flood was at the curb inlet on Chantilly Road. Therefore, backwater from North Fork Peachtree Creek was not included in this study.

The DeKalb County drainage engineer made a special report on drainage along Brook Valley Lane on December 27, 1968. His report suggests increasing culvert sizes at various points in the lower reaches of the Watershed from calculations based on the use of Talbot's formula. Additional culverts were suggested in order to raise "c" to 0.8. Additionally, the report stressed the importance of adequate channels on private property.

e. <u>Current Land Use</u>

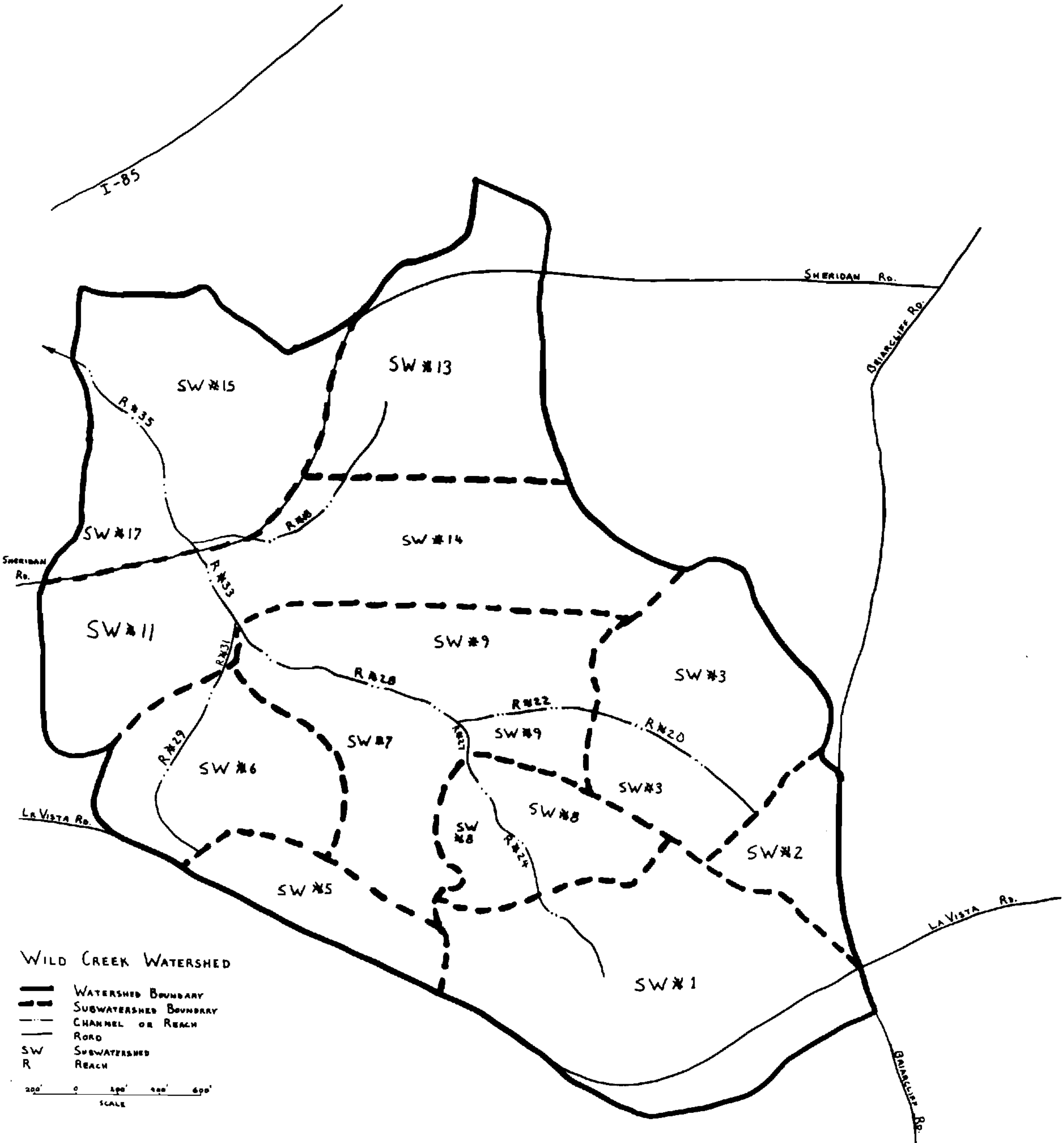
Most of the Wild Creek Watershed is developed. The vast majority of this development is single family residential. Commercial and multi-family residential development has taken place near the intersection of Briarcliff and LaVista Roads. A portion of this area is in the study watershed. Sheridan Road traverses the lower reaches of the watershed. LaVista Road borders the watershed on the south. On the east, Briarcliff Road touches the watershed.

f. Projected Land Use

The projection is for little increase in the intensity of development in the Wild Creek Basin. Interviews with local residents indicate the possibility of an apartment house complex on Sheridan Road. Widening of LaVista Road would bring commercial development if the adjacent land is rezoned.

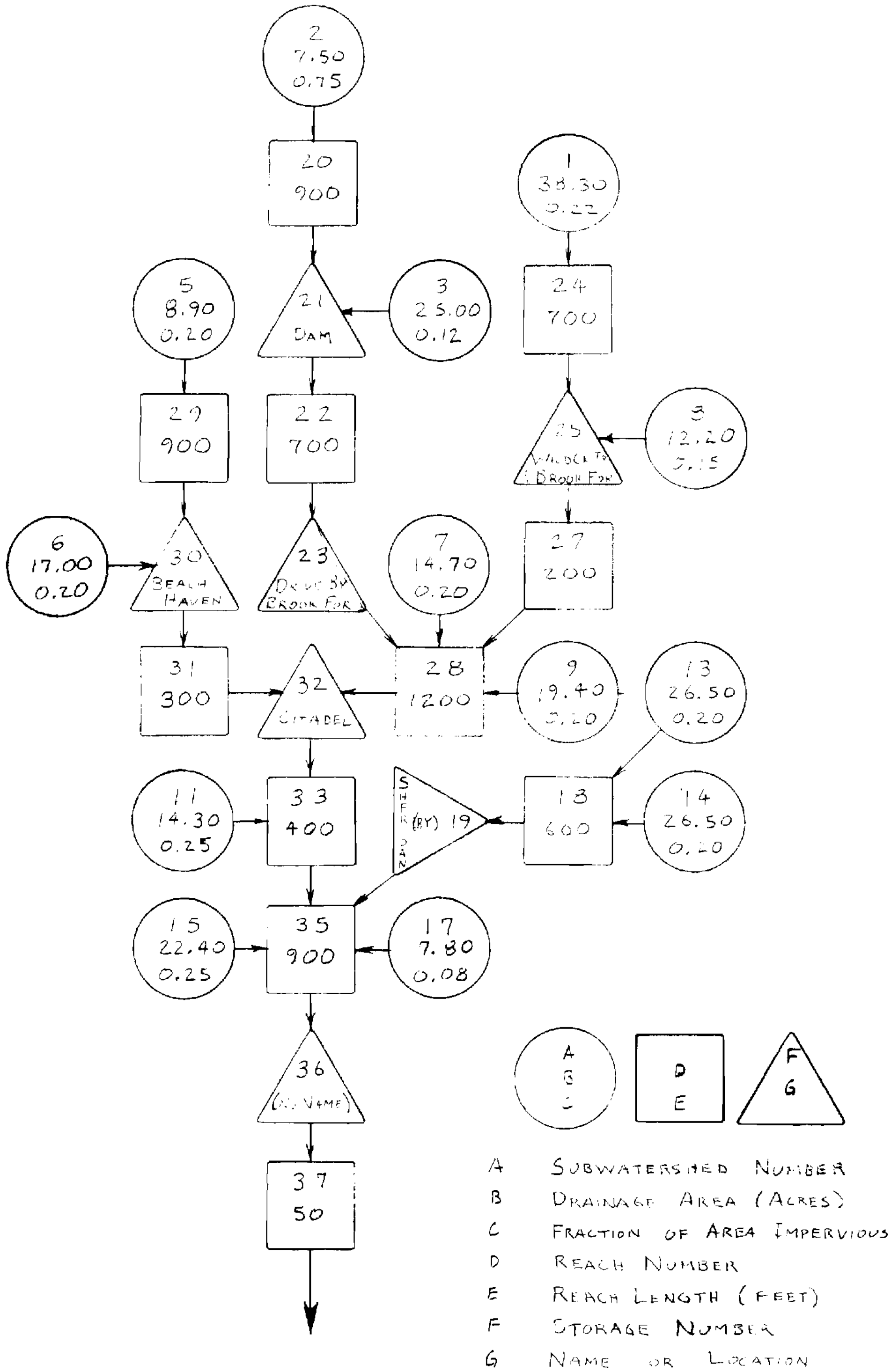
II. Map of the Watershed

Figure 3-1 is a map of the Wild Creek Watershed showing the division of the basin into subwatersheds and channel reaches. Figure 3-2 is a schematic diagram of the subarea, channel, and storage segments used to model the Wild Creek Basin. It shows the general drainage pattern in the basin. Each subarea is represented by a circle showing its area in acres and impervious fraction. Channel reaches are symbolized by squares. Length in feet is given along with channel number. Storage areas are symbolized by triangles containing the segment number and location of each site. It should be noted that each storage shown on the schematic may not be included in every simulation run.



I-85
Sheridan Rd.
Briarcliff Rd.
SW #13
SW #15
R #35
SW #17
Sheridan Rd.
SW #14
R #33
SW #11
SW #9
SW #3
R #28
R #22
R #20
R #31
SW #7
SW #9
R #29
SW #6
SW #3
SW #8
SW #2
La Vista Rd.
SW 8
R #24
SW #5
La Vista Rd.
SW #1
Briarcliff Rd.
Wild Creek Watershed
Watershed Boundary
Subwatershed Boundary
Channel or Reach
Road
SW Subwatershed
R Reach
200' 0 200' 400' 600'
Scale

Figure 3 - 1



WILD CREEK SCHEMATIC DIAGRAM

FIGURE 3 - 2

III. Drainage Problems

At the present time, there are two major problem areas in the Wild Creek watershed. The major problem is along Brook Valley Lane. Residents in this area have long complained of recurring back yard flooding that has at times damaged basements and the ground levels of homes along the creek.

An additional problem exists along Wild Creek Trail where front yard flooding is the major problem. Residents stated that in very large storms their driveways were overtopped by floodwaters.

In order to evaluate these problems, flows were simulated for existing land use on the watershed as well as for two past and two future situations. The five runs were as follows:

Run Number	Watershed Condition
1	Natural conditions - no development
2	Existing development but without the existing detention structure at the intersection of Wild Creek Trail and Brook Forest Drive
3	Existing development with the detention structure at Wild Creek Trail and Brook Forest Drive
4	Run 3 with the addition of a dam above Brook Forest Drive
5	Run 3 with a smaller culvert at Beach Haven Road

IV. Description of Simulation Runs

1. Input specifications

Figure 3-2 shows segment type and links the 37 segments of the Wild Creek watershed. It should be noted that each of the numbered segments is not included in all runs. If a segment is not included, the water is directed to the next downstream segment.

Table 3-2 lists the subareas used in the analysis of the Wild Creek Watershed, and Table 3-3 lists the channels. Channel reach 28 includes the problem area along Wild Creek Trail, and channel 35 includes the problems along Brook Valley Lane. Storage segment descriptions are found in Table 3-4. Figures 3-3 and 3-4 are the rating curves show the effects of the small natural channels in these reaches. The water surface elevation rises quickly until the flow begins to spill out onto the flood plain. After this point, the flow increases with little elevation change.

2. Simulation Runs

Run #1. This simulation was based on natural conditions of no impervious area, natural channels, and no storage segments in the watershed. The results in Tables 3- 5 and 3- 6 show peak flows and frequencies for the two major problem areas, channel segments 28 and 35. For segment 28 (channel parallel to Wild Creek Trail) the mean annual peak flow was 102.2 cfs, and the standard deviation of the series of annual flood peaks was 71.8. This leads to estimates for a 2-year flood of 90 cfs (elevation 1.8 feet) and a 100-year flood of 328 cfs (elevation 3.7 feet). For Reach 35 (channel adjacent to Brook Valley Lane) the mean annual peak flow was 198 cfs with a standard deviation of 138 cfs. The corresponding 2-year flood was 175 cfs (elevation 6.3 feet) and the 100-year flood was 630 cfs (elevation 8.0 feet).

Table 3-2. Specifications for Area Segments for Wild Creek

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE: RAPID	MODERATE	SLOW	IMPERVIOUS	STORAGE CONSTANT	LENGTH-FT	SLOPE	ROUGHNESS
1	38.300	.000	.780	.000	.220	11.	.0000	.0000	.0000
2	7.500	.000	.250	.000	.750	3.	.0000	.0000	.0000
3	25.000	.000	.880	.000	.120	10.	.0000	.0000	.0000
5	8.900	.000	.800	.000	.200	5.	.0000	.0000	.0000
6	17.000	.000	.800	.000	.200	7.	.0000	.0000	.0000
7	14.700	.000	.800	.000	.200	7.	.0000	.0000	.0000
8	12.200	.000	.850	.000	.150	7.	.0000	.0000	.0000
9	19.400	.000	.800	.000	.200	8.	.0000	.0000	.0000
11	14.300	.000	.750	.000	.250	6.	.0000	.0000	.0000
13	26.500	.000	.800	.000	.200	9.	.0000	.0000	.0000
14	26.500	.000	.800	.000	.200	9.	.0000	.0000	.0000
15	22.400	.000	.750	.000	.250	8.	.0000	.0000	.0000
17	7.800	.000	.920	.000	.080	6.	.0000	.0000	.0000

Table 3-3. Specifications for Channel Segments for Wild Creek

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS	CHANNEL APARM	CHANNEL MPARM	FLOOD PLAIN APARM	FLOOD PLAIN MPARM	AVERAGE TRAVEL TIME(SEC)
18	4	600.00	.02850	.050	1.469	1.500	10.497	.922	74.5
20	4	900.00	.06110	.050	2.379	1.500	5.398	1.168	110.1
22	4	700.00	.01430	.050	1.151	1.500	2.611	1.168	177.0
24	4	700.00	.08070	.050	2.204	1.500	9.451	1.117	47.4
27	3	200.00	.05000	.025	91.622	1.000			
28	4	1200.00	.01670	.050	1.125	1.500	8.035	.922	194.8
29	4	900.00	.05000	.050	2.152	1.500	4.883	1.168	121.7
31	4	300.00	.05000	.050	1.805	1.500	10.831	.988	28.9
33	4	400.00	.00390	.050	.504	1.500	3.025	.988	138.1
35	4	900.00	.00390	.050	.504	1.500	3.025	.988	310.7
37	4	50.00	.00390	.050	.504	1.500	3.025	.988	17.3

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

Table 3-4. Specifications for Storage Segments for Wild Creek

SPECIFICATIONS FOR STORAGE SEGMENT 19 (All runs but #1)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	828.000	.000	.000
.037	16.375	829.000	1.000	.074
.197	32.750	830.000	2.000	.294
.591	49.125	831.000	3.000	.413
1.037	56.725	832.000	4.000	.574
1.759	66.592	833.000	5.000	.792
2.619	79.200	834.000	6.000	1.004
3.775	184.027	835.000	7.000	1.240
4.931	288.854	.000	.000	.000

SPECIFICATIONS FOR STORAGE SEGMENT 21 (Run #4 and 5)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	853.000	.000	.000
.010	5.789	854.000	1.000	.020
.053	10.028	855.000	2.000	.080
.176	14.001	856.000	3.000	.157
.375	16.923	857.000	4.000	.257
.698	19.766	858.000	5.000	.382
1.147	22.579	859.000	6.000	.530
1.771	84.638	860.000	7.000	.717
2.591	946.312	861.000	8.000	.932

SPECIFICATIONS FOR STORAGE SEGMENT 23 (All runs but #1)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	845.000	.000	.000
.004	13.370	846.000	1.000	.009
.019	26.740	847.000	2.000	.021
.048	32.750	848.000	3.000	.037
.094	43.111	849.000	4.000	.057
.164	49.891	850.000	5.000	.083
.262	152.985	851.000	6.000	.113
.404	335.920	852.000	7.000	.186

Table 3-4. (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 25 (Run #2)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	836.000	.000	.000
.003	32.750	837.000	1.000	.006
.017	65.500	838.000	2.000	.028
.059	98.251	839.000	3.000	.050
.119	113.450	840.000	4.000	.077
.214	133.183	841.000	5.000	.107
.334	158.400	842.000	6.000	.141
.512	175.094	843.000	7.000	.222
.791	191.462	844.000	8.000	.342
1.187	207.614	845.000	9.000	.434
1.668	223.628	846.000	10.000	.550
2.281	239.559	847.000	11.000	.650
2.968	345.452	848.000	12.000	.750

SPECIFICATIONS FOR STORAGE SEGMENT 25 (Runs #3, 4, and 5)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	836.000	.000	.000
.003	5.800	837.000	1.000	.006
.017	10.000	838.000	2.000	.028
.059	14.000	839.000	3.000	.050
.119	16.900	840.000	4.000	.077
.214	19.800	841.000	5.000	.107
.334	22.600	842.000	6.000	.141
.512	24.600	843.000	7.000	.222
.791	109.100	844.000	8.000	.342
1.187	207.600	845.000	9.000	.434
1.668	223.600	846.000	10.000	.550
2.281	239.600	847.000	11.000	.650
2.968	345.500	848.000	12.000	.750

Table 3-4. (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 30 (Run #2, 3, and 4)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	833.000	.000	.000
.007	16.375	834.000	1.000	.014
.032	32.750	835.000	2.000	.041
.094	49.125	836.000	3.000	.083
.199	56.725	837.000	4.000	.126
.349	66.592	838.000	5.000	.179
.562	79.200	839.000	6.000	.249
.852	87.547	840.000	7.000	.331
1.232	95.731	841.000	8.000	.434
1.724	200.287	842.000	9.000	.551

SPECIFICATIONS FOR STORAGE SEGMENT 30 (Run #5)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	[illegible].684	833.000	.000	.000
.007	11.772	834.000	1.000	.014
.032	15.476	835.000	2.000	.041
.094	1[illegible].2[illegible]1	836.000	3.000	.083
.199	21.174	837.000	4.000	.126
.349	23.[illegible]21	838.000	5.000	.179
.562	25.632	839.000	6.000	.249
.852	27.5[illegible]3	840.000	7.000	.331
1.232	29.431	841.000	8.000	.434
1.724	127.728	842.000	9.000	.551

Table 3-4. (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 32 (All runs but #1)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	816.000	.000	.000
.006	55.599	817.000	1.000	.012
.025	111.197	818.000	2.000	.028
.066	166.796	819.000	3.000	.057
.141	192.599	820.000	4.000	.092
.277	226.099	821.000	5.000	.207
.555	268.909	822.000	6.000	.321
.920	297.249	823.000	7.000	.436
1.427	325.036	824.000	8.000	.551
2.029	352.456	825.000	9.000	.689
2.817	443.962	826.000	10.000	.861
3.604	535.468	.000	.000	.000

SPECIFICATIONS FOR STORAGE SEGMENT 36 (All runs but #1)

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	810.000	.000	.000
.011	56.725	811.000	1.000	.022
.048	113.450	812.000	2.000	.055
.127	170.175	813.000	3.000	.105
.272	226.900	814.000	4.000	.198
.521	253.682	815.000	5.000	.285
.847	277.895	816.000	6.000	.386
1.302	322.673	817.000	7.000	.514
1.955	365.809	818.000	8.000	.882
3.090	394.806	819.000	9.000	1.322
4.625	423.338	820.000	10.000	1.837
6.807	451.526	821.000	11.000	2.479
9.615	479.463	822.000	12.000	3.214
13.183	668.022	823.000	13.000	3.788

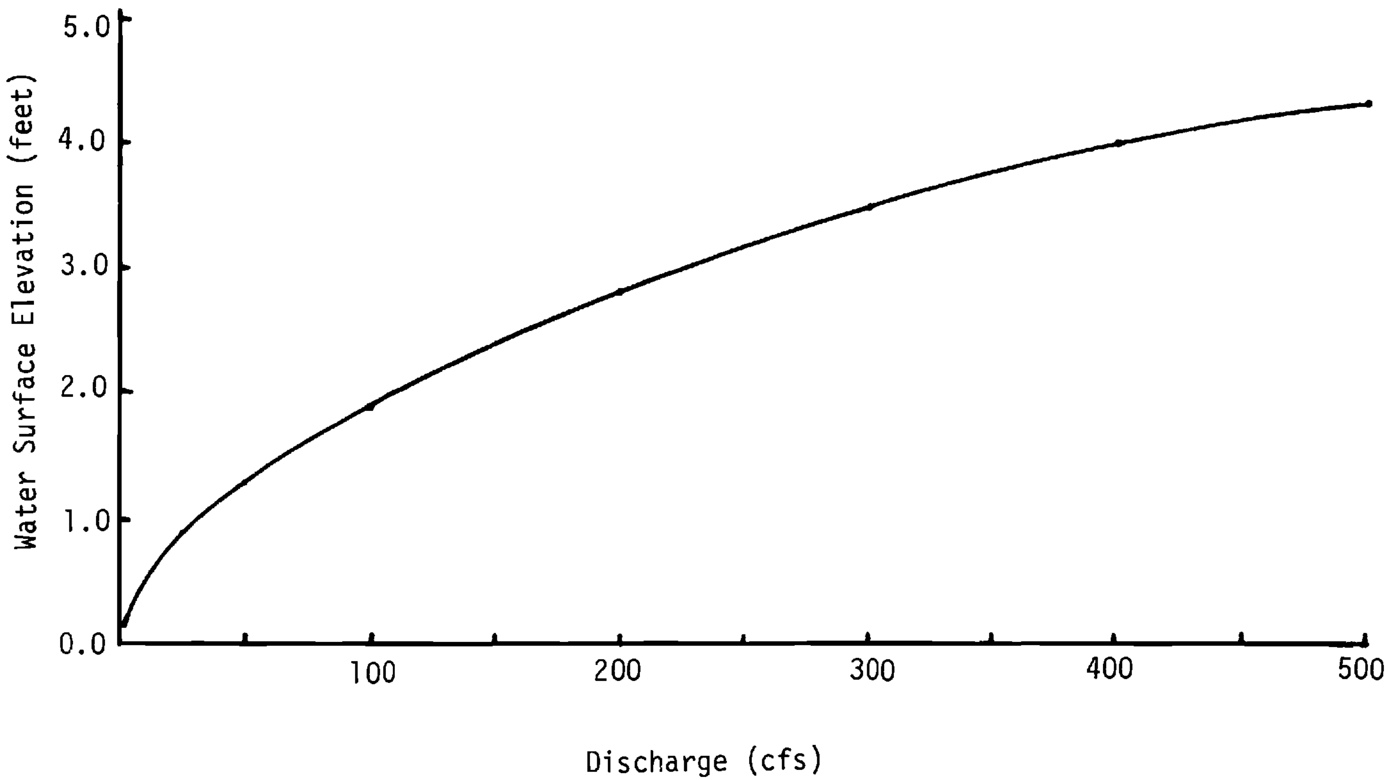


Figure 3-3. Rating Curve for Section 28 Wild Creek

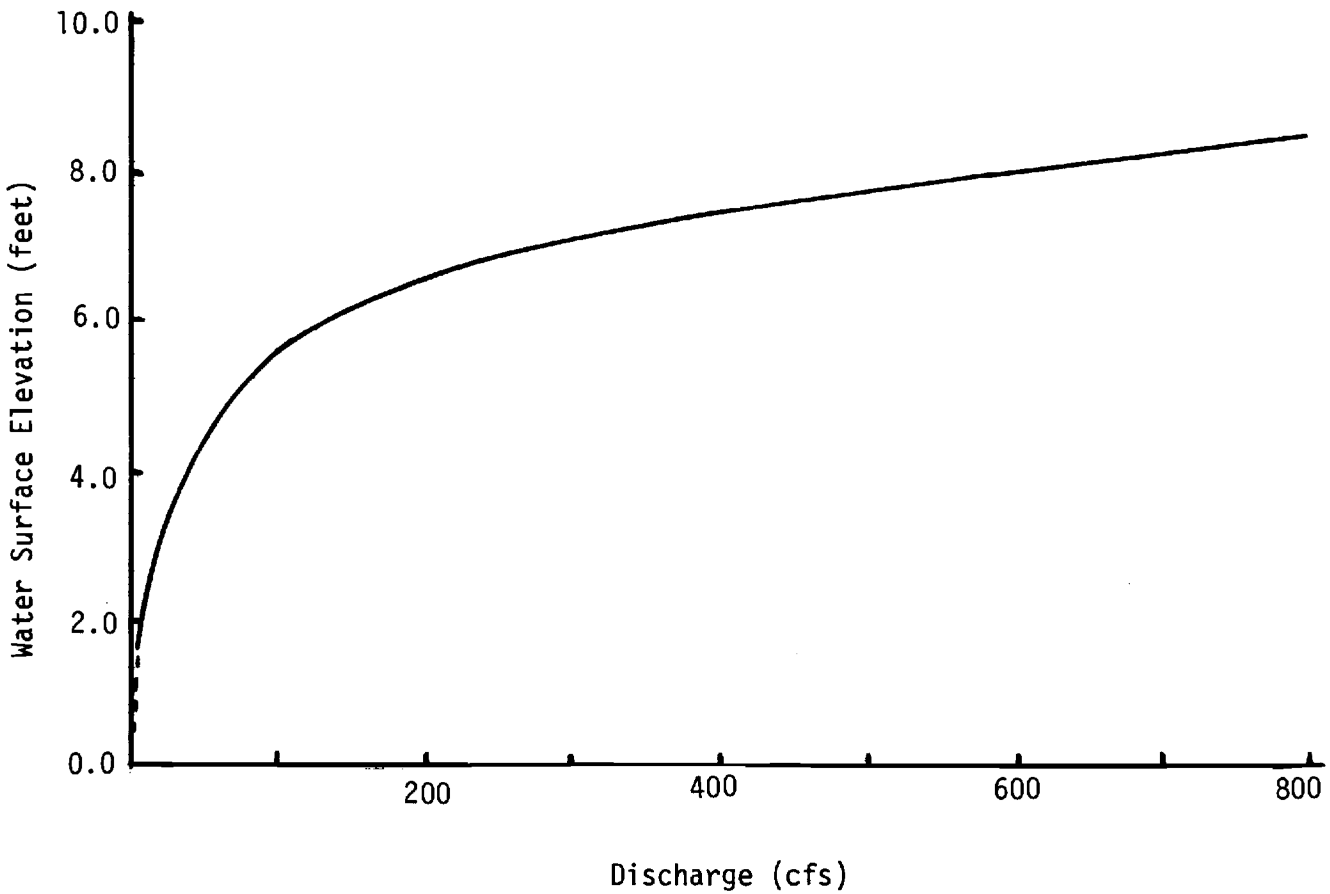


Figure 3-4. Rating Curve for Section 35 of Wild Creek

Table 3-5. Run 1 Annual Peak Flows and Frequency Analysis (Segment 28)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 28 FOR WILD CREEK - NATURAL

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	11.7	1918	APR.	7
1919	88.3	1919	MAY	6
1920	233.8	1920	MAR.	12
1921	234.3	1921	AUG.	24
1922	96.9	1922	MAR.	10
1923	61.8	1923	MAR.	12
1924	58.3	1924	MAY	27
1925	46.4	1925	JAN.	17
1926	219.9	1926	AUG.	11
1927	65.3	1927	FEB.	23
1928	268.7	1928	JULY	10
1929	106.0	1929	MAR.	14
1930	34.8	1930	JAN.	28
1931	15.7	1930	NOV.	16
1932	91.9	1932	JUNE	18
1933	73.5	1933	JUNE	10
1934	62.6	1934	JULY	19
1935	19.3	1935	MAR.	5
1936	111.6	1936	FEB.	3
1937	114.5	1937	JUNE	17
1938	156.5	1938	JUNE	25
1939	73.9	1939	JUNE	22
1940	142.0	1940	SEPT	10
1941	42.5	1941	AUG.	13
1942	125.2	1942	MAR.	20

MEAN = 102.2 STANDARD DEVIATION = 71.8

FLOOD FREQUENCY FOR FLOWPOINT 28 FOR WILD CREEK - NATURAL

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	90.4	1.8
5-YEAR	20.0	153.9	2.4
10-YEAR	10.0	195.9	2.8
25-YEAR	4.0	249.0	3.2
50-YEAR	2.0	288.4	3.4
100-YEAR	1.0	327.5	3.7
200-YEAR	0.5	366.5	3.8

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 3-6. Run 1 Annual Peak Flows and Frequency Analysis (Segment 35)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 35 FOR WILD CREEK - NATURAL

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	22.9	1918	APR.	7
1919	175.2	1919	MAY	6
1920	443.2	1920	MAR.	12
1921	434.7	1921	AUG.	24
1922	192.5	1922	MAR.	10
1923	119.8	1923	MAR.	12
1924	112.8	1924	MAY	27
1925	94.8	1925	JAN.	17
1926	431.7	1926	AUG.	11
1927	129.5	1927	FEB.	23
1928	536.0	1928	JULY	10
1929	207.1	1929	MAR.	14
1930	66.6	1930	JAN.	28
1931	29.5	1930	NOV.	16
1932	184.5	1932	JUNE	18
1933	141.6	1933	JUNE	10
1934	118.6	1934	JULY	19
1935	39.5	1935	MAR.	5
1936	214.1	1936	FEB.	3
1937	218.5	1937	JUNE	17
1938	283.7	1938	JUNE	25
1939	149.0	1939	JUNE	22
1940	262.2	1940	SEPT	10
1941	81.3	1941	AUG.	13
1942	249.5	1942	MAR.	20

MEAN = 197.5 STANDARD DEVIATION = 137.9

FLOOD FREQUENCY FOR FLOWPOINT 35 FOR WILD CREEK - NATURAL

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	174.9	6.3
5-YEAR	20.0	296.7	7.0
10-YEAR	10.0	377.4	7.3
25-YEAR	4.0	479.3	7.6
50-YEAR	2.0	555.0	7.8
100-YEAR	1.0	630.0	.0
200-YEAR	0.5	704.8	.0

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Run #2. This simulation was based on the conditions which existed in the watershed before the recent construction of the detention structure near the intersection of Wild Creek Trail and Brook Forest Dr. Instead of this structure, a 3' by 6' elliptical culvert was used at storage segment 25 because of information that this was the original structure at this point. The results of this run are shown in Tables 3- 7 and 3- 8. For reach 28 the mean annual peak flow was 138.6 cfs with a standard deviation of 78.5. This mean leads to a 2-year flood of 126 cfs (elevation 2.2 feet) and a 100-year flood Of 385 cfs (elevation 3.9 feet). The mean annual peak flow in Reach 35 was 229.6 cfs with a standard deviation of 121.2. From this mean a 2-year flood of 210 cfs (elevation 6.6 feet) and a 100-year flood of 610 cfs (elevation 7.9 feet) were determined.

Run #3. This simulation was based on the current watershed conditions including the detention structure. Tables 3- 9 and 3-10 show peak flows and frequencies from this run. In reach 28 the mean annual peak flow was 131.4 cfs with a standard deviation of 81.2 cfs. The resulting 2-year flow is 118 cfs (elevation 2.1 feet), and the resulting 100-year flow is 386 cfs (elevation 3.9 feet). Downstream in Reach 35, the mean annual peak flow was 222.1 cfs with a standard deviation of 120.9. The 2-year flood is 202 cfs (elevation 6.5 feet), and the 100-year flood is 601 cfs (elevation 7.9 feet).

Table 3-7. Run 2 Annual Peak Flows and Frequency Analysis (Segment 28)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 28 FOR WILD CREEK - NO STORAGE EXCEPT EXISTING CULVERTS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	29.9	1918	APR.	7
1919	121.5	1919	MAY	6
1920	264.2	1920	MAR.	12
1921	281.4	1921	AUG.	24
1922	109.5	1922	MAR.	10
1923	81.2	1923	MAR.	12
1924	92.9	1924	MAY	27
1925	79.8	1924	DEC.	8
1926	265.6	1926	AUG.	11
1927	76.3	1927	FEB.	23
1928	292.1	1928	JULY	10
1929	120.1	1929	MAR.	14
1930	49.1	1930	JAN.	28
1931	55.7	1931	JULY	28
1932	122.8	1932	JUNE	18
1933	166.1	1933	JUNE	10
1934	139.0	1934	JULY	19
1935	31.3	1935	MAR.	5
1936	127.5	1936	SEPT	29
1937	158.4	1937	JUNE	17
1938	237.4	1938	JUNE	25
1939	101.5	1939	JUNE	22
1940	216.1	1940	SEPT	10
1941	110.5	1941	AUG.	13
1942	134.7	1942	MAR.	20

MEAN = 138.6 STANDARD DEVIATION = 78.4

FLOOD FREQUENCY FOR FLOWPOINT 28 FOR WILD CREEK - NO STORAGE EXCEPT EXISTING CULVERTS

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	125.7	2.2
5-YEAR	20.0	195.0	2.8
10-YEAR	10.0	240.9	3.1
25-YEAR	4.0	298.9	3.5
50-YEAR	2.0	341.9	3.7
100-YEAR	1.0	384.6	3.9
200-YEAR	0.5	427.2	4.1

* PERCENT CHANCE IN <u>ANY</u> YEAR OF GETTING A FLOOD <u>GREATER</u> THAN THAT INDICATED IN THE TABLE.

Table 3-8. Run 2 Annual Peak Flows and Frequency Analysis (Segment 35)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 35 FOR WILD CREEK - NO STORAGE EXCEPT EXISTING CULVERTS

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	55.8	1918	APR.	7
1919	194.7	1919	MAY	6
1920	410.1	1920	AUG.	15
1921	433.9	1921	AUG.	24
1922	197.1	1922	MAR.	10
1923	138.7	1923	MAR.	12
1924	155.3	1924	MAY	27
1925	134.9	1924	DEC.	8
1926	438.9	1926	AUG.	11
1927	134.8	1927	FEB.	23
1928	488.9	1928	JULY	10
1929	213.5	1929	MAR.	14
1930	86.3	1930	JAN.	28
1931	96.7	1931	JULY	28
1932	226.0	1932	JUNE	18
1933	260.8	1933	JUNE	10
1934	211.8	1934	JULY	19
1935	59.9	1935	MAR.	5
1936	216.4	1936	FEB.	3
1937	253.1	1937	JUNE	17
1938	361.8	1938	JUNE	25
1939	182.2	1939	JUNE	22
1940	342.0	1940	SEPT	10
1941	199.0	1941	AUG.	13
1942	248.0	1942	MAR.	20

MEAN = 229.6 STANDARD DEVIATION = 121.2

FLOOD FREQUENCY FOR FLOWPOINT 35 FOR WILD CREEK - NO STORAGE EXCEPT EXISTING CULVERTS

RETURN PERIOD	PROBABILITY *	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	209.7	6.6
5-YEAR	20.0	316.8	7.1
10-YEAR	10.0	387.7	7.4
25-YEAR	4.0	477.3	7.6
50-YEAR	2.0	543.8	7.8
100-YEAR	1.0	609.7	7.9
200-YEAR	0.5	675.5	8.1

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 3-9. Run 3 Annual Peak Flows and Frequency Analysis (Segment 28)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 28 FOR WILD CREEK - STORAGE DEVICE AT 25

WATER YEAR	PEAK FLOW (CFS)	YEAR	MONTH	DAY
1918	29.6	1918	APR.	7
1919	99.2	1919	MAY	6
1920	269.3	1920	MAR.	12
1921	279.7	1921	AUG.	24
1922	191.3	1922	MAR.	10
1923	67.9	1923	MAR.	12
1924	76.4	1924	MAY	27
1925	69.4	1924	DEC.	8
1926	265.1	1926	AUG.	11
1927	66.1	1927	FEB.	23
1928	299.0	1928	JULY	10
1929	199.3	1929	MAR.	14
1930	43.9	1930	JAN.	28
1931	51.0	1931	JULY	28
1932	119.2	1932	JUNE	18
1933	157.4	1933	JUNE	10
1934	115.9	1934	JULY	19
1935	30.7	1935	MAR.	5
1936	116.4	1936	FEB.	3
1937	149.2	1937	JUNE	17
1938	249.9	1938	JUNE	25
1939	92.9	1939	JUNE	22
1940	217.2	1940	SEPT	10
1941	193.8	1941	AUG.	13
1942	133.7	1942	MAR.	20

MEAN = 131.4 STANDARD DEVIATION = 81.2

FLOOD FREQUENCY FOR FLOWPOINT 28 FOR WILD CREEK - STORAGE DEVICE AT 25

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	118.1	2.1
5-YEAR	20.0	189.9	2.7
10-YEAR	10.0	237.4	3.1
25-YEAR	4.0	297.4	3.5
50-YEAR	2.0	342.0	3.7
100-YEAR	1.0	386.2	3.9
200-YEAR	0.5	430.2	4.1

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 3-10. Run 3 Annual Peak Flows and Frequency Analysis (Segment 35)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 35 FOR WILD CREEK - STORAGE DEVICE AT 25

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	55.2	1918	APR.	7
1919	179.2	1919	MAY	6
1920	404.3	1920	AUG.	15
1921	427.1	1921	AUG.	24
1922	186.8	1922	MAR.	10
1923	130.0	1923	MAR.	12
1924	145.2	1924	MAY	27
1925	127.2	1924	DEC.	8
1926	434.7	1926	AUG.	11
1927	122.3	1927	FEB.	23
1928	484.5	1928	JULY	10
1929	205.6	1929	MAR.	14
1930	83.3	1930	JAN.	28
1931	91.1	1931	JULY	28
1932	219.9	1932	JUNE	18
1933	259.3	1933	JUNE	10
1934	195.5	1934	JULY	19
1935	59.3	1935	MAR.	5
1936	206.8	1936	FEB.	3
1937	240.7	1937	JUNE	17
1938	354.2	1938	JUNE	25
1939	139.6	1939	JUNE	22
1940	332.5	1940	SEPT	10
1941	186.3	1941	AUG.	13
1942	242.1	1942	MAR.	20

MEAN = 222.1   STANDARD DEVIATION = 120.9

FLOOD FREQUENCY FOR FLOWPOINT 35 FOR WILD CREEK - STORAGE DEVICE AT 25

RETURN PERIOD	PROBABILITY *	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	202.2	6.5
5-YEAR	20.0	309.1	7.0
10-YEAR	10.0	373.8	7.3
25-YEAR	4.0	469.2	7.6
50-YEAR	2.0	535.5	7.8
100-YEAR	1.0	601.3	7.9
200-YEAR	0.5	666.8	8.1

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Run #4. This simulation was made to evaluate the effect of placing an embankment across the outlet of channel segment 21 in order to provide detention storage. Outlet structures include an 18" circular culvert at the base and a 20' wide spillway at an elevation 6 feet above the channel bottom. Tables 3-11 and 3-12 show results of this run. Segment 28 has a mean annual peak flow of 113.2 cfs with a standard deviation of 70.7. The corresponding 2-year flood is 102 cfs (elevation 1.9 feet), and the 100-year flood crests at 335 cfs (elevation 3.7 feet). In segment 35, the mean annual peak flow is 208.5 cfs with a standard deviation of 113.9. This leads to a 2-year flood Of 190 cfs (elevation 6.4 feet) and a 100-year flood of 566 cfs (elevation 7.8 feet).

Run #5. In this simulation the hydrologic effects were investigated of replacng a 36" circular culvert at Beach Haven Road with a 18" circular culvert. Tables 3-13 and 3-14 show the results. Since segment 22 is upstream, it is not affected. For reach 35, the mean annual peak flow is 215.0 cfs with a standard deviation of 113.9. This leads to a 2-year flood of 196 cfs (elevation 6.5 feet) and a 100-year flood of 572.4 cfs (elevation 7.8 feet).

V. Summary and Conclusions

Tables 3-15 and 3-16 summarize the results of all five simulation runs. They also show the percentage change in flow from prior conditions.

It can be seen from Table 3-15 that with development.

Table 3-11. Run 4 Annual Peak Flows and Frequency Analysis (Segment 28)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 28 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	27.8	1918	APR.	7
1919	82.9	1919	MAY	6
1920	222.3	1920	MAR.	12
1921	240.2	1921	AUG.	24
1922	88.7	1922	MAR.	10
1923	58.0	1923	MAR.	12
1924	65.0	1924	MAY	27
1925	56.3	1924	DEC.	8
1926	240.9	1926	AUG.	11
1927	58.8	1927	FEB.	23
1928	269.8	1928	JULY	10
1929	94.3	1929	MAR.	14
1930	39.7	1930	JAN.	28
1931	43.4	1931	JULY	28
1932	104.5	1932	JUNE	18
1933	124.5	1933	JUNE	10
1934	93.6	1934	JULY	19
1935	30.4	1935	MAR.	5
1936	98.9	1936	FEB.	3
1937	124.2	1937	JUNE	17
1938	196.0	1938	JUNE	25
1939	86.4	1939	JUNE	22
1940	178.4	1940	SEPT	10
1941	88.8	1941	AUG.	13
1942	115.3	1942	MAR.	20

MEAN = 113.2 STANDARD DEVIATION = 70.7

FLOOD FREQUENCY FOR FLOWPOINT 28 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	101.5	1.9
5-YEAR	20.0	164.1	2.5
10-YEAR	10.0	205.4	2.9
25-YEAR	4.0	257.7	3.2
50-YEAR	2.0	296.5	3.5
100-YEAR	1.0	335.0	3.7
200-YEAR	0.5	373.4	3.9

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 3-12. Run 4 Annual Peak Flows and Frequency Analysis (Segment 35)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 35 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	54.5	1918	APR.	7
1919	168.3	1919	MAY	6
1920	381.2	1920	AUG.	15
1921	401.2	1921	AUG.	24
1922	175.8	1922	MAR.	10
1923	122.6	1923	MAR.	12
1924	138.1	1924	MAY	27
1925	118.5	1924	DEC.	8
1926	414.1	1926	AUG.	11
1927	124.0	1927	FEB.	23
1928	462.1	1928	JULY	10
1929	191.8	1929	MAR.	14
1930	80.5	1930	JAN.	28
1931	83.8	1931	JULY	28
1932	207.2	1932	JUNE	18
1933	225.6	1933	JUNE	10
1934	179.1	1934	JULY	19
1935	58.6	1935	MAR.	5
1936	191.9	1936	FEB.	3
1937	221.3	1937	JUNE	17
1938	329.9	1938	JUNE	25
1939	174.2	1939	JUNE	22
1940	309.2	1940	SEPT	10
1941	174.0	1941	AUG.	13
1942	226.1	1942	MAR.	20

MEAN = 208.5 STANDARD DEVIATION = 113.9

FLOOD FREQUENCY FOR FLOWPOINT 35 FOR WILD CREEK - STORAGE DEVICE AT 21 AND 25

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	189.8	6.4
5-YEAR	20.0	290.5	6.9
10-YEAR	10.0	357.1	7.2
25-YEAR	4.0	441.3	7.5
50-YEAR	2.0	505.8	7.7
100-YEAR	1.0	565.8	7.8
200-YEAR	0.5	627.6	8.0

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 3-13. Run 5 Annual Peak Flows and Frequency Analysis (Segment 28)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 28 FOR WILD CREEK - STORAGE DEVICE AT 25 AND 30

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	28.6	1918	APR.	7
1919	99.8	1919	MAY	6
1920	262.3	1920	MAR.	12
1921	278.7	1921	AUG.	24
1922	101.8	1922	MAR.	10
1923	67.9	1923	MAR.	12
1924	76.4	1924	MAY	27
1925	69.4	1924	DEC.	8
1926	265.1	1926	AUG.	11
1927	66.1	1927	FEB.	23
1928	292.9	1928	JULY	10
1929	109.8	1929	MAR.	14
1930	43.9	1930	JAN.	28
1931	51.0	1931	JULY	28
1932	119.2	1932	JUNE	18
1933	153.4	1933	JUNE	10
1934	115.9	1934	JULY	19
1935	30.7	1935	MAR.	5
1936	116.4	1936	FEB.	3
1937	149.2	1937	JUNE	17
1938	242.9	1938	JUNE	25
1939	92.9	1939	JUNE	22
1940	217.2	1940	SEPT	10
1941	103.8	1941	AUG.	13
1942	133.7	1942	MAR.	20

MEAN = 131.4 STANDARD DEVIATION = 81.2

FLOOD FREQUENCY FOR FLOWPOINT 28 FOR WILD CREEK - STORAGE DEVICE AT 25 AND 30

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	118.1	2.1
5-YEAR	20.0	189.9	2.7
10-YEAR	10.0	237.4	3.1
25-YEAR	4.0	297.4	3.5
50-YEAR	2.0	342.0	3.7
100-YEAR	1.0	386.2	3.9
200-YEAR	0.5	430.2	4.1

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 3-14. Run 5 Annual Peak Flows and Frequency Analysis (Segment 35)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 35 FOR WILD CREEK - STORAGE DEVICE AT 25 AND 30

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	55.2	1918	APR.	7
1919	173.5	1919	MAY	6
1920	384.7	1920	AUG.	15
1921	404.4	1921	AUG.	24
1922	184.7	1922	MAR.	10
1923	127.4	1923	MAR.	12
1924	144.1	1924	MAY	27
1925	128.1	1924	DEC.	8
1926	415.5	1926	AUG.	11
1927	126.7	1927	FEB.	23
1928	468.4	1928	JULY	10
1929	202.0	1929	MAR.	14
1930	84.0	1930	JAN.	28
1931	91.0	1931	JULY	28
1932	213.9	1932	JUNE	18
1933	240.9	1933	JUNE	10
1934	188.1	1934	JULY	19
1935	59.2	1935	MAR.	5
1936	200.6	1936	FEB.	3
1937	230.9	1937	JUNE	17
1938	335.4	1938	JUNE	25
1939	179.4	1939	JUNE	22
1940	316.2	1940	SEPT	10
1941	184.6	1941	AUG.	13
1942	236.4	1942	MAR.	20

MEAN = 215.0   STANDARD DEVIATION = 113.9

FLOOD FREQUENCY FOR FLOWPOINT 35 FOR WILD CREEK - STORAGE DEVICE AT 25 AND 30

RETURN PERIOD	PROBABILITY*	FLOW IN CFS	W S ELEV IN FT
2-YEAR	50.0	196.3	6.5
5-YEAR	20.0	297.0	7.0
10-YEAR	10.0	363.7	7.3
25-YEAR	4.0	447.9	7.5
50-YEAR	2.0	510.4	7.7
100-YEAR	1.0	572.4	7.8
200-YEAR	0.5	634.2	8.0

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 3-15. Summary Table for Segment 28

	Run #1 Natural Watershed	Run #2 Existing Land-Use No Detention	Run #3 Existing Land-Use Existing Detention	Run #4 Existing Land-Use Added Detention 1
Mean Flow (cfs)	102.2	138.6	131.4	113.2
Standard Deviation	71.8	78.4	81.2	70.7
Flow Changes (%)				
from Run #1	-	+36	+29	+11
from Run #2	-	-	-5	-18
from Run #3	-	-	-	-14
2-year flood				
Flow (cfs)	90.4	125.7	118.1	101.5
Elevation (ft)	1.8	2.2	2.1	1.9
Flow changes (%)				
from Run #1	-	+39	+31	+12
from Run #2	-	-	-6	-19
from Run #3	-	-	-	-14
100-year flood				
Flow (cfs)	327.5	389.6	386.2	335.0
Elevation (ft)	3.7	3.9	3.9	3.7
Flow changes (%)				
from Run #1	-	+17	+18	+2
from Run #2	-	-	0	-13
from Run #3	-	-	-	-13

Note: The detention storage in Run 5 is downstream from this site and hence does not affect flows in channel segment 28.

