AGUA FRIA RIVER

SIDE DRAINAGE ANALYSIS

SECTION - I BUCKEYE ROAD TO 1500 FEET SOUTH OF INTERSTATE 10

SECTION - II INTERSTATE 10 TO MCDOWELL ROAD AND
THOMAS ROAD TO CAMELBACK ROAD

Prepared by:

Simons, Li & Associates

SECTION - I

BUCKEYE ROAD TO 1500 FEET SOUTH OF INTERSTATE 10

AQUA FRIA RIVER FLOOD CONTROL PROJECT

ANALYSIS OF SIDE DRAINAGE REQUIREMENTS

BUCKEYE ROAD TO 1500 FEET SOUTH OF INTERSTATE 10

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I. INTRODUCTION

As part of the channelization/levee design for the Agua Fria River from Buckeye Road to Interstate 10, Simons, Li & Associates, Inc. (SLA) conducted an analysis of the drainage problem created by the proposed levee blocking the natural outlet of local runoff into the river. This analysis consisted of delineating drainage areas that would contribute local runoff to this reach of the river and then selecting hydrologic parameters that could be used to develop hydrographs at various concentration points along the levee alignment. The peak discharge from each of the hydrographs was used to design flap gate culverts through the levee embankment, thus eliminating the potential for ponded water along the landside toe of the levee.

At the request of the Flood Control District of Maricopa County, the local drainage analysis was based on a 10 year storm return interval and existing land uses. For those sub-basins along the west bank of the river that are located within the City of Avondale, the District requested that the drainage analysis be based on a storm return interval compatible with City policy. Contact with the City of Avondale indicated their policy for drainage calculations is a 10 year-2 hour storm. In order to be consistent with this policy and to comply with the District's request, a 10 year-2 hour storm duration was used in generating all the hydrographs presented in this report.

The District also requested that the inlet design headwater used to size the local drainage cluverts not be at a higher elevation than the water surface elevation for the 100 year flood for existing conditions on the Aqua Fria River. This criteria was complied with at all side drainage inlets from Buckeye Road to 1500 feet downstream of the I-10 bridge.

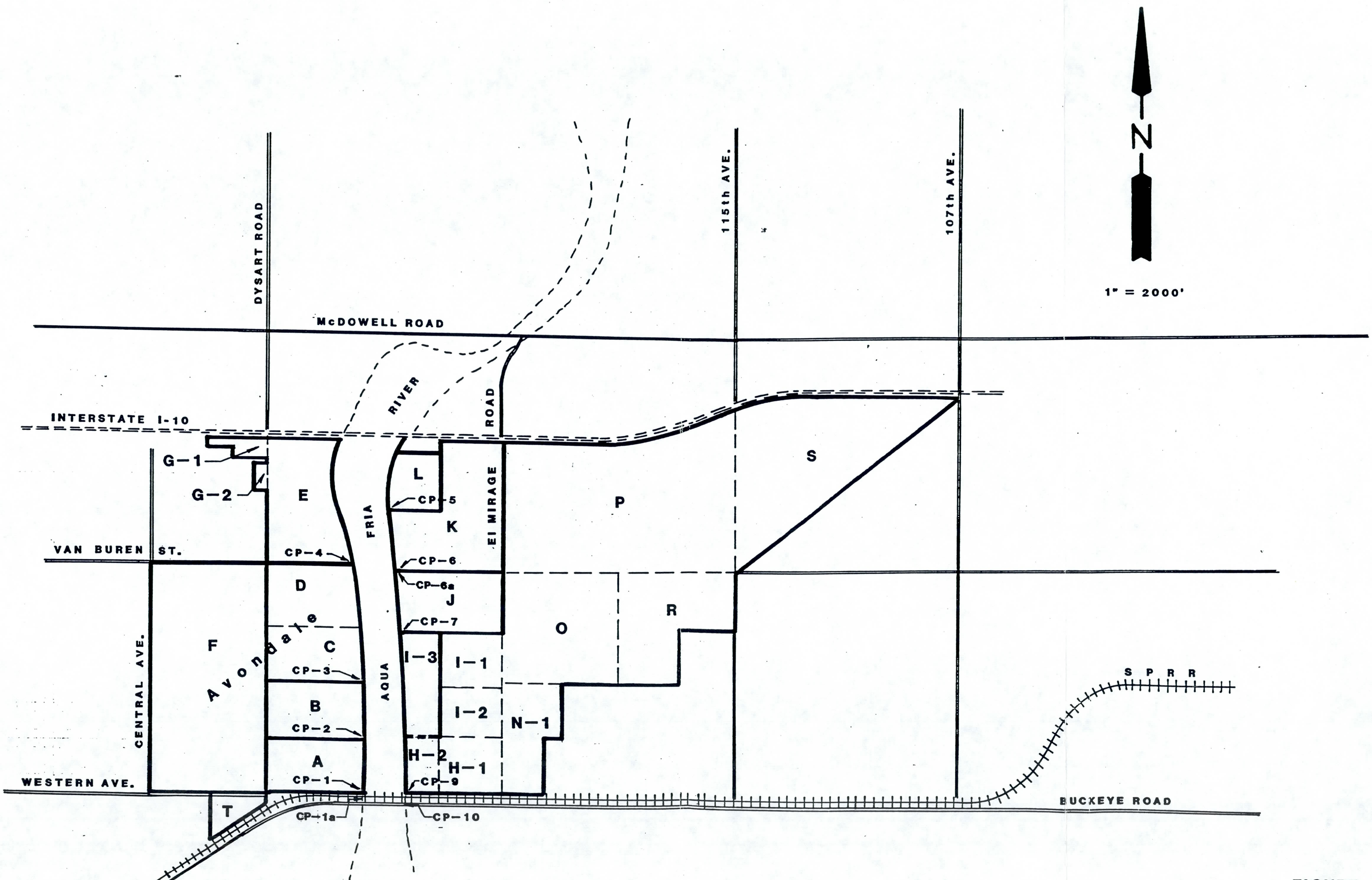
II. WATERSHED DESCRIPTION AND DELINEATION OF DRAINAGE AREAS

A precise delineation of the local drainage area was difficult in the absence of detailed topographic maps. This problem was further complicated by the flatness of the area and the existance of roads, drainage ditches, and numerous irrigation laterals which all tend to divert and intercept overland flow, thus distorting the natural drainage pattern of the area. Accordingly, a significant amount of judgement and some simplifying assumptions were used in the hydrologic analysis.

The watershed boundaries that were used in the analysis were based on the results of several field inspections by SLA staff, a 7.5 minute USGS quadrangle map (1"=2,000', 5' C.I., 1957) and the topographic map (1"=400', 2' C.I., 1981) used for the 1984 conceptual report. Additional input on drainage patterns along the east side of the river were obtained from a local resident who farms this area. The results of this investigation led to the delineation of several sub-basins which will contribute runoff to concentration points at various locations along the levee. These sub-basins and their concentration points are shown on Figure 1. Figure 2 presents a USGS quadrangle map of the drainage area.

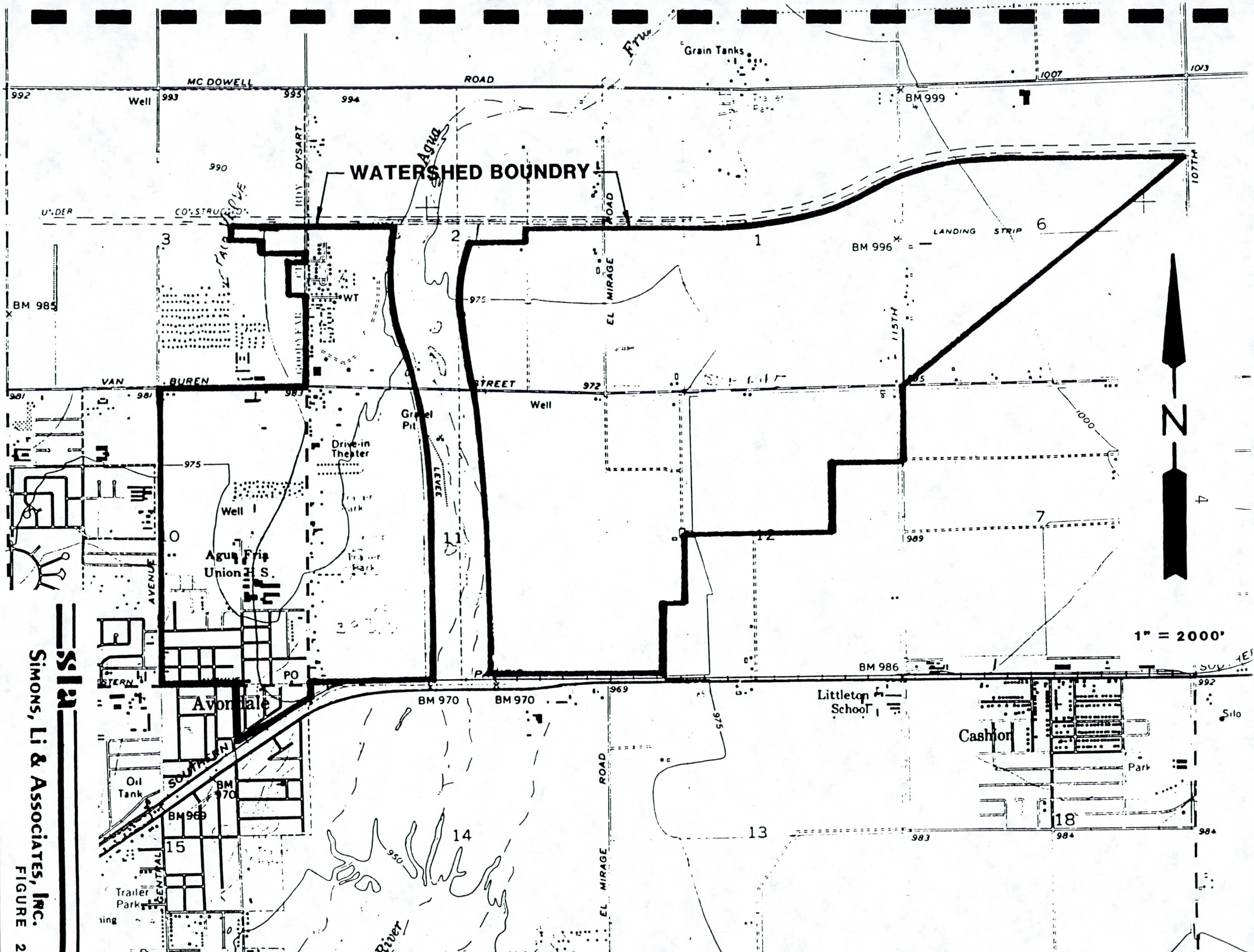
The Soil Conservation Service (SCS) classification for soils within the local drainage area is Hydrologic Group B. This classification is based on soils maps published in the <u>Soil Survey of Maricopa County, Arizona, Central Part</u>, U.S. Department of Agriculture, SCS, September 1977. The soils are primarily loam and sandy loam. Permeability rates are in the range of 0.2 to 2.0 inches per hour.

The sub-basins on the east side of the river consist entirely of irrigated agricultural or open pasture land. Irrigation laterals, roads, drainage ditches and natural topographic features were considered in establishing the boundaries of each sub-basin. In order to simplify the determination of flow paths through sub-basin I-S on the east side of the river, an assumption was made that the irrigation laterals would have no impact on lengthening the flow path used to compute the time of concentration (Tc) for this sub-basin. This is a conservative assumption since it results in a shorter Tc which will cause a higher peak discharge. Similarly, no consideration was given to the impact that plowed furrows in a given field might have in causing water to move



N
1" = 2000'
DYSART ROAD
McDOWELL ROAD
115th AVE.
107th AVE.
INTERSTATE I-10
RIVER
FRIA
AQUA
El MIRAGE
ROAD
VAN BUREN ST.
CENTRAL AVE.
WESTERN AVE.
BUCKEYE ROAD
S P R R
Avondale
G-1
G-2
E
L
K
P
S
D
J
C
F
B
A
T
O
R
I-3
I-1
I-2
N-1
H-2
H-1
CP-1
CP-1a
CP-2
CP-3
CP-4
CP-5
CP-6
CP-6a
CP-7
CP-9
CP-10

FIGURE 1



sla

Simons, Li & Associates, Inc.

FIGURE 2

across it at right angles rather than diagonally. This latter assumption can also be supported by the possibility that any given field may not be plowed for row crops when the design storm occurs.

The contribution of flow from the northeast corner of sub-basin I-S was complicated by the elevated road surface near the intersection of 115th Avenue and Van Buren Street. A weir type analysis was used at this intersection to establish a ratio which was used to pro-rate the number of acres from sub-basin S which would flow westerly and be contained within the project drainage boundaries. The remainder of this area will flow southerly and be lost from the local runoff accumulation behind the levee system. This analysis indicated that 74% or 348.5 acres of sub-basin S would contribute to local runoff along the levee.

The west side of the river consists primarily of residential, commercial, and industrial land use with some pockets of open space. Existing roads and drainage channels were used to establish sub-basin boundaries on this side of the river. Since plans are presently being prepared for industrial development on portions of sub-basins A, B, C, and D, curve numbers representative of these conditions were used in the hydrology analysis rather than curve numbers for the undeveloped condition that currently exists.

III. HYDROLOGIC ANALYSIS

Unit hydrograph theory was used to develop runoff hydrographs at the concentration points of each sub-basin. The SCS dimensionless unit hydrograph was used to develop a unit hydrograph for each sub-basin. The unit hydrographs were then applied to a 2 hour thunderstorm rainfall distribution at unit time intervals computed for each sub-basin. The unit time intervals were computed in accordance with SCS recommendations that they be approximately 0.133 x Tc but no greater than 0.25 x Tp (time to peak). A summary of the hydrologic calculations is shown in Table 1. Plotted hydrographs are included in the Appendix to this report.

The Tc values shown in Table 1 were computed with the Kirpich equation shown at the bottom of the table. Judgement must be used in applying this equation since it was developed from data from small, hilly agricultural areas in Tennessee. Adjustments for different topography and land uses are made by applying a dimensionless "k" factor. Modern Sewer Design, published by the American Iron and Steel Institute (First Edition 1980, page 68), lists adjustment factors ranging from 0.2 for concrete channels to 2.0 for overland flow on grassed surfaces. For the prupose of this study, a "k" value of 1.75 was selected for the flat agricultural fields on the east side of the river. A "k" value of 1.0 to 1.5 was used for the residential, commercial, and industrial sub-basins on the west side. The selected "k" values for each sub-basin are listed in Table 1.

The land slopes used in the Tc calculations for the agricultural fields were based on spot elevations taken from the 1"=400', 1981 topographic map used for the conceptual report. Since these fields have been leveled for agricultural use, the USGS quadrangle map does not truly represent the flat, terraced characteristics of these fields. Spot elevations were only available for those fields immediatley adjacent to the east bank of the river. Slopes in those fields further to the east were assumed to be comparable to those along the river.

Land slopes for sub-basins on the west bank were based on elevation data taken from the USGS 7.5 minute quadrangle map (Tolleson, Arizona, 1957, photorevised 1982).

SCS curve numbers (CN) used to represent the runoff potential of each sub-basin are also listed in Table 1. The selected values were based on

TABLE 1

SUMMARY OF HYDROLOGY CALCULATIONS

Sub-Basin	D.A. ($mi.^2$)	TC (hr.)	CN	2 hr. Thunderstorm[1] Q10 (cfs)	2 hr. Thunderstorm[1] Volume (ac-ft.)	Comments[2]
A	0.10	0.40	80	44	2.19	K=1.0 for Tc
B	0.10	0.39	80	44	2.19	K=1.0 for Tc
C + D	0.20	0.55	81	84	4.80	K=1.0 for Tc
E+(G-1)+(G-2)	0.174	0.75	76	37	2.69	K=1.43 for Tc
F	0.51	1.22	82	128[3]	13.06	K=1.5 for Tc
I-3	0.067	0.96	81	19	1.61	K=1.75 for Tc
J	0.12	1.10	81	31	2.88	K=1.75 for Tc
K	0.19	1.20	81	46	4.56	K=1.75 for Tc
L	0.05	0.67	81	18	1.20	K=1.75 for Tc
I-1, I-2, H-1, H-2, N-1, O, P, R, S	1.91	3.70	81	166 185[4]	45.83	K=1.75 for Tc
T	0.033	0.29	87	30[3]	1.25	K=1.0 for Tc

[1] Point precipitation for a 10 year, 2 hour storm is 1.74". This value is derived from NOAA Atlas 2, Volume VIII, Arizona.

[2] $Tc=K\left(\frac{0.04593\ L^{.77}}{S^{.385}}\right)$, Tc (minutes)
L (feet)
S (%)
K dimensionless factor for land use and cover

[3] These peak discharge values were subsequently attenuated and translated and then added to the runoff from sub-basin A in order to get a design discharge for CP-1. See Table 3 for culvert design discharges.

