George V. Sabol Consulting Engineers, Inc.
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BRIGHTON, COLORADO 80601
(303) 457-4015

3 January 1992

Mr. Stephen D. Waters
Flood Control District of Maricopa County
2801 West Durango
Phoenix, Arizona 85009

Subject: Revision of Verification Report for Academy Acres and Walnut Gulch Watersheds (November 1991)

Dear Steve:

I submitted the subject report to you under cover letter of 29 November 1991 and we discussed this at the District on 17 December. At that time, you questioned the values of the Green and Ampt parameters that were used for Academy Acres and the Walnut Gulch watersheds. I was to review those parameter values and to revise the report accordingly, if needed. This was done and a revised report is enclosed.

As you noted, the Green and Ampt parameters for Academy Acres are incorrect. The parameters should have been calculated based on a sandy loam soil texture for both soils in this watershed. This correction was made and this resulted in increased flood discharges from the model and those results correspond very closely to the flood frequency results.

I rechecked the Green and Ampt parameters for the two Walnut Gulch watersheds (63.011 and 63.008) and the parameter values are correct. I suspect that the confusion resulted from the fact that the log area-averaged XKSAT values were calculated along with the corresponding values of PSIF and DTHETA, and then the XKSAT value was corrected for 30% vegetation cover. The log area-averaged value of XKSAT of 0.18 in/hr produces PSIF of 5.8 inch and DTHETA (dry) of 0.39. The XKSAT value after adjustment for 30% vegetation cover is 0.22 in/hr, and these values are correct, I believe. Therefore, there was no need to revise the models or results for the Walnut Gulch watersheds.

I have replaced some of the calculation sheets in the report appendix to more clearly illustrate how the Green and Ampt parameters were calculated. I think that this will help.

A few other minor editorial changes were made to the report. All necessary changes and corrections are made to the January 1992 version of the report.

Please call me if you have any questions. I am incorporating these verification results in the Documentation/Verification Report, as decided during our meeting in December, and will provide this when completed.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo V Sabol

George V. Sabol

Enclosures: Verification Report for Academy Acres and Walnut Gulch Watersheds (January 1992)

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

VERIFICATION REPORT

FOR ACADEMY ACRES AND

WALNUT GULCH WATERSHEDS

George V. Sabol Consulting Engineers, Inc.
Brighton, Colorado and Phoenix, Arizona

January 1992

INTRODUCTION

This report presents the results of verification testing of the procedures in the Maricopa County Hydrologic Design Manual (Manual). Frequency simulation results are presented for three watersheds with basin characteristics and streamgage data as described below:

Watershed	Size sq.mi.	Land-use	Length of Gage Record years
Academy Acres at Albuquerque, NM	0.12	residential	14
Walnut Gulch 63.011 at Tombstone	3.18	desert rangeland	27
Walnut Gulch 63.008 at Tombstone	5.98	desert rangeland	27

The Beaver Creek #8 watershed, about 50 miles south of Flagstaff, was to be used for these test verification purposes also. However, after initial investigations of the watershed it was discovered that volcanic cinder exists in the watershed that negates the use of the Green and Ampt infiltration equation with parameter values based on soil texture. Therefore, that watershed was deleted from further consideration since watersheds of volcanic cinder are not known to be represented in Maricopa County.

Watershed maps and watershed information that were used in developing the models are shown in the appendix. The streamgage data and calculations for the flood frequency analyses for each watershed are also provided in the appendix.

Graphical flood frequency analyses were performed for each of the watersheds using the available gage data. The analyses were performed using the procedure that was developed for the ADOT Hydrology Manual (1991 draft). This includes establishing a best fit line to the data when plotted on probability paper, and also includes 90 percent confidence limit bands about the best fit line. The graphical flood frequency analyses for the three watersheds are shown in Figures 1 through 3.

The following is a discussion of the modeling and results for each of the watersheds. The model results are compared to the flood magnitudes from the flood frequency analyses.

ACADEMY ACRES, ALBUQUERQUE, NM

The drainage area is a small (0.12 sq. mi.) residential area developed on an alluvial fan in the Northeast Heights of Albuquerque. Streets are the major conveyance for storm runoff. There is a relatively short concrete lined channel at the outlet of the watershed and the streamgage is located in the channel. The area consists of 191 single-family and 44 duplex residential units with a density of about 5 units per acre. A church and paved parking lot are contained in the upper part of the watershed. The residences are generally landscaped with irrigated lawns with a small amount of native vegetation that occasionally may be of gravel underlain by plastic.

Rainfall for the frequency simulation was developed by procedures in the Manual using rainfall statistics for this location from the NOAA Atlas for New Mexico (Miller and others, 1973a). For a watershed of this size (0.12 sq. mi.) this required Pattern No. 1 and a depth-area reduction factor of 1.0.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. The soil is a sandy loam and the Green and Ampt parameters are; hydraulic conductivity (XKSAT) of 0.40 in/hr, capillary suction (PSIF) of 4.3 inches, and soil moisture deficit (DTHETA) of 0.25. The surface retention (IA) was estimated as 0.20 inch, and the effective impervious area (RTIMP) as 28%.

A single-basin model using the Clark unit hydrograph was developed. The unit hydrograph parameters were calculated based on an area (A) of 0.124 square miles, watercourse length (L) of 0.9 mile, slope of 105 ft/mi, and resistance coefficient of 0.028. T_c and R varied for each flood return period since the procedures to estimate these parameters are a function of rainfall excess intensity, which varies for each flood return period. The synthetic urban time-area relation was used.

The results of the graphical flood frequency analysis and results of the frequency simulation are shown in Table 1 and Figure 1. The record length is only 14 years, and therefore the accuracy of the flood frequency analysis to represent the "true" flood frequency relation may be questionable. However, the results are very close to the flood frequency analysis best estimates for all three return periods.

TABLE 1

Verification results for the Academy acres Watershed
All values in cfs

Return Period years	Flood Frequency Results Best Estimate	Upper Limit	Lower Limit	Model Results
10	95	140	60	92
25	130	210	80	133
100	190	340	110	197

WALNUT GULCH 63.011 AND 63.008, TOMBSTONE, AZ

These watersheds are instrumented subbasins of the Walnut Gulch Experimental Watershed that is operated by the USDA, Agricultural Research Service near Tombstone, Arizona. Watershed 63.008 is 5.98 square miles and it contains the 3.18 square mile watershed 63.011 as a subbasin. The watersheds consist of undeveloped rangeland, and the vegetation is predominantly native brush, grasses, and cacti with about 30% cover.

The models were run using two different sources for rainfall statistics; the NOAA Atlas for Arizona (Miller and others, 1973b) and site-specific rainfall statistics as developed from information in Osborn and Renard (1988). There is considerable difference between the rainfall statistics from these two sources. The site-specific rainfall statistics are appreciably higher than the NOAA Atlas statistics. For example, the NOAA 100-yr, 1-hr rainfall is 2.43 inches and the site-specific 100-yr, 1-hr rainfall is 3.07 inches (a 26 percent increase over the NOAA statistic). Osborn and Renard do not provide rainfall depth-duration-frequency statistics for durations in excess of 1 hour. However, the Maricopa County procedure requires a 6-hr rainfall depth to define the design storm. Therefore, the Osborn and Renard rainfall statistics were plotted on graph paper along with the NOAA statistics and the Osborn and Renard statistics were extended to 6 hours to follow the same slope of the NOAA lines. This graph is shown in the appendix. This may or may not represent severe storms for the watershed. According to Osborn (personal communication, October 1991) the peak discharges on both watersheds 63.011 and 63.008 resulted from rains of durations less than 1 hour. Although 6-hr type storms may occur over the Walnut Gulch watershed that have similar characteristics to the Maricopa County design storm (1954 Queen Creek storm), such storms apparently have not occurred in that area since the watershed was instrumented. Therefore, modeling of the Walnut Gulch watersheds using the 6-hr Maricopa County design storm may not be representative of the appropriate regional meteorologic conditions. Nonetheless, modeling of these watersheds was performed using 6-hr rainfall depths as described. For watershed 63.011 (3.18 sq. mi.) this required Pattern No. 2.07 and a depth-area reduction factor of 0.97, and for watershed 63.008 (5.98 sq. mi.) the Pattern No. is 2.44 and the depth-area reduction factor is 0.96.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. Watershed 63.011 is a subbasin of 63.008 and the same rainfall loss parameters were calculated for both watersheds. The Green and Ampt parameters were area averaged with hydraulic conductivity (XKSAT) of 0.22 in/hr after correction for the 30% vegetation cover, capillary suction (PSIF) of 5.8 inches, and soil moisture deficit (DTHETA dry) of 0.39. The surface retention (IA) was estimated as 0.35 inch, and the impervious area (RTIMP) as 0%.

These two watersheds were modeled using both the Clark unit hydrograph and the Phoenix Valley S-graph. Watershed 63.008 was modeled as a single-basin and also as a two subbasin model. The watershed characteristics that were used to calculate the Clark unit hydrograph parameters, T_C and R, and the S-graph Lag are shown in Table 2. The synthetic natural time-area relation was used with the Clark unit hydrograph.

TABLE 2

Watershed characteristics used in calculating unit hydrograph parameters for Walnut Gulch watersheds 63.011 and 63.008

Area A sq.mi.	Length L miles	Centroid Length L_{ca} miles	Slope S ft/mi	Resistance Coefficients K_b Clark	Resistance Coefficients K_n S-Graph
Watershed 63.011 Single-Basin Models					
3.18	4.0	1.8	100	0.033	.03
Watershed 63.008 Single-Basin Models					
5.98	8.0	3.6	75	0.033	.03
Watershed 63.008 Multi-Basin Models (Clark only)					
3.18	4.0	---	100	0.033	------
2.80	4.0	---	75	0.033	------

The model results, shown in Tables 3 and 4 and Figures 2 and 3, are within the 90 percent confidence levels for both watersheds when the site-specific rainfall statistics are used. The model results are consistently less than the lower 90 percent confidence level values when the NOAA Atlas rainfall statistics are used. Since the site-specific rainfall statistics more accurately reflect the actual rainfall regime than do the NOAA Atlas statistics, it seems appropriate to evaluate the model performance based on the site-specific rainfall statistics. Because of this, all results that are discussed are the results using the site-specific rainfall statistics that were developed from Osborn and Renard (1988).

Watershed 63.011 is smaller than the recommended 5 square mile upper limit for application of the Clark unit hydrograph, and, therefore, this watershed was modeled as a single basin. Model results for watershed 63.011 are shown in Table 3 and Figure 2. The model results using both the Clark unit hydrograph and the S-graph are very close to the best estimate of the 10-yr flood peak discharge. The results are not as good at the 25- and 100-yr return periods, but are within the 90 percent confidence levels. Considering that this watershed is outside of Maricopa County and that the design rainfall criteria that were applied may not be completely representative of the regional severe storm characteristics, the results are reasonable.

TABLE 3

Verification results for the Walnut Gulch 63.011 watershed
All values in cfs

Return Period years	Flood Frequency Results: Best Estimate	Flood Frequency Results: Upper Limit	Flood Frequency Results: Lower Limit	Using NOAA Statistics: Clark u-hg	Using NOAA Statistics: S-graph	Using Site-Specific Statistics: Clark u-hg	Using Site-Specific Statistics: S-graph
10	1,950	3,220	1,180	560	960	1,760	2,050
25	2,950	6,040	1,850	1,170	1,570	2,030	2,290
100	6,500	13,290	3,180	2,300	2,500	4,380	4,190

Watershed 63.008 is a little larger than the recommended 5 square mile upper limit for application of the Clark unit hydrograph, but is smaller than the absolute 10 square mile upper limit for application. Therefore, this watershed was modeled as a single basin using the Clark unit hydrograph and the S-graph, and was also modeled as a multi-basin (two subbasins) watershed using the Clark unit hydrograph. When modeled as a single basin, the calculated T_C exceeded the duration of rainfall excess indicating that this watershed should not be modeled as a single basin when using the Clark unit hydrograph procedure as described in the Manual.

The model results, shown in Table 4 and Figure 3, are not particularly good when the watershed is treated as a single basin with the Clark unit hydrograph. This provides evidence that the size recommendations for the Clark unit hydrograph procedure should not be exceeded if the calculated T_C exceeds the duration of rainfall excess.

TABLE 4

Verification results for the Walnut Gulch 63.008 watershed
All values in cfs

Return Period years	Flood Frequency Results: Best Estimate	Flood Frequency Results: Upper Limit	Flood Frequency Results: Lower Limit	Using NOAA Statistics: Clark u-hg	Using NOAA Statistics: S-graph	Using NOAA Statistics: Multi-Basin	Using Site-Specific Statistics: Clark u-hg	Using Site-Specific Statistics: S-graph	Using Site-Specific Statistics: Multi-Basin
10	2,100	3,260	1,340	780	1,060	920	1,790	2,450	2,070
25	3,300	5,720	2,010	1,330	1,830	1,540	2,010	2,750	2,320
100	6,200	11,620	3,270	2,220	3,030	2,450	3,820	5,250	5,190

The results of the single basin, S-graph model are reasonable. This indicates that, for small, desert rangeland watersheds, the Phoenix Valley S-graph is a viable unit hydrograph procedure and it can be used in certain applications where the Clark unit hydrograph is either inappropriate (exceeds size limitations) or where expedience may warrant the use of an S-graph rather than the Clark unit hydrograph.

The multi-basin, Clark unit hydrograph model yielded reasonable results for the full range of return periods. This indicates that the Clark unit hydrograph can be used for larger watersheds where it is either necessary or desirable to model the watershed as a system of subbasins.

The model results for both watersheds 63.011 and 63.008 are highly dependent upon the ability of the rainfall input to reflect local, severe storm rainfall characteristics. The rainfall criteria that were applied to these watersheds was developed from an historic 7-hr duration local storm in Maricopa County as represented by the 6-hr design rainfall criteria in the Manual. That rainfall may not be representative of the spatial and temporal distributions of rainfall that actually occur in the Tombstone area. Therefore, the accuracy of the developed rainfall-runoff models to reproduce a recorded flood frequency relation must be interpreted within this assumption.

SUMMARY OF VERIFICATIONS

Flood frequency simulations were performed for three watersheds that have streamgage records. None of the watersheds are in Maricopa County; two are in Cochise County in southeast Arizona, and one is a fully urbanized watershed in Albuquerque, New Mexico. Flood peak discharges were estimated using procedures in the Manual for the watersheds for return periods of 10-, 25- and 100-yr. The ratio of the flood peak discharge, as estimated by the most appropriate model, to the discharge from the best fit flood frequency line are shown in Table 5.

TABLE 5

Results of verifications for the Academy Acres and Walnut Gulch 63.011 and 63.008 watersheds

Return Period years	Ratio of flood peak discharges estimated by procedures in the Manual to discharges from flood frequency analyses		
	Academy Acres[a]	Walnut Gulch 63.011[b]	Walnut Gulch 63.008[c]
10	.97	.90	.99
25	1.02	.69	.70
100	1.04	.67	.84

[a] - Table 1, Model Results: Best Estimate
[b] - Table 3, Clark unit hydrograph and Site-Specific Rain: Best Estimate
[c] - Table 4, Multi-Basin and Site-Specific Rain: Best Estimate

It is also interesting to note that the single-basin Clark unit hydrograph model and the single-basin S-graph model for watershed 63.011 (Table 3) generally yield similar results, and that the multi-basin Clark unit hydrograph model and the single-basin S-graph model for watershed 63.008 (Table 4) generally yield similar results. This provides technical support for the applicability of these unit hydrograph procedures for natural watersheds.

Considering the assumption of the applicability of the Maricopa County design rainfall criteria to these watersheds that are not within Maricopa County, the results seem appropriate in serving as a verification of the modeling procedure.

REFERENCES

Osborn, H.B., and Renard, K.G., 1988, Rainfall intensities for southeastern Arizona; Amer. Soc. of Civil Engineers, Journal of Irrigation and Drainage Engineering, Vol. 114, No. 1, pp. 195-199.

Miller, J.F., Frederick, R.H., and Tracey, R.J., 1973a, NOAA Atlas 2, Precipitation-frequency atlas of the western United States, Volume IV, New Mexico: National Weather Service, U.S. Dept of Commerce, Silver Springs, MD.

Miller, J.F., Frederick, R.H., and Tracey, R.J., 1973b, NOAA Atlas 2, Precipitation-frequency atlas of the western United States, Volume VIII, Arizona: National Weather Service, U.S. Dept of Commerce, Silver Springs, MD.

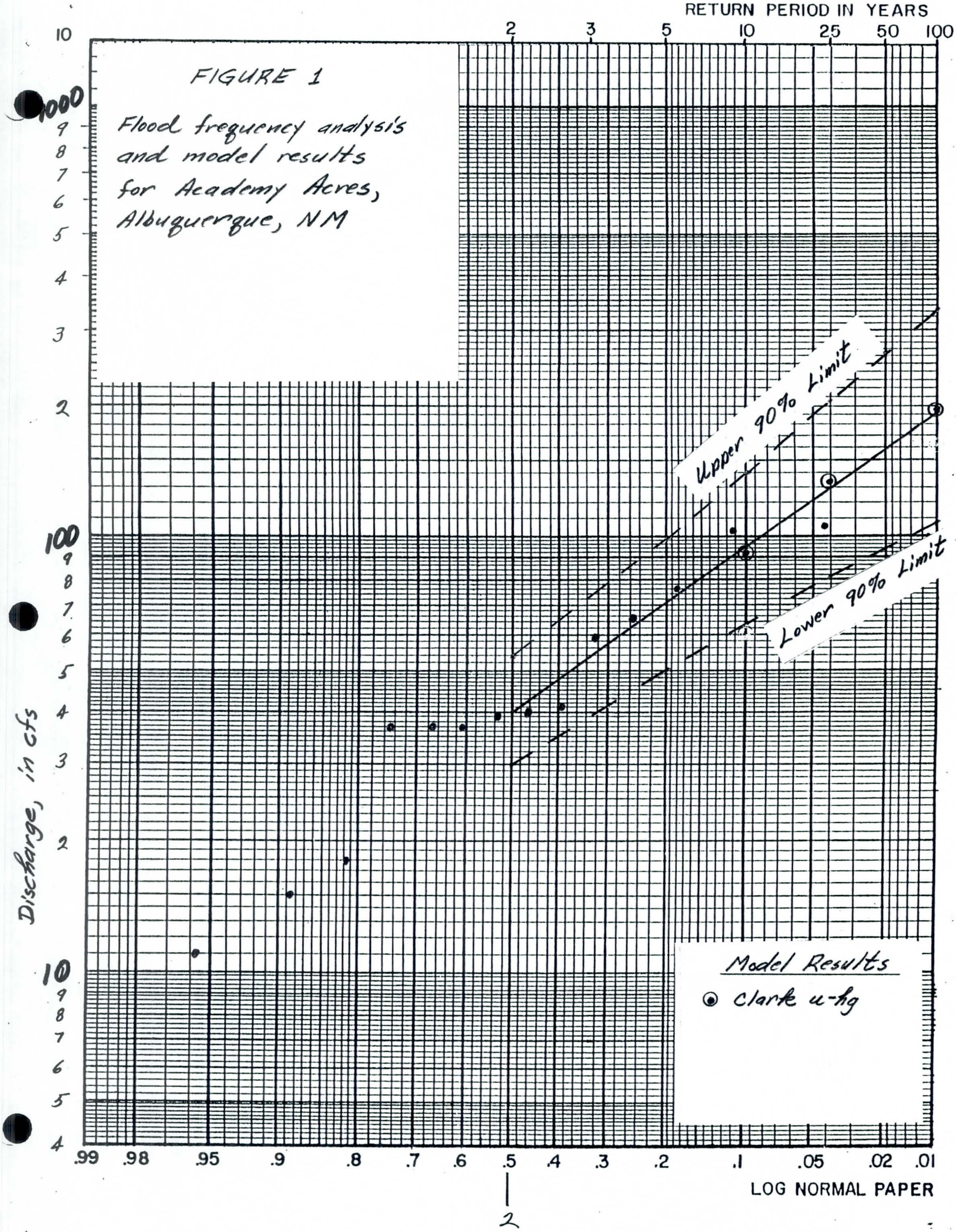
FIGURE 1
Flood frequency analysis and model results for Academy Acres, Albuquerque, NM
RETURN PERIOD IN YEARS
2
3
5
10
25
50
100
Upper 90% Limit
Lower 90% Limit
Model Results
Clark u-hg
Discharge, in cfs
10
1000
9
8
7
6
5
4
3
2
100
10
.99
.98
.95
.9
.8
.7
.6
.5
.4
.3
.2
.1
.05
.02
.01
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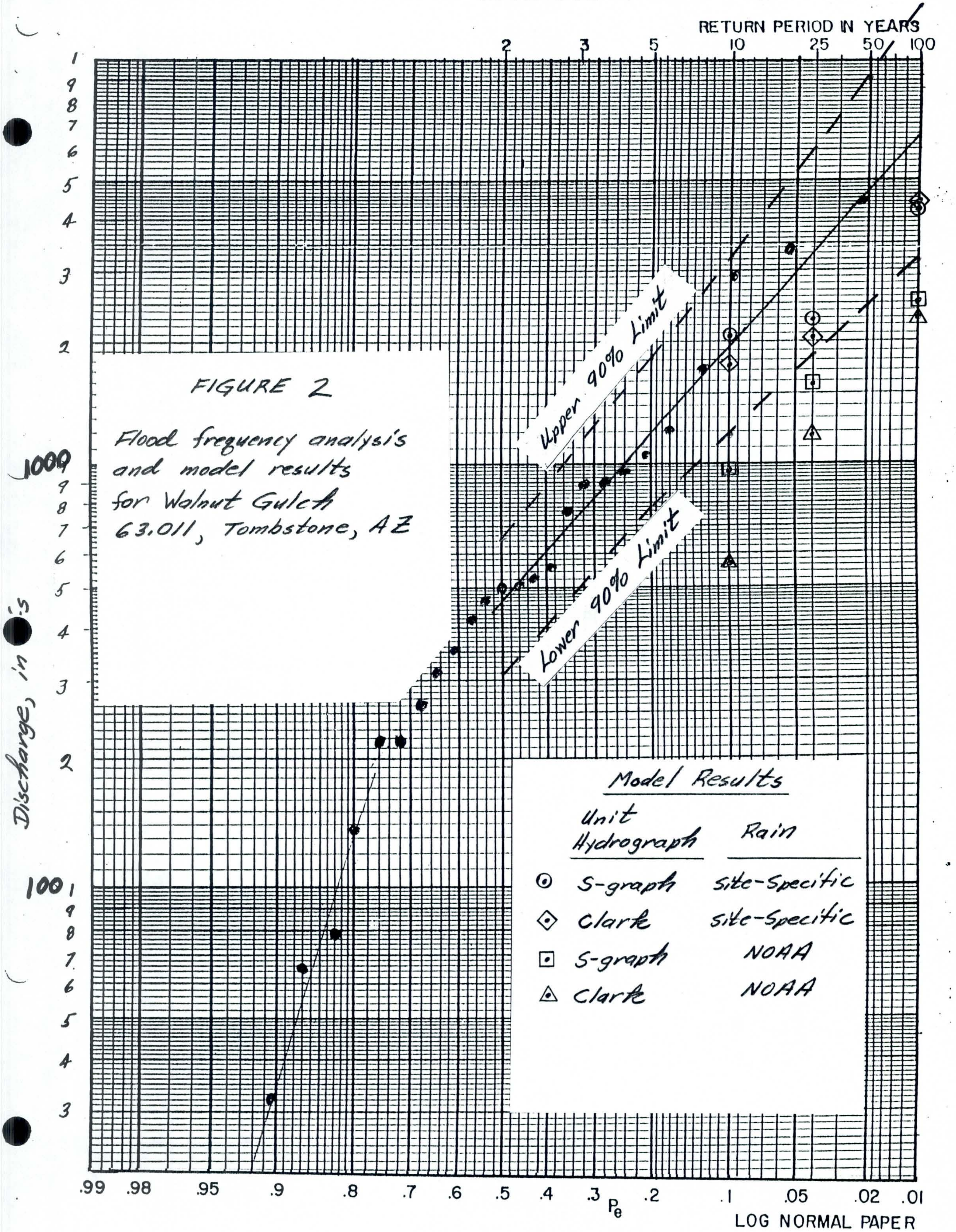
2


RETURN PERIOD IN YEARS
2
3
5
10
25
50
100
Upper 90% Limit
Lower 90% Limit
FIGURE 2
Flood frequency analysis and model results for Walnut Gulch 63.011, Tombstone, AZ
Model Results
Unit Hydrograph
Rain
S-graph
site-Specific
Clark
site-Specific
S-graph
NOAA
Clark
NOAA
Discharge, in cfs
1000
100
.99
.98
.95
.9
.8
.7
.6
.5
.4
.3
.2
.1
.05
.02
.01
P_e
LOG NORMAL PAPER

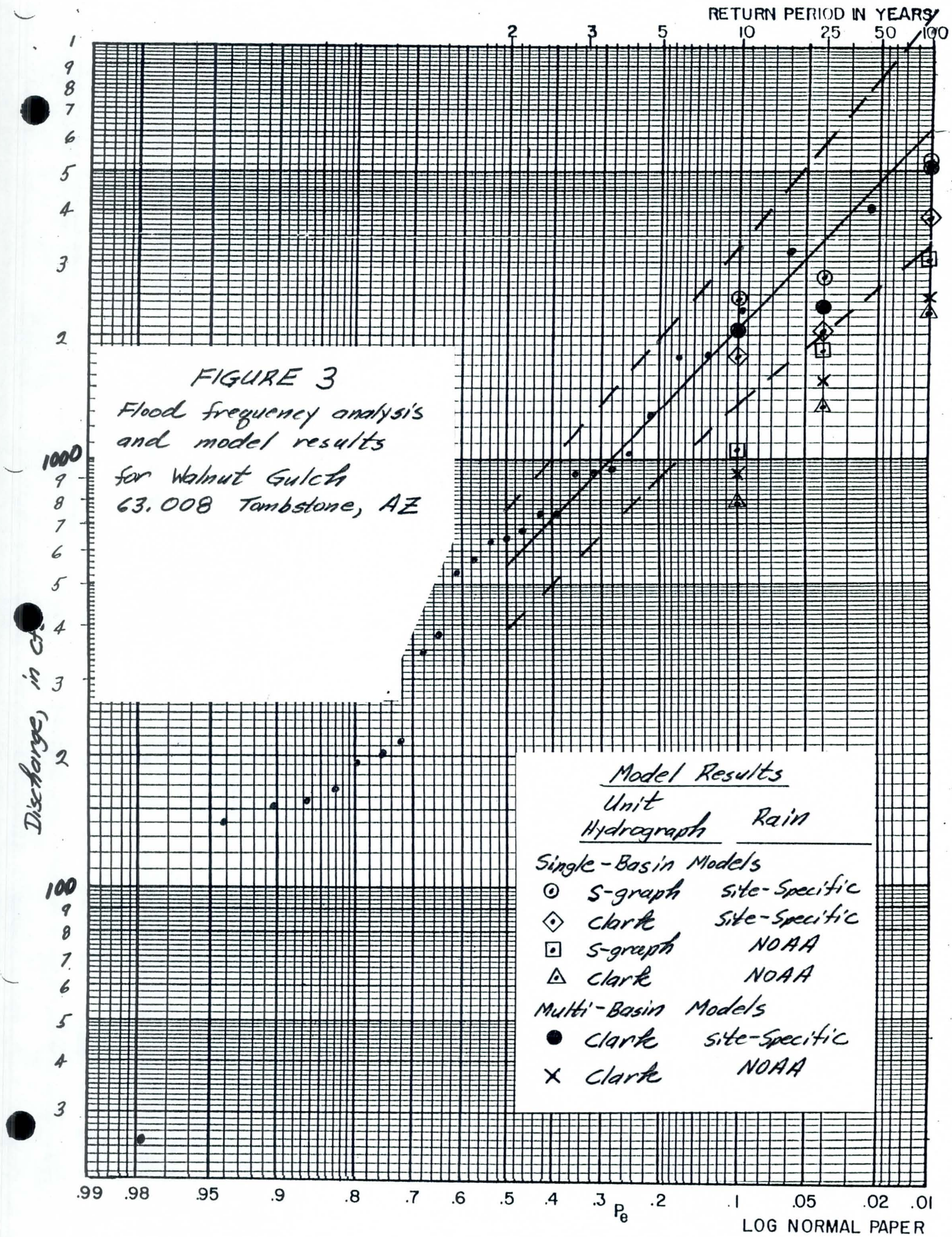


FIGURE 3
Flood frequency analysis and model results for Walnut Gulch 63.008 Tombstone, AZ