Table 3-16. Summary Table for Segment 35

	Run #1 Natural Watershed	Run #2 Existing Land-Use No Detention	Run #3 Existing Land-Use Existing Detention	Run #4 Existing Land-Use Added Detention 1	Run #5 Existing Land-Use Added Detention 2
Mean Flow (cfs)	197.5	229.6	222.1	208.5	215.0
Standard Deviation	137.9	121.2	120.9	113.9	113.9
Flow Changes (%)					
from Run #1	-	+16	+12	+6	+9
from Run #2	-	-	-3	-9	-6
from Run #3	-	-	-	-6	-3
2-year Flood					
Flow (cfs)	174.9	209.7	202.2	189.8	196.3
Elevation (ft)	6.3	6.6	6.5	6.4	6.5
Flow Changes (%)					
from Run #1	-	+20	+16	+9	+12
from Run #2	-	-	-4	-9	-6
from Run #3	-	-	-	-6	-3
100-year Flood					
Flow (cfs)	630.0	609.7	601.3	565.8	572.4
Elevation (ft)	-	7.9	7.9	7.8	7.8
Flow changes (%)					
from Run #1	-	-3	-5	-10	-9
from Run #2	-	-	-1	-7	-6
from Run #3	-	-	-	-6	-5

maller peaks were increased the most; the 2-year flood increased 39% while the 100-year flood increased only 17%. The recently completed detention device at segment 25 has been helpful in reach 28; the mean annual peak flow has decreased 5% over the developed condition prior to construction of the detention device. Again low flows were affected most. The 2-year flood is 6% less while the 100-year flood remained constant. The dam at segment 21 would reduce flows further; the mean would decrease 14% from the present and the 2- and 100-year floods would decrease a similar amount. It should be noted, however, that flood elevations have remained fairly constant through all these changes. The 39% increase in the 2-year flood flow caused by development caused only a 0.4' increase in the flood elevation. Measures to reduce flows have had a similarly small effect on flood level reductions.

Table 3-16 shows the effects of watershed changes on reach 35. In this channel, development caused a 16% increase in mean annual peak flow. Again the smaller peaks were effected most. The 2-year flood is 20% greater for the developed watershed than the natural, while the 100-year is 3% less. The storage device at segment 25 has less effect in this reach than it had in reach 28. Here the mean annual peak flow has decreased 3% over the developed conditions prior to construction of the detention device. The dam at 21 would decrease the flow 3%. Again it should be noted that flood elevations are not at all sensitive to these changes. Development only increased the 2-year flood level 0.3 feet, and the most effective al-alternative only decreases the flood elevation by 0.2 feet.

From this study the following conclusions may be drawn:

a) Development has increased flood flows in the Wild Creek watershed, particularly the smaller floods.

b) The increase in flood elevations due to the increased flows has been small.

c) Attempts to slow and detain water upstream do not substantially reduce flood flows or flood elevations.

One solution to the particularly severe problem in reach 35 would be to relocate the 4 houses on Brook Valley Lane particularly affected by the flooding. Though not simulated, improvement of the channel below Sheridan Drive and enlargement of the culvert under the driveway to Lanier Electronics may provide some relief to the four houses and not create greater problems downstream.

SECTION 3

Honey Creek

I. Physical Attributes of Watershed

a. Location

The portion of Honey Creek watershed included in this study is the upper headwater area. The stream rises in the city of Lithonia in southeast DeKalb County and flows generally southeast. About one fourth of the city of Lithonia drains into the creek. Even though the stream reach studied lies within DeKalb County, a small tributary of the Creek included in the study drains from Rockdale County.

b. Size

The watershed studied has a total area of 1444 acres. About 120 acres are located within the city limits of Lithonia, and about 70 acres are located in Rockdale County. Three roads cross the creek within the study limits. Table 4-1 shows drainage areas and size of culverts at these crossings.

Table 4-1

DRAINAGE AREAS AT ROAD CROSSINGS

Road Crossing	Drainage Area (Acres)	Bridge Opening
Covington Highway	144	5' x 5' Box Culvert
Interstate Route 20	194	5.5' x 7' Box Culvert
Turner Hill Road	804	Three, 6' x 10' Box Culverts

c. General Drainage Patterns

As can be seen on Figure 4-1, most of the watershed is presently

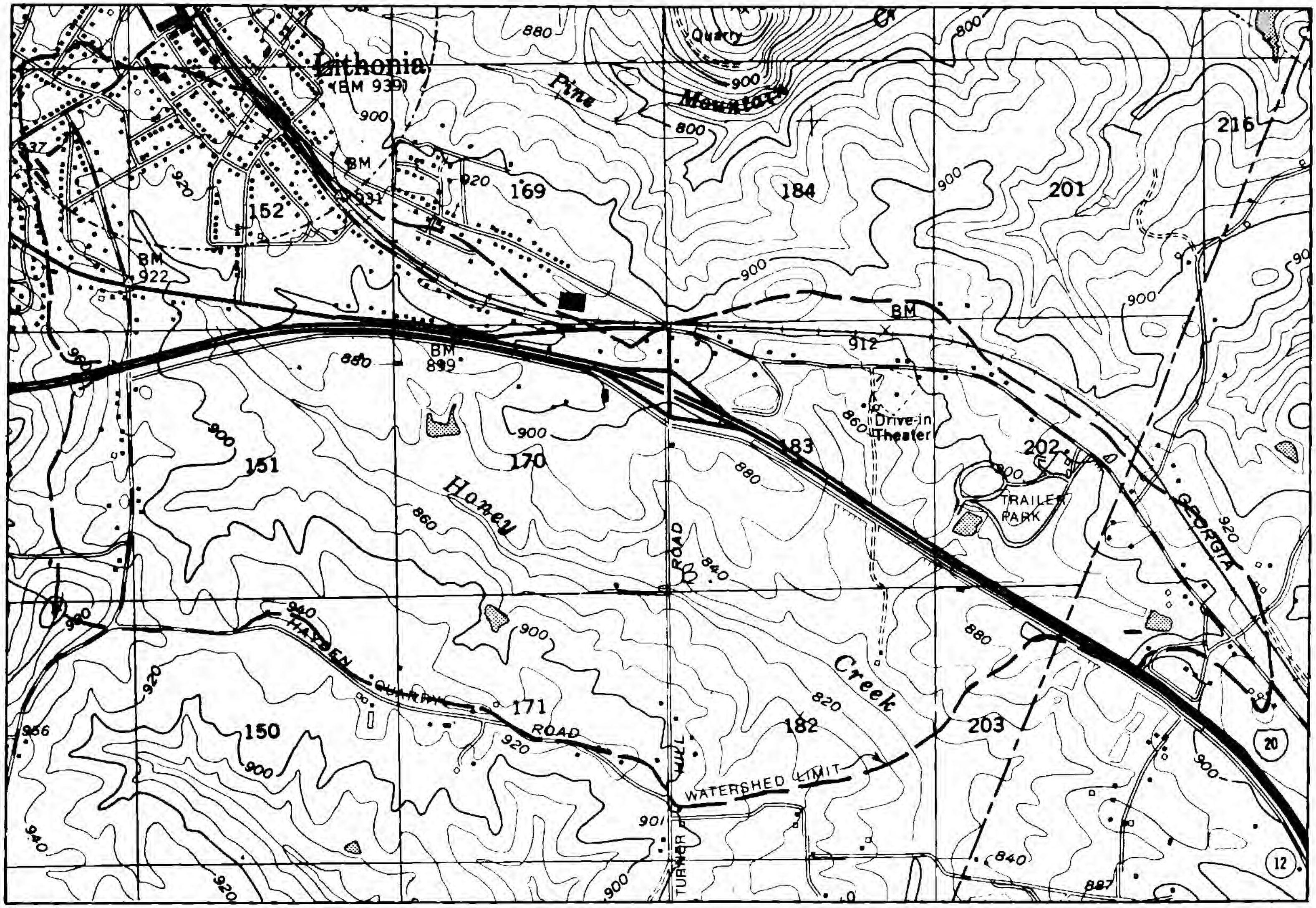


Figure 4-1. Topographic Map of Honey Creek

undeveloped with the exception of the portion in and near the city of Lithonia. Drainage for the most part is in natural channels. The main stream channel varies in width from about 20 feet at the lower end of the study to about 5 feet in the vicinity of I-20. The main stream channel has a length of about 9,800 feet from Covington Highway to the lower study limits.

d) Other Studies

The U.S. Army Corps of Engineers completed a Flood Plain Information Report on Honey Creek in April, 1974. The study limits extended to the Interstate Highway 20 crossing. The study was based on existing conditions in the watershed. Table 4-2 shows a comparison of the peak flows for the 100-year flood under existing conditions developed by this study and those developed by the Corps for the Honey Creek report.

Table 4-2

COMPARISON OF PEAK FLOWS WITH CORPS REPORT

Location	100-year flood discharge (cfs)	
	This Study	Corps Study
Turner Hill Road	1190	1800
Hayden Quarry Road	2195[1]	2340

e) Current Land Use

The portion of the watershed within the city limits of Lithonia

1 - At end of reach 76 upsteam of Hayden Quarry Road

has been almost completely developed. Most of the development is single family residences with one small development of multi-family residences in the extreme northwest portion of the watershed. The only other significant development is a large mobile home park in the area above I-20 and just west of the DeKalb-Rockdale County Line. The remainder of the watershed is rural with a few scattered homes. Klondike and Turner Hill Roads traverse the watershed from north to south. Interstate Highway 20 cuts across the middle of the watershed from east to west. Hayden Quarry Road borders the watershed on the south, and Georgia railroad borders it on the north.

f. Projected Land Use

The current projection is for rapid development in the Honey Creek Basin with most of the new development expected to be single family residential. It is reasonable to expect, however, that multi-family residential and commercial development will occur along I-20. The "Windswept" development project is currently being planned to convert six land lots to single family residential development. The planned installation of sewage treatment facilities serving the area is expected to accelerate development in the area.

II. Map of Watershed

Figure 4-1 shows the watershed on a topographic map, and Figure 4-2 shows the area, channel, and storage segments used in flow simulation. The watershed was divided into subareas and the drainageways were divided into channel segments. Existing small lakes provide some storage at locations 60 and 66. The culvert at Covington

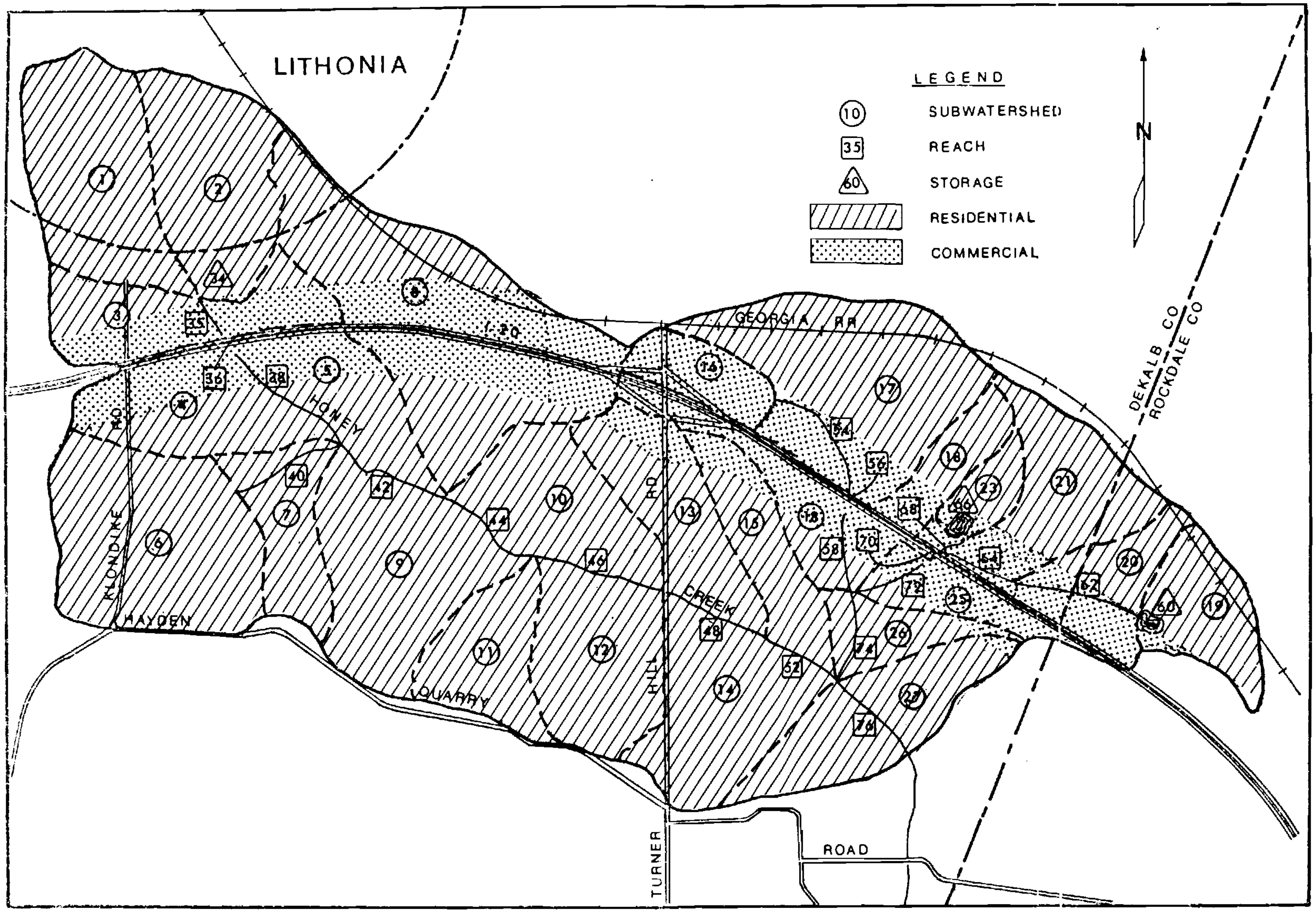


Figure 4-2. Map of Honey Creek for Run 2

Highway is small enough to create some backwater storage effect. The other road crossings are large enough so as not to have significant backwater storage effects. A schematic diagram of the modeling segments is shown on Figure 4-3.

III. Drainage Problems

At present there are no significant drainage problems within the reach studied. The purpose of this study was largely to demonstrate what could be done in arranging development patterns and providing flood control structures to keep such problems from occurring in newly developing areas.

In order to evaluate this situation, flows were simulated for existing land-use on the watershed as well as for five alternative future land development possibilities. The six runs were as follows:

1. Current land use and channel conditions
2. Complete development of the watershed as shown on Figure 4-2.
3. Complete development of the lower half of the watershed with the upper half remaining as presently developed (Figure 4-4).
4. Complete development of the upper half of the watershed with the lower half remaining as presently developed (Figure 4-5).
5. Complete development of the watershed as in run 2 but with detention storage located on a tributary at the upper end of reach 74.
6. Complete development of land lots 151, 170, 171, 182, 183, and 203. These are land lots within the watershed that will be developed by the proposed "Windswept" development. This scheme is shown on Figure 4-6.

Total residential development was simulated under scheme Number 6. For the other development schemes, residential development was

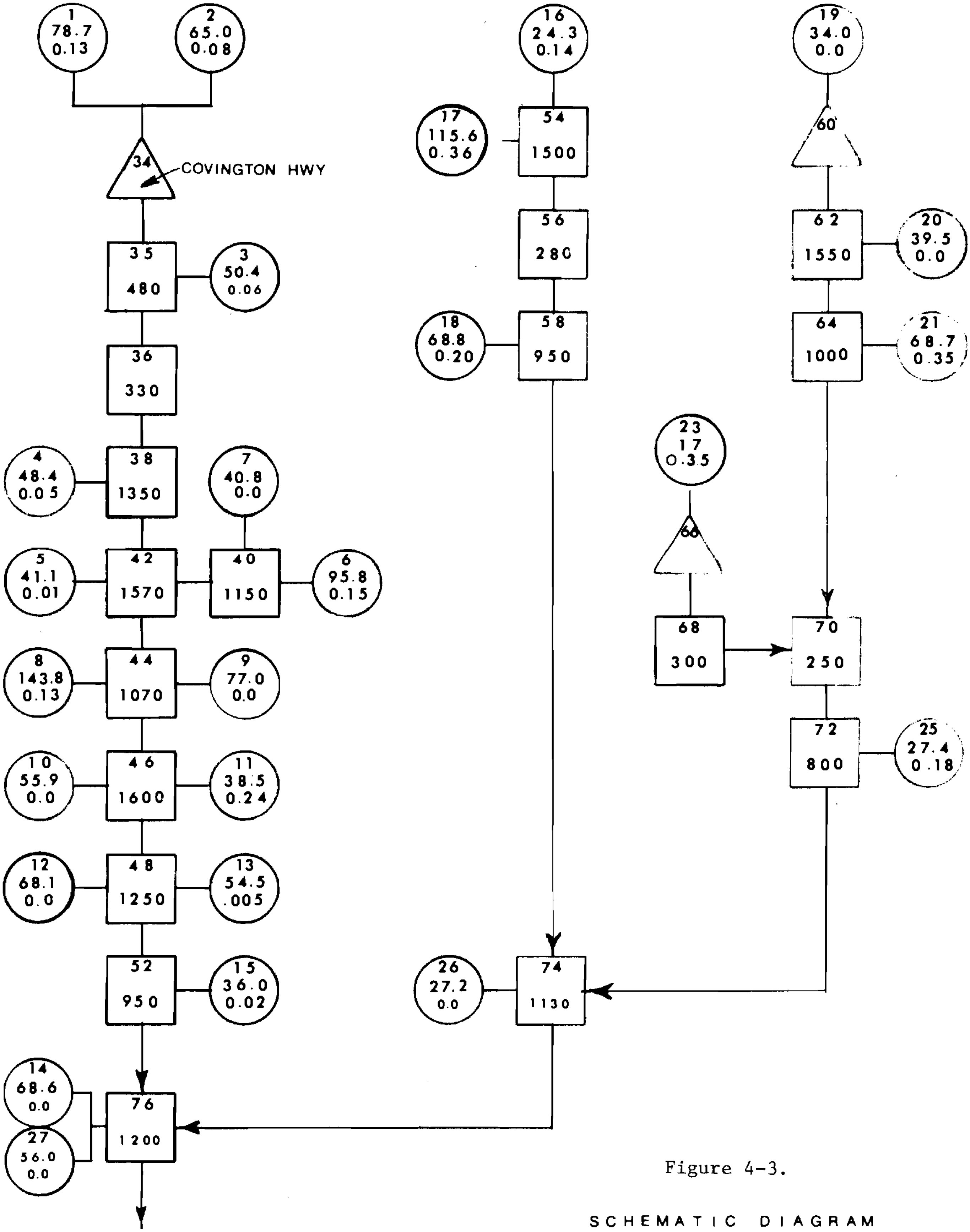


Figure 4-3.

SCHEMATIC DIAGRAM
HONEY CREEK

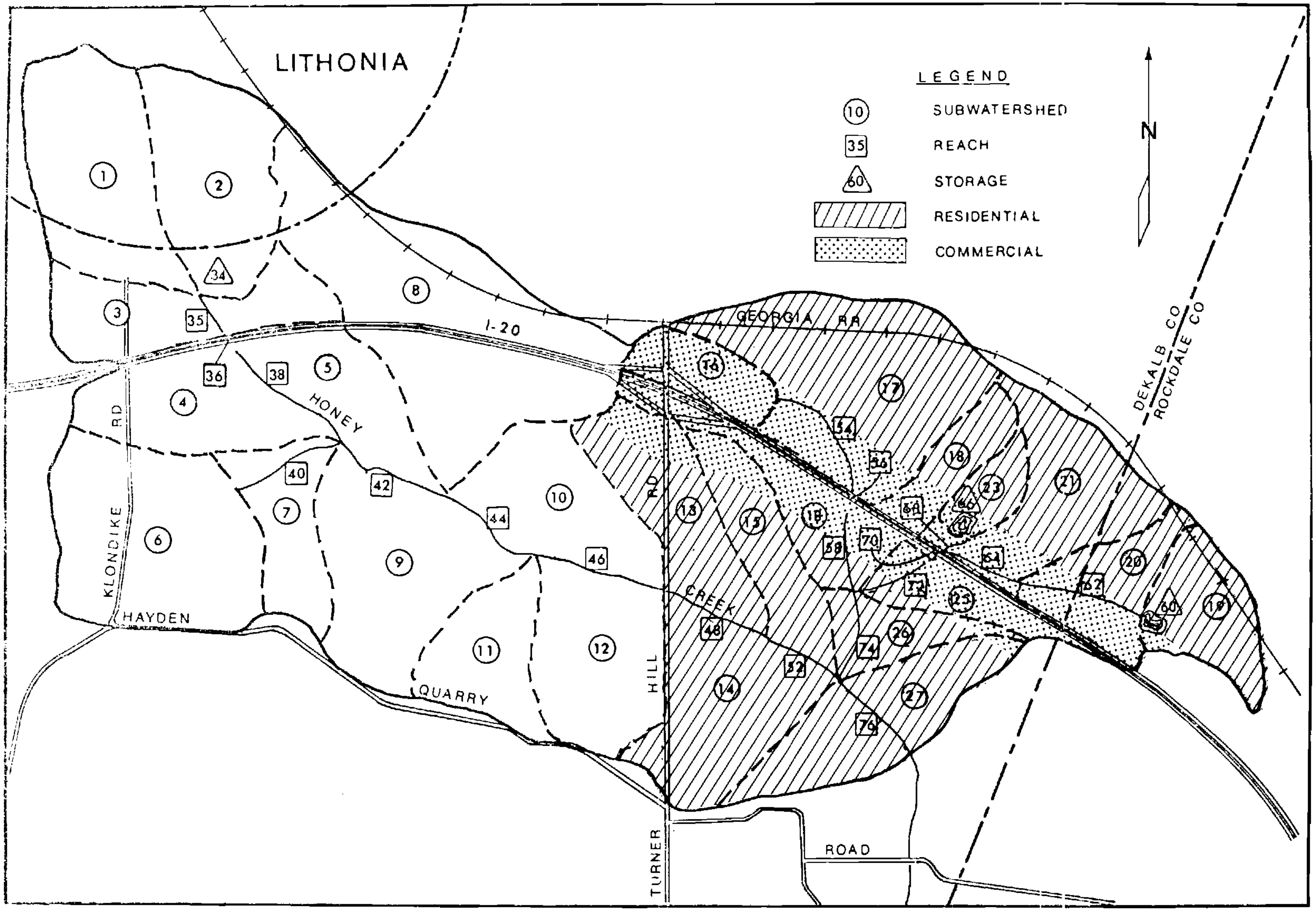


Figure 4-4. Map of Honey Creek for Run 3

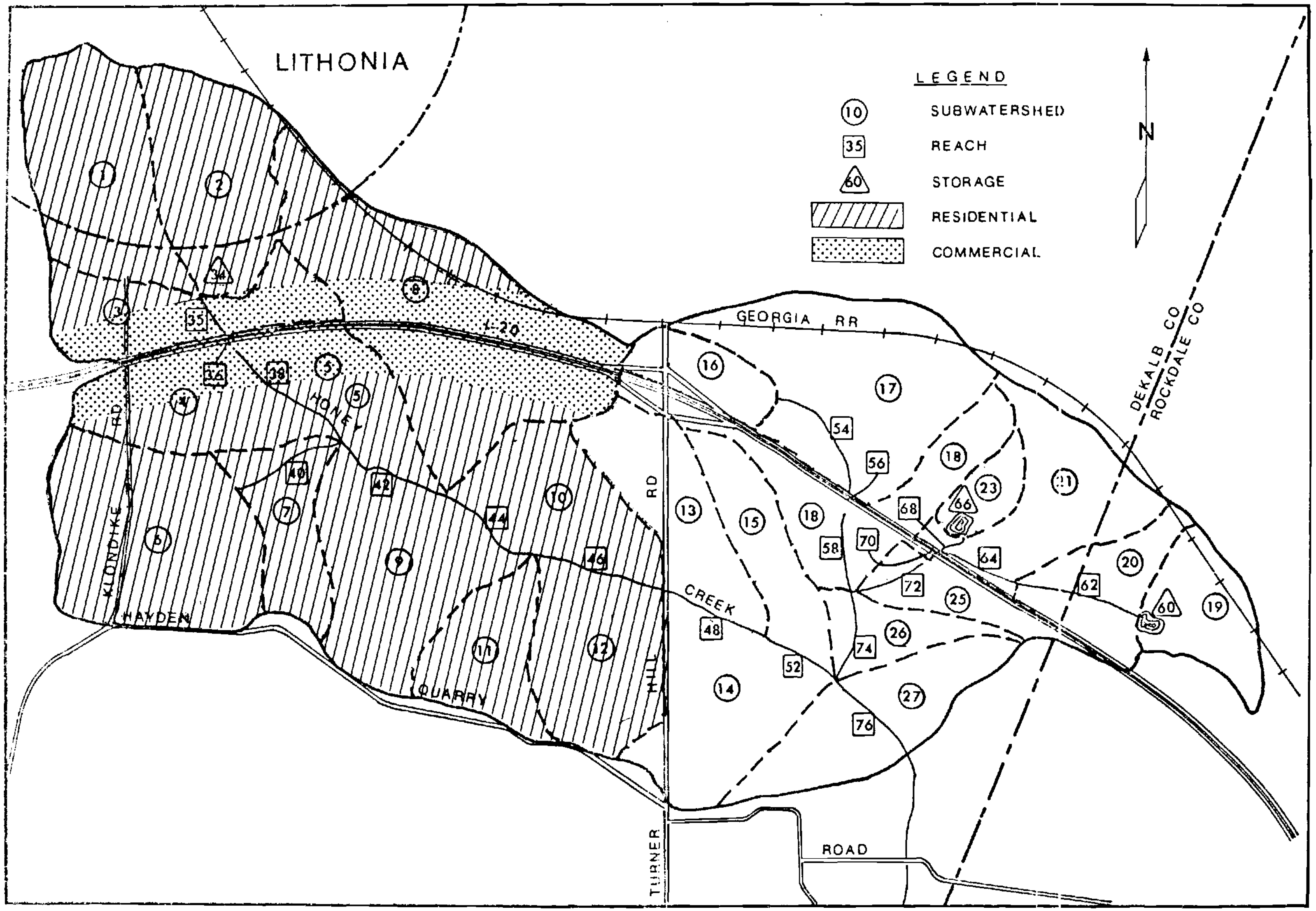


Figure 4-5. Map of Honey Creek for Run 4

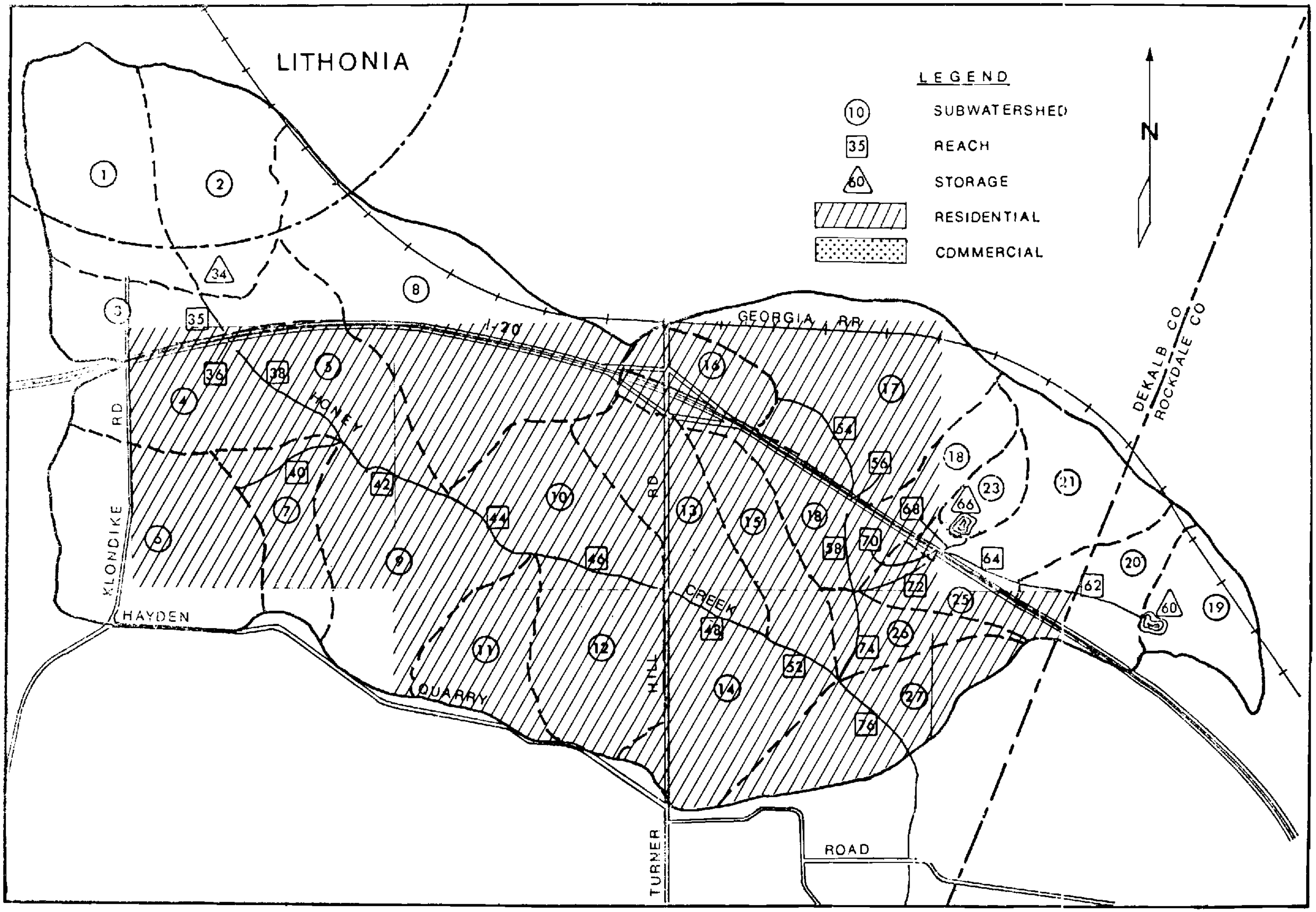


Figure 4-6. Map of Honey Creek for Run 6

considered for the developed area with the exception of a strip commercial development along I-20.

IV. Descriptions of Simulation Runs

1) Existing Development

This simulation was based on present level of watershed development. Table 4-3 shows specifications for source areas and channel segments. Tables 4-4 and 4-6 show the annual peak flows for water years 1918-1942 and results of flood frequency analyses for reaches 46, 74, and 76. As can be seen on Figures 4-2 thru 4-5, channel segment 46 ends at Turner Hill Road, channel segment 74 is the lower reach of a tributary entering from the left bank below Turner Hill Road,and segment 76 is the lower end of study.

2) Complete Watershed Development

This simulation was based on complete watershed development as shown on Figure 4-2. It was assumed that 80% of commercial development and 20% of residential development was impervious. Table 4-7 shows specifications for source areas. Channel segments were assumed to remain in their present conditions as reflected in Table 4-3. Tables 4-8 through 4-10 show the annual peak flows and frequency analyses for the same reaches as for simulation 1. Comparison of existing development with complete development shows, for reach 76, a 45% increase for the 2-year flood and a 24% increase for the 100-year flood. At the lower end of the study these increases would mean about a 1 foot increase in elevation of the 2-year flood and about a 1/2 foot increase in the 100-year flood.

Table 4-3. Specifications for Source Areas and Channel Segments for Run 1

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	RAPID	MODERATE	SLOW	IMPERVIOUS
		FRACTION OF AREA WITH EACH SOIL TYPE			
1	78.700	.070	.800	.000	.130
2	65.000	.060	.860	.000	.080
3	50.400	.140	.800	.000	.060
4	48.400	.400	.550	.000	.050
5	41.100	.500	.490	.000	.010
6	95.800	.360	.490	.000	.150
7	40.800	.420	.580	.000	.000
8	143.800	.140	.730	.000	.130
9	77.000	.340	.660	.000	.000
10	55.900	.410	.590	.000	.000
11	38.500	.200	.560	.000	.240
12	68.100	.340	.660	.000	.000
13	54.500	.000	.995	.000	.005
14	68.600	.230	.770	.000	.000
15	36.000	.000	.980	.000	.020
16	24.300	.000	.860	.000	.140
17	115.600	.000	.640	.000	.360
18	68.800	.000	.800	.000	.200
19	34.000	.000	1.000	.000	.000
20	39.500	.000	1.000	.000	.000
21	68.700	.000	.650	.000	.350
23	17.000	.000	.650	.000	.350
25	27.400	.000	.820	.000	.180
26	27.200	.000	1.000	.000	.000
27	56.000	.125	.875	.000	.000

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS
35	4	480.00	.01150	.050
36	1	330.00	.00610	.015
38	4	1350.00	.01560	.050
40	4	1150.00	.01520	.050
42	4	1570.00	.00730	.050
44	4	1070.00	.00790	.040
46	4	1600.00	.00720	.040
48	4	1250.00	.00440	.040
52	4	950.00	.00580	.040
54	4	1500.00	.02000	.040
56	1	280.00	.00530	.015
58	4	950.00	.01320	.040
62	4	1550.00	.01100	.040
64	4	1000.00	.02100	.040
68	4	300.00	.06000	.040
70	1	250.00	.00600	.015
72	4	800.00	.01500	.040
74	4	1130.00	.00620	.040
76	4	1200.00	.00430	.035

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

Table 4-4. Run 1 Annual Peak Flows and Frequency Analysis (Segment 46)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 46 FOR HONEY CREEK RUN 1

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	78.9	1918	APR.	7
1919	342.4	1919	MAY	6
1920	783.6	1920	MAR.	12
1921	736.4	1921	AUG.	24
1922	596.1	1921	NOV.	16
1923	671.6	1923	FEB.	13
1924	251.2	1924	MAY	27
1925	532.2	1925	JAN.	11
1926	814.7	1926	AUG.	11
1927	308.4	1927	FEB.	23
1928	995.0	1928	JULY	10
1929	452.0	1929	MAR.	14
1930	165.3	1930	MAY	17
1931	86.4	1930	NOV.	16
1932	383.7	1932	JUNE	18
1933	326.0	1933	JUNE	10
1934	245.2	1934	JULY	19
1935	150.7	1934	OCT.	5
1936	438.8	1936	FEB.	3
1937	467.7	1937	APR.	29
1938	573.7	1938	APR.	8
1939	378.6	1939	JUNE	22
1940	522.3	1940	SEPT	10
1941	249.2	1941	AUG.	13
1942	566.5	1942	MAR.	20

MEAN = 444.7 STANDARD DEVIATION = 237.5

FLOOD FREQUENCY FOR FLOWPOINT 46 FOR HONEY CREEK RUN 1

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	405.7
5-YEAR	20.0	615.6
10-YEAR	10.0	754.5
25-YEAR	4.0	930.1
50-YEAR	2.0	1060.4
100-YEAR	1.0	1189.7
200-YEAR	0.5	1318.5

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-5. Run 1 Annual Peak Flows and Frequency Analysis (Segment 74)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 74 FOR HONEY CREEK RUN 1

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	87.8	1918	APR.	7
1919	296.3	1919	MAY	6
1920	684.6	1920	AUG.	15
1921	744.6	1921	AUG.	24
1922	588.7	1921	NOV.	16
1923	488.6	1923	FEB.	13
1924	244.5	1924	MAY	27
1925	363.2	1925	JAN.	11
1926	762.8	1926	AUG.	11
1927	215.4	1927	FEB.	23
1928	855.6	1928	JULY	10
1929	331.8	1929	MAR.	14
1930	159.9	1930	MAY	17
1931	139.9	1931	JULY	28
1932	357.5	1932	JUNE	18
1933	408.8	1933	JUNE	10
1934	310.2	1934	JULY	19
1935	126.8	1934	OCT.	5
1936	335.3	1936	SEPT	29
1937	381.1	1937	JUNE	17
1938	594.6	1938	JUNE	25
1939	297.1	1939	JUNE	22
1940	552.3	1940	SEPT	10
1941	307.3	1941	AUG.	13
1942	396.5	1942	MAR.	20

MEAN = 401.2 STANDARD DEVIATION = 209.1

FLOOD FREQUENCY FOR FLOWPOINT 74 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	366.9
5-YEAR	20.0	551.6
10-YEAR	10.0	674.0
25-YEAR	4.0	828.6
50-YEAR	2.0	943.2
100-YEAR	1.0	1057.1
200-YEAR	0.5	1170.5

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-6. Run 1 Annual Peak Flows and Frequency Analysis (Segment 76)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 76 FOR HONEY CREEK RUN 1

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	167.2	1918	APR.	7
1919	640.0	1919	MAY	6
1920	1464.2	1920	MAR.	12
1921	1416.1	1921	AUG.	24
1922	1148.6	1921	NOV.	16
1923	1223.3	1923	FEB.	13
1924	511.3	1924	MAY	27
1925	968.9	1925	JAN.	11
1926	1524.0	1926	AUG.	11
1927	580.1	1927	FEB.	23
1928	1840.8	1928	JULY	10
1929	813.8	1929	MAR.	14
1930	331.5	1930	MAY	17
1931	187.1	1930	NOV.	16
1932	747.8	1932	JUNE	18
1933	674.0	1933	JUNE	10
1934	525.9	1934	JULY	19
1935	299.0	1934	OCT.	5
1936	787.4	1936	FEB.	3
1937	852.3	1937	APR.	29
1938	1066.8	1938	JUNE	25
1939	702.6	1939	JUNE	22
1940	1017.6	1940	SEPT	10
1941	530.9	1941	AUG.	13
1942	1026.6	1942	MAR.	20

MEAN = 841.9 STANDARD DEVIATION = 431.4

FLOOD FREQUENCY FOR FLOWPOINT 76 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	771.0
5-YEAR	20.0	1152.3
10-YEAR	10.0	1404.7
25-YEAR	4.0	1723.6
50-YEAR	2.0	1960.2
100-YEAR	1.0	2195.1
200-YEAR	0.5	2429.1

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-7. Specifications for Area Segments for Run 2

SPECIFICATIONS FOR SOURCE AREAS

		FRACTION OF AREA WITH EACH SOIL TYPE			
NUMBER	AREA-ACRE	RAPID	MODERATE	SLOW	IMPERVIOUS
1	78.700	.060	.640	.000	.300
2	65.000	.050	.690	.000	.260
3	50.400	.060	.320	.000	.620
4	48.400	.220	.300	.000	.480
5	41.100	.270	.270	.000	.460
6	95.800	.400	.390	.000	.210
7	40.800	.340	.460	.000	.200
8	143.800	.070	.380	.000	.550
9	77.000	.270	.530	.000	.200
10	55.900	.330	.470	.000	.200
11	38.500	.160	.450	.000	.390
12	68.100	.270	.530	.000	.200
13	54.500	.000	.690	.000	.310
14	68.600	.180	.620	.000	.200
15	36.000	.000	.680	.000	.320
16	24.300	.000	.170	.000	.830
17	115.600	.000	.440	.000	.560
18	68.800	.000	.350	.000	.650
19	34.000	.000	.750	.000	.250
20	39.500	.000	.480	.000	.520
21	68.700	.000	.520	.000	.480
23	17.000	.000	.410	.000	.590
25	27.400	.000	.270	.000	.730
26	27.200	.000	.750	.000	.250
27	56.000	.100	.700	.000	.200

Table 4-8. Run 2 Annual Peak Flows and Frequency Analysis (Segment 46)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 46 FOR HONEY CREEK RUN 2

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	198.2	1918	APR.	7
1919	469.4	1919	MAY	6
1920	919.8	1920	MAR.	12
1921	1031.4	1921	AUG.	24
1922	857.7	1921	NOV.	16
1923	757.0	1923	FEB.	13
1924	448.7	1924	MAY	27
1925	604.3	1925	JAN.	11
1926	1147.5	1926	AUG.	11
1927	362.9	1927	FEB.	23
1928	1155.1	1928	JULY	10
1929	524.8	1929	MAR.	14
1930	276.1	1930	MAY	17
1931	315.9	1931	JULY	28
1932	650.4	1932	JUNE	18
1933	681.1	1933	JUNE	10
1934	510.0	1934	JULY	19
1935	228.5	1934	OCT.	5
1936	612.5	1936	SEPT	29
1937	548.3	1937	APR.	29
1938	903.9	1938	JUNE	25
1939	621.7	1939	JUNE	22
1940	873.0	1940	SEPT	10
1941	627.0	1941	AUG.	13
1942	630.6	1942	MAR.	20

MEAN = 638.2 STANDARD DEVIATION = 268.2

FLOOD FREQUENCY FOR FLOWPOINT 46 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	594.2
5-YEAR	20.0	831.2
10-YEAR	10.0	988.1
25-YEAR	4.0	1186.4
50-YEAR	2.0	1333.5
100-YEAR	1.0	1479.5
200-YEAR	0.5	1624.9

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-9. Run 2 Annual Peak Flows and Frequency Analysis (Segment 74)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 74 FOR HONEY CREEK RUN 2

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	169.9	1918	APR.	7
1919	421.8	1919	MAY	6
1920	872.3	1920	AUG.	15
1921	1033.1	1921	AUG.	24
1922	815.0	1921	NOV.	16
1923	556.8	1923	FEB.	13
1924	388.6	1924	MAY	27
1925	536.3	1924	DEC.	8
1926	1050.0	1926	AUG.	11
1927	266.4	1927	FEB.	23
1928	977.7	1928	JULY	10
1929	388.1	1929	MAR.	14
1930	279.6	1930	MAY	17
1931	384.0	1931	JULY	28
1932	539.2	1932	JUNE	18
1933	801.0	1933	JUNE	10
1934	608.8	1934	JULY	19
1935	207.2	1934	OCT.	5
1936	578.5	1936	SEPT	29
1937	568.2	1937	JUNE	17
1938	966.9	1938	JUNE	25
1939	566.7	1939	JUNE	22
1940	907.0	1940	SEPT	10
1941	669.5	1941	AUG.	13
1942	446.9	1942	MAR.	20

MEAN = 600.0 STANDARD DEVIATION = 264.9

FLOOD FREQUENCY FOR FLOWPOINT 74 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	556.5
5-YEAR	20.0	790.6
10-YEAR	10.0	945.6
25-YEAR	4.0	1141.4
50-YEAR	2.0	1286.7
100-YEAR	1.0	1430.9
200-YEAR	0.5	1574.6

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-10. Run 2 Annual Peak Flows and Frequency Analysis (Segment 76)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 76 FOR HONEY CREEK RUN 2

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	384.9	1918	APR.	7
1919	868.0	1919	MAY	6
1920	1696.7	1920	MAR.	12
1921	1920.2	1921	AUG.	24
1922	1598.6	1921	NOV.	16
1923	1376.6	1923	FEB.	13
1924	859.2	1924	MAY	27
1925	1091.3	1925	JAN.	11
1926	2093.7	1926	AUG.	11
1927	685.0	1927	FEB.	23
1928	2105.9	1928	JULY	10
1929	938.3	1929	MAR.	14
1930	533.7	1930	MAY	17
1931	618.3	1931	JULY	28
1932	1242.9	1932	JUNE	18
1933	1316.5	1933	JUNE	10
1934	997.0	1934	JULY	19
1935	445.8	1934	OCT.	5
1936	1207.3	1936	SEPT	29
1937	1056.4	1937	JUNE	17
1938	1697.9	1938	JUNE	25
1939	1157.6	1939	JUNE	22
1940	1645.5	1940	SEPT	10
1941	1219.2	1941	AUG.	13
1942	1133.8	1942	MAR.	20

MEAN = 1195.6 STANDARD DEVIATION = 487.0

FLOOD FREQUENCY FOR FLOWPOINT 76 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	1115.6
5-YEAR	20.0	1546.0
10-YEAR	10.0	1831.0
25-YEAR	4.0	2191.0
50-YEAR	2.0	2458.0
100-YEAR	1.0	2723.2
200-YEAR	0.5	2987.3

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

3) Complete Development of Lower Half with Upper Half Remaining in its Present Condition

This scheme as shown on Figure 4-4 was simulated to determine what effect staging of watershed development had on peak flows. Table 4-11 shows specifications for source areas. Channels were again assumed to remain in their natural state. Tables 4-12 through 4-14 show the annual peak flows and frequency analyses for the same reaches as for simulation 1. Comparison with existing development shows, for reach 76, a 24% increase for the 2-year flood and a 22% increase for the 100-year flood.