[4] Including 19 cfs diverted from sub-basin I-3.

guidelines provided by Technical Release No. 55, Urban Hydrology For Small Watersheds, SCS, January 1975, Table 2-2, page 2-5. A copy of this table is presented as Table 2 in this report. This table was used with Hydrologic Soil Group B.

Even though the irrigated agricultural fields would be considered "cultivated land with conservation treatment", they were treated as being "without conservation treatment" for the purpose of selecting a curve number. This provides a higher curve number with more runoff potential and thus gives a factor of safety to the hydrograph calculations for the complex drainage patterns that exist in the fields served by irrigation laterals. Where more than one land use exists in a given sub-basin, a composite curve number was computed on the basis of an area weighted average.

The runoff volumes listed in Table 1 were computed by multiplying the drainage area of each sub-basin by the amount of direct runoff. This calculation was then checked by computing the area under each runoff hydrograph and converting it to an equivalent volume. Agreement between the two computations was obtained for all sub-basins.

The rainfall distribution used for the hydrograph development was based on NOAA recommendations for short duration storms. A plot of this distribution is illustrated in Figure 3. This rainfall distribution has been applied to actual precipitation data in Pima County and was found to produce synthetic hydrographs which correlated well with measured hydrographs. Accordingly, this distribution curve is considered to be more descriptive of the short duration, high intensity thunderstorms occuring in Arizona than is the standard SCS 2 hour distribution. The SCS curve presents a less severe rainfall distribution which results in lower peak discharge values.

The precipitation data used to dimension the rainfall distribution curve was taken from the NOAA Atlas 2, Volume VIII, Arizona. Using this publication, the 10 year-2 hour point precipitation for the study area was 1.74". No areal adjustment factor was applied to this value since the total drainage area being analyzed was less than 10 square miles.

TABLE 2

RUNOFF CURVE NUMBERS FOR SELECTED AGRICULTURAL, SUBURBAN, AND URBAN LAND USE
(Antecendent moisture condition II, and I_a = 0.25)

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land[1]: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover[2]	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious).	81	88	91	93
Residential:[3]				
Average lot size — Average % Impervious[4]				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.[5]	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers[5]	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

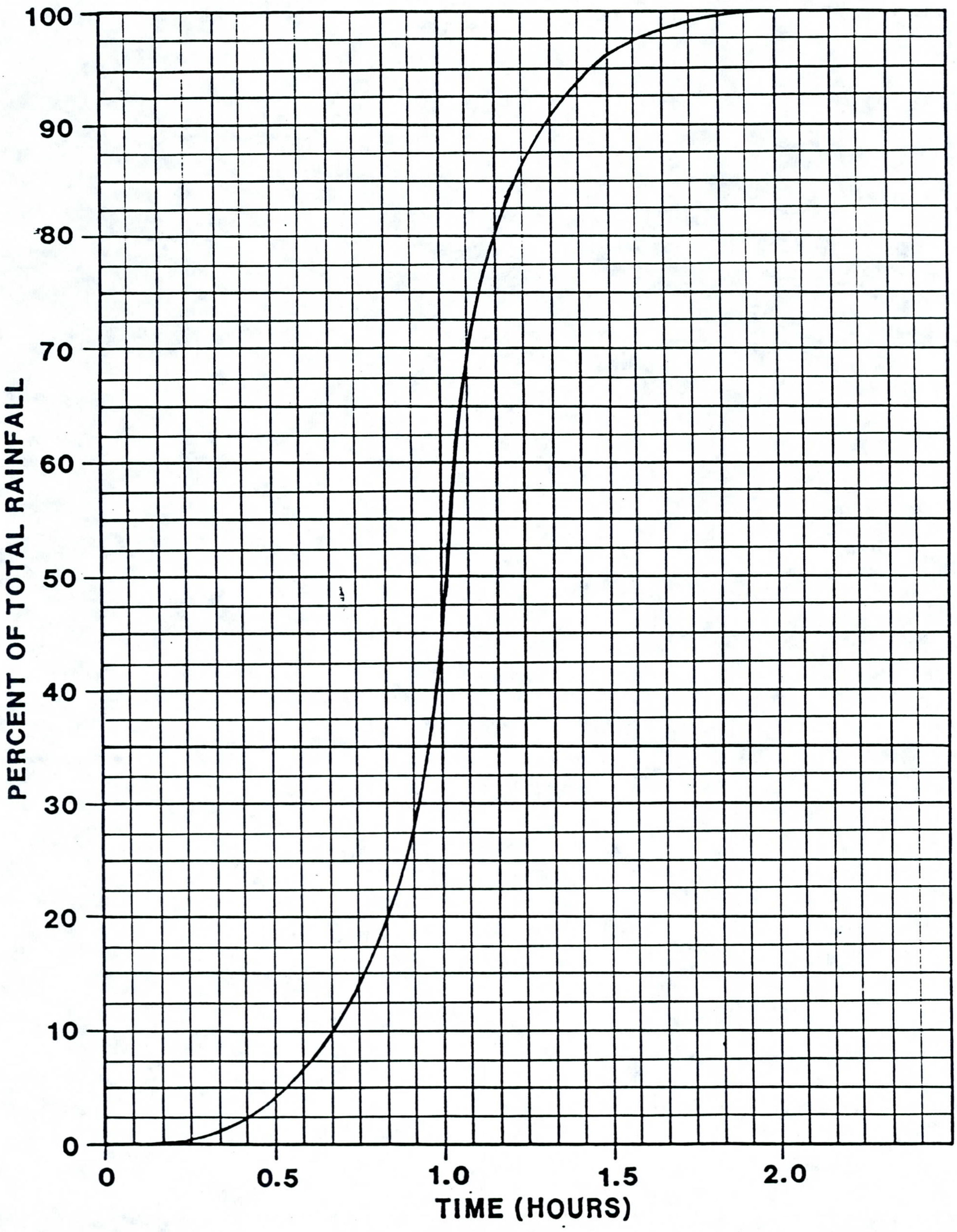
[1] For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

[2] Good cover is protected from grazing and litter and brush cover soil.

[3] Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

[4] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

[5] In some warmer climates of the country a curve number of 95 may be used.



RAINFALL DISTRIBUTION CURVE
USED FOR LOCAL DRAINAGE
HYDROLOGY ANALYSIS

FIGURE 3

IV. LOCATION AND DESIGN OF LOCAL DRAINAGE OUTLETS

Once runoff hydrographs had been developed for each sub-basin, the local drainage outlets through the levee embankment were designed to pass the peak discharge from the hydrographs. Except for sub-basins F and T, all hydrograph concentration points are located along the landside slope of the levee. An outlet culvert was also provided at CP-6a to allow an existing irrigation lateral to have continued access to the river for disposal of excess irrigation water. Another small culvert (18") was provided at CP-1a to alleviate ponding in a small depression located on the west side of the river between the SPRR and Buckeye Road.

A decision was made to route the runoff from sub-basin I-3 to CP-9 rather than providing an additional concentration point at the southern boundary of sub-basin I-3. The southern portion of basin I-3 encompasses a scoured area of the riverbank that will be filled as part of the proposed levee/channelization project. Rather than construct an earth berm along the southern boundary of I-3 to contain runoff within this filled portion of the sub-basin, it was decided to grade a small swale along the landside toe of the levee in order to allow runoff from I-3 to reach CP-9. Consequently, a conservative assumption was made to size the culvert at CP-9 to pass the sum of the peak discharges from sub-basins I-3 and I-S.

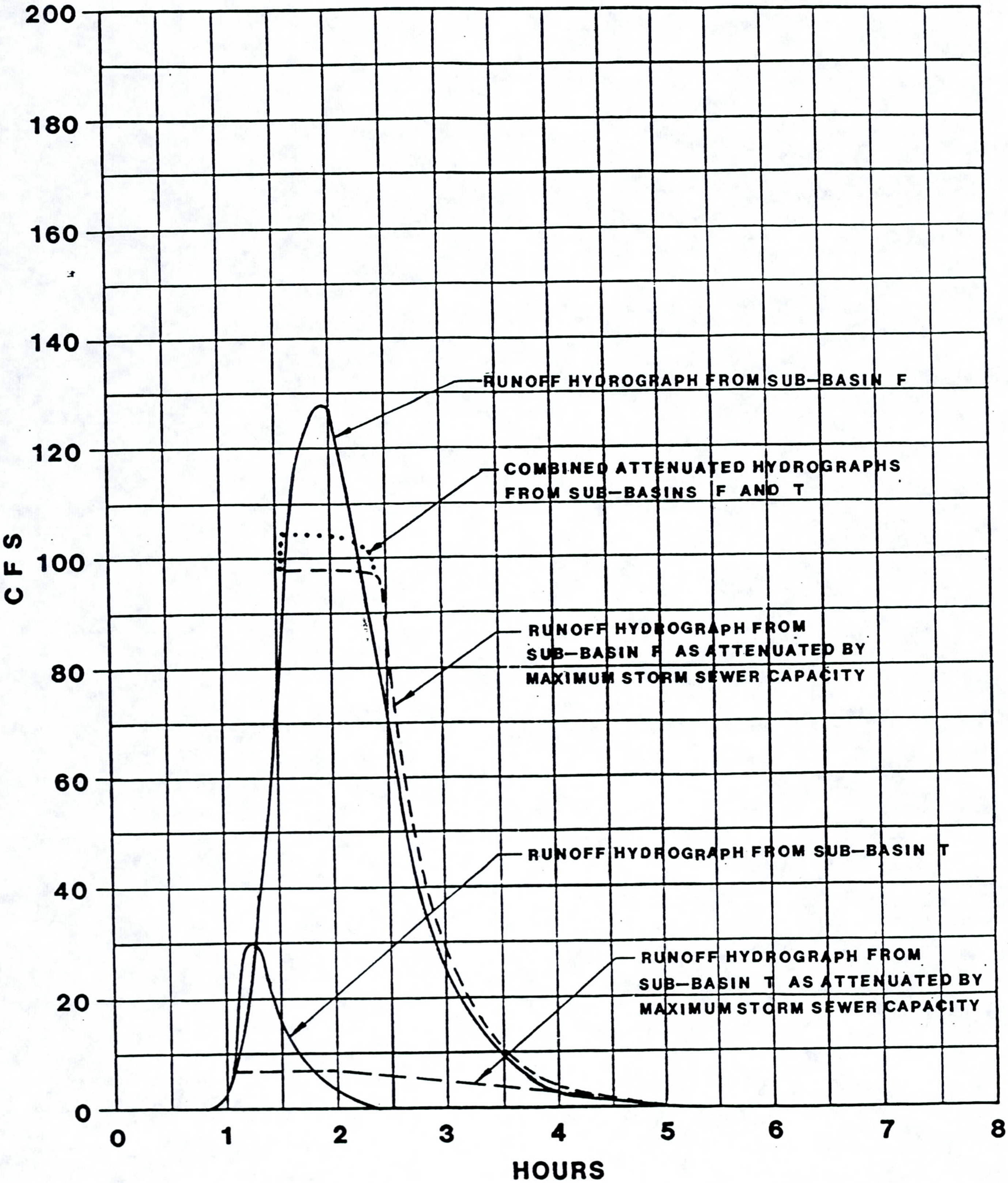
According to information provided by the City of Avondale, sub-basins F and T are drained by an underground storm sewer system. Unfortunately, information on the design capacity of this system was not available. The sewer pipe outlets for these sub-basins were located on the east side of Dysart Road along an extension of Western Avenue. At this location, the discharge from these pipes empties into a drainage ditch which conveys the water to the Aqua Fria River. Since no design information was available on the storm sewer system, an analysis was required to provide an estimate of the amount of runoff from sub-basins F and T that would exit the system and combine with runoff from sub-basin A. To provide this information, a runoff hydrograph was developed individually for both sub-basins F and T (unit hydrograph procedures were used as described previously) using the assumption of overland flow. The capacities of the storm sewer outlets were then estimated under the assumption that they were flowing full with a velocity of approximately 6 fps, which is about mid-way between the 3 fps and 10 fps minimum and maximum limits recommended for storm sewer systems in Water Supply and Waste Disposal (Hardenbergh and Rodie, 1961). Based on these assumptions, the peak sewer capacity for sub-basin F was estimated at 97.5 cfs while that for sub-basin T was 3 cfs.

Using the estimated sewer capacity calculations, the runoff hydrographs from each sub-basin were attenuated at the peak discharge value calculated for the storm sewer systems. These "clipped" hydrographs, which were positioned relative to the beginning of the storm, were then added together to get a single combined, attenuated hydrograph representative of the total discharge from the 2 storm sewer systems (F+T). This hydrograph is representative of what an observer would see if he were positioned at the outlet of the storm sewer system. The hydrograph attenuation/combination process is graphically illustrated in Figure 4.

It should be noted that the areas under each of the individual runoff hydrographs were preserved during the attenuation process by visually extending the recession limb of the attenuated hydrographs to provide an increase in area equivalent to that which was "clipped" from the peak.

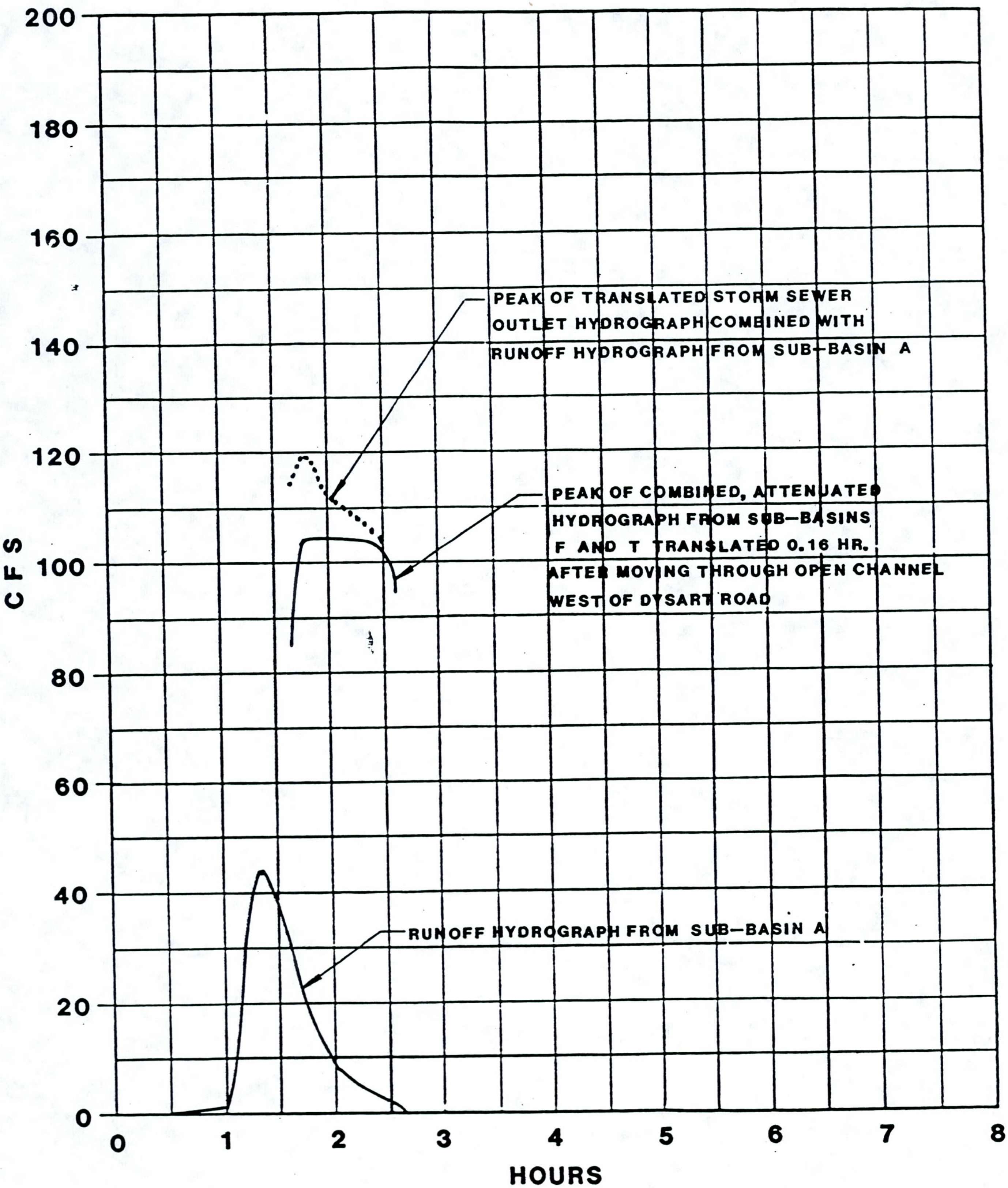
Once the hydrograph from the storm sewer system had been developed, it had to be routed through the open drainage ditch and combined with the runoff hydrograph from sub-basin A. Using Mannings Equation with a peak discharge of 104 cfs, an assumed slope of 0.004 ft./ft., and a cross section considered typical of the drainage ditch, a velocity of 3.38 fps was computed for use in determining the travel time from the sewer outlet to the concentration point of sub-basin A. The storm sewer hydrograph was then translated this amount (0.16 hours) and added to the hydrograph for sub-basin A to determine the peak discharge for use in sizing the drainage outlet at CP-1 (see Figure 5).