APPENDIX

Watershed and Streamgage Data for

Academy Acres

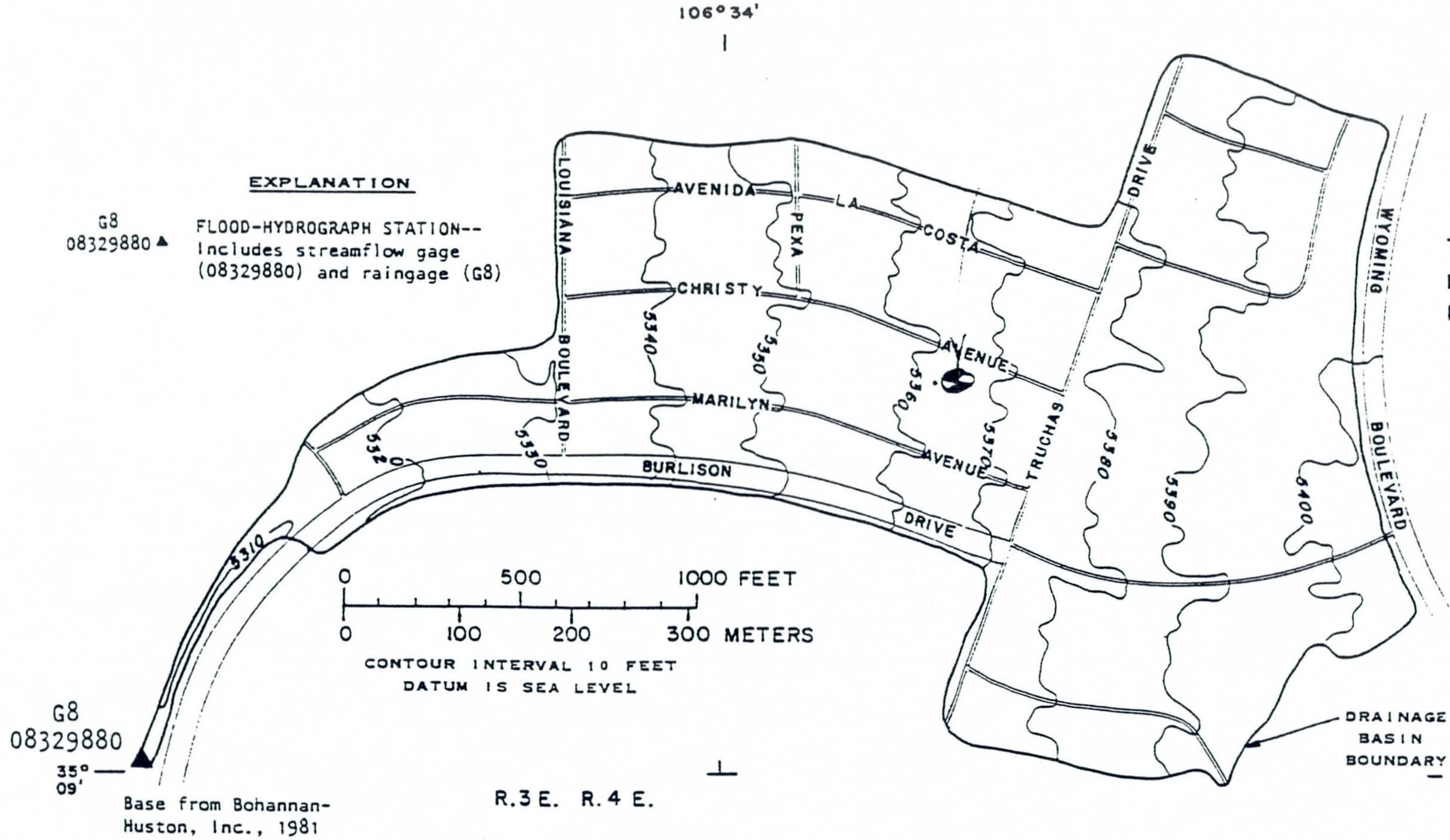
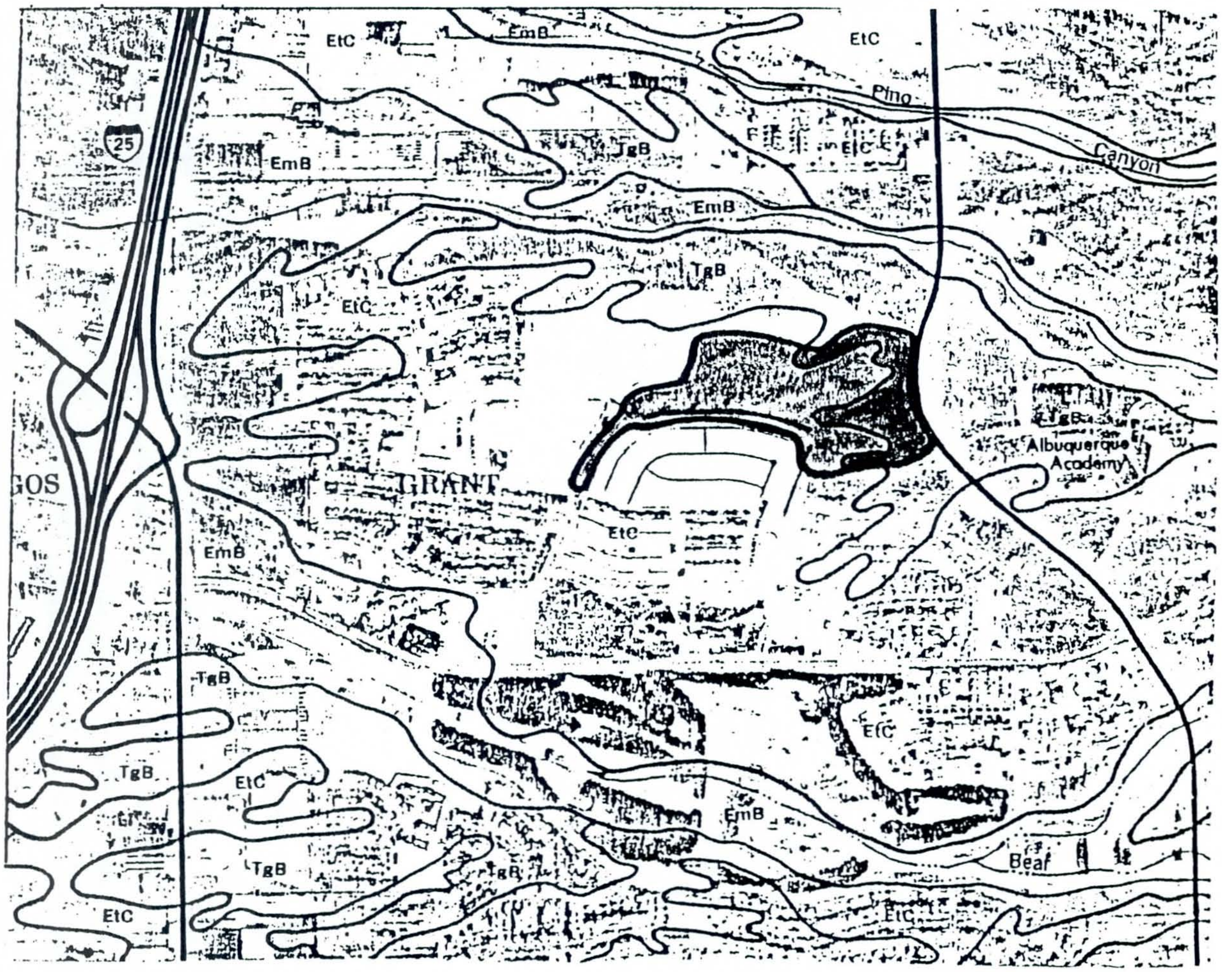


Figure 4.--Drainage basin for Academy Acres Drain (08329880).



Scale: 1 inch = 2,000 feet

Figure 3.- SCS soil classification for the Academy Acres drainage basin (SCS, 1977).

1

*** O U T P U T   D A T A ***

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR ACADEMY ACRES
PRIMARY ZONE NUMBER=   7
SHORT-DURATION ZONE NUMBER=   6

LATITUDE   35.15N   LONGITUDE 106.57W   ELEVATION   5306 FEET

POINT VALUES

RETURN PERIOD

DURATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.32	.41	.48	.57	.64	.71	.87	5-MIN
10-MIN	.50	.64	.74	.88	.99	1.10	1.36	10-MIN
15-MIN	.60	.78	.91	1.09	1.23	1.37	1.69	15-MIN
30-MIN	.76	1.01	1.18	1.42	1.60	1.79	2.21	30-MIN
1-HR	.92	1.23	1.45	1.74	1.97	2.20	2.74	1-HR
2-HR	.98	1.31	1.53	1.84	2.08	2.32	2.88	2-HR
3-HR	1.03	1.36	1.59	1.91	2.16	2.40	2.97	3-HR
6-HR	1.11	1.46	1.70	2.03	2.29	2.55	3.15	6-HR
12-HR	1.18	1.55	1.80	2.16	2.44	2.71	3.34	12-HR
24-HR	1.25	1.64	1.91	2.29	2.58	2.87	3.54	24-HR

* IF YOUR SITE IS IN ARIZONA OR NEW MEXICO, PLEASE CONSULT THE
FOLLOWING PAPER FOR REVISED DEPTH-AREA VALUES:
DEPTH-AREA RATIOS IN THE SEMI-ARID SOUTHWEST UNITED STATES
NOAA TECHNICAL MEMORANDUM NWS HYDRO-40
ZEHR AND MYERS
AUGUST 1984

INPUT DATA

PROJECT NAME=ACADEMY ACRES
✓ZONE= 7   SHORT-DURATION ZONE= 6✓
✓LATITUDE= 35.15   LONGITUDE= 106.57   ELEVATION= 5306
✓2-YR, 6-HR PCPN= 1.11   100-YR, 6-HR PCPN= 2.55
✓2-YR, 24-HR PCPN= 1.25   100-YR, 24-HR PCPN= 2.87

* * * * E N D   O F   R U N * * * *

George V. Sabol Consulting Engineers, Inc.

SHEET ____ OF ____

CLIENT FCDMC

DATE 19 Dec 91

PROJECT Testing

BY GVS

SUBJECT Academy Acres, Alb, NM

PROJECT NO. S-5F

Watershed Characteristics:

$A = .124$ sq. mi. (single-family residential)

$L = .9$ mi

$L_{ca} = .52$ mi

$S = 105$ ft/mi

$K_b = -.00625 \log(640 * .124) + .04$

$= .028$

Green and Ampt Parameters:

Map Symbol (1)	Soil Name (2)	Texture (3)	XKSAT in/hr (4)	% Area (5)
EtC	Embudo-Tijeras	gravelly sandy loam	.40	80
TgB	Tijeras	sandy loam	.40	20

$\overline{XKSAT} = .40$ in/hr

PSIF = 4.3 inches

DTHETA = .25

RTIMP = 28 %

IA = .20 inch

Table 7.--Station description, daily mean discharge values, monthly rainfall totals, and selected rainfall-discharge unit-value data for ACADEMY ACRES DRAIN AT ALBUQUERQUE (08329880), 1976-83.

STATION DESCRIPTION

LOCATION.--Lat 35 deg 09 min 02 sec, long 106 deg 34 min 18 sec, in NE1/4 SE1/4 sec.25, T.11 N., R.3 E., Bernalillo County, Hydrologic Unit 13020203, on left bank of concrete lined channel, at intersection of Burlison Dr and Leander Ave, 230 ft (70 m) north of intersection of Esther Ave and Burlison Dr and 0.4 mi (0.6 km) north of Academy Rd, in Albuquerque.

DRAINAGE AREA.--0.124 sq mi (0.321 sq km).

PERIOD OF RECORD.--June 1976 to December 1978, May 1979 to current year (no winter records).

GAGE.--Water-stage recorder and V-notch sharp-crested weir. Prior to May 1, 1978, concrete trapezoidal weir. Altitude of gage is 5,306 ft (1,617.27 m), from topographic orthophoto map.

REMARKS.--Records good except those for June 1976 to October 1979, which are poor. Recording rain gage at station. Additional recording rain gage near upstream end of basin since September, 1981. Basin drains residential area.

STAGE-DISCHARGE RELATION.--Rating developed on basis of weir-flow computations and discharge measurements at discharges of 0.10 cu ft per sec (0.003 cu m per sec), 0.20 cu ft per sec (0.006 cu m per sec), 0.35 cu ft per sec (0.01 cu m per sec), 1.00 cu ft per sec (0.03 cu m per sec), 2.50 cu ft per sec (0.07 cu m per sec) and 5.0 cu 5.0 cu ft per sec (0.14 cu m per sec) June 1976 to May 1978. Rating developed on basis of V-notch sharp-crested weir computation and discharge measurements at discharges of 0.10 cu ft per sec (0.003 cu m per sec), 0.60 cu ft per sec (0.02 cu m per sec), 1.85 cu ft per sec (0.05 cu m per sec), 2.08 cu ft per sec (0.06 cu m per sec) and slope-area measurement at discharge of 100 cu ft per sec (2.83 cu m per sec).

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 103 cu ft per sec (2.92 cu m per sec) Aug. 3, 1978, gage height, 4.09 ft (1.247 m) from rating curve extended above 2.0 cu ft per sec (0.57 cu m per sec) on basis of slope-area measurement of peak flow; no flow most of the time.

EXTREMES.--Maximum discharge during period June to December 1976, 20 cu ft per sec (0.55 cu m per sec) July 31, gage height, 0.68 ft (0.207 m); no flow most of the time.

Calendar year 1977: Maximum discharge, 15 cu ft per sec (0.42 cu m per sec) Sept. 5, gage height, 0.58 ft (0.177 m); no flow most of the time.

Calendar year 1978: Maximum discharge, 103 cu ft per sec (2.92 cu m per sec) Aug. 3, gage height, 4.09 ft (1.247 m) from rating curve extended above 2.0 cu ft per sec (0.57 cu m per sec) on basis of slope-area measurement of peak flow; no flow most of the time.

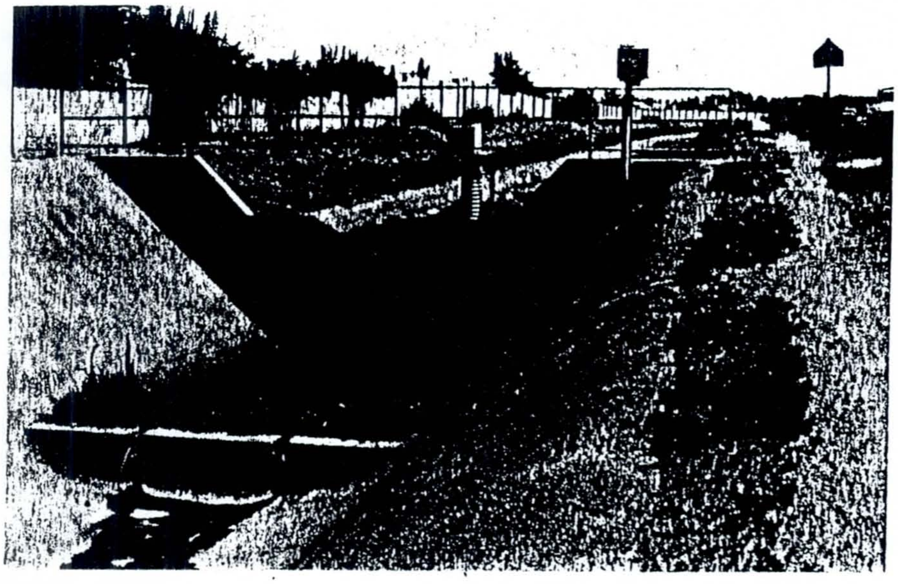
Calendar year 1979: Maximum discharge, 65 cu ft per sec (1.82 cu m per sec) July 16, gage height, 3.68 ft (1.122 m); no flow most of the time.

Calendar year 1980: Maximum discharge, 101 cu ft per sec (2.88 cu m per sec) Aug. 14, gage height, 4.07 ft (1.241 m); no flow most of the time.

Calendar year 1981: Maximum discharge, 59 cu ft per sec (1.67 cu m per sec) July 7, gage height, 3.61 ft (1.100 m); no flow most of the time.

Calendar year 1982: Maximum discharge, 37 cu ft per sec (1.05 cu m per sec) Aug. 12, gage height, 3.25 ft (0.991 m); no flow most of the time.

Calendar year 1983: maximum discharge, 39 cu ft per sec (1.10 cu m per sec) June 25, gage height, 3.29 ft (1.003 m); no flow most of the time.



Photograph 9.--Academy Acres Drain (08329880).

Frequency analysis for input data from file: ACADEMY.DAT
Station Name: ACADEMY ACRES, ALB, NM 0839988
Units: cfs
Regional Skew: None
14 data points used in the analysis

PLOTTING POSITIONS

Month	Day	Year	Magnitude	Gringorten	Cunnane	Weibull	Hazen
8	3	78	103.000	0.040	0.042	0.067	0.036
8	14	89	101.000	0.110	0.113	0.133	0.107
8	14	90	76.000	0.181	0.183	0.200	0.179
7	16	79	65.000	0.252	0.254	0.267	0.250
7	7	81	59.000	0.323	0.324	0.333	0.321
7	22	86	41.000	0.394	0.394	0.400	0.393
6	25	88	40.000	0.465	0.465	0.467	0.464
6	25	83	39.000	0.535	0.535	0.533	0.536
8	12	82	37.000	0.606	0.606	0.600	0.607
9	19	89	37.000	0.677	0.676	0.667	0.679
8	11	87	37.000	0.748	0.746	0.733	0.750
4	18	85	18.000	0.819	0.817	0.800	0.821
9	5	77	15.000	0.890	0.887	0.867	0.893
10	3	83	11.000	0.960	0.958	0.933	0.964

Arithmetic Mean:	48.50000
Standard Deviation:	29.00862
Skew Coefficient:	0.76421
Coefficient of Variation:	0.59812
Arithmetic Mean of Logs:	1.60464
Standard Deviation of Logs:	0.29100
Skew Coefficient of Logs:	-0.48671
Coefficient of Variation of Logs:	0.18135
Regional Skew Coefficient:	None
Weighted Skew Coefficient:	None

MAGNITUDES FOR SELECTED PROBABILITIES

Probability	Extreme Value	Log Extreme Value	Station LP III	Log Normal
0.99000	-2.48	12.39	5.24	7.16
0.90000	14.53	18.36	16.58	17.04
0.80000	23.32	22.49	23.42	22.89
0.50000	44.00	36.27	42.54	40.24
0.42920	49.20	40.90	47.77	45.31
0.20000	71.82	68.95	71.41	70.74
0.10000	90.24	105.52	90.89	95.00
0.04000	113.51	180.62	114.98	130.07
0.02000	130.77	269.13	132.36	159.35
0.01000	147.91	399.82	149.12	191.21
0.00400	170.47	673.31	165.22	226.08
0.00200	187.51	997.98	185.78	276.78

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CLIENT ______ SHEET ____ OF ____

PROJECT ______ DATE Revised 15 Oct 91

BY GVS

SUBJECT FIGURE 1-9 PROJECT NO. ______

Work Sheet for Log-Normal Confidence Limits

Watershed ACADEMY ACRES Albuquerque, N.M.

Station No. ______

$Q_{2\text{-}yr}$ 40 cfs

$Q_{100\text{-}yr}$ 190 cfs

Confidence Level (C.L.) 90 %

$a = \frac{100 - C.L.}{100}$ 0.10

$u_{1-a/2}$ 1.645

N 14

$\bar{Y} = \log_{10}(Q_{2\text{-}yr}) = \log_{10}(\ 40\ ) =$ 1.602

$S_y = \frac{\log_{10} Q_{100\text{-}yr} - \log_{10} Q_{2\text{-}yr}}{2.327} = \frac{\log_{10}(190) - \log_{10}(40)}{2.327}$ 0.2908

T years (1)	$u_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits (c) Upper (5)	Lower (6)
2	0.0	1.602	.0777	54	30
5	.842	1.847	.0905	99	50
10	1.282	1.975	0.1049	140	63
25	1.751	2.111	0.1237	206	81
50	2.052	2.199	0.1369	266	94
100	2.327	2.279	0.1496	335	108

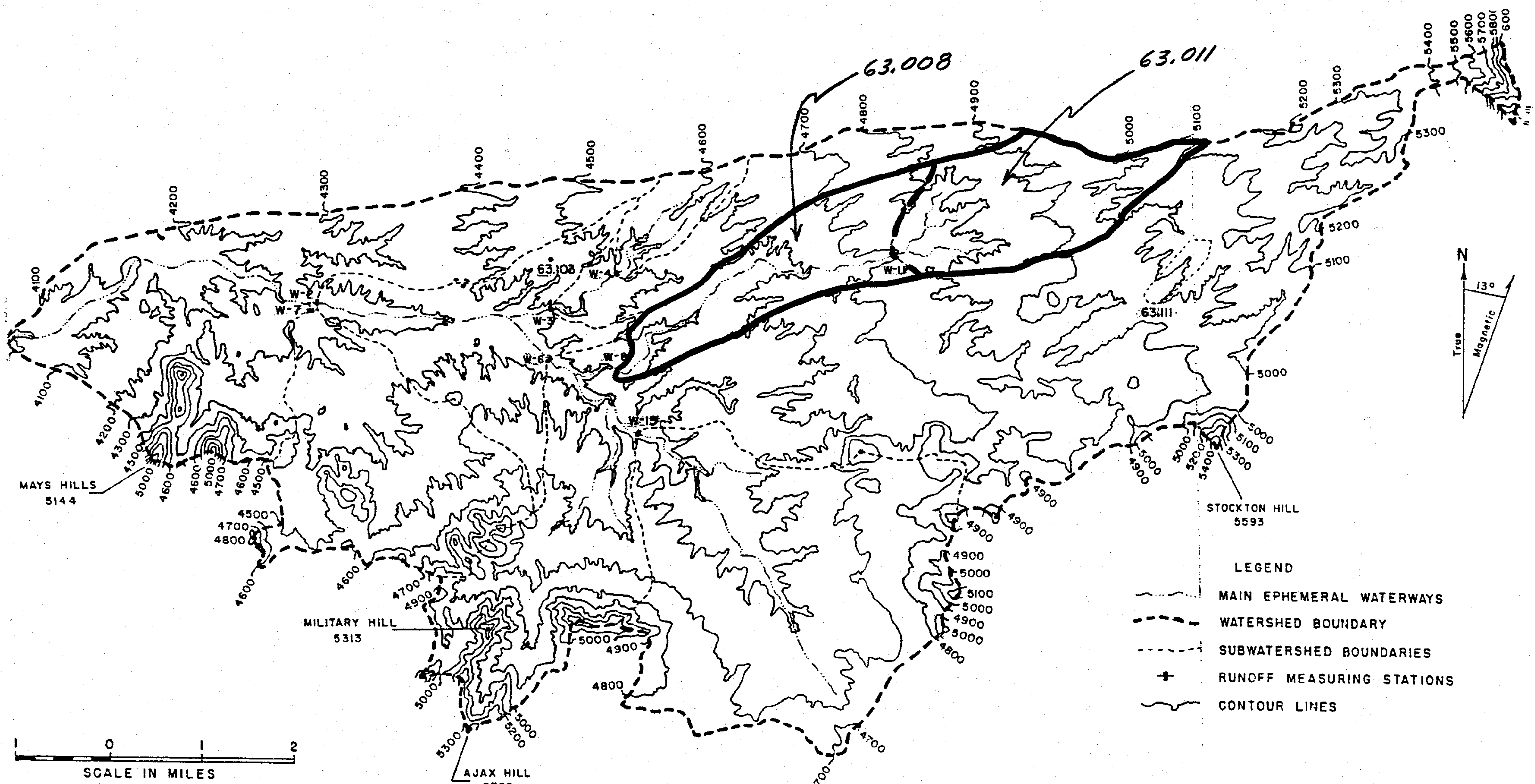
(a) $Y_T = \bar{Y} + u_{1-\frac{1}{T}} S_y$

(b) $S_T = \left[\left(S_y^2/N\right)\left(1 + .5\, u^2_{1-\frac{1}{T}}\right)\right]^{1/2}$

(c) $Q_L = 10^{\left(Y_T \pm u_{1-a/2} S_T\right)}$

Watershed and Streamgage Data for

Walnut Gulch 63.008



For additional cultural, topographic features and raingage network, see map page 63.1-5 in "Hydrologic Data for Experimental Agricultural Watersheds in the United States 1956-1959", Misc. Pub. No. 945, U.S.D.A., Agricultural Research Service, November 1963.

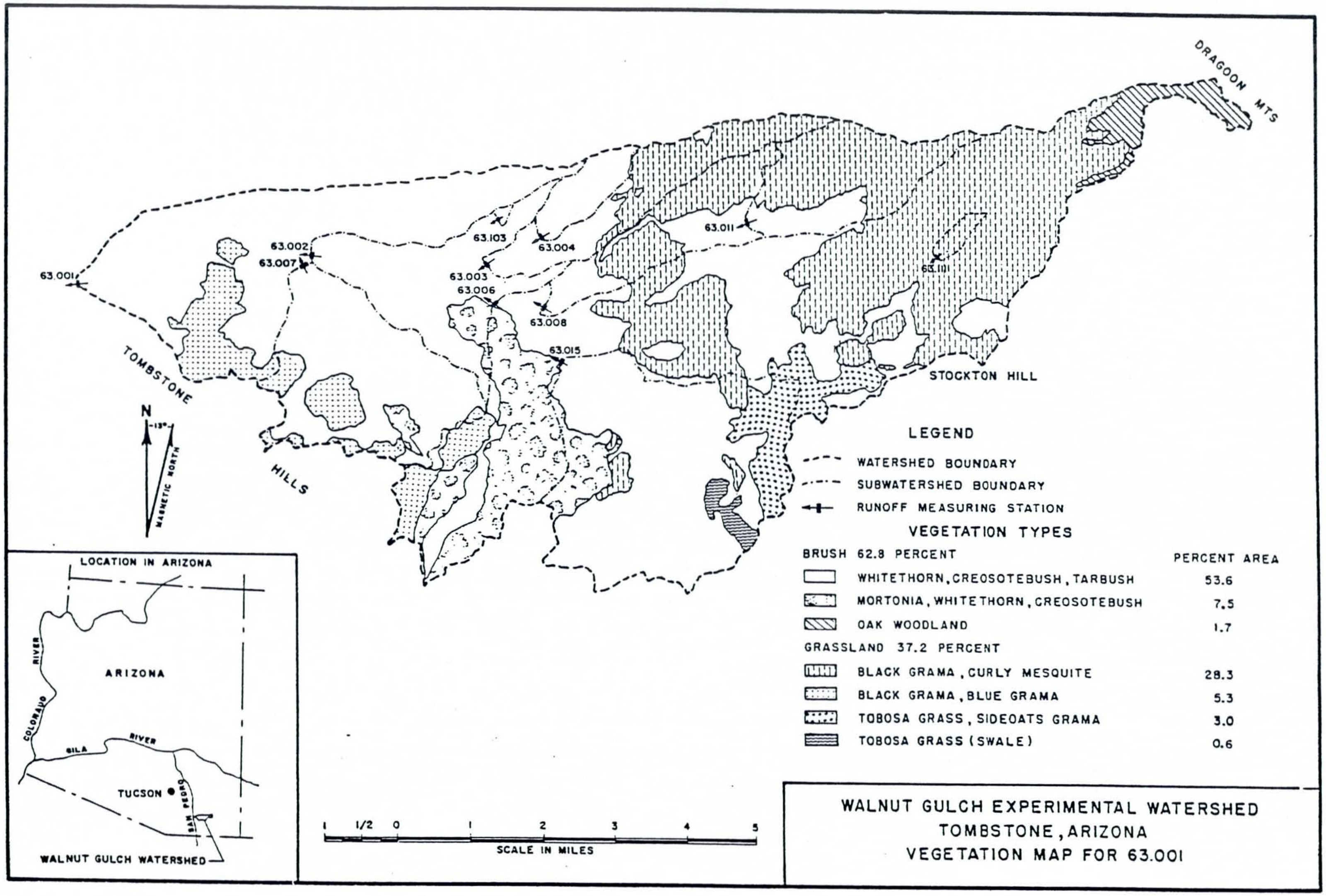
DRAINAGE AREAS

63.001	W-1	36,900 ACRES	57.7 SQUARE MILES
63.002	W-2	28,100 ACRES	43.9 SQUARE MILES
63.003	W-3	2,220 ACRES	3.47 SQUARE MILES
63.004	W-4	560 ACRES	0.88 SQUARE MILES
63.006	W-6	23,500 ACRES	36.7 SQUARE MILES
63.007	W-7	3,340 ACRES	5.22 SQUARE MILES
63.008	W-8	3,830 ACRES	5.98 SQUARE MILES
63.011	W-11	2,035 ACRES	3.18 SQUARE MILES
63.015	W-15	5,912 ACRES	9.24 SQUARE MILES
63.103		8.3 ACRES	
63.111		143 ACRES	

WALNUT GULCH EXPERIMENTAL WATERSHED

TOMBSTONE, ARIZONA

CONTOUR MAP



WALNUT GULCH EXPERIMENTAL WATERSHED
TOMBSTONE, ARIZONA
VEGETATION MAP FOR 63.001

1

*** O U T P U T   D A T A ***

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR WALNUT GULCH
PRIMARY ZONE NUMBER= 7
SHORT-DURATION ZONE NUMBER= 8

LATITUDE 31.60N   LONGITUDE 110.15W   ELEVATION 4400 FEET

POINT VALUES

	RETURN PERIOD							
DURATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.40	.47	.52	.60	.67	.73	.87	5-MIN
10-MIN	.59	.71	.80	.92	1.02	1.12	1.34	10-MIN
15-MIN	.72	.89	1.00	1.17	1.30	1.43	1.73	15-MIN
30-MIN	.96	1.19	1.35	1.58	1.76	1.94	2.36	30-MIN
1-HR	1.17	1.46	1.67	1.97	2.20	2.43	2.96	1-HR
2-HR	1.28	1.61	1.84	2.17	2.43	2.69	3.28	2-HR
3-HR	1.36	1.71	1.96	2.32	2.59	2.87	3.50	3-HR
6-HR	1.50	1.90	2.18	2.58	2.89	3.20	3.91	6-HR
12-HR	1.65	2.11	2.44	2.89	3.25	3.60	4.42	12-HR
24-HR	1.80	2.33	2.69	3.20	3.60	4.00	4.92	24-HR

* IF YOUR SITE IS IN ARIZONA OR NEW MEXICO, PLEASE CONSULT THE FOLLOWING PAPER FOR REVISED DEPTH-AREA VALUES:
DEPTH-AREA RATIOS IN THE SEMI-ARID SOUTHWEST UNITED STATES
NOAA TECHNICAL MEMORANDUM NWS HYDRO-40
ZEHR AND MYERS
AUGUST 1984

INPUT DATA

PROJECT NAME=WALNUT GULCH
ZONE= 7   SHORT-DURATION ZONE= 8
LATITUDE= 31.60   LONGITUDE= 110.15   ELEVATION= 4400
2-YR, 6-HR PCPN= 1.50   100-YR, 6-HR PCPN= 3.20
2-YR, 24-HR PCPN= 1.80   100-YR, 24-HR PCPN= 4.00

* * * * E N D   O F   R U N * * * *

Rainfall Intensities for Southeastern Arizona

By Herbert B. Osborn, Member, ASCE[1] and
Kenneth G. Renard, Fellow, ASCE[2]

Introduction

Small watershed storm runoff in the southwestern United States is dominated by intense, short-duration convective rains of limited areal extent. Storm drainage design is often based on rainfall information published by the National Weather Service in the National Oceanic and Atmospheric Administration (NOAA) Atlas 2 series (Miller et al. 1973). In NOAA Atlas 2, short-duration rainfall is derived by an extrapolation procedure from maps of 6-hr and 24-hr rainfall amounts with different frequencies. In this study, intensity-duration-frequency values for 1 hr and less, based on data from a dense network of rain gauges in southeastern Arizona, are compared to similar values derived from NOAA Atlas 2. Differences in rainfall intensities obtained from the two methods are illustrated by simulating and comparing peaks and volumes of runoff.