4) Complete Development of Upper Half with Lower Half Remaining as Present

This scheme as shown on Figure 4-5 was also simulated to determine effect of staging development. Table 4-15 shows specifications for source areas. Channels were assumed to remain in their present condition. Tables 4-16 through 4-18 show peak flows and frequency analyses for the same reaches as for simulation No. 1. Comparison with existing development shows, for reach 76, a 22% increase in peak flow for the 2-year flood and an 11% increase in the 100-year flood.

5) Complete Development of Watershed with a Detention Storage Segment at Confluence of Reaches 58 and 72

This simulation was made to determine effect of placing detention storage in the downstream part of a watershed on peak flows. The storage segment was assumed to be a dam with a 36" drop inlet 5 feet below a trapezpoidal broad crested weir with side slopes of 2 to 1. The specifications for the storage segment are shown in Table 4-19.

Table 4-11. Specifications for Area Segments for Run 3

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA*ACRE	RAPID	MODERATE	SLOW	IMPERVIOUS
		FRACTION OF AREA WITH EACH SOIL TYPE			
1	78.700	.060	.640	.000	.300
2	65.000	.050	.690	.000	.260
3	50.400	.060	.320	.000	.620
4	48.400	.220	.300	.000	.480
5	41.100	.270	.270	.000	.460
6	95.800	.400	.390	.000	.210
7	40.800	.340	.460	.000	.200
8	143.800	.070	.380	.000	.550
9	77.000	.270	.530	.000	.200
10	55.900	.330	.470	.000	.200
11	38.500	.160	.450	.000	.390
12	68.100	.270	.530	.000	.200
13	54.500	.000	.995	.000	.005
14	68.600	.230	.770	.000	.000
15	36.000	.000	.980	.000	.020
16	24.300	.000	.860	.000	.140
17	115.600	.000	.640	.000	.360
18	68.800	.000	.800	.000	.200
19	34.000	.000	1.000	.000	.000
20	39.500	.000	1.000	.000	.000
21	68.700	.000	.650	.000	.350
23	17.000	.000	.650	.000	.350
25	27.400	.000	.820	.000	.180
26	27.200	.000	1.000	.000	.000
27	56.000	.125	.875	.000	.000

Table 4-12. Run 3 Annual Peak Flows and Frequency Analysis (Segment 46)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 46 FOR HONEY CREEK RUN 3

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	198.2	1918	APR.	7
1919	469.4	1919	MAY	6
1920	919.8	1920	MAR.	12
1921	1031.4	1921	AUG.	24
1922	857.7	1921	NOV.	16
1923	757.0	1923	FEB.	13
1924	448.7	1924	MAY	27
1925	604.3	1925	JAN.	11
1926	1147.5	1926	AUG.	11
1927	362.9	1927	FEB.	23
1928	1155.1	1928	JULY	10
1929	524.8	1929	MAR.	14
1930	276.1	1930	MAY	17
1931	315.9	1931	JULY	28
1932	650.4	1932	JUNE	18
1933	681.1	1933	JUNE	10
1934	510.0	1934	JULY	19
1935	228.5	1934	OCT.	5
1936	612.5	1936	SEPT	29
1937	548.3	1937	APR.	29
1938	903.9	1938	JUNE	25
1939	621.7	1939	JUNE	22
1940	873.0	1940	SEPT	10
1941	627.0	1941	AUG.	13
1942	630.6	1942	MAR.	20

MEAN = 638.2 STANDARD DEVIATION = 268.2

FLOOD FREQUENCY FOR FLOWPOINT 46 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	594.2
5-YEAR	20.0	831.2
10-YEAR	10.0	988.1
25-YEAR	4.0	1186.4
50-YEAR	2.0	1333.5
100-YEAR	1.0	1479.5
200-YEAR	0.5	1624.9

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-13. Run 3 Annual Peak Flows and Frequency Analysis (Segment 74)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 74 FOR HONEY CREEK RUN 3

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	87.8	1918	APR.	7
1919	296.3	1919	MAY	6
1920	684.6	1920	AUG.	15
1921	744.6	1921	AUG.	24
1922	588.7	1921	NOV.	16
1923	488.6	1923	FEB.	13
1924	244.5	1924	MAY	27
1925	363.2	1925	JAN.	11
1926	762.8	1926	AUG.	11
1927	215.4	1927	FEB.	23
1928	855.6	1928	JULY	10
1929	331.8	1929	MAR.	14
1930	159.9	1930	MAY	17
1931	139.9	1931	JULY	28
1932	357.5	1932	JUNE	18
1933	408.8	1933	JUNE	10
1934	310.2	1934	JULY	19
1935	126.8	1934	OCT.	5
1936	335.3	1936	SEPT	29
1937	381.1	1937	JUNE	17
1938	594.6	1938	JUNE	25
1939	297.1	1939	JUNE	22
1940	552.3	1940	SEPT	10
1941	307.3	1941	AUG.	13
1942	396.5	1942	MAR.	20

MEAN = 401.2 STANDARD DEVIATION = 209.1

FLOOD FREQUENCY FOR FLOWPOINT 74 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	366.9
5-YEAR	20.0	551.6
10-YEAR	10.0	674.0
25-YEAR	4.0	828.6
50-YEAR	2.0	943.2
100-YEAR	1.0	1057.1
200-YEAR	0.5	1170.5

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-14. Run 3 Annual Peak Flows and Frequency Analysis (Segment 76)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 76 FOR HONEY CREEK RUN 3

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	286.8	1918	APR.	7
1919	757.6	1919	MAY	6
1920	1595.1	1920	MAR.	12
1921	1660.9	1921	AUG.	24
1922	1376.1	1921	NOV.	16
1923	1310.9	1923	FEB.	13
1924	693.1	1924	MAY	27
1925	1042.1	1925	JAN.	11
1926	1823.7	1926	AUG.	11
1927	642.0	1927	FEB.	23
1928	1999.0	1928	JULY	10
1929	890.0	1929	MAR.	14
1930	410.5	1930	MAY	17
1931	385.4	1931	JULY	28
1932	1020.3	1932	JUNE	18
1933	944.4	1933	JUNE	10
1934	713.4	1934	JULY	19
1935	371.4	1934	OCT.	5
1936	992.4	1936	SEPT	29
1937	944.8	1937	APR.	29
1938	1354.2	1938	JUNE	25
1939	968.8	1939	JUNE	22
1940	1311.1	1940	SEPT	10
1941	839.8	1941	AUG.	13
1942	1095.9	1942	MAR.	20

MEAN = 1017.2 STANDARD DEVIATION = 455.0

FLOOD FREQUENCY FOR FLOWPOINT 76 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	942.5
5-YEAR	20.0	1344.6
10-YEAR	10.0	1610.8
25-YEAR	4.0	1947.2
50-YEAR	2.0	2196.7
100-YEAR	1.0	2444.4
200-YEAR	0.5	2691.2

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-15. Specifications for Area Segments for Run 4

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE RAPID	MODERATE	SLOW	IMPERVIOUS
1	78.700	.070	.800	.000	.130
2	65.000	.060	.860	.000	.080
3	50.400	.140	.800	.000	.060
4	48.400	.400	.550	.000	.050
5	41.100	.500	.490	.000	.010
6	95.800	.360	.490	.000	.150
7	40.800	.420	.580	.000	.000
8	143.800	.140	.730	.000	.130
9	77.000	.340	.660	.000	.000
10	55.900	.410	.590	.000	.000
11	38.500	.200	.560	.000	.240
12	68.100	.340	.660	.000	.000
13	54.500	.000	.690	.000	.310
14	68.600	.180	.620	.000	.200
15	36.000	.000	.680	.000	.320
16	24.300	.000	.170	.000	.830
17	119.600	.000	.440	.000	.560
18	68.800	.000	.350	.000	.650
19	34.000	.000	.750	.000	.250
20	39.500	.000	.480	.000	.520
21	68.700	.000	.520	.000	.480
23	17.000	.000	.410	.000	.590
25	27.400	.000	.270	.000	.730
26	27.200	.000	.750	.000	.250
27	56.000	.100	.700	.000	.200

Table 4-16. Run 4 Annual Peak Flows and Frequency Analysis (Segment 46)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 46 FOR HONEY CREEK RUN 4

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	78.9	1918	APR.	7
1919	342.4	1919	MAY	6
1920	783.6	1920	MAR.	12
1921	736.4	1921	AUG.	24
1922	596.1	1921	NOV.	16
1923	671.6	1923	FEB.	13
1924	251.2	1924	MAY	27
1925	532.2	1925	JAN.	11
1926	814.7	1926	AUG.	11
1927	308.4	1927	FEB.	23
1928	995.0	1928	JULY	10
1929	452.0	1929	MAR.	14
1930	165.3	1930	MAY	17
1931	86.4	1930	NOV.	16
1932	383.7	1932	JUNE	18
1933	326.0	1933	JUNE	10
1934	245.2	1934	JULY	19
1935	150.7	1934	OCT.	5
1936	438.8	1936	FEB.	3
1937	467.7	1937	APR.	29
1938	573.7	1938	APR.	8
1939	378.6	1939	JUNE	22
1940	522.3	1940	SEPT	10
1941	249.2	1941	AUG.	13
1942	566.5	1942	MAR.	20

MEAN = 444.7 STANDARD DEVIATION = 237.5

FLOOD FREQUENCY FOR FLOWPOINT 46 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	405.7
5-YEAR	20.0	615.6
10-YEAR	10.0	754.5
25-YEAR	4.0	930.1
50-YEAR	2.0	1060.4
100-YEAR	1.0	1189.7
200-YEAR	0.5	1318.5

*PERCENT CHANCE IN <u>ANY</u> YEAR OF GETTING A FLOOD <u>GREATER</u> THAN THAT INDICATED IN THE TABLE.

Table 4-17. Run 4 Annual Peak Flows and Frequency Analysis (Segment 74)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 74 FOR HONEY CREEK RUN 4

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	169.9	1918	APR.	7
1919	421.8	1919	MAY	6
1920	872.3	1920	AUG.	15
1921	1033.1	1921	AUG.	24
1922	815.0	1921	NOV.	16
1923	556.8	1923	FEB.	13
1924	388.6	1924	MAY	27
1925	536.3	1924	DEC.	8
1926	1050.0	1926	AUG.	11
1927	266.4	1927	FEB.	23
1928	977.7	1928	JULY	10
1929	388.1	1929	MAR.	14
1930	279.6	1930	MAY	17
1931	384.0	1931	JULY	28
1932	539.2	1932	JUNE	18
1933	801.0	1933	JUNE	10
1934	608.8	1934	JULY	19
1935	207.2	1934	OCT.	5
1936	578.5	1936	SEPT	29
1937	568.2	1937	JUNE	17
1938	966.9	1938	JUNE	25
1939	566.7	1939	JUNE	22
1940	907.0	1940	SEPT	10
1941	669.5	1941	AUG.	13
1942	446.9	1942	MAR.	20

MEAN = 600.0 STANDARD DEVIATION = 264.9

FLOOD FREQUENCY FOR FLOWPOINT 74 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	556.5
5-YEAR	20.0	790.6
10-YEAR	10.0	945.6
25-YEAR	4.0	1141.4
50-YEAR	2.0	1286.7
100-YEAR	1.0	1430.9
200-YEAR	0.5	1574.6

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-18. Run 4 Annual Peak Flows and Frequency Analysis (Segment 76)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 76 FOR HONEY CREEK RUN 4

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	270.3	1918	APR.	7
1919	763.1	1919	MAY	6
1920	1567.0	1920	MAR.	12
1921	1697.2	1921	AUG.	24
1922	1380.4	1921	NOV.	16
1923	1288.5	1923	FEB.	13
1924	670.5	1924	MAY	27
1925	1019.8	1925	JAN.	11
1926	1812.5	1926	AUG.	11
1927	615.6	1927	FEB.	23
1928	1952.5	1928	JULY	10
1929	863.1	1929	MAR.	14
1930	468.8	1930	MAY	17
1931	490.6	1931	JULY	28
1932	970.7	1932	JUNE	18
1933	1091.4	1933	JUNE	10
1934	851.3	1934	JULY	19
1935	375.3	1934	OCT.	5
1936	925.1	1936	SEPT	29
1937	920.3	1937	JUNE	17
1938	1446.3	1938	JUNE	25
1939	885.8	1939	JUNE	22
1940	1388.1	1940	SEPT	10
1941	941.2	1941	AUG.	13
1942	1064.1	1942	MAR.	20

MEAN = 1028.8 STANDARD DEVIATION = 446.8

FLOOD FREQUENCY FOR FLOWPOINT 76 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	955.4
5-YEAR	20.0	1350.3
10-YEAR	10.0	1611.7
25-YEAR	4.0	1942.0
50-YEAR	2.0	2187.0
100-YEAR	1.0	2430.3
200-YEAR	0.5	2672.6

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-19

Specifications for Storage Segment 73

Storage (Acre-feet)	Discharge (CFS)	Elevation (Feet-MSL)	Head (Feet)	Surface Area (Acres)
0	0	825	0	0.40
2.23	45.9	827.5	2.5	1.38
7.42	105.3	830.0	5.0	2.98
28.83	1264.3	835.0	10.0	7.98

Specifications for source areas and channel segments were the same as simulation No. 2. Tables 4-20 through 4-22 show annual peak flows and frequency analyses for the same reaches as in simulation No.1. Comparison of this simulation with simulation number 2 for reach No. 74 shows that the storage segment reduced the peak flow of the 2-year flood by 25% and the 100-year flood by 19%.

In segment 76, the 2-year flood was reduced by 6% while the 100-year flood was reduced by 3%.

6) Simulation of Proposed Development "Windswept"

This development scheme as shown on Figure 4-6 simulated conditions that would exist if land lots 151 170, 183, 171, 182, and 203 were developed with single family residences. The development was assumed to have 20% impervious area. Table 4-23 shows specifications for source areas. Channel reaches were assumed to remain in their present condition. Tables 4-24 through 4-26 show annual peak flows and frequency analyses for same reaches as for simulation 1. Comparison of this development scheme with existing shows, for reach 76, a 19% increase in the 2-year flood flow and a 9% increase in the 100-year flood.

Table 4-20. Run 5 Annual Peak Flows and Frequency Analysis (Segment 46)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 46 FOR HONEY CREEK RUN 5

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	198.2	1918	APR.	7
1919	469.4	1919	MAY	6
1920	919.8	1920	MAR.	12
1921	1031.4	1921	AUG.	24
1922	857.7	1921	NOV.	16
1923	757.0	1923	FEB.	13
1924	448.7	1924	MAY	27
1925	604.3	1925	JAN.	11
1926	1147.5	1926	AUG.	11
1927	362.9	1927	FEB.	23
1928	1155.1	1928	JULY	10
1929	524.8	1929	MAR.	14
1930	276.1	1930	MAY	17
1931	315.9	1931	JULY	28
1932	650.4	1932	JUNE	18
1933	681.1	1933	JUNE	10
1934	510.0	1934	JULY	19
1935	228.5	1934	OCT.	5
1936	612.5	1936	SEPT	29
1937	548.3	1937	APR.	29
1938	903.9	1938	JUNE	25
1939	621.7	1939	JUNE	22
1940	873.0	1940	SEPT	10
1941	627.0	1941	AUG.	13
1942	630.6	1942	MAR.	20

MEAN = 638.2 STANDARD DEVIATION = 268.2

FLOOD FREQUENCY FOR FLOWPOINT 46 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	594.2
5-YEAR	20.0	831.2
10-YEAR	10.0	988.2
25-YEAR	4.0	1186.5
50-YEAR	2.0	1333.6
100-YEAR	1.0	1479.6
200-YEAR	0.5	1625.1

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-21. Run 5 Annual Peak Flows and Frequency Analysis (Segment 74)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 74 FOR HONEY CREEK RUN 5

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	90.1	1918	APR.	7
1919	306.6	1919	MAY	6
1920	647.2	1920	AUG.	15
1921	825.3	1921	AUG.	24
1922	642.8	1921	NOV.	16
1923	488.7	1923	FEB.	13
1924	326.6	1924	MAY	27
1925	347.9	1925	JAN.	11
1926	870.2	1926	AUG.	11
1927	219.6	1927	FEB.	23
1928	793.0	1928	JULY	10
1929	280.2	1929	MAR.	14
1930	121.3	1930	MAY	17
1931	193.8	1931	JULY	28
1932	489.8	1932	JUNE	18
1933	550.6	1933	JUNE	10
1934	367.9	1934	JULY	19
1935	105.4	1934	OCT.	5
1936	486.2	1936	SEPT	29
1937	382.7	1937	JUNE	17
1938	721.7	1938	JUNE	25
1939	445.6	1939	JUNE	22
1940	693.6	1940	SEPT	10
1941	520.0	1941	AUG.	13
1942	389.6	1942	MAR.	20

MEAN = 452.3 STANDARD DEVIATION = 226.3

FLOOD FREQUENCY FOR FLOWPOINT 74 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	415.1
5-YEAR	20.0	615.0
10-YEAR	10.0	747.4
25-YEAR	4.0	914.7
50-YEAR	2.0	1038.8
100-YEAR	1.0	1162.0
200-YEAR	0.5	1284.7

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-22. Run 5 Annual Peak Flows and Frequency Analysis (Segment 76)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 76 FOR HONEY CREEK RUN 5

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	318.9	1918	APR.	7
1919	816.1	1919	MAY	6
1920	1616.0	1920	MAR.	12
1921	1834.4	1921	AUG.	24
1922	1511.3	1921	NOV.	16
1923	1347.7	1923	FEB.	13
1924	827.1	1924	MAY	27
1925	1045.1	1925	JAN.	11
1926	2012.5	1926	AUG.	11
1927	672.3	1927	FEB.	23
1928	2002.2	1928	JULY	10
1929	879.4	1929	MAR.	14
1930	417.4	1930	MAY	17
1931	509.1	1931	JULY	28
1932	1211.3	1932	JUNE	18
1933	1191.6	1933	JUNE	10
1934	862.2	1934	JULY	19
1935	370.5	1934	OCT.	5
1936	1164.1	1936	SEPT	29
1937	987.9	1937	APR.	29
1938	1578.7	1938	JUNE	25
1939	1158.6	1939	JUNE	22
1940	1537.6	1940	SEPT	10
1941	1137.8	1941	AUG.	13
1942	1121.6	1942	MAR.	20

MEAN = 1125.3 STANDARD DEVIATION = 480.2

FLOOD FREQUENCY FOR FLOWPOINT 76 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	1046.4
5-YEAR	20.0	1470.7
10-YEAR	10.0	1751.7
25-YEAR	4.0	2106.7
50-YEAR	2.0	2370.1
100-YEAR	1.0	2631.5
200-YEAR	0.5	2891.9

* PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-23. Specifications for Area
Segment for Run 6

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE RAPID	MODERATE	SLOW	IMPERVIOUS
1	78.700	.070	.800	.000	.130
2	65.000	.060	.860	.000	.080
3	50.400	.140	.780	.000	.080
4	48.400	.340	.460	.000	.200
5	41.100	.400	.390	.000	.210
6	95.800	.330	.450	.000	.220
7	40.800	.340	.480	.000	.180
8	143.800	.130	.660	.000	.210
9	77.000	.290	.550	.000	.160
10	55.900	.330	.470	.000	.200
11	38.500	.160	.450	.000	.390
12	68.100	.270	.530	.000	.200
13	54.500	.000	.800	.000	.200
14	68.600	.180	.620	.000	.200
15	36.000	.000	.780	.000	.220
16	24.300	.000	.720	.000	.280
17	115.600	.000	.540	.000	.460
18	68.800	.000	.670	.000	.330
19	34.000	.000	1.000	.000	.000
20	39.500	.000	.930	.000	.070
21	68.700	.000	.650	.000	.350
23	17.000	.000	.650	.000	.350
25	27.400	.000	.710	.000	.290
26	27.200	.000	.800	.000	.200
27	56.000	.100	.700	.000	.200

Table 4-24. Run 6 Annual Peak Flows and Frequency Analysis (Segment 46)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 46 FOR HONEY CR. RUN NO 6

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	120.5	1918	APR.	7
1919	384.8	1919	MAY	6
1920	834.8	1920	MAR.	12
1921	836.5	1921	AUG.	24
1922	689.5	1921	NOV.	16
1923	705.1	1923	FEB.	13
1924	315.9	1924	MAY	27
1925	558.5	1925	JAN.	11
1926	924.0	1926	AUG.	11
1927	328.5	1927	FEB.	23
1928	1053.9	1928	JULY	10
1929	476.4	1929	MAR.	14
1930	207.2	1930	MAY	17
1931	176.6	1931	JULY	28
1932	467.8	1932	JUNE	18
1933	451.5	1933	JUNE	10
1934	341.5	1934	JULY	19
1935	185.7	1934	OCT.	5
1936	463.2	1936	SEPT	29
1937	495.3	1937	APR.	29
1938	670.5	1938	JUNE	25
1939	456.9	1939	JUNE	22
1940	645.4	1940	SEPT	10
1941	381.1	1941	AUG.	13
1942	591.0	1942	MAR.	20

MEAN = 510.5   STANDARD DEVIATION = 242.0

FLOOD FREQUENCY FOR FLOWPOINT 46 FOR HONEY CREEK

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	470.8
5-YEAR	20.0	684.6
10-YEAR	10.0	826.2
25-YEAR	4.0	1005.1
50-YEAR	2.0	1137.8
100-YEAR	1.0	1269.6
200-YEAR	0.5	1400.8

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-25. Run 6 Annual Peak Flows and Frequency Analysis (Segment 74)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 74 FOR HONEY CR. RUN NO 6

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	109.8	1918	APR.	7
1919	329.3	1919	MAY	6
1920	735.7	1920	AUG.	15
1921	818.6	1921	AUG.	24
1922	653.1	1921	NOV.	16
1923	506.4	1923	FEB.	13
1924	280.1	1924	MAY	27
1925	377.8	1925	JAN.	11
1926	838.4	1926	AUG.	11
1927	229.7	1927	FEB.	23
1928	895.4	1928	JULY	10
1929	347.6	1929	MAR.	14
1930	197.6	1930	MAY	17
1931	196.4	1931	JULY	28
1932	405.6	1932	JUNE	18
1933	515.4	1933	JUNE	10
1934	407.1	1934	JULY	19
1935	142.0	1934	OCT.	5
1936	405.2	1936	SEPT	29
1937	441.2	1937	JUNE	17
1938	690.6	1938	JUNE	25
1939	355.1	1939	JUNE	22
1940	654.9	1940	SEPT	10
1941	411.3	1941	AUG.	13
1942	409.4	1942	MAR.	20

MEAN = 454.1 STANDARD DEVIATION = 221.7

FLOOD FREQUENCY FOR FLOWPOINT 74 FOR HONEY CR.

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	417.7
5-YEAR	20.0	613.6
10-YEAR	10.0	743.3
25-YEAR	4.0	907.2
50-YEAR	2.0	1028.8
100-YEAR	1.0	1149.5
200-YEAR	0.5	1269.8

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

Table 4-26. Run 6 Annual Peak Flows and Frequency Analysis (Segment 76)

ANNUAL PEAK FLOWS FOR SEGMENT NO. 76 FOR HONEY CR. RUN NO 6

WATER YEAR	PEAK FLOW(CFS)	YEAR	MONTH	DAY
1918	265.0	1918	APR.	7
1919	737.6	1919	MAY	6
1920	1568.9	1920	MAR.	12
1921	1622.7	1921	AUG.	24
1922	1342.5	1921	NOV.	16
1923	1292.3	1923	FEB.	13
1924	643.8	1924	MAY	27
1925	1019.6	1925	JAN.	11
1926	1761.4	1926	AUG.	11
1927	621.0	1927	FEB.	23
1928	1963.6	1928	JULY	10
1929	870.7	1929	MAR.	14
1930	419.7	1930	MAY	17
1931	399.1	1931	JULY	28
1932	939.6	1932	JUNE	18
1933	964.0	1933	JUNE	10
1934	737.2	1934	JULY	19
1935	359.9	1934	OCT.	5
1936	905.3	1936	SEPT	29
1937	914.2	1937	APR.	29
1938	1346.6	1938	JUNE	25
1939	880.2	1939	JUNE	22
1940	1289.3	1940	SEPT	10
1941	830.4	1941	AUG.	13
1942	1076.1	1942	MAR.	20

MEAN = 990.8 STANDARD DEVIATION = 445.7

FLOOD FREQUENCY FOR FLOWPOINT 76 FOR HONEY CR.

RETURN PERIOD	PROBABILITY*	FLOW IN CFS
2-YEAR	50.0	917.6
5-YEAR	20.0	1311.5
10-YEAR	10.0	1572.3
25-YEAR	4.0	1901.7
50-YEAR	2.0	2146.2
100-YEAR	1.0	2388.8
200-YEAR	0.5	2630.6

*PERCENT CHANCE IN ANY YEAR OF GETTING A FLOOD GREATER THAN THAT INDICATED IN THE TABLE.

V. Summary and Conclusions

It was found that all development schemes had a greater effect on the more frequent floods than the larger, less frequent floods. Increases in the 2-year flood at the lower end of the study, reach 76, ranged from 19% to 45%. The 100-year flood increased from 9% to 24%.

The inclusion of a storage segment on a tributary in the lower part of the watershed showed that the storage reduced the mean peak flow in the tributary by 25% but decreased the flow downstream on the main stream of Honey Creek only 1%. Table 4-27 summarizes the results of the simulation runs for reaches 46, 74, and 76. Each projected development pattern can be compared with existing development. Table 4-28 shows the results of placing a storage segment upstream of reach 74.

Table 4-27

Summary of Simulation Runs with No Storage

Simulation	Reach 46 Discharge (CFS)		Reach 74 Discharge (CFS)		Reach 76 Discharge (CFS)	
	2-yr.	100-yr.	2-yr.	100-yr.	2-yr.	100-yr.
Existing Development	406	1190	367	1057	771	2195
Complete Development	594	1480	557	1431	1116	2723
Lower Half Development	406	1190	557	1431	955	2763
Upper Half Development	594	1480	367	1057	943	2444
"Windswept" Development	471	1270	418	1150	918	2389

Table 4-28

Summary of Simulation Runs

With and Without Storage Segment 73

Simulation	Reach 46 Discharge (CFS)		Reach 74 Discharge (CFS)		Reach 76 Discharge (CFS)	
	2-yr.	100-yr.	2-yr.	100-yr.	2-yr.	100-yr.
Complete Development	594	1480	557	1431	1116	2723
Complete Development with storage Segment 73	594	1480	415	1162	1046	2632

SECTION 5

Womack Creek

I. Physical Attributes of Watershed

a. Location

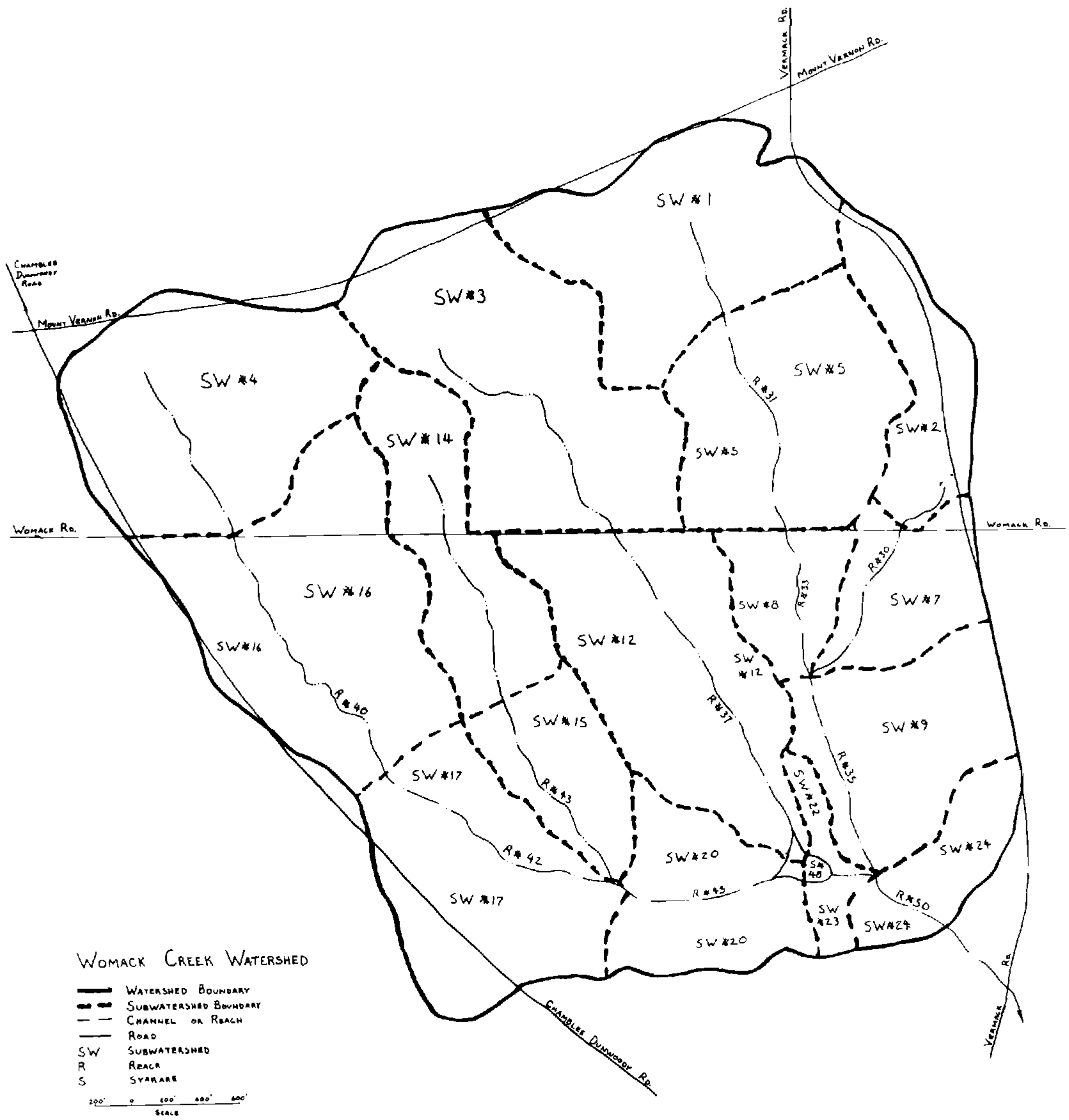
The headwater area of Womack Creek is examined in this study. The stream rises near the intersection of Mount Vernon and Chamblee-Dunwoody roads in Northwest DeKalb County and flows southeast to join Nancy Creek. Most of the area enclosed by Mt. Vernon, Vermack and Chamblee - Dunwoody Roads drains into the Creek (Figure 5-1).

b. Size

The watershed has a total area of 420 acres, most of which is developed as single family residential units. Womack Road, the only major road passing through the watershed, crosses the Creek and its tributaries at five places. Tributaries are also crossed at seven other places by streets serving residential units. Table 5-1 lists the drainage areas and culvert sizes at these crossings.

c. General Drainage Patterns

As shown on Figure 5-2, about 80 percent of the watershed is developed with most of the undeveloped areas located along the roads at the watershed divide. The major drainageways remain natural channels. Main stream channel widths vary from about 23 feet at the lower end of the study area to about 2 to 3 feet in the upper reaches. The main channel has a length of about 6100 feet from the divide in subwatershed number 4 (near Mr. Vernon Road) to the lower study limits. The main stream flows southeast along the western boundary of the watershed and then east along the south boundary where three major tributaries join from the north.



Womack Creek Watershed
Watershed Boundary
Subwatershed Boundary
Channel or Reach
Road
SW Subwatershed
R Reach
Mount Vernon Rd.
Vermack Rd.
Chamblee Dunwoody Road
Womack Rd.
Chamblee Dunwoody Rd.
SW#1
SW#2
SW#3
SW#4
SW#5
SW#7
SW#8
SW#9
SW#12
SW#14
SW#15
SW#16
SW#17
SW#20
SW#22
SW#23
SW#24
R#30
R#31
R#33
R#35
R#37
R#40
R#42
R#43
R#45
R#50

FIGURE 5 - 1

TABLE 5-1

Drainage Areas at Road Crossings

Road Crossing	Drainage Area (Acres)	Bridge Opening
Womack Road - 1	45	*
Womack Road - 2	9	*
Womack Road - 3	45	*
Womack Road - 4	66	*
Womack Road - 5	16	*
Springfield Dr. (U/S of confluence)	22	*
Springfield Dr. (D/S of confluence, S44)**	160	two 5' dia C.M.P.
Courtleigh Dr. (S38)	84	6.5' dia C.M.P. partly silted.
Cambridge Dr. (S48)	266	two 6' dia C.M.P. partly silted.
Parliament Dr.	23	*
Leeds Court	30	*
Leeds Way (S34)	110	5.5' dia, C.M.P.

* Drainage areas at these crossings are small for the size of low relative to the bridge openings, and no backwater storage effects occur.

** For locations of S34, S38, S44 and S48, refer to Plate 1.

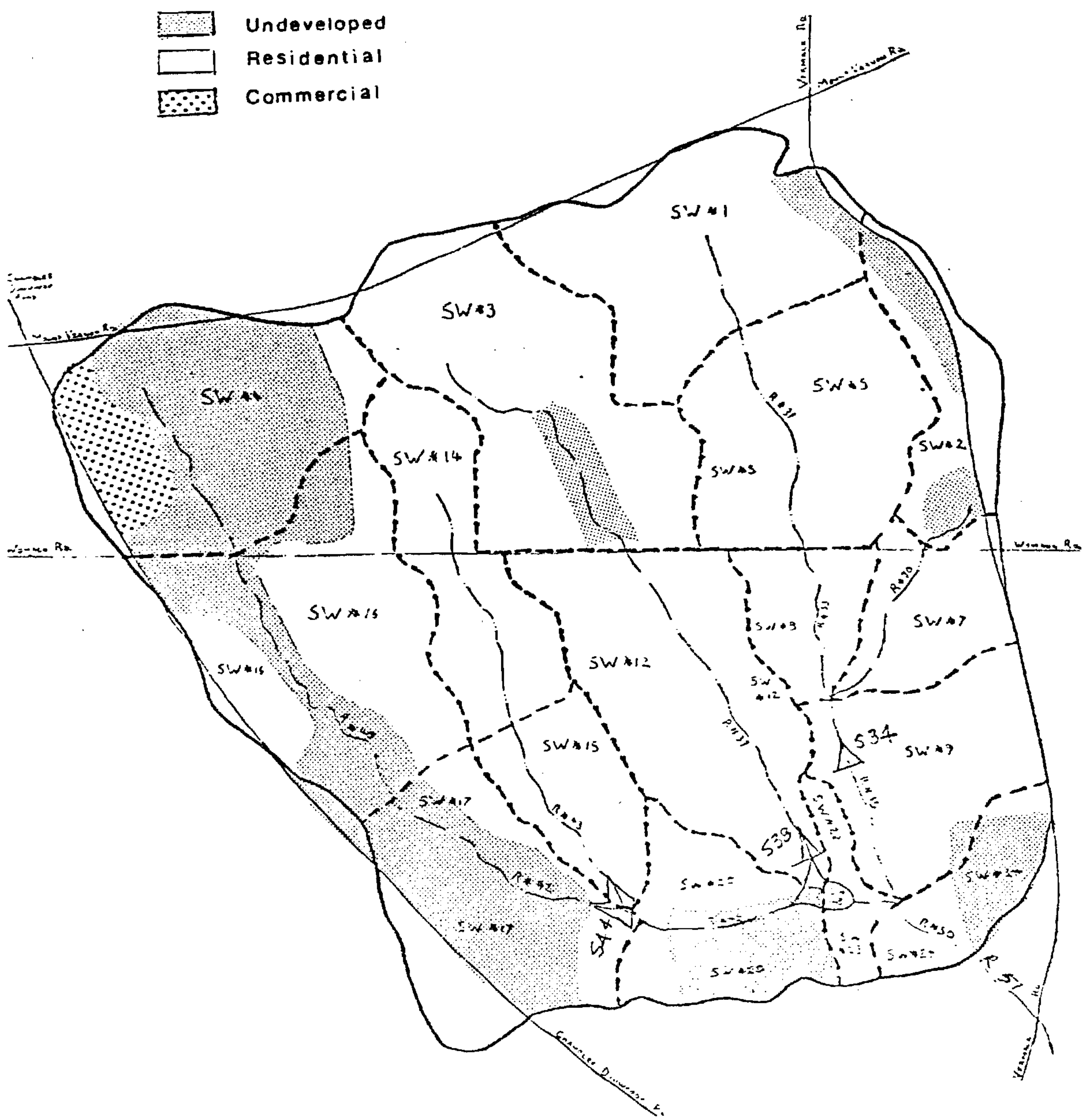


Figure 5-2. Map of Land Use for Womack Creek

d <u>Other Studies</u>

The U. S. Army Corps of Engineers completed a special Flood Hazard Information Report including the Nancy Creek Basin in DeKalb County in October, 1971, but the coverage did not extend into the Womack basin.

e <u>Current Land Use</u>

The area is about 80 percent developed. Most of the development is single family residences, but a small area is developed for commercial use in the extreme northwest portion of the watershed. Subwatershed impervious areas range from 5 to 25 percent of their total areas.

f <u>Projected Land Use</u>

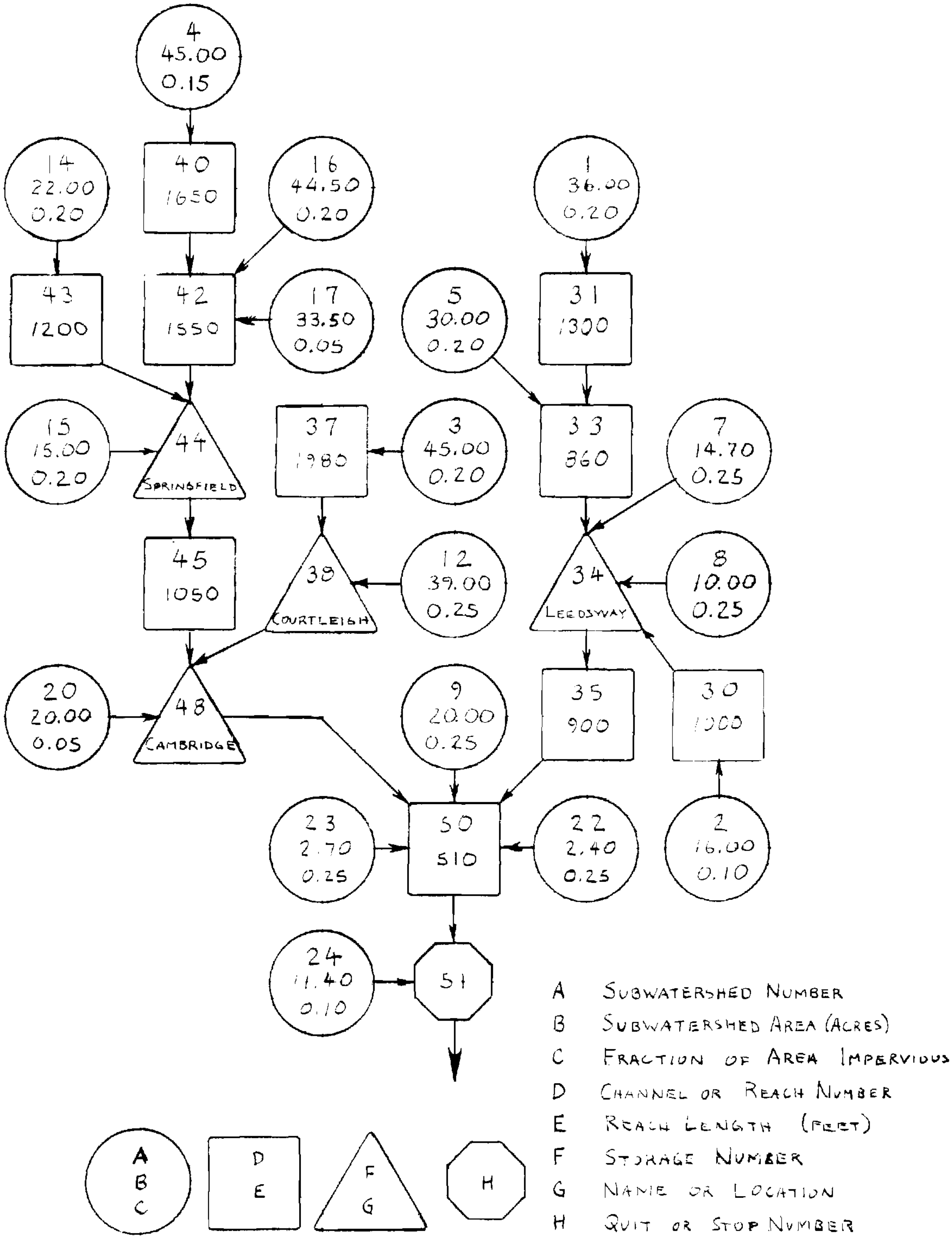
The patches of undeveloped land remaining in various parts of the watershed were projected to be developed for single family residences except for portions of subwatershed 4 which were projected for commercial development. Although parts of subwatersheds 16 and 17 could be developed for commercial uses or part of subwatershed 4 for residential use, the above projection provides a quick and reasonable approximation.

II. Map of Subwatersheds.

Figure 5-1 shows the overall drainage pattern, stream locations, and basin subareas. The watershed area was subdivided for analysis into 17 subwatersheds, and the drainageways were divided into 10 channel segments. The culverts on Leeds Way and Courtleigh, Springfield and Cambridge Drives are likely to cause some backwater effect. The drainage areas at the other culverts (Table 5-1) are small and the culvert sizes are sufficient to cause little detention of storm runoff. A schematic diagram of the segments is shown in Figure 5-3.