The assumption used in selecting the design discharges for sub-basin I-S and the drainage outlet between Buckeye Road and the Southern Pacific RR (SPRR) required an analysis of 3 existing culverts. The reader is referred to Figure 6 for a plan view of the system under discussion. Although sub-basin I-S has an existing southern outlet through a 3½' x 6' box at culvert #1, a backwater condition sufficiently severe to block any appreciable southerly flow at culvert #1 was assumed. Under this assumption, all the runoff from sub-basin I-S (166 cfs) must be brought through the levee at CP-9, located north of the railroad.



ATTENUATION AND COMBINATION OF RUNOFF HYDROGRAPHS FROM SUB-BASINS F AND T

FIGURE 4



TRANSLATION AND COMBINATION OF ATTENUATED HYDROGRAPH FROM SUB-BASINS F AND T WITH HYDROGRAPH FROM SUB-BASIN A

FIGURE 5

SCHEMATIC USED FOR DRAINAGE ANALYSIS ALONG BUCKEYE ROAD, SPRR, AND SUB-BASIN H-1, H-2, N-1

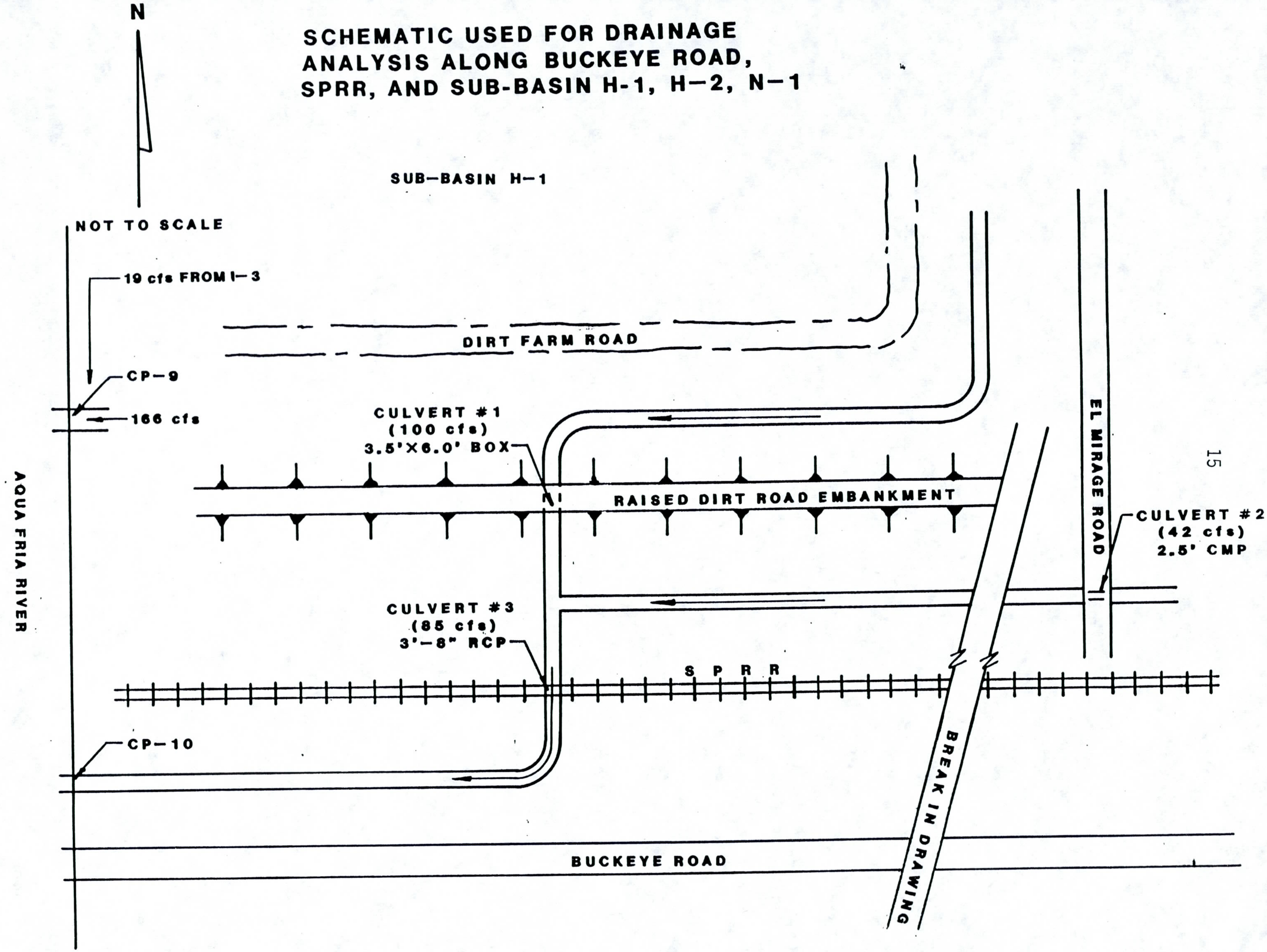


FIGURE 6

Culvert #2 was next analyzed as a control point in estimating the amount of water being conveyed to culvert #3 via the open channel between the SPRR and the raised road embankment to the north. Assuming inlet control at culvert #2, the maximum discharge was estimated to be 42 cfs. This value was then compared to the capacity at culvert #3 which was found to be 85 cfs. Since culvert #3 has a greater capacity then culvert #2, there should be no appreciable reverse flow through culvert #1 which would add to the runoff being handled by the proposed outlet for sub-basin I-S (CP-9).

The outlet (CP-10) for the runoff being discharged through culvert #3 was sized on the assumption that the maximum discharge through culvert #3 was 85 cfs. This assumption was based on an inlet control calculation with a maximum available headwater depth of 5.17 feet. Any additional runoff that may enter the channel between the SPRR and Buckeye Road west of culvert #3 would merely pond in the depression between the railroad and highway. The extra head provided by this additional runoff would serve to increase the discharge through the levee outlet. As a result, the proposed drainage outlet between the railroad and highway was designed for 85 cfs on the assumption of a headwater depth of 4.94 feet. Should water pond to the top of the levee at this point, a headwater depth of 13.36 feet would exist which would produce a culvert discharge of 175 cfs. Beyond this depth the levee crest would be overtopped but not the railroad or highway since they are both higher than the levee crest at this location.

The design of the local drainage culverts at each CP were based on inlet control. The inlets were designed so that the headwater depths required to pass the peak discharge would not pond water higher than the elevation of the 100-year water surface profile for existing conditions on the Aqua Fria River. In order to meet this criteria, drop inlets were required at CP-1, 2, 3, 4, and 7. Depending upon specific conditions at each location, the invert of the culvert outlets were set at 1 to 4 feet above the channel bed of the river.

The culvert capacities were determined using a nomograph for concrete pipe culverts from Hydraulic Engineering Circular No. 5, Bureau of Public Roads. An investigation was made to determine the impact that flap gates have on reducing the capacity of pipe culverts. Research conducted by the Hydraulic Laboratory of Iowa State University indicates that the head loss through flap gates is so small that it has little effect on the discharge capacity of drainage outlets. A small allowance was made for this additional head loss by sizing the culvert capacities for a "projecting groove end" rather than a "groove end with headwall" which is more representative of actual design conditions.

Because of anticipated installation problems resulting from warped levee slopes near the bridges, flap gates were not proposed for the outlets of culverts at CP-1a and CP-10. Reverse flow at these locations will only pond water between the SPRR and Buckeye Road embankments. These ponding areas are very small and should not create any problems at these locations. A summary of the recommended culvert sizes for each CP is shown in Table 3.

In summary, the assumptions used in the hydrologic analysis and drainage outlet design are considered conservative. No consideration was given to the possible detention capacity that many of the bermed, agricultural fields may provide for rainfall runoff. Inlet control was also assumed for the analysis of the three culverts upstream of CP-10 (Figure 6). Again, this is conservative since a tailwater will probably be present downstream of the 3 culverts which would reduce the discharge from that obtained assuming inlet control.

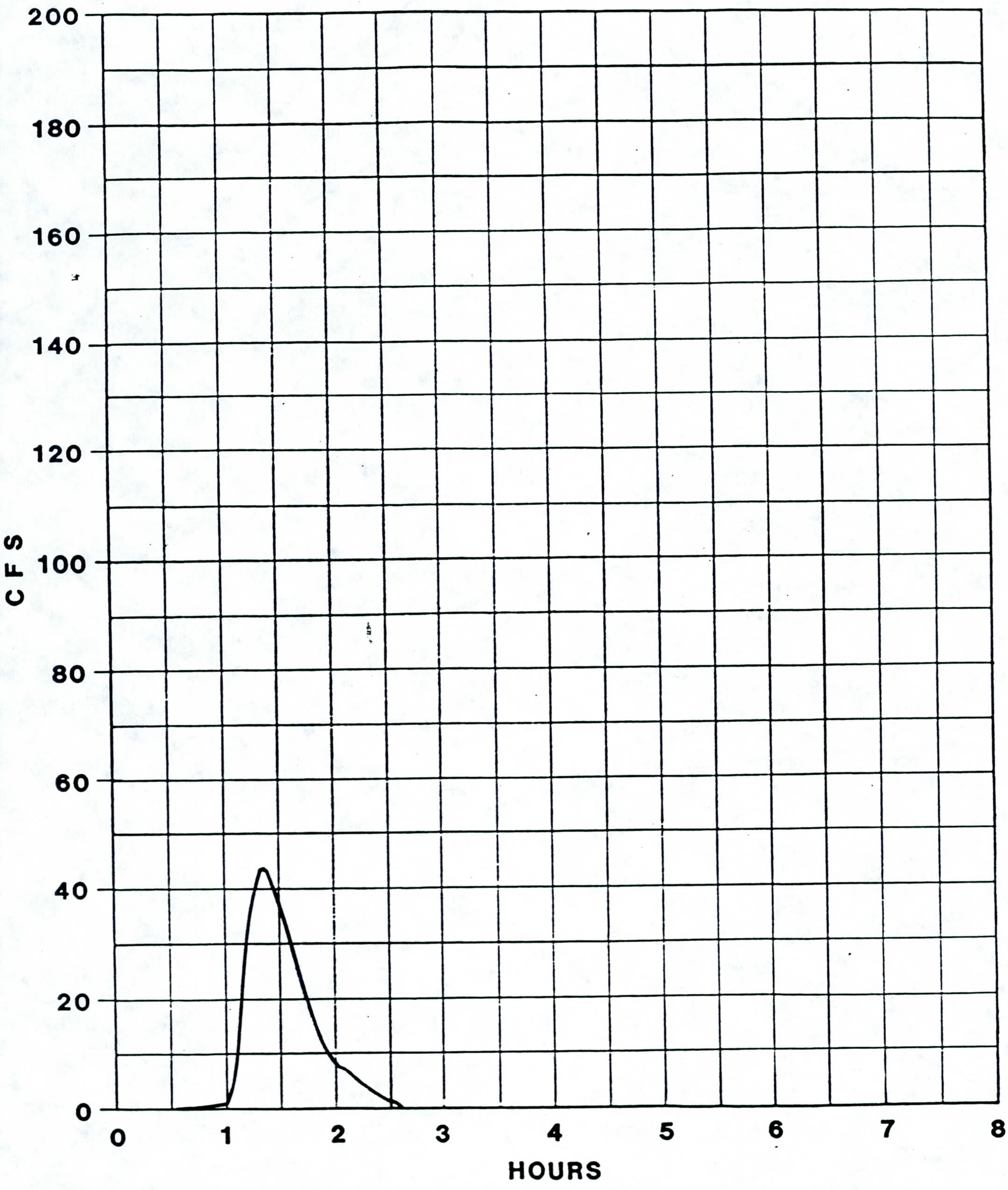
R-1/R634

TABLE 3

SUMMARY OF RECOMMENDED CULVERT SIZES FOR PASSING LOCAL DRAINAGE THROUGH LEVEE

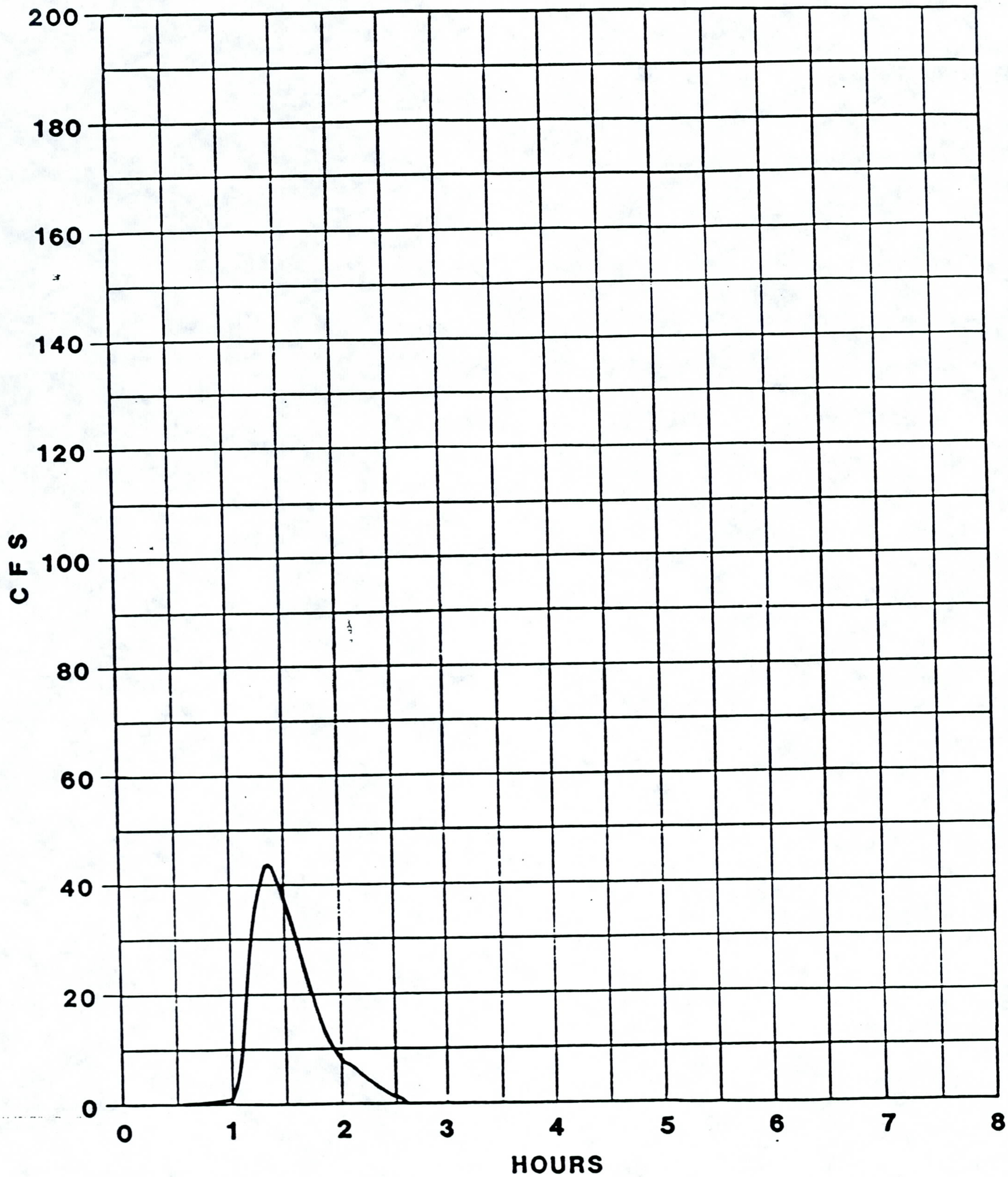
Concentration Point	Contributing Sub-Basins	Design Q (cfs)	Culvert Size (inches)	Inlet Headwater Elevation @ Qp (feet, MSL)	Water Surface of 100-Year Flood On Aqua Fria River, Existing Conditions (feet, MSL)
CP 1	A, F, T	119	48	961.38	962.33
CP 1a	Sm. Depressed Area between SPRR & Buck-eye Road	15	18	959.49	961.48
CP 2	B	44	36	964.33	965.24
CP 3	C, D	84	42	967.90	970.48
CP 4	E, G-1, G-2	37	36	972.94	973.83
CP 5	L	18	24	974.60	975.61
CP 6	K	46	36	973.42	973.75
CP 6a	Irrigation Lateral	18.5	24	970.18	973.70
CP 7	J	31	36	967.88	971.83
CP 9	H-1, H-2, I-1, I-2, I-3 N-1, O, P, R, S	185	(3)36	960.10	962.33
CP 10	Drainage Ditch Along SPRR From 115th Ave. to Agua Fria River	85	42	960.75	961.48