Previous Studies

In 1973, The National Weather Service (Miller et al. 1973) published an 11-volume atlas for rainfall in the 11 western states. Equations are provided in the publication to estimate 1-hr rainfall from 6-hr and 24-hr rainfall maps for different frequencies. Ratios published in Technical Paper No. 40 (Hershfield 1961) are used to estimate rainfall for 5-, 10-, 15-, and 30-min durations from the 1-hr estimates. Reich (1978) showed the value of computers in developing intensity-duration-frequency relationships from NOAA Atlas 2, but he also warned that computer output was no better than the data from which the estimates were made. Most recently, Petersen (1986) found that estimates for short-duration intensities based on recording rain gauge records near Billings, Montana, were significantly larger for recurrence intervals from 2–100 yrs, than those based on NOAA Atlas 2.

Rainfall Analysis

Data from a dense-recording rain gauge network on the U. S. Dept. of Agric., Agricultural Research Service's Walnut Gulch experimental watershed, in southeastern Arizona, were used to estimate 2-, 5-, 10-, 25-, 50-, and 100-yr rains for 5-, 10-, 15-, 30-, and 60-min durations. Based on the assumption of independent sampling points for well-separated rain gauges (Reich and Osborn 1982) and the station-year method (Hafstad 1942), three sets of four gauges each were selected to create records of 90, 91, and 92

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Note. Discussion open until July 1, 1988. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on March 19, 1987. This paper is part of the *Journal of Irrigation and Drainage Engineering*, Vol. 114, No. 1, February, 1988. ©ASCE, ISSN 0733-9437/88/0001-0195/$1.00 + $.15 per page. Paper No. 22230.

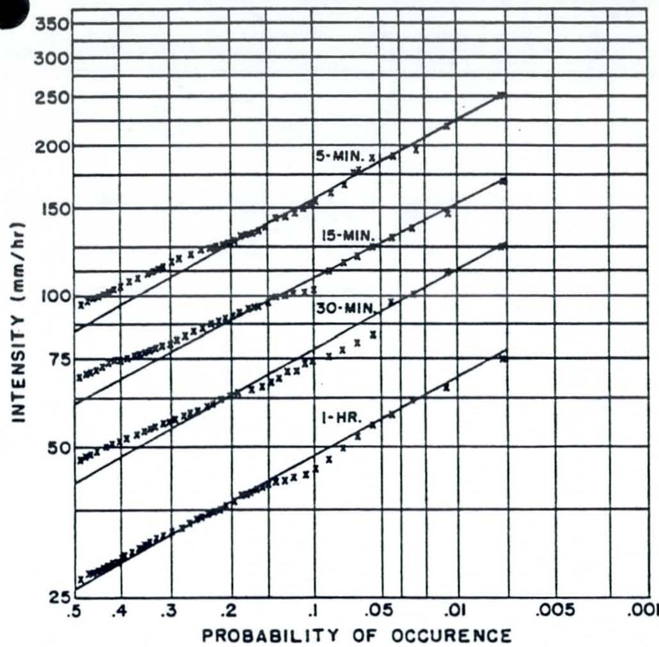

FIG. 1. Intensity-Frequency Relationships for 5-, 15-, 30-min, and 1-hr Durations for Walnut Gulch, Arizona

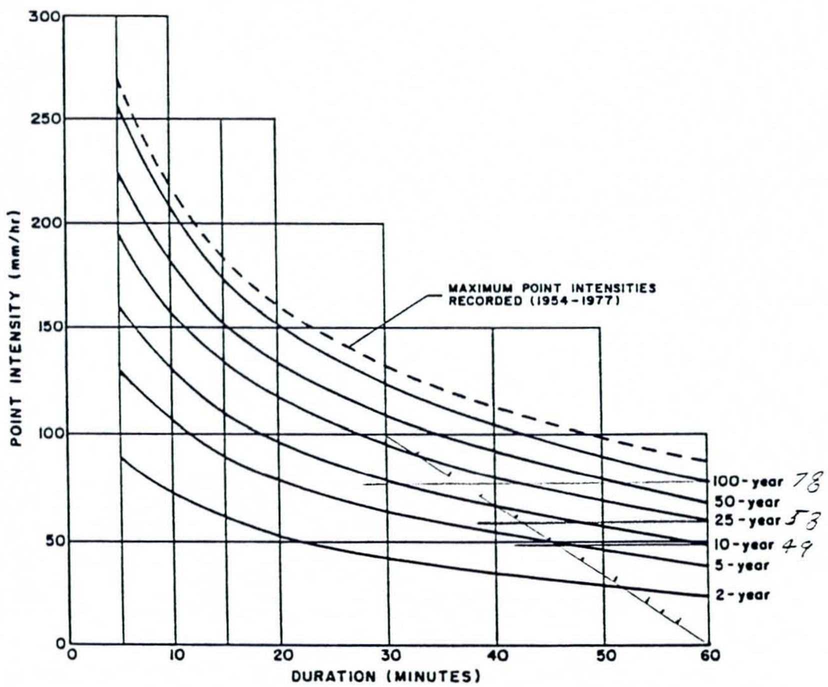

FIG. 2. Point Rainfall Intensities for Durations of 5–60 min.

TABLE 1. Point Depth, Duration, Frequency Rainfall for South[illegible]ern Arizona, Annual Series

Return Period and Duration (min) (1)	Walnut Gulch Data (mm) (2)	inches	National Oceanic and Atmospheric Administration Atlas (mm) (3)	inches
	(*a*) 2 Years			
5	7.0		7.6	
10	11.4		12.0	
15	14.6		15.2	
30	21.0		21.6	
	(*b*) 5 Years			
5	10.2		10.2	
10	17.8		16.5	
15	22.8		21.6	
30	31.8		29.2	
	(*c*) 10 Years			
5	12.0	.47	12.0	.47
10	21.6	.85	20.3	.80
15	28.0	1.10	25.4	1.00
30	38.0	1.50	34.3	1.35
60	49.0	1.93		1.67
	(*d*) 25 Years			
5	16.5	.65	15.2	.60
10	25.4	1.00	22.8	.90
15	34.3	1.35	29.2	1.15
30	48.2	1.90	39.4	1.55
60	58.0	2.28		1.97
	(*e*) 50 Years			
5	19.0		16.5	
10	30.5		25.4	
15	38.0		31.8	
30	55.8		44.4	
	(*f*) 100 Years			
5	21.6	.85	18.0	.71
10	35.6	1.40	28.0	1.10
15	44.4	1.75	35.6	1.40
30	63.5	2.50	48.0	1.89
60	76.0	3.07		2.43

yrs. Twelve different gauges made up the three sets, and the four gauges in each set were separated by at least four mi. Estimates for 5-min, 15-min, 30-min, and 1-hr rainfall were plotted on log probability paper (Fig. 1). The relationships from Fig. 1 were used to derive intensity-duration-frequency curves (Fig. 2). The 6-hr and 24-hr rainfall maps in NOAA Atlas 2, Vol. 8 (Arizona) were used, along with the appropriate equations and ratios, to derive depths for 2-, 5-, 10-, 25-, 50-, and 100-yr rainfall for 5-, 10-, 15-, 30-,

TABLE 2. Comparison of Runoff Peaks and Volumes for a 100-yr, 1-hr Storm Based on Walnut Gulch and NOAA Atlas 2 Estimates

		Peak (cms/ha)				Volume (mm)			
		"Dry"		"Wet"		"Dry"		"Wet"	
Watershed (1)	Area (ha) (2)	Walnut Gulch (3)	National Oceanic and Atmospheric Administration (4)	Walnut Gulch (5)	National Oceanic and Atmospheric Administration (6)	Walnut Gulch (7)	National Oceanic and Atmospheric Administration (8)	Walnut Gulch (9)	National Oceanic and Atmospheric Administration (10)
63105	0.24	0.50	0.35	0.54	0.41	50.0	33.5	62.5	45.7
63011	810	0.12	0.06	0.17	0.12	30.5	15.2	42.0	26.7

and 60-min durations (Table 1). Rainfall depths for the same return periods and durations were derived from Walnut Gulch data and compared to the NOAA Atlas 2 estimate (Table 1). The differences appeared appreciable for the less frequent events, particularly the 50- and the 100-yr storms, as opposed to the Peterson (1986) study in which the difference were appreciable for all recurrence intervals.

Runoff

One method of illustrating the significance of differing estimates of short-duration rainfall intensities is to study the differences in flood peaks and volumes when the rainfall estimates are entered into a mathematical rainfall-runoff model. A kinematic cascade rainfall-runoff model, KINEROS (Rovey et al. 1977), which has been adapted for use on Walnut Gulch (Osborn 1984), was used in this evaluation. KINEROS is a well-tested nonlinear, deterministic, distributed parameter model. Inputs are: (1) Hyetographs of actual or simulated rainfall; (2) the watershed surface geometry and topography; (3) parameters for surface roughness; (4) infiltration parameters (based on Green-Ampt); and (5) the channel network, including slope, cross-sectional area, cross-sectional shape, hydraulic roughness, and a subroutine for channel abstraction (Smith 1981; Osborn 1984). Data from two natural rangeland watersheds were used to validate the model—a very small (0.24 ha, 0.6 ac) watershed, and a large (810 ha, 2,000 ac) watershed. Rainfall was assumed to cover the 0.24 ha watershed evenly, but was varied both in time and space over the 810 ha watershed using an elliptical model based on earlier modeling efforts (Osborn and Laursen 1973). Runoff peaks and volumes were obtained for "wet" and "dry" antecedent conditions for the 100-yr, 1-hr rain (Table 2). "Dry" antecedent conditions normally prevail in southeastern Arizona, but "wet" antecedent conditions, often assumed in engineering design, occur occasionally. Runoff peak and volume estimates based on Walnut Gulch rainfall data were substantially greater than those based on the NOAA Atlas estimates for all durations for the 50- and 100-yr storms, somewhat greater for the 25-yr storms, and substantially the same for the more frequent events.

Conclusions

For southeastern Arizona, estimates of short-duration precipitation intensities, based on NOAA Atlas 2, were substantially lower than estimates based on data from a dense rain gauges network for less frequent events (50- and 100-year frequencies). Runoff peaks and volumes, as estimated with a distributed mathematical rainfall-runoff model, were underestimated for the less frequent events, particularly for the 100-yr storm, based on NOAA Atlas 2.

Appendix I. References

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Reich, B. M., and Osborn, H. B. (1982). "Improving point rainfall prediction with experimental data." *Int. Statistical Analysis of Rainfall and Runoff.* Int'l. Symp. on Rainfall/Runoff, Mississippi State Univ., Water Res. Publications, P.O. Box 2841, Littleton, CO, 41–54.

Rovey, E. W., Woolhiser, D. A., and Smith, R. E. (1977). "A distributed kinematic model of upland watersheds." *Hydrology Paper 93*, Colorado State University, Ft. Collins, CO.

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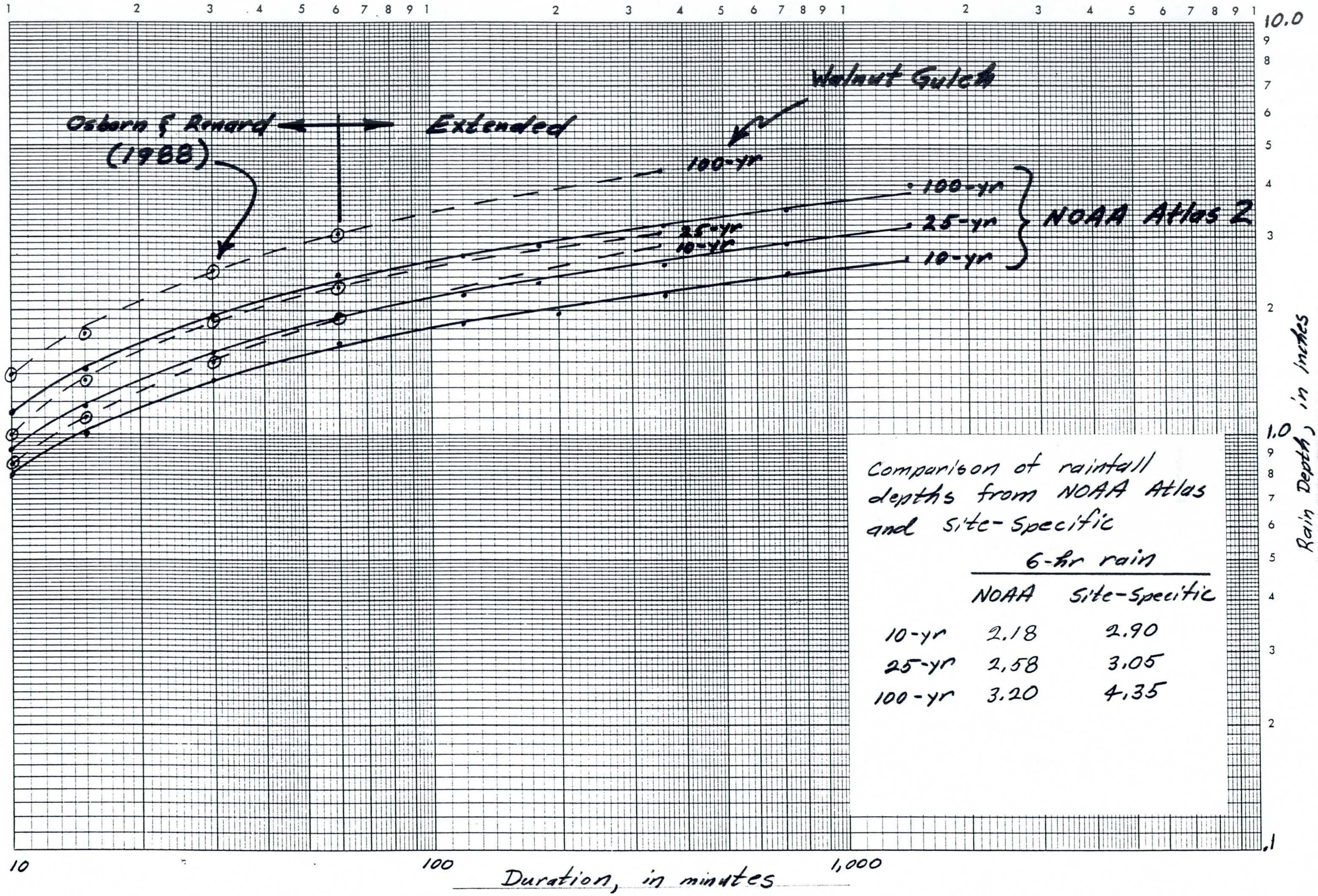
25 oct 91



Comparison of rainfall depths from NOAA Atlas and Site-Specific

	6-hr rain	
	NOAA	Site-Specific
10-yr	2.18	2.90
25-yr	2.58	3.05
100-yr	3.20	4.35

TOMBSTONE, ARIZONA   WALNUT GULCH WATERSHED 63.008

LOCATION: Cochise County, Ariz., 1½ miles northeast of Tombstone; Walnut Gulch, San Pedro River, Gila River, Colorado River Basin.

AREA: 3,830 acres (5.98 sq. miles)

SLOPES:

Slope - Percent	0-3	3-10	10-20	20-35
Percent of area 1/	4	56	28	12

1/ Estimated

SOILS: Not available

EROSION:

Erosion Class	1	2
Percent of area	98	2

LAND CAPABILITY:

Class	VI
Percent of area	100

GEOLOGY: One hundred percent of the subwatershed consists of Quaternary and Tertiary alluvium of the Tombstone pediment. The alluvium is made up of permeable lensed and interbedded sand, gravel, conglomerate, caliche conglomerate, and some clay. Two series of conglomerate are recognized beneath the recent alluvium of the Tombstone pediment. A younger conglomerate whose bedding is nearly conformable to the pediment surface and probably considerably older than that surface, and an older Tertiary conglomerate lying unconformably beneath that. These conglomerates are known to persist to depths exceeding 1,200 feet. Topographic expression of the alluvium is that of low undulating hills dissected by present stream channels. Caliche conglomerates of the unit are fairly resistant to erosion and form steep cliffs of low relief in some of the present stream channels. The southeast tip and fluvial outlet of the watershed is underlain by the remnant of a highly fractured intrusive basalt plub. The regional watertable is about 425 feet deep.

Stratigraphy and Hydrogeology of Walnut Gulch Subwatershed 63.008

System	Formation and percent of area		Description
Quaternary & Tertiary	Recent alluvium	99%	Gravel, sand, and clay.
	Younger conglomerate	< 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.
	Older conglomerate	< 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.
	Basalt	< 1%	Intrusive olivine basalt plug, secondary calcite vein filling.

Source of data: General Geology of Central Cochise County, Arizona, by James Gilluly, U. S. Geological Survey, Professional Paper 281, 1956, and extended field studies by project staff.

SURFACE DRAINAGE: Good, length of principal waterway is 8.0 miles with 2 major tributaries; a natural watershed with surface flow in well defined water courses; includes gaged watershed 63.011.

CHARACTER OF FLOW: Ephemeral

INSTRUMENTATION: Precipitation: Measured by 11 24-hour weighing rain gages. Runoff: Critical depth flume (precalibrated), AD-35 analog strip chart water level recorder.

WATERSHED CONDITIONS: (Includes Watershed 63.011) Vegetation cover: Approximately one-third of the area is dominated by desert shrubs (whitethorn, creosotebush, tarbush) with a crown spread of approximately 30 percent and an understory of grasses with less than 1 percent basal area. The remaining two-thirds of the area is dominated by grasses (black grama, curly mesquite grass, sideoats grama), with a basal area of about 2.5 percent, interspersed by desert shrubs with a crown spread of about 5 percent.

GENERALLY REPRESENTS: Desert grassland ranges in the Southeastern Arizona Basin and Rangeland Resources Area (D-41).

Determination of Green and Ampt parameters

Map Unit (1)	Soil Name (2)	Soil Texture (3)	% Soil in Map Unit (4)	XKSAT in/hr (5)
HbC	Hathaway	loam	70	.25
	Bernardino	clay loam	25	.04
	Misc.	sandy loam	5	.40
HnC	Hathaway	loam	60	.25
	Nickel	loam	35	.25
	Kimbaugh	loam	5	.25
HbB	Hathaway	loam	60	.25
	Bernardino	clay loam	30	.04
	Nickel	loam	5	.25
	Sonoita	sandy loam	5	.40
BhC	Bernardino	clay loam	60	.04
	Hathaway	loam	35	.25
	Misc.	loamy sand	5	1.2
BeB	Bernardino	clay loam	95	.04
	Misc.	sandy loam	5	.40
RcB	Rillito	loam	60	.25
	Cave	loam	35	.25
	Misc.	loam	5	.25
RlB	Rillito	loam	70	.25
	Laveen	sandy loam	25	.40
	Misc.	sandy loam	5	.40
RlC	Rillito	loam	75	.25
	Laveen	sandy loam	20	.40
	Misc.	sandy loam	5	.40
BhB	Bernardino	clay loam	70	.04
	Hathaway	loam	30	.25
Lo	loamy alluvial	sandy loam	100	.40
Co	Corona	sandy loam	100	.40
So	Sonoita	sandy loam	100	.40

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SHEET ____ OF ____
DATE 2 Jan 92
BY GVS
PROJECT NO. 5-5

Map Unit Green and Ampt parameters (WG8 & WG11)

Map Unit (1)	Calculation (2)	$XKSAT_{mu}$ (3)
HbC	$\text{antilog}[(\log .25).70 + (\log .04).25 + (\log .40).05]$	.16
HnC	(100% loam)	.25
HbB	$\text{antilog}[(\log .25).65 + (\log .04).30 + (\log .40).05]$	.15
BhC	$\text{antilog}[(\log .04).60 + (\log .25).35 + (\log 1.2).05]$	.09
BeB	$\text{antilog}[(\log .04).95 + (\log .40).05]$	.04
RcB	(100% loam)	.25
RlB	$\text{antilog}[(\log .25).70 + (\log .40).30]$	.29
RlC	$\text{antilog}[(\log .25).75 + (\log .40).25]$	.28
BhB	$\text{antilog}[(\log .04).70 + (\log .25).30]$	.07
Lo	(100% sandy loam)	.40
CO	(100% sandy loam)	.40
So	(100% sandy loam)	.40

log area-averaged Green and Ampt parameters for WG 8

Soil Name (1)	Area sq. mi. (2)	% Area (3)	$XKSAT_{mu}$ (4)
HbC	2.96	49	.16
HnC	1.11	19	.25
So, Co, Lo	.55	9	.40
HbB	.57	10	.15
RIC	.30	5	.28
BhC	.24	4	.09
BhB	.10	2	.07
RcB	.08	1	.25
BeB	.07	1	.04
	5.98	100	

$$\overline{XKSAT} = \text{antilog}\Big[(\log.16).49 + (\log.25).19 + (\log.40).09 + (\log.15).10 + (\log.28).05 + (\log.09).04 + (\log.07).02 + (\log.25).01 + (\log.04).01\Big]$$

$\overline{XKSAT}$ = .18 in/hr

PSIF = 5.8 inches

DTHETA (dry) = .39

Correction of $\overline{XKSAT}$ for vegetation cover ($V_c = 30\%$)

$$C_k = \left(\frac{V_c - 10}{90}\right) + 1.0$$

$$C_k = 1.22$$

$$XKSAT = C_k * \overline{XKSAT}$$

$$= (1.22)(.18) = .22 \text{ in/hr}$$

IA = .35 inch

RTIMP = 0 %

Calculation of Lag for WG8

$A = 5.98$ sq.mi.
$L = 8.0$ miles
$L_{ca} = 3.6$ miles
$S = 75$ ft/mile

USBR Lag with $K_n = 0.03$

$$Lag = 26 K_n \left(\frac{L \cdot L_{ca}}{S^{1/2}} \right)^{.33}$$

$$= 26(.03)\left(\frac{8.0 * 3.6}{75^{1/2}} \right)^{.33}$$

$= 1.16$ hrs

Frequency analysis for input data from file: 63008.DAT
Station Name: WALNUT GULCH 63.008
Units: CFS
Regional Skew: None
27 data points used in the analysis

PLOTTING POSITIONS

Month	Day	Year	Magnitude	Gringorten	Cunnane	Weibull	Hazen
7	12	64	4053.300	0.021	0.022	0.036	0.019
8	27	82	3392.100	0.058	0.059	0.071	0.056
7	12	75	2297.600	0.094	0.096	0.107	0.093
7	7	67	1797.900	0.131	0.132	0.143	0.130
1	22	77	1787.600	0.168	0.169	0.179	0.167
7	23	71	1280.800	0.205	0.206	0.214	0.204
7	14	65	1022.800	0.242	0.243	0.250	0.241
8	1	78	951.000	0.279	0.279	0.286	0.278
9	20	83	932.500	0.316	0.316	0.321	0.315
8	10	86	929.200	0.353	0.353	0.357	0.352
3	6	70	743.200	0.389	0.390	0.393	0.389
7	31	63	740.900	0.426	0.426	0.429	0.426
7	20	66	671.000	0.463	0.463	0.464	0.463
7	27	76	641.400	0.500	0.500	0.500	0.500
8	4	80	628.100	0.537	0.537	0.536	0.537
7	2	68	569.200	0.574	0.574	0.571	0.574
8	22	87	529.400	0.611	0.610	0.607	0.611
7	12	73	380.700	0.647	0.647	0.643	0.648
8	16	84	346.300	0.684	0.684	0.679	0.685
7	19	74	214.200	0.721	0.721	0.714	0.722
7	31	81	201.900	0.758	0.757	0.750	0.759
6	6	72	191.600	0.795	0.794	0.786	0.796
7	17	69	167.200	0.832	0.831	0.821	0.833
8	16	89	157.100	0.869	0.868	0.857	0.870
8	20	88	152.100	0.906	0.904	0.893	0.907
7	17	85	140.100	0.942	0.941	0.929	0.944
8	18	79	26.000	0.979	0.978	0.964	0.981

Arithmetic Mean:	923.89630
Standard Deviation:	987.72526
Skew Coefficient:	1.95800
Coefficient of Variation:	1.06909
Arithmetic Mean of Logs:	2.73678
Standard Deviation of Logs:	0.49318
Skew Coefficient of Logs:	-0.49692
Coefficient of Variation of Logs:	0.18020

MAGNITUDES FOR SELECTED PROBABILITIES

Probability	Extreme Value	Log Extreme Value	Station LP III	Log Normal
0.99000	-765.82	78.10	17.06	29.26
0.90000	-205.10	148.96	121.42	127.21
0.80000	84.63	207.84	217.91	209.66
0.50000	766.08	454.96	599.39	545.48
0.42920	937.60	554.14	729.51	666.91
0.20000	1682.96	1305.52	1441.91	1419.16
0.10000	2290.02	2623.57	2170.11	2339.01
0.04000	3057.04	6336.82	3232.87	3984.12
0.02000	3626.06	12189.51	4103.51	5620.38
0.01000	4190.87	23334.78	5022.75	7654.39
0.00400	4934.54	54868.81	5975.82	10167.30
0.00200	5496.08	104641.18	7289.61	14326.69

GEORGE V. SABOL CONSULTING ENGINEERS, INC.

SHEET ____ OF ____

CLIENT ____

DATE *Revised 15 Oct 91*

PROJECT ____

BY *GVS*

SUBJECT *FIGURE 1-9*

PROJECT NO. ____

Work Sheet for Log-Normal Confidence Limits

Watershed Walnut Gulch 63.008

Station No. ____

$Q_{2\text{-}yr}$ 550 cfs

$Q_{100\text{-}yr}$ 6200 cfs

Confidence Level (C.L.) 90 %

$a = \frac{100 - C.L.}{100}$ 0.10

$u_{1-a/2}$ 1.645

N 27

$\bar{Y} = \log_{10}(Q_{2\text{-}yr}) = \log_{10}(550) =$ 2.74

$S_Y = \frac{\log_{10} Q_{100\text{-}yr} - \log_{10} Q_{2\text{-}yr}}{2.327} = \frac{\log_{10}(6200) - \log_{10}(550)}{2.327}$ 0.452

T years (1)	$u_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits (c) Upper (5)	Limits (c) Lower (6)
2	0.0	2.74	0.0869	764	395
5	.842	3.12	0.1012	1934	899
10	1.282	3.32	0.1174	3259	1339
25	1.751	3.53	0.1384	5724	2006
50	2.052	3.67	0.1532	8356	2618
100	2.327	3.79	0.1674	11624	3270

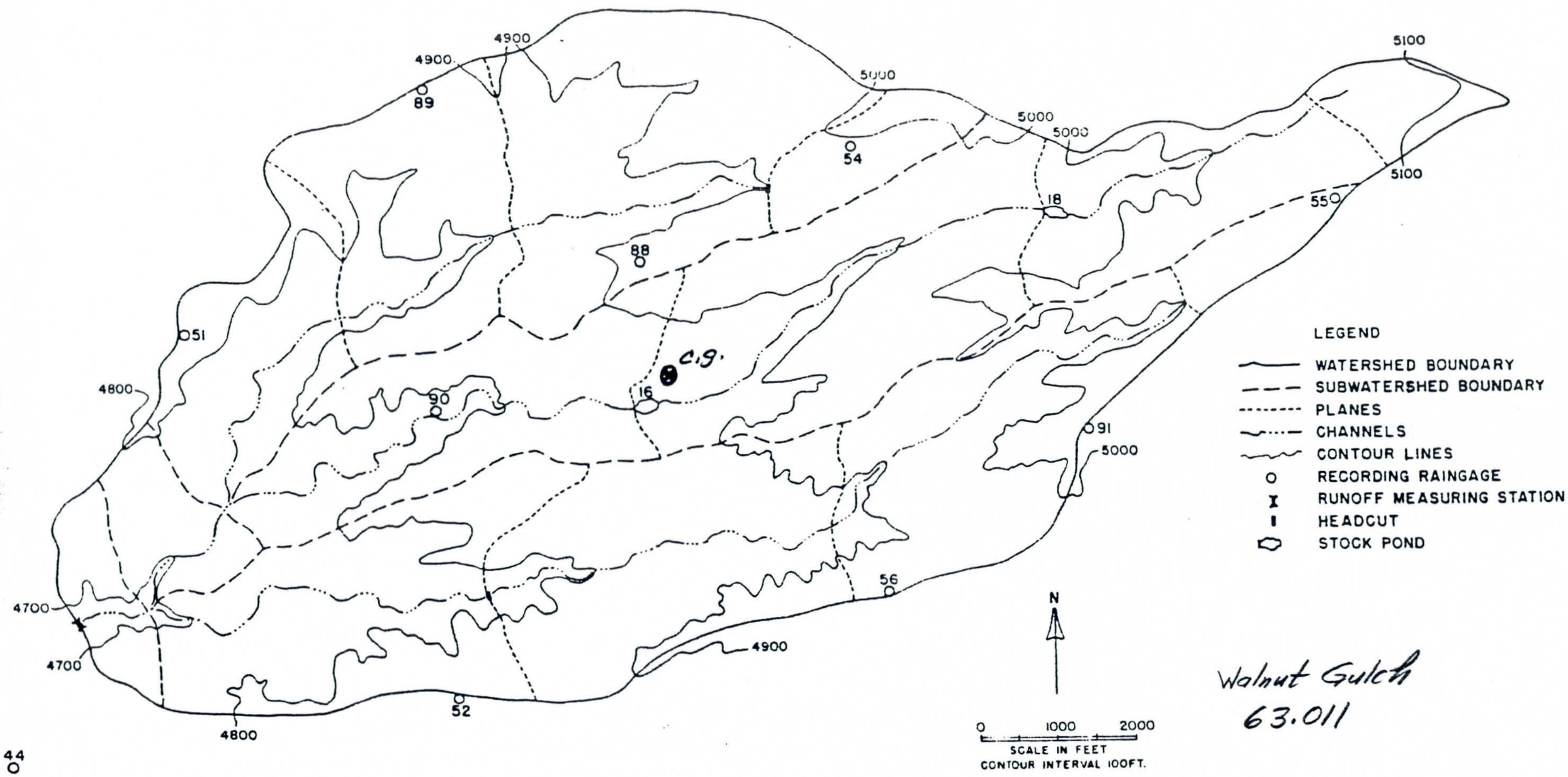
(a) $Y_T = \bar{Y} + u_{1-\frac{1}{T}} S_Y$

(b) $S_T = \left[(S_Y^2 / N)(1 + .5\, u^2_{1-\frac{1}{T}}) \right]^{1/2}$

(c) $Q_L = 10^{(Y_T \pm u_{1-a/2} S_T)}$

Watershed and Streamgage Data for

Walnut Gulch 63.011



Walnut Gulch
63.011

TOMBSTONE, ARIZONA   WALNUT GULCH WATERSHED 63.011

LOCATION: Cochise County, Ariz.; 4 1/3 miles northeast of Tombstone; Walnut Gulch, San Pedro River, Gila River, Colorado River Basin.