III. Drainage Problems.

The drainage problems in the area are concentrated in the general



WOMACK CREEK SCHEMATIC DIAGRAM

FIGURE 5-3

area of the intersection of Courtleigh with Cambridge Drive as shown on the sketch in Figure 5-4. The flood of June, 1974 overtopped the Cambridge Drive culvert (Point B). The house at Point A had 3 feet of water at its back and 1 foot at its front. Two 72" corregated metal pipes at Point D were one third filled with silt at the time surveyed. There was some erosion at Point D. A courtyard flooded at House H. The creek appeared to be silting at Point C but eroding at Point E. The channel is constricted at Point F. At Leeds Way the channel is badly eroded (see Point G).

IV. Description of Simulation Runs

1. Existing Development and Channel System

The initial simulation sought to determine flood flows and flood stages with the existing land use and flood stages in the watershed. The specific purpose was to estimate expected flood levels for various return periods at the problem locations. Table 5-2 shows the specifications for the source areas, channel segments, and the four storage segments. Table 5-3 shows the frequency analyses for channel segment 35, storage segments 48. The culvert on Cambridge Drive (Road elevation 980.0) is overtopped by a 10-year flood and that the 100-year flood crosses the road at a depth greater than 1.0 foot. The results for Reach 35 indicate that the 25-year flood has a depth of water of 4.1 feet and that the 100-year flood flows at a depth of 4.5 feet. The maximum depth of water than can be contained within that channel is only about 3 feet.

2. Projected development and existing channel system

This simulation is based on complete watershed development and on an assumption that the current channel system will remain unchanged from present conditions. The projected development for subwatershed 4

Figure 5-4. Map of Problem Areas on Womack Creek

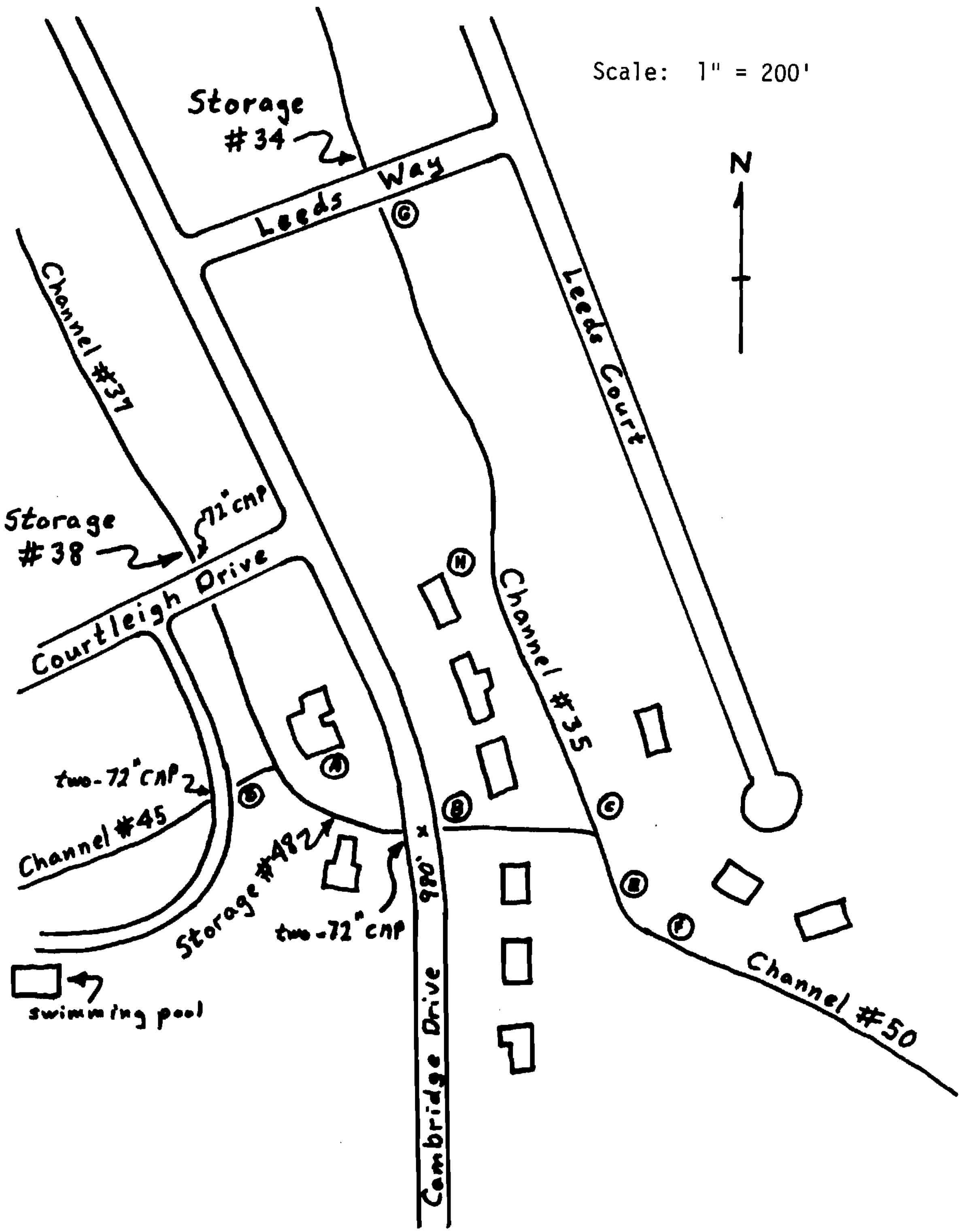


Table 5-2. Specifications for Source Areas and Channel Segments for Run 1

SPECIFICATIONS FOR SOURCE AREAS

		FRACTION OF AREA WITH EACH SOIL TYPE			
NUMBER	AREA-ACRE	RAFID	MODERATE	SLOW	IMPERVICUS
1	36.000	.000	.800	.000	.200
2	16.000	.000	.900	.000	.100
3	45.000	.000	.800	.000	.200
4	45.000	.000	.850	.000	.150
5	30.000	.000	.800	.000	.200
7	14.700	.000	.750	.000	.250
8	10.000	.000	.750	.000	.250
9	20.100	.000	.750	.000	.250
12	39.000	.000	.750	.000	.250
14	22.000	.000	.800	.000	.200
15	15.000	.000	.800	.000	.200
16	44.800	.000	.800	.000	.200
17	33.500	.000	.950	.000	.050
20	20.000	.000	.950	.000	.050
22	2.400	.000	.750	.000	.250
23	2.700	.000	.750	.000	.250
24	11.400	.000	.900	.000	.100

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS
30	4	1000.00	.03750	.035
31	4	1300.00	.03846	.035
33	4	860.00	.02326	.035
35	4	900.00	.01220	.035
37	4	1980.00	.01768	.035
40	4	1650.00	.03455	.035
42	4	1550.00	.01613	.040
43	4	1200.00	.03750	.035
45	4	1050.00	.00714	.030
50	4	510.00	.02350	.030

SPECIFICATIONS FOR STORAGE SEGMENT 34

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	980.000	.000	.000
.023	40.649	981.000	1.000	.046
.098	81.297	982.000	2.000	.110
.240	121.946	983.000	3.000	.165
.429	162.595	984.000	4.000	.220
.715	203.243	985.000	5.000	.376
1.173	233.509	986.000	6.000	.505
1.728	252.218	987.000	7.000	.643
2.458	269.633	988.000	8.000	.781
3.290	409.769	989.000	9.000	.918
4.299	782.332	990.000	10.000	1.067

Table 5-2 (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 38

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	978.000	.000	.000
.032	41.980	979.000	1.000	.064
.138	83.960	980.000	2.000	.156
.377	125.939	981.000	3.000	.344
.845	167.919	982.000	4.000	.588
1.506	192.376	983.000	5.000	.693
2.232	210.737	984.000	6.000	.799
3.153	351.083	985.000	7.000	1.051
4.259	724.984	986.000	8.000	1.074
5.463	1375.909	987.000	9.000	1.579

SPECIFICATIONS FOR STORAGE SEGMENT 44

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	981.000	.000	.000
.009	70.467	982.000	1.000	.018
.038	140.935	983.000	2.000	.039
.087	211.402	984.000	3.000	.060
.158	281.869	985.000	4.000	.083
.302	352.336	986.000	5.000	.253
.675	385.965	987.000	6.000	.459
1.220	416.890	988.000	7.000	.666
2.014	459.044	989.000	8.000	.895
3.016	515.253	990.000	9.000	1.139
4.268	568.038	991.000	10.000	1.309
5.638	716.205	992.000	11.000	1.492
7.253	1091.819	993.000	12.000	1.676
8.990	1753.659	994.000	13.000	1.860
10.985	2752.734	995.000	14.000	2.080
12.979	3751.810	.000	.000	.000

SPECIFICATIONS FOR STORAGE SEGMENT 48

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	973.500	.000	.000
.014	37.817	974.000	.500	.055
.087	113.450	975.000	1.500	.129
.272	189.084	976.000	2.500	.207
.566	264.717	977.000	3.500	.542
1.393	320.885	978.000	4.500	.987
2.573	354.752	979.000	5.500	1.469
4.334	400.118	980.000	6.500	1.956
6.531	572.946	981.000	7.500	2.577
9.488	959.265	982.000	8.500	3.197
12.925	1624.222	983.000	9.500	3.816
17.122	2626.181	984.000	10.500	4.436
21.800	4009.697	985.000	11.500	5.060

Table 5-3. Flood Frequency for Segments 35 and 48 on Womack Creek

Return Period (years)	Probability of Exceedance (%)	Flood Flow (cfs)	Water Surface Elevation (feet)
Segment 35			
2	50	130	3.1[1]
5	20	190	3.6
10	10	230	3.8
25	4	280	4.1
50	2	320	4.3
100	1	350	4.5
Segment 48			
2	50	250	976.9[2]
5	20	360	979.3
10	10	440	980.3
25	4	530	980.8
50	2	600	981.1
100	1	670	981.3

[1] From bottom of channel.

[2] From mean sea level.

is commercial with the rest of the watershed residential. It is assumed that 25% of the residential development and 100% of commercial development will be impervious. Comparison of existing development with projected development shows, for segment 48 (at the culvert on Cambridge Drive), a 27% increase in the 2-year flood and a 19% in the 100-year flood. These increases would cause about a one-foot increase in elevation of the 2-year flood and about a 0.3 foot increase in the 100-year flood. With the projected development a 5-year flood would overtop the culvert. The flow in Reach 35 increases by less than 5 percent, and stages increase by less than 0.1 foot.

3. Conclusions

Several alternatives to reduce the flooding around the Cambridge Road area remain to be tested with the model. Enlargement of the culvert at Cambridge Drive and some channel improvements above and below the road are possibilities that may offer some relief. The erosion and sediment problems are a result of the channel system readjusting to the new land use on the watershed. Efforts at bank stabilizations and better energy disipation at culverts could help with some of the problems.

SECTION 6

West Branch Snapfinger Creek

(Susan Creek)

I. Physical Attributes of Watershed

a. Location

The west branch of Snapfinger Creek, which was examined in this study, rises in the western part of the city of Clarkston and flows southeast until it joins a tributary which drains the northeast portion of the study area. The creek then flows south until it reaches DeKalb Junior College and Technical High School where it makes a sharp bend to the left to pass under Memorial Drive (Figure 6-1). Section 6 deals with the drainage problems upstream from Memorial Drive,and problems further downstream are discussed in Section 7.

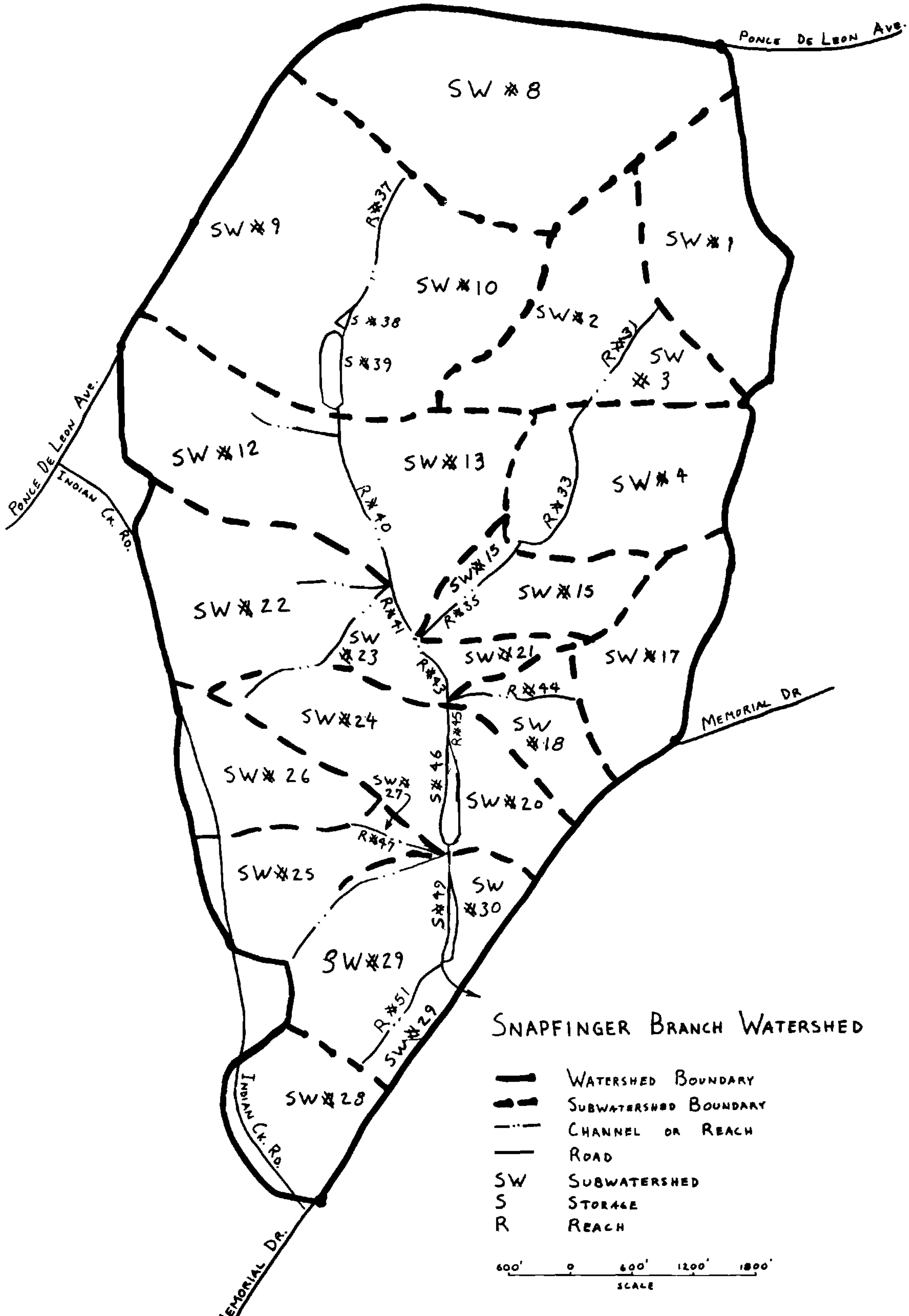
The area enclosed by Ponce De Leon Avenue on the north, Memorial Drive on the south, Rays Road on the East, and the Indian Creek Road on the west drains into the creek.

b. Size

The watershed has a total area of about 1200 acres, 23 percent of which is impervious. Six roads cross the creek or its major tributaries. Table 6-1 lists the drainage areas and the size of the culverts at these crossings. The capacity estimates are culvert discharges under the head at which flows just begin to overtop the road.

c. General Drainage Patterns

The drainage configuration is shown on Figure 6-2. About 85 percent of the basin is developed, and the drainage is in natural



SNAPFINGER BRANCH WATERSHED
WATERSHED BOUNDARY
SUBWATERSHED BOUNDARY
CHANNEL OR REACH
ROAD
SW SUBWATERSHED
S STORAGE
R REACH
600' 0 600' 1200' 1800'
SCALE
PONCE DE LEON AVE.
INDIAN CK. RD.
MEMORIAL DR.
SW #8
SW #9
SW #10
SW #1
SW #2
SW #3
SW #4
SW #12
SW #13
SW #15
SW #17
SW #18
SW #20
SW #21
SW #22
SW #23
SW #24
SW #25
SW #26
SW #27
SW #28
SW #29
SW #30
S #38
S #39
S #46
S #49
R #31
R #33
R #35
R #37
R #40
R #41
R #43
R #44
R #45
R #47
R #51

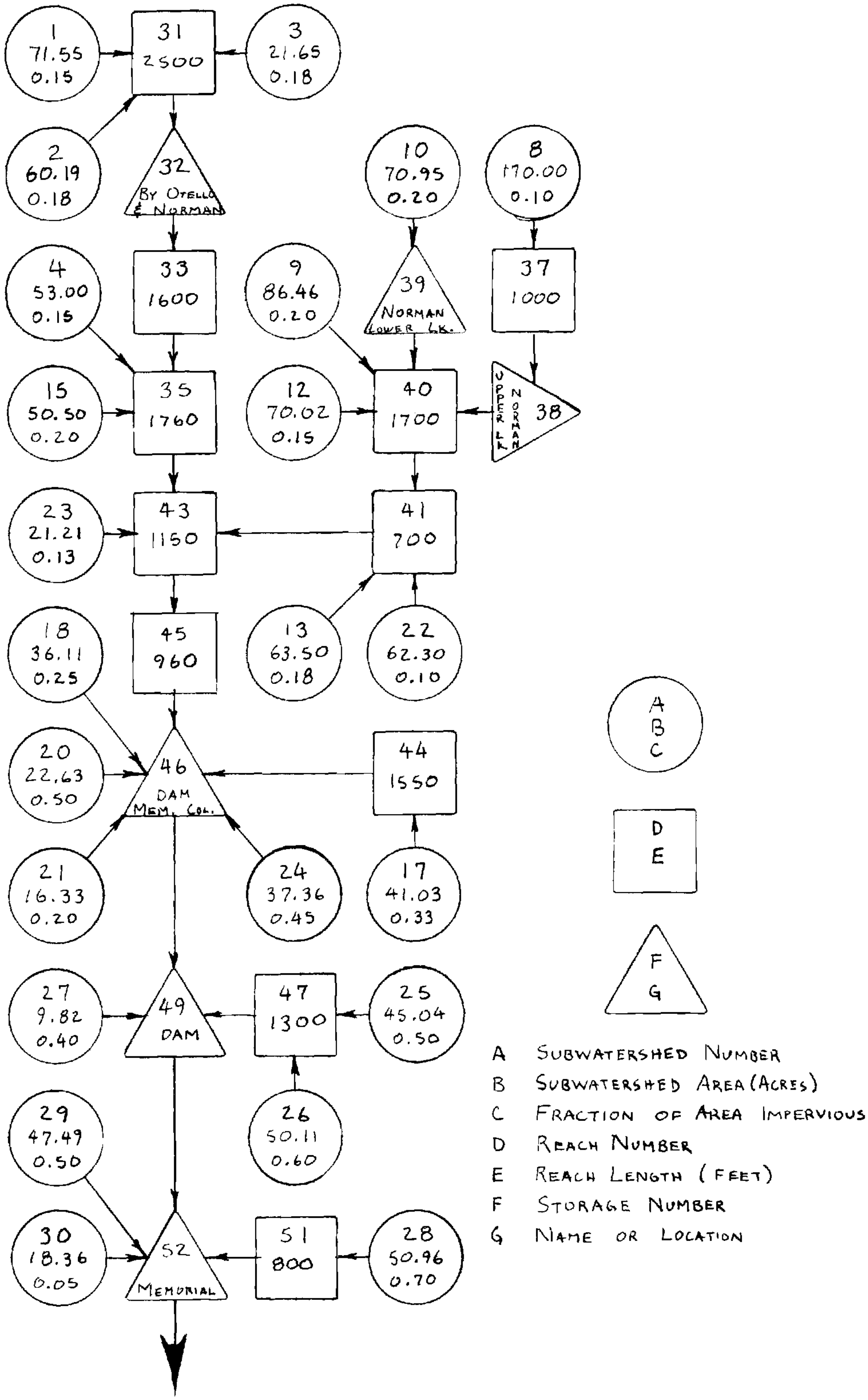
FIGURE 6-1

TABLE 6-1

Drainage Areas at Culverts

Road	Drainage Area (Acres)	Culvert Description	Capacity (cfs)
Norman Road	327.4	The area drains into Storages 38 and 39 which have multiple outlets draining water into Reach 40 (Figure 6-1)	
Norman Rd. (on trib.) (S32)	153.4	1-54" Circular Pipe 1-48"x42" Elliptical Pipe	393
Cunlieth Rd.	437.	2-84"x72" Rectangular	*not simulated
Holly Hedge Rd.	257.	2-72"x60" Rectangular	*not simulated
Memorial College Ave.	1038.	2-120"x96" Rectangular	*not simulated
Memorial Dr. (S52)	1200.	2-96"x96" Rectangular	1743

*Culvert size is large enough so as not to cause potential backwater effect.



Snapfinger Branch Schematic Diagram

Figure 6 - 2

channels. The main channel has a length of about 8500 feet from the point it leaves subwatershed 8 to Memorial Drive. The tributary in the northeast portion of the basin has a length of 3600 feet from the point it leaves subwatershed 1 to the junction. The channel width varies from about 10 feet in the upper reaches to about 20 feet in the lower reaches. Almost complete development of the basin (about 95%) and the relatively flat slope of Reach 49 have contributed to the flooding.

d. Other Studies

The U.S. Army Corps of Engineers completed a Flood Plain Information Report on the Snapfinger Creek flood plain in March, 1968, but only a small portion of the West Branch flood plain is included in that study. The mapped 100-year flood plain in Corps'study indicates that the culvert on Memorial Drive can safely pass the 100-year flood and the flood waters extend to about 150 to 250 feet on either side of the Creek in Reach 49.

e. Current Land Use

As of 1975, 85 percent of the watershed has been developed. Multi-family apartment complexes, DeKalb Junior College, and Technical High School occupy the Southern part of the Watershed. Single family houses on quarter to half acre lots occupy most of the remaining area. Some commercial property and a multi-family apartment complex are also located along Ponce De Leon Avenue in the northwestern portion of the watershed. Subwatershed impervious percentages range from 5 to 70% of their total area.

f. Projected Land Use

It is expected that the patches of undeveloped land remaining in various parts of the watershed (Figure 6-1) will be eventually developed. It is projected that the undeveloped areas located along major roads will be developed as multi-family apartment complexes and commercial property and the interior areas as single family houses.

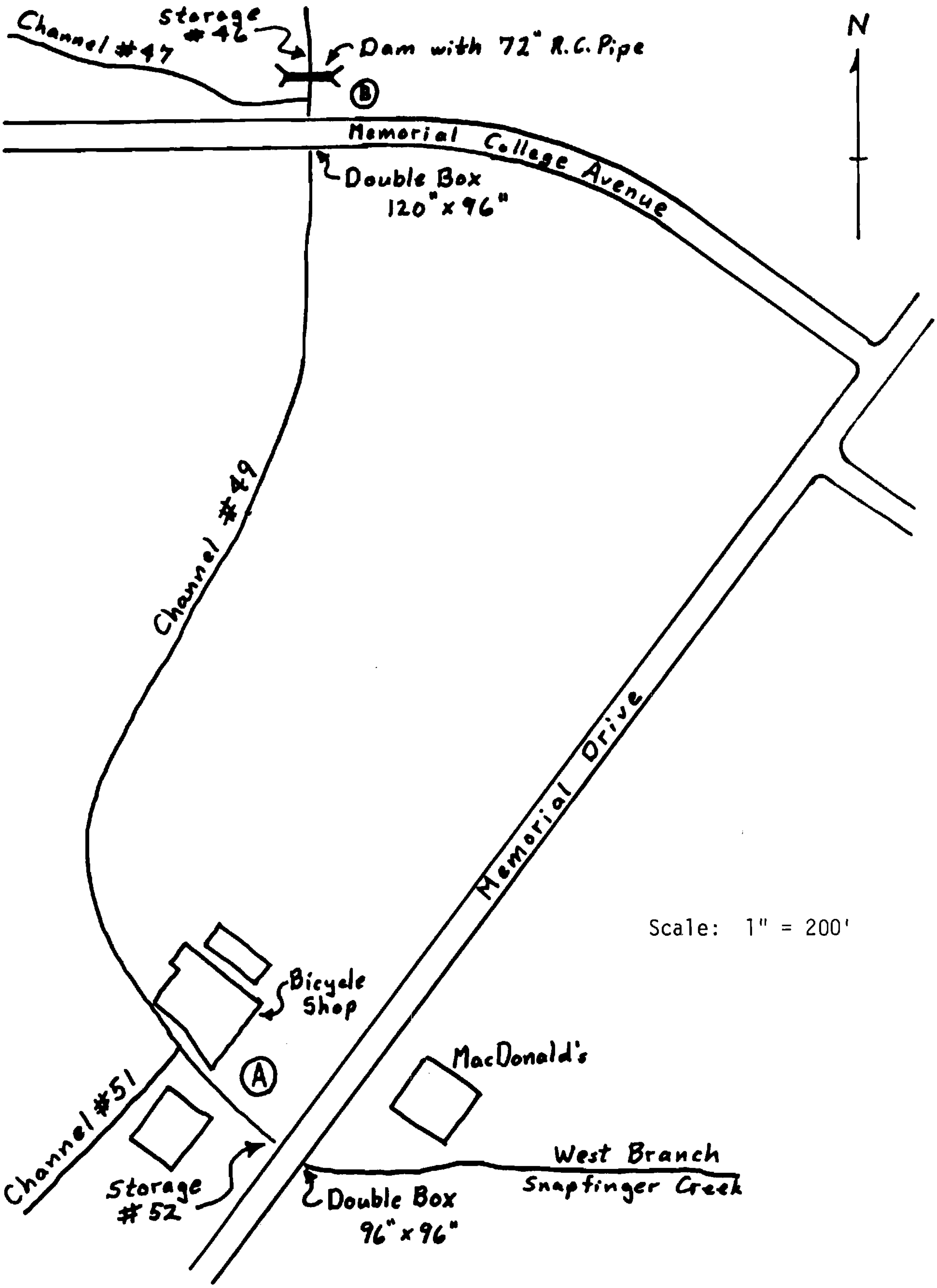
II. Map of Watershed

Figure 6-1 outlines the total watershed and the sub-areas into which it is divided for analysis. Major drainageways are shown and divided into channel segments. Two road crossings listed in Table 6-1 involve culverts small enough to have a potential backwater effect, and a storage segment was added for each. In addition, the storage effects of two existing lakes (S-38 and S-39) and an existing reservoir (S-46) were modeled. A schematic diagram of the segments of all three types is shown on Figure 6-2.

III. Drainage Problems

The drainage problems in the watershed are concentrated in the area immediately upstream of Memorial Drive at Point A, Storage 52, Reach 51, and Reach 49 as shown on Figure 6-3. Approximately 5 businesses in this area were flooded during the June, 1973, floods. The bicycle shop on left bank was flooded to a depth of 9 inches over the main floor, and flood proofing efforts by the owner probably reduced the depth over what it was outside. The small manmade channel with vertical sides downstream from the junction of Reaches 49 and 51 constricts the flow, contributes to upstream backwater storage, and is frequently overtopped.

Figure 6-3. Map of Problem Areas on Snapfinger Branch



Memorial Drive was not overtopped in June, 1973, and downstream conditions do not create any backwater upstream of Memorial Drive.

A small retention structure was located about 100 feet upstream of Memorial College Avenue (See point B on Figure 6-3).

Specific problems studies were

1) Extent of flooding in the problem areas with current development and channel conditions,

2) Overtopping culverts within the watershed,

3) Potential aggravation of the problems by additional urban development, and

4) Reduction of the present flooding problem by improving channels and adding storages.

IV. Description of Simulation Runs.

1. Existing Development and Channel System

The initial simulation sought to determine flood flows and flood elevations with the existing land use and channel conditions in the watershed. The specific purpose was to estimate expected flood levels for various return periods at the problem locations and near the culverts and storages. Table 6-2 shows the specifications for the source areas, channel segments and storage segments. Table 6-3 shows the results of the flood frequency analysis for storage segments 46 and 52. Results for other storage segments are not shown in this report, but they were examined and found not to pose a problem.

The results indicate that the storage provided by the retention

Table 6-2. Specifications for Area, Channel and Storage Segments for Existing Conditions

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	RAPID	MODERATE	SLOW	IMPERVIOUS
		FRACTION OF AREA WITH EACH SOIL TYPE			
1	71.550	.000	.850	.000	.150
2	60.190	.000	.820	.000	.180
3	21.650	.000	.820	.000	.180
4	53.000	.000	.850	.000	.150
8	170.000	.000	.900	.000	.100
9	86.460	.000	.800	.000	.200
10	70.950	.000	.800	.000	.200
12	70.020	.000	.850	.000	.150
13	63.500	.000	.820	.000	.180
15	50.500	.000	.800	.000	.200
17	41.030	.000	.670	.000	.330
18	36.110	.000	.750	.000	.250
20	22.630	.000	.500	.000	.500
21	16.330	.000	.800	.000	.200
22	62.300	.000	.900	.000	.100
23	21.210	.000	.870	.000	.130
24	37.360	.000	.550	.000	.450
25	45.040	.000	.500	.000	.500
26	50.110	.000	.400	.000	.600
27	9.820	.000	.600	.000	.400
28	50.960	.000	.300	.000	.700
29	47.490	.000	.500	.000	.500
30	18.360	.000	.950	.000	.050

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS	AVERAGE TRAVEL TIME(SEC)
31	4	2500.00	.01120	.025	242.3
33	4	1600.00	.00623	.025	137.8
35	4	1760.00	.01990	.025	129.5
37	4	1000.00	.02000	.040	116.0
40	4	1700.00	.01177	.040	242.9
41	4	700.00	.01000	.035	100.2
43	4	1150.00	.00435	.035	234.8
44	4	1550.00	.02581	.035	152.2
45	4	960.00	.00323	.035	329.9
47	4	1300.00	.01690	.035	143.6
49	4	1400.00	.00286	.035	511.0
51	4	800.00	.01250	.035	112.9

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

Table 6-2 (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 32

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	958.000	.000	.000
.011	52.669	959.000	1.000	.023
.098	158.006	961.000	3.000	.103
1.329	298.893	965.000	7.000	.882
6.529	392.771	968.000	10.000	1.882
10.782	1280.893	970.000	12.000	2.929

SPECIFICATIONS FOR STORAGE SEGMENT 38

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	947.500	.000	1.446
1.481	12.686	948.500	1.000	1.515
2.262	18.106	949.000	1.500	1.538
3.788	65.308	950.000	2.500	1.561
5.384	227.106	951.000	3.500	1.584
6.957	582.034	952.000	4.500	1.607

SPECIFICATIONS FOR STORAGE SEGMENT 39

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	939.000	.000	3.600
3.620	50.671	940.000	1.000	3.640
7.281	124.915	941.000	2.000	3.680
10.981	213.109	942.000	3.000	3.720
14.723	1034.877	943.000	4.000	3.760
18.503	2658.194	944.000	5.000	3.800

Table 6-2 (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 46

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	900.500	.000	.000
.250	115.790	903.000	2.500	.200
.700	162.105	904.000	3.500	.400
1.048	208.421	905.000	4.500	.593
2.041	254.737	906.000	5.500	1.250
3.876	289.242	907.000	6.500	2.890
7.412	310.696	908.000	7.500	3.300
8.950	320.885	908.500	8.000	4.200
11.620	684.400	909.000	8.500	5.140
17.055	2326.207	910.000	9.500	7.654
45.644	12714.872	913.000	12.500	15.270

SPECIFICATIONS FOR STORAGE SEGMENT 52

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	893.500	.000	.000
.136	272.376	895.000	1.500	.181
1.105	1180.297	900.000	6.500	.294
2.779	1361.881	901.000	7.500	1.286
3.884	1497.381	902.000	8.500	2.902
8.752	1583.014	903.000	9.500	5.023
14.321	1664.246	904.000	10.500	8.310
25.615	1741.694	905.000	11.500	12.314
90.013	13008.841	910.000	16.500	18.110

Table 6-3. Floods for Segments 46 and 52 on West Branch Snapfinger Creek With Existing Conditions

Return Period (years)	Probability of Exceedance (%)	Flood Flow (cfs)	Water Surface Elevation (feet)
Segment 46			
2-year	50	660	908.9[1]
5-year	20	1040	909.2
10-year	10	1300	909.4
25-year	4	1620	909.6
50-year	2	1850	909.7
100-year	1	2090	909.9
Segment 52			
2-year	50	830	897.5[1]
5-year	20	1200	900.1
10-year	10	1440	901.9
25-year	4	1760	904.4
50-year	2	1990	905.1
100-year	1	2210	905.3

[1]From mean sea level.

structure upstream of Memorial College Avenue is not effective in reducing downstream flood peaks because of its small storage capacity. As shown by Table 6-3 the spillway crest elevation of 908.5 is overtopped by a 2-year flood by 0.6 foot and by a 100-year flood by 1.6 feet. Water spills over a length of about 350 feet. Memorial Drive is most likely to be overtopped at a saddle point of elevation 904.5 on the road to the right of the culvert by floods exceeding a 25-year return period. The depth of water over the road during these floods ranges up to 0.8 foot for a 100-year flood. At these elevations, large areas would be inundated upstream.

2. Projected Development and Existing Channel System

This simulation is based on complete watershed development and an assumption that the current channel system will remain unchanged from present conditions. Table 6-4 shows the projected specifications for source areas. Table 6-5 shows the results of the flood frequency analysis for the same segments as the first simulation. Comparison of existing development with projected development shows, for segment 52 (near the culvert on Memorial Drive) a 10% increase in the 2-year and a 2% increase in the 100-year flood discharges. With the projected development, a 25-year flood would overtop the road. At segment 46 (retention structure upstream of Memorial College Avenue), increases of 44% and 27% in the 2-year and 100-year discharges, respectively, were found with an increase of 0.2 and 0.3 foot in 2-year and 100-year elevations respectively.

3. Hydrologic Analyses of Remedial Measures based on 1928 Storm

In order to develop the hydrology required to evaluate remedial

Table 6-4. Specifications for Area Segments for Future Development

SPECIFICATIONS FOR SOURCE AREAS

		FRACTION OF AREA WITH EACH SOIL TYPE			
NUMBER	AREA-ACRE	RAPID	MODERATE	SLOW	IMPERVIOUS
1	71.550	.000	.750	.000	.250
2	60.190	.000	.750	.000	.250
3	21.650	.000	.750	.000	.250
4	53.000	.000	.750	.000	.250
8	170.000	.000	.300	.000	.700
9	86.460	.000	.650	.000	.350
10	70.950	.000	.750	.000	.250
12	79.020	.000	.750	.000	.250
13	63.500	.000	.750	.000	.250
15	50.500	.000	.750	.000	.250
17	41.030	.000	.400	.000	.600
18	36.110	.000	.250	.000	.750
20	22.630	.000	.250	.000	.750
21	16.330	.000	.750	.000	.250
22	62.300	.000	.750	.000	.250
23	21.210	.000	.750	.000	.250
24	37.360	.000	.550	.000	.450
25	45.040	.000	.500	.000	.500
26	50.110	.000	.400	.000	.600
27	9.820	.000	.600	.000	.400
28	50.960	.000	.300	.000	.700
29	47.490	.000	.500	.000	.500
30	18.360	.000	.250	.000	.750

Table 6-5. Floods for Segments 46 and 52 on West Branch Snapfinger Creek With Future Development

Return Period (years)	Probability of Exceedance (%)	Flood Flow (cfs)	Water Surface Elevation (feet)
Segment 46			
2-year	50	950	909.0[1]
5-year	20	1410	909.5
10-year	10	1710	909.7
25-year	4	2090	909.9
50-year	2	2370	910.0
100-year	1	2650	910.1
Segment 52			
2-year	50	910	897.0[1]
5-year	20	1270	900.3
10-year	10	1510	902.8
25-year	4	1810	904.8
50-year	2	2030	905.2
100-year	1	2250	905.3

[1]From mean sea level.

measures for dealing with the flood flows were simulated for the storm event producing the largest flows in the problem area for all the previous simulations, the event of July 10, 1928. The remedial measures considered were

1) Enlarge the existing detention structure just upstream of Memorial College Avenue,

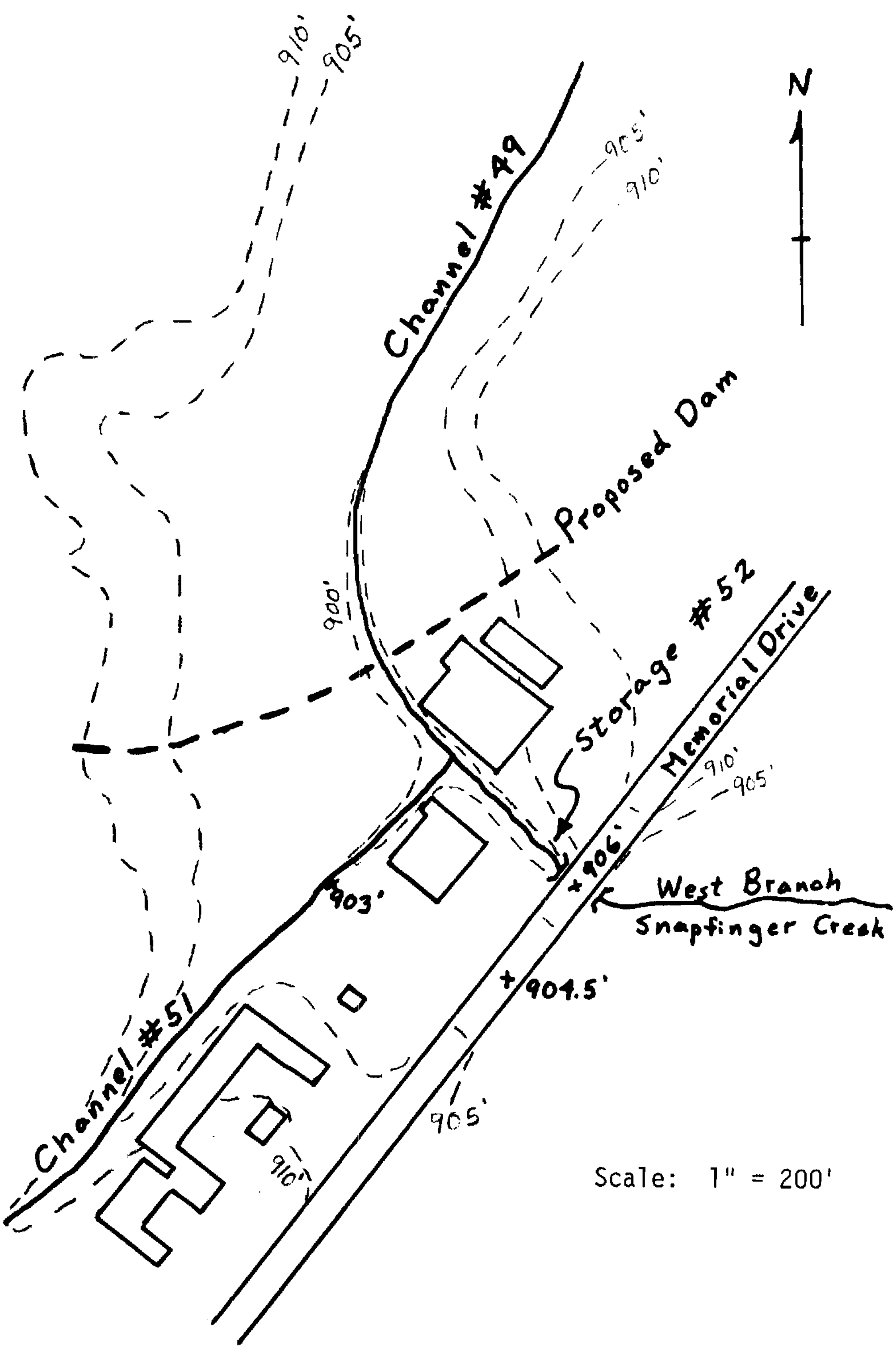
2) Provide a detention structure about 300 feet upstream of Memorial Drive (Figure 6-4),

3) Combine these two storages if each singly is not adequate.

Simulation runs made with each of the measures mentioned above. The changes in input necessary for each run are the size of the outlet works for storage segments 46 and 49 and conversion of reach 49 to storage 49. The specifications for source areas and channel segments are the same as those under Run 1. The specifications for the finally selected storage capacities at segments 46, 49, and 52 are shown in Table 6-6.

The purpose of the first series of runs was to explore whether enlargement of the detention structure just upstream of Memorial College Avenue would provide adequate protection downstream. The topography revealed that the water level behind the detention structure could rise to 913.0 with no damage to property. However, a series of 5 runs indicated that with storage at segment 46 and a design water surface elevation of 913.0, the flows released would still inundate large areas near segments 49, 51 and 52. Hence, enlargement of the detention structure just upstream of Memorial College Avenue alone would not provide the

Figure 6-4. Locations Map of Proposed Dam



Scale: 1" = 200'

Table 6-6. Specifications for Storage Segments for Detention Dams

SPECIFICATIONS FOR STORAGE SEGMENT 46

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	900.500	.000	.000
.250	74.742	903.000	2.500	.200
.700	123.810	904.000	3.500	.400
1.048	180.498	905.000	4.500	.593
2.041	243.892	906.000	5.500	1.250
3.876	289.242	907.000	6.500	2.890
7.412	310.696	908.000	7.500	3.300
8.950	320.885	908.500	8.000	4.200
11.620	444.464	909.000	8.500	5.140
17.055	949.236	910.000	9.500	7.654
45.644	3529.910	913.000	12.500	15.270

SPECIFICATIONS FOR STORAGE SEGMENT 49

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	895.000	.000	.000
1.375	269.165	900.000	5.000	.550
18.754	479.628	905.000	10.000	9.050
83.141	1666.381	910.000	15.000	12.300

SPECIFICATIONS FOR STORAGE SEGMENT 52

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	893.500	.000	.000
.136	117.942	895.000	1.500	.181
1.105	1063.905	900.000	6.500	.294
2.779	1318.636	901.000	7.500	1.286
3.884	1497.381	902.000	8.500	2.902
8.752	1583.014	903.000	9.500	5.023
14.321	1664.246	904.000	10.500	8.310
25.615	1741.694	905.000	11.500	12.314
90.013	13008.841	910.000	16.500	18.110

necessary protection downstream.

The aim of the second series of runs was to examine whether a detention structure located about 300 feet upstream of Memorial Drive would, when combined with the existing detention structure near the Memorial College Avenue, provide adequate protection. From the existing topography, it was determined that the water surface behind the detention structure could rise to an elevation of 910 with no damage to property. A series of 3 runs indicated that the detention structure contemplated was effective in reducing peak flows through the problem area. However, since enlargement of the detention structure near Memorial College Avenue is relatively easy, runs were made to evaluate the combined effect of both storages.

The most effective scheme was, first, to raise the existing detention structure near Memorial College Avenue. The initial design would have a target water surface elevation of 913.0 and provide a 100-foot wide spillway at elevation 908.5, the elevation of the existing spillway. Second, a detention structure 300 feet upstream from Memorial Drive is needed. The initial design would have a target water surface elevation of 910.0 and discharge water through a six-foot square box culvert at the channel bed and a 30 foot rectangular broad crested weir at elevation 905.0. Table 6-7 shows the results of the flood frequency analysis for storage segments 46 and 52. If these two storage detention locations were adopted, the 25-year and 100-year flood stages would fall by 5.3 and 4.6 feet respectively at storage segment 52. These reductions in flood stages would eliminate major

Table 6-7. Floods for Segments 46 and 52 on West Branch Snapfinger Creek With Added Detention Dams

Return Period (years)	Probability of Exceedance (%)	Flood Flow (cfs)	Water Surface Elevation (feet)*
Segment 46			
2-year	50	600	909.4[1]
5-year	20	950	910.0
10-year	10	1170	910.4
25-year	4	1460	910.8
50-year	2	1670	911.1
100-year	1	1890	911.3
Segment 52			
2-year	50	520	896.5[1]
5-year	20	720	897.6
10-year	10	850	898.4
25-year	4	1010	899.1
50-year	2	1130	899.9
100-year	1	1250	900.7

*The values for segment 46 are higher than those shown in Table 6-3 because more of the floodwater is being stored.