APPENDIX



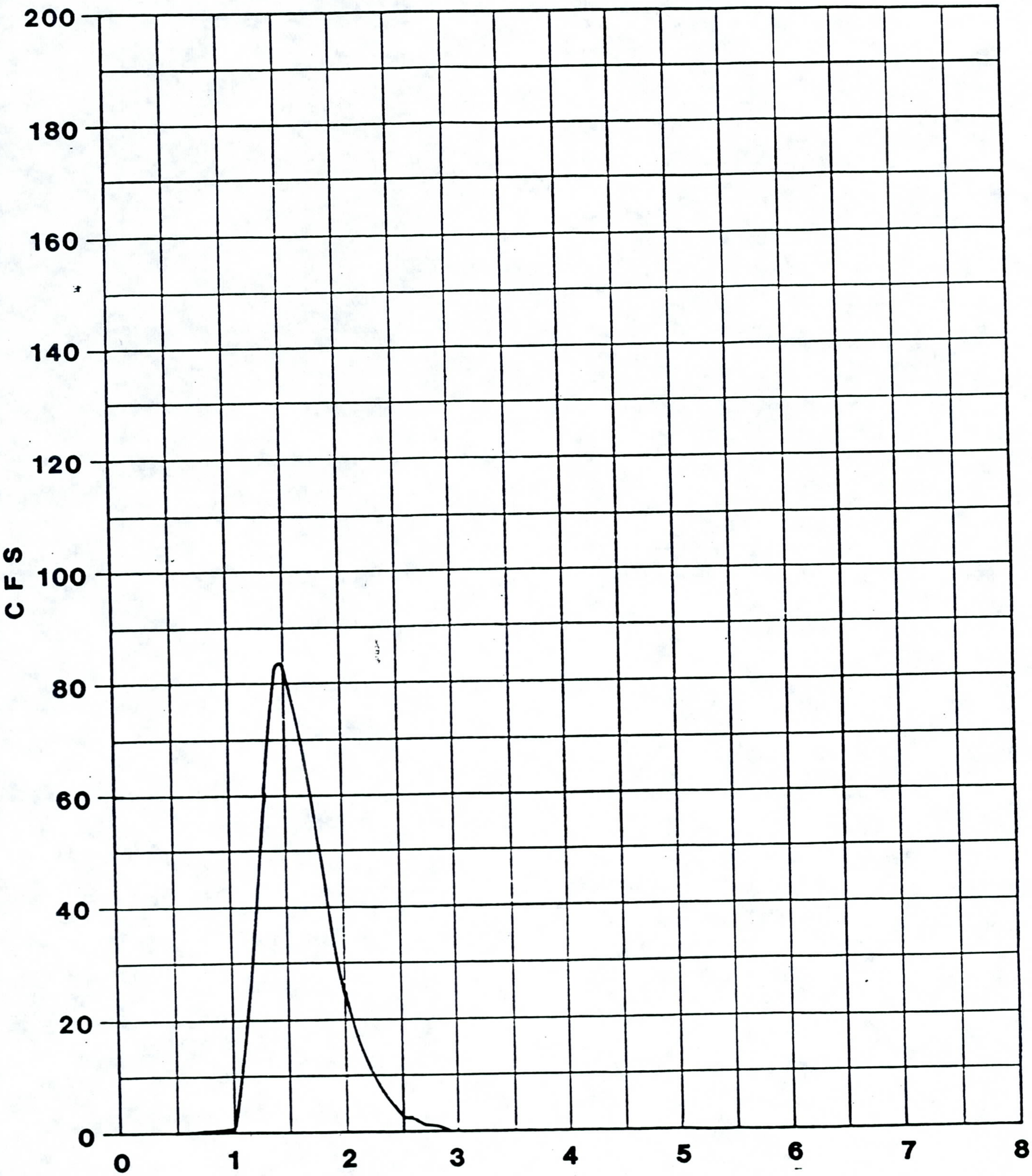
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN A

FIGURE A—1



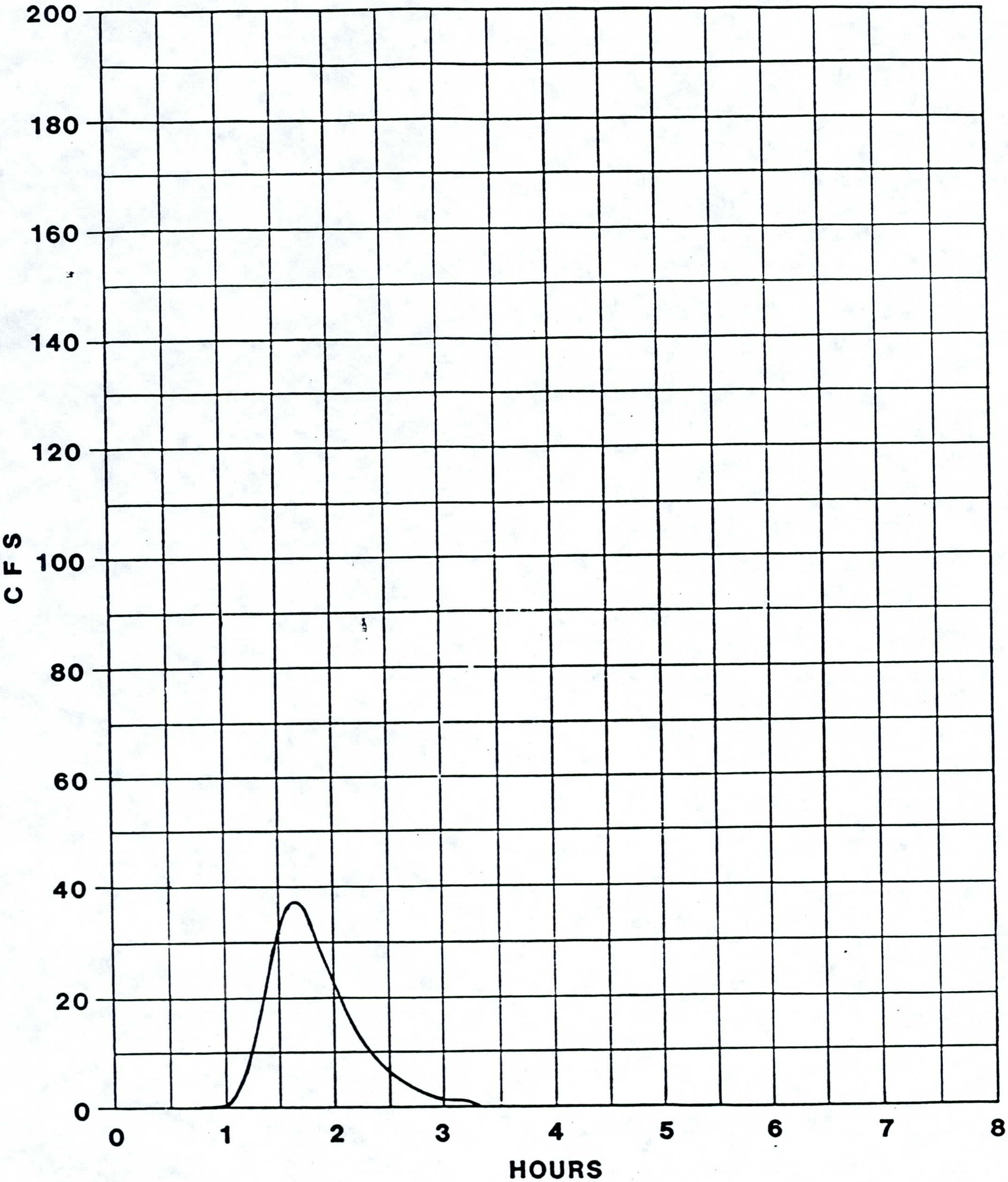
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN B

FIGURE A—2



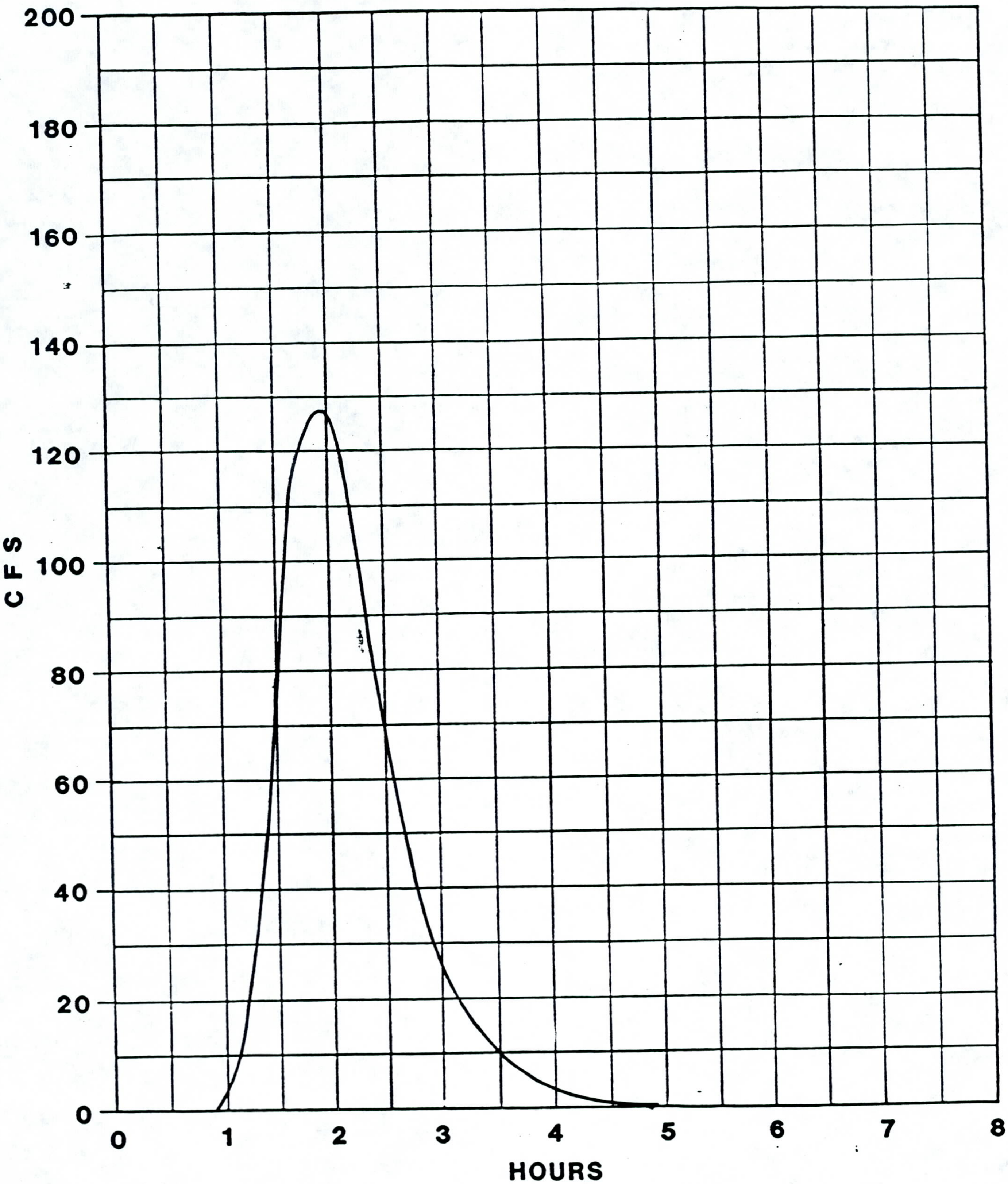
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN C, D

FIGURE A—3



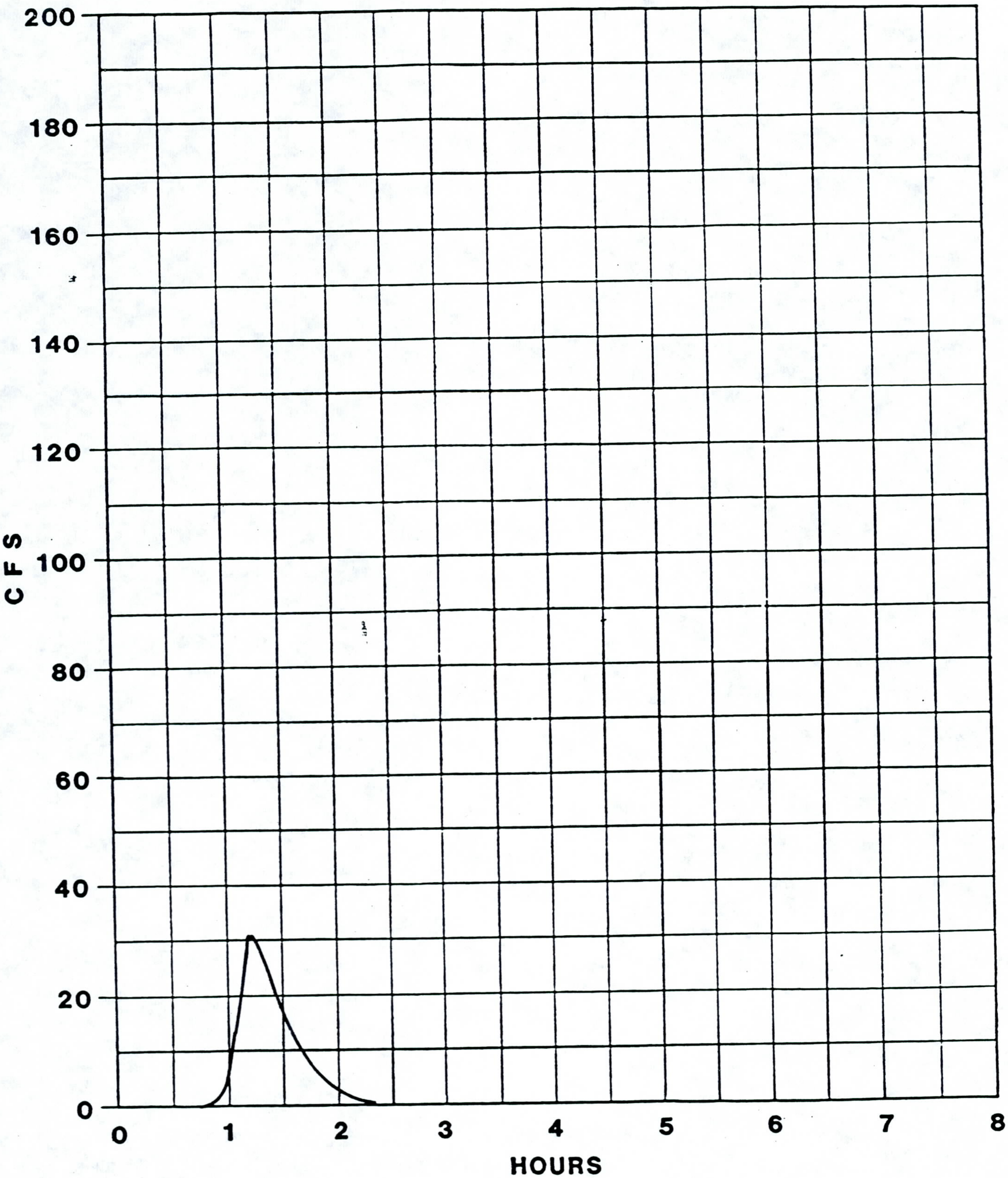
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB-BASIN E, G-1, G-2

FIGURE A-4



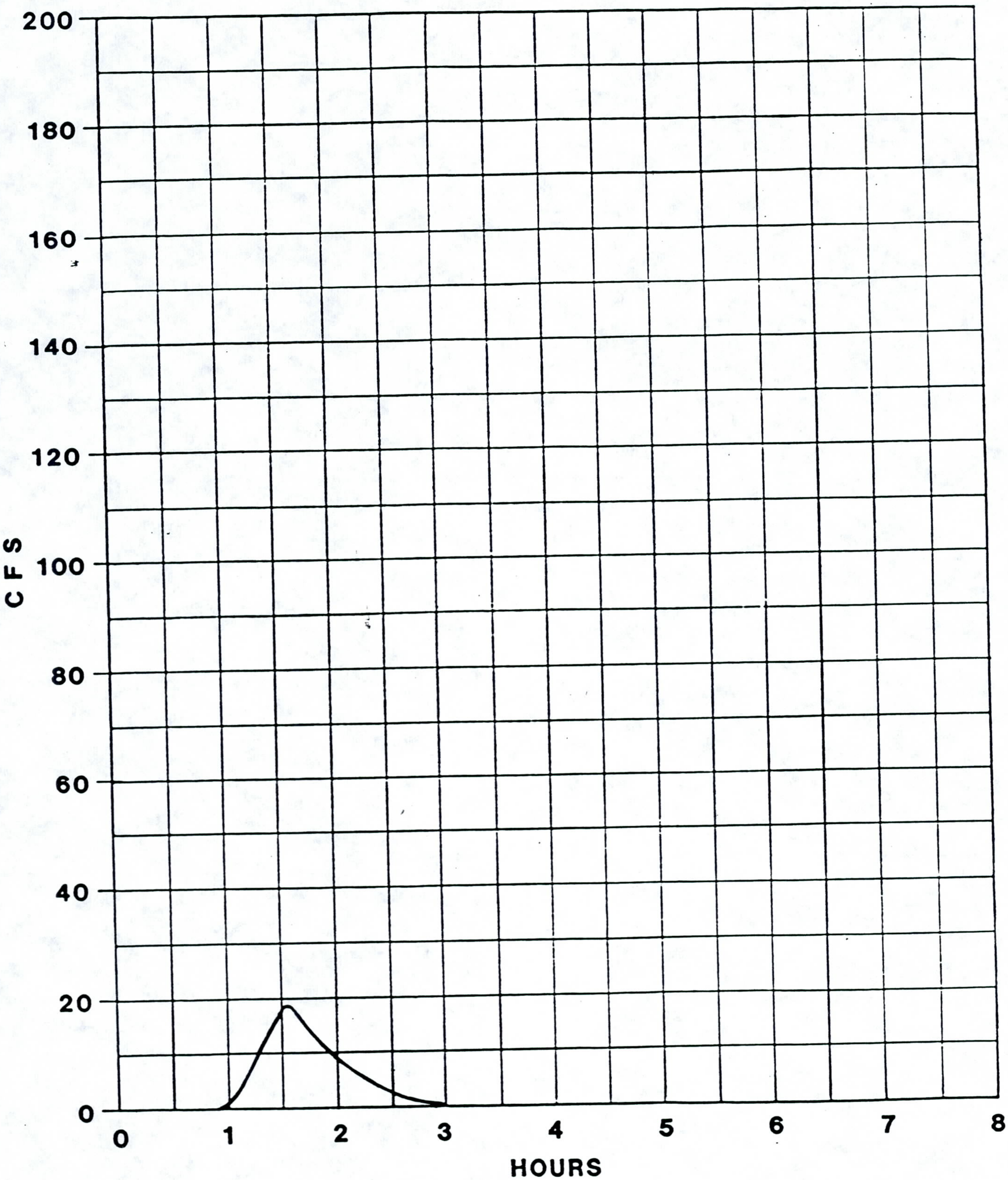
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN F