AREA: 2,035 acres (3.18 sq. miles)

SLOPES:

Slope - Percent	0-3	3-10	10-20	20-35
Percent of area 1/	4	52	28	16

1/ Estimated

SOILS: Not available

EROSION:

Erosion Class	1	2
Percent of area	98	2

LAND CAPABILITY:

Class	VI
Percent of area	100

GEOLOGY: One hundred percent of the subwatershed consists of Quaternary and Tertiary alluvium of the Tombstone pediment. The alluvium is made up of permeable lensed and interbedded sand, gravel, conglomerate, caliche conglomerate, and some clay. Two series of conglomerate are recognized beneath the recent alluvium of the Tombstone pediment. A younger conglomerate whose bedding is nearly conformable to the pediment surface and probably older than that surface, and an older Tertiary conglomerate lying unconformably beneath that. These conglomerates are known to persist to depths exceeding 1,200 feet. Topographic expression of the alluvium is that of low undulating hills dissected by present stream channels. Caliche conglomerates of this unit are fairly resistant to erosion and form steep cliffs of low relief in some of the present stream channels. The regional watertable is about 425 feet deep.

Stratigraphy and Hydrogeology of Walnut Gulch Subwatershed 63.011

System	Formation and percent of area	Description
Quaternary & Tertiary	Recent alluvium 99%	Gravel, sand, and clay.
	Younger conglomerate < 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.
	Older conglomerate < 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.

Source of data: General Geology of Central Cochise County, Arizona, by James Gilluly, U. S. Geological Survey, Professional Paper 281, 1956 and extended field studies by project staff.

SURFACE DRAINAGE: Good, length of principal waterway is 4.0 miles with 2 major tributaries; a natural watershed with surface flow in well defined water courses.

CHARACTER OF FLOW: Ephemeral.

INSTRUMENTATION: Precipitation: Measured by 5 24-hour weighing rain gages. Runoff: Critical depth flume (precalibrated) AD-35 analog strip chart water level recorder.

WATERSHED CONDITIONS: Vegetation cover: Approximately 20 percent of the area dominated by desert shrubs (whitethorn, creosotebush, tarbush) with a crown spread of approximately 30 percent cover and an understory of grasses with basal area of less than 1 percent. The remaining 80 percent of the area supports a grass cover (black grama, curly mesquite grass, sideoats grama) with a basal cover of about 2.5 percent interspersed with desert shrubs averaging less than 5 percent crown cover.

GENERALLY REPRESENTS: Desert grassland ranges in the Southeastern Arizona Basin and Rangeland resources area (D-41).

(See worksheets for WG8 for calculation of Map Unit XKSAT values.)

log area-averaged Green and Ampt parameters for WG11

Soil Name (1)	Area sq. mi. (2)	% Area (3)	$XKSAT_{mu}$ (4)
HbC	2.23	70	.16
HnC	.92	29	.25
Lo	.03	1	.40
	3.18	100	

$\overline{XKSAT} = \text{antilog}\left[(\log .16).70 + (\log .25).29 + (\log .40).01\right]$

$\overline{XKSAT} = .18$ in/hr

PSIF = 5.8 inches

DTHETA (dry) = .39

Correction of $\overline{XKSAT}$ for vegetation cover ($V_c = 30\%$)

$$C_k = \left(\frac{V_c - 10}{90}\right) + 1.0$$

$$C_k = 1.22$$

$XKSAT = C_k * \overline{XKSAT}$

$= (1.22)(.18) = .22$ in/hr

IA = .35 inch

RTIMP = 0 %

Note: The Green and Ampt parameters are the same for watersheds WG8 and WG11.

GEORGE V. SABOL CONSULTING ENGINEERS, INC.

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PROJECT

SUBJECT

SHEET ____ OF ____

DATE 2 Jan 92

BY GVS

PROJECT NO. 5-5

Calculation of Lag for WG 11

$A = 3.18$ sq. mi.
$L = 4.0$ mi.
$L_{ca} = 1.8$ mi.
$S = 100$ ft/mi

USBR Lag with $K_n = 0.03$

$$Lag = 26\,K_n \left(\frac{L \cdot L_{ca}}{S^{1/2}}\right)^{.33}$$

$$= 26\,(.03)\left(\frac{4.0 * 1.8}{100^{1/2}}\right)^{.33}$$

$$= 0.7 \text{ hr}$$

Frequency analysis for input data from file: 63011.DAT
Station Name: WALNUT GULCH 63.011
Units: CFS
Regional Skew: None
27 data points used in the analysis

PLOTTING POSITIONS

Month	Day	Year	Magnitude	Gringorten	Cunnane	Weibull	Hazen
7	12	64	4381.000	0.021	0.022	0.036	0.019
8	27	82	3355.200	0.058	0.059	0.071	0.056
1	22	77	2879.200	0.094	0.096	0.107	0.093
7	7	67	1703.700	0.131	0.132	0.143	0.130
7	23	75	1175.200	0.168	0.169	0.179	0.167
7	14	65	1044.700	0.205	0.206	0.214	0.204
7	20	66	955.300	0.242	0.243	0.250	0.241
8	4	80	894.000	0.279	0.279	0.286	0.278
8	2	68	875.200	0.316	0.316	0.321	0.315
7	31	63	751.600	0.353	0.353	0.357	0.352
8	29	86	547.400	0.389	0.390	0.393	0.389
2	5	76	517.900	0.426	0.426	0.429	0.426
3	19	73	502.000	0.463	0.463	0.464	0.463
7	20	70	492.400	0.500	0.500	0.500	0.500
8	1	78	457.600	0.537	0.537	0.536	0.537
7	23	71	433.400	0.574	0.574	0.571	0.574
7	15	81	345.800	0.611	0.610	0.607	0.611
9	20	83	311.000	0.647	0.647	0.643	0.648
7	28	69	262.100	0.684	0.684	0.679	0.685
8	16	89	216.800	0.721	0.721	0.714	0.722
7	17	85	214.500	0.758	0.757	0.750	0.759
7	3	74	175.300	0.795	0.794	0.786	0.796
10	20	88	77.400	0.832	0.831	0.821	0.833
8	16	84	65.200	0.869	0.868	0.857	0.870
6	6	72	64.900	0.906	0.904	0.893	0.907
7	15	87	31.500	0.942	0.941	0.929	0.944
6	7	79	9.500	0.979	0.978	0.964	0.981

Arithmetic Mean: 842.21481
Standard Deviation: 1072.20404
Skew Coefficient: 2.20068
Coefficient of Variation: 1.27308

Arithmetic Mean of Logs: 2.59971
Standard Deviation of Logs: 0.61702
Skew Coefficient of Logs: -0.68427
Coefficient of Variation of Logs: 0.23734

Regional Skew Coefficient: None
Weighted Skew Coefficient: None

MAGNITUDES FOR SELECTED PROBABILITIES

Probability	Extreme Value	Log Extreme Value	Station LP III	Log Normal
0.99000	-992.02	35.01	4.08	10.24
0.90000	-303.34	78.42	59.87	64.37
0.80000	-68.84	118.97	129.50	120.28
0.50000	670.90	317.05	469.12	397.84
0.42920	857.09	405.76	596.43	511.59
0.20000	1666.20	1185.48	1344.31	1315.96
0.10000	2325.18	2838.67	2136.23	2458.86
0.04000	3157.80	8555.87	3299.57	4787.59
0.02000	3775.49	19396.52	4224.92	7363.34
0.01000	4388.61	43707.81	5176.70	10836.90
0.00400	5195.89	127386.78	6138.97	15458.26
0.00200	5805.45	285698.16	7394.79	23741.13

George V. Sabol Consulting Engineers, Inc.

SHEET ____ OF ____

CLIENT

DATE Revised 15 Oct 91

PROJECT

BY GVS

SUBJECT FIGURE 1-9

PROJECT NO.

Work Sheet for Log-Normal Confidence Limits

Watershed Walnut Gulch 63.011

Station No. 09471120

$Q_{2\text{-yr}}$ 445 cfs

$Q_{100\text{-yr}}$ 6500 cfs

Confidence Level (C.L.) 90 %

$a = \frac{100 - C.L.}{100}$ 0.10

$u_{1-a/2}$ 1.645

N 26

$\bar{Y} = \log_{10}(Q_{2\text{-yr}}) = \log_{10}(\ 445\ ) =$ 2.6484

$S_Y = \frac{\log_{10} Q_{100\text{-yr}} - \log_{10} Q_{2\text{-yr}}}{2.327} = \frac{\log_{10}(6500) - \log_{10}(445)}{2.327}$ 0.5004

T years	$u_{1-\frac{1}{T}}$	Y_T (a)	S_T (b)	Limits (c) Upper	Lower
(1)	(2)	(3)	(4)	(5)	(6)
2	0.0	2.6484	0.0981	645	307
5	.842	3.0697	0.1142	1810	762
10	1.282	3.2899	0.1324	3219	1181
25	1.751	3.5246	0.1561	6045	1853
50	2.052	3.6752	0.1729	9112	2459
100	2.327	3.8128	0.1889	13290	3177

(a) $Y_T = \bar{Y} + u_{1-\frac{1}{T}} S_Y$

(b) $S_T = \left[\left(S_Y^2/N\right)\left(1 + .5\, u^2_{1-\frac{1}{T}}\right)\right]^{1/2}$

(c) $Q_L = 10^{\left(Y_T \pm u_{1-a/2} S_T\right)}$

PC Shell Version 7 Directory Print ID = None

Path=B:\

	Name	Ext	Size	#Clu	Date	Time	Attributes	
Academy Acres	AA10	DAT	818	2	10-21-91	11:40a	A	Single - Basin Clark u-hg
	AA10	OUT	20065	40	10-21-91	11:40a	A	
	AA100	DAT	819	2	10-21-91	11:46a	A	
	AA100	OUT	19963	39	10-21-91	11:46a	A	
	AA25	DAT	818	2	10-21-91	11:42a	A	
	AA25	OUT	20015	40	10-21-91	11:43a	A	
Walnut Gulch 63.011	WG11A10	DAT	831	2	10-21-91	1:28p	A	Single-Basin Clark u-hg (NOAA statistics)
	WG11A10	OUT	8443	17	10-21-91	1:28p	A	
	WG11A100	DAT	831	2	10-21-91	1:29p	A	
	WG11A100	OUT	8443	17	10-21-91	1:30p	A	
	WG11A25	DAT	831	2	10-21-91	1:29p	A	
	WG11A25	OUT	8443	17	10-21-91	1:29p	A	
	WG11B10	DAT	1145	3	11-11-91	12:10p	A	Single-Basin S-graph (NOAA statistics)
	WG11B10	OUT	8880	18	11-11-91	12:10p	A	
	WG11B100	DAT	1144	3	11-11-91	12:20p	A	
	WG11B100	OUT	8880	18	11-11-91	12:20p	A	
	WG11B25	DAT	1145	3	11-11-91	12:14p	A	
	WG11B25	OUT	8880	18	11-11-91	12:15p	A	
	WG11C10	DAT	862	2	10-25-91	10:36a	A	Single - Basin Clark u-hg (Walnut Gulch site-specific statistics)
	WG11C10	OUT	8443	17	10-25-91	10:37a	A	
	WG11C100	DAT	791	2	10-25-91	10:24a	A	
	WG11C100	OUT	32785	65	10-25-91	10:24a	A	
	WG11C25	DAT	791	2	10-25-91	10:31a	A	
	WG11C25	OUT	8443	17	10-25-91	10:31a	A	
	WG11D10	DAT	1159	3	11-11-91	12:06p	A	Single-Basin S-graph (Walnut Gulch site-specific statistics)
	WG11D10	OUT	9097	18	11-11-91	12:07p	A	
	WG11D100	DAT	1160	3	11-11-91	12:25p	A	
	WG11D100	OUT	8880	18	11-11-91	12:25p	A	
	WG11D25	DAT	1163	3	11-11-91	12:12p	A	
	WG11D25	OUT	8880	18	11-11-91	12:12p	A	

Walnut Gulch
63.008

WG8A10	DAT	831	2	10-21-91	1:21p	A	Single-Basin	
WG8A10	OUT	8443	17	10-21-91	1:21p	A	Clark u-hg	
WG8A100	DAT	832	2	10-21-91	1:20p	A		
WG8A100	OUT	8443	17	10-21-91	1:20p	A		NOAA
WG8A25	DAT	831	2	10-21-91	1:21p	A		Statistics
WG8A25	OUT	8443	17	10-21-91	1:22p	A		
WG8B10	DAT	1227	3	11-11-91	11:42a	A	Single-Basin	
WG8B10	OUT	9207	18	11-11-91	11:43a	A	S-graph	
WG8B100	DAT	1215	3	11-11-91	11:30a	A		
WG8B100	OUT	8990	18	11-11-91	11:30a	A		
WG8B25	DAT	1223	3	11-11-91	11:47a	A		
WG8B25	OUT	8990	18	11-11-91	11:48a	A		
WG8C10	DAT	862	2	11-25-91	10:17a	A	Single-Basin	
WG8C10	OUT	8443	17	11-25-91	10:18a	A	Clark u-hg	
WG8C100	DAT	791	2	10-25-91	10:14a	A		
WG8C100	OUT	33417	66	10-25-91	10:14a	A		Walnut Gulch
WG8C25	DAT	778	2	11-25-91	10:14a	A		Site-Specific
WG8C25	OUT	8443	17	11-25-91	10:15a	A		Statistics
WG8D10	DAT	1182	3	11-11-91	11:40a	A		
WG8D10	OUT	8990	18	11-11-91	11:40a	A	Single-Basin	
WG8D100	DAT	1214	3	11-11-91	4:24p	A	S-graph	
WG8D100	OUT	8990	18	11-11-91	4:25p	A		
WG8D25	DAT	1244	3	11-11-91	11:45a	A		
WG8D25	OUT	8990	18	11-11-91	11:46a	A		
WG8E10	DAT	1339	3	11-12-91	8:42a	A		
WG8E10	OUT	12626	25	11-12-91	8:43a	A	Multi-Basin	NOAA
WG8E100	DAT	2065	5	11-11-91	8:59a	A	Clark u-hg	Statistics
WG8E100	OUT	26104	51	11-11-91	9:00a	A		
WG8E25	DAT	1337	3	11-11-91	4:14p	A		
WG8E25	OUT	12626	25	11-11-91	4:14p	A		
WG8F10	DAT	1354	3	11-12-91	8:33a	A		Walnut Gulch
WG8F10	OUT	12626	25	11-12-91	8:34a	A	Multi-Basin	Site-Specific
WG8F100	DAT	2101	5	11-12-91	9:20a	A	Clark u-hg	Statistics
WG8F100	OUT	466556	912	11-12-91	9:23a	A		
WG8F25	DAT	1352	3	11-11-91	4:07p	A		
WG8F25	OUT	12626	25	11-11-91	4:08p	A		

66 files LISTed 897,579 byte 66 files in subdir = 897,579 byte
5 0 files SELECTed 0 bytes Available on volume = 542,720 byte

George V. Sabol Consulting Engineers, Inc.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

SDW

FLOOD CONTROL DISTRICT
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DEC 0 9 1991

CH ENG		P & PM
DEP	/	HYDRO
ADMIN		LMGT
FINANCE		FILE
C & O		
ENGR		

REMARKS

29 November 1991

Mr. Stephen D. Waters
Flood Control District of Maricopa County
2891 West Durango
Phoenix, Arizona 85009

DRAFT

Subject: Additional Verification of the Hydrology Manual

Dear Steve:

We have completed the additional verification testing of the Maricopa County Hydrologic Design Manual as described in our letter of 9 September 1991 and approved by the District in its letter of 20 September 1991.

I believe that the results are reasonable and lend additional credence to the procedures in the Manual. These results can be used to support our conclusion that the Manual provides reasonable estimates of design discharges in Maricopa County. These results certainly do not indicate that the procedure leads to overestimation of design discharges as I have occassionaly heard comment by reviewers of the Manual. In fact, these results, taken alone, could indicate that design discharges are underestimated. However, the data base isn't large enough for me to make any conclusions that the procedures are biased toward either under- or overestimating design discharges. In my opinion, our previous verification results (see Table 4-10 of the Documentation/Verification Report) along with these results do not indicate a bias.

The report was prepared in a format that is compatible to that used in Part 4 of the Documentation/Verification Report, and as such, it could be incorporated into that report without much additional effort. Or this report could be used as a supplement or annex to that report. It may be best to not hold-up release of the Documentation/Verification Report to incorporate this information. We can discuss this after you have reviewed the enclosed.

A disk of model input and output files is enclosed with a brief description of model input for each file.

Please call either me or Mr. Joe Rumann if you wish to discuss our analyses or if you have questions.

As always, it has been our pleasure to be of service to you and the District.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo V Sabol

George V. Sabol

Enclosure: Verification Report for Academy Acres and Walnut Gulch Watersheds

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAFT Final Submitted 3 Jan 92

VERIFICATION REPORT

FOR ACADEMY ACRES AND

WALNUT GULCH WATERSHEDS

George V. Sabol Consulting Engineers, Inc.
Brighton, Colorado and Phoenix, Arizona

November 1991

INTRODUCTION

This report presents the results of verification testing of the procedures in the Maricopa County Hydrologic Design Manual (Manual). Frequency simulation results are presented for three watersheds with basin characteristics and streamgage data as described below:

Watershed	Size sq.mi.	Land-use	Length of Gage Record years
Academy Acres at Albuquerque, NM	0.12	residential	14
Walnut Gulch 63.011 at Tombstone	3.12	desert rangeland	27
Walnut Gulch 63.008 at Tombstone	5.98	desert rangeland	27

The Beaver Creek #8 watershed, about 50 miles south of Flagstaff, was to be used for these test verification purposes also. However, after initial investigations of the watershed it was discovered that volcanic cinder exists in the watershed that negates the use of the Green and Ampt infiltration equation with parameter values based on soil texture. Therefore, that watershed was deleted from further consideration since watersheds of volcanic cinder are not known to be represented in Maricopa County.

Watershed maps and watershed information that were used in developing the models are shown in the appendix. The streamgage data and calculations for the flood frequency analyses for each watershed are also provided in the appendix.

Graphical flood frequency analyses were performed for each of the watersheds using the available data. The analyses were performed using the procedure that was developed for the ADOT Hydrology Manual (1991 draft). This includes establishing a best fit line to the data when plotted on probability paper, and also includes 90 percent confidence limit bands about the best fit line. The graphical flood frequency analyses for the three watersheds are shown in Figures 1 through 3.

The following is a discussion of the modeling and results for each of the watersheds. The model results are compared to the flood magnitudes from the flood frequency analyses.

ACADEMY ACRES, ALBUQUERQUE, NM

The drainage area is a small (0.12 sq. mi.) residential area developed on an alluvial fan in the Northeast Heights of Albuquerque. Streets are the major conveyance for storm runoff. There is a relatively short concrete lined channel at the outlet of the watershed and the streamgage is located in the channel. The area consists of 191 single-family and 44 duplex residential units with a density of about 5 units per acre. A church and paved parking lot are contained in the upper part of the watershed. The residences are generally landscaped with irrigated lawns with a small amount of native vegetation that occasionally may be of gravel underlain by plastic.

Rainfall for the frequency simulation was developed by procedures in the Manual using rainfall statistics for this location from the NOAA Atlas for New Mexico (Miller and others, 1973a). For a watershed of this size (0.12 sq. mi.) this required Pattern No. 1 and a depth-area reduction factor of 1.0.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. The Green and Ampt parameters were area averaged with hydraulic conductivity (XKSAT) of 0.96 in/hr, capillary suction (PSIF) of 2.8 inches, and soil moisture deficit (DTHETA) of 0.29. The surface retention (IA) was estimated as 0.20 inch, and the effective impervious area (RTIMP) as 28%.

A single-basin model using the Clark unit hydrograph was developed. The unit hydrograph parameters were calculated based on an area (A) of 0.124 square miles, watercourse length (L) of 0.9 mile, slope of 105 ft/mi, and resistance coefficient of 0.028. T_c and R varied for each flood return period since the procedures to estimate these parameters are a function of rainfall excess intensity, which varies for each flood return period. The synthetic urban time-area relation was used.

The results of the graphical flood frequency analysis and results of the frequency simulation are shown in Table 1 and Figure 1. The record length is only 14 years, and therefore the accuracy of the flood frequency analysis to represent the "true" flood frequency relation may be questionable. However, there should be relatively high confidence that the true flood magnitudes are contained within the 90 percent confidence level bands. The model produces flood peak discharges that are within the 90 percent confidence limits for all return periods. The model results are lower than the best estimates from the flood frequency analysis, and the results are somewhat better at the 100-yr than at the 10-yr return period.

TABLE 1

Verification results for the Academy acres Watershed
All values in cfs

Return Period years	Flood Frequency Results			Model Results
	Best Estimate	Upper Limit	Lower Limit	
10	95	140	60	60
25	130	210	80	95
100	190	340	110	160

WALNUT GULCH 63.011 AND 63.008, TOMBSTONE, AZ

These watersheds are instrumented subbasins of the Walnut Gulch Experimental Watershed that is operated by the USDA, Agricultural Research Service near Tombstone, Arizona. Watershed 63.008 is 5.98 square miles and it contains watershed 63.011 as a subbasin. The watersheds consist of undeveloped rangeland, and the vegetation is predominantly native brush, grasses, and cacti.

The models were run using two different sources for rainfall statistics; the NOAA Atlas for Arizona (Miller and others, 1973b) and site-specific rainfall statistics as developed from information in Osborn and Renard (1988). There is considerable difference between the rainfall statistics from these two sources. The site-specific rainfall statistics are appreciably higher than the NOAA Atlas statistics. For example, the NOAA 100-yr, 1-hr rainfall is 2.43 inches and the site-specific 100-yr, 1-hr rainfall is 3.07 inches (a 26 percent increase over the NOAA statistic). Osborn and Renard do not provide rainfall depth-duration-frequency statistics for durations in excess of 1 hour. However, the Maricopa County procedure requires a 6-hr rainfall depth to define the design storm. Therefore, the Osborn and Renard rainfall statistics were plotted on graph paper along with the NOAA statistics and the Osborn and Renard statistics were extended to 6 hours to follow the same slope of the NOAA lines. This graph is shown in the appendix. This may or may not represent severe storms for the watershed. According to Osborn (personal communication, October 1991) the peak discharges on both watersheds 63.011 and 63.008 resulted from rains of durations less than 1 hour. Although 6-hr type storms may occur over the Walnut Gulch watershed that have similar characteristics to the Maricopa County design storm (1954 Queen Creek storm), such storms apparently have not occurred in that area since the watershed was instrumented. Therefore, modeling of the Walnut Gulch watersheds using the 6-hr Maricopa County design storm may not be representative of the appropriate regional meteorologic conditions. Nonetheless, modeling of these watersheds was performed using 6-hr rainfall depths as described. For watershed 63.011 (3.12 sq. mi.) this required Pattern No. 2.07 and a depth-area reduction factor of 0.97, and for watershed 63.008 (5.98 sq. mi.) the Pattern No. is 2.44 and the depth-area reduction factor is 0.96.

Rainfall losses were calculated by the Green and Ampt infiltration equation and surface retention loss. Watershed 63.011 is a subbasin of 63.008 and the same rainfall loss parameters were used for both watersheds. The Green and Ampt parameters were area averaged with hydraulic conductivity (XKSAT) of 0.22 in/hr, capillary suction (PSIF) of 5.8 inches, and soil moisture deficit (DTHETA) of 0.39. The surface retention (IA) was estimated as 0.35 inch, and the impervious area (RTIMP) as 0%.

These two watersheds were modeled using both the Clark unit hydrograph and the Phoenix Valley S-graph. Watershed 63.008 was modeled as a single-basin and also as a two subbasin model. The watershed characteristics that were used to calculate the Clark unit hydrograph parameters, T_c and R, and the S-graph Lag are shown in Table 2. The synthetic natural time-area relation was used with the Clark unit hydrograph.

TABLE 2

Watershed characteristics used in calculating unit hydrograph parameters for Walnut Gulch watersheds 63.011 and 63.008

Area A sq.mi.	Length L miles	Centroid Length L_{ca} miles	Slope S ft/mi	Resistance Coefficients K_b Clark	Resistance Coefficients K_n S-Graph
Watershed 63.011 Single-Basin Models					
3.18	4.0	1.8	100	0.033	.03
Watershed 63.008 Single-Basin Models					
5.98	8.0	3.6	75	0.033	.03
Watershed 63.008 Multi-Basin Models (Clark only)					
3.18	4.0	---	100	0.033	------
2.80	4.0	---	75	0.033	------

The model results are within the 90 percent confidence levels for both watersheds when the site-specific rainfall statistics are used. The model results are consistently less than the lower 90 percent confidence level values when the NOAA Atlas rainfall statistics are used. Since the site-specific rainfall statistics more accurately reflect the actual rainfall regime than do the NOAA Atlas statistics, it seems appropriate to evaluate the model performance based on the site-specific rainfall statistics. Because of this, all results that are discussed are the results using the site-specific rainfall statistics that were developed from Osborn and Renard (1988).

Watershed 63.011 is smaller than the recommended 5 square mile upper limit for application of the Clark unit hydrograph, and, therefore, this watershed was modeled as a single basin. Model results for watershed 63.011 are shown in Table 3 and Figure 2. The model results using both the Clark unit hydrograph and the S-graph are very close to the best estimate of the 10-yr flood peak discharge. The results are not as good at the 25- and 100-yr return periods, but are within the 90 percent confidence levels. Considering that this watershed is outside of Maricopa County and that the design rainfall criteria that were applied may not be completely representative of the regional severe storm characteristics, the results are reasonable.

TABLE 3

Verification results for the Walnut Gulch 63.011 watershed
All values in cfs

Return Period years	Flood Frequency Results Best Estimate	Flood Frequency Results Upper Limit	Flood Frequency Results Lower Limit	Using NOAA Statistics Clark u-hg	Using NOAA Statistics S-graph	Using Site-Specific Statistics Clark u-hg	Using Site-Specific Statistics S-graph
10	1,950	3,220	1,180	560	960	1,760	2,050
25	2,950	6,040	1,850	1,170	1,570	2,030	2,290
100	6,500	13,290	3,180	2,300	2,500	4,380	4,190

Watershed 63.008 is a little larger than the recommended 5 square mile upper limit for application of the Clark unit hydrograph, but is smaller than the absolute 10 square mile upper limit for application. Therefore, this watershed was modeled as a single basin using the Clark unit hydrograph and the S-graph, and was also modeled as a multi-basin (two subbasins) watershed using the Clark unit hydrograph. When modeled as a single basin, the calculated T_c exceeded the duration of rainfall excess indicating that this watershed should not be modeled as a single basin when using the Clark unit hydrograph procedure as described in the Manual.

The model results, shown in Table 4 and Figure 3, are not particularly good when the watershed is treated as a single basin with the Clark unit hydrograph. This provides evidence that the size recommendations for the Clark unit hydrograph procedure should not be exceeded if the calculated T_c exceeds the duration of rainfall excess.

TABLE 4

Verification results for the Walnut Gulch 63.008 watershed
All values in cfs

Return Period years	Flood Frequency Results Best Estimate	Flood Frequency Results Upper Limit	Flood Frequency Results Lower Limit	Using NOAA Statistics Clark u-hg	Using NOAA Statistics S-graph	Using NOAA Statistics Multi-Basin	Using Site-Specific Statistics Clark u-hg	Using Site-Specific Statistics S-graph	Using Site-Specific Statistics Multi-Basin
10	2,100	3,260	1,340	780	1,060	920	1,790	2,450	2,070
25	3,300	5,720	2,010	1,330	1,830	1,540	2,010	2,750	2,320
100	6,200	11,620	3,270	2,220	3,030	2,450	3,820	5,250	5,190

The results of the single basin, S-graph model are reasonable. This indicates that, for small, desert rangeland watersheds, the Phoenix Valley S-graph is a viable unit hydrograph procedure and it can be used in certain applications where the Clark unit hydrograph is either inappropriate (exceeds size limitations) or where expedience may warrant the use of an S-graph rather than the Clark unit hydrograph.

The multi-basin, Clark unit hydrograph model yielded reasonable results for the full range of return periods. This indicates that the Clark unit hydrograph can be used for larger watersheds where it is either necessary or desirable to model the watershed as a system of subbasins.

The model results for both watersheds 63.011 and 63.008 are highly dependent upon the ability of the rainfall input to reflect local, severe storm rainfall characteristics. The rainfall criteria that were applied to these watersheds was developed from an historic 7-hr duration local storm in Maricopa County as represented by the 6-hr design rainfall criteria in the Manual. That rainfall may not be representative of the spatial and temporal distributions of rainfall that actually occur in the Tombstone area. Therefore, the accuracy of the developed rainfall-runoff models to reproduce recorded flood frequency relation must be interpreted within this assumption.