[1]From mean sea level.

flood damages at Memorial Drive for floods as large as the 100-year event.

V. Summary and Conclusions

It was found that under the existing conditions a number of buildings would be flooded just upstream of Memorial Drive even by relatively frequent storms. This is partly due to a relatively flat channel reach upstream of Memorial Drive and the constriction of the channel leading to the culvert under Memorial Drive. The present detention dam near Memorial College Avenue was found to provide insufficient storage to protect these buildings.

A detention dam about 300 feet upstream of Memorial Drive together with the enlargement of the present detention dam just upstream of Memorial College Avenue was found to reduce the 25-year and 100-year flood stages by about 5.3 and 4.6 feet respectively in the problem areas and protect the building from flooding. Comparison of Table 6-3 with Table 6-7 summarizes the effect at storage segment 52. An economic analysis should be made to determine whether the reduction in flood damage to these buildings would justify the cost of providing this detention storage. Benefits further downstream than described in this section of the report are described in the next section and should be evaluated in the analysis.

SECTION 7

Snapfinger Creek

I. Physical Attributes of Watershed

a. Location

The headwater portion of the Snapfinger Creek watershed rises in the northeast corner of the basin near Ponce de Leon Avenue. Ponce de Leon Avenue forms most of the north boundary of the basin and Memorial Drive passes through about middle of the basin in an east-west direction. Three main tributaries flow into the creek from the north, draining western, middle and eastern portions of the watershed. The west and middle branches join the creek below Memorial Drive while the east branch joins the main creek above Memorial Drive. The West Branch, discussed in Section 6, drains the western portion.

b. Size

The portion of the watershed studied has a total area of 4865 acres. Several roads cross the creek and its tributaries within the study area,and the more significant crossings are listed in Table 7-1. The drainage areas at the road crossings and the culvert sizes are also given in Table 7-1.

c. General Drainage Patterns

The basin and channel configuration is shown in Figure 7-1. About 70% of the basin is developed,and the drainage is in natural channels. The main channel has a length of about 5.2 miles and the west, middle, and east branches are about 2.0, 1.5, and 0.9 miles respectively in length. The channel width varies from about 10

TABLE 7-1

Drainage Areas at Culverts

Road	Drainage Area (Acres)	Culvert Description
Central Drive (Main Creek in the East	486	**
N. Hairston Rd.	752	2-10' x 10' boxes
Central Drive (East Br.)	449	2-5' x 6' boxes
Memorial Dr. (Main creek in the East)	1470	2-16' x 8' boxes
Abingdon Dr.	371	1-8' x 10' Elliptical 1-1.5' Circular
Memorial Dr. (Central Br.)	536	**
Memorial College Ave. (West Br.)	1038	2-10' x 8' boxes
Memorial Dr. (West Br.)	1200	2-8' x 8' boxes
Beaver Road	2011	2-7' Circular
Rock Bridge Road	4350	**Bridge 15'-20' x 70'

** Culvert size is large enough so as not to have significant storage effects from these culverts.

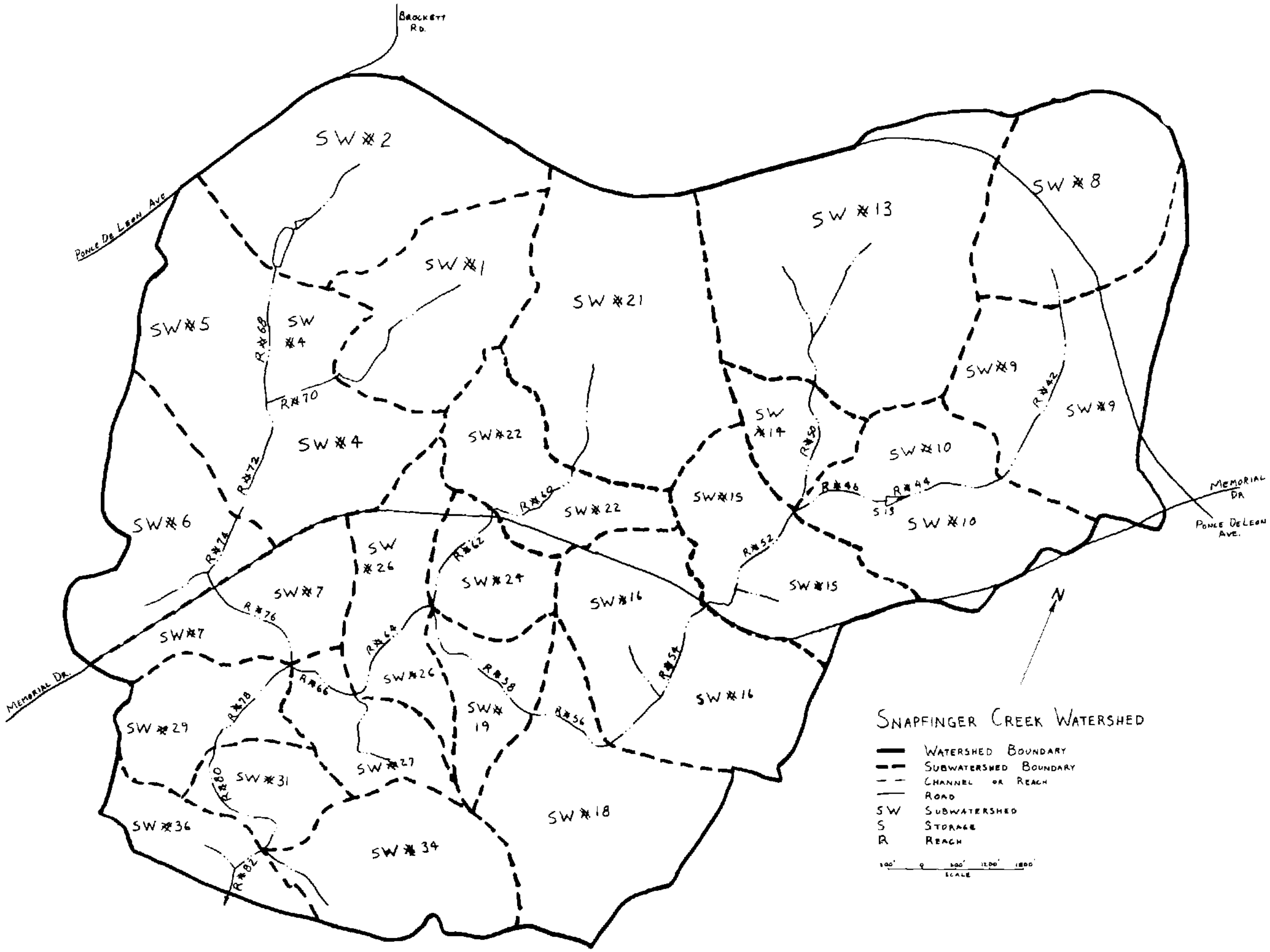

BROCKETT RD.
PONCE DE LEON AVE.
MEMORIAL DR.
SW #2
SW #1
SW #21
SW #13
SW #8
SW #5
SW #4
SW #9
SW #22
SW #14
SW #10
SW #6
SW #15
SW #26
SW #24
SW #16
SW #7
SW #29
SW #19
SW #27
SW #31
SW #36
SW #34
SW #18
R #68
R #70
R #72
R #74
R #76
R #78
R #80
R #82
R #64
R #66
R #62
R #60
R #58
R #56
R #54
R #52
R #50
R #46
R #44
R #42
S 13
SNAPFINGER CREEK WATERSHED
WATERSHED BOUNDARY
SUBWATERSHED BOUNDARY
CHANNEL OR REACH
ROAD
SW SUBWATERSHED
S STORAGE
R REACH
SCALE

FIGURE 7-1

feet in the upper reaches to about 30 feet in the lower reaches.

d. Other studies

The U. S. Army Corps of Engineers completed a Flood Plain Information Report on the entire Snapfinger Creek flood plain including Barbashela and Indian Creeks, in March, 1968. The watershed shown on Figure 7-1 comprises the upper 20% of the area studied by the Corps. While the Corps' study did not provide detailed information on the 100-year flood flows for the different reaches, the areas delineated as the 100-year flood plain indicated that about 18 houses in reaches 66, 76, 78, 80 and 82 would be inundated. This agrees with the results of this study. The Corps study estimates the 100-year discharge at the Redan Road Bridge (about 1.2 miles downstream of Reach 82 and with a tributary areas of 8450 acres) as 5920 cfs. This discharge compares with the 100-year discharge of 5590 cfs in Reach 82 for 4865 acres found in the study. The two figures are in reasonable agreement considering that much of the basin was undeveloped at the time of the Corps' study.

e. Current Land Use

About 70% of the watershed is already developed. Half of the undeveloped area lies along the three major roads passing through the basin, namely Ponce de Leon Avenue, Central Drive and Memorial Drive. While most current development is single family residence, some commercial property and multi-family apartment complexes are located along the three major roads. Subwatershed impervious percentages range from 7 to 60% of their total area.

f. Projected Land Use

It is expected that the patches of undeveloped land remaining in various parts of the watershed will be eventually developed It is projected that the undeveloped areas located along major roads will be developed as multi-family apartment complexes or commercial property and the interior areas as single family houses.

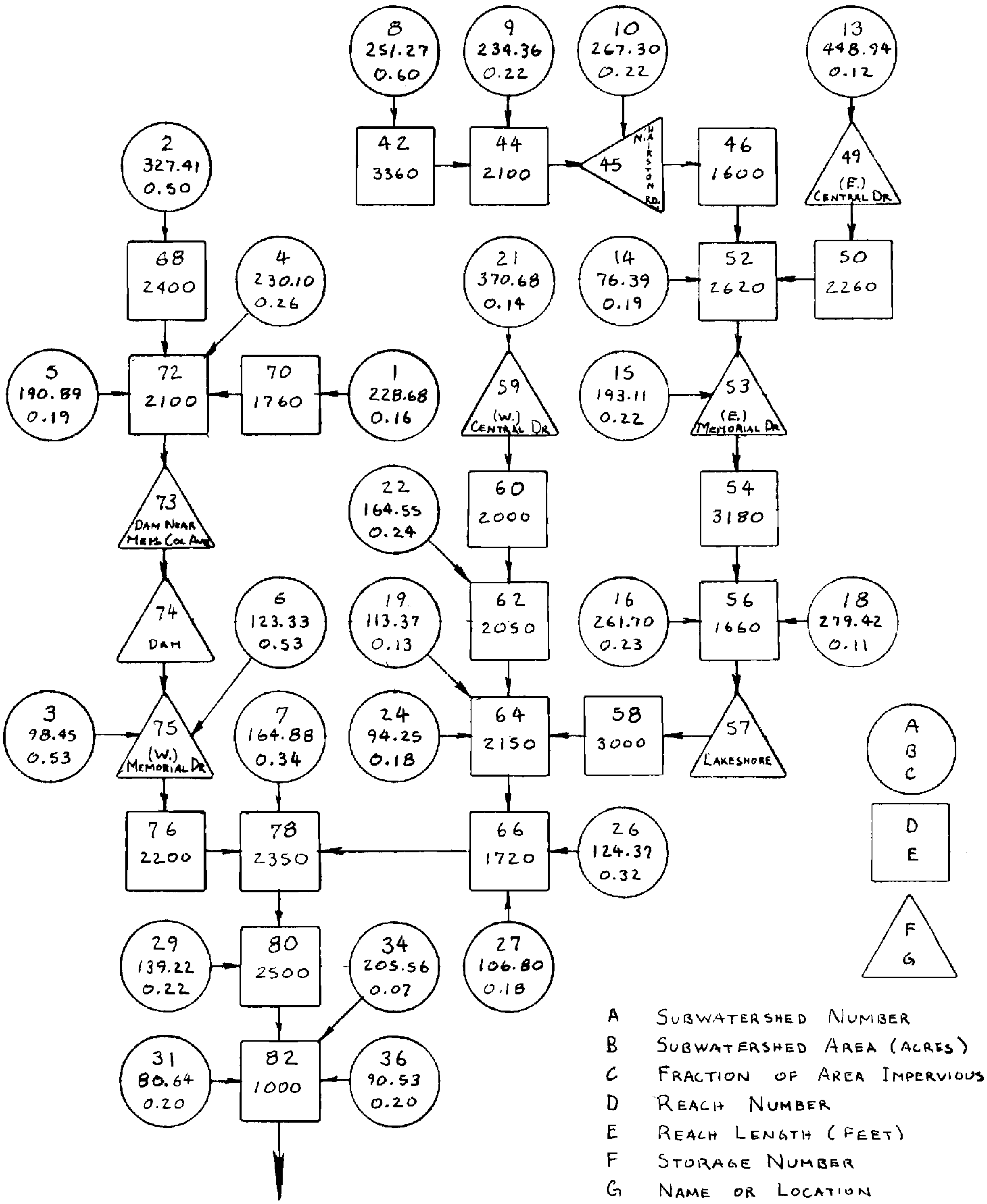
II. Map of Watershed

Figure 7-1 outlines the total watershed and the sub-areas into which it is divided for analysis. Major drainageways are shown and divided into channel segments. Seven road crossings listed in Table 7-1 involve culverts small enough to have a potential backwater effect, and a storage segment was added for each. A schematic diagram of the segments of all three types is shown on Figure 7-2.

III. Drainage Problems

Flooding regularly occurs along the main branch downstream from its junction with the West Branch and along both branches just upstream from the junction. Property damages are concentrated along Susan Creek Drive (Reaches 76 and 78, Figure 7-3) and Indian Lakes Circle (Reaches 80 and 82, Figure 7-4). On Susan Creek Drive, House A has had 6 feet of water in the backyard. The owner said that the creek bed has filled with about 10 feet of sediment during the last 5 years. Three houses near point B were also flooded at the same time. Susan Creek Drive was overtopped from points C to D. On Indian Lakes Circle, House A has been flooded twice at an estimated loss of $35,000. The deepest water occurred in August, 1972, and was 23 inches deep in the house. The conditions analyzed were

1) The extent of the flooding in the problem areas under current development and channel conditions,
2) Overtopping of various culverts within the watershed,
3) Potential aggravation of flooding and drainage problems by additional urban development,
4) The effects of flood flows and flood stages from improving channels and adding storages



SNAPFINGER CREEK SCHEMATIC DIAGRAM

FIGURE 7 - 2

Figure 7-3. Map of Problem Area Along Susan Creek Drive

Scale: 1" = 200'

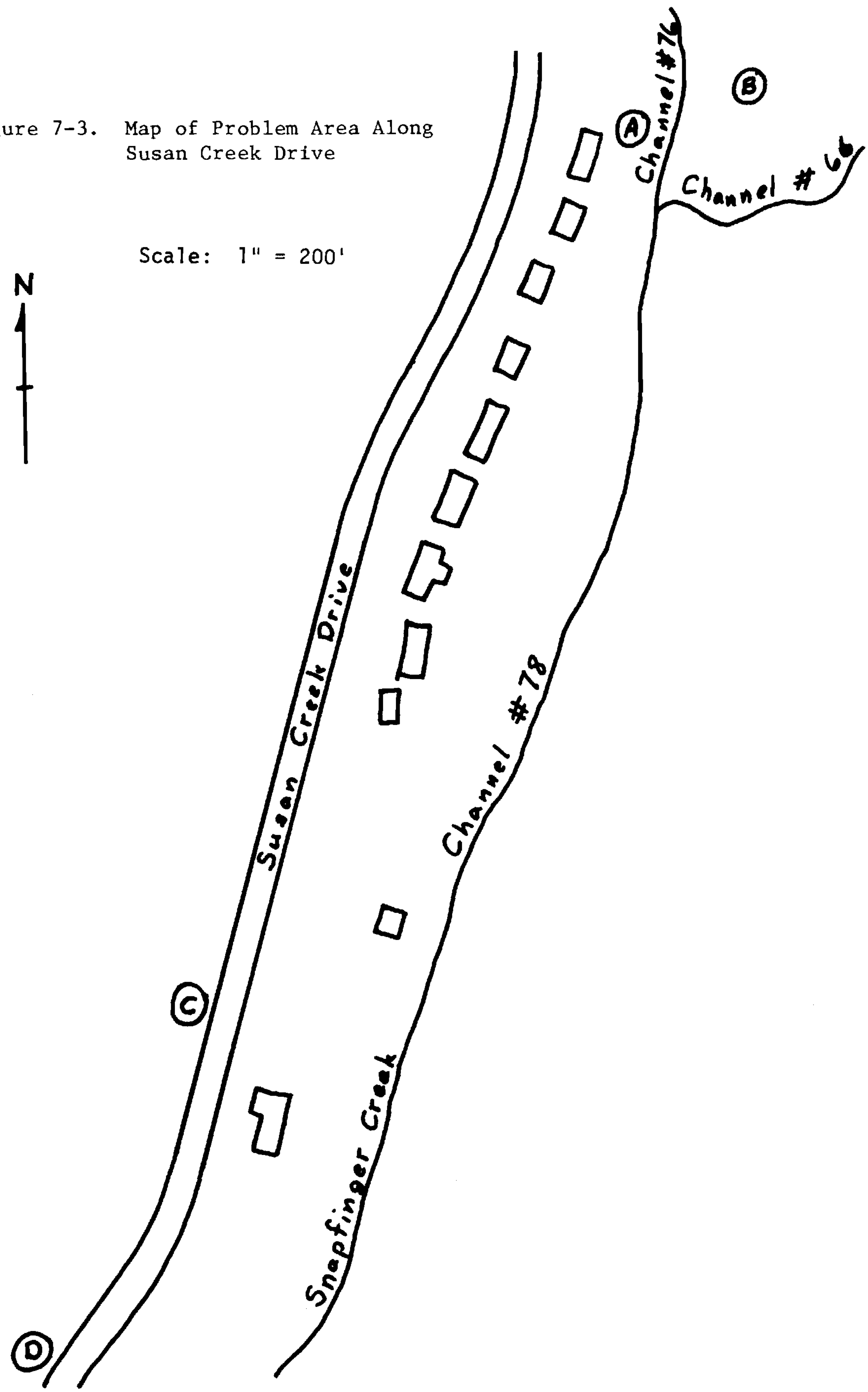
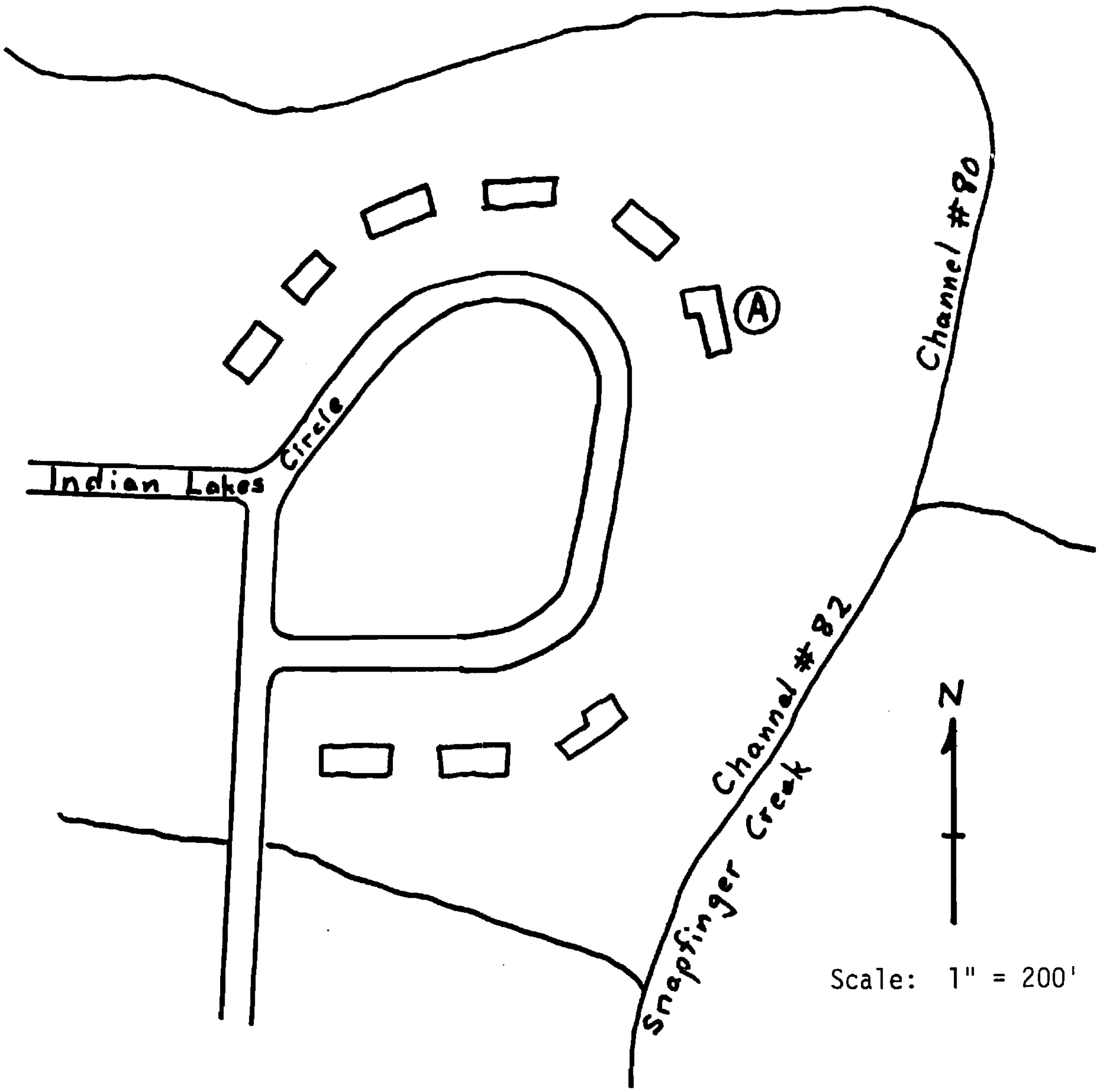


Figure 7-4. Map of Problem Area Along Indian Lakes Circle



IV. Description of Simulation Runs

1. Existing Development

The initial simulation sought to determine flood flows and flood elevations with existing land use and channel conditions in the watershed. The specific purpose was to estimate expected flood levels for various return periods at the problem locations and near the culverts and storages. Table 7-2 shows the specifications for the source areas, channel segments, and seven storage segments. Table 7-3 shows the results of the flood frequency analysis for Reaches 78, 80 and 82 which cover the problem areas.* Flooding problems along the west branch are examined in Section 6. All culverts, except those on Beaver Road and West Branch at Memorial Drive, were found not to be overtopped by the 100-year flood. The culvert on Beaver Road is overtopped by 0.1 foot by the 25-year flood and by 0.9 foot by 100-year flood, but no damageable property is located in the flooded areas. Section 6 describes the situation for West Branch at Memorial Drive.

Table 7-4 summarizes the channel capacities and the 2, 25 and 100-year flood discharges from frequency analysis for Reaches 78,80 and 82.

TABLE 7-4

Simulated Flood Discharges under Existing Conditions

Reach No.	Capacity cfs	Flood Discharges cfs.		
		2-year	25-year	100-year
78	679	2050	4220	5290
80	675	2080	4250	5330
82	849	2190	4460	5590

*Flooding at the downstream end of Reaches 66 and 76 is probably largely caused by backwater from Reach 78.

Table 7-2. Specifications for Area, Channel and Storage Segments for Snapfinger Creek

SPECIFICATIONS FOR SOURCE AREAS

		FRACTION OF AREA WITH EACH SOIL TYPE			
NUMBER	AREA-ACRE	RAPID	MODERATE	SLOW	IMPERVIOUS
1	228.680	.000	.840	.000	.160
2	327.410	.000	.850	.000	.150
4	230.100	.000	.740	.000	.260
5	190.890	.000	.810	.000	.190
6	221.780	.000	.470	.000	.530
7	164.880	.000	.660	.000	.340
8	251.270	.000	.400	.000	.600
9	234.360	.000	.780	.000	.220
10	267.300	.000	.780	.000	.220
13	448.940	.000	.880	.000	.120
14	76.390	.000	.810	.000	.190
15	193.110	.000	.780	.000	.220
16	261.700	.000	.770	.000	.230
18	279.420	.000	.890	.000	.110
19	113.370	.000	.870	.000	.130
21	370.680	.000	.860	.000	.140
22	164.590	.000	.760	.000	.240
24	94.250	.000	.820	.000	.180
26	124.370	.000	.680	.000	.320
27	106.800	.000	.820	.000	.180
29	139.220	.000	.780	.000	.220
31	80.640	.000	.800	.000	.200
34	205.560	.000	.930	.000	.070
36	90.530	.000	.800	.000	.200

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS	AVERAGE TRAVEL TIME(SEC)
42	4	3360.00	.01042	.035	457.4
44	4	2100.00	.01071	.035	235.0
46	4	1600.00	.00938	.035	191.4
50	4	2260.00	.00973	.035	230.0
52	4	2620.00	.00572	.040	274.9
54	4	3180.00	.00943	.035	454.9
56	4	1660.00	.00271	.030	417.0
58	4	3000.00	.00467	.035	448.2
60	4	2000.00	.00800	.040	213.1
62	4	2050.00	.01073	.035	196.6
64	4	2150.00	.00326	.035	443.6
66	4	1720.00	.00349	.035	342.8
68	4	2400.00	.01125	.040	350.8
70	4	1760.00	.01990	.025	129.5
72	4	2100.00	.00381	.035	664.1
74	4	1620.00	.00386	.035	508.7
76	4	2200.00	.00454	.035	324.3
78	4	2350.00	.00425	.035	269.0
80	4	2500.00	.00420	.035	288.0
82	4	1000.00	.00200	.035	190.5

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

SPECIFICATIONS FOR STORAGE SEGMENT 45

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	962.000	.000	.000
.015	253.772	963.000	1.000	.030
.200	1015.086	966.000	4.000	.180
1.824	2030.172	970.000	8.000	.800
10.357	2893.441	975.000	13.000	3.390
36.914	3711.126	980.000	18.000	6.690

SPECIFICATIONS FOR STORAGE SEGMENT 49

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	971.000	.000	.000
.015	107.666	972.000	1.000	.030
.480	430.665	975.000	4.000	.600
8.883	787.249	980.000	9.000	4.220
47.306	1084.563	985.000	14.000	10.610

SPECIFICATIONS FOR STORAGE SEGMENT 53

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	939.000	.000	.000
.035	256.799	940.000	1.000	.070
1.610	1540.793	945.000	6.000	1.330
18.972	2824.786	950.000	11.000	5.970
69.291	4108.780	955.000	16.000	15.390

SPECIFICATIONS FOR STORAGE SEGMENT 57

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	916.000	.000	.000
.048	116.729	917.000	1.000	.096
.372	350.188	919.000	3.000	.360
1.421	466.917	920.000	4.000	1.210
11.434	926.509	925.000	9.000	5.860
102.291	4779.123	930.000	14.000	35.860

Table 7-2 (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 59

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	934.000	.000	.000
.017	89.135	935.000	1.000	.034
.715	713.079	942.000	8.000	.400
4.558	850.160	945.000	11.000	1.090
12.130	1696.515	950.000	16.000	3.800

SPECIFICATIONS FOR STORAGE SEGMENT 73

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	900.500	.000	.000
.250	115.790	903.000	2.500	.200
.700	162.105	904.000	3.500	.400
1.048	208.421	905.000	4.500	.593
2.041	254.737	906.000	5.500	1.250
3.876	289.242	907.000	6.500	2.890
7.412	310.696	908.000	7.500	3.300
8.950	320.885	908.500	8.000	4.200
11.620	684.400	909.000	8.500	5.140
17.055	2326.207	910.000	9.500	7.654
45.644	12714.872	913.000	12.500	15.270

SPECIFICATIONS FOR STORAGE SEGMENT 75

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	893.500	.000	.000
.136	272.376	895.000	1.500	.181
1.105	1180.297	900.000	6.500	.294
2.779	1361.881	901.000	7.500	1.286
3.884	1497.381	902.000	8.500	2.902
8.752	1583.014	903.000	9.500	5.023
14.321	1664.246	904.000	10.500	8.310
25.615	1741.694	905.000	11.500	12.314
90.013	13008.841	910.000	16.500	18.110

Table 7-3. Floods for Segments 78, 80 and 82 on Snapfinger Creek with Existing Conditions.

Return Period (years)	Probability of Exceedance (%)	Flood Flow (cfs)	Water Surface Elevation (feet)
Segment 78			
2 - year	50	2050	8.0[1]
5 - year	20	2920	9.1
10 - year	10	3490	9.8
25 - year	4	4220	10.6
50 - year	2	4760	11.2
100 - year	1	5290	11.9
Segment 80			
2 - year	50	2080	7.8[1]
5 - year	20	2950	8.8
10 - year	10	3540	9.4
25 - year	4	4250	10.1
50 - year	2	4790	10.7
100 - year	1	5330	11.4
Segment 82			
2 - year	50	2190	7.9[1]
5 - year	20	3100	8.4
10 - year	10	3800	8.8
25 - year	4	4460	9.2
50 - year	2	5030	9.5
100 - year	1	5590	9.8

[1]From bottom of channel.

Even though these capacity estimates are very approximate, the channel capacities in all three problem reaches are obviously very low, possibly due to silting, and the flood flows invariably overtop the banks and flow onto the floodplain.

2) Projected Development with Current Channel System

This simulation is based on complete watershed development and the assumption that the current channel system will remain unchanged from present conditions. Table 7-5 shows the projected specifications for the source areas. Table 7-6 shows the results of the frequency analysis for the same segments as Table 7-3. The percent increase in discharges caused by projected development for Reaches 78, 80 and 82 are shown in Table 7-7.

The rise in peak stages due to the above increases in discharges was of the order of 0.6 to 1.1 feet in Reach 78, 0.6 to 0.7

TABLE 7-7

Percent Increase in Discharges Under Projected Development

	% increase in flood discharges		
Reach No.	2-year	25-year	100-year
78	26	20	19
80	24	17	15
82	22	13	12

foot in Reach 80 and 0.3 to 0.4 foot in Reach 82. These stage increases would also aggravate backwater flooding in Reaches 66 and 76, but no areas were discovered where the projected development would be likely to induce new flooding.

3. Hydrologic Analysis of Remedial Measures Based on 1928 Storm

In order to study the hydrologic effectiveness of potential

Table 7-5. Specifications for Area Segments for Snapfinger Creek with Future Development

SPECIFICATIONS FOR SOURCE AREAS

		FRACTION OF AREA WITH EACH SOIL TYPE			
NUMBER	AREA-ACRE	RAPID	MODERATE	SLOW	IMPERVIOUS
1	228.680	.000	.750	.000	.250
2	327.410	.000	.850	.000	.150
4	230.100	.000	.550	.000	.450
5	190.890	.000	.700	.000	.300
6	221.780	.000	.200	.000	.800
7	164.880	.000	.550	.000	.450
8	251.270	.000	.200	.000	.800
9	234.360	.000	.400	.000	.600
10	267.300	.000	.350	.000	.650
13	448.940	.000	.750	.000	.250
14	76.390	.000	.600	.000	.400
15	193.110	.000	.700	.000	.300
16	261.700	.000	.500	.000	.500
18	279.420	.000	.750	.000	.250
19	113.370	.000	.750	.000	.250
21	370.680	.000	.750	.000	.250
22	164.550	.000	.700	.000	.300
24	94.250	.000	.400	.000	.600
26	124.370	.000	.600	.000	.400
27	106.800	.000	.750	.000	.250
29	139.220	.000	.700	.000	.300
31	80.640	.000	.750	.000	.250
34	205.560	.000	.750	.000	.250
36	90.530	.000	.750	.000	.250

Table 7-6. Floods for Segments 78,80, and 82 on Snapfinger Creek with Future Development

Return Period (years)	Probability of Exceedance (%)	Flood Flow (cfs)	Water Surface Elev. (feet)
Segment 78			
2-year	50	2590	8.6[1]
5-year	20	3580	9.8
10-year	10	4240	10.6
25-year	4	5080	11.6
50-year	2	5690	12.3
100-year	1	6300	13.0
Segment 80			
2-year	50	2580	8.4[1]
5-year	20	3540	9.5
10-year	10	4170	10.2
25-year	4	4970	10.9
50-year	2	5560	11.6
100-year	1	6150	12.1
Segment 82			
2-year	50	2680	8.2[1]
5-year	20	3630	8.7
10-year	10	4260	9.1
25-year	4	5060	9.6
50-year	2	5650	9.8
100-year	1	6240	10.2

[1]From bottom of channel.

measures for dealing with the flood problems in the areas indicated, flows were simulated for the storm event producing the largest flows in the problem area for all the previous simulations with current or projected land use, that of July 10, 1928. The remedial measures considered were

1) The series of three small dams on the West Branch represented by storages 73, 74, and 75 on Figure 7-1 and recommended for consideration in Section 6 of this series of studies.
2) Reduction of culvert sizes at locations where the flood water can be temporarily stored upstream from a road embankment without flooding any damageable property (the four locations studied were storage segments 45 at North Hairston Road, 49 at Central Drive, 53 at Memorial Drive, and 59 at Abingdon Drive).
3) Two small dams on the Main Branch on Reaches 62 and 64, and
4) Channelization of the reaches with flood problems, segments 66, 76, 78, 80, and 82.

Simulation runs were made with seven combinations of the measures listed above. The changes in input data largely involve addition of the required storage segments or outlet size changes for cases where the culvert size was reduced. The specifications for the source areas and channel segments remain the same as those under simulation 1 except that reach 74 was converted to storage 74 in West Branch, and channelized sections were used for the last two runs for five channel segments.

The results of the series of seven runs with the 1928 storm are summarized as follows.

1) The possible storage on West Branch reduced the flood discharges in the three problem reaches by 10 to 12%, but this storage

alone would not reduce the peaks to anywhere near the existing capacity of the channels.

2) Of the four culverts on the main branch for which a size reduction was considered, storage at Memorial Drive (Storage segment 53) gives the largest reduction in flood discharge, but the peak flow for the 1928 storm would still be several times the channel capacity.

3) The detention dams on the main branch were still not able to reduce the flood peaks to an acceptable range as sufficient storage capacity is not available at these sites.

4) Channelization of the problem reaches would require large lined channels.

In summary, these seven runs show that structural solution of the flood problem will require an expensive combination of storage and channelization. An economic analysis of costs and benefits would be required to decide whether a structural approach would be justified and to evaluate the possible nonstructural alternatives.

4. Hydrologic analysis based on 25-year flood series

The final simulation with the complete 25-year series was based on the existing level of development plus the proposed dams on West Branch and reduction of the culvert size at Memorial Drive on the Main Branch by 50 percent. Though these storages would not solve the flood problem, this simulation was made for a frequency analysis of the reduction in flood peaks and flood stages that would be achieved in these problem areas by these measures. Table 7-8 shows the results of the flood frequency analysis for Reaches 78, 80 and 82.

Table 7-8. Floods for Segments 78, 80 and 82 on Snapfinger Creek With Added Storage

Return Period (years)	Probability of Exceedance (%)	Flood Flow (cfs)	Water Surface Elevation (Ft)
Segment 78			
2-year	50	1870	7.6 [1]
5-year	20	2560	8.6
10-year	10	3020	9.3
25-year	4	3600	9.9
50-year	2	4040	10.4
100-year	1	4460	10.9
Segment 80			
2-year	50	1910	7.6 [1]
5-year	20	2620	8.4
10-year	10	3090	8.9
25-year	4	3680	9.6
50-year	2	4120	10.1
100-year	1	4560	10.5
Segment 82			
2-year	50	2050	7.8 [1]
5-year	20	2820	8.3
10-year	10	3330	8.6
25-year	4	3980	8.9
50-year	2	4460	9.2
100-year	1	4930	9.5

[1] From bottom of channel.

Reducing the size of the entrance to the culvert at Memorial Drive on the Main Branch by one half would increase the 2-year, 25-year and 100-year upstream flood stages by 2.9, 5.3, and 8.0 feet respectively. Though Memorial Drive would not be overtopped (elev. 960.0) by a 100-year flood, the backwaters would inundate about 25 acres including the culvert on Anderson Road and several residences. Furthermore, comparison of Table 7-3 and 7-8 show that flood stages in the problem reaches would only be reduced by amounts ranging from 0 to 0.9 foot for return periods 2 to 100 years. Therefore, it seems unlikely that storage at the Memorial Drive site can be economically justified.

V. Summary and Conclusions

The existing channels in reaches 78, 80 and 82 were found to have very low capacities and regularly flood. Detention storage on the West Branch (see Section 6) would not lower the flood stages in the problem areas significantly, and no effective storage sites have been found on the main branch to lower downstream flood peaks even though six sites were investigated. Under these circumstances, the following approaches deserve further study and economic analysis.

1) Channelize these reaches,
2) Buy the property flooded along these reaches for use as recreation or natural areas,
3) Flood proof the properties where feasible. Zoning regulations should be strictly enforced to prevent further development in the area of high flood hazard.

SECTION 8

Cobbs Creek

I. Physical Attributes of Watershed

a. Location

Cobbs Creek, a tributary of the South River, has its headwaters in Avondale Estates. Within the study area which lies above Snapfinger Road, the stream flows generally southeast draining an extensively developed area.

b. Size

The watershed has a total area of about 2300 acres with about 29% impervious. Thirteen roads cross the creek or its major tributaries. Table 8-1 shows the drainage areas and the size of the culverts at these road crossings. A few minor roads which are in the upper reaches of the tributaries are not listed.

c. General Drainage Patterns

The basin and channel configurations are shown on Figure 8-1. Over 90 percent of the basin is developed,and the drainage is in natural channels. The main channel has a length of about 3.4 miles from the divide on area 2 to Snapfinger Road. The Creek has one main tributary which drains the eastern portion of the watershed and has a length of about 3,500 feet. The channel width varies from about 15 feet in the upper reaches to about 30 feet in the lower reaches.

d. Other Studies

The U. S. Army Corps of Engineers completed a Flood Plain Information Report of Cobbs Creek in June, 1968. The study limits extended up to the Snapfinger Road crossing which is the lower limit of this study. In the Corps study, using records of known flood data for all gaging stations in the region with a record exceeding 5 years, the 100-year flood discharge at Snapfinger Road crossing was statistically estimated as 2600 cfs. Based on 25-year simulated peak flows for the existing development, the 100-year flood at the same location

TABLE 8-1

Drainage Areas at Culverts

Road	Drainage Area (Acres)	Culvert Description
Hess Dr.	193	1-5.3' x 7.8' Elliptical
Memorial Dr. (Branch)		1-6' x 6' Box
Memorial Dr. (Cr.)	428	1-7' x 8' Box
*Bobbie Lane	550	3-4' Circular 1-4'-5'9" Elliptical
Beech-Bonway Dr.	607	2-5' x 5.7' Boxes
*Convair Lane	690	2-6' x 9' Arch
*Midway Rd.	740	2-6.5'Circular
*Alverado Way	1004	1-6' x 20' Bridge
*Peach Crest Rd.	1113	1-6' x 20' Bridge
Brookfield La. (west)	1216	2-7.5' x 8.0' Boxes
Sherry Dale La.	474	2-7.5' x 7' Elliptical
Brookfield La. (east)	568	1-6' x 11.6' Box
*Gleenwood Rd.	2061	4-10'x 10' Boxes
*Snapfinger Rd.	2292	*

*Culvert size is large enough so as not to cause potential backwater effect.

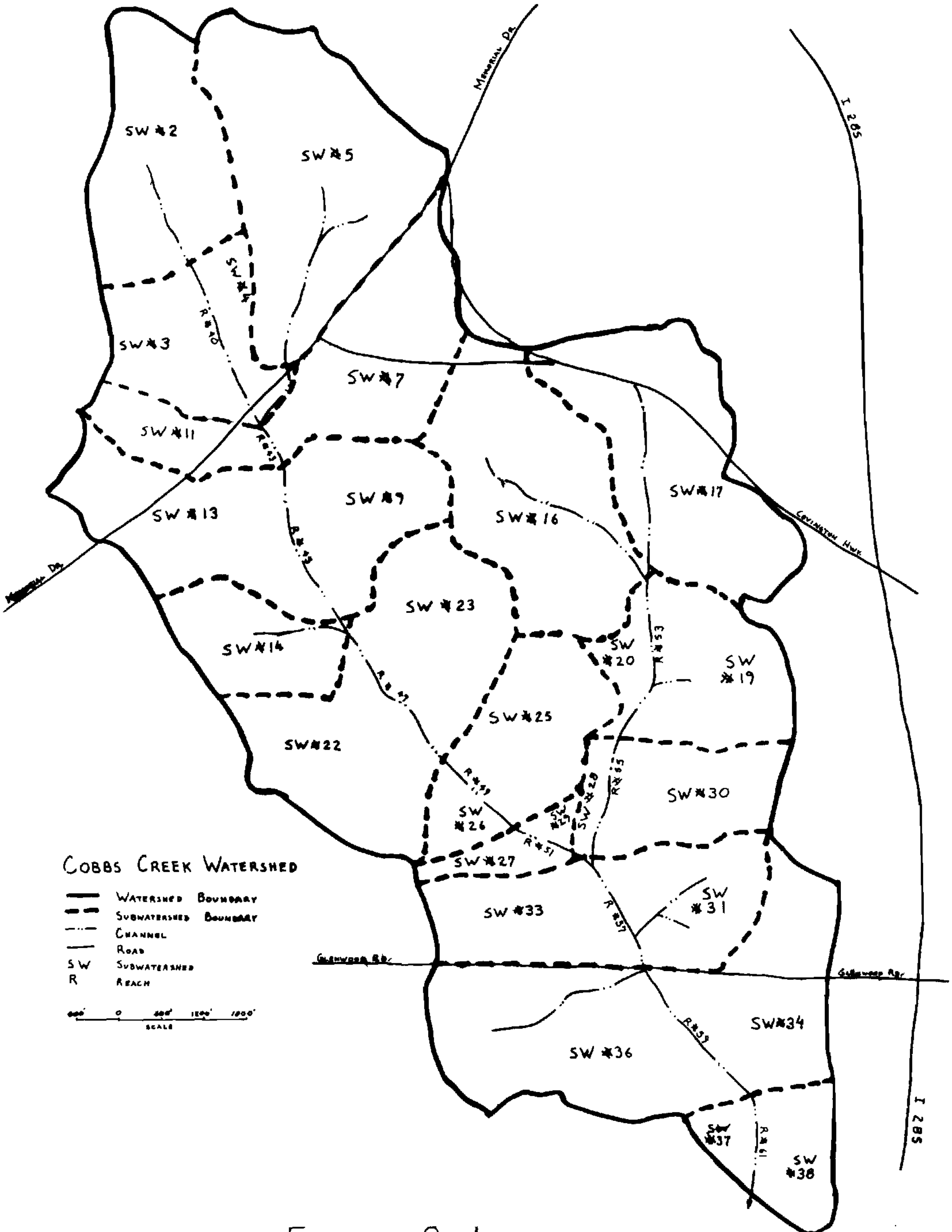

Cobbs Creek Watershed
Watershed Boundary
Subwatershed Boundary
Channel
Road
SW Subwatershed
R Reach
Memorial Dr.
I 285
Covington Hwy.
Glenwood Rd.
SW #2
SW #5
SW #3
SW #7
SW #11
SW #13
SW #9
SW #16
SW #17
SW #23
SW #14
SW #20
SW #19
SW #25
SW #22
SW #30
SW #26
SW #27
SW #33
SW #31
SW #34
SW #36
SW #37
SW #38

FIGURE 8-1

has been estimated as 4990 cfs in this study. Part of the discrepancy was due to the added development that has taken place since 1968 when the Corps study was completed.

e. Current Land Use

Over 90% of the watershed has already been developed. The undeveloped area is mostly wooded area along the creek and its tributaries. Most of the developed area is single family residential with some commercial property and multifamily apartment complexes along the main highway. Subwatershed impervious areas range from 5 to 65% of the total.

f. Projected Land Use

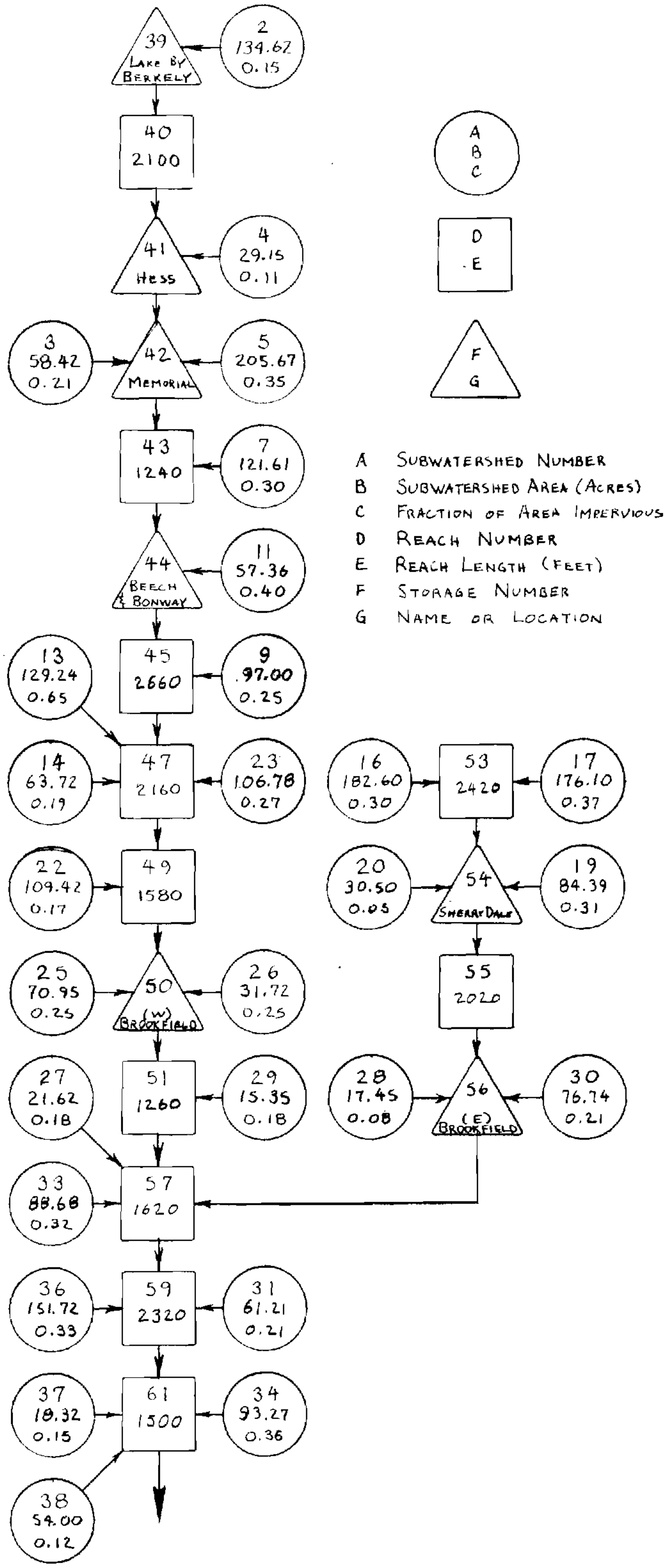
The undeveloped land remaining in the basin is mostly wooded area along the creek and its tributaries. This area is likely to be left as flood plain and should not be developed. A few acres of land remaining undeveloped in subwatersheds 25, 34 and 38 may eventually be developed as multifamily apartment complexes as indicated by the nearby development. However, the complete development of the basin should not significantly alter the peak flood flows.