FIGURE A—5



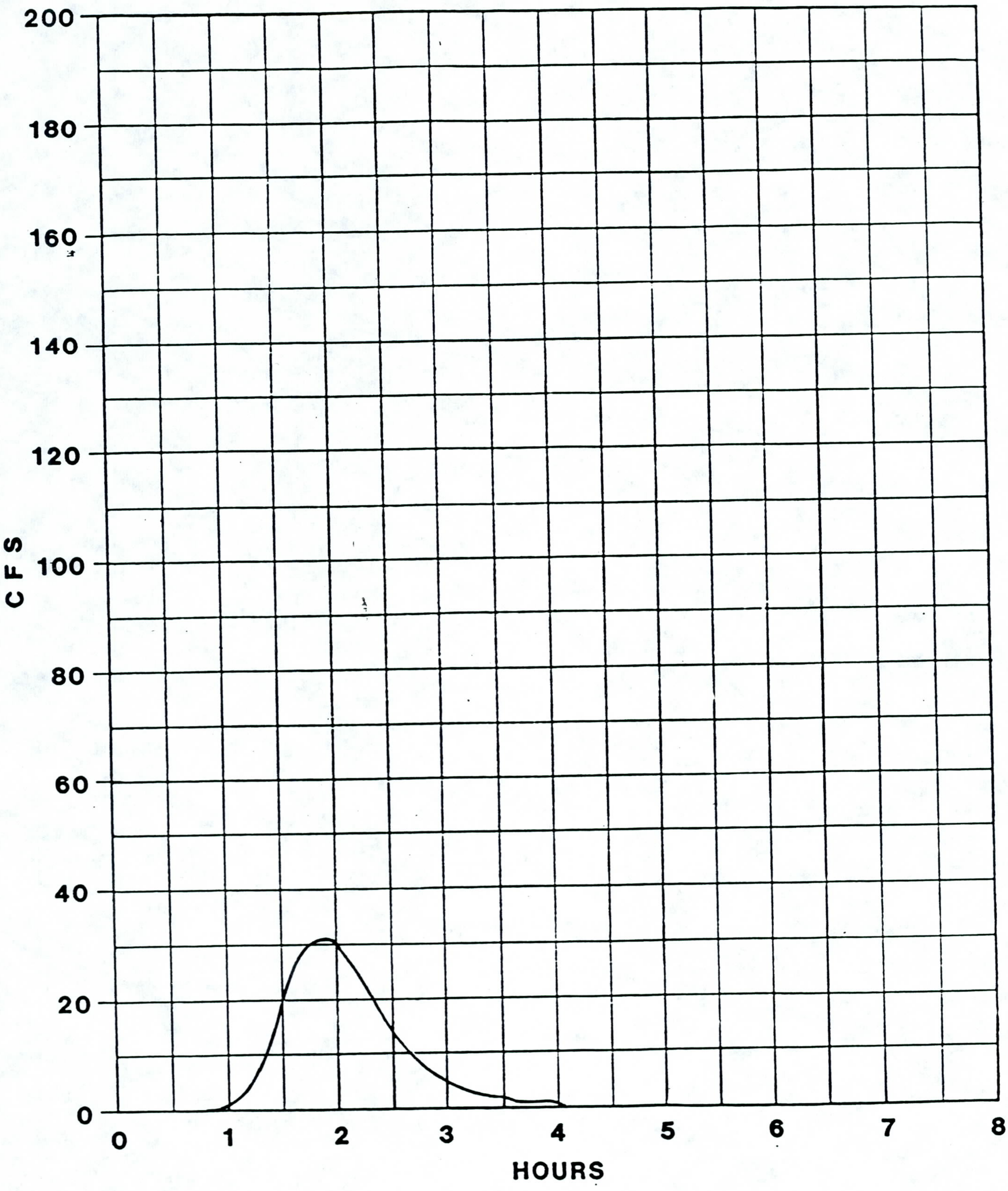
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN T

FIGURE A—6



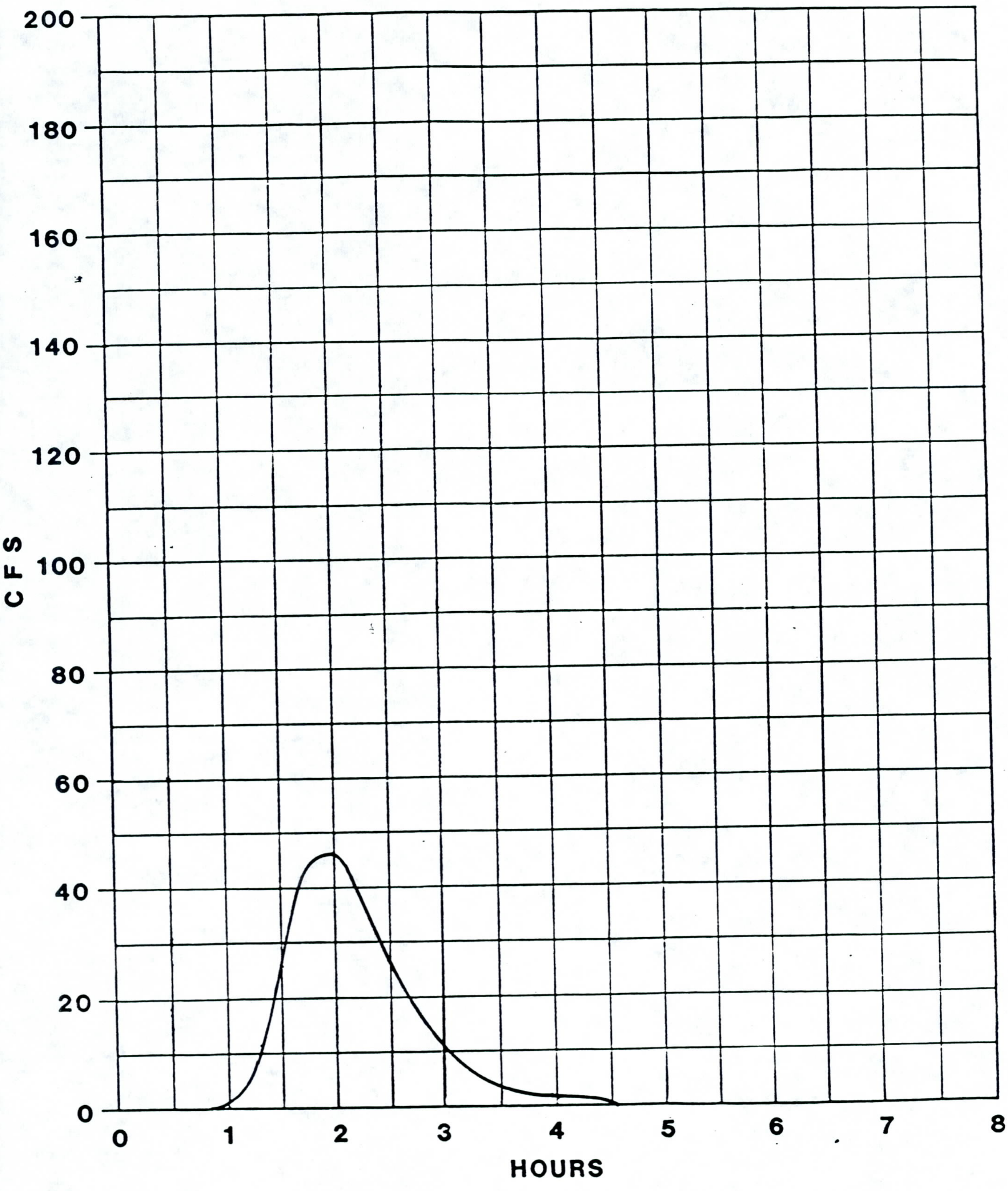
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN I—3

FIGURE A—7



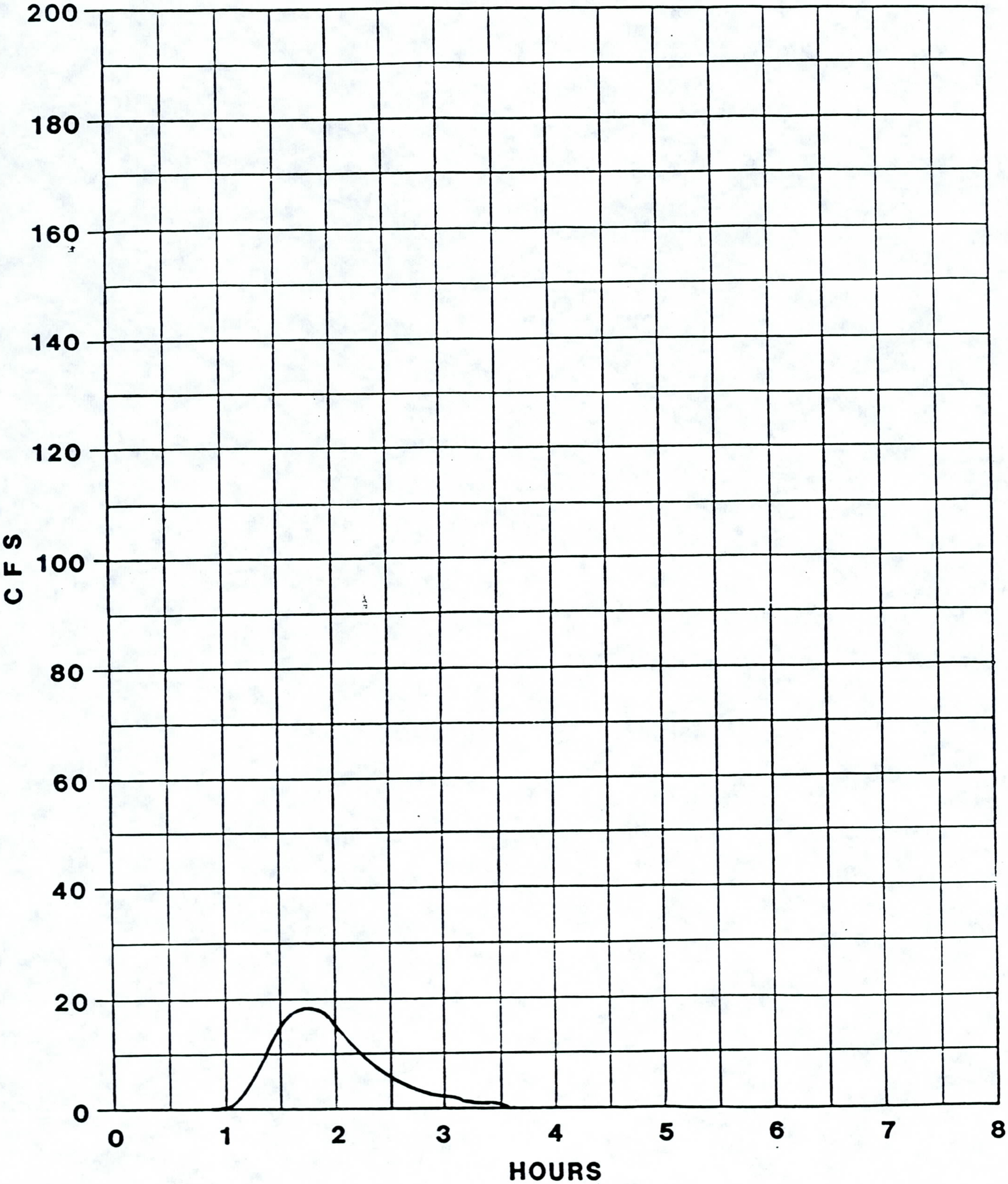
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN J

FIGURE A—8



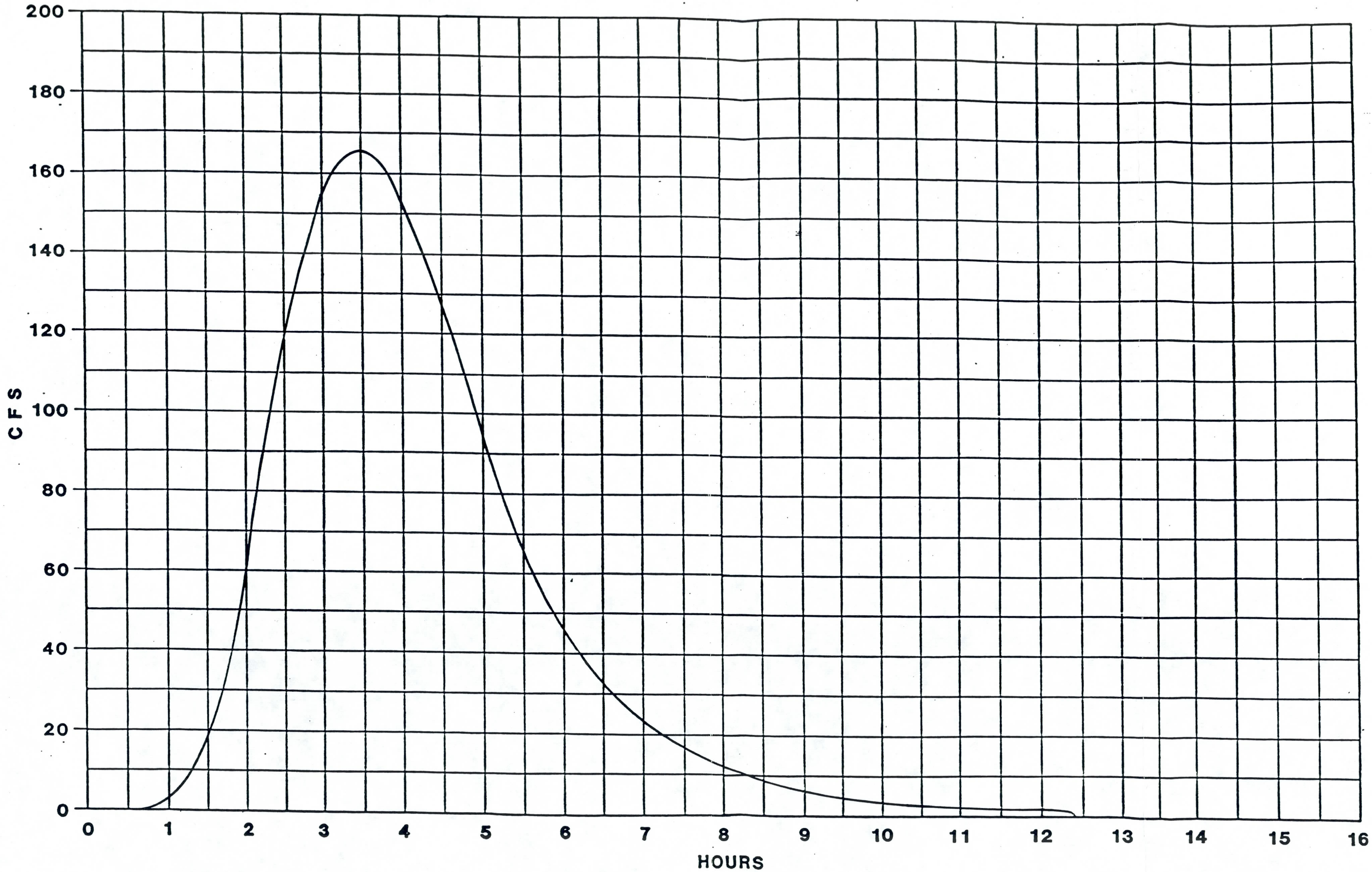
RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN K

FIGURE A—9



RUNOFF HYDROGRAPH
10 YR. — 2 HR. THUNDERSTORM
SUB—BASIN L

FIGURE A—10



RUNOFF HYDROGRAPH
10 YR. – 2 HR. THUNDERSTORM
SUB–BASIN I–1, I–2,
H–1, H–2, N–1, O, P, R, S

FIGURE A–11

SECTION - II

INTERSTATE 10 TO MCDOWELL ROAD AND THOMAS ROAD TO CAMELBACK ROAD

sla

SIMONS, LI & ASSOCIATES, INC.