SUMMARY OF VERIFICATIONS

Flood frequency simulations were performed for three watersheds that have streamgage records. None of the watersheds are in Maricopa County; two are in Cochise County in southeast Arizona, and one is a fully urbanized watershed in Albuquerque, New Mexico. Flood peak discharges were estimated using procedures in the Manual for the watersheds for return periods of 10-, 25- and 100-yr. The ratio of the flood peak discharge as estimated by the most appropriate model to the discharge from the best fit flood frequency line are shown in Table 5.

TABLE 5

Results of verifications for the Academy Acres and Walnut Gulch 63.011 and 63.008 watersheds

Return Period	Ratio of flood peak discharges estimated by procedures in the Manual to discharges from flood frequency analyses		
years	Academy Acres[a]	Walnut Gulch 63.011[b]	Walnut Gulch 63.008[c]
10	.63	.90	.99
25	.73	.69	.70
100	.84	.67	.84

[a] - Table 1, Model Results: Best Estimate
[b] - Table 3, Clark unit hydrograph and Site-Specific Rain: Best Estimate
[c] - Table 4, Multi-Basin and Site-Specific Rain: Best Estimate

It is also interesting to note that the single-basin Clark unit hydrograph model and the single-basin S-graph model for watershed 63.011 generally yield similar results, and that the multi-basin Clark unit hydrograph model and the single-basin S-graph model for watershed 63.008 generally yield similar results. This provides technical support for the applicability of these unit hydrograph procedures for natural watersheds.

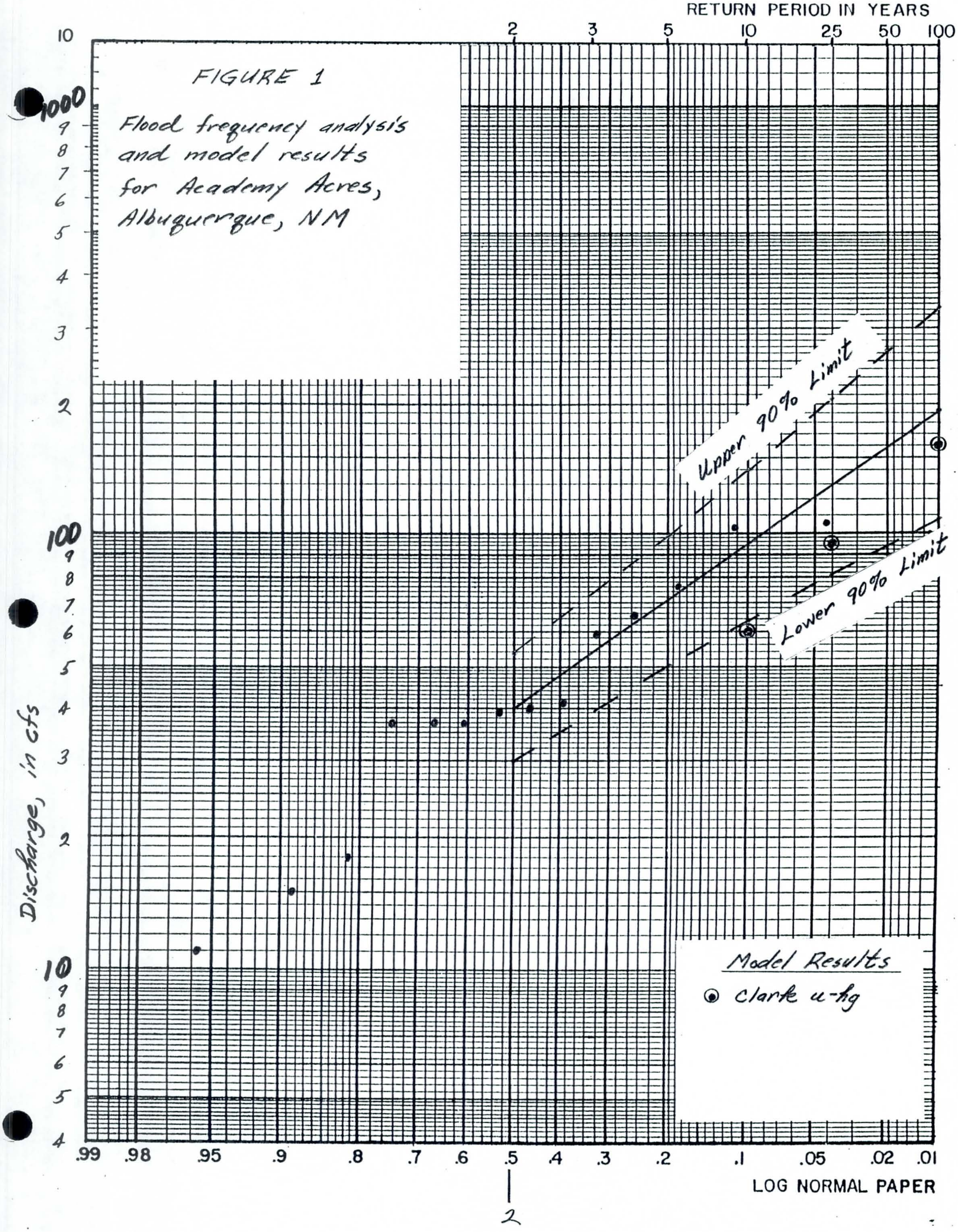
Considering the assumption of the applicability of the Maricopa County design rainfall criteria to these watersheds that are not within Maricopa County, the results seem appropriate in serving as a verification of the modeling procedure.

REFERENCES

Osborn, H.B., and Renard, K.G., 1988, Rainfall intensities for southeastern Arizona; Amer. Soc. of Civil Engineers, Journal of Irrigation and Drainage Engineering, Vol. 114, No. 1, pp. 195-199.

Miller, J.F., Frederick, R.H., and Tracey, R.J., 1973a, NOAA Atlas 2, Precipitation-frequency atlas of the western United States, Volume IV, New Mexico: National Weather Service, U.S. Dept of Commerce, Silver Springs, MD.

Miller, J.F., Frederick, R.H., and Tracey, R.J., 1973b, NOAA Atlas 2, Precipitation-frequency atlas of the western United States, Volume VIII, Arizona: National Weather Service, U.S. Dept of Commerce, Silver Springs, MD.


RETURN PERIOD IN YEARS
2
3
5
10
25
50
100
FIGURE 1
Flood frequency analysis and model results for Academy Acres, Albuquerque, NM
Upper 90% Limit
Lower 90% Limit
Model Results
Clark u-hg
Discharge, in cfs
10
1000
9
8
7
6
5
4
3
2
100
10
.99
.98
.95
.9
.8
.7
.6
.5
.4
.3
.2
.1
.05
.02
.01
LOG NORMAL PAPER
2

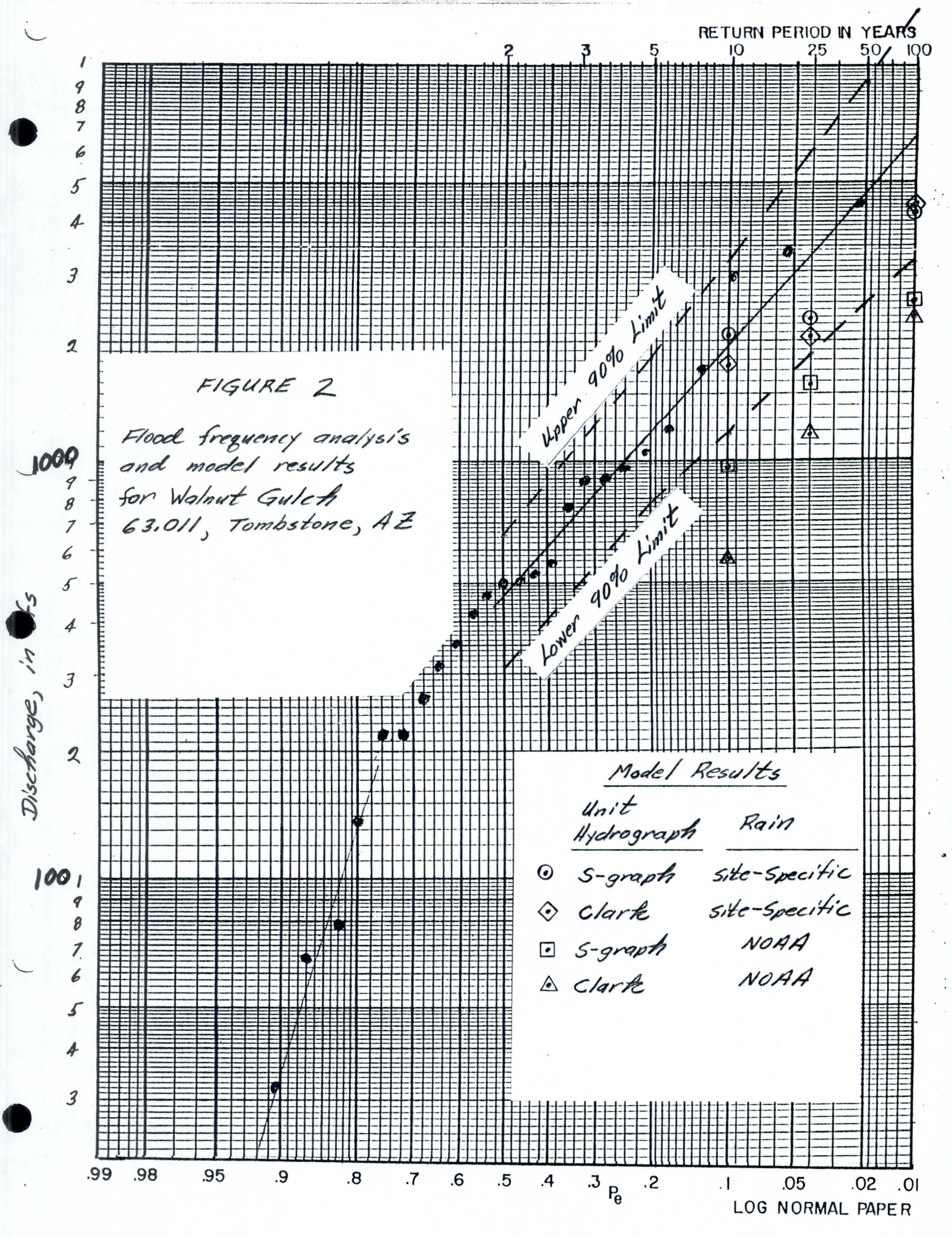
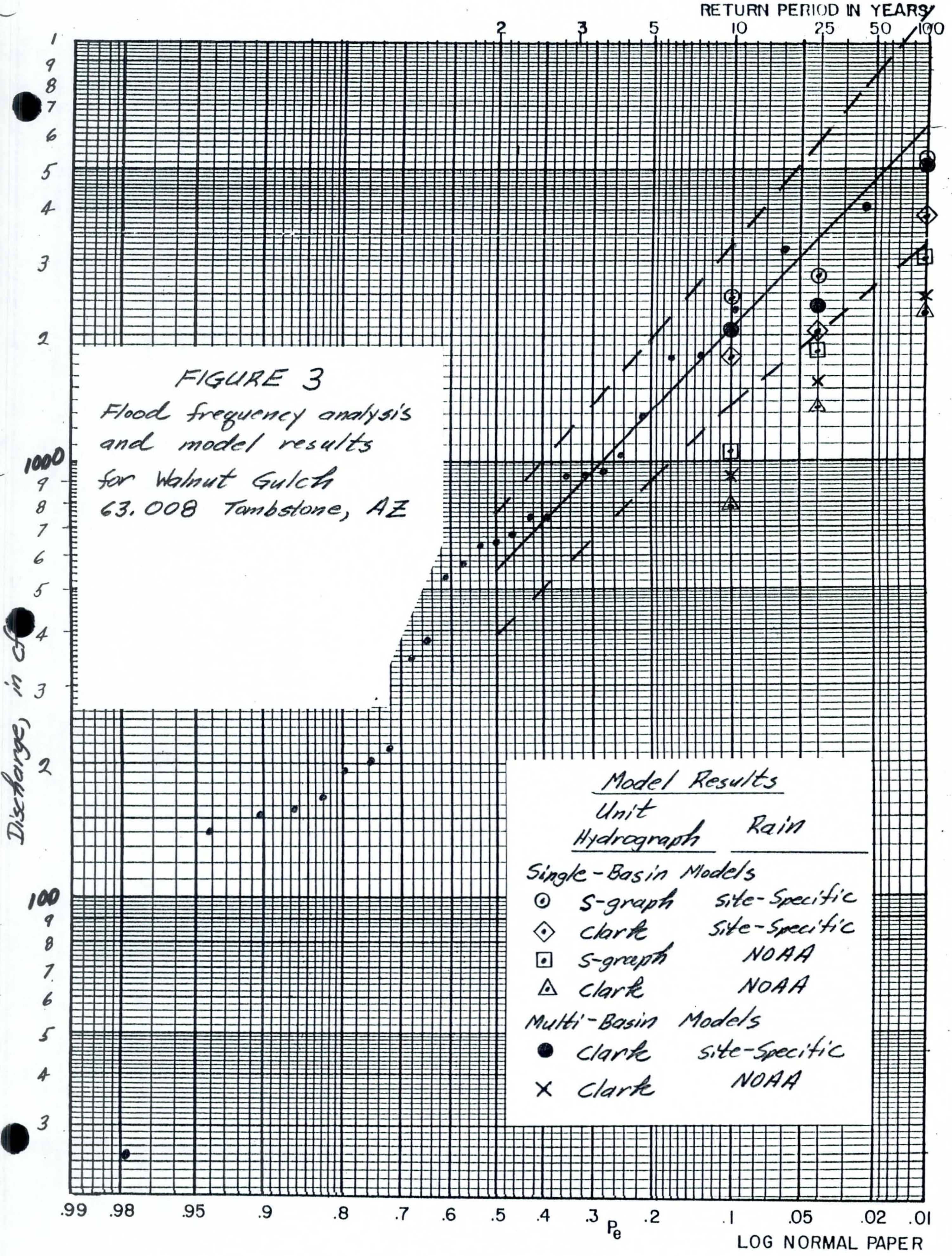


FIGURE 2

Flood frequency analysis and model results for Walnut Gulch 63.011, Tombstone, AZ



RETURN PERIOD IN YEARS
2
3
5
10
25
50
100
FIGURE 3
Flood frequency analysis and model results for Walnut Gulch 63.008 Tombstone, AZ
Discharge, in cfs
1000
100
.99
.98
.95
.9
.8
.7
.6
.5
.4
.3
P_e
.2
.1
.05
.02
.01
LOG NORMAL PAPER
Model Results
Unit Hydrograph
Rain
Single-Basin Models
S-graph site-Specific
Clark site-Specific
S-graph NOAA
Clark NOAA
Multi-Basin Models
Clark site-Specific
Clark NOAA

APPENDIX

Watershed and Streamgage Data for

Academy Acres

Table 7.--Station description, daily mean discharge values, monthly rainfall totals, and selected rainfall-discharge unit-value data for ACADEMY ACRES DRAIN AT ALBUQUERQUE (08329880), 1976-83.

STATION DESCRIPTION

LOCATION.--Lat 35 deg 09 min 02 sec, long 106 deg 34 min 18 sec, in NE1/4 SE1/4 sec.25, T.11 N., R.3 E., Bernalillo County, Hydrologic Unit 13020203, on left bank of concrete lined channel, at intersection of Burlison Dr and Leander Ave, 230 ft (70 m) north of intersection of Esther Ave and Burlison Dr and 0.4 mi (0.6 km) north of Academy Rd, in Albuquerque.

DRAINAGE AREA.--0.124 sq mi (0.321 sq km).

PERIOD OF RECORD.--June 1976 to December 1978, May 1979 to current year (no winter records).

GAGE.--Water-stage recorder and V-notch sharp-crested weir. Prior to May 1, 1978, concrete trapezoidal weir. Altitude of gage is 5,306 ft (1,617.27 m), from topographic orthophoto map.

REMARKS.--Records good except those for June 1976 to October 1979, which are poor. Recording rain gage at station. Additional recording rain gage near upstream end of basin since September, 1981. Basin drains residential area.

STAGE-DISCHARGE RELATION.--Rating developed on basis of weir-flow computations and discharge measurements at discharges of 0.10 cu ft per sec (0.003 cu m per sec), 0.20 cu ft per sec (0.006 cu m per sec), 0.35 cu ft per sec (0.01 cu m per sec), 1.00 cu ft per sec (0.03 cu m per sec), 2.50 cu ft per sec (0.07 cu m per sec) and 5.0 cu ft per sec (0.14 cu m per sec) June 1976 to May 1978. Rating developed on basis of V-notch sharp-crested weir computation and discharge measurements at discharges of 0.10 cu ft per sec (0.003 cu m per sec), 0.60 cu ft per sec (0.02 cu m per sec), 1.85 cu ft per sec (0.05 cu m per sec), 2.08 cu ft per sec (0.06 cu m per sec) and slope-area measurement at discharge of 100 cu ft per sec (2.83 cu m per sec).

EXTREMES FOR PERIOD OF RECORD.--Maximum discharge, 103 cu ft per sec (2.92 cu m per sec) Aug. 3, 1978, gage height, 4.09 ft (1.247 m) from rating curve extended above 2.0 cu ft per sec (0.57 cu m per sec) on basis of slope-area measurement of peak flow; no flow most of the time.

EXTREMES.--Maximum discharge during period June to December 1976, 20 cu ft per sec (0.55 cu m per sec) July 31, gage height, 0.68 ft (0.207 m); no flow most of the time.

Calendar year 1977: Maximum discharge, 15 cu ft per sec (0.42 cu m per sec) Sept. 5, gage height, 0.58 ft (0.177 m); no flow most of the time.

Calendar year 1978: Maximum discharge, 103 cu ft per sec (2.92 cu m per sec) Aug. 3, gage height, 4.09 ft (1.247 m) from rating curve extended above 2.0 cu ft per sec (0.57 cu m per sec) on basis of slope-area measurement of peak flow; no flow most of the time.

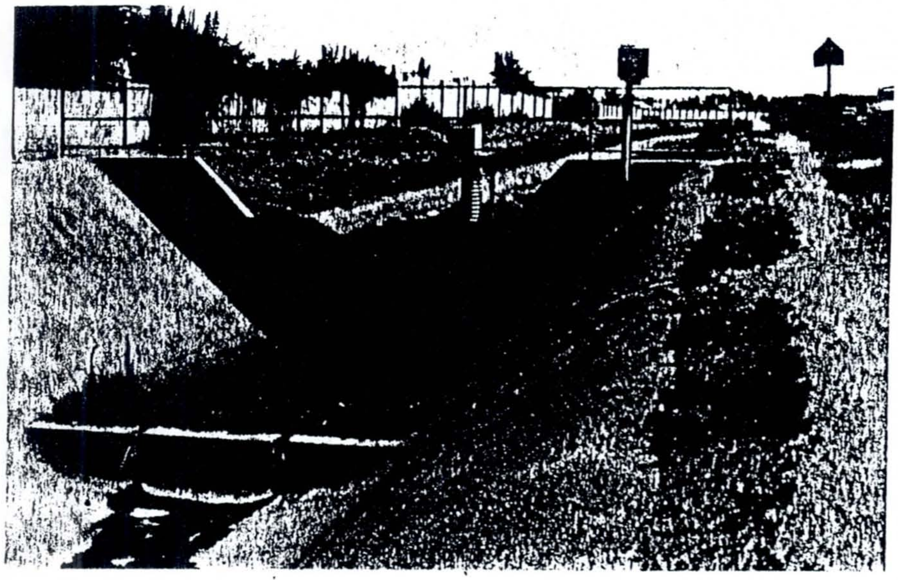
Calendar year 1979: Maximum discharge, 65 cu ft per sec (1.82 cu m per sec) July 16, gage height, 3.68 ft (1.122 m); no flow most of the time.

Calendar year 1980: Maximum discharge, 101 cu ft per sec (2.88 cu m per sec) Aug. 14, gage height, 4.07 ft (1.241 m); no flow most of the time.

Calendar year 1981: Maximum discharge, 59 cu ft per sec (1.67 cu m per sec) July 7, gage height, 3.61 ft (1.100 m); no flow most of the time.

Calendar year 1982: Maximum discharge, 37 cu ft per sec (1.05 cu m per sec) Aug. 12, gage height, 3.25 ft (0.991 m); no flow most of the time.

Calendar year 1983: maximum discharge, 39 cu ft per sec (1.10 cu m per sec) June 25, gage height, 3.29 ft (1.003 m); no flow most of the time.



Photograph 9.--Academy Acres Drain (08329880).

Frequency analysis for input data from file: ACADEMY.DAT
Station Name: ACADEMY ACRES, ALB, NM 0839988
Units: cfs
Regional Skew: None
14 data points used in the analysis

PLOTTING POSITIONS

Month	Day	Year	Magnitude	Gringorten	Cunnane	Weibull	Hazen
8	3	78	103.000	0.040	0.042	0.067	0.036
8	14	89	101.000	0.110	0.113	0.133	0.107
8	14	90	76.000	0.181	0.183	0.200	0.179
7	16	79	65.000	0.252	0.254	0.267	0.250
7	7	81	59.000	0.323	0.324	0.333	0.321
7	22	86	41.000	0.394	0.394	0.400	0.393
6	25	88	40.000	0.465	0.465	0.467	0.464
6	25	83	39.000	0.535	0.535	0.533	0.536
8	12	82	37.000	0.606	0.606	0.600	0.607
9	19	89	37.000	0.677	0.676	0.667	0.679
8	11	87	37.000	0.748	0.746	0.733	0.750
4	18	85	18.000	0.819	0.817	0.800	0.821
9	5	77	15.000	0.890	0.887	0.867	0.893
10	3	83	11.000	0.960	0.958	0.933	0.964

Arithmetic Mean: 48.50000
Standard Deviation: 29.00862
Skew Coefficient: 0.76421
Coefficient of Variation: 0.59812

Arithmetic Mean of Logs: 1.60464
Standard Deviation of Logs: 0.29100
Skew Coefficient of Logs: -0.48671
Coefficient of Variation of Logs: 0.18135

Regional Skew Coefficient: None
Weighted Skew Coefficient: None

MAGNITUDES FOR SELECTED PROBABILITIES

Probability	Extreme Value	Log Extreme Value	Station LP III	Log Normal
0.99000	-2.48	12.39	5.24	7.16
0.90000	14.53	18.36	16.58	17.04
0.80000	23.32	22.49	23.42	22.89
0.50000	44.00	36.27	42.54	40.24
0.42920	49.20	40.90	47.77	45.31
0.20000	71.82	68.95	71.41	70.74
0.10000	90.24	105.52	90.89	95.00
0.04000	113.51	180.62	114.98	130.07
0.02000	130.77	269.13	132.36	159.35
0.01000	147.91	399.82	149.12	191.21
0.00400	170.47	673.31	165.22	226.08
0.00200	187.51	997.98	185.78	276.78

GEORGE V. SABOL CONSULTING ENGINEERS, INC.

SHEET ____ OF ____

CLIENT ______

DATE Revised 15 Oct 91

PROJECT ______

BY GVS

SUBJECT FIGURE 1-9

PROJECT NO. ______

Work Sheet for Log-Normal Confidence Limits

Watershed ACADEMY ACRES Albuquerque, N.M.

Station No. ______

$Q_{2\text{-yr}}$ 40 cfs

$Q_{100\text{-yr}}$ 190 cfs

Confidence Level (C.L.) 90 %

$a = \frac{100 - C.L.}{100}$ 0.10

$u_{1-a/2}$ 1.645

N 14

$\bar{Y} = \log_{10}(Q_{2\text{-yr}}) = \log_{10}(\ 40\ ) =$ 1.602

$S_y = \frac{\log_{10} Q_{100\text{-yr}} - \log_{10} Q_{2\text{-yr}}}{2.327} = \frac{\log_{10}(190) - \log_{10}(40)}{2.327}$ 0.2908

T years (1)	$u_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits (c) Upper (5)	Limits (c) Lower (6)
2	0.0	1.602	.0777	54	30
5	.842	1.847	.0905	99	50
10	1.282	1.975	0.1049	140	63
25	1.751	2.111	0.1237	206	81
50	2.052	2.199	0.1369	266	94
100	2.327	2.279	0.1496	335	108

(a) $Y_T = \bar{Y} + u_{1-\frac{1}{T}} S_y$

(b) $S_T = \left[(S_y^2/N)(1 + .5\,u^2_{1-\frac{1}{T}})\right]^{1/2}$

(c) $Q_L = 10^{(Y_T \pm u_{1-a/2} S_T)}$

1

*** O U T P U T   D A T A ***

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR ACADEMY ACRES
PRIMARY ZONE NUMBER= 7
SHORT-DURATION ZONE NUMBER= 6

LATITUDE 35.15N   LONGITUDE 106.57W   ELEVATION 5306 FEET

POINT VALUES

	RETURN PERIOD							
DURATION	2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.32	.41	.48	.57	.64	.71	.87	5-MIN
10-MIN	.50	.64	.74	.88	.99	1.10	1.36	10-MIN
15-MIN	.60	.78	.91	1.09	1.23	1.37	1.69	15-MIN
30-MIN	.76	1.01	1.18	1.42	1.60	1.79	2.21	30-MIN
1-HR	.92	1.23	1.45	1.74	1.97	2.20	2.74	1-HR
2-HR	.98	1.31	1.53	1.84	2.08	2.32	2.88	2-HR
3-HR	1.03	1.36	1.59	1.91	2.16	2.40	2.97	3-HR
6-HR	1.11	1.46	1.70	2.03	2.29	2.55	3.15	6-HR
12-HR	1.18	1.55	1.80	2.16	2.44	2.71	3.34	12-HR
24-HR	1.25	1.64	1.91	2.29	2.58	2.87	3.54	24-HR

* IF YOUR SITE IS IN ARIZONA OR NEW MEXICO, PLEASE CONSULT THE FOLLOWING PAPER FOR REVISED DEPTH-AREA VALUES:
DEPTH-AREA RATIOS IN THE SEMI-ARID SOUTHWEST UNITED STATES
NOAA TECHNICAL MEMORANDUM NWS HYDRO-40
ZEHR AND MYERS
AUGUST 1984

INPUT DATA

PROJECT NAME=ACADEMY ACRES
✓ZONE= 7   SHORT-DURATION ZONE= 6✓
✓LATITUDE= 35.15   LONGITUDE= 106.57   ELEVATION= 5306
✓2-YR, 6-HR PCPN= 1.11   100-YR, 6-HR PCPN= 2.55
✓2-YR, 24-HR PCPN= 1.25   100-YR, 24-HR PCPN= 2.87

* * * * E N D   OF   R U N * * * *

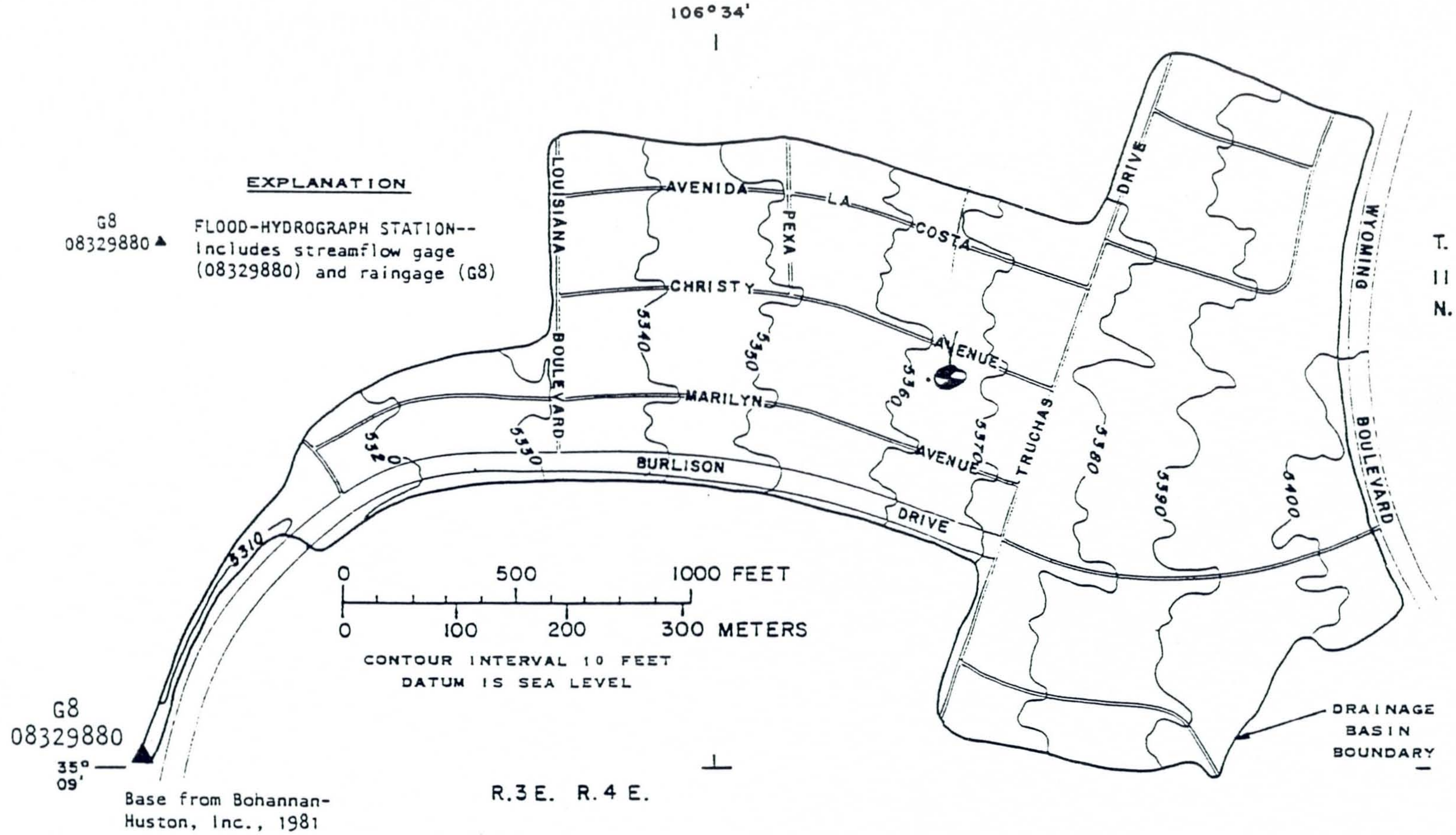
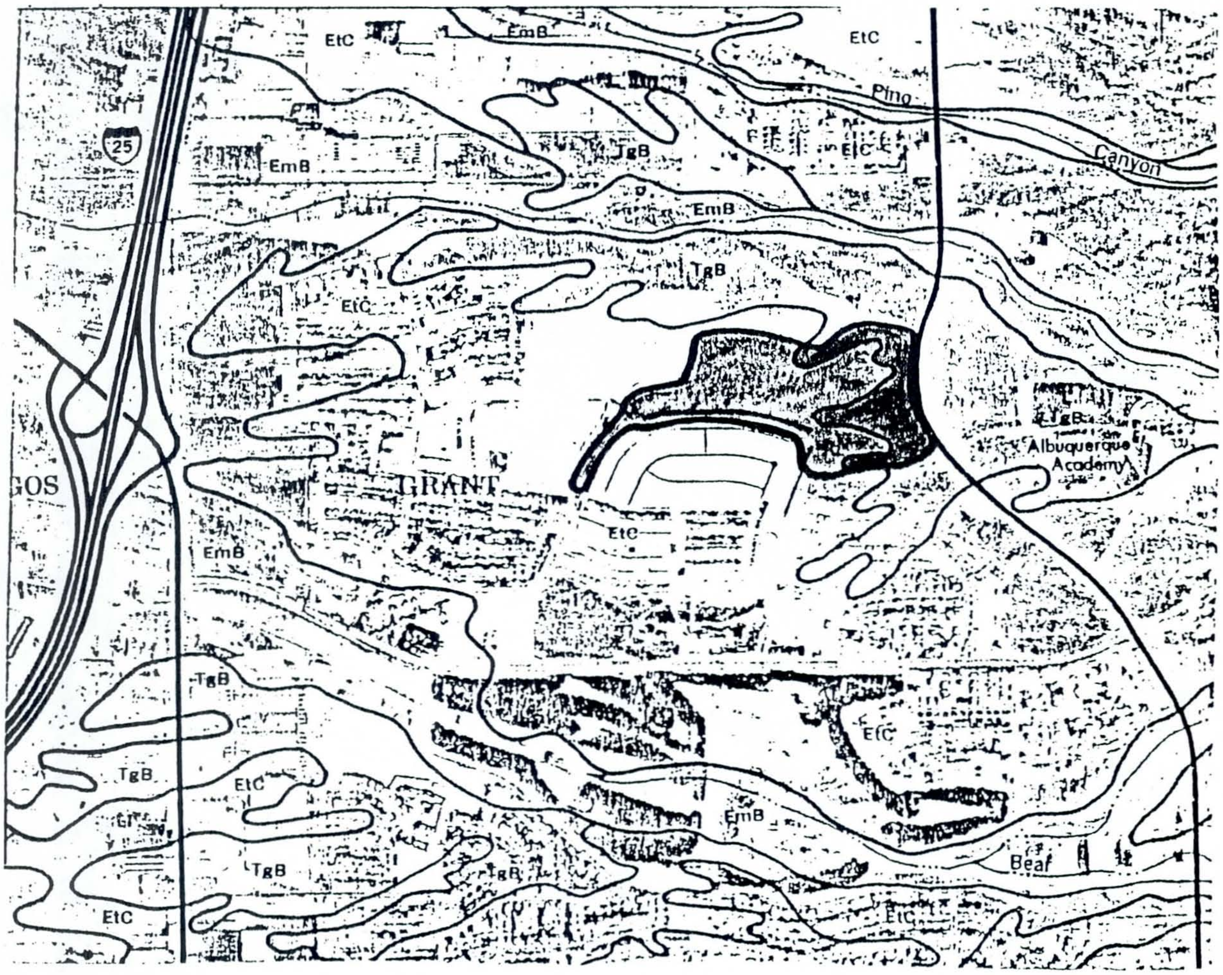


Figure 4.--Drainage basin for Academy Acres Drain (08329880).