II. Map of Watershed

Figure 8-1 outlines the total watershed and the subareas into which it is divided for analysis. Major drainageways are shown and divided into channel segments. Six road crossings listed in Table 8-1 involve culverts small enough to have a potential backwater effect and a storage segment was added for each. In addition, the storage effect of the lake at the intersection of Berkely Road and Wiltshire Drive (subarea 2) was also considered. A schematic diagram of the segments of all three types is shown in Figure 8-2.

III. Drainage Problems

The drainage problems in the watershed are concentrated in the general areas located near Brookfield Lane and Misty Valley Drive (Figure 8-3) and the junction of Beech and Bonway Drives (Figure 8-4). Homes along Brookfield



Cobbs Creek Schematic Diagram

FIGURE 8-2

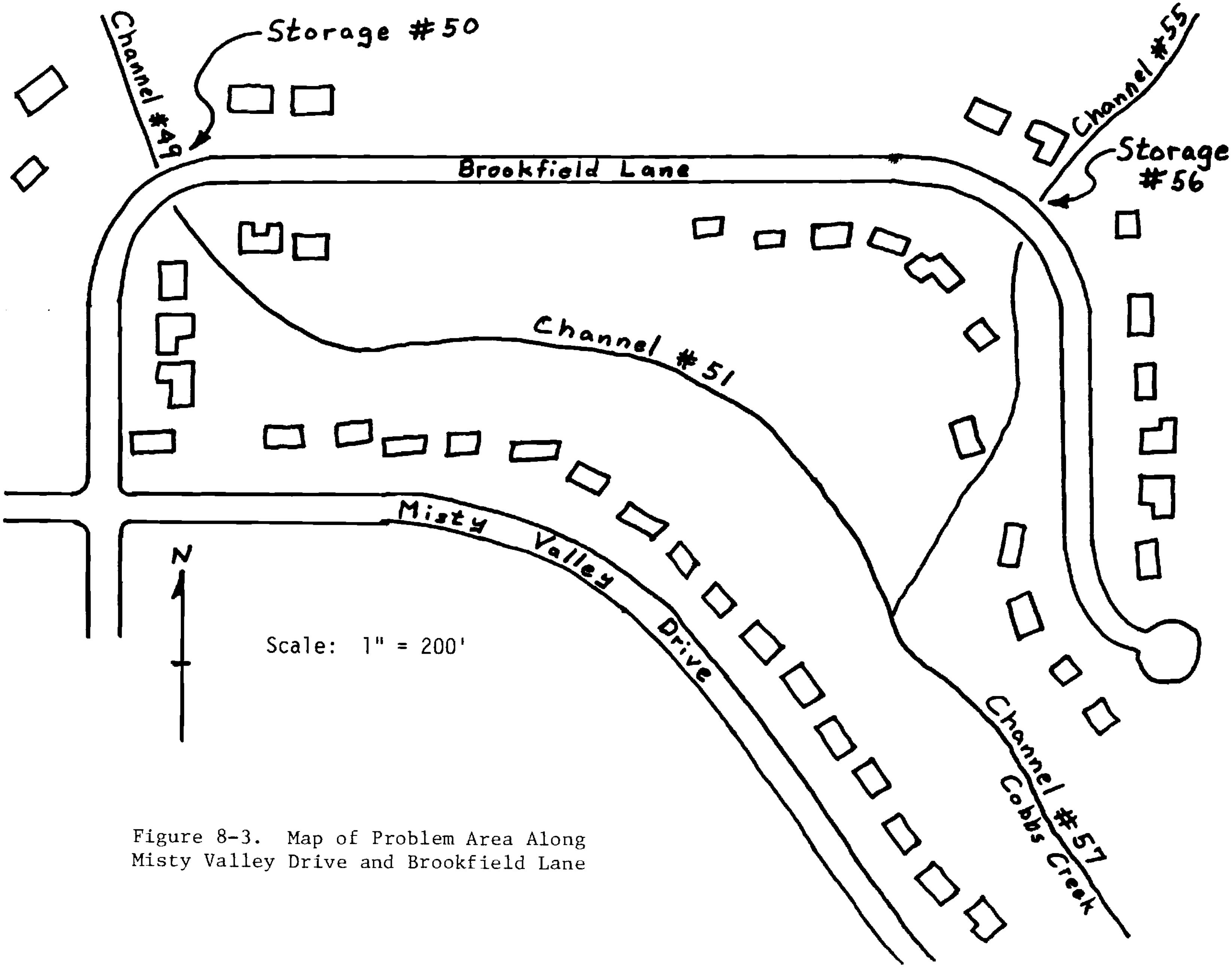
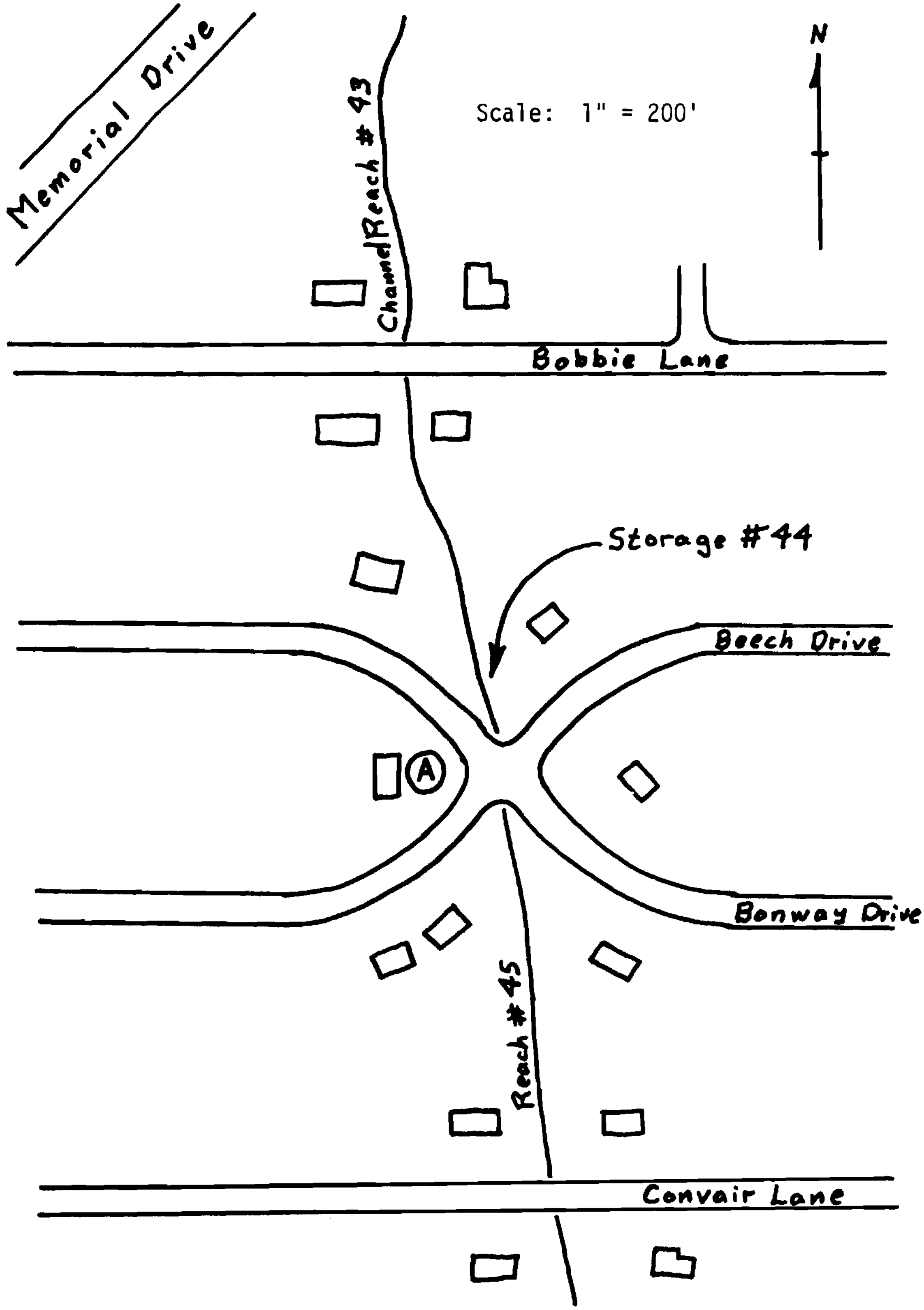


Figure 8-3. Map of Problem Area Along Misty Valley Drive and Brookfield Lane

Figure 8-4. Map of Problem Area at Beech and Bonway Drives



Lane and Misty Valley Drive represent one of the major areas with flood damage in the county. The Brookfield Lane has been overtopped by two feet of water. On Beech Drive, water has been 3 feet deep in the front yard of house (A) and resulted in basement flooding. The specific problems studied were

1) Overtopping of various culverts within the watershed,
2) Flooding of the houses at the intersection of Beech and Bonway Drives,
3) Flooding of the general area near Brookfield Lane and Misty Valley Drive,
4) Some possible solutions to reduce the present flooding in the watershed.

IV. Description of Simulation Runs

1. Existing Development

The initial simulation sought to determine flood flows and flood elevations with the existing level of development in the watershed. The specific purpose was to estimate expected flood levels for various return periods at the problem locations and near the culverts and storages. Table 8-2 shows the specifications for source areas, channel segments and the 7 storage segments. Table 8-3 shows the results of the flood frequency analysis for segments 44, 50, 56, 57, and 61. Segment 44 covers the problem areas at Brookfield Lane and Misty Valley Drive. Segment 61 is the channel segment at the lower end of the study and gives flows at the Spapfinger Road crossing.

Table 8-3 shows that a 5-year flood overtops the road (elev. 934.0) at the culvert on Beech Drive by 0.2 of a foot and the 100-year flood by 1.6 feet. This would mean that all floods of frequency 5-years and above would enter the front yard of houses A (See Figure 8-3). The 5-year flood also overtops Brookfield Lane to the west (Figure 8-3) and overflow occurs over a large length of the road flooding several houses. Brookfield Lane is overtopped to the east (elev. about 882.0) by a 10-year flood,and the backwaters near the culvert are likely to flood some 10 houses in the area.

Table 8-2. Specifications for Area, Channel and Storage Segments for Cobbs Creek

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE: RAPID	MODERATE	SLOW	IMPERVIOUS	STORAGE CONSTANT
2	134.620	.000	.850	.000	.150	22.
3	59.420	.000	.790	.000	.210	13.
4	29.150	.000	.890	.000	.110	11.
5	205.670	.000	.650	.000	.350	21.
7	121.610	.000	.700	.000	.300	17.
9	97.000	.000	.750	.000	.250	16.
11	57.360	.000	.600	.000	.400	10.
13	129.240	.000	.350	.000	.650	12.
14	63.720	.000	.810	.000	.190	14.
16	182.600	.000	.700	.000	.300	21.
17	176.100	.000	.630	.000	.370	19.
19	84.390	.000	.690	.000	.310	14.
20	30.500	.000	.950	.000	.050	12.
22	109.420	.000	.830	.000	.170	19.
23	106.780	.000	.730	.000	.270	16.
25	70.950	.000	.750	.000	.250	14.
26	31.720	.000	.750	.000	.250	9.
27	21.620	.000	.820	.000	.180	8.
28	17.450	.000	.920	.000	.080	9.
29	15.350	.000	.820	.000	.180	7.
30	76.740	.000	.790	.000	.210	15.
31	61.210	.000	.790	.000	.210	13.
33	88.680	.000	.680	.000	.320	14.
34	93.270	.000	.640	.000	.360	14.
36	151.720	.000	.670	.000	.330	13.
37	18.320	.000	.850	.000	.150	8.
38	54.000	.000	.880	.000	.120	14.

Table 8-2 (cont'd)

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS	AVERAGE TRAVEL TIME(SEC
40	4	2100.00	.01429	.035	245.7
43	4	1240.00	.00806	.035	139.6
45	4	2660.00	.00451	.035	400.4
47	4	2160.00	.00833	.035	253.6
49	4	1580.00	.00633	.035	212.8
51	4	1260.00	.00714	.035	159.8
53	4	2420.00	.01529	.025	142.8
55	4	2020.00	.00198	.035	488.4
57	4	1620.00	.00309	.040	292.6
59	4	2320.00	.00474	.035	324.7
61	4	1500.00	.00667	.035	177.0

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

SPECIFICATIONS FOR STORAGE SEGMENT 39

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	3.913	977.000	.000	5.000
5.250	45.019	978.000	1.000	5.500
11.002	63.131	979.000	2.000	6.000
17.166	78.667	930.000	3.000	6.240

SPECIFICATIONS FOR STORAGE SEGMENT 41

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	945.700	.000	.000
.017	73.506	947.000	1.300	.026
.064	130.156	948.000	2.300	.064
.161	186.745	949.000	3.300	.150
.563	243.335	950.000	4.300	.830
6.662	879.753	955.000	9.300	3.030

Table 8-2 (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 42

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	940.500	.000	.000
.016	143.899	941.500	1.000	.032
.064	287.799	942.500	2.000	.064
.144	431.698	943.500	3.000	.096
.730	647.547	945.000	4.500	1.150
10.878	1148.022	950.000	9.500	5.210
55.758	1625.693	955.000	14.500	11.020

SPECIFICATIONS FOR STORAGE SEGMENT 44

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	926.000	.000	.000
.016	102.263	927.000	1.000	.032
.192	409.131	930.000	4.000	.160
3.913	807.866	935.000	9.000	2.250
23.747	1593.893	940.000	14.000	4.970

SPECIFICATIONS FOR STORAGE SEGMENT 50

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	880.000	.000	.000
.020	175.818	881.000	1.000	.040
.087	351.636	882.000	2.000	.100
.543	879.090	885.000	5.000	.344
3.109	1522.629	890.000	10.000	.790
13.552	27662.115	895.000	15.000	4.300

Table 8-2 (cont'd)

SPECIFICATIONS FOR STORAGE SEGMENT 54

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	888.500	.000	.000
.012	120.826	889.500	1.000	.024
.151	483.365	892.500	4.000	.131
.990	785.370	895.000	6.500	.410
8.267	1716.258	900.000	11.500	4.720

SPECIFICATIONS FOR STORAGE SEGMENT 56

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	876.000	.000	.000
.055	114.011	877.000	1.000	.110
1.100	664.066	882.000	6.000	.660
8.617	4449.426	885.000	9.000	3.670

Table 8-3. Floods for Segments 44,50,56,57 and 61 on Cobbs Creek with Existing Conditions

Return Period (years)	Probability of Exceedance (%)	Flood Flows (cfs)	Water Surface Elevation (ft)
Segment 44			
2-year	50	530	931.2[1]
5-year	20	760	934.2
10-year	10	910	935.1
25-year	4	1100	935.3
50-year	2	1250	935.5
100-year	1	1390	935.6
Segment 50			
2-year	50	1030	887.8
5-year	20	1500	889.7
10-year	10	1810	890.0
25-year	4	2200	890.0
50-year	2	2500	890.0
100-year	1	2790	890.1
Segment 56			
2-year	50	510	881.1
5-year	20	750	881.8
10-year	10	920	882.2
25-year	4	1120	882.4
50-year	2	1270	882.6
100-year	1	1420	882.7
Segment 57			
2-year	50	1490	9.2[2]
5-year	20	2160	9.8
10-year	10	2600	10.3
25-year	4	3160	10.6
50-year	2	3580	10.8
100-year	1	3990	11.0
Segment 61			
2-year	50	1670	-[3]
5-year	20	2420	-
10-year	10	2910	-
25-year	4	2910	-
50-year	2	3540	-
100-year	1	4470	-

[1] from mean sea level

[2] from bottom of channel

[3] elevation not estimated

The channel along Brookfield Lane and Misty Valley Drive has a depth of about 7 feet and the two-year flood has a depth around 9 feet. This indicates that the flows are in the floodplain which is flat in this area and several houses on the right bank are likely to be flooded frequently.

2. Natural Conditions

This simulation is based on the watershed in its natural conditions and an assumption that the current channel system (without lakes and culverts) is the same as existed when there was no development in the watershed. The specific purpose was to estimate the natural flows occuring in the watershed and to evaluate how much urbanization increased the flood peaks. All source areas were assumed to be entirely pervious, and the specifications for channel reaches remained the same as given by Table 8-2. Table 8-4 summarizes selected peak flows that were estimated to occur under natural conditions. Increases due to urbanization range from 11% to 48% depending upon the storm.

3. Study with Storage Segment 48.

In order to determine the effect of retention storage at the Peachcrest Road crossing (storage segment 48), flows were simulated for selected storm events. No new retention structure was proposed. Only the existing culvert size at Peachcrest Road was reduced to 4'x20' with a 30' trapezoidal weir with 1:1 side slopes at Elev. 903.0. The aim was to see that the 100-year flood elevation would not exceed elevation 905.0. Above that elevation one house would be flooded and at elevation 910, five houses would be flooded by the backwaters from the retention.

Table 8-5 shows the specifications for storage segment 48. The specifications for other segments would remain the same as those given by Table 8-2. Table 8-6 summarizes the results of this simulation and shows that the reduction in peak flows due to storage 48 would be insignificant for smaller storm events while in case of the largest events (July 10, 1928) the reductions were 16% in both segments 57 and 61. A reduction of 16% in the larger peak flows would not begin to eliminate

Table 8-4. Comparison of Selected Floods for Segments 57 and 61 Before and After Development

Date of Storm	Peak Flow (cfs) Existing Conditions	Peak Flow (cfs) Before Development	Percent Increase
Segment 57			
Aug. 24, 1921	2814	1896	48%
Jan. 11, 1925	1483	1342	11%
July 10, 1928	3205	2811	14%
Segment 61			
Aug 24, 1921	3095	2095	48%
Jan 11, 1925	1723	1546	11%
July 10, 1928	3630	3199	13%

Table 8-5. Specifications For Storage Segment 48

Storage (Acre-Feet)	Discharge (cfs)	Elevation (Feet-MSL)	Head (Feet)	Surface Area (Acres)
.000	.000	893.000	.000	.000
.050	226.980	895.000	2.000	.050
.287	641.997	897.000	4.000	.230
1.317	912.978	900.000	7.000	.550
5.388	1197.802	903.000	10.000	2.670
14.811	1646.716	905.000	12.000	4.960
24.235	2095.630	.000	.000	.000

Table 8-6. Results of Study With Storage Segment 48

Date of Storm	Peak Flow (cfs) Without Storage	Peak Flow (cfs) With Storage	Percent Decrease
Segment 57			
Aug 24, 1921	2814	2502	11%
Jan 11, 1925	1483	1458	2%
July 10, 1928	3205	2707	16%
Segment 61			
Aug 24, 1921	3095	2823	9%
Jan 11, 1925	1723	1707	1%
July 10, 1928	3630	3102	16%

the flood problem in the Brookfield Lane and Misty Valley Drive area. Moreover, it was observed that the flood elevation due to the flows of July 10, 1928 would exceed the target elevation of 905.0.

V. Summary and Conclusion

It was found that under the existing conditions frequent flooding would occur at the intersection of Beech and Bonway Drives and the Brookfield Lane and Misty Valley Drive area. This is mainly due to almost complete urban development of the watershed. The present study indicated that near Misty Valley Drive urbanization increased flood flows from 10 to 50%. Retention behind Peachcrest Road was found to be ineffective to eliminate present flood problems in Brookfield Lane and Misty Valley Drive area.

Channel and culvert improvements from Bobbie Lane to Convair Lane may reduce flood problems at the junction of Beech and Bonway Drives.

An economic analysis of measures for dealing with flood problems on Brookfield Lane and Misty Valley Drive should consider removal of houses frequently flooded, flood proofing of homes less frequently flooded, and sizeable channel improvements.

SECTION 9

Nancy Creek

I. Physical Attributes of Watershed

a. Location

The portion of the Nancy Creek watershed examined in this study is the upper head water area. The Creek originates in Gwinnett County and flows west crossing Peachtree Industrial Boulevard and I-285 at about 1.48 and 2.34 miles respectively below the point of origin. The drainage area lying in Gwinnett County is only about 160 acres. The west branch, the largest of 4 branches joining Nancy Creek, originates near the intersection of Mount Vernon and Chamblee-Dunwoody Road, flows approximately southeast for about 1.6 miles and then flows south for about 1 mile to join the Creek below I-285 at North Shallowford Road. The study area ends about 2000 feet below North Shallowford Road.

b. Size

The watershed has a total area of about 6790 acres. Several roads cross the creek and its tributaries within the study area with the more important roads, the drainage area at each road crossing, and the culvert sizes and capacities given in the Table 9-1. The capacity estimates are culvert discharges under the head at which flows just begin to overtop the road.

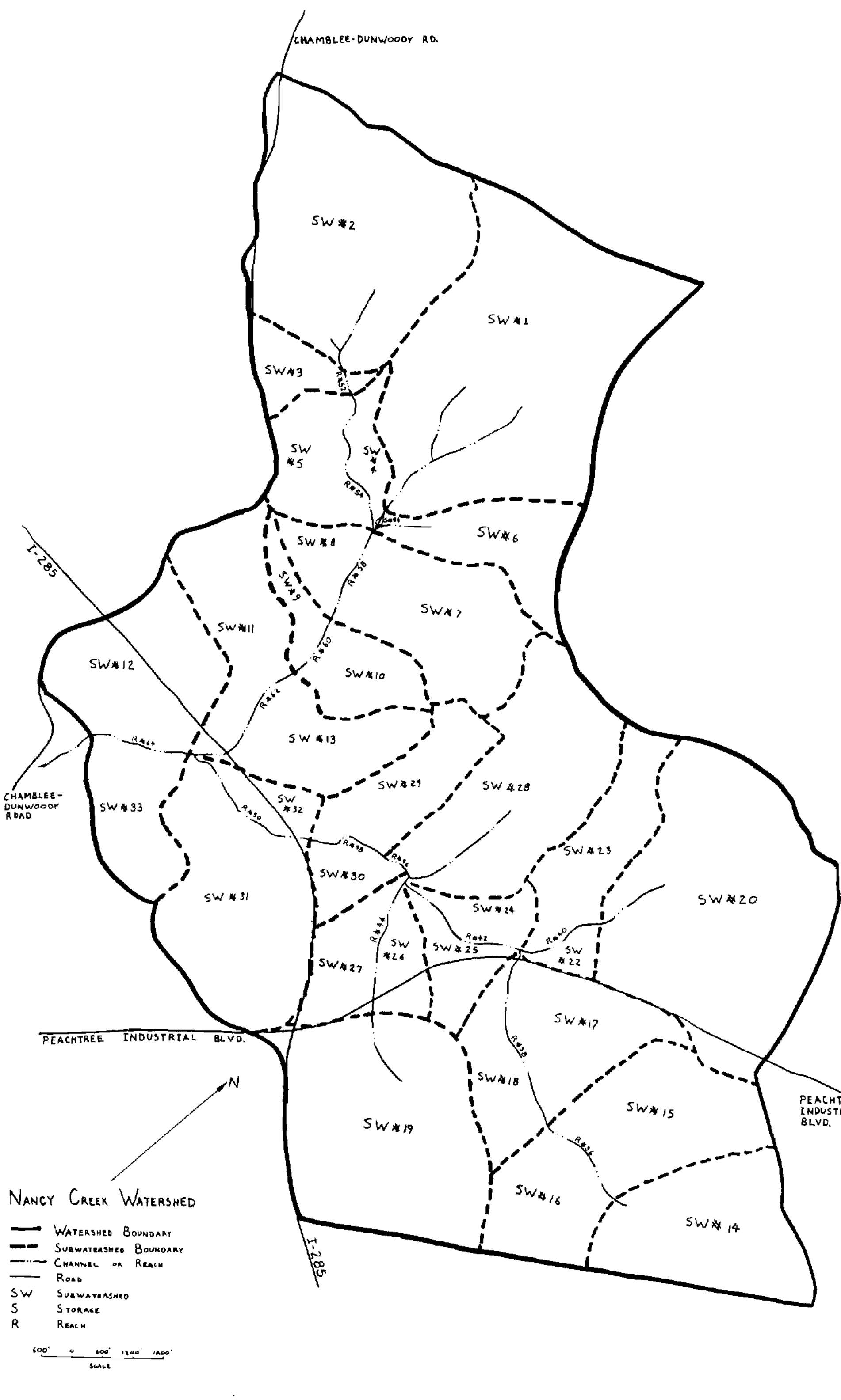
c. General Drainage Patterns

The basin and channel configurations are shown on Figure 9-1. About 70% of the area is developed, and the drainage is in natural channels. The main channel has a length of about 3.4 miles from its origin in subwatershed 14 to the end of study area, reach 64.

TABLE 9-1

Drainage Areas at Culverts

Road	Drainage Area Acres	Culvert Description	Capacity cfs.
eachtree Ind. Blvd.	726	3 - 4'X8' boxes	1722.
illy Mill Rd.	1278	1 - 5.5'X20' Bridge	1248
eeler Rd. (West Br.)	538	2 - 4'X8' boxes	958.
eeler Rd. (East Br.)	637	2 - 4'X8' boxes	1086.
eachford Rd.	1656	3 - 7'X9' boxes	2569
. Shallowford Rd.	4500	4 - 8'x8' boxes	3578



CHAMBLEE-DUNWOODY RD.
SW #1
SW #2
SW #3
SW #4
SW #5
SW #6
SW #7
SW #8
SW #9
SW #10
SW #11
SW #12
SW #13
SW #14
SW #15
SW #16
SW #17
SW #18
SW #19
SW #20
SW #22
SW #23
SW #24
SW #25
SW #26
SW #27
SW #28
SW #29
SW #30
SW #31
SW #32
SW #33
I-285
CHAMBLEE-DUNWOODY ROAD
PEACHTREE INDUSTRIAL BLVD.
N
NANCY CREEK WATERSHED
WATERSHED BOUNDARY
SUBWATERSHED BOUNDARY
CHANNEL OR REACH
ROAD
SW SUBWATERSHED
S STORAGE
R REACH
SCALE

FIGURE 9-1

The west branch with 4 tributaries drains about 40% of the study area and has a length of 2.65 miles. In addition, the Creek has two middle branches of 7000 and 4000 feet draining the lower and upper middle basin respectively and an eastern branch of 4500 feet. Channel widths vary from about 10 feet in the upper reaches to about 40 feet in the lower reaches.

d. Other Studies

The U.S. Army Corps of Engineers completed a special flood Hazard Information Report including the Nancy and Peachtree Creek Basins in DeKalb County in October 1971. Since the study described herein covers only a small portion of the Corps' Study area,the Corps report does not go into sufficient detail to give such information as 100-year flood flows in different reaches for direct comparison. However, the 100-year flood stages determined in Corps Study agree approximately with the 100-year flood stages estimate from the simulated flows in this study.

e. Current Land Use

About 70% of the basin has already been developed. More than half of the undeveloped area lies along the major highways. While most development is single family residences, multi-family apartment complexes and industrial and commercial property are located along the major highways. Subwatershed impervious areas range from 2% to 72% of their respective total areas.

f. Projected Land Use

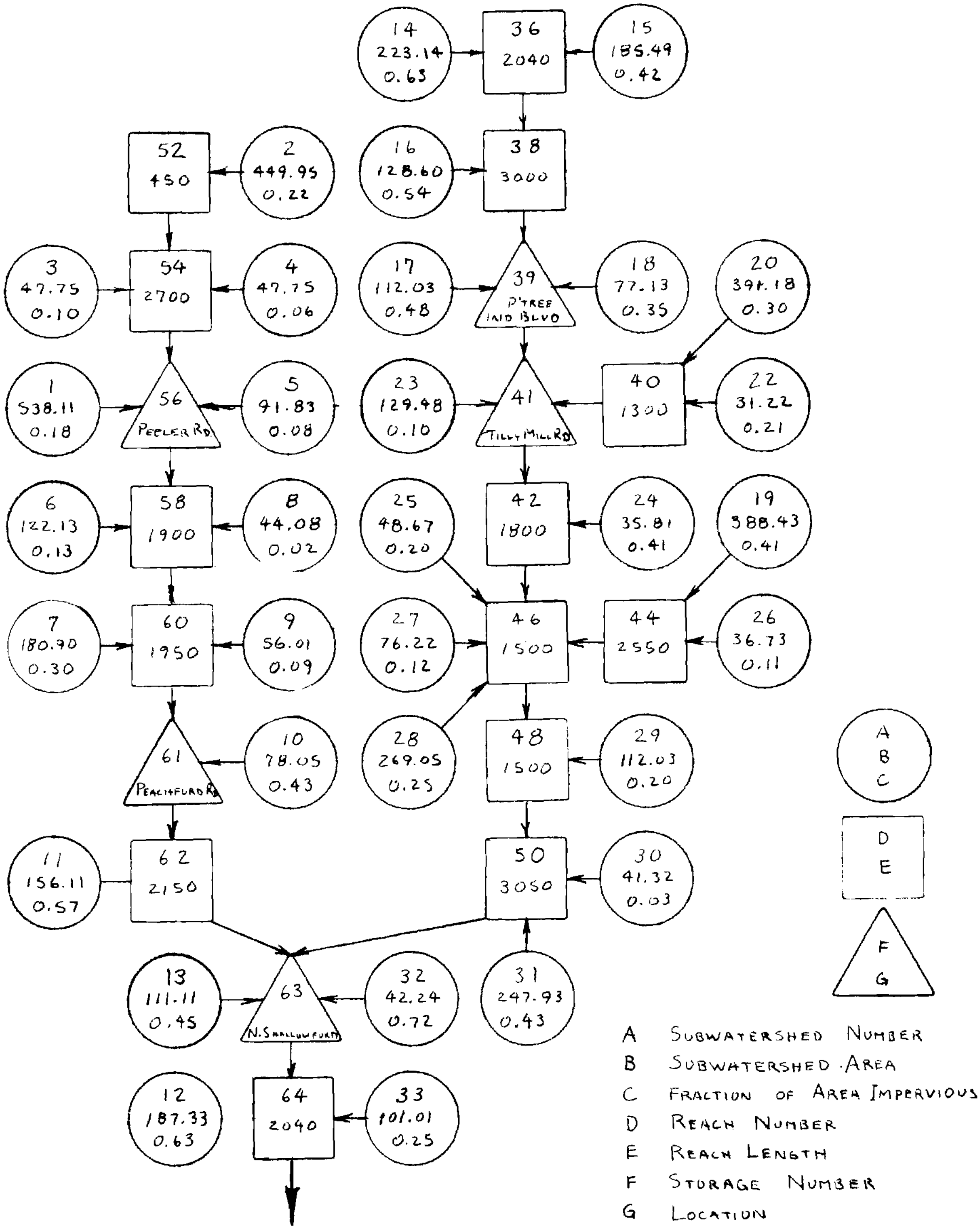
It is expected that the patches of undeveloped land remaining in various parts of the watershed will be eventually developed. For this study, it is projected that the undeveloped areas located along major roads will be developed as multi-family apartment complexes and commercial property and the interior areas as single family houses.

II. Map of Watershed

Figure 9-1 outlines the total watershed and the subareas into which it is divided for analysis. Major drainageways are shown and divided into channel segments. The six road crossings listed in Table 9-1 involve culverts small enough to have a potential backwater effect, and a storage segment was added for each. The two culverts on Peeler Road were very close to each other; hence, only one storage segment was added to represent their combined effect. A schematic diagram of the segments of all three types is shown on Figure 9-2.

III. Drainage Problems

There are three general areas which have been affected by floods in the basin. 1) Tilly Mill Road: Water flowed two feet deep over the bridge during the 1973 floods, but no damageable property was located in the flood plain. 2) Vintage Lane: Severe residentail flooding occurred in 1973. A house near point A (Figure 9-3) had water two feet deep in living room. Flow in the tributary coming from the north aggravates the problem,but Nancy Creek is the main source of flooding. 3) Gainesborough Drive and Royal Court Area: Severe residential flooding has occurred in this area. Flood waters have been three feet deep at point A (Figure 9-4). The house near point B had water three feet in the backyard during the December, 1973 flood. The flood plain is very wide and flat through this reach.



NANCY CREEK SCHEMATIC DIAGRAM

FIGURE 9 - 2

Figure 9-3. Map of Problem Area at Vintage Lane

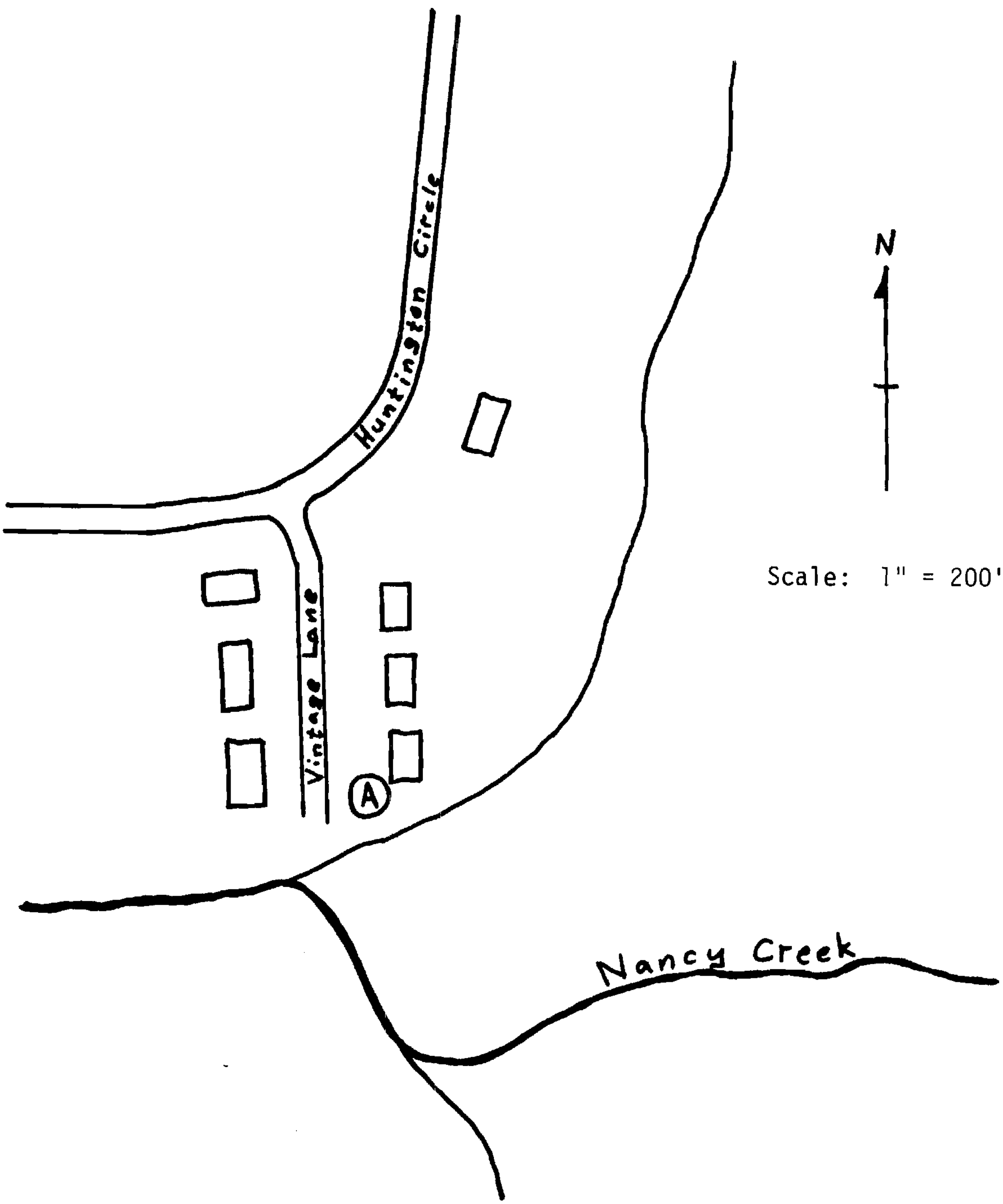
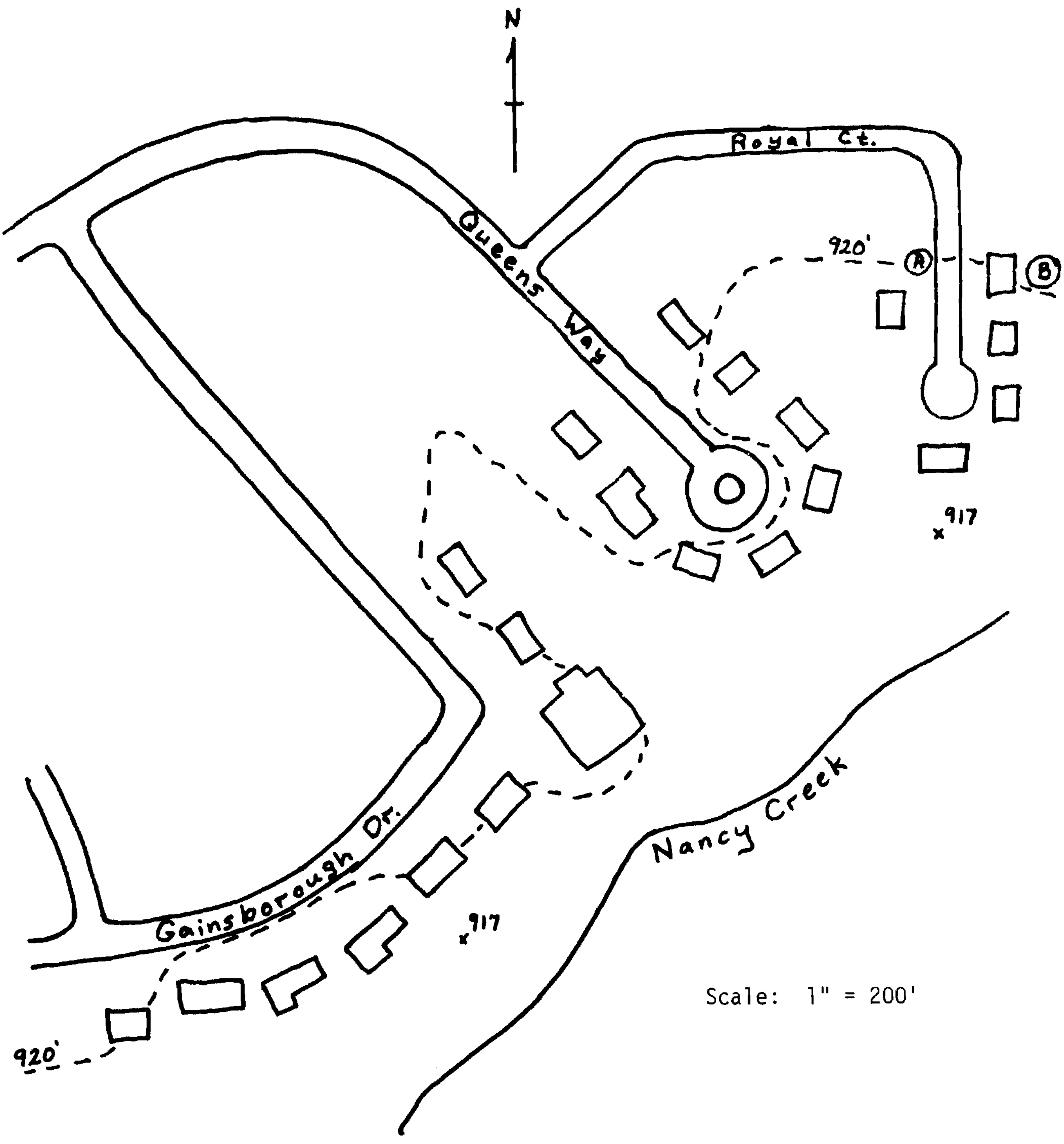


Figure 9-4. Map of Problem Area at Gainsborough Drive, Queens Way and Royal Court



Specific problems studied were

1) Extent of potential flooding under the current development and channel conditions.
2) Overtopping various culverts within the study watershed,
3) Potential aggravation of flooding and drainage problems in the watershed by additional urban development,
4) The hydrologic implications of addressing the present flooding problem by improving channels and adding storages.

V. Description of Simulation Runs

1. Existing Development

The initial simulation sought to determine flood flows and flood elevation with the existing level of development in the watershed. The specific purpose was to estimate expected flood levels for various return periods at the problem locations and near the culverts. Table 9-2 shows the specifications for the source areas, channel segments, and the 5 storage segments. Table 9-3 shows the results of the flood frequency analysis for storage segment 41 and reaches 46 and 64 which cover the problem areas.

The bridge on Tilly Mill Road (Elev. 952.8) is overtopped by all floods having a 10-year return period or greater.

Capacities of the existing channels in the two problem reaches were too low, and the flood flows were invariably in the flood plain. Since the flood plain in these reaches are broad and flat, vast areas are inundated. This inundation does, however, provide flood storage to reduce damages downstream.