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James K. Larrington, P.E.
Bayard T. Stevenson III, P.E.

November 7, 1984

Mr. Richard Perreault
Flood Control District of Maricopa County
3335 W. Durango
Phoenix, Arizona 85009

RE: Agua Fria River Channelization Side Drainage Analysis

Dear Mr. Perreault:

Enclosed herewith are two copies of our analysis of requirements for providing drainage through the proposed levee system being designed as part of the channelization of the Agua Fria River between Interstate 10 and McDowell Road, and Thomas Road to Camelback Road. Our analysis looked at both the 100-year peak discharges anticipated from the present, undeveloped watersheds and the minimum provisions required to insure a backwater/ponding situation from these watersheds which would be no worse than the flood elevations which could have been anticipated from the standard project flood on the unimproved system through the aforementioned reaches.

Please review this material in order that a decision may be made regarding the design parameter to be utilized in providing for drainage through the levee system.

Should you require additional information or have any questions regarding this subject, please contact either myself or Michael Zeller.

FLOOD CONTROL DISTRICT
RECEIVED

NOV 08 '84

	CH ENG		HYDRO
	ASST		LMgt
	ADMIN		SUSP
	C & O	3	FILE
1	[illegible]		DESTROY
	FINANCE		
REMARKS			

LH3.1

Very truly yours,

SIMONS, LI & ASSOCIATES, INC.

John B. Lynch

John B. Lynch, P.E.
Vice President for
Design and Construction Services

JBL:ec

Enclosure

R24/R576CL

FCD FILE: LAF 3.1

SIDE DRAINAGE RECOMMENDATIONS FOR
THE AGUA FRIA CHANNELIZATION

The following summary report discusses the results of the hydrologic investigation performed to determine the design of overbank drainage measures recommended for incorporation into the Agua Fria River Channelization Project between Camelback Road and Thomas Road, and between McDowell Road and I-10.

Figure 1, on page 5, shows the delineation of the drainage areas and concentration points of overbank flows entering the Agua Fria within the study sections (see Figure 2, on page 6, for complete delineation of areas concentrating at Points 3 and 7). Culvert installations are recommended at each concentration point shown except number 7. Table 1, on page 7, lists the 100-year peak (i.e., the design flow rate) and recommended culvert type and size for each location.

The rational method was used, as shown on the attached calculation sheets, to determine the peak flow rates in Table 1. U.S.G.S. (7.5 min.) quad sheets were used to determine the drainage areas shown on Figure 1. No attempt was made to account for irrigation water in determining the peak-flow rates. The following conditions were assumed in determining the drainage areas, concentration points, and design flow rates:

1. The existing land usage (i.e., predominantly agricultural) was assumed in determining the hydrologic parameters used in the rational method per the request of the Maricopa County Flood Control District.

2. The western boundary of the drainage areas concentrating at Points 1 and 2 reflect the existence of a large drainage ditch which acts to drain upstream runoff to the south into the RID Canal.

3. In delineating the drainage areas concentrating at Points 3 and 7, it was assumed that the Grand Canal

acts as a drainage control feature diverting upstream runoff to the west along the Bethany Home Road alignment. It is uncertain, however, as to what extent the Grand Canal will act in this capacity. If overtopping of this canal occurs, it will result in significantly higher peak flows at Points 3 and 7 than those shown in Table 1.

4. With the exception of Items 2 and 3 above, it was assumed that irrigation canals and ditches within the various drainage areas, including the RID Canal where it crosses the area draining to Point 7, do not act to divert runoff during the 100-year event.

5. The drainage area concentrating at Point 3 assumes the installation of the floodwall proposed for construction along Indian School Road adjacent to the Agua Fria River.

6. Concentration Points 2 and 3 assume that the proposed RID siphon design will create a levee condition where the elevated flume presently exists.

7. The culvert installation at Point 4 assumes the excavation of a channel extending approximately 500 feet to the east to intercept an existing drainageway.

8. The drainage areas concentrating at Point 6 and 7 assume the installation of the culverts proposed for Points 4 and 5, respectively.

Additionally, it should be noted that there is no culvert recommendation for Concentration Point 7. Current plans for the proposed McDowell Road Bridge over the Agua Fria River, being prepared by Dibble & Associates, call for a culvert installation along the east approach embankment to drain the flows concentrating at Point 7. These flows would then discharge into the planned siltation basin at the outlet of the I-10 collector channel. Due to the uncertain effects of such an additional discharge on the operation of the siltation basin, along with the potentially high cost of integrating the McDowell Road culvert installation into the siltation basin design, it is recommended that the culvert installation be relocated to drain directly into the Agua Fria River on the north side of the approach embankment. It is also recommended that this culvert be designed to accommodate the discharge shown in Table 1.

The following conditions were assumed in arriving at the recommendations shown in Table 1:

1. It was assumed, for design purposes, that all side-drainage culvert installations would operate under inlet control.

2. Flap gates are only available on a stock-item basis for circular culverts of 48-inch diameter and smaller. It was assumed that flap gates are required at all installations; therefore, no circular culverts larger than 48 inches are being recommended.

3. It was assumed, for purposes of culvert sizing, that uniformity in dimensions would result in reduced costs associated with specialized fabrication of flap gates for box culvert installations that would be needed to accommodate a 100-year flood.

4. It was assumed that inundation of land due to ponding of water at culvert inlets would be kept to a minimum. However, recommendations in Table 1 do allow for some

local inundation at most locations due to the magnitude of the design flow rates when compared with the limited headwater available.

5. Final design of these culvert installations shall require grading, diking, and channelization work at all locations.

R24/R576

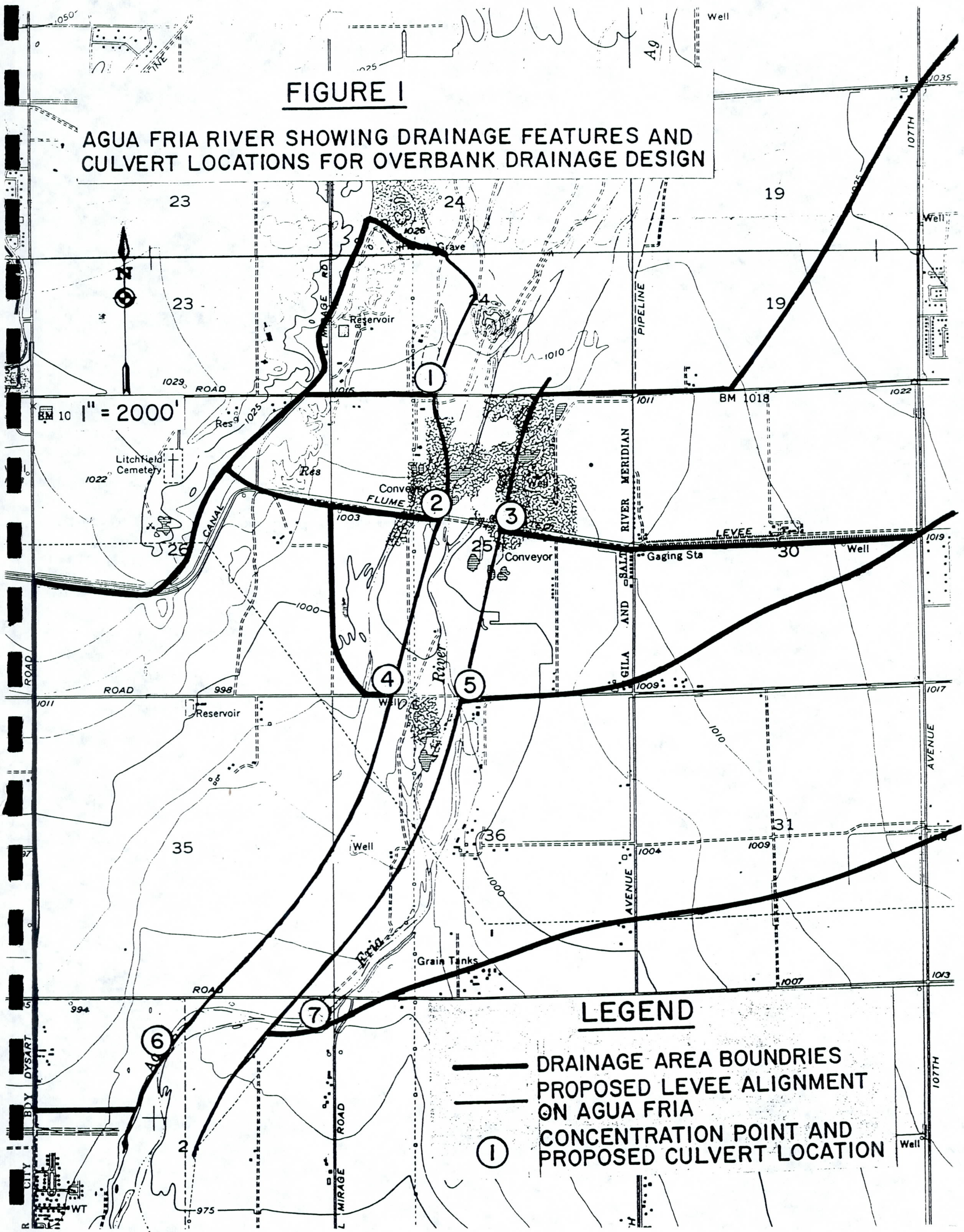

FIGURE I

AGUA FRIA RIVER SHOWING DRAINAGE FEATURES AND CULVERT LOCATIONS FOR OVERBANK DRAINAGE DESIGN

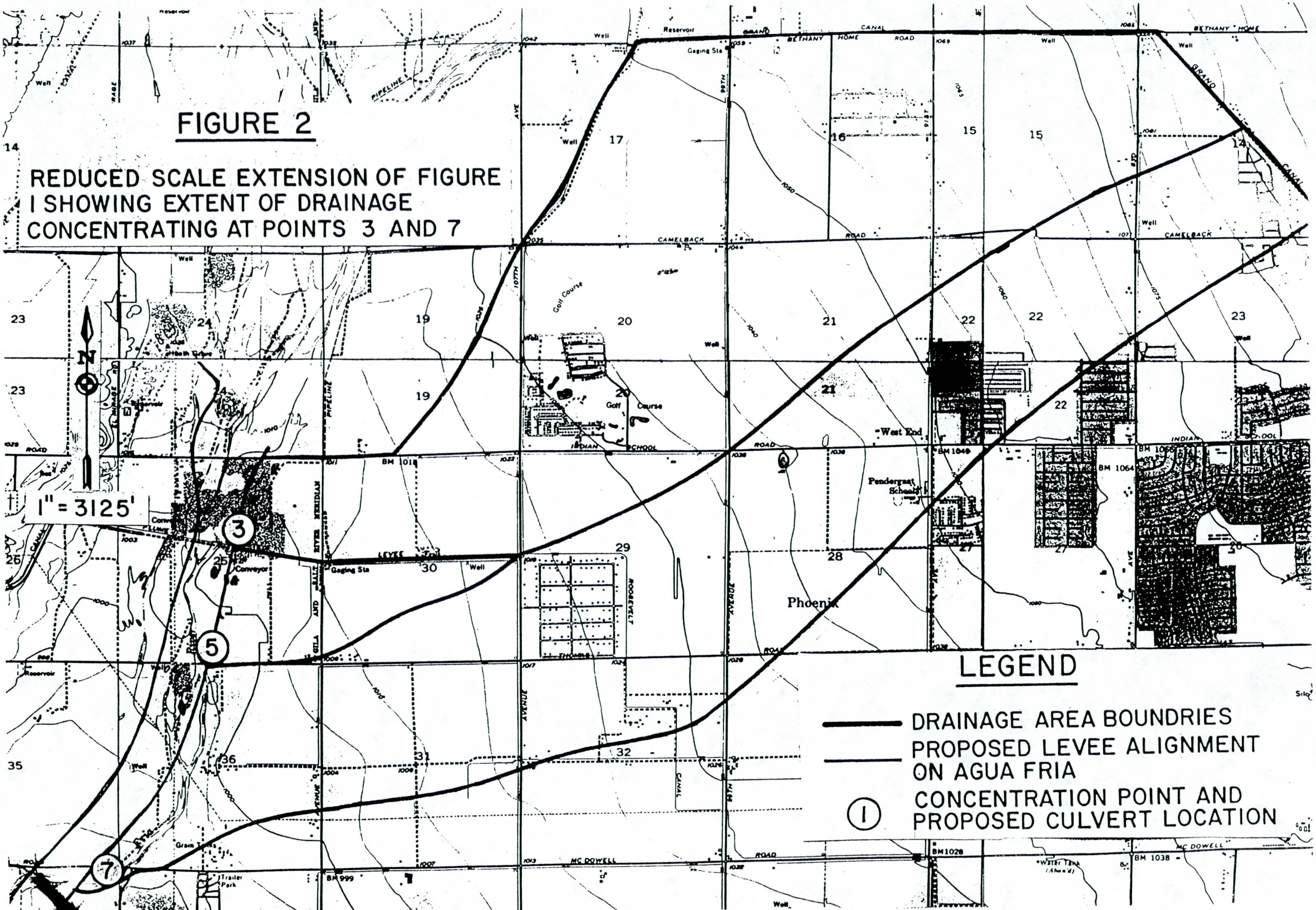

FIGURE 2

REDUCED SCALE EXTENSION OF FIGURE I SHOWING EXTENT OF DRAINAGE CONCENTRATING AT POINTS 3 AND 7

1" = 3125'

TABLE 1

SUMMARY OF RECOMMENDATIONS FOR
OVERBANK DRAINAGE ON THE AGUA FRIA
BETWEEN CAMELBACK ROAD AND THOMAS ROAD, AND
BETWEEN MCDOWELL ROAD AND I-10

Location No. (See Fig. 1)	Location within Agua Fria Channelization	Design Discharge for Q_{100} (cfs)	Culvert Recommendation*
1	North of Indian School Road, West Bank	383	2 CBC's
2	North of Proposed RID Siphon, West Bank	292	2 CBC's
3	North of Proposed RID Siphon, West Bank	1194	5 CBC's
4	North of Thomas Road, West Bank	211	3 RCP's or 2 CBC's
5	North of Thomas Road, East Bank	243	3 RCP's or 2 CBC's
6	Approximately 1200 feet North of I-10, West Bank	715	3 CBC's
7	North of McDowell Road, East Bank	942	See Text

* All CBC's are 8'x4'
All RCP's are 48"

SIDE DRAINAGE RECOMMENDATIONS FOR THE AGUA FRIA CHANNELIZATION

100 YR DESIGN CALCULATION SHEETS

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ______ County ______
Location ______
Project No. ______ Station ______
~~Name of Stream~~ CONCENTRATION POINT # 1

DESIGN DATA

Design Frequency		100	years
Drainage Area	A_1	138.7	acres
	A_2		acres
	A_3		acres
Drainage Length		3000	feet
Elevation			
Top of Drainage Area		1022	feet
At Structure		1008	feet
Drainage Area Slope		0.5	%
Precipitation			
P = 6-hour		3.15	inches
P = 24-hour		3.63	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		2.56	inches
Time of Concentration	Tc	29	minutes
Rainfall Intensity	i	4.18	inches/hour
Runoff Coefficient	C_1	0.66	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.66	
Peak Discharge Qp = CiA =		383	cfs

Computed by JMW Date 10-10-84

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ______ County ______
Location ______
Project No. ______ Station ______
~~Name of Stream~~ CONCENTRATION POINT #2

DESIGN DATA

Design Frequency		100	years
Drainage Area	A_1	145.1	acres
	A_2		acres
	A_3		acres
Drainage Length		2800	feet
Elevation			
Top of Drainage Area		1010	feet
At Structure		1000	feet
Drainage Area Slope		0.36	%
Precipitation			
P = 6-hour			inches
P = 24-hour			inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		2.56	inches
Time of Concentration	Tc*	46	minutes
Rainfall Intensity	i	3.10	inches/hour
Runoff Coefficient	C_1	0.64 (50%)	
	C_2	0.66 (50%)	
	C_3		
Weighted Runoff Coefficient	C	0.65	
Peak Discharge Qp = CiA =		292	cfs