Scale: 1 inch = 2,000 feet

Figure 3.- SCS soil classification for the Academy Acres drainage basin (SCS, 1977).

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DATE 18 Sept, 91

PROJECT Hydrology Manual

BY JMR

SUBJECT Single Basin Testing

PROJECT NO. 15

Academy Acres

Area = 0.124 mi^2 = 80 ac^2
L = .9 mi = 4750 ft.
L_{ca} = .52 mi = 2750 ft.
S = 105 ft/mi

USBR - Lag

k_n = 0.03 for urban, residential watershed

$$Lag = 26 k_n \left(\frac{(L)(L_{ca})}{\sqrt{S}} \right)^{.33}$$

= 0.28 hrs = 17 min.

k_n = 0.015

Lag = 0.14 hrs. = 9 min.

Green-Ampt Parameters:

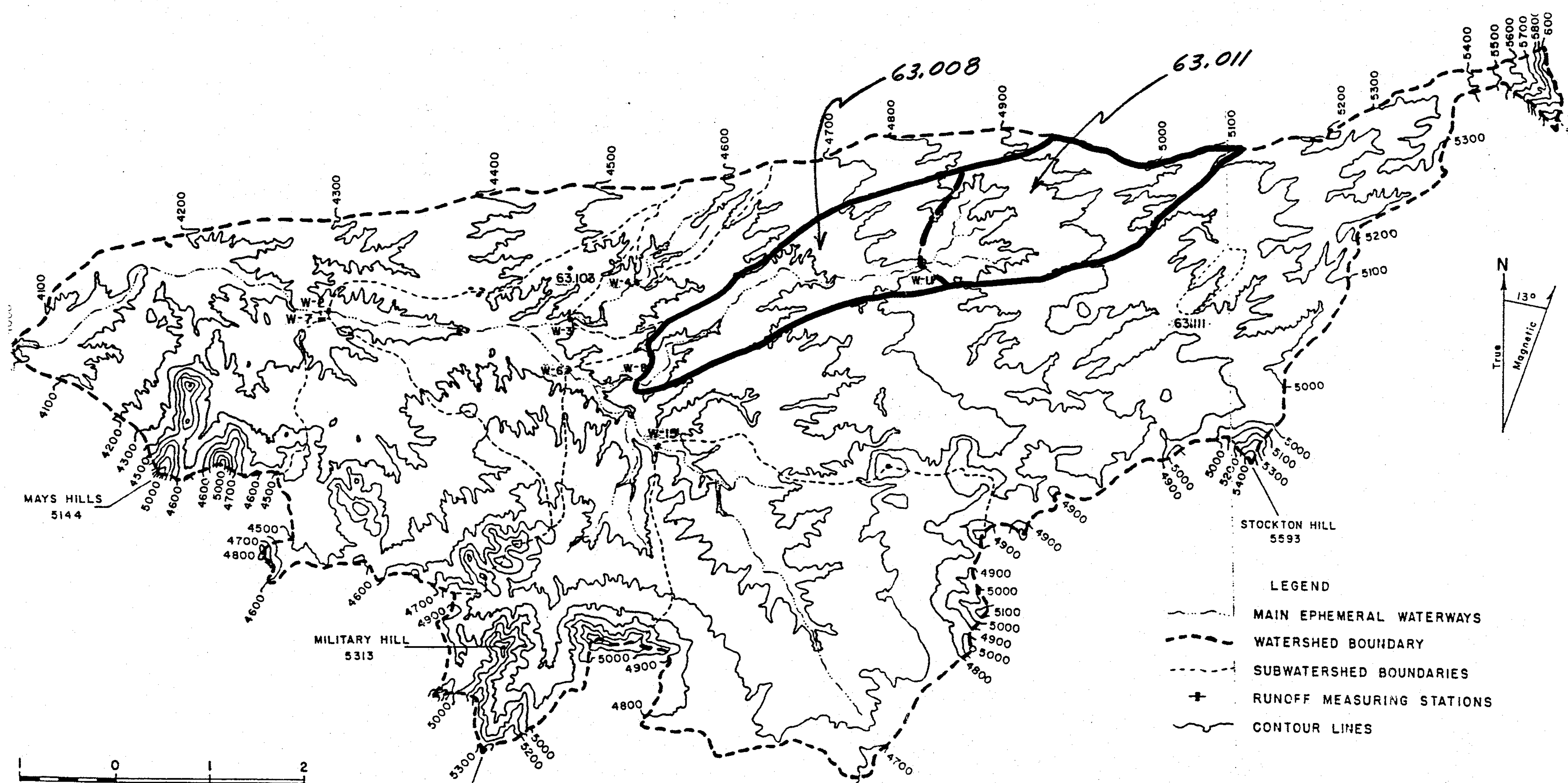
Map symbol	Etc	
Soil Name	Embudo-Tijeras	T
Textural Class	gravelly sandy loam	sa
% area	80	
DTHETA	.3	
PSIF	2.4	
XKSAT	1.2	0

XKSAT log-averaged by area = antilog[.8(log 1.
USING FIG. PSIF = 2.8 DTHETA = 0.
IA for developed lawn & turf = 0.2
% impervious = 38 %

gsL → LS ?
DISCUSS

Watershed and Streamgage Data for

Walnut Gulch 63.008



CONTOUR INTERVAL 100 FEET

For additional cultural, topographic features and raingage network, see map page 63.1-5 in "Hydrologic Data for Experimental Agricultural Watersheds in the United States 1956-1959", Misc. Pub. No. 945, U.S.D.A., Agricultural Research Service, November 1963.

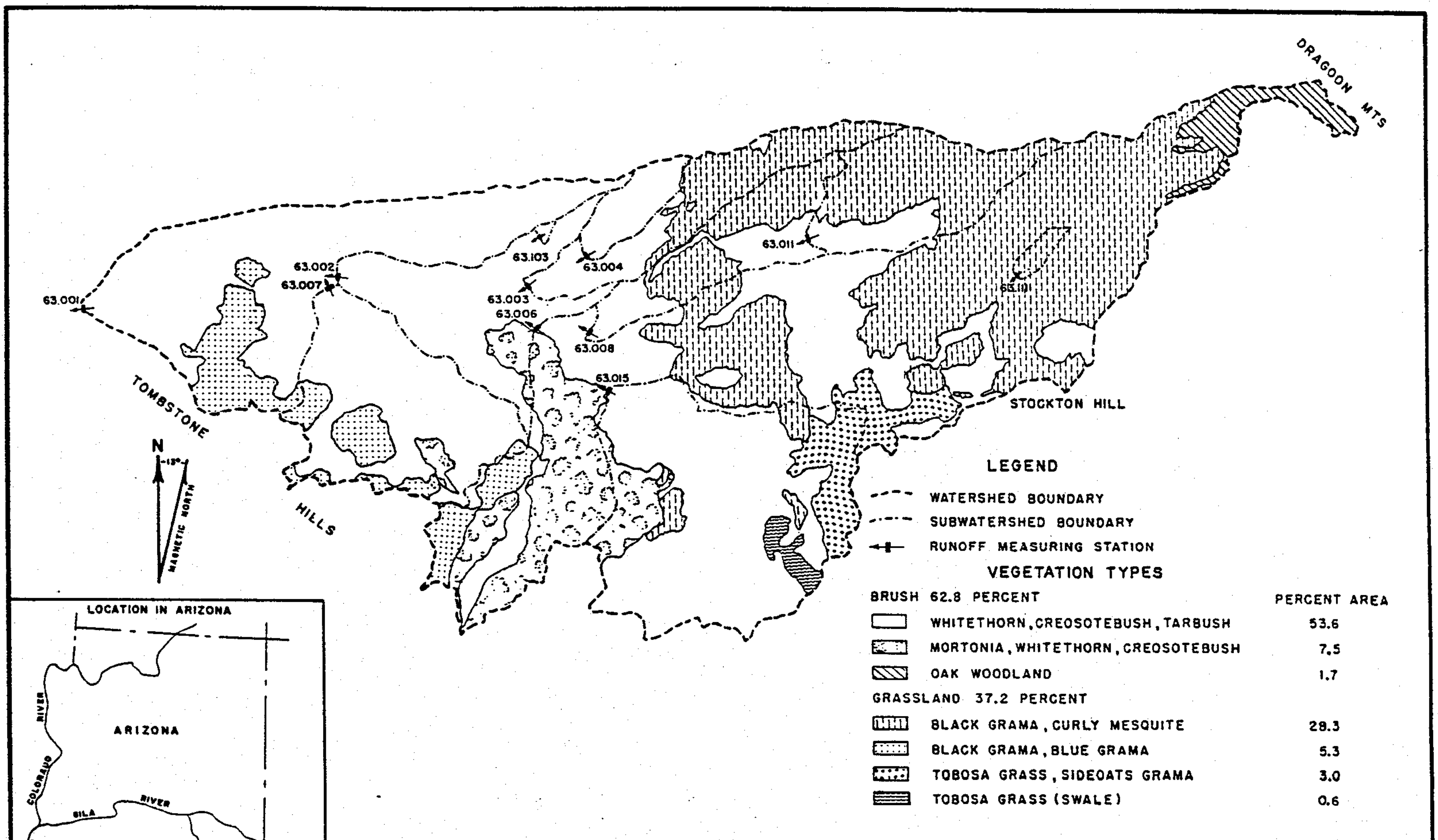
DRAINAGE AREAS

63.001	W-1	36,900 ACRES	57.7 SQUARE MILES
63.002	W-2	28,100 ACRES	43.9 SQUARE MILES
63.003	W-3	2,220 ACRES	3.47 SQUARE MILES
63.004	W-4	560 ACRES	0.88 SQUARE MILES
63.006	W-6	23,500 ACRES	36.7 SQUARE MILES
63.007	W-7	3,340 ACRES	5.22 SQUARE MILES
63.008	W-8	3,830 ACRES	5.98 SQUARE MILES
63.011	W-11	2,035 ACRES	3.18 SQUARE MILES
63.015	W-15	5,912 ACRES	9.24 SQUARE MILES
63.103		8.3 ACRES	
63.111		143 ACRES	

WALNUT GULCH EXPERIMENTAL WATERSHED

TOMBSTONE, ARIZONA

CONTOUR MAP



WALNUT GULCH EXPERIMENTAL WATERSHED
TOMBSTONE, ARIZONA
VEGETATION MAP FOR 63.001

*** O U T P U T   D A T A ***

REVISED JUNE 1988 TO UPDATE COMPUTATION OF SHORT-DURATION VALUES

PRECIPITATION FREQUENCY VALUES FOR WALNUT GULCH
PRIMARY ZONE NUMBER= 7
SHORT-DURATION ZONE NUMBER= 8

LATITUDE 31.60N   LONGITUDE 110.15W   ELEVATION 4400 FEET

POINT VALUES

DURATION	RETURN PERIOD 2-YR	5-YR	10-YR	25-YR	50-YR	100-YR	500-YR	
5-MIN	.40	.47	.52	.60	.67	.73	.87	5-MIN
10-MIN	.59	.71	.80	.92	1.02	1.12	1.34	10-MIN
15-MIN	.72	.89	1.00	1.17	1.30	1.43	1.73	15-MIN
30-MIN	.96	1.19	1.35	1.58	1.76	1.94	2.36	30-MIN
1-HR	1.17	1.46	1.67	1.97	2.20	2.43	2.96	1-HR
2-HR	1.28	1.61	1.84	2.17	2.43	2.69	3.28	2-HR
3-HR	1.36	1.71	1.96	2.32	2.59	2.87	3.50	3-HR
6-HR	1.50	1.90	2.18	2.58	2.89	3.20	3.91	6-HR
12-HR	1.65	2.11	2.44	2.89	3.25	3.60	4.42	12-HR
24-HR	1.80	2.33	2.69	3.20	3.60	4.00	4.92	24-HR

* IF YOUR SITE IS IN ARIZONA OR NEW MEXICO, PLEASE CONSULT THE
  FOLLOWING PAPER FOR REVISED DEPTH-AREA VALUES:
    DEPTH-AREA RATIOS IN THE SEMI-ARID SOUTHWEST UNITED STATES
    NOAA TECHNICAL MEMORANDUM NWS HYDRO-40
    ZEHR AND MYERS
    AUGUST 1984

INPUT DATA

PROJECT NAME=WALNUT GULCH
ZONE= 7   SHORT-DURATION ZONE= 8
LATITUDE= 31.60   LONGITUDE= 110.15   ELEVATION= 4400
2-YR, 6-HR PCPN= 1.50   100-YR, 6-HR PCPN= 3.20
2-YR, 24-HR PCPN= 1.80   100-YR, 24-HR PCPN= 4.00

* * * * E N D   O F   R U N * * * *

Rainfall Intensities for Southeastern Arizona

By Herbert B. Osborn, Member, ASCE[1] and
Kenneth G. Renard, Fellow, ASCE[2]

Introduction

Small watershed storm runoff in the southwestern United States is dominated by intense, short-duration convective rains of limited areal extent. Storm drainage design is often based on rainfall information published by the National Weather Service in the National Oceanic and Atmospheric Administration (NOAA) Atlas 2 series (Miller et al. 1973). In NOAA Atlas 2, short-duration rainfall is derived by an extrapolation procedure from maps of 6-hr and 24-hr rainfall amounts with different frequencies. In this study, intensity-duration-frequency values for 1 hr and less, based on data from a dense network of rain gauges in southeastern Arizona, are compared to similar values derived from NOAA Atlas 2. Differences in rainfall intensities obtained from the two methods are illustrated by simulating and comparing peaks and volumes of runoff.

Previous Studies

In 1973, The National Weather Service (Miller et al. 1973) published an 11-volume atlas for rainfall in the 11 western states. Equations are provided in the publication to estimate 1-hr rainfall from 6-hr and 24-hr rainfall maps for different frequencies. Ratios published in Technical Paper No. 40 (Hershfield 1961) are used to estimate rainfall for 5-, 10-, 15-, and 30-min durations from the 1-hr estimates. Reich (1978) showed the value of computers in developing intensity-duration-frequency relationships from NOAA Atlas 2, but he also warned that computer output was no better than the data from which the estimates were made. Most recently, Petersen (1986) found that estimates for short-duration intensities based on recording rain gauge records near Billings, Montana, were significantly larger for recurrence intervals from 2–100 yrs, than those based on NOAA Atlas 2.

Rainfall Analysis

Data from a dense-recording rain gauge network on the U. S. Dept. of Agric., Agricultural Research Service's Walnut Gulch experimental watershed, in southeastern Arizona, were used to estimate 2-, 5-, 10-, 25-, 50-, and 100-yr rains for 5-, 10-, 15-, 30-, and 60-min durations. Based on the assumption of independent sampling points for well-separated rain gauges (Reich and Osborn 1982) and the station-year method (Hafstad 1942), three sets of four gauges each were selected to create records of 90, 91, and 92

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Note. Discussion open until July 1, 1988. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on March 19, 1987. This paper is part of the ***Journal of Irrigation and Drainage Engineering***, Vol. 114, No. 1, February, 1988. ©ASCE, ISSN 0733-9437/88/0001-0195/$1.00 + $.15 per page. Paper No. 22230.

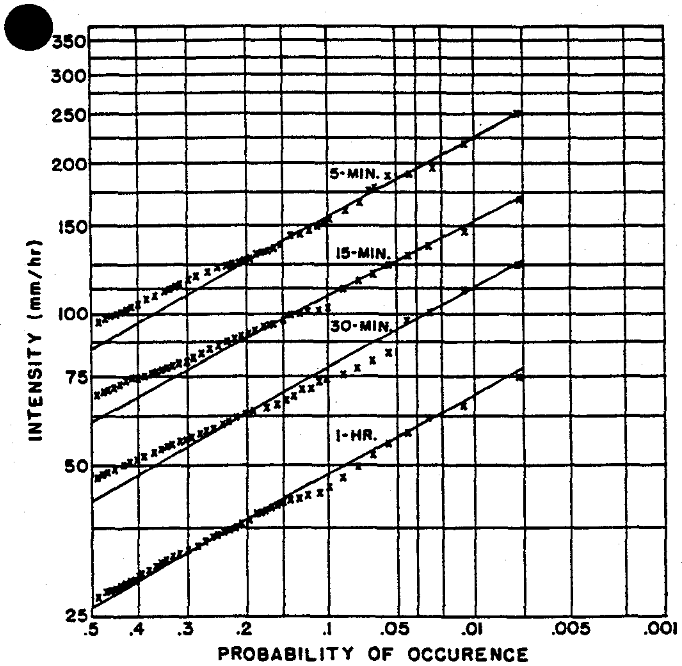

FIG. 1. Intensity-Frequency Relationships for 5-, 15-, 30-min, and 1-hr Durations for Walnut Gulch, Arizona

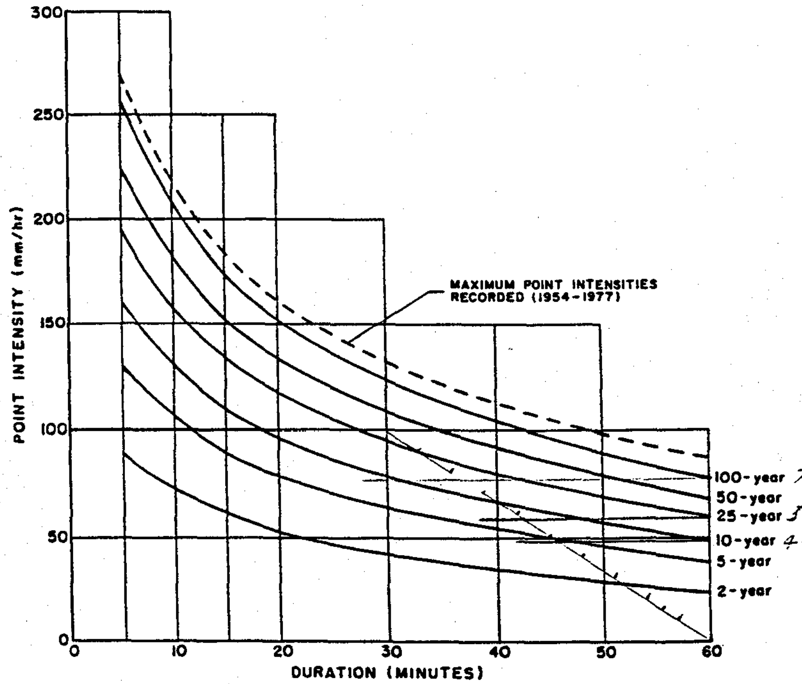

FIG. 2. Point Rainfall Intensities for Durations of 5–60 min.

TABLE 1. Point Depth, Duration, Frequency Rainfall for Southwestern Arizona, Annual Series

Return Period and Duration (min) (1)	Walnut Gulch Data (mm) (2)	inches	National Oceanic and Atmospheric Administration Atlas (mm) (3)	inches
		(*a*) 2 Years		
5	7.0		7.6	
10	11.4		12.0	
15	14.6		15.2	
30	21.0		21.6	
		(*b*) 5 Years		
5	10.2		10.2	
10	17.8		16.5	
15	22.8		21.6	
30	31.8		29.2	
		(*c*) 10 Years		
5	12.0	.47	12.0	.47
10	21.6	.85	20.3	.80
15	28.0	1.10	25.4	1.00
30	38.0	1.50	34.3	1.35
60	49.0	1.93		1.67
		(*d*) 25 Years		
5	16.5	.65	15.2	.60
10	25.4	1.00	22.8	.90
15	34.3	1.35	29.2	1.15
30	48.2	1.90	39.4	1.55
60	58.0	2.28		1.97
		(*e*) 50 Years		
5	19.0		16.5	
10	30.5		25.4	
15	38.0		31.8	
30	55.8		44.4	
		(*f*) 100 Years		
5	21.6	.85	18.0	.71
10	35.6	1.40	28.0	1.10
15	44.4	1.75	35.6	1.40
30	63.5	2.50	48.0	1.89
60	78.0	3.07		2.43

yrs. Twelve different gauges made up the three sets, and the four gauges in each set were separated by at least four mi. Estimates for 5-min, 15-min, 30-min, and 1-hr rainfall were plotted on log probability paper (Fig. 1). The relationships from Fig. 1 were used to derive intensity-duration-frequency curves (Fig. 2). The 6-hr and 24-hr rainfall maps in NOAA Atlas 2, Vol. 8 (Arizona) were used, along with the appropriate equations and ratios, to derive depths for 2-, 5-, 10-, 25-, 50-, and 100-yr rainfall for 5-, 10-, 15-, 30-,

TABLE 2. Comparison of Runoff Peaks and Volumes for a 100-yr, 1-hr Storm Based on Walnut Gulch and NOAA Atlas 2 Estimates

		Peak (cms/ha)				Volume (mm)			
		"Dry"		"Wet"		"Dry"		"Wet"	
Watershed (1)	Area (ha) (2)	Walnut Gulch (3)	National Oceanic and Atmospheric Administration (4)	Walnut Gulch (5)	National Oceanic and Atmospheric Administration (6)	Walnut Gulch (7)	National Oceanic and Atmospheric Administration (8)	Walnut Gulch (9)	National Oceanic and Atmospheric Administration (10)
63105	0.24	0.50	0.35	0.54	0.41	50.0	33.5	62.5	45.7
63011	810	0.12	0.06	0.17	0.12	30.5	15.2	42.0	26.7

and 60-min durations (Table 1). Rainfall depths for the same return periods and durations were derived from Walnut Gulch data and compared to the NOAA Atlas 2 estimate (Table 1). The differences appeared appreciable for the less frequent events, particularly the 50- and the 100-yr storms, as opposed to the Peterson (1986) study in which the difference were appreciable for all recurrence intervals.

Runoff

One method of illustrating the significance of differing estimates of short-duration rainfall intensities is to study the differences in flood peaks and volumes when the rainfall estimates are entered into a mathematical rainfall-runoff model. A kinematic cascade rainfall-runoff model, KINEROS (Rovey et al. 1977), which has been adapted for use on Walnut Gulch (Osborn 1984), was used in this evaluation. KINEROS is a well-tested nonlinear, deterministic, distributed parameter model. Inputs are: (1) Hyetographs of actual or simulated rainfall; (2) the watershed surface geometry and topography; (3) parameters for surface roughness; (4) infiltration parameters (based on Green-Ampt); and (5) the channel network, including slope, cross-sectional area, cross-sectional shape, hydraulic roughness, and a subroutine for channel abstraction (Smith 1981; Osborn 1984). Data from two natural rangeland watersheds were used to validate the model—a very small (0.24 ha, 0.6 ac) watershed, and a large (810 ha, 2,000 ac) watershed. Rainfall was assumed to cover the 0.24 ha watershed evenly, but was varied both in time and space over the 810 ha watershed using an elliptical model based on earlier modeling efforts (Osborn and Laursen 1973). Runoff peaks and volumes were obtained for "wet" and "dry" antecedent conditions for the 100-yr, 1-hr rain (Table 2). "Dry" antecedent conditions normally prevail in southeastern Arizona, but "wet" antecedent conditions, often assumed in engineering design, occur occasionally. Runoff peak and volume estimates based on Walnut Gulch rainfall data were substantially greater than those based on the NOAA Atlas estimates for all durations for the 50- and 100-yr storms, somewhat greater for the 25-yr storms, and substantially the same for the more frequent events.

Conclusions

For southeastern Arizona, estimates of short-duration precipitation intensities, based on NOAA Atlas 2, were substantially lower than

estimates based on data from a dense rain gauges network for less frequent events (50- and 100-year frequencies). Runoff peaks and volumes, as estimated with a distributed mathematical rainfall-runoff model, were underestimated for the less frequent events, particularly for the 100-yr storm, based on NOAA Atlas 2.

Appendix I. References

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Rovey, E. W., Woolhiser, D. A., and Smith, R. E. (1977). "A distributed kinematic model of upland watersheds." *Hydrology Paper 93*, Colorado State University, Ft. Collins, CO.

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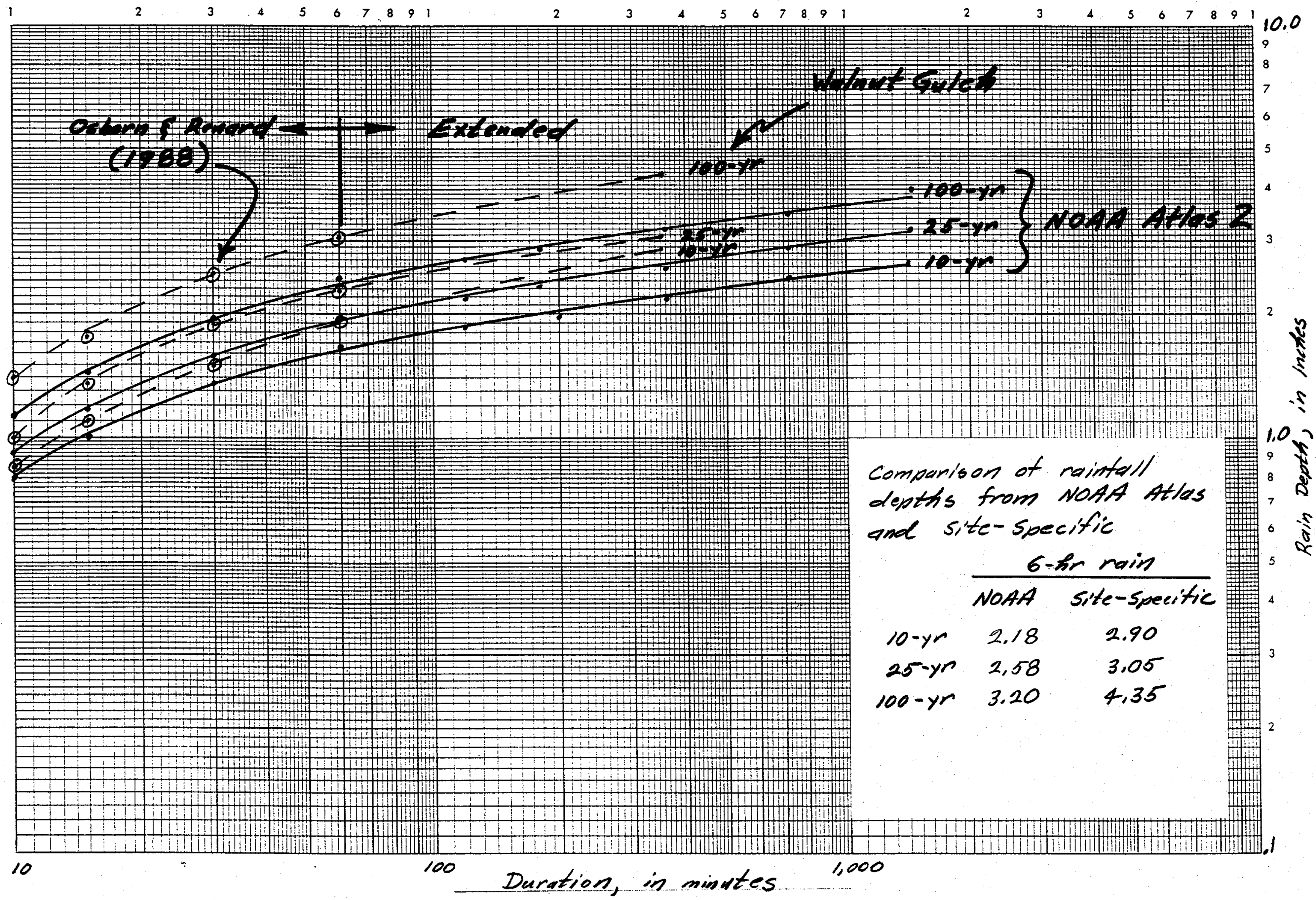
GVS
25 oct 91



Comparison of rainfall depths from NOAA Atlas and site-Specific

	6-hr rain	
	NOAA	Site-Specific
10-yr	2.18	2.90
25-yr	2.58	3.05
100-yr	3.20	4.35

TOMBSTONE, ARIZONA   WALNUT GULCH WATERSHED 63.008

LOCATION: Cochise County, Ariz., 1¼ miles northeast of Tombstone; Walnut Gulch, San Pedro River, Gila River, Colorado River Basin.