Table 9-2. Specifications for Area, Channel and Storage Segments for Nancy Creek

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE			
		RAPID	MODERATE	SLOW	IMPERVIOUS
1	538.110	.000	.820	.000	.180
2	449.950	.000	.780	.000	.220
3	47.750	.000	.900	.000	.100
4	47.750	.000	.940	.000	.060
5	91.830	.000	.920	.000	.080
6	122.130	.000	.870	.000	.130
7	180.900	.000	.700	.000	.300
8	44.080	.000	.980	.000	.020
9	56.010	.000	.910	.000	.090
10	78.050	.000	.570	.000	.430
11	156.110	.000	.430	.000	.570
12	187.330	.000	.370	.000	.630
13	111.110	.000	.550	.000	.450
14	223.140	.000	.370	.000	.630
15	185.490	.000	.580	.000	.420
16	128.600	.000	.460	.000	.540
17	112.030	.000	.520	.000	.480
18	77.130	.000	.650	.000	.350
19	388.430	.000	.590	.000	.410
20	391.180	.000	.700	.000	.300
22	31.220	.000	.790	.000	.210
23	129.480	.000	.900	.000	.100
24	35.810	.000	.590	.000	.410
25	48.670	.000	.800	.000	.200
26	36.730	.000	.890	.000	.110
27	76.220	.000	.880	.000	.120
28	269.050	.000	.750	.000	.250
29	112.030	.000	.800	.000	.200
30	41.320	.000	.970	.000	.030
31	247.930	.000	.570	.000	.430
32	42.240	.000	.280	.000	.720
33	101.010	.000	.750	.000	.250

Table 9-2 (cont'd)

SPECIFICATIONS FOR CHANNEL SEGMENTS

NUMBER	TYPE	LENGTH-FT	SLOPE	ROUGHNESS	AVERAGE TRAVEL TIME(SEC)
36	4	2040.00	.01005	.035	279.7
38	4	3000.00	.00683	.035	420.4
40	4	1300.00	.00846	.035	194.3
42	4	1800.00	.00556	.035	311.3
44	4	2550.00	.00784	.035	303.0
46	4	1500.00	.00233	.035	405.4
48	4	1500.00	.00367	.035	260.5
50	4	3050.00	.00213	.035	695.4
52	4	450.00	.00890	.035	53.9
54	4	2700.00	.00630	.035	457.4
58	4	1900.00	.00526	.035	370.1
60	4	1950.00	.00436	.035	417.3
62	4	2150.00	.00279	.035	428.3
64	4	2040.00	.00196	.040	559.1

DEFINITION OF NUMERIC CHANNEL TYPES

1 RECTANGULAR
2 TRIANGULAR
3 CIRCULAR
0 IRREGULAR WITH NO FLOOD PLAIN
4 IRREGULAR WITH FLOOD PLAIN

SPECIFICATIONS FOR STORAGE SEGMENT 39

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	950.000	.000	.275
12.862	861.329	955.000	5.000	4.870
49.602	1437.362	960.000	10.000	10.000
90.038	1722.176	963.000	13.000	13.000
114.748	3140.884	965.000	15.000	16.160

SPECIFICATIONS FOR STORAGE SEGMENT 41

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	944.800	.000	.000
.016	188.202	945.800	1.000	.033
3.219	978.650	950.000	5.200	3.581
26.980	1248.391	952.800	8.000	8.750
47.328	3726.467	955.000	10.200	14.340

SPECIFICATIONS FOR STORAGE SEGMENT 56

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	948.000	.000	.000
1.100	513.598	950.000	2.000	1.100
18.425	1460.765	955.000	7.000	11.390
61.162	1656.047	956.000	8.000	13.390
60.604	3511.627	958.000	10.000	19.390
139.564	10473.691	960.000	12.000	26.630

SPECIFICATIONS FOR STORAGE SEGMENT 61

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	923.000	.000	.000
.172	573.265	925.000	2.000	.172
2.285	2006.429	930.000	7.000	1.270
22.680	2569.089	934.000	11.000	9.750
45.643	3576.377	935.000	12.000	11.750

SPECIFICATIONS FOR STORAGE SEGMENT 63

STORAGE (ACRE-FEET)	DISCHARGE (CFS)	ELEVATION (FEET-MSL)	HEAD (FEET)	SURFACE AREA (ACRES)
.000	.000	916.000	.000	.000
.015	181.584	917.000	1.000	.030
2.529	1235.906	920.000	4.000	3.673
44.507	2811.881	925.000	9.000	18.640
166.156	3578.082	929.000	13.000	30.830
190.194	5226.805	930.000	14.000	32.830
382.850	30896.957	935.000	19.000	54.500

Table 9-3. Floods for Segments 41, 46 and 64 for Nancy Creek with Existing Conditions

Return Period (Years)	Probability of Exceedance (%)	Flood Flows (cfs)	Water Surface Elevations (feet)
Segment 41			
2-year	50	904	949.8[1]
5-year	20	1270	952.0
10-year	10	1480	953.1
25-year	4	1750	953.4
50-year	2	1950	953.5
100-year	1	2150	953.7
Segment 46			
2-year	50	1460	6.8[2]
5-year	20	1980	8.1
10-year	10	2320	8.6
25-year	4	2750	9.0
50-year	2	3070	9.3
100-year	1	3390	9.6
Segment 64			
2-year	50	2210	8.3[2]
5-year	20	2830	9.5
10-year	10	3250	10.1
25-year	4	3770	10.6
50-year	2	4150	10.8
100-year	1	4540	11.2

[1]From mean sea level.

[2]From channel bottom.

2. Projected Development with Current Channel System

This simulation is based on complete watershed development and an assumption that the current channel system will remain unchanged from present conditions. Table 9-4 shows the projected specifications for the source areas. Table 9-5 shows the results of the flood frequency analyses for the same segments as simulation 1. Increases in peak flows ranged from 30% for segment 41 to 5% for segment 64.

3. Inclusion of a Dam on West Branch

Flows were simulated for the storm event producing the largest flows in the first simulation, July 10, 1928, in order to evaluate the effect of a dam on the west branch in reducing downstream peak flows. The suitable site for a dam appeared to be the end of reach 58 with a design water surface at elevation of 950.0. Segment 58 was converted into a storage segment with a detention structure at its lower end. The changes in input data are the outlet works of storage segment 58. The specifications for all other elements remain the same as those given in Table 9-2.

The results of this simulation shows that the proposed detention storage is not effective in reducing peak flows downstream.

V. Summary and Conclusions

It was found that prevention of flood damages in the portion of the Nancy Creek basin included in the present study would require channels of much larger capacity that those existing. Channelization, however, would be costly and would aggravate flooding downstream. Construction of a detention dam on West Branch did not substantially reduce downstream flood problems An economic analysis of the flood

Table 9-4. Specifications for Area Segments With Projected Development

SPECIFICATIONS FOR SOURCE AREAS

NUMBER	AREA-ACRE	FRACTION OF AREA WITH EACH SOIL TYPE RAPID	MODERATE	SLOW	IMPERVIOUS
1	538.110	.000	.660	.000	.340
2	449.950	.000	.680	.000	.320
3	47.750	.000	.580	.000	.420
4	47.750	.000	.750	.000	.250
5	91.830	.000	.580	.000	.420
6	122.130	.000	.750	.000	.250
7	180.900	.000	.400	.000	.600
8	44.080	.000	.400	.000	.600
9	56.010	.000	.400	.000	.600
10	78.050	.000	.300	.000	.700
11	156.110	.000	.150	.000	.850
12	187.330	.000	.300	.000	.700
13	111.110	.000	.400	.000	.600
14	223.140	.000	.150	.000	.850
15	185.490	.000	.150	.000	.850
16	128.600	.000	.300	.000	.700
17	112.030	.000	.520	.000	.480
18	77.130	.000	.600	.000	.400
19	388.430	.000	.500	.000	.500
20	391.180	.000	.400	.000	.600
22	31.220	.000	.750	.000	.250
23	129.480	.000	.750	.000	.250
24	35.810	.000	.590	.000	.410
25	48.670	.000	.400	.000	.600
26	36.730	.000	.600	.000	.400
27	76.220	.000	.700	.000	.300
28	269.050	.000	.750	.000	.250
29	112.030	.000	.750	.000	.250
30	41.320	.000	.750	.000	.250
31	247.930	.000	.500	.000	.500
32	42.240	.000	.150	.000	.850
33	101.010	.000	.750	.000	.250

Table 9-5. Floods for Segments 41, 46 and 64 for Nancy Creek with Future Development

Return Period (years)	Probability of Exceedance (%)	Flood Flows (cfs)	Water Surface Elevations (feet)
Segment 41			
2-year	50	1190	952.8[1]
5-year	20	1630	953.2
10-year	10	1920	953.5
25-year	4	2290	953.8
50-year	2	2570	954.1
100-year	1	2840	954.3
Segment 46			
2-year	50	1700	7.5[2]
5-year	20	2330	8.5
10-year	10	2740	9.0
25-year	4	3270	9.5
50-year	2	3660	9.8
100-year	1	4040	10.1
Segment 64			
2-year	50	2420	9.0[2]
5-year	20	2990	10.0
10-year	10	3380	10.2
25-year	4	3860	10.7
50-year	2	4220	11.0
100-year	1	4570	11.2

[1]From mean sea level.

[2]From bottom of channel.

control alternatives is needed to select suitable methods for relieving the problems in Nancy Creek. Purchase of property in the flood plain and flood proofing need to be considered.

UROS04: URBAN FLOOD SIMULATION MODEL

Part III. A Gaging System for Flood Measurements in DeKalb County

for
Board of Commissioners
DeKalb County, Georgia

by
James R. Wallace
School of Civil Engineering

March 1976

1976

GEORGIA INSTITUTE OF TECHNOLOGY
Atlanta, Georgia

UROS4: URBAN FLOOD SIMULATION MODEL

Part 3. A Gaging System for Flood Measurements in DeKalb County

for

Board of Commissioners

DeKalb County, Georgia

by

James R. Wallace

School of Civil Engineering

Georgia Institute of Technology

Atlanta, Georgia

March, 1976

ACKNOWLEDGMENTS

This report is one of a series of three reports presenting the work at Georgia Institute of Technology for DeKalb County, Georgia on a project entitled "Utilization of a Computer Model to Determine the Impact of Urban Development of Flooding in DeKalb County." Work on the project began December, 1973, and was completed March, 1975. Alan M. Lumb, with the assistance of J. R. Wallace and L. D. James, directed the project. Dr. James assisted with the development and application of UROS4, while Dr. Wallace directed the work in gage site selection and installation. Paul Sanders and W. M. Sangster provided invaluable administrative assistance.

Eleven students worked on the project, and three made major contributions. John Clerici worked on the project for the entire period and took major responsibility in the collection of the field data. Jack Kittle worked on the programming of UROS4, the creation of the runoff files and program applications. D. Rao assisted with the statistical analysis and was very productive in program applications. Three graduate students supported by the Corps of Engineers, Carvel Deese, Brian Tarras, and Roy Powell, worked on the project as part of their studies and their efforts are gratefully appreciated. Although only working with the project one quarter, Tom Debo developed the procedures for and performed most of the work on the determination of the fraction of impervious area. The services of six other research assistants, Ed Sing, Paul Nowak, Adnan Saad, Tim Hassett, Jaime Nino-Pinto and Steve Todd, are gratefully acknowledged. Mr. Lloyd Willard, a technician in the School of Civil Engineering, constructed the instrument shelters and raingage recepticles and assisted in gage installation.

ABSTRACT

Urban development has occurred so rapidly in the Atlanta Metropolitan Area that the citizens and their governments have not been able to deal adequately with the associated flood and drainage problems. As the idealistic approach of locating everyone and everything on higher ground is costly if not impossible, the welfare of DeKalb County can best be served by a combination of 1) tributary area land use planning, 2) flood plain management including land use planning and regulation of flood plain building practices, and 3) structural measures involving detention storage and drainage system improvements. Selection of a successful combination requires information on how land surfaces and drainageways respond to a variety of precipitation patterns. Since watershed configurations and precipitation patterns are so complex and varied, hydrologic simulation is the only method powerful enough to determine fully the effects of land use and channel changes on flood elevations.

In order to provide a working simulation model for use by DeKalb County, the Urban Flood Simulation Model was developed. Rainfall, streamflow, and soils data were analyzed with the Stanford Watershed Model to develop an historic data file of rainfall excess for the range of land surface conditions found in DeKalb County. The Urban Flood Simulation Model simulates floods given the data file and prescribed physical characteristics of as many as 100 area, channel, and storage segments in a selected drainage area (Snapfinger Creek for example). The Model will calculate flood elevations and assocaited probabilities for critical points specified in the input data. Though collecting, coding, and checking the data on the physical characteristics may take a man-month or more, depending on watershed size and resolution, once the coding is complete it is relatively easy to explore the effects of changing land-use, altering the drainage system, or adding detention storage.

The procedures used in developing the file of runoff data, the computational framework, the computer programming, and the recommended procedures for collecting and coding data on drainage characteristics are all described in a companion report, "Part 1. Documentation and Users Manual." The second report, "Part 2. Applications to Selected DeKalb County Watersheds," illustrates use of the model in hydrologic studies. Eight DeKalb County watersheds were studied in varying degrees of detail, and preliminary assessments were made of the hydrologic aspects of the problems and potential solutions. This report, Part 3, describes elements of the study associated with the establishment of six DeKalb County gaging stations for the collection of rainfall and streamflow data. These gages, which will be operated by the county, will provide hydrologic data which can be used to refine present model calibaration.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SECTION I

Introduction

DeKalb County is currently faced with many difficult hydrologic problems such as flooding, erosion, and diminishing water quality. Hydrologic simulation is a tool that planners and engineers can use to help solve some of these problems. In order to provide a working simulation model for use by DeKalb County, the Urban Flood Simulation Model (UROS4) was developed at Georgia Tech. This model simulates the hydrologic processes which determine the rate and volumes of storm runoff and the resulting streamflow. These processes include precipitation, evaporation, transpiration of plants, infiltration of water into soil, drainage of storm water over the ground surface, movement of water down the stream channel system, and storage and detention of water on the ground surface, in the soil, in stream channels and in lakes and reservoirs.

The DeKalb Model, UROS4, is general in nature and can be used to simulate a wide range of hydrologic conditions and watershed characteristics. By the correct selection of the values of the constants (parameters) that occur in the mathematical expressions which make up the model, this general model can be used to simulate specific watersheds and drainage systems.

The selection of appropriate parameter values is referred to as model calibration and is one of the most important parts of a simulation study. Model calibration is an iterative procedure which involves (1) initial estimation of parameter values, (2) simulation of historical (previously measured and recorded) storm runoff events (floods),

(3) comparison of simulated and measured floods, and (4) adjustment of parameter values to bring simulated floods in line with historical data. This process is repeated until the model is capable of reproducing the historical events.

Calibration of the DeKalb Model has, by necessity, been accomplished with data from watersheds outside of DeKalb County but within the metropolitan Atlanta area (see ref. 1). At least two years of rainfall and streamflow records are required for model calibration, and records of this length were not available within the County at the time of the Georgia Tech study. Therefore, as part of the Georgia Tech study of the DeKalb flooding situation, rainfall and streamflow gages were installed and will be operated by the County to collect additional information which can be used to refine the present model calibration. Collection of this additional data is needed even though some data of this type are currently being collected in the County by the U. S. Geological Survey.

The characteristics of the soils in a watershed can significantly affect the quantity of water which moves rapidly to a stream following a rainstorm. Therefore, the Georgia Tech gages have been located on some watersheds which contain very porous soils and on some which contain very tight or impermeable soils so that the entire range of soil types present in DeKalb County will be included. The degree to which the watershed has been developed is also a significant factor in determining the amount and rate at which runoff builds up in the streams, and the gages have been located to cover a range of land use and development.

The streamflow gaging procedures adopted for this study require the gaging stations to be located near the entrance to culverts through which the stream is flowing. Hydraulic characteristics of a culvert are affected by the geometry of the culvert, by the geometries of the channels upstream and downstream of the culvert, by the slope of the culvert, and by the presence of silt or debris in the channel and culvert. Many culverts, because of their undesirable hydraulic characteristics, are not suitable for gage location. Since rainfall is recorded at the site of the streamgage, the site must be free of overhanging vegetation and should not be too close to tall trees or buildings which could affect the rain gage catch. Therefore, an important phase of establishing the location of the gages was to eliminate potential sites which did not have acceptable hydraulic characteristics or which did not permit reliable rainfall measurements.

The purpose of this report is to describe in detail the various activities and criteria that were involved in the selection of the gage locations, to describe the watersheds selected for gaging, and to describe the gage site and the associated flow rating curves.

SECTION II

Criteria for Site Selection

For an orderly study to be conducted, it was necessary to develop criteria to be used for selection of the gaging sites. These criteria were then used as a basis for the development of a procedure for site selection.

Preliminary Data Collection

One of the initial steps was the collection of numerous forms of data which were analyzed and stored for future use. A composite map was developed which showed the location and watershed boundaries of all U.S.G.S. gages in the County, along with the twelve watersheds which the County Planning Commission had indicated as problem areas. This map also served as an index for the U.S.G.S. quadrangle sheets and the large-scale (1" - 200') planning maps provided by the County. Another map was developed which showed the limits of all Corps flood plain information reports. (None of the proposed Georgia Tech gage sites were covered by the Corps reports). Meetings were held with County planning officials and the County drainage engineer. Both the Mobile and Savannah Districts of the Corps of Engineers were contacted. The Mobile District furnished cross-sections for Burnt Fork, Lullwater, and Peavine Creeks, copies of flood plain information reports, and a set of curves depicting discharge versus drainage area for 10-, 100-, and 500-year floods. The Savannah District furnished copies of flood insurance studies for both the City of Decatur and DeKalb County, a summary list of all flood plain information studies in the area, and regional flood frequency curves developed from data collected in the vicinity around Atlanta.

General Criteria

Three general criteria were developed which centered around (1) the soils to be found in watersheds of the County, (2) the land use and stability of land use in the watersheds, and (3) the hydraulics of the culverts where the gages were to be installed.

Analysis of Soils Data. The permeability of a soil is a measure of the rate at which water can be transmitted through the soil, and it is a factor in estimating the rate at which rainfall can be absorbed by the soil. Thus, it was considered likely that soils of different permeabilities would have different runoff characteristics. Consequently, it was decided to cover by the gaging progrms the entire range of permeability values which are found in DeKalb County.

To evaluate the permeability of the various soil associations, several meetings were first held with the U.S. Soil Conservation Service (S.C.S.) which has a comprehensive soils study presently underway for DeKalb County. Although this study was not to have been completed until after the watershed study, the agency was very cooperative in providing basic soils data which had already been collected. In addition, a General Soils Map and brochure entitled "Soils Interpretation for Regional Planning in Metropolitan Atlanta" was completed by the S.C.S. for the Atlanta Regional Commission in 1967 and was available for the Tech study.

The 1967 soils map shows the distribution of soil associations throughout the county (Fig. 26, Ref. 1). Soil associations represent soils that occur together in a characteristic pattern, and may consist of a few or many soils which may be of similar or different types. Although closely associated geographically, the soils of an association may differ in permeability. Thus, it was necessary to examine the permeability of the individual soils comprising the associations.

The individual soils found in DeKalb County are listed in Table 1 along with average permeabilities and the hydrologic soil group to which each soil belongs. The average permeability assigned to each soil is a weighted value based on the thickness and permeability of various layers which make up the soil profile. Information on the layering of the soils was furnished by the S.C.S. The permeability of each layer was classified by the S.C.S. according to the scale shown in Table 2, and the averaging was done by Georgia Tech. For comparison, the description of the four hydrologic soil groups used by the S.C.S. are presented in Table 3 and the group pertaining to each soil is listed in Table 1.

The next step in the soils analysis was to combine the permeability data for each soil to provide a weighted permeability for each soil association. The weighted average permeability was computed by multiplying the average permeability of each soil, as given in Table 1, by the percentage of occurance of each soil in the association. For example, soil association number 2, as shown in Table 4, is composed of 50% Wilkes, 20% Iredell, and 20% Mecklenburg with 10% unspecified. The resulting weighted permeability is

$$\frac{(50 \times 0.6) + (20 \times 0.2) + (20 \times 0.2)}{90} = 0.42 \text{ inches/hr.}$$

Table 1. Permeability of DeKalb County Soils

Soil	Average Permeability (inches/hour)	Hydrologic Soil Group
Altavista	2.0	C
Appling	1.8	B
Alluvial	4.0	B
Cecil	1.8	B
Chewacla	1.3	C
Congaree	1.9	B
Davidson	1.8	B
Gwinnett	1.3	B
Iredell	0.2	D
Linker	2.2	B
Louisa	2.2	B
Louisburg	13.0	B
Madison	1.3	B
Mecklenburg	0.2	C
Musella	2.2	B
Pacolet	1.8	B
Red Bay	1.3	B
Wedowee	2.2	B
Wehadkee	1.8	D
Wickham	1.8	B
Wilkes	0.6	C

Table 2. Classification of Soil Permeabilities

Permeability Class	Numerical Range (inches per hour)	Average Rate (inches per hour)
Very slow (VS)	Less than 0.06	-
Slow (S)	0.06 - 0.2	0.13
Moderately slow (MS)	0.2 - 0.6	0.40
Moderate (M)	0.6 - 2.0	1.30
Moderately rapid (MR)	2.0 - 6.0	4.00
Rapid (R)	6.0 -20.0	13.00
Very rapid (VR)	More than 20	-

Table 3. Hydrologic Soil Groups (S.C.S.)

Hydrologic Soil Group	Description
A	Soil with lowest runoff potential. Deep sands with very little silt and clay.
B	Mostly sandy soils less deep than A.
C	Shallow soils and soils containing considerable clay.
D	Soil with highest runoff potential. Clay soils and shallow soils with impermeable subhorizons.

Table 4. Permeabilities Assigned to Soil Associations

SOIL#	SOIL ASSOCIATION	%COMPONENTS			PERMEABILITY				HYDROLOGIC GROUP			
		1st	2nd	3rd	1st	2nd	3rd	wt ave	1st	2nd	3rd	wt ave
1	Alluvial Land Chewacla-Weh Adkee	60	20	10	MR	M	M	3.16	B	C	D	B
1A	Congaree-Chewacla Weh Adkee	50	20	15	M	M	M	1.74	B	C	D	B
2	Wilkes-Iredell Mecklenburg	50	20	20	MS	S	S	.42	C	D	C	C
3	Madison-Louisa Pacolet	45	25	15	M	MR	M	1.65	B	B	B	B
4	Appling-Cecil Madison	45	30	10	M	M	M	1.74	B	B	B	B
5	Madison-Pacolet Gwinnett	40	25	20	M	M	M	1.45	B	B	B	B
6	Gwinnett-Pacolet Musella	45	25	15	M	M	MR	1.61	B	B	B	B
7	Gwinnett-Davidson Musella	45	25	15	M	M	MR	1.61	B	B	B	B
8	Louisburg-Wedowee Pacolet	40	30	20	R	M	M	6.91	B	B	B	B
9	Appling-Louisburg Pacolet	50	25	15	M	R	M	4.91	B	B	B	B
10	Madison-Pacolet Gwinnett	40	30	20	M	M	M	1.47	B	B	B	B

11	Linker-Louisburg Musella	60	15	10	MR	R	MR	4.11	B	B	B	B
12	Pacolet-Gwinnett	40	35	10	M	M	R	2.91	B	B	B	B
13	Wickham-Altavista Red Bay	65	20	10	M	M	M	1.79	B	C	B	B
14	Appling-Pacolet Louisburg	60	20	10	M	M	R	3.04	B	B	B	B
15	Wilkes-Gwinnett Musella	50	30	10	MS	M	MR	1.01	C	B	B	C
16	Appling-Pacolet Gwinnett	40	35	10	M	M	M	1.84	B	B	B	B
17	Louisburg-Pacolet Wedowee	37	35	21	R	M	M	6.35	B	B	B	B
18	Rock Outcrop	-	-	-	VS			0.0	D			D
19	Made Land	-	-	-								
20	Unclassified											

On the basis of the average weighted permeabilities shown in Table 4, each soil association was assigned to one of the S.C.S. groups shown in Table 5.

Objectives of the gaging program were to measure runoff from watersheds representing the range of permeabilities found in DeKalb County and to supplement other gaging programs which have recently been initiated. To accomplish these objectives the watersheds being gaged by the U.S.G.S. were analyzed to determine their average permeabilities. This was done by measuring the percentage of each U.S.G.S. watershed covered by the various soil associations and computing an average permeability for each watershed. The results (see Table 6) show that most of the U.S.G.S. watersheds have moderate permeability and that no watersheds with slow or rapid permeabilities are being gaged. Inspection of the soils map (Fig. 26, Ref. 1) shows the presence of soils with slow permeability in the southwest part of the county and soils with rapid permeability in the east boundary of the county. Therefore, potential gage locations were determined to be in these areas.

Analyses of Land Use. Land use can have a strong influence on runoff characteristics. The U.S. Geological Survey had divided land use in their gaged watersheds into the following six catagories:

1. Single-family residences
2. Multi-family residences
3. Commercial
4. Industrial
5. Parks
6. Undeveloped

Land use in the U.S.G.S. watersheds was analyzed and an attempt was made

Table 5. General Classification of Average Permabilities of Soil Associations

Permeability	Soil Association Number
Very slow (VS)	18
Moderately Slow (MS)	2
Moderate (M)	1A, 3,4,5,6,7,10,13,15,16
Moderately Rapid (MR)	1,9,11,12,14
Rapid (R)	8,17

Table 6. Permeabilities of Watersheds Gaged by U.S.G.S.

U.S.G.S. Site Number *	Soils Associations and Percent in Drainage Basin				Weighted Permeability for Drainage Basin (inches/hour)
1	1-9%	10-38%	5-53%		1.61
7	1-11%	12-25%	10-12%	5-52%	2.01
8	1-11%	12-33%	5-56%		2.12
9	1-9%	12-15%	10-26%	5-50%	1.83
12	1-12%	10-35%	5-31%	20-22%	1.72
16	1-14%	10-39%	5-47%		1.69
21	1-10%	12-47%	5-43%		2.31
26	1-7%	10-9%	5-15%	20-69%	1.82
27	1-9%	10-32%	5-59%		1.61
29	12-44%	5-59%			2.09
32	1-8%	12-43%	5-49%		2.21
57	1-14%	10-41%	5-45%		1.70
2-3371	1-10%	12-51%	5-39%		2.37
2-3367	1-4%	10-44%	5-52%		1.53
2-2136	20-100%				–

* For site locations see Figures 21 and 22, Ref. 1.

to augment their coverage with the Georgia Tech gages wherever possible. The land-use characteristics of the recently installed U.S.G.S. gages are shown in Table 7. It was also desirable for the pattern of land use to be relatively stable in each watershed, a requirement for a straightforward evaluation of streamflow records and watershed model calibration.

The land use characteristics of the watersheds tentatively selected for the Georgia Tech gaging program were subdivided in a manner similar to that used by the U.S.G.S. Photo revised quadrangle maps, aerial photographs, and field inspections were used in making this analysis.

Data previously compiled by the County was obtained and studied to gain an understanding of present and expected future patterns of land use. DeKalb County planning officials were also consulted about the possibilities of future development in the watersheds proposed for gaging. Three separate map sources were used by the County planners as a basis for forming opinions concerning future land use. The watersheds were located on the General Soils Map to determine the suitability of the soils in the area for future development. The watersheds were also located on long-range sewerage plans developed by the County. Since rapid changes in land use normally follow new sewerage systems, this method provides a good indication of stability. Finally, composite land use maps were investigated.

Culvert Hydraulics. Consideration of culvert hydraulics is a very important aspect in the selection of sites for streamflow gaging stations. The placement of a roadway fill and culvert in a stream may

Table 7. Land Use in Watersheds Gaged by U.S.G.S.

GAGE LOCATION	*LAND USE IN%					
USGS Gage Sites	SFR 1	MFR 2	COM 3	IND 4	PRKS 5	UND 6
Shoal Cr. at Line St.	62	8	7	2	16	5
Cobb Cr. at Snapfinger Rd.	42	21	7	0	0	30
Trib. to Shoal Cr at Glendale Rd.	72	0	9	0	9	10
Shoal Cr. at Rainbow Dr.	44	20	4	0	12	20
Sugar Cr. at Clifton Church Rd.	52	7	6	2	8	25
S. Fork Peachtree Cr. at Montreal Rd.	30	6	6	18	0	40
Trib. to N. Fork at Drew Valley Rd.	90	0	0	10	0	0
N. Fork at Shallowford Rd.	29	1	7	12	6	45
Trib. to S. Fork at E. Rock Springs Rd.	64	18	2	0	8	8
Trib. to S. Fork at Scott Blvd.	40	13	15	3	19	10
Trib. to N. Fork at Meadowcliff Rd.	72	0	0	0	8	20
Trib. to Nancy Cr. at Plantation Ln.	14	0	18	58	0	10
South Utoy Cr at Adams Dr.	54	11	9	8	8	10
Camp Cr. at Park Terrace	73	11	1	0	0	15
Trib. to S. Utoy at Headland Dr.	85	15	0	0	0	0
Trib. to S. Utoy at Woodberry Dr.	60	0	10	0	0	30
Trib. to S. Utoy Cr. at Ft. Valley Dr.	72	9	0	0	9	10
S. Fork Peachtree Cr. at Willivee Dr.	35	7	7	21	0	30

cause an abrupt change in the character of flow, producing rapidly varied flow in which acceleration rather than boundary friction plays a primary role in determining the elevation of the water surface. The flow through a culvert can be classified as one of six types with with only two of these types of flow being well suited for the type of indirect flow measurement considered feasible for this study. This indirect method is used extensively to measure flood discharges from small drainage basins such as proposed for gaging by Georgia Tech, and has the advantage of eliminating the need for a control structure such as a weir. The six types of culvert flow can be classified on the basis of the control section and the relative heights of the headwater and tailwater elevations as summarized in Table 8. (2). The following general classification can be made from Table 8 and Figure 1:

If h_4/D is equal to or less than 1.0 and $(h_1-Z)/D$ is less than 1.5, only types 1, 2, and 3 flow are possible.

If h_4/D is greater than 1.0 only type 4 flow can exist.

If h_4/D is equal to or less than 1.0 and $(h_1 - Z)/D$ is equal to or greater than 1.5, only types 5 and 6 flow are possible.

For flow types 1 and 2 various critical depths can be assumed, and the corresponding discharges and headwater elevations computed. Stage-discharge relationships can thus be readily calculated for culverts where this type flow exists, and gage readings can be accurately interpreted from the time of installation. A rating curve for type 3 flow is not readily developed, because the discharge is a function of both the outlet area and the fall between the headwater and tailwater pools. To develop a rating curve for type 3 flow the discharge must be computed for several combinations of tailwater elevation as well as fall. These discharges are then plotted against fall through the culvert (head-

Table 8. Classification of Culvert Flow

Flow Type	Description	Barrel Flow	Location of terminal section	Kind of Control
1	Critical Depth at Inlet	Partly Full	Inlet	Critical Depth
2	Critical Depth at Outlet	Partly Full	Outlet	Critical Depth
3	Tranquil Flow Throughout	Partly Full	Outlet	Backwater
4	Submerged	Full	Outlet	Backwater
5	Rapid Flow at Inlet	Partly Full	Inlet	Entrance Geometry
6	Full Flow, Free Outfall	Full	Outlet	Entrance and Barrel Geometry

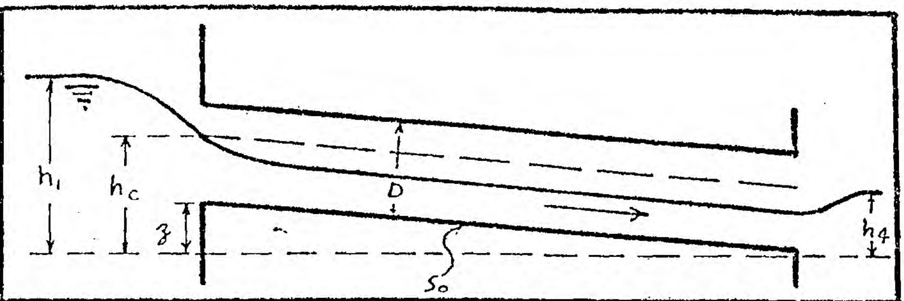
TYPE FLOW EXAMPLE

1
Critical Depth At Inlet

$\frac{h_1 - z}{D} < 1.5$

$h_4/h_c < 1.0$

$S_o > S_c$

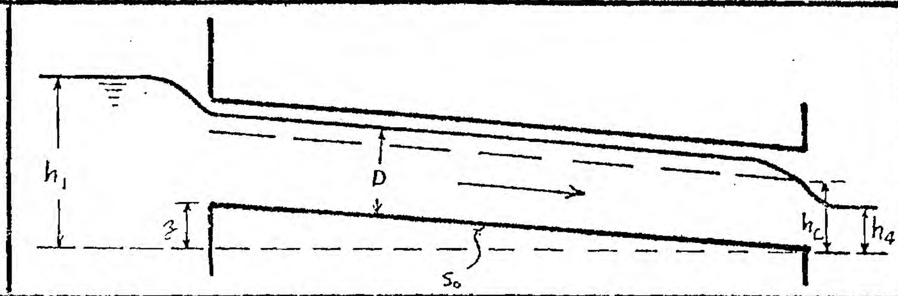


2
Critical Depth At Outlet

$\frac{h_1 - Z}{D} < 1.5$

$h_4/h_c < 1.0$

$S_o < S_c$

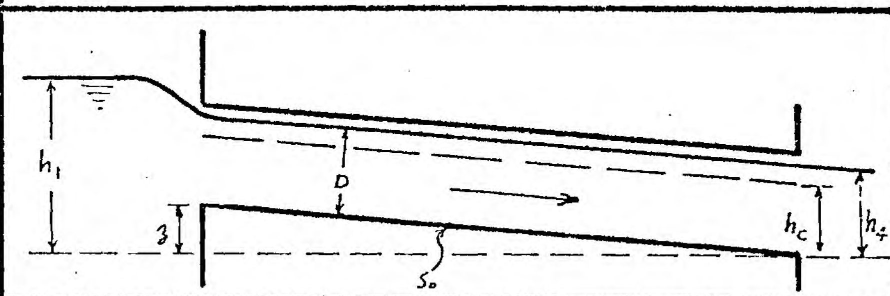


3
Tranquil Flow Throughout

$\frac{h_1 - Z}{D} < 1.5$

$h_4/D \leq 1.0$

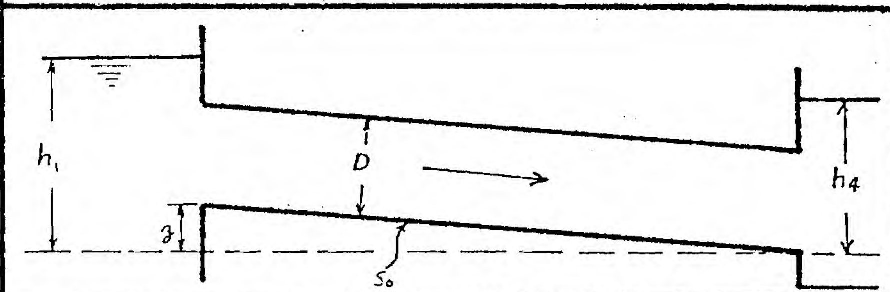
$h_4/h_c > 1.0$



4
Submerged Outlet

$\frac{h_1 - Z}{D} > 1.0$

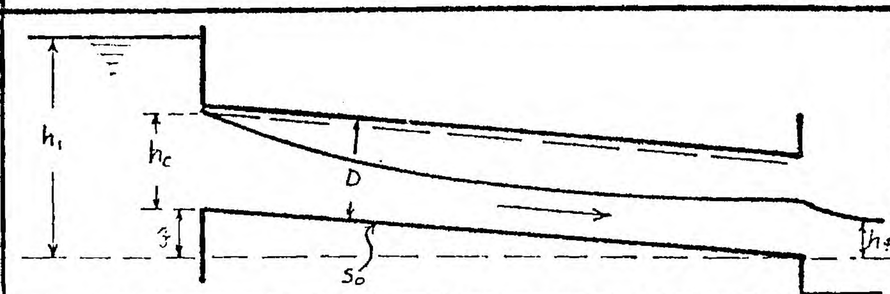
$h_4/D > 1.0$



5
Rapid Flow at Inlet

$\frac{h_1 - Z}{D} \geq 1.5$

$h_4/D \leq 1.0$



6
Full Flow Free Outlet

$\frac{h_1 - Z}{D} \geq 1.5$

$h_4/D \leq 1.0$

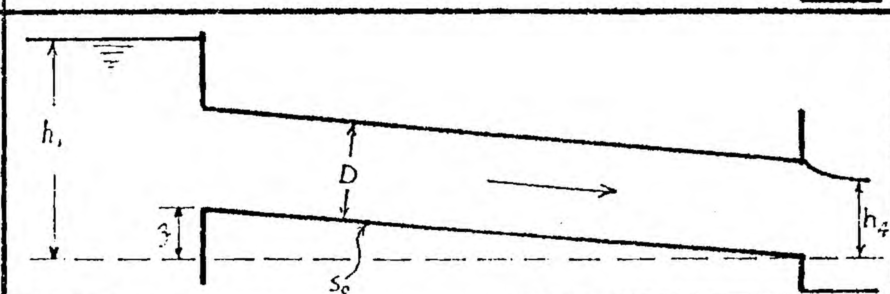


Figure 1. Classification of Culvert Flow

water minus tailwater) and curves drawn connecting points of equal tailwater. Computed rating curves for type 3 flow should be verified by current-meter measurements. Such measurements must be gathered over a wide range of flow conditions for adequate verification of the computed rating curve, which usually results in a long period of time elapsing before gage data can be of value. A recent check by the U.S.G.S. on such a culvert in DeKalb County indicated that actual discharges were 75% less than those calculated using their culvert rating program.

At many small-area gaging stations experience has shown that either types 1, 2, or 4 may be expected through a considerable range of discharge. At high heads, types 1 and 2 flow will usually change to either type 5 or 6 flow, respectively. For example, a steep culvert with free getaway might always support type 1 flow until the headwater-diameter (H/D) ratio reaches 1.5, at which time flow may become either type 5 or 6. (2).

For type 1 flow very reliable discharge computations may be expected up to a H/D ratio of 1.25. Good results may also be expected for type 5 flow when the flow condition is definitely known and the H/D ratio is greater than 1.75. For type 6 flow, good flow measurements may always be expected. In the range of transition between types of flow, better measurements may be expected from circular rather than box culverts, but the results should not be rated better than fair in either case. (2).

For type 4 flow ponded conditions generally exist, which is not a desirable condition for accurate discharge measurements.

For all kinds of indirect measurements the quality of the field data will be a factor in the accuracy of the measurement. Factors to be considered are:

1. Accuracy to which headwater and tailwater elevations can be determined.

2. Stability of the approach channel.

3. Closeness of the entrance conditions to that of a standard.

4. Shape and condition of the culvert.

5. Scour or fill in the culvert.

6. Possibility of the culvert being partially plugged by debris at the time of peak. (2)

Field Analysis of Culvert Hydraulics

A preliminary field screening of proposed sites was conducted before detailed field data were obtained. Some watersheds which contain ponds or other forms of detention structures were eliminated from further consideration. This factor is important in small watersheds because a greater proportion of the total watershed is controlled by a given size lake.

Both the approach to and the exit from the culvert should be fairly straight and perpendicular to the culvert headwall. Where the approach channel is curved, it is necessary to install a crest gage on each side of the culvert on the upstream side. This criterion should also be used for the exit channel. The U.S.G.S. follows such a policy, and has recorded as much as a foot of difference between two crest gages on opposite banks of a curved approach section. In all cases the recording gages are supplemented by at least one crest gage upstream and one downstream of the culvert.

It is also important that the carrying capacity of the approach channel be greater than that of the culvert. If this condition is not met, the required sudden contraction of the cross-sectional area between the approach section and the culvert will not occur. No tributaries should enter the stream between the approach section and the culvert. Such conditions would lead to erroneous discharge measurements since discharges are computed from stages measured at the approach section, and such measurements would not reflect the additional discharge from the tributary. This is an inherent problem for urban study areas. It is quite common in urban areas for storm drainage to follow the streets to a low point such as a creek and to be discharged at the culvert. This inflow is not accurately reflected in the discharge measurement.

The culvert should be free of silt deposits. Cleaning the culvert will often not solve the problem, because the siltation problem is frequently caused by backwater effects which are independent of the culvert geometry. Many of the sites which otherwise appeared satisfactory were eliminated due to the presence of silt.

A very important factor of site selection is having the proper type of flow occurring in the culvert. The immediate downstream reach should be inspected for obstructions which might cause tailwater conditions sufficient to drown out critical flow in the culvert. As pointed out in the previous section, flows must reach critical depth either at the entrance or the exit of a culvert in order to obtain accurate data over a short period of time. The type of culvert should also be noted. If standard culvert rating procedures are to be used only circular pipes, pipearches or rectangular box culverts should be considered.

Adequate freeboard above the top of the culvert is desirable to insure that the road is not likely to be overtopped. If overtopping occurs a rating curve developed by a culvert rating will not reflect the actual flood discharge. The possibility of a large amount of ponding upstream of the culvert entrance should also be checked during preliminary screening. This condition can cause inaccurate discharge measurement since the gage would, in effect, be measuring the outflow from a small reservoir rather than the inflow.

(It is advisable to check with the owners of the land on which the gages are to be located early in the site selection procedure to gain tentative approval. For the DeKalb study this matter was handled by the County Right-of-Way Department.)

Field data were obtained during the preliminary screening process and during gage installation. While the required field data are neither highly sophisticated nor time consuming to collect, the data should be thoroughly understood in order to eliminate return trips to any sites. There are two main functions of field data. First, it is the basis for determining the type of flow which occurs in the culvert. When final selection of sites is completed, it then provides the input data for computing the rating curve at the gaging station.

It is convenient to use the elevation of culvert invert at the downstream end of the culvert as the zero gage elevation for flow calculations. The elevation of the upstream and downstream inverts are required along with the length of the culvert in order to determine its slope. Complete details of culvert dimensions must be obtained. These include culvert projections, wingwall angles, size of fillets and chamfers, degree of entrance rounding, size and shape of opening, and type of entrance. The material of which the culvert is made must be described as well as its condition (good, fair, poor). For multiple culverts the minimum web thickness between each barrel should be recorded.

Roughness coefficients (n) for use in the Manning equation were selected (at the time of the field survey) for both the approach section and the culvert.

As previously discussed, the capacity of the approach section must be greater than that of the culvert. On the other hand, if the area is more than five times the area of the culvert, then zero approach velocity may be assumed and the section is not required. To avoid the possibility of the approach section being within the drawdown region, the section should be located at least one culvert width upstream from the culvert entrance. Where wingwalls exist, the proper location is at a distance upstream from the end of the wingwalls equal to the width between wingwalls at their upstream end. If the wingwalls do not cause a significant contraction, the section may be closer, but not closer than one culvert width. One culvert width at multiple culverts may be considered as the sum of the individual culvert widths. The cross section should be taken at right angles to the channel. Roughness coefficients must be assigned to the channel and each overbank.

High-water marks from previous storms are extremely valuable in estimating the type of flow which will occur in the culvert. These data provide the best method for determining if type 3 flow is occuring. The occurance of type 3 flow may eliminate the site from consideration for culvert rating purposes. High-water marks in the approach channel were obtained along the banks from the culvert entrance upstream a distance of at least two culvert diameters. Tailwater elevations were taken along the downstream embankment or channel. The elevation of several marks on each side of the culvert were obtained. The points on each side were averaged if each appeared representative of the actual peak flood elevation. The elevation of the low point in the road profile gives quick indication if overtopping by floodwaters is a problem. While it is not desirable, the site can still be used by considering the roadway as a broad-crested weir (3) and adjusting the rating curve accordingly. One method of accomplishing this would be to use a computer program such as HEC-2 (3) which handles bridge routines involving combinations of flow conditions such as weir flow and culvert flow. Successive computer runs with a varied discharge can then be made to develop the composite rating curve.

Determination of Flow Type

It is not readily apparent by field inspection of a culvert what type of flow will occur. For type 1 flow to occur, the culvert slope must be steep, i.e., normal depth must be less than critical depth. For type 2 flow to occur conditions must exist such that critical depth occurs at the culvert outlet. For either case to occur, the elevation of critical depth above datum (downstream invert) must be greater than the tailwater elevation.

A quick check for type 1 flow and type 2 flow was made with graphs such as those shown in Figure 9, Ref. 2.

It was next determined if backwater drowns out the critical depth and causes subcritical flow. The occurance of backwater can be established, if it exists, from high water marks in the downstream channel. In the absence of highwater marks an approximate method will provide an indication of flow conditions. Given a flow rate in the upstream channel and using any standard culvert design procedure, the headwater may be approximated in order to calculate a critical depth for the culvert using the procedure described above. For the tailwater a trapazoidal channel may be used to approximate the actual downstream conditions and normal depth computed using Manning's equation. This is obviously not as reliable as true measurements. A more sophisticated solution, which involves more field data, would be to acutally compute a backwater curve from some point downstream.

Gage Instrumentation

The purposes of the instruments installed at each gage site are (1) to provide a continuous record of the water surface elevation in the creeks upstream of the culvert, (2) to provide a continuous record of the rainfall accumulated at the site, and (3) to provide a separate record of the maximum elevation of the water level at the gages and at the downstream end of the culverts at which the gages are located. The layout of a typical installation is shown on Figure 2.