Computed by JMW Date 10-10-84

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

~~Highway~~ __________ County __________

Location __________

Project No. __________ Station __________

~~Name of Stream~~ CONCENTRATION POINT # 3

DESIGN DATA

Design Frequency		100	years
Drainage Area	A_1	3657.5	acres
	A_2		acres
	A_3		acres
Drainage Length		27200	feet
Elevation			
Top of Drainage Area		1085	feet
At Structure		1000	feet
Drainage Area Slope		0.31	%
Precipitation			
P = 6-hour		3.15	inches
P = 24-hour		3.63	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		2.56	inches
Time of Concentration	Tc *	373	minutes
Rainfall Intensity	i	0.51	inches/hour
Runoff Coefficient	C_1	0.64	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.64	
Peak Discharge Qp = CiA =		1194	cfs

Computed by JMW Date 10-11-84

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ______ County ______
Location ______
Project No. ______ Station ______
~~Name of Stream~~ CONCENTRATION POINT #4

DESIGN DATA

Design Frequency		100	years
Drainage Area	A_1		acres
	A_2	111.1	acres
	A_3		acres
Drainage Length		3200	feet
Elevation			
Top of Drainage Area		1000	feet
At Structure		992	feet
Drainage Area Slope		0.25	%
Precipitation			
P = 6-hour		3.15	inches
P = 24-hour		3.63	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		2.56	inches
Time of Concentration	Tc^*	51	minutes
Rainfall Intensity	i	2.88	inches/hour
Runoff Coefficient	C_1	0.66	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.66	
Peak Discharge Qp = CiA =		211	cfs

Computed by JMW Date 10-10-84

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ________ County ________

Location ________

Project No. ________ Station ________

~~Name of Stream~~ CONCENTRATION POINT # 5

DESIGN DATA

Design Frequency		100	years
Drainage Area	A_1	308.5	acres
	A_2	—	acres
	A_3	—	acres
Drainage Length (L)		8200	feet
Elevation			
Top of Drainage Area		1019	feet
At Structure		992	feet
Drainage Area Slope (S)		0.33	%
Precipitation			
P = 6-hour		3.15	inches
P = 24-hour		3.63	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		2.56	inches
Time of Concentration	Tc *	146	minutes
Rainfall Intensity	i	1.23	inches/hour
Runoff Coefficient	C_1	0.64	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.64	
Peak Discharge Qp = CiA =		243	cfs

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ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ______ County ______
Location ______
Project No. ______ Station ______
~~Name of Stream~~ CONCENTRATION POINT #6

DESIGN DATA

Design Frequency		100	years
Drainage Area	A_1	1053.3	acres
	A_2		acres
	A_3		acres
Drainage Length		10,900	feet
Elevation			
Top of Drainage Area		1015	feet
At Structure		975	feet
Drainage Area Slope		0.37	%
Precipitation			
P = 6-hour		3.15	inches
P = 24-hour		3.63	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		2.56	inches
Time of Concentration	Tc*	173	minutes
Rainfall Intensity	i	1.06	inches/hour
Runoff Coefficient	C_1	0.64	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.64	
Peak Discharge Qp = CiA =		715	cfs

Computed by JMW Date 10-10-84

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ________ County ________
Location ________
Project No. ________ Station ________
~~Name of Stream~~ CONCENTRATION POINT #7

DESIGN DATA

Design Frequency		100	years
Drainage Area	A_1	3590.5	acres
	A_2		acres
	A_3		acres
Drainage Length		36,600	feet
Elevation			
Top of Drainage Area		1095	feet
At Structure		980	feet
Drainage Area Slope		0.31	%
Precipitation			
P = 6-hour		3.15	inches
P = 24-hour		3.63	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		2.56	inches
Time of Concentration	Tc	468	minutes
Rainfall Intensity	i	0.41	inches/hour
Runoff Coefficient	C_1	0.64	
	C_2		
	C_3		
Weighted Runoff Coefficient	C		
Peak Discharge Qp = CiA =		942	cfs

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DERIVATION OF RAINFALL - RUN-OFF RATIO C FOR DRAINAGE AREA CONCENTRATING AT POINT 1

FROM: HYDROLOGIC DESIGN ~~FOR~~ FOR HIGHWAY DRAINAGE IN ARIZONA (ADOT)

FOR DESERT BRUSH W/ 10% COVER SOIL TYPE C ; CN = 89

THE FOLLOWING METHODOLOGY FOR DETERMINING THE RAINFALL TO RUNOFF RATIO, C, IS TAKEN FROM THE "PIMA COUNTY HYDROLOGY MANUAL FOR ENGINEERING DESIGN" - PP 104-109

$$CN^* = \frac{R_1(P_1 - 0.88) + R_2}{P_1}$$

WHERE: CN^* = ADJUSTED CURVE #
P_1 = 100 YR, 1 HR RAINFALL
R_1 = 96.0 } SEE PG 108
R_2 = 72.6 } PIMA CTY MANUAL

$$= \frac{96.0(2.56 - 0.88) + 72.60}{2.56}$$

$$CN^* = 91.4$$

$$C = \frac{(P_1 - 0.2S)^2}{P_1(P_1 + 0.8S)}$$

WHERE $S = \frac{1000}{CN^*} - 10 = \frac{1000}{91.4} - 10 = 0.95$

$$C = \frac{(2.56 - 0.2(0.95))^2}{2.56(2.56 + 0.8(0.95))} = 0.66$$

FOR ALL DRAINAGE AREAS:

TIME OF CONCENTRATION COMPUTED BY THE FOLLOWING EQN ON FIG. 3-1 OF ADOT MANUAL*:

$$T_c = \frac{.04593\, L^{.77}}{S^{.385}}$$

WHERE L = DRAINAGE LENGTH
S = " SLOPE (

(* EXCEPT WHERE ADJUSTED AS NOTED ON CALC SHEETS)

RAINFALL INTENSITY, i, DETERMINED FROM:
FIGURE 3-2 OF THE ABOVE ADOT MANUAL FOR $T_c < 60$ MIN
FIGURE 2-2 " " " " " " $1 HR < T_c < 6 H$
TABLE 1 " " " " " " $T_c > 6$ HR

DERIVATION OF RAINFALL - RUNOFF RATIO C
FOR DRAINAGE AREA CONCENTRATING @ POINT 2:

FROM: "HYDROLOGIC DESIGN FOR HIGHWAY DRAINAGE IN ARIZONA" (ADOT) AND SCS NAT'L ENGINEERING HANDBOOK

* FOR: DESERT BRUSH w/ 10% COVER (ADOT) } CN = 89

FOR: STRAIGHT ROW CROPS IN POOR HYDROLOGIC CONDITION (SCS) } CN = 88

DRAINAGE AREA IS APPROXIMATLY 50% DESERT BRUSH
50% ROW CROPS

FOR: DESERT BRUSH AREA, C = 0.66 (SEE AREA #1 CALCS)

FOR: STRAIGHT ROW CROPS IN POOR IN POOR HYDROLOGIC CONDITION, C = 0.64 (SEE AREA # 3, 5, & 6 CALCS)

USE C = 0.65

FOR THIS DRAINAGE AREA THE TIME OF CONCENTRATION WAS ADJUSTED UPWARD BY A FACTOR OF 2 FOR THE 50% OF FLOW OCCURING AS OVERLAND FLOW THROUGH CULTIVATED AREAS TO REFLECT THE REDUCED VELOCITY OF FLOWS EXPECTED THROUGH THESE AREAS

I.E. ADJUSTED $T_c = T_c^* = 0.5(T_c) + 0.5(2T_c)$

$$T_c = 1.5\, T_c$$

WHERE T_c IS THE TIME OF CONCENTRATION GIVEN BY FIGURE 3.1 OF THE ADOT HYDROLOGY MANUAL

T_c^* WAS THEN USED TO DETERMINE THE RAINFALL INTENSITY, i, AS DESCRIBED EARLIER

DERIVATION OF RAINFALL - RUNOFF RATIO C FOR DRAINAGE AREA CONCENTRATING AT POINT 4 :

FROM : HYDROLOGIC DESIGN FOR HIGHWAY DRAINAGE IN ARIZONA (ADOT) AND SCS NAT'L ENGINEERING HANDBOOK

FOR: DESERT BRUSH W/ 10% COVER (ADOT) } CN = 89

FOR: STRAIGHT ROW CROPS IN POOR HYDROLOGICAL CONDITION (SCS) } CN = 88

DRAINAGE AREA IS : 70% DESERT BRUSH (C = 0.66, SEE AREA 1 CALCS)
10% ROW CROPS (C = 0.64, SEE AREA 3 CALCS)

USE $C = 0.66$.

FOR THIS DRAINAGE AREA THE TIME OF CONCENTRATION T_c WAS ADJUSTED UPWARD BY A FACTOR OF 2 FOR THE 30% OF FLOW OCCURING AS OVERLAND FLOW THROUGH CULTIVATED AREAS I.E.

ADJUSTED $T_c = T_c^* = 0.7(T_c) + 0.3(2T_c)$

$T_c^* = 1.3\, T_c$

THE ADJUSTED T_c WAS THEN USED TO DERIVE THE RAINFALL INTENSITY, i.

DERIVATION OF RAINFALL - RUNOFF RATIO C FOR DRAINAGE AREAS CONCENTRATING @ POINTS 3, 5, 6, & 7

FROM: SCS NAT'L ENGINEERING HANDBOOK

FOR: STRAIGHT ROW CROPS IN POOR HYROLOGICAL COND } CN = 88

FROM PIMA COUNTY HYDROLOGY MANUAL:

$$CN^* = \frac{R_1 (P_1 - 0.88) + R_2}{P_1}$$

$$CN^* = \frac{95.5 (2.56 - 0.88) + 71.72}{2.56} = 90.7$$

$$C = \frac{(P_1 - 0.2 S)^2}{P_1 (P_1 + 0.8 S)}$$

WHERE $S = \frac{1000}{CN^*} - 10 = \frac{1000}{90.7} - 10 = 1.03$

$$C = \frac{(2.56 - 0.2(1.03))^2}{2.56 (2.56 + 0.8(1.03))}$$

$$C = 0.64$$

FOR THESE DRAINAGE AREAS Tc WAS ADJUSTED UPWARD BY A FACTOR OF 2 FOR 100% OVERLAND FLOW THROUGH CULTIVATED LAND. THIS ADJUSTED Tc (Tc^*) WAS USED TO DERIVE THE RAINFALL INTENSITY i.

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CLIENT ________ JOB NO. ________ PAGE ________
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DETERMINATION OF PRECIPITATION VALUES FOR VARIOUS DURATIONS AND RETURN INTERVALS:

$$Y_{100} = 0.494 + 0.755 \cdot (X_3^2 / X_4)$$

$$Y_2 = -0.011 + 0.942 \, (X_1^2 / X_2)$$

WHERE:

Y_{100}	=	100 YR	, 1 HR	VALUE		
Y_2	=	2 "	, " "	"		
X_1	=	" "	, 6 "	"	FROM TABLE 1	
X_2	=	" "	, 24 "	"	" " "	
X_3	=	100 "	, 6 "	"	" " "	
X_4	=	" "	, 24 "			

$$Y_{100} = 0.494 + 0.755 \left((3.15)^2 / (3.63) \right)$$

$$= 2.56 \text{ IN.}$$

$$Y_2 = -0.011 + 0.942 \left((1.2)^2 / (1.41) \right)$$

$$= 0.95 \text{ IN}$$

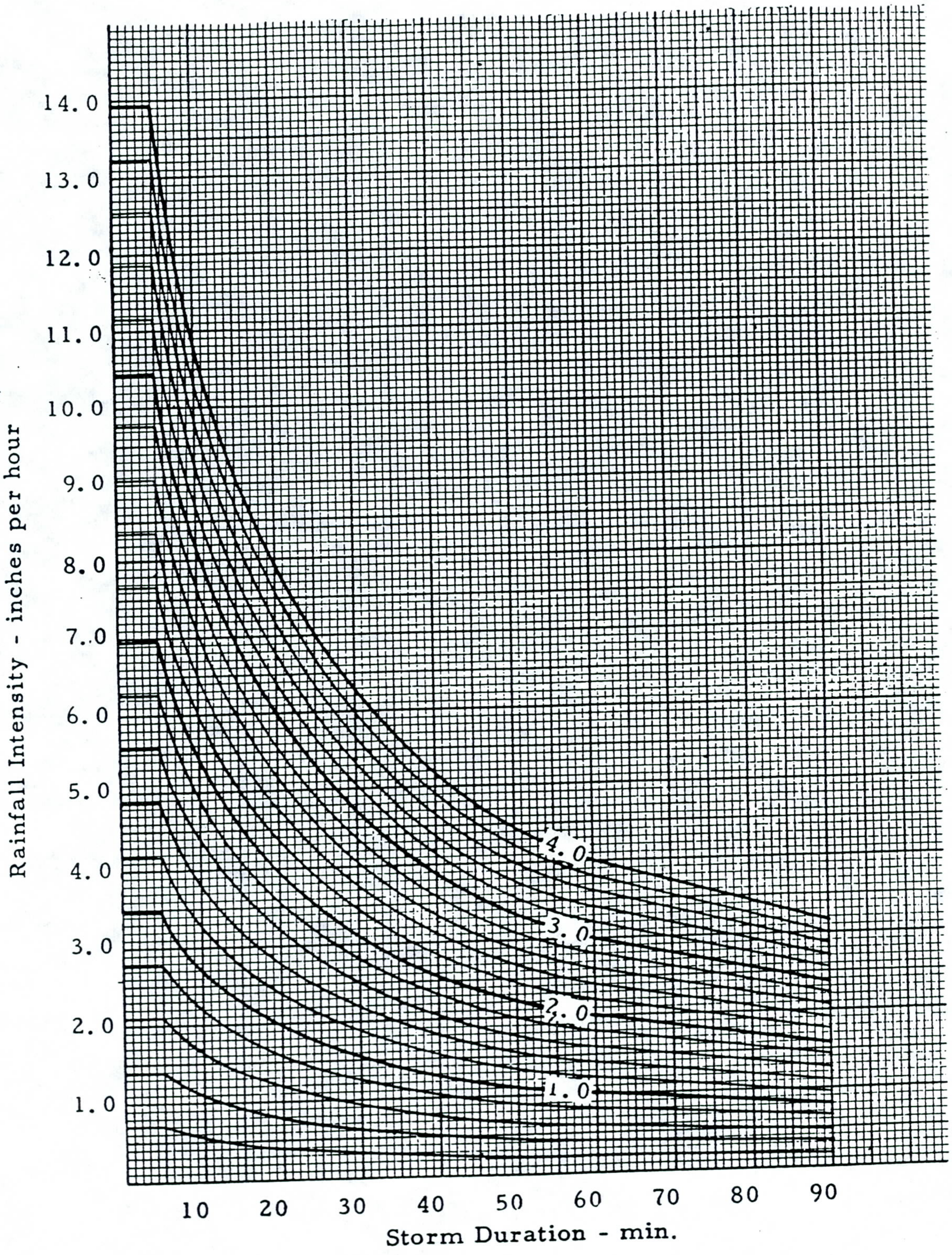


Fig. 3-2

STANDARD DURATION RAINFALL - INTENSITY CURVES

Based on:
Weather Bureau
Technical Paper No. 40

ADDENDUM to "HYDROLOGIC DESIGN FOR HIGHWAY DRAINAGE IN ARIZONA" April 1975

Steps to be used to determine precipitation values for various durations and return periods.

STEP 1. From the precipitation maps in the manual "Hydrologic Design for Highway Drainage in Arizona", determine the precipitation values for the 6 and 24 hour duration storms for return periods of 2, 5, 10, 25, 50 and 100 years. Tabulate these values in Table 1 in the column headed 'Map Values'

TABLE 1

Return Period (Years)	Precipitation Values (inches)			
	6 hour duration		24 hour duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.2	1.2	1.4	1.41
5	1.7	1.7	2.0	1.95
10	2.0	2.0	2.3	2.33
25	2.4	2.4	2.8	2.78
50	2.8	2.8	3.2	3.21
100	3.0	3.15	3.8	3.63

NOTE: There is a possibility of making an error while reading the maps because, (1) a site is not easy to locate precisely on a series of 12 maps, (2) there may be some slight registration differences in printing, and (3) precise interpolation between isolines is difficult. In order to minimize any errors in reading the maps, these values should be plotted on the diagram "Precipitation Depth versus Return Period" Fig. 1.