AREA: 3,830 acres (5.98 sq. miles)

SLOPES:

Slope - Percent 1/	0-3	3-10	10-20	20-35
Percent of area	4	56	28	12

1/ Estimated

SOILS: Not available

EROSION:

Erosion Class	1	2
Percent of area	98	2

LAND CAPABILITY:

Class	VI
Percent of area	100

GEOLOGY: One hundred percent of the subwatershed consists of Quaternary and Tertiary alluvium of the Tombstone pediment. The alluvium is made up of permeable lensed and interbedded sand, gravel, conglomerate, caliche conglomerate, and some clay. Two series of conglomerate are recognized beneath the recent alluvium of the Tombstone pediment. A younger conglomerate whose bedding is nearly conformable to the pediment surface and probably considerably older than that surface, and an older Tertiary conglomerate lying unconformably beneath that. These conglomerates are known to persist to depths exceeding 1,200 feet. Topographic expression of the alluvium is that of low undulating hills dissected by present stream channels. Caliche conglomerates of the unit are fairly resistant to erosion and form steep cliffs of low relief in some of the present stream channels. The southeast tip and fluvial outlet of the watershed is underlain by the remnant of a highly fractured intrusive basalt plub. The regional watertable is about 425 feet deep.

Stratigraphy and Hydrogeology of Walnut Gulch Subwatershed 63.008

System	Formation and percent of area	Description
Quaternary & Tertiary	Recent alluvium 99%	Gravel, sand, and clay.
	Younger conglomerate < 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.
	Older conglomerate < 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.
	Basalt < 1%	Intrusive olivine basalt plug, secondary calcite vein filling.

Source of data: General Geology of Central Cochise County, Arizona, by James Gilluly, U. S. Geological Survey, Professional Paper 281, 1956, and extended field studies by project staff.

SURFACE DRAINAGE: Good, length of principal waterway is 8.0 miles with 2 major tributaries; a natural watershed with surface flow in well defined water courses; includes gaged watershed 63.011.

CHARACTER OF FLOW: Ephemeral

INSTRUMENTATION: Precipitation: Measured by 11 24-hour weighing rain gages. Runoff: Critical depth flume (precalibrated), AD-35 analog strip chart water level recorder.

WATERSHED CONDITIONS: (Includes Watershed 63.011) Vegetation cover: Approximately one-third of the area is dominated by desert shrubs (whitethorn, creosotebush, tarbush) with a crown spread of approximately 30 percent and an understory of grasses with less than 1 percent basal area. The remaining two-thirds of the area is dominated by grasses (black grama, curly mesquite grass, sideoats grama), with a basal area of about 2.5 percent, interspersed by desert shrubs with a crown spread of about 5 percent.

GENERALLY REPRESENTS: Desert grassland ranges in the Southeastern Arizona Basin and Rangeland Resources Area (D-41).

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BY JMR

SUBJECT Single Basin Testing

PROJECT NO. 15

WALNUT GULCH 63.008

Area = 5.98 mi^2

$L = 8.0$ mi
$L_{ca} = 3.6$ mi
$S = 75$ ft/mi

USBR - LAG

$k_n = 0.03$

$$Lag = 26(0.03)\left(\frac{(8.0)(3.6)}{\sqrt{75}}\right)^{.33} = 1.16 \text{ hrs.}$$

Green-Ampt Parameters (see attached tables) from SCS soil survey:

IA = 0.15
DTHETA = .25
PSIF = 5.8
XKSAT = .22
IMP = 0

RESULTS:

	A	B	C
$k_n = 0.03$	4169	5962	4459

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PROJECT Hydrology Manual Testing
SUBJECT Soils 63.008

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PROJECT NO. S-15

Surface Retention: IA Natural Hillslope - Sonoran Desert = 0.15

MAP UNIT	% MAP UNIT	Soil Name	Textural Class	DTHETA (normal)	PSIF	XKSAT
HbC	70	Hathaway	loam	.25	3.5	.25
	25	Bernardino	clay loam	.15	8.2	0.04
	5	Misc.	sandy loam	.25	4.3	0.4
HnC	60	Hathaway	loam	.25	3.5	.25
	35	Nickel	loam			
	5	Kimbrough	loam			
HbB	60	Hathaway	loam	.25	3.5	.25
	30	Bernardino	clay loam	.15	8.2	0.04
	10	Nickel	loam	.25	3.5	.25
		Sonoita	sandy loam	.25	4.3	0.4
BhC	60	Bernardino	clay loam	.15	8.2	0.04
	35	Hathaway	loam	.25	3.5	.25
	5	Misc.	loamy sand	.30	2.4	1.2
BeB	95	Bernardino	clay loam	.15	8.2	0.04
	5	Misc.	sandy loam	.25	4.3	0.4
RcB	60	Rillito	loam	.25	3.5	.25
	35	Cave	loam			
	5	Misc.	loam			
* RlB	70	Rillito	loam	.25	3.5	.25
	25	Laveen	sandy loam	.25	4.3	0.4
	5	Misc.	sandy loam			
RlC	75	Rillito	loam	.25	3.5	.25
	20	Laveen	sandy loam	.25	4.3	0.4
	5	Misc.	sandy loam			

XKSAT log = .16

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SUBJECT Soils 63.008

SHEET ____ OF ____
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PROJECT NO. ____

* not in study area

MAP UNIT	% MAP UNIT	Soil Name	Textural Class	DTHETA (normal)	PSIF	XKSAT
BhB	70 30	Bernardino Hathaway	clay loam loam	.15 .25	8.2 3.5	0.04 .25
Lo	100	loamy Alluvial	sandy loam	.25	4.3	0.4
Co	100	Corona	sandy loam	.25	4.3	0.4
So	100	Sonoita	sandy loam	.25	4.3	0.4

Weighted Parameters — Log

HbB
DTHETA = .70(.35) + .3(.25) = .32 — .32
PSIF = .65(3.5) + .3(8.2) + .05(4.3) = 2.68 — 4.57
XKSAT = .65(.25) + .3(0.04) + .05(.4) = 0.19 — 0.15 ✓

BhC
DTHETA = .6(.25) + .4(.35) = .29 — .29
PSIF = .6(8.2) + .35(3.5) + .05(2.4) = 6.27 — 5.72
XKSAT = .6(.04) + .35(.25) + .05(1.2) = 0.17 — 0.09 ✓

BeB
DTHETA = .95(.25) + .05(.35) = 0.26
PSIF = .95(8.2) + .05(4.3) = 8.01
XKSAT = .95(.04) + .05(.4) = 0.06

?

* RlB
DTHETA = .35
PSIF = .7(3.5) + .3(4.3) = 3.74
XKSAT = .7(.25) + .3(.4) = 0.30

RlC
DTHETA = .35 — .35
PSIF = .75(3.5) + .25(4.3) = 3.70 — 3.68
XKSAT = .75(.25) + .25(.4) = 0.29 — 0.28 ✓

Lo, Co, So =
DTHETA = .35
PSIF = 4.3
XKSAT = 0.4 ✓

RcB
DTHETA = .35
PSIF = 3.5
XKSAT = .25 ✓

BhB
D = .7(.25) + .3(.35) = 0.28 — .28
P = .7(8.2) + .3(3.5) = 6.79 — 6.35
X = .7(.04) + .3(.25) = 0.10 — 0.07 ✓

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PROJECT Hydrology Manual Testing

BY JMR

SUBJECT Soils 63.008

PROJECT NO. 15

Weighted for Single Basin:

Soil	AREA	%
HbC	2.96	.49
HnC	1.11	.19
So, Co, Lo	.55	.09
HbB	.57	.10
RIC	.30	0.05
BhC	.24	.04
BhB	.1	.02
RcB	.08	.01
BcB	.07	.01
Total	5.98	100

DTHETA = .49(.33) + .19(.35) + .09(.35) + .1(.32) + .05(.35) + .04(.29) + .02(.28) + .01(.35) + .01(.26)
= .49(.33) + .34(.35) + .1(.32) + .04(.29) + .02(.28) + .01(.26)
= .33

PSIF = .49(4.72) + .19(3.5) + .09(4.3) + .1(2.68) + .05(3.7) + .04(6.27) + .02(6.79) + .01(3.5) + .01(8.01)
= 4.32

XKSAT = .49(.21) + .19(.25) + .09(.4) + .1(.19) + .05(.29) + .04(.17) + .02(.1) + .01(.25) + .01(.06)
= .23

Vegetative cover adjustment - 30% 1.22(.23) = 0.28

Log. Average for Area:

DTHETA = [.49(log .32) + .19(log .35) + .09(log .35) + .1(log .32) + .05(log .35) + .04(log .29) + .02(log .28) + .01(log .35) + .01(log .25)] = 0.33 No

PSIF = [.49(log 4.37) + .19(log 3.5) + .09(log 4.3) + .1(log 4.57) + .05(log 3.68) + .04(log 5.72) + .02(log 6.35) + .01(log 3.5) + .01(log 7.94)] = 4.26 No

XKSAT = antilog [.49(log .16) + .19(log .25) + .09(log .4) + .1(log .15) + .05(log .28) + .04(log .09) + .02(log .07) + .01(log .25) + .01(log .04)] = 0.18 OK

Vegetative cover adjustment - 30% 1.22(.18) = .22 OK

Frequency analysis for input data from file: 63008.DAT
Station Name: WALNUT GULCH 63.008
Units: CFS
Regional Skew: None
27 data points used in the analysis

PLOTTING POSITIONS

Month	Day	Year	Magnitude	Gringorten	Cunnane	Weibull	Hazen
7	12	64	4053.300	0.021	0.022	0.036	0.019
8	27	82	3392.100	0.058	0.059	0.071	0.056
7	12	75	2297.600	0.094	0.096	0.107	0.093
7	7	67	1797.900	0.131	0.132	0.143	0.130
1	22	77	1787.600	0.168	0.169	0.179	0.167
7	23	71	1280.800	0.205	0.206	0.214	0.204
7	14	65	1022.800	0.242	0.243	0.250	0.241
8	1	78	951.000	0.279	0.279	0.286	0.278
9	20	83	932.500	0.316	0.316	0.321	0.315
8	10	86	929.200	0.353	0.353	0.357	0.352
3	6	70	743.200	0.389	0.390	0.393	0.389
7	31	63	740.900	0.426	0.426	0.429	0.426
7	20	66	671.000	0.463	0.463	0.464	0.463
7	27	76	641.400	0.500	0.500	0.500	0.500
8	4	80	628.100	0.537	0.537	0.536	0.537
7	2	68	569.200	0.574	0.574	0.571	0.574
8	22	87	529.400	0.611	0.610	0.607	0.611
7	12	73	380.700	0.647	0.647	0.643	0.648
8	16	84	346.300	0.684	0.684	0.679	0.685
7	19	74	214.200	0.721	0.721	0.714	0.722
7	31	81	201.900	0.758	0.757	0.750	0.759
6	6	72	191.600	0.795	0.794	0.786	0.796
7	17	69	167.200	0.832	0.831	0.821	0.833
8	16	89	157.100	0.869	0.868	0.857	0.870
8	20	88	152.100	0.906	0.904	0.893	0.907
7	17	85	140.100	0.942	0.941	0.929	0.944
8	18	79	26.000	0.979	0.978	0.964	0.981

Arithmetic Mean: 923.89630
Standard Deviation: 987.72526
Skew Coefficient: 1.95800
Coefficient of Variation: 1.06909

Arithmetic Mean of Logs: 2.73678
Standard Deviation of Logs: 0.49318
Skew Coefficient of Logs: -0.49692
Coefficient of Variation of Logs: 0.18020

MAGNITUDES FOR SELECTED PROBABILITIES

Probability	Extreme Value	Log Extreme Value	Station LP III	Log Normal
0.99000	-765.82	78.10	17.26	29.26
0.90000	-205.10	148.96	121.42	127.21
0.80000	84.63	207.84	217.91	209.66
0.50000	766.08	454.96	599.39	545.48
0.42920	937.60	554.14	729.51	666.91
0.20000	1682.96	1305.52	1441.91	1419.16
0.10000	2290.02	2623.57	2170.11	2339.01
0.04000	3057.04	6336.82	3232.87	3984.12
0.02000	3626.06	12189.51	4103.51	5620.38
0.01000	4190.87	23334.78	5022.75	7654.39
0.00400	4934.54	54868.81	5975.82	10167.30
0.00200	5496.08	104641.18	7289.61	14326.69

Work Sheet for Log-Normal Confidence Limits

Watershed Walnut Gulch 63.008

Station No. ____

$Q_{2\text{-yr}}$ 550 cfs

$Q_{100\text{-yr}}$ 6200 cfs

Confidence Level (C.L.) 90 %

$a = \frac{100 - C.L.}{100}$ 0.10

$u_{1-a/2}$ 1.645

N 27

$\bar{Y} = \log_{10}(Q_{2\text{-yr}}) = \log_{10}(550) =$ 2.74

$S_y = \dfrac{\log_{10} Q_{100\text{-yr}} - \log_{10} Q_{2\text{-yr}}}{2.327} = \dfrac{\log_{10}(6200) - \log_{10}(550)}{2.327}$ 0.452

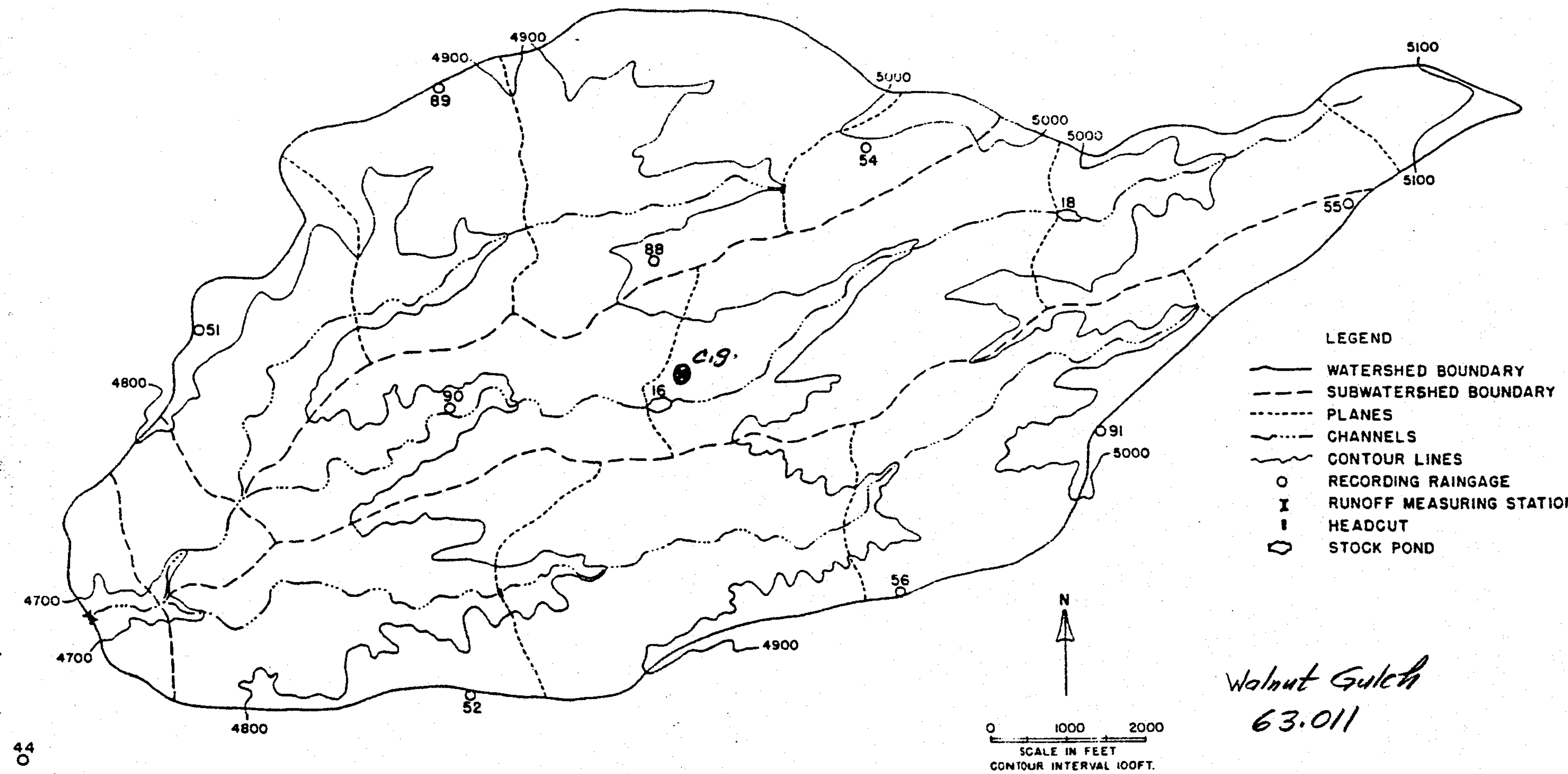
T years (1)	$u_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits (c) Upper (5)	Lower (6)
2	0.0	2.74	0.0869	764	395
5	.842	3.12	0.1012	1934	899
10	1.282	3.32	0.1174	3259	1339
25	1.751	3.53	0.1384	5724	2006
50	2.052	3.67	0.1532	8356	2618
100	2.327	3.79	0.1674	11624	3270

(a) $Y_T = \bar{Y} + u_{1-\frac{1}{T}} S_Y$

(b) $S_T = \left[\left(S_Y^2/N\right)\left(1 + .5\, u^2_{1-\frac{1}{T}}\right)\right]^{1/2}$

(c) $Q_L = 10^{\left(Y_T \pm u_{1-a/2} S_T\right)}$

Watershed and Streamgage Data for
Walnut Gulch 63.011



Walnut Gulch
63.011

LOCATION: Cochise County, Ariz.; 4 1/3 miles northeast of Tombstone; Walnut Gulch, San Pedro River, Gila River, Colorado River Basin.

AREA: 2,035 acres (3.18 sq. miles)

SLOPES:

Slope - Percent	0-3	3-10	10-20	20-35
Percent of area 1/	4	52	28	16

1/ Estimated

SOILS: Not available

EROSION:

Erosion Class	1	2
Percent of area	98	2

LAND CAPABILITY:

Class	VI
Percent of area	100

GEOLOGY: One hundred percent of the subwatershed consists of Quaternary and Tertiary alluvium of the Tombstone pediment. The alluvium is made up of permeable lensed and interbedded sand, gravel, conglomerate, caliche conglomerate, and some clay. Two series of conglomerate are recognized beneath the recent alluvium of the Tombstone pediment. A younger conglomerate whose bedding is nearly conformable to the pediment surface and probably older than that surface, and an older Tertiary conglomerate lying unconformably beneath that. These conglomerates are known to persist to depths exceeding 1,200 feet. Topographic expression of the alluvium is that of low undulating hills dissected by present stream channels. Caliche conglomerates of this unit are fairly resistant to erosion and form steep cliffs of low relief in some of the present stream channels. The regional watertable is about 425 feet deep.

Stratigraphy and Hydrogeology of Walnut Gulch Subwatershed 63.011

System	Formation and percent of area	Description
Quaternary & Tertiary	Recent alluvium 99%	Gravel, sand, and clay.
	Younger conglomerate < 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.
	Older conglomerate < 1%	Gravel, sand, conglomerate, caliche conglomerate, and clay, some boulders.

Source of data: General Geology of Central Cochise County, Arizona, by James Gilluly, U. S. Geological Survey, Professional Paper 281, 1956 and extended field studies by project staff.

SURFACE DRAINAGE: Good, length of principal waterway is 4.0 miles with 2 major tributaries; a natural watershed with surface flow in well defined water courses.

CHARACTER OF FLOW: Ephemeral.

INSTRUMENTATION: Precipitation: Measured by 5 24-hour weighing rain gages. Runoff: Critical depth flume (precalibrated) AD-35 analog strip chart water level recorder.

WATERSHED CONDITIONS: Vegetation cover: Approximately 20 percent of the area dominated by desert shrubs (whitethorn, creosotebush, tarbush) with a crown spread of approximately 30 percent cover and an understory of grasses with basal area of less than 1 percent. The remaining 80 percent of the area supports a grass cover (black grama, curly mesquite grass, sideoats grama) with a basal cover of about 2.5 percent interspersed with desert shrubs averaging less than 5 percent crown cover.

GENERALLY REPRESENTS: Desert grassland ranges in the Southeastern Arizona Basin and Rangeland resources area (D-41).

WALNUT GULCH 63.011

Area = 3.18 mi^2

L = 4.0 mi
L_{ca} = 1.8 mi
S = 100 ft/mi

USBR-LAG

K_n = 0.03

$$Lag = 26(0.03)\left(\frac{(4.0)(1.8)}{\sqrt{100}}\right)^{.33} = 0.7 \text{ hrs.}$$

Green-Ampt Parameters (see attached tables) from SCS soil surveys:

IA = 0.15
DTHETA = 0.25
PSIF = 5.8
XKSAT = .21
IMP = 0

RESULTS:

	A	B	C
k_n = 0.03	3529	5302	4862

A = SCS-UH USBR-LAG
B = SCS-UH LAG = .6TC
C = Clark-UH TC = MC

100 yr - 24 hr Hypothetical Rainfall
NOAA Atlas 2 Depth-Area Reduction
Green-Ampt Losses

GEORGE V. SABOL CONSULTING ENGINEERS, INC.

CLIENT ADOT / ATRC
PROJECT Hydrology Manual Testing
SUBJECT Soils 63,011

SHEET ____ OF ____
DATE 4 Sept. 91
BY JMR
PROJECT NO. 5-15

Surface Retention: 1A Natural Hillslopes / Sonoran Desert 0.15

MAP UNIT	% MAP UNIT	Soil name	Textural Class	DTHETA	PSIF	XKSAT
HbC	70	Hathoway	loam	.35	3.5	.25
	25	Barnardino	clay loam	.25	8.2	0.04
	5	Misc.	sandy loam	.35	4.3	0.4
HnC	60	Hathoway	loam	.35	3.5	.25
	35	Nickel	loam			
	5	Kimbrough	loam			
Lo	100	Loamy alluvial	sandy loam	.35	4.3	0.4

Weighted Parameters:

		Log
HbC	DTHETA = .75(.35) + .25(.25) = 0.33	0.32
	PSIF = .7(3.5) + .25(8.2) + .05(4.3) = 4.72	4.37
	XKSAT = .7(.25) + .25(.04) + .05(.4) = 0.21	0.16

HnC DTHETA = .35
PSIF = 3.5
XKSAT = .25

FIG 4.3
FOR XKSAT = 0.16 :
PSIF = 6.0 in
$d\theta$ = .395

Lo DTHETA = .35
PSIF = 4.3
XKSAT = 0.4

Weighted for Single Basin:

HbC = 2.23 mi² .70%
HnC = 0.92 mi² .29%
Lo = 0.03 mi² .01%
3.18

DTHETA = .7(.33) + .30(.35) = 0.34	0.33
PSIF = .7(4.72) + .29(3.5) + .01(4.3) = 4.36	4.10
XKSAT = .7(.21) + .29(.25) + .01(0.4) = 0.22	0.18
Vegetation Cover Adjustment: XKSAT = 0.22(1.17) = .26	0.21

8-15% slopes assume vegetation cover 25%
3-8% slopes assume vegetation cover 50%

Frequency analysis for input data from file: 63011.DAT
Station Name: WALNUT GULCH 63.011
Units: CFS
Regional Skew: None
27 data points used in the analysis

PLOTTING POSITIONS

Month	Day	Year	Magnitude	Gringorten	Cunnane	Weibull	Hazen
7	12	64	4381.000	0.021	0.022	0.036	0.019
8	27	82	3355.200	0.058	0.059	0.071	0.056
1	22	77	2879.200	0.094	0.096	0.107	0.093
7	7	67	1703.700	0.131	0.132	0.143	0.130
7	23	75	1175.200	0.168	0.169	0.179	0.167
7	14	65	1044.700	0.205	0.206	0.214	0.204
7	20	66	955.300	0.242	0.243	0.250	0.241
8	4	80	894.000	0.279	0.279	0.286	0.278
8	2	68	875.200	0.316	0.316	0.321	0.315
7	31	63	751.600	0.353	0.353	0.357	0.352
8	29	86	547.400	0.389	0.390	0.393	0.389
2	5	76	517.900	0.426	0.426	0.429	0.426
3	19	73	502.000	0.463	0.463	0.464	0.463
7	20	70	492.400	0.500	0.500	0.500	0.500
8	1	78	457.600	0.537	0.537	0.536	0.537
7	23	71	433.400	0.574	0.574	0.571	0.574
7	15	81	345.800	0.611	0.610	0.607	0.611
9	20	83	311.000	0.647	0.647	0.643	0.648
7	28	69	262.100	0.684	0.684	0.679	0.685
8	16	89	216.800	0.721	0.721	0.714	0.722
7	17	85	214.500	0.758	0.757	0.750	0.759
7	3	74	175.300	0.795	0.794	0.786	0.796
10	20	88	77.400	0.832	0.831	0.821	0.833
8	16	84	65.200	0.869	0.868	0.857	0.870
6	6	72	64.900	0.906	0.904	0.893	0.907
7	15	87	31.500	0.942	0.941	0.929	0.944
6	7	79	9.500	0.979	0.978	0.964	0.981

Arithmetic Mean: 842.21481
Standard Deviation: 1072.20404
Skew Coefficient: 2.20068
Coefficient of Variation: 1.27308

Arithmetic Mean of Logs: 2.59971
Standard Deviation of Logs: 0.61702
Skew Coefficient of Logs: -0.68427
Coefficient of Variation of Logs: 0.23734

Regional Skew Coefficient: None
Weighted Skew Coefficient: None

MAGNITUDES FOR SELECTED PROBABILITIES

Probability	Extreme Value	Log Extreme Value	Station LP III	Log Normal
0.99000	-992.02	35.01	4.08	10.24
0.90000	-383.34	78.42	59.87	64.37
0.80000	-68.84	118.97	129.50	120.28
0.50000	670.90	317.05	469.12	397.84
0.42920	857.09	405.76	596.43	511.59
0.20000	1666.20	1185.48	1344.31	1315.96
0.10000	2325.18	2838.67	2136.23	2458.86
0.04000	3157.80	8555.87	3299.57	4787.59
0.02000	3775.49	19396.52	4224.92	7363.34
0.01000	4388.61	43707.81	5176.70	10836.90
0.00400	5195.89	127386.78	6138.97	15458.26
0.00200	5805.45	285698.16	7394.79	23741.13

George V. Sabol Consulting Engineers, Inc.