A single clock driven recorder was used to provide the continuous records of water elevation and precipitation. A Stevens Type A Model 71 Water Level Recorder with a mechanical rainfall recording accessory was selected for this purpose because it was the least costly reliable instrument available which allowed for the simultaneous recording of both rainfall and streamflow. This instrument records, on a strip chart, the elevation of a float in the gage stilling well and the elevaton of a float in the rain collector. Water from the creek flows into the stilling well through an inlet pipe laid from the bottom of the creek to the stilling well. Thus, the water in the stilling well stays at the same elevation as the water in the creek upstream of the culvert. The rain collector is mounted on the side of the box housing the

Point	Description
(1)	Invert at downstream end of culvert
(2)	Water elevation in channel at downstream end of culvert-low flow
(3)	Elevation of zero mark on downstream crest gage
(4)	Invert at upstream end of culvert
(5)	Elevation of zero mark on upstream crest gage
(6)	Top of reference rod
(7)	Elevation of invert of intake to stilling well
(8)	Street elevation

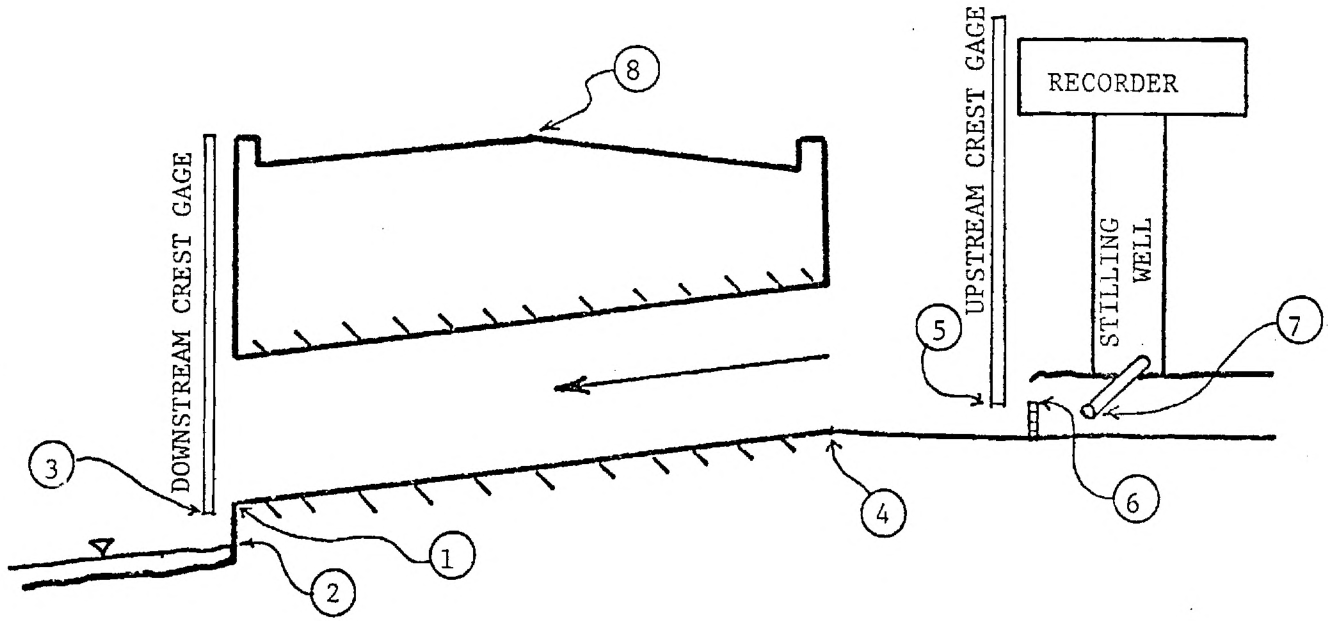


Figure 2. Typical Gage Installation

recorder. A float in the rain collector is connected to a pen on the recorder, and thus a continuous record of the accumulated rainfall is obtained along with a record of the water surface elevation.

Crest stage gages were installed at the upstream and downstream end of each culvert used for gaging. The purpose of these gages is to provide a record of the maximum water level which occurs during the period between visits to the site for data collection and instrument maintenance. A crest stage gage consists of a 2-inch plastic pipe with a graduated rod mounted inside. A small cup is attached to the bottom of the rod and the cup is filled with powdered cork. The pipe is mounted vertically on a post or on a culvert headwall, and when the water in the creek rises, water enters several small holes in the bottom of the pipe and floats the powdered cork. When the water recedes after a storm, the cork adheres to the graduated rod at the highest elevation reached by the water. When the gage is serviced, the powdered cork is wiped from the graduated rod and the cup refilled in preparation for the next period of measurement. The high water marks on the crest stage gages make it possible to check the high water elevation shown on the recorder chart, and, in the event that the recorder malfunctions during the storm, they provide a back up measurement which permits the peak flow during the storm to be determined. In those installations where the stream approaches the culvert at an angle, crest gages are installed on both sides of the creek to determine if the water level is higher on one side than the other. In this case the recorder values may be either too high or too low, and the values can be adjusted by averaging the elevations across the creek.

A crest stage gage was also installed at the down stream end of each culvert. The flow rate through the culvert is, in some cases, determined by both the headwater and tailwater elevations (see previous discussion on culvert hydraulics). To determine the type of flow occuring at the culvert, and hence the rate of flow, upstream and downstream water surface elevatons are required.

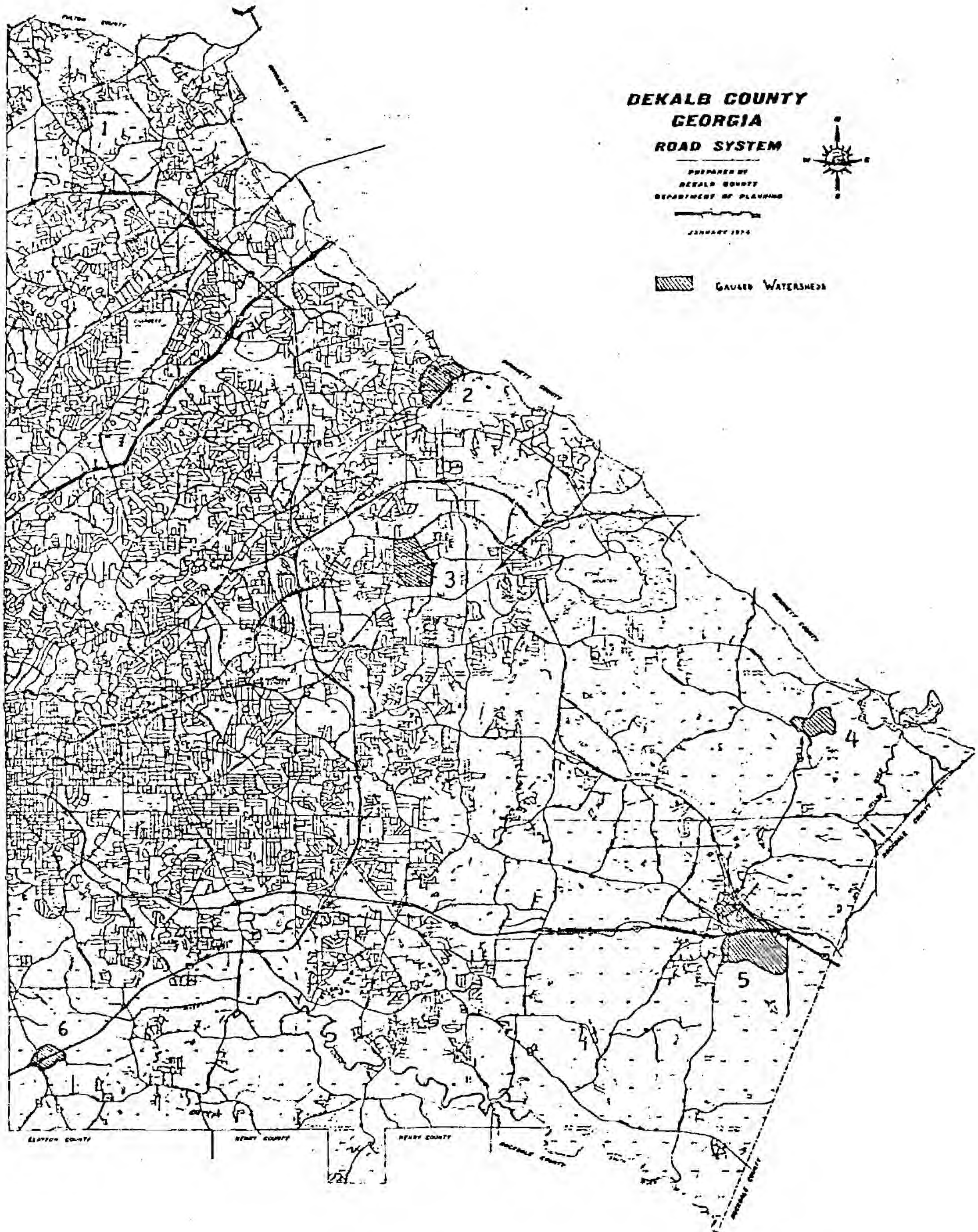


Figure 3. Location of Gage Sites

SECTION III

Watersheds Selected for Gaging

Approximately 60 sites were evaluated by the procedures previously discussed, and six of these sites were judged acceptable for gaging. The culverts at these six sites appeared to be experiencing either type 1 or type 2 flow, and all sites were adequate for the location of rain gages. The sites selected and summaries of their characteristics are discussed below, and the locations of the gaged watersheds are shown on Figure 3. The layout of a typical gage installation is shown in Figure 2.

SITE 1: Womack Creek Gage. The gage site is at a culvert which passes under Courtleigh Drive approximately 150 feet north of the intersection of Courtleigh Drive and Cambridge Drive. The creek is a headwater tributary to Womack Creek and drains a watershed of about 84 acres. The watershed is completely developed in single family residences and development conditions are expected to remain stable, with about 25 per cent of the watershed estimated to be covered by impervious surfaces. The soils in the watershed are of moderately rapid permeability. This watershed is a subarea of the Womack Creek watershed that was studied with the simulation model. The results of this study are described in reference 4.

The culvert under Courtleigh Drive is a 79" x 49" corrugated metal pipe arch 48.8 feet long with upstream and downstream wingwalls at approximately 30° angles to the centerline of the pipe. The slope of the pipe is 0.0236. The water level recorder is located about 12 feet upstream of the headwall. Specific elevations for the Womack Creek installation are as follows:

ITEM	ELEVATION(ft. above downstream invert)
(1) Invert at downstream end of culvert	0.00
(2) Water elevation in channel at downstream end of culvert under low flow conditions	-1.2' (approx.)
(3) Elevation of zero mark on downstream crest gage	-0.63'
(4) Invert at upstream end of culvert	1.15'
(5) Elevation of zero mark on upstream crest gage	3.36'
(6) Top of reference rod	2.57'
(7) Elevation of invert of intake to stilling well	1.41'
(8) Street elevation	7.0' (approx.)

The geometry of this installation indicates that type 1 flow will occur through the range of flow conditions expected at this site. The rating curve for type 1 flow at this site is shown on Figure 4. The rating curve is based on a datum of 0.0 feet at the downstream invert of the culvert. To correlate the reading on the recorder with the zero datum, a reference rod has been placed in the stream near the intake to the stilling well. The top of the reference rod is 2.57 feet above zero, and marks are inscribed on the rod at 0.1' intervals. When the recorder is serviced, the water level on the reference rod is read. This value is recorded by the reader and marked on the recorder chart. All water levels recorded on the recorder chart are then referenced to this elevation.

Site 2: Jackson Creek Gage. The gage on Jackson Creek is located at a culvert which carries the creek beneath West Hampton Drive in Tucker. A short distance downstream from the gage the creek flow into Gwinnett County where it is tributary to the Yellow River. Upstream of the gage the creek drains 278 acres. The watershed is bounded approximately by Lawrenceville Highway on the south and east and on the west and north by Old Norcross Road. About 60 per cent of the area has been developed and the land use is single family residences, with medium to low density development. This watershed contains soils of high permeability. Approximately 15 per cent of the watershed is covered by impervious surfaces, and a small pond is located in the upstream part of the watershed.

Two 7-foot circular corrugated metal pipes carry the flow beneath West Hampton Drive. The culverts are 60.6 feet long and have slightly

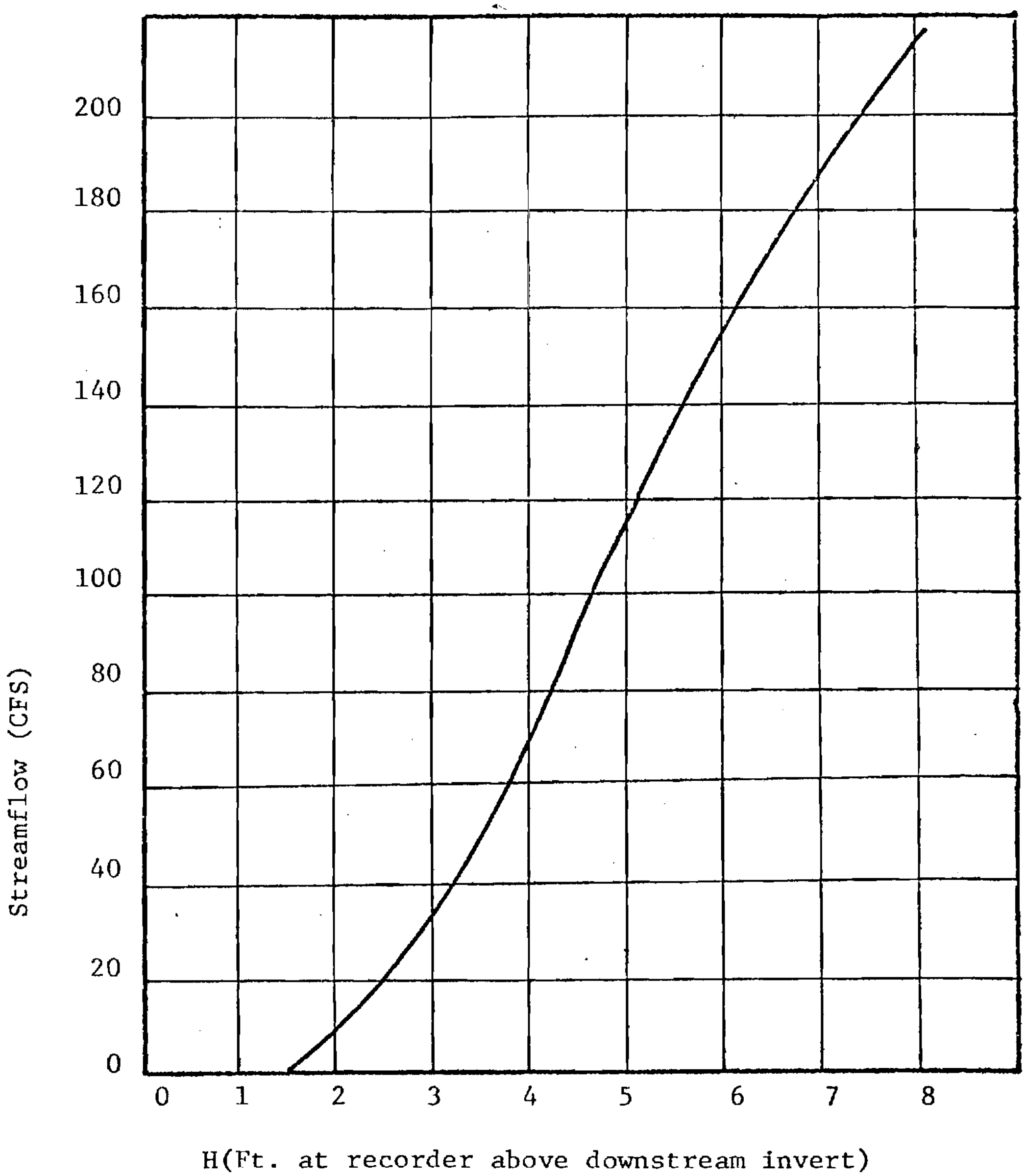


Figure 4. Rating Curve for Womack Creek at Courtleigh Drive

different slopes; the slope of the pipe on the right hand side (looking downstream) is 0.0129 and that on the left side is 0.0152 Masonry headwalls have been constructed at the upstream and downstream end of the culverts, and there are wingwalls at about 30° angles to the headwall. The center line of the creek approached the culvert headwall at a slight angle. Because of this angled approach two upstream crest stage gages were installed, one on either side of the creek. The recorder and stilling well are attached to the upstream headwall, and the inlet pipe extends upstream about 30 feet at which point a 90° bend in the inlet pipe carries the pipe to the center of the stream. Pertinent elevations for the Jackson Creek Gage are as follows (see Figure 2. for layout):

	Item	Elevation (Ft. above downstream invert)
(1)	Downstream invert	
	Pipe on right side	0.23
	Pipe on left side	0.00
(2)	Water surface in downstream channel at low flow	0.1 (Approx.)
(3)	Zero mark - downstream crest gage	1.25
(4)	Upstream invert	
	Pipe on right side	1.0
	Pipe on left side	0.92
(5)	Zero mark - upstream crest gage	
	Gage on right bank	4.58
	Gage on left bank	4.06
(6)	Top of reference rod	2.99
(7)	Invert of inlet to stilling well	1.90
(8)	Street Elevation	12.5 (approx.)

The geometry of this installation indicates that type 1 flow will occur through the range of flow conditions expected at this site. The rating curve for type 1 flow at this site is shown on Figure 5.

Site 3: Snapfinger Creek Gage. This gage is located on a tributary of Snapfinger Creek in the headwaters of the Snapfinger Creek watershed where the tributary passes beneath Abingdon Drive. At this point the creek drains an area of 371 acres. The watershed is bounded on the north by Ponce de Leon Avenue, on the east by Hamrick Drive, on the west by Rays Road, and on the south by Abingdon Drive. About one-third of the watershed is undeveloped, while the other two-thirds of the area is developed with medium density single-family residences. The watershed contains soils of moderate permeability, and approximately 15 per cent of the watershed is covered with impervious surfaces. This watershed was included in the simulation study of Snapfinger Creek watershed (see ref. 4).

The rating curve (see Figure 6) for this gage is based on type 1 flow occurring through the 11'-10" by 7'-7" pipe-arch culvert under Abingdon Drive. There is an 18-inch circular culvert about 0.2 feet above the pipe-arch culvert, but it was not taken into account in the rating curve because water will not normally rise high enough to cause flow through this smaller culvert. The slope of the pipe-arch culvert is 0.0169 and it is 44.5 feet long. The recorder and stilling well are located on the left bank of the creek approximately 32 feet upstream of the culvert. Pertinent elevations for the Snapfinger gage are as follows (see Figure 3 for layout):

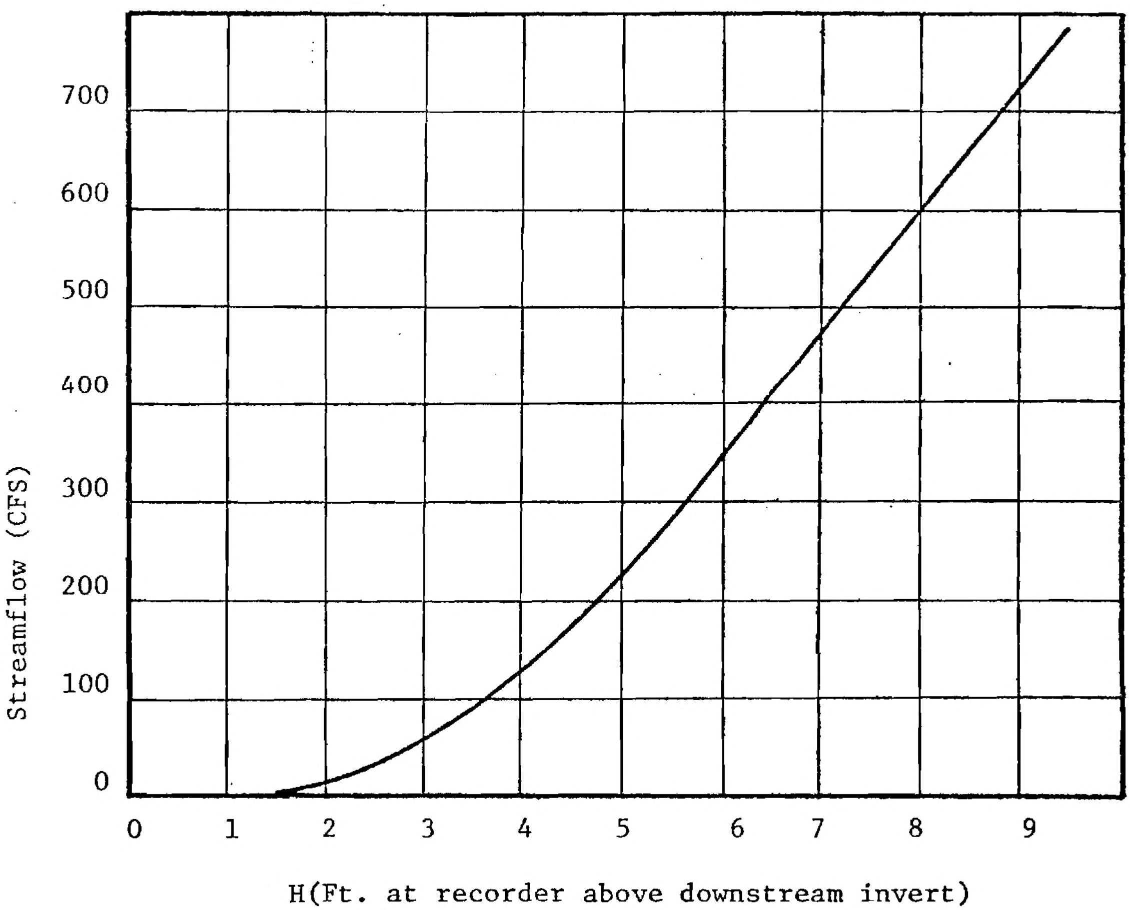


Figure 5. Rating Curve for Jackson Creek at West Hampton Drive

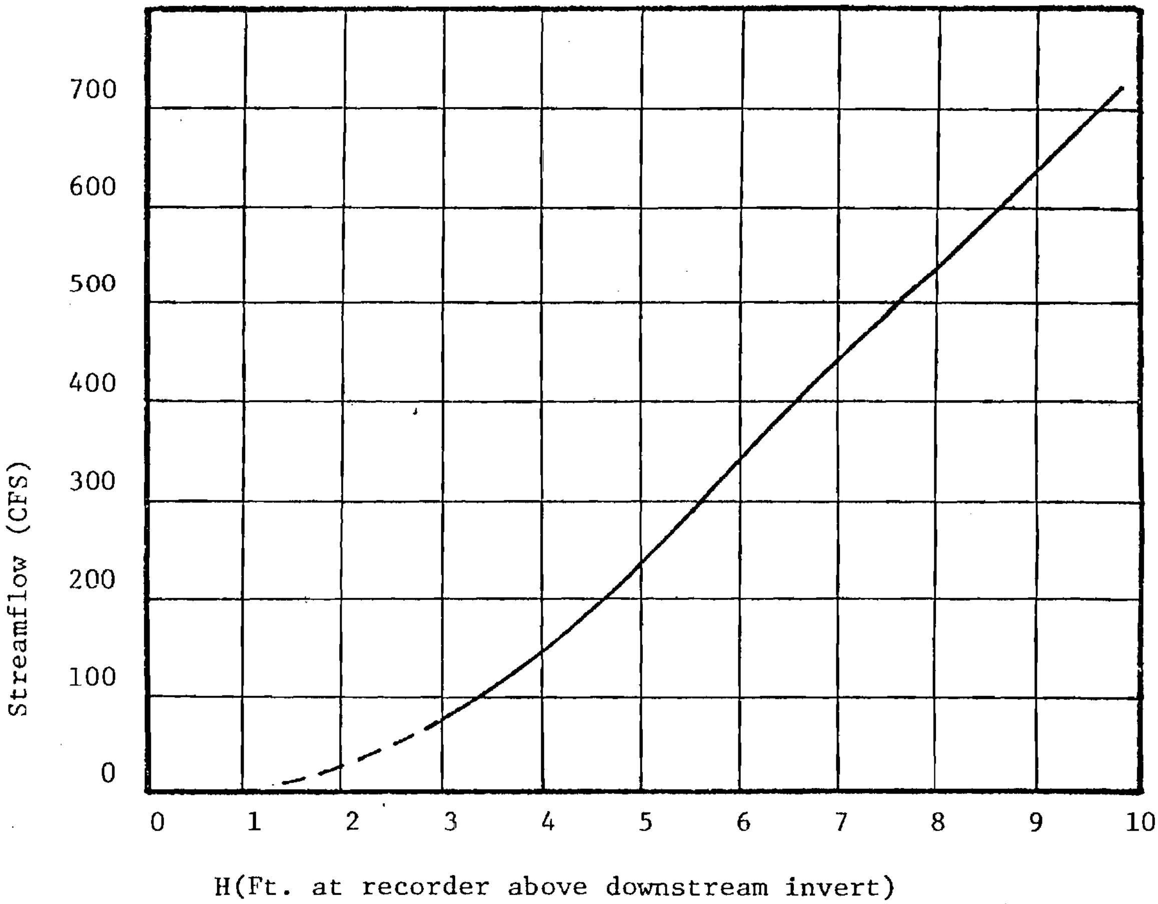


Figure 6. Rating Curve for Snapfinger Creek tributary at Abingdon Drive

Item	Elevation (Ft. above downstream invert)
(1) Downstream culvert invert	0.0
(2) Water surface in downstream channel at low flow	-1.0 (approx.)
(3) Zero mark - downstream crest gage	0.31
(4) Upstream culvert invert	0.75
(5) Zero mark-upstream crest gage	2.53
(6) Top of reference rod	1.03
(7) Invert of inlet to stilling well	0.81
(8) Street Elevation	11.5 (approx.)

Site 4: Wesley Branch Gage: Wesley Branch watershed is located in the eastern part of DeKalb County near the Gwinnett County border. The permeability of the soils in this watershed range from near zero (rock outcrops) to rapid. The drainage area above the gage at Hightower Trail is 201 acres with 191 acres undeveloped. There are a few isolated residences in the watershed, and there is a school at the upstream end of the watershed. The undeveloped portions of the area are forested. About 1 to 2 per cent of the watershed is impervious.

The culvert at which the gage is located is a currugated metal pipe 3 feet in diameter. Originally there was no headwall for this culvert, but a temporary plywood headwall was installed at the time of the gage installation in order to improve entrance conditions for flow in the culvert. The culvert is 36.6 feet long and the slope of the culvert is 0.0172. The rating curve for the culvert is based on type 1 flow and is shown in Figure 7. . The inlet to the gage stilling well is on the left bank approximately 11 feet upstream from the

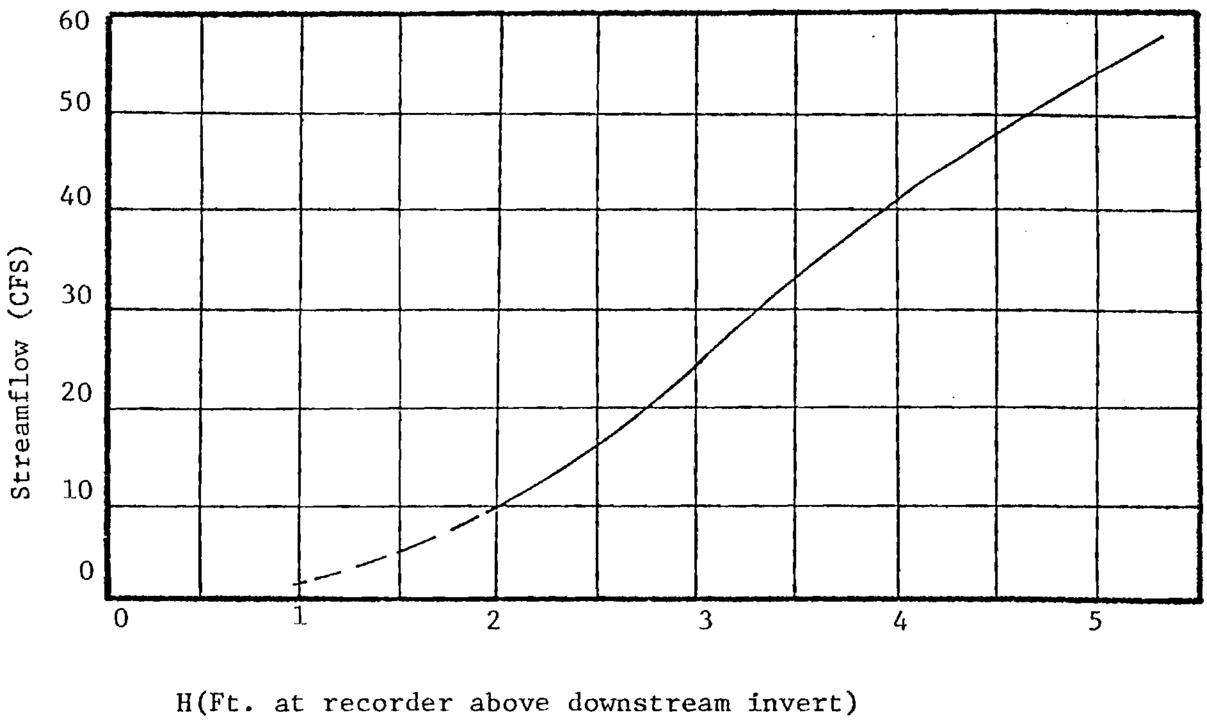


Figure 7. Rating Curve for Wesley Branch at Hightower Trail

culvert. The low point on the road above the culvert is approximately 100 feet east of the culvert and is only about 4.3 feet above the culvert invert. (However, local residents stated that they had never observed the road to be overtopped.) Flows somewhat greater than the mean annual flood should be carried by the culvert without overtopping the road. Rare floods may overtop the road, in which case it will be necessary to extend the rating curve by considering the road to act as a broad crested weir. Important elevations at this site are as follows:

	Item	Elevation (Ft. above downstream invert)
(1)	Downstream culvert invert	0.00
(2)	Water surface in downstream channel at low flow	0.44
(3)	Zero mark-downstream crest gage	0.20
(4)	Upstream culvert invert	0.63
(5)	Zero mark-upstream crest gage	1.81
(6)	Top of reference rod	2.16
(7)	Invert of inlet to stilling well	1.53
(8)	Street Elevation	4.3 (approx.)

Site 5: Honey Creek Gage. The Honey Creek gage is located at the intersection of Honey Creek and Turner Hill Road in southeast DeKalb County. At this point the creek drains a watershed of 804 acres. Most of the watershed has not been urbanized, but a portion of Lithonia does drain into the creek. Overall, it is estimated that 8 per cent of the watershed is impervious, and the soils are

predominantly of moderate permeability. The purpose of locating a gage on this watershed was to observe the change in flow which is expected to occur as the watershed undergoes urbanization. The planned installation of sewage treatment facilities in the area is expected to accelerate development of the watershed.

The culverts at Turner Hill Road consist of three 6' x 10' concrete boxes and the creek approaches the culverts at an angle of about 60°, with the culverts being perpendicular to the roadway. The culverts have wingwalls upstream and downstream of the road, and the wingwalls are at 45° to the face of the culvert. The culverts are 44.5 feet long and the average slope is 0.00562. The stilling well and gage are located on the right bank approximately 50 feet upstream from Turner Hill Road. Analysis of the culvert hydraulics indicates that type 1 flow can be expected. The rating curve for type 1 flow at the Honey Creek Gage is shown on Figure 8, and the elevation of the gage components are as follows:

	Item	Elevation (Ft. above downstream invert)
(1)	Downstream culvert invert (center of left box)	0.00
(2)	Water surface in downstream channel at low flow	-1.0 (approx.)
(3)	Zero mark-downstream crest gage	0.29
(4)	Upstream culvert invert (center of left box)	0.30
(5)	Zero mark-upstream crest gage	0.88
(6)	Top of reference rod	1.78

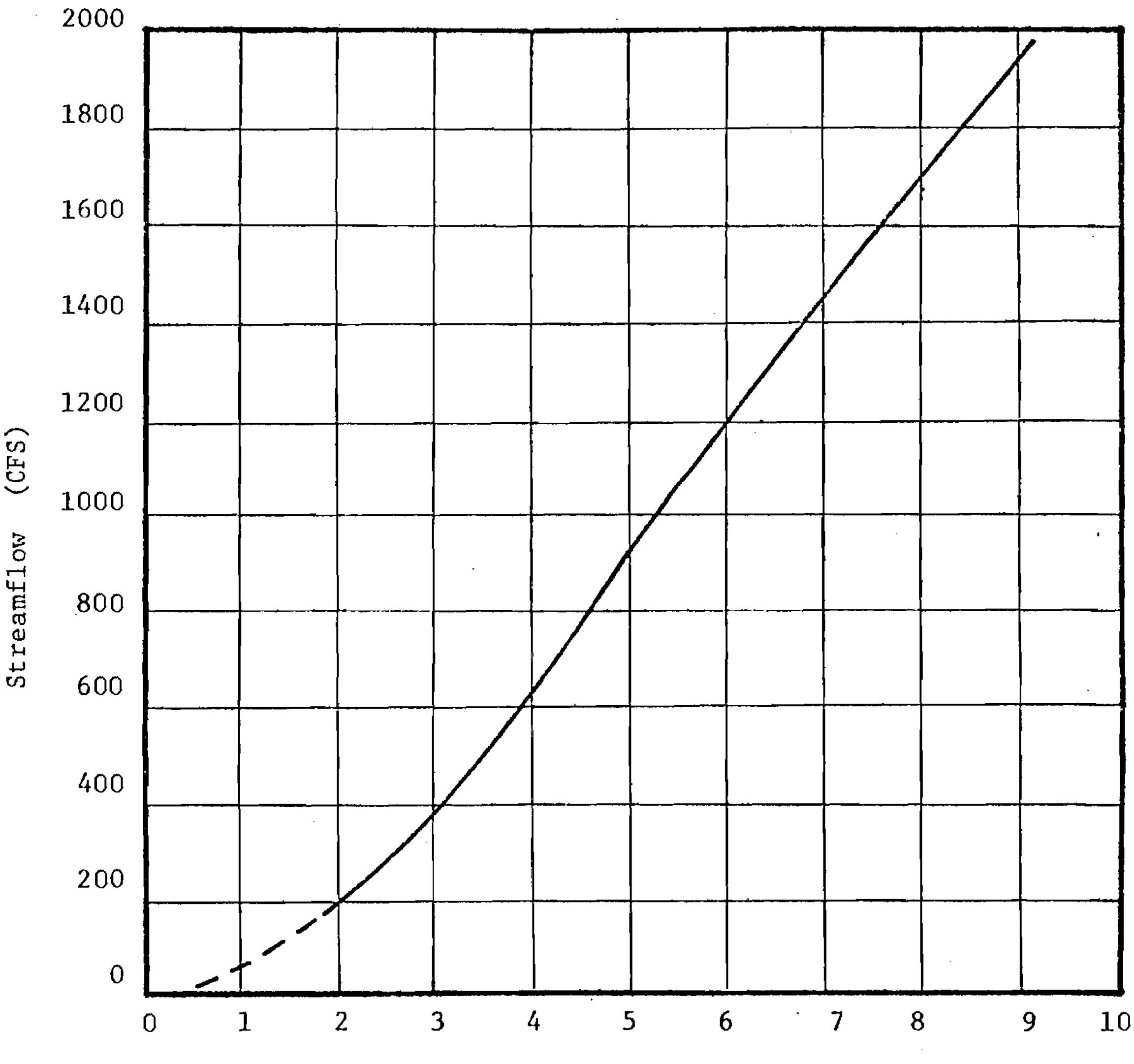


Figure 8. Rating Curve for Honey Creek at Turner Hill Road

(7)	Invert to inlet of stilling well	0.46
(8)	Street Elevation	12 (approx.)

Site 6: Henrico Gage. The headwaters of a small tributary to the South River intersects Henrico Road in southwest DeKalb County about 400 feet south of Interstate 285. The drainage area upstream from the Henrico Road crossing is 112 acres; about 67 acres are undeveloped, 6 acres are in residential development, and 22 acres are in commercial use. The remaining 17 acres correspond to the impervious surface of Interstate 285, which roughly bisects the watershed from northeast to southwest. Overall, it is estimated that 23 per cent of the watershed is impervious. This watershed was selected for study because a significant percentage of the soils in the watershed are of low permeability. Comparison of the runoff from this watershed with that from watersheds with more permeable soils should provide a better estimate of the effect of soils on storm runoff in DeKalb County.

The water level recorder is located about 23 feet upstream from the 6' x 6' concrete box culvert which carries the flow beneath Henrico Road. The culvert is 59.15 feet long and has a slope of 0.00947. There are upstream and downstream wingwalls constructed at 45° angles to the culvert, and the culvert is at an angle of about 75° with the centerline of the road. The condition of the culvert is good and there is no sediment build-up in it. Type 1 flow is expected to occur in this culvert; the rating curve is shown in Figure 9 and the elevations of the gage and culvert components are as follows:

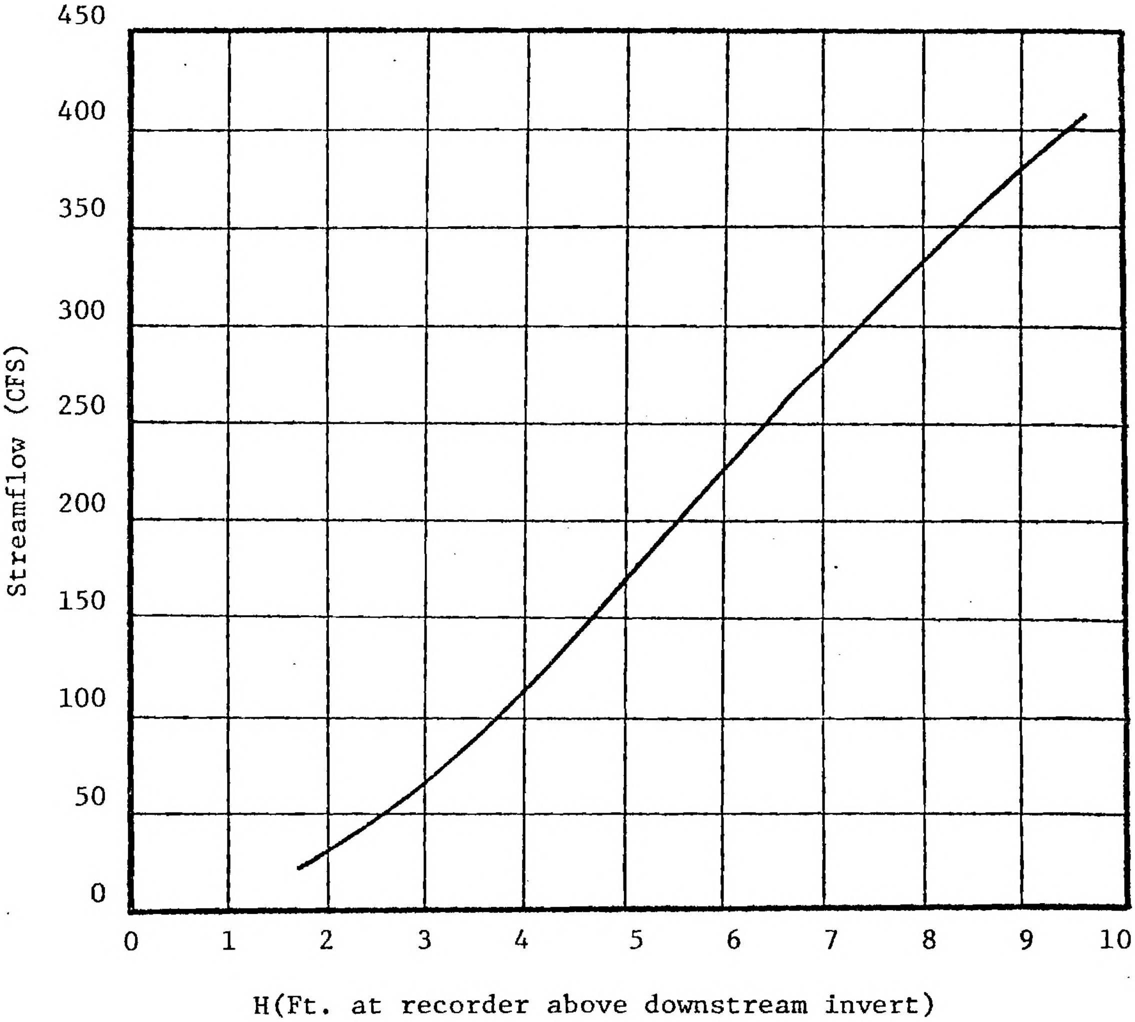


Figure 9. Rating Curve for Gage at Henrico Road

	Item	Elevation (Ft. above downstream invert)
(1)	Downstream culvert invert (center of box)	0.00
(2)	Water surface in downstream channel at low flow	-1.5 (approx.)
(3)	Zero mark-downstream crest gage	0.10
(4)	Upstream Culvert Invert (center of box)	0.56
(5)	Zero mark-upstream crest gage	1.72
(6)	Top of reference rod	1.71
(7)	Invert to inlet of stilling well	0.79
(8)	Street elevation	12

SECTION IV

Summary and Recommendations

Summary of Characteristics of Gaged Watersheds. The watersheds on which the gages were located are distributed throughout the County, and ranged in size from 84 to 804 acres (see Table 9). The land use in these watersheds is predominately in two catagories--Single Family Residences and Undeveloped--and the impervious area ranged from 1-2 per cent for the least developed watershed to a high of 25 for a watershed completely developed in single family residences.

One criterion for site selection was that the soils in the watershed should cover the range of soil permeabilities found in DeKalb County. This was accomplished by including a watersheds with soils of high permeability and a watershed with slowly permeable soils (Henrico Road Creek). Although high and low permeability soils represent only a small portion of the soils in DeKalb County, it is anticipated that the gaging of the runoff from these soil extremes will indicate the effect of soil permeability on runoff and that it can then be determined if such an effect needs to be taken into account when estimating flows from ungaged county streams. The installed gages will provide a continuous record of streamflow in the gaged creeks.

Recommendations. The streamflow rate can be determined by reading the water surface elevation from the recorder chart and finding the associated flow rate on the culvert rating curve provided in Section III of this report. It is emphasized that the rating curves are only estimates based on site geometry and have not been checked against field measurements. The reliability of the rating curves can be determined by comparing measured upstream and downstream water surface elevations, at determined from the crest gages at each site, with the water levels corresponding to type 1 flow, which is the flow type on which

Table 9. Summary of Watershed Characteristics

W/S Name	Area (acres)	Estimated Weighted Permeability (in./hr.)	Impervious Area (%)	Major land Use
Womack Ck.	84	2.19	25	SFR-100%
Jackson Ck.	278	4.22	15	SFR-60% Undev.-40%
Snapfinger Ck.	371	1.61	15	SFR-67% Undev.-40%
Wesley Br.	191	5.04	1-2	Undeveloped
Honey Ck.	804	5.28	8	Undeveloped- (82%)
Henrico Rd.Ck.	112	0.71	23	Undeveloped-60% Commercial-20 % Interstate 285-15 %

these curves are based. (See Section III for the required relationship of headwater and tailwater for different flow types). In addition, it is recommended that discharge measurements based on current meter readings should be made at each site to completely validate the rating curves.

Reliable records from the gages can only be achieved if the culverts remain unobstructed by debris and siltation. Each time a gage is serviced any accumulation of sediment or debris should be noted and removed as soon as possible. It is recommended that these gages not be used for normal or low flow measurements because the accurate interpretation of the rating curves depends on the cross sectional area of the streamflow being constructed by the culvert; at low flows such construction does not occur.

REFERENCES

1. Lumb, A.M., "UROS4: Urban Flood Simulation Model, Part 1. Documentation and Users Manuel," School of Civil Engineering, Georgia Institute of Technology, Atlanta, Ga., March 1975.

2. Bodhaine, G.L., "Measurement of Peak Discharge at Culverts by Indirect Methods," Ch. A3, Techniques of Water-Resources Investigatons of the United States Geological Survey, Government Printing Office, Washington, D.C., 1968.

3. HEC-2 Water Surface Profiles, Users Manuel, Hydrologic Engineering Center, Corps of Engineers, U.S. Army, Davis, CA., 1971.

4. James, L.D. and A.M. Lumb, "UROS4: Urban Flood Simulation Model Part 2. Applications to Selected DeKalb County Watersheds," School of Civil Engineering, Georgia Institute of Technology, Atlanta, GA, May 1975.