Project No. ______________ Station ______________

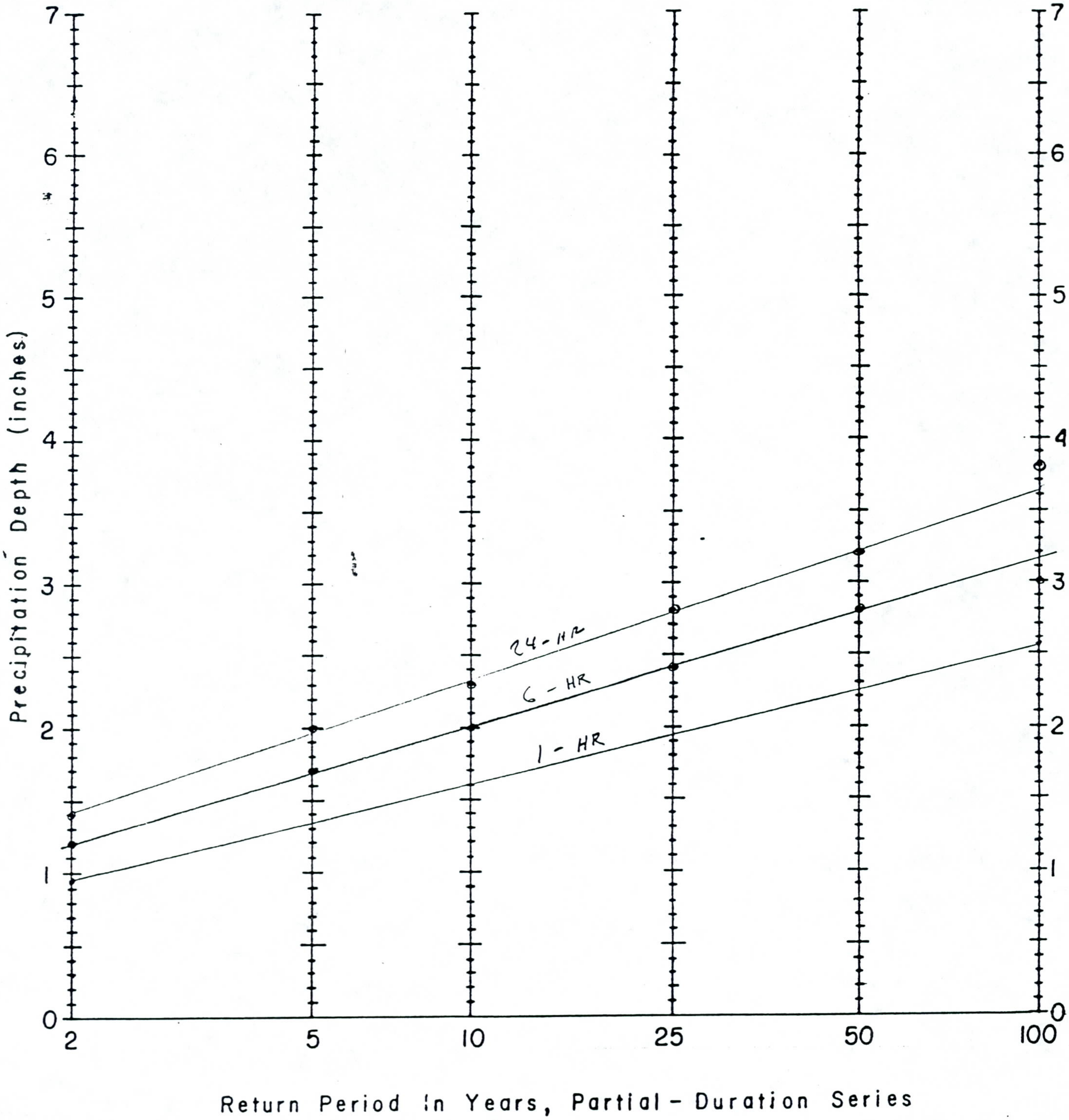
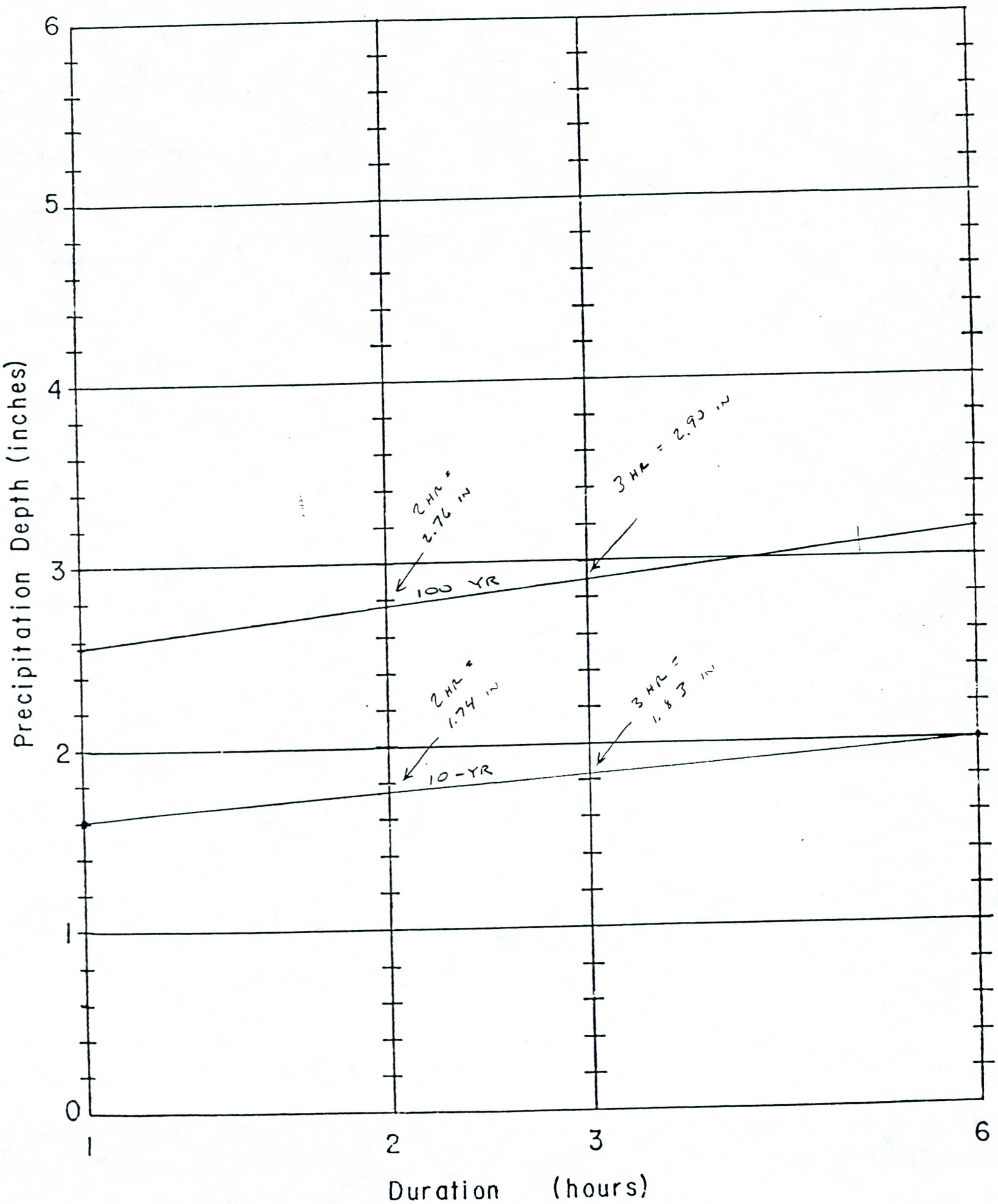


Figure 1 Precipitation Depth Versus Return Period for Partial-Duration Series

Figure 2-2 (revised 4-75)

Precipitation Depth — Duration Diagram (1—6 hours)



ALTERNATIVE SIDE DRAINAGE RECOMMENDATIONS FOR THE AGUA FRIA CHANNELIZATION

The following alternatives in addressing side-drainage culvert recommendations for the Agua Fria Channelization that consider other than the 100-year design were investigated at the request of the Maricopa County Flood Control District.

These recommendations are alternatives to the 100-year design recommendations contained in the summary report entitled "Side Drainage Recommendations for the Agua Fria Channelization" to which this report is attached.

These alternative recommendations are based on a design criteria stipulated by the Maricopa County Flood Control District whereby the culvert design shall be for a 10-year return interval flow such that the headwater elevations required to accommodate the design flow shall not exceed the water-surface elevations corresponding to the Standard Project Flood on the Agua Fria River under existing conditions at the same point.

Table 1A on the following page lists the culvert recommendations based on the above design criteria, along with the design flow rate and approximate headwater elevation. The culvert locations are the same as those found in Table 1 of the 100-year design summary report. The derivation of the 10-year peak flows shown in Table 1 were based on the same methodology and assumptions listed in the 100-year design summary. Calculation sheets are attached.

It should be noted that no recommendations are made for Concentration Points 2 and 3. At these locations, the limiting headwater elevations, as defined by the above design criteria, is above the elevation of the top of the proposed levee on the Agua Fria at this point, but below the elevation of the top of the bermed canal proposed to replace the RID flume at this same location point (see 100-year design summary report). This indicates that the maximum headwater elevation at these locations cannot exceed the limiting headwater regardless of the design flow rate.

Any water impounded behind the above-noted levee and canal is expected to drain either into the gravel pits located on the north side of the above canal, or into the gravel pits on the south side of the above canal by way of conduits which pass under the canal at either location.

The culverts proposed for Concentration Points 4 and 5 require headwater elevations considerably lower than the limiting headwater as defined in the design criteria. Preliminary investigation of these two sites indicates that the cost associated with construction of the extensive spur dikes which would be required to accommodate a higher headwater elevation due to the flat terrain would outweigh the benefits realized from the installation of fewer or smaller culverts. The culvert proposals shown for these two sites, however, will require some diking and channelization work regardless of the culvert design. The installation at Concentration Point 4 will still require the excavation of a 500-foot drainageway, as described in the 100-year design summary report.

The recommendation for Concentration Point 7 remains unchanged from the one given in the 100-year design summary report with the exception of the change in the design flow rate and the design headwater elevation.

Finally, it is recommended that at each location in Table 1A, an 80-foot plus or minus section of the Agua Fria channelization levee be constructed with an 8-inch to 12-inch facing of gunite rather than the 9-foot soil-cement facing. Soil cement would, however, be utilized in the toe area of this 80-foot plus or minus section to approximately the river flow line. The gunite would be keyed into the soil cement on the sides and bottom. Such a section could be easily removed at a minimum expense, should improvement to the side drainage installations be deemed necessary at a later date. This recommendation also applies to Points 2 and 3, where no culvert is recommended in Table 1A.

TABLE 1A

SUMMARY OF ALTERNATIVE RECOMMENDATIONS FOR OVERBANK DRAINAGE ON THE AGUA FRIA

*Location Number	10-Year Discharge (cfs)	Culvert Recommendation	Approximate Headwater Elevation at Culvert for 10-Year Discharge	Approximate SPF WSEL on Agua Fria Riv. Under Existing Conditions
1	149	1, 42" RCP	1013.5'	1019.14'
2	114	See Text		
3	468	See Text		
4	86	2, 36" RCP's	995.24'	1002.28'
5	90	2, 36" RCP's	995.36'	1002.28'
6	265	3, 42" RCP's	982.25'	984.13'
7	388	See Text		

*See Figure 1 of 100-Year Design Summary Report.

sla
SIMONS, LI & ASSOCIATES, INC.

CLIENT	JOB NO.	PAGE
PROJECT	DATE CHECKED	DATE
DETAIL	CHECKED BY	COMPUTED BY

ALTERNATIVE SIDE DRAINAGE RECOMMENDATIONS
FOR THE AGUA FRIA CHANNELIZATIONS

10-YR DESIGN CALCULATION SHEETS

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway __________ County __________

Location __________

Project No. __________ Station __________

Name of Stream C.P. #1

DESIGN DATA

Design Frequency		10	years
Drainage Area	A_1	138.7	acres
	A_2		acres
	A_3		acres
Drainage Length		3000	feet
Elevation			
Top of Drainage Area		1022	feet
At Structure		1008	feet
Drainage Area Slope		0.5	%
Precipitation			
P = 6-hour		2.00	inches
P = 24-hour		2.33	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.60	inches
Time of Concentration	Tc	29	minutes
Rainfall Intensity	i	2.50	inches/hour
Runoff Coefficient	C_1	0.43	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.43	
Peak Discharge Qp = CiA =		149	cfs

Computed by __________ Date __________

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ________ County ________
Location ________
Project No. ________ Station ________
Name of Stream CP #2

DESIGN DATA

Design Frequency		10	years
Drainage Area	A_1	145.1	acres
	A_2		acres
	A_3		acres
Drainage Length		2800 -	feet
Elevation			
Top of Drainage Area		1010	feet
At Structure		1000	feet
Drainage Area Slope		0.36	%
Precipitation			
P = 6-hour		2.00	inches
P = 24-hour		2.33	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.60	inches
Time of Concentration	Tc *	46	minutes
Rainfall Intensity	i	1.90	inches/hour
Runoff Coefficient	C_1	0.43 (50%)	
	C_2	0.40 (50%)	
	C_3		
Weighted Runoff Coefficient	C	0.415	
Peak Discharge Qp = CiA =		114	cfs

Computed by ________ Date ________

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ______ County ______
Location ______
Project No. ______ Station ______
Name of Stream C.P. * 3

DESIGN DATA

Design Frequency		10	years
Drainage Area	A_1	3657.5	acres
	A_2		acres
	A_3		acres
Drainage Length		27,200 -	feet
Elevation			
Top of Drainage Area		1085	feet
At Structure		1000	feet
Drainage Area Slope		0.31	%
Precipitation			
P = 6-hour		2.00	inches
P = 24-hour		2.33	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.60	inches
Time of Concentration	Tc^*	373	minutes
Rainfall Intensity	i	0.32	inches/hour
Runoff Coefficient	C_1	0.40	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.40	
Peak Discharge Qp = CiA =		468	cfs

Computed by ______ Date ______

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ____________ County ____________
Location ____________
Project No. ____________ Station ____________
Name of Stream C.P. #4

DESIGN DATA

Design Frequency		10	years
Drainage Area	A_1	111.1	acres
	A_2		acres
	A_3		acres
Drainage Length		3200 -	feet
Elevation			
Top of Drainage Area		1000	feet
At Structure		992	feet
Drainage Area Slope		0.25	%
Precipitation			
P = 6-hour		2.00	inches
P = 24-hour		2.33	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.60	inches
Time of Concentration	Tc*	51	minutes
Rainfall Intensity	i	1.80	inches/hour
Runoff Coefficient	C_1	0.43	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.43	
Peak Discharge Qp = CiA =		86	cfs

Computed by ____________ Date ____________

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ____ County ____
Location ____
Project No. ____ Station ____
Name of Stream C.P. #5

DESIGN DATA

Design Frequency		10	years
Drainage Area	A_1	308.5	acres
	A_2		acres
	A_3		acres
Drainage Length		8200 -	feet
Elevation			
Top of Drainage Area		1019	feet
At Structure		992	feet
Drainage Area Slope		0.33	%
Precipitation			
P = 6-hour		2.00	inches
P = 24-hour		2.33	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.60 -	inches
Time of Concentration	Tc *	146	minutes
Rainfall Intensity	i	0.73	inches/hour
Runoff Coefficient	C_1	0.40	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.40	
Peak Discharge Qp = CiA =		90	cfs

Computed by ____ Date ____

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ______ County ______
Location ______
Project No. ______ Station ______
* Name of Stream C.P. #6

DESIGN DATA

Design Frequency		10	years
Drainage Area	A_1	1053.3	acres
	A_2		acres
	A_3		acres
Drainage Length		10,900-	feet
Elevation			
Top of Drainage Area		1015	feet
At Structure		975	feet
Drainage Area Slope		0.37	%
Precipitation			
P = 6-hour		2.00	inches
P = 24-hour		2.33	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.60	inches
Time of Concentration	Tc *	173	minutes
Rainfall Intensity	i	0.63	inches/hour
Runoff Coefficient	C_1	0.40	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.40	
Peak Discharge Qp = CiA =		265	cfs

Computed by ______ Date ______

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET
RATIONAL METHOD

LOCATION DATA

Highway ______ County ______
Location ______
Project No. ______ Station ______
Name of Stream C. P. # 7

DESIGN DATA

Design Frequency		10	years
Drainage Area	A_1	3590.5	acres
	A_2		acres
	A_3		acres
Drainage Length		36,600	feet
Elevation			
Top of Drainage Area		1095	feet
At Structure		980	feet
Drainage Area Slope		0.31	%
Precipitation			
P = 6-hour		2.00	inches
P = 24-hour		2.33	inches

DESIGN COMPUTATIONS

Precipitation P_1 = 1-hour		1.60	inches
Time of Concentration	Tc*	468	minutes
Rainfall Intensity	i	0.27	inches/hour
Runoff Coefficient	C_1	0.40	
	C_2		
	C_3		
Weighted Runoff Coefficient	C	0.40	
Peak Discharge Qp = CiA =		388	cfs

Computed by ______ Date ______

DERIVATION OF RAINFALL - RUNOFF RATIOS, "C"
FOR 10 - YR EVENT

FOR DESERT BRUSH:

$$CN^{*} = \frac{96.0\,(1.60 - 0.88) + 72.6}{1.60} = 88.6$$

$$S = \frac{1000}{88.6} - 10 = 1.29$$

$$C = \frac{(1.60 - 0.2(1.29))^2}{1.60\,(1.60 + 0.8(1.29))} = \boxed{0.43}$$

FOR ROW CROPS:

$$CN^{*} = \frac{95.5\,(1.60 - 0.88) + 71.72}{1.60} = 87.8$$

$$S = \frac{1000}{87.8} - 10 = 1.39$$

$$C = \frac{(1.60 - 0.2(1.39))^2}{1.6\,(1.60 + 0.8(1.39))} = \boxed{0.40}$$