SHEET ______ OF ______

CLIENT ______

DATE Revised 15 Oct 91

PROJECT ______

BY GVS

SUBJECT FIGURE 1-9

PROJECT NO. ______

Work Sheet for Log-Normal Confidence Limits

Watershed Walnut Gulch 63.011

Station No. 09471120

$Q_{2\text{-}yr}$ 445 cfs

$Q_{100\text{-}yr}$ 6500 cfs

Confidence Level (C.L.) 90 %

$a = \frac{100 - C.L.}{100}$ 0.10

$u_{1-a/2}$ 1.645

N 26

$\bar{Y} = \log_{10}(Q_{2\text{-}yr}) = \log_{10}(\ 445\ ) =$ 2.6484

$S_y = \frac{\log_{10} Q_{100\text{-}yr} - \log_{10} Q_{2\text{-}yr}}{2.327} = \frac{\log_{10}(6500) - \log_{10}(445)}{2.327}$ 0.5004

T years (1)	$u_{1-\frac{1}{T}}$ (2)	Y_T (a) (3)	S_T (b) (4)	Limits (c) Upper (5)	Limits (c) Lower (6)
2	0.0	2.6484	0.0981	645	307
5	.842	3.0697	0.1142	1810	762
10	1.282	3.2899	0.1324	3219	1181
25	1.751	3.5246	0.1561	6045	1853
50	2.052	3.6752	0.1729	9112	2459
100	2.327	3.8128	0.1889	13290	3177

(a) $Y_T = \bar{Y} + u_{1-\frac{1}{T}} S_Y$

(b) $S_T = \left[(S_Y^2/N)(1 + .5\, u^2_{1-\frac{1}{T}}) \right]^{1/2}$

(c) $Q_L = 10^{(Y_T \pm u_{1-a/2} S_T)}$

PC Shell Version 7 Directory Print ID = None

Path=B:\

	Name	Ext	Size	#Clu	Date	Time	Attributes		
	AA10	DAT	818	2	10-21-91	11:40a	A		
Academy	AA10	OUT	20065	40	10-21-91	11:40a	A	Single - Basin	
	AA100	DAT	819	2	10-21-91	11:46a	A	Clark u-hg	
Acres	AA100	OUT	19963	39	10-21-91	11:46a	A		
	AA25	DAT	818	2	10-21-91	11:42a	A		
	AA25	OUT	20015	40	10-21-91	11:43a	A		
Walnut Gulch 63.008	WG11A10	DAT	831	2	10-21-91	1:28p	A	Single - Basin	
	WG11A10	OUT	8443	17	10-21-91	1:28p	A		
	WG11A100	DAT	831	2	10-21-91	1:29p	A	Clark u-hg	
	WG11A100	OUT	8443	17	10-21-91	1:30p	A		
	WG11A25	DAT	831	2	10-21-91	1:29p	A		NOAA
	WG11A25	OUT	8443	17	10-21-91	1:29p	A		
	WG11B10	DAT	1145	3	11-11-91	12:10p	A		statistics
	WG11B10	OUT	8880	18	11-11-91	12:10p	A	Single-Basin	
	WG11B100	DAT	1144	3	11-11-91	12:20p	A	S-graph	
	WG11B100	OUT	8880	18	11-11-91	12:20p	A		
	WG11B25	DAT	1145	3	11-11-91	12:14p	A		
	WG11B25	OUT	8880	18	11-11-91	12:15p	A		
	WG11C10	DAT	862	2	10-25-91	10:36a	A	Single - Basin	
	WG11C10	OUT	8443	17	10-25-91	10:37a	A		
	WG11C100	DAT	791	2	10-25-91	10:24a	A	Clark u-hg	Walnut Gulch
	WG11C100	OUT	32785	65	10-25-91	10:24a	A		
	WG11C25	DAT	791	2	10-25-91	10:31a	A		site-specific
	WG11C25	OUT	8443	17	10-25-91	10:31a	A		
	WG11D10	DAT	1159	3	11-11-91	12:06p	A		statistics
	WG11D10	OUT	9097	18	11-11-91	12:07p	A	Single-Basin	
	WG11D100	DAT	1160	3	11-11-91	12:25p	A	s-graph	
	WG11D100	OUT	8880	18	11-11-91	12:25p	A		
	WG11D25	DAT	1163	3	11-11-91	12:12p	A		
	WG11D25	OUT	8880	18	11-11-91	12:12p	A		

Walnut Gulch 63.011

WG8A10	DAT	831	2	10-21-91	1:21p	A	Single-Basin
WG8A10	OUT	8443	17	10-21-91	1:21p	A	Clark u-hg
WG8A100	DAT	832	2	10-21-91	1:20p	A	
WG8A100	OUT	8443	17	10-21-91	1:20p	A	
WG8A25	DAT	831	2	10-21-91	1:21p	A	NOAA
WG8A25	OUT	8443	17	10-21-91	1:22p	A	Statistics
WG8B10	DAT	1227	3	11-11-91	11:42a	A	
WG8B10	OUT	9207	18	11-11-91	11:43a	A	Single-Basin
WG8B100	DAT	1215	3	11-11-91	11:30a	A	S-graph
WG8B100	OUT	8990	18	11-11-91	11:30a	A	
WG8B25	DAT	1223	3	11-11-91	11:47a	A	
WG8B25	OUT	8990	18	11-11-91	11:48a	A	
WG8C10	DAT	862	2	11-25-91	10:17a	A	Single-Basin
WG8C10	OUT	8443	17	11-25-91	10:18a	A	Clark u-hg
WG8C100	DAT	791	2	10-25-91	10:14a	A	
WG8C100	OUT	33417	66	10-25-91	10:14a	A	Walnut Gulch
WG8C25	DAT	778	2	11-25-91	10:14a	A	Site-Specific
WG8C25	OUT	8443	17	11-25-91	10:15a	A	statistics
WG8D10	DAT	1182	3	11-11-91	11:40a	A	
WG8D10	OUT	8990	18	11-11-91	11:40a	A	Single-Basin
WG8D100	DAT	1214	3	11-11-91	4:24p	A	
WG8D100	OUT	8990	18	11-11-91	4:25p	A	S-graph
WG8D25	DAT	1244	3	11-11-91	11:45a	A	
WG8D25	OUT	8990	18	11-11-91	11:46a	A	
WG8E10	DAT	1339	3	11-12-91	8:42a	A	
WG8E10	OUT	12626	25	11-12-91	8:43a	A	Multi-Basin NOAA
WG8E100	DAT	2065	5	11-11-91	8:59a	A	Clark u-hg statistics
WG8E100	OUT	26104	51	11-11-91	9:00a	A	
WG8E25	DAT	1337	3	11-11-91	4:14p	A	
WG8E25	OUT	12626	25	11-11-91	4:14p	A	
WG8F10	DAT	1354	3	11-12-91	8:33a	A	
WG8F10	OUT	12626	25	11-12-91	8:34a	A	Multi-Basin Walnut Gulch
WG8F100	DAT	2101	5	11-12-91	9:20a	A	Site-Specific
WG8F100	OUT	466556	912	11-12-91	9:23a	A	Clark u-hg statistics
WG8F25	DAT	1352	3	11-11-91	4:07p	A	
WG8F25	OUT	12626	25	11-11-91	4:08p	A	

66 files LISTed 897,579 byte 66 files in subdir = 897,579 byte

s 0 files SELECTed 0 bytes Available on volume = 542,720 byte

GEORGE V. SABOL CONSULTING ENGINEERS, INC.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

FLOOD CONTROL DISTRICT
RECEIVED
DEC 13 1991

CH ENG		P & PM	
		HYDRO	
ADMIN		LMGT	
FINANCE		FILE	
C & O			
ENGR			

REMARKS

9 December 1991

Mr. Stephen D. Waters
Flood Control District of Maricopa County
2801 West Durango
Phoenix, Arizona 85009

Subject: Event Simulation for storm of 4 Aug 1980 over Walnut Gulch 63.011 (WG11)

Dear Steve:

Enclosed is a brief report on the event simulation that I performed for WG11 using data that was supplied by the USDA-ARS in Tucson. The results are surprisingly good -almost too good! As I informed you on the phone, all of the model input was selected based on the procedures as described. The initial model input was not changed after the initial (and final) run of the model. Therefore, none of the model input was adjusted after initial selection to improve the model performance. The HEC-1 model files are SG.DAT and SG.OUT.

Using the same model and inputing the Maricopa County 6-hr rainfall pattern, the peak discharge is about 5,400 cfs. This compares with the previously reported (single-basin) model results of 4,200 cfs. Therefore, the 8 subbasin model yields results that are similar to the single-basin model. The HEC-1 model files for this are SG100.DAT and SG100.OUT.

A disk is enclosed with the HEC-1 files and the ARS data for WG11.

We should discuss how these additional verification results are to be presented when we meet next week.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

George V. Sabol

Enclosure: Report w/disk

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

VERIFICATION REPORT

EVENT SIMULATION FOR

WALNUT GULCH 63.011

4 August 1980 Storm

George V. Sabol Consulting Engineers, Inc.
Brighton, Colorado and Phoenix, Arizona

December 1991

raingage records that were used as input to each subbasin are as follows:

Subbasin	Raingage
1	#51
2	Composite average of #88, #89, and #90
3	#54
4	Composite average of #88, #89, and #90
5	Composite average of #54 and #91
6	#55
7	Composite average of #52 and #56
8	#91

WATERSHED CHARACTERISTICS

The Green and Ampt infiltration parameters were assumed to be the same for each subbasin as were used for the entire watershed as reported previously (November 1991). Those parameter values are:

IA	.35 inch
DTHETA	.39
PSIF	5.8 inches
XKSAT	.22 in/hr
RTIMP	0%

The watershed characteristics that were used to develop the subbasin unit hydrographs are shown in Table 1. The Phoenix Valley S-graph was used as the unit hydrograph. The Clark unit hydrograph could not be used because the rainfall excess duration was less than T_c for each of the subbasins. The routing parameters are shown in Table 2. Kinematic routing of the subbasin hydrographs was used.

RESULTS AND DISCUSSION

The multi-basin model with input as described resulted in 0.26 inches of storm runoff with a peak discharge of 886 cfs at time 14:05 (Figure 11). The recorded runoff had 0.23 inches of runoff with a peak discharge of 894 cfs at time 13:47 (Figure 1).

The results are exceptionally good. Some runoff volume would be lost by channel transmission, which was not modeled, and this would account for some or all of the overestimation of the runoff volume. Although incorporation of transmission loss into the model would result in some reduction in runoff volume, the peak discharge would only be reduced by a small amount.

The results of the model seem to validate the rainfall loss procedure and indicates that the Phoenix Valley S-graph can be used for small, desert rangeland watersheds in Arizona.

TABLE 1

Watershed Characteristics and Unit Hydrograph Parameters

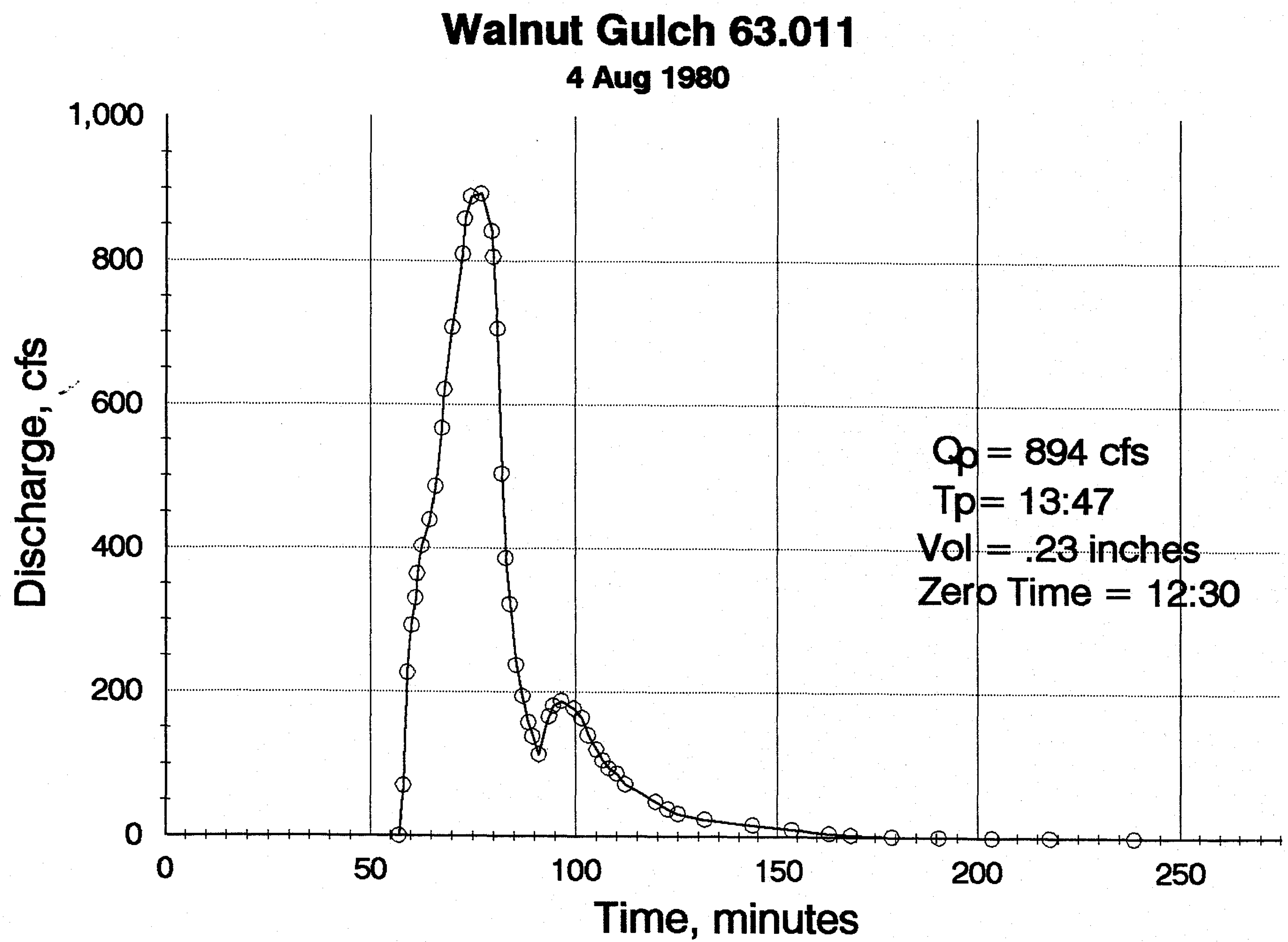
Subbasin	Area sq. mi.	L miles	L_{ca} miles	S ft/mi	K_b	K_n	Lag min.
1	.31	1.0	.4	75	.09	.03	13
2	.56	1.2	.6	75	.09	.03	17
3	.20	.6	.2	75	.10	.03	8
4	.78	1.8	.9	75	.08	.03	23
5	.21	.8	.4	.75	.10	.03	12
6	.21	.9	.4	75	.10	.03	13
7	.62	1.8	.9	75	.09	.03	23
8	.23	.8	.4	75	.10	.03	12

$$Lag = 24\ K_n\ \left(\frac{LL_{ca}}{S^{1/2}}\right)^{.38} * 60$$

TABLE 2

Routing Parameters

Reach	L ft	S ft/ft	n	shape	WD ft	side slope 1H:<u>Z</u>V
G-F	9500	.014	.03	TRAP	15	10
F-A	3000	.014	.03	TRAP	20	10
H-E	4200	.014	.03	TRAP	10	10
E-B	9500	.014	.03	TRAP	20	10
D-C	6300	.014	.03	TRAP	20	10
C-B	2300	.014	.03	TRAP	30	10
B-A	2700	.014	.03	TRAP	30	10



Walnut Gulch 63.011
4 Aug 1980
1,000
800
600
400
200
0
Discharge, cfs
0
50
100
150
200
250
Time, minutes
Qp = 894 cfs
Tp= 13:47
Vol = .23 inches
Zero Time = 12:30

FIGURE 1

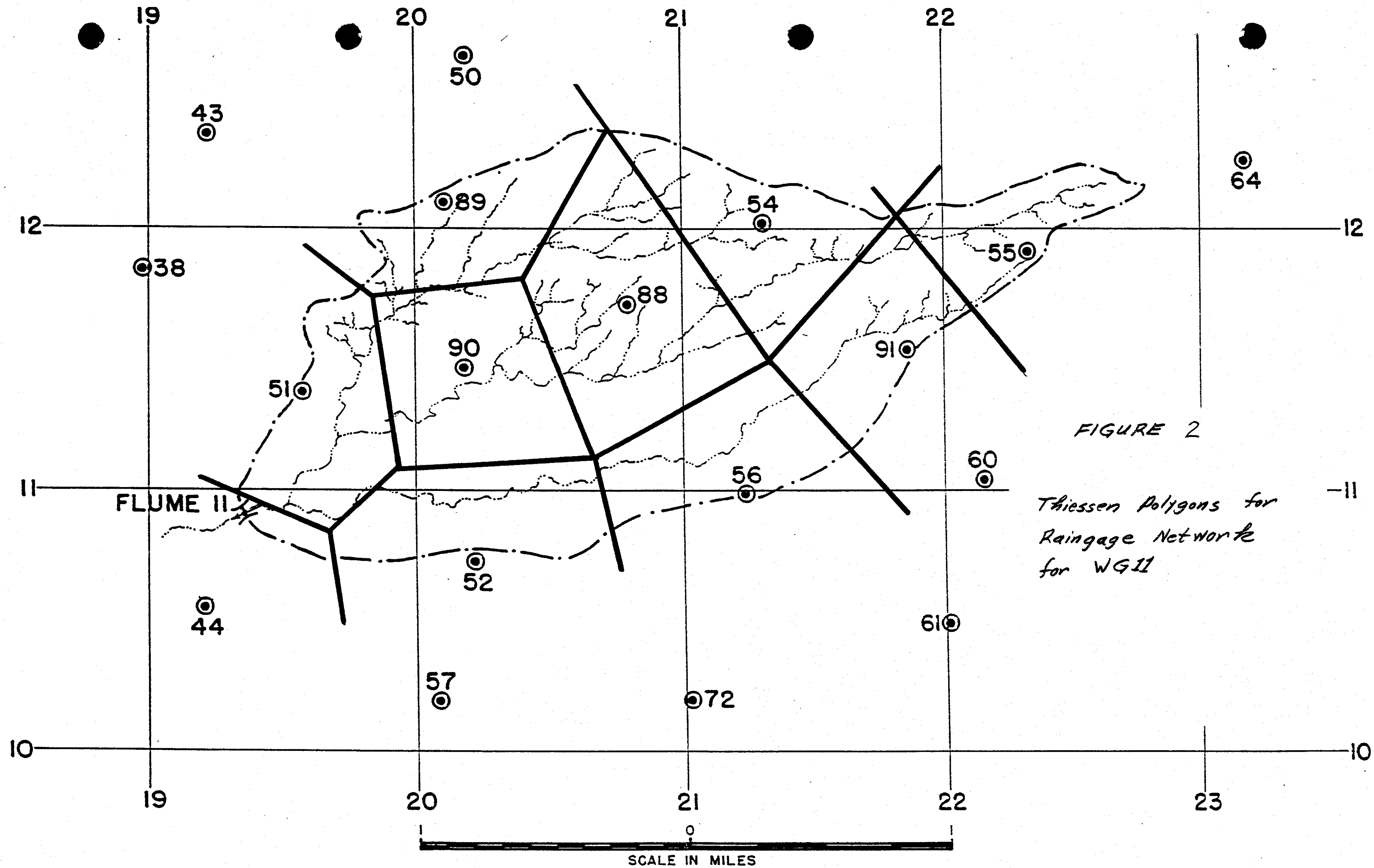


FIGURE 2

Thiessen Polygons for Raingage Network for WG11

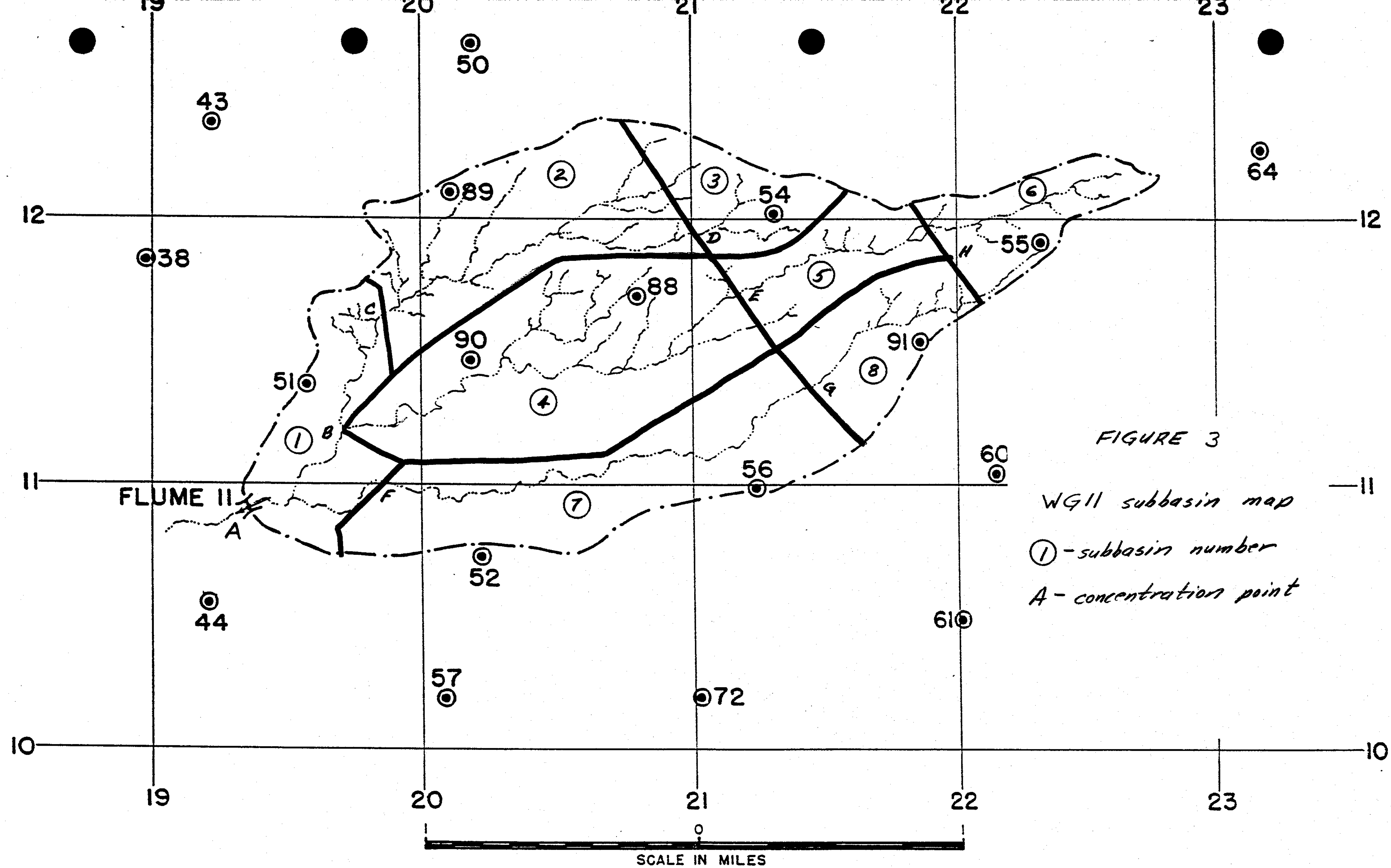
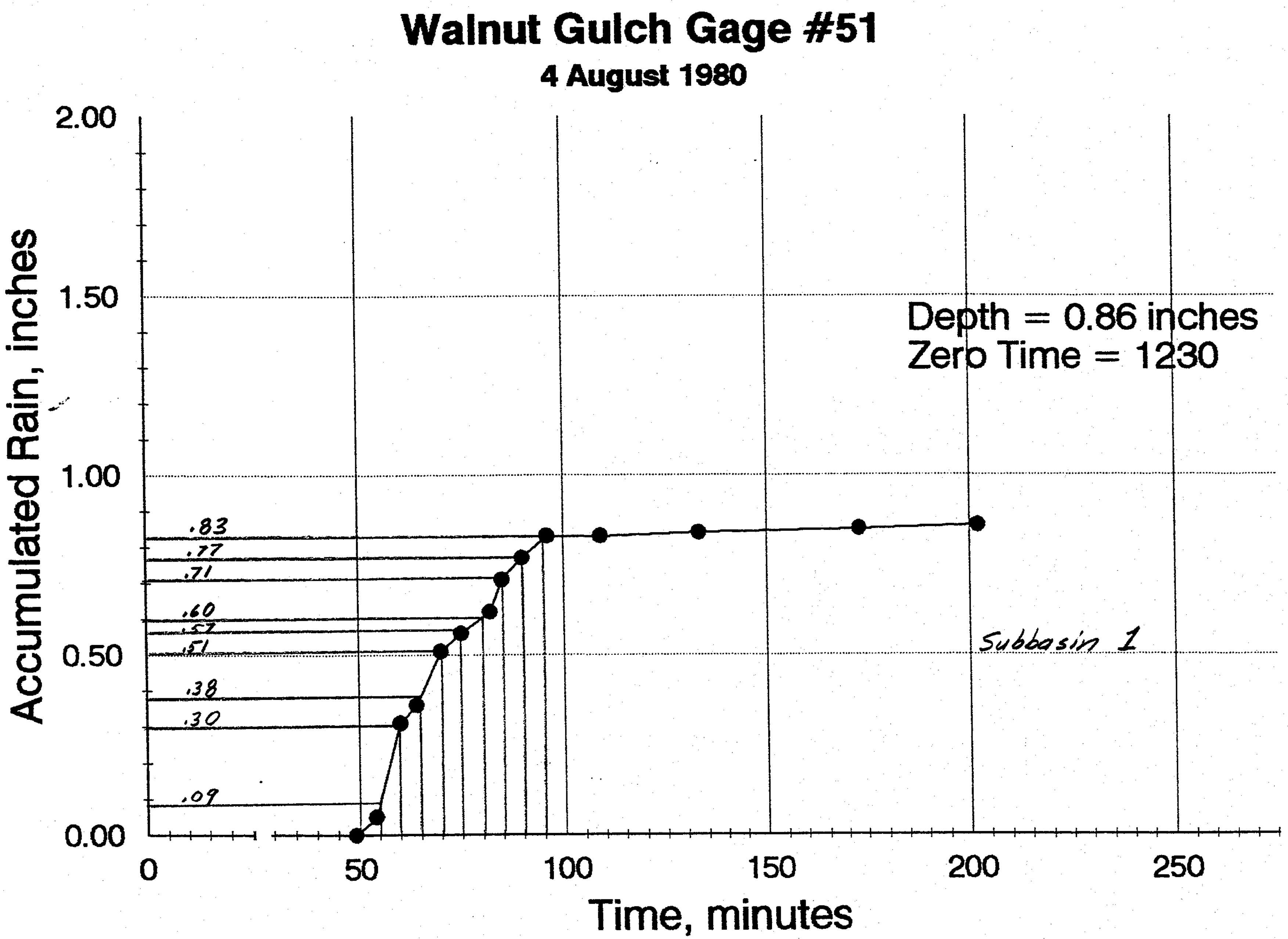


FIGURE 3

WG11 subbasin map

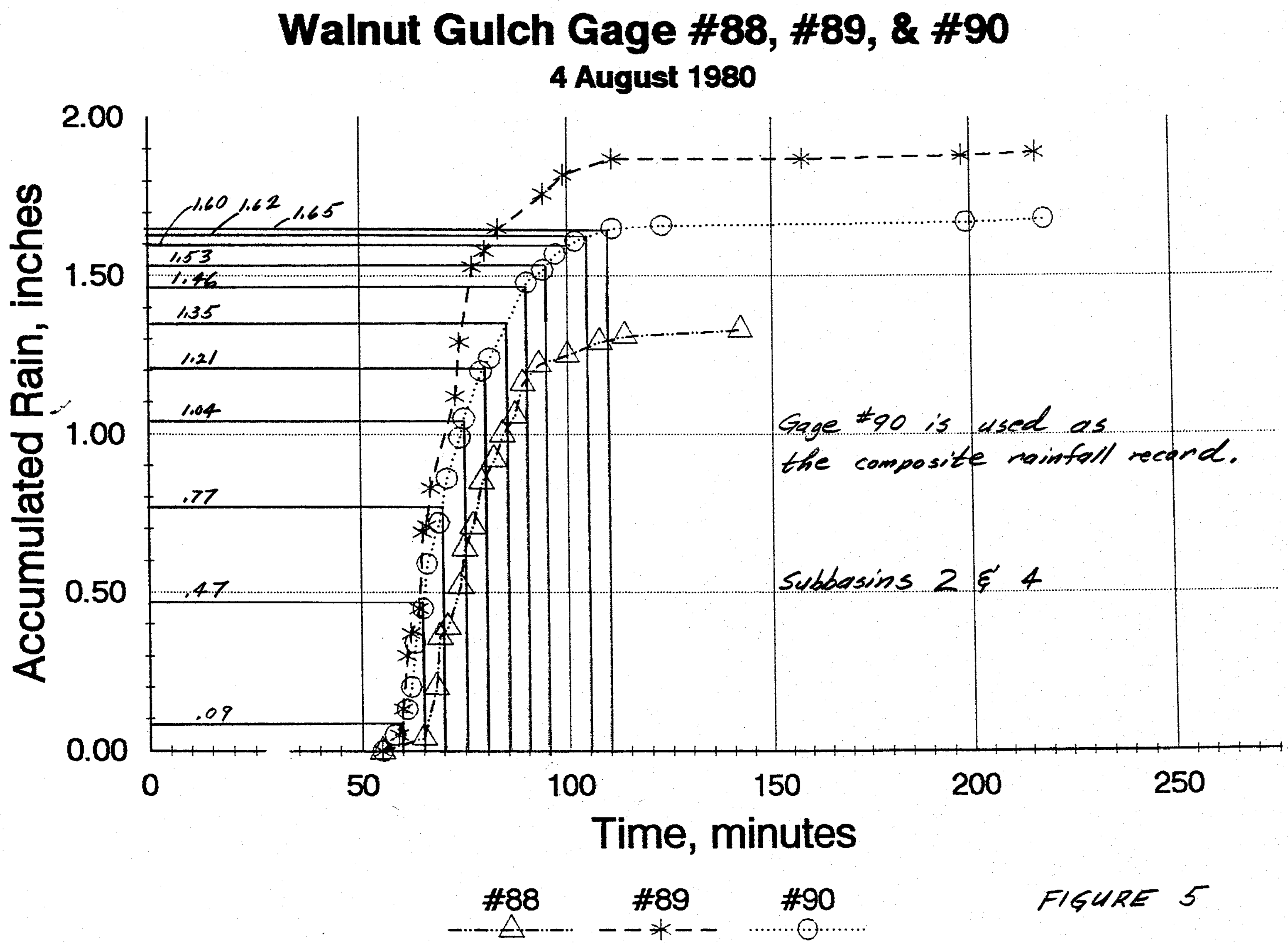
(1) - subbasin number

A - concentration point



Walnut Gulch Gage #51
4 August 1980
Depth = 0.86 inches
Zero Time = 1230
Accumulated Rain, inches
Time, minutes
Subbasin 1
.83
.77
.71
.60
.57
.51
.38
.30
.09
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250

FIGURE 4



Walnut Gulch Gage #88, #89, & #90
4 August 1980
Accumulated Rain, inches
Time, minutes
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
1.60
1.62
1.65
1.53
1.46
1.35
1.21
1.04
.77
.47
.09
Gage #90 is used as the composite rainfall record.
Subbasins 2 & 4
#88
#89
#90

FIGURE 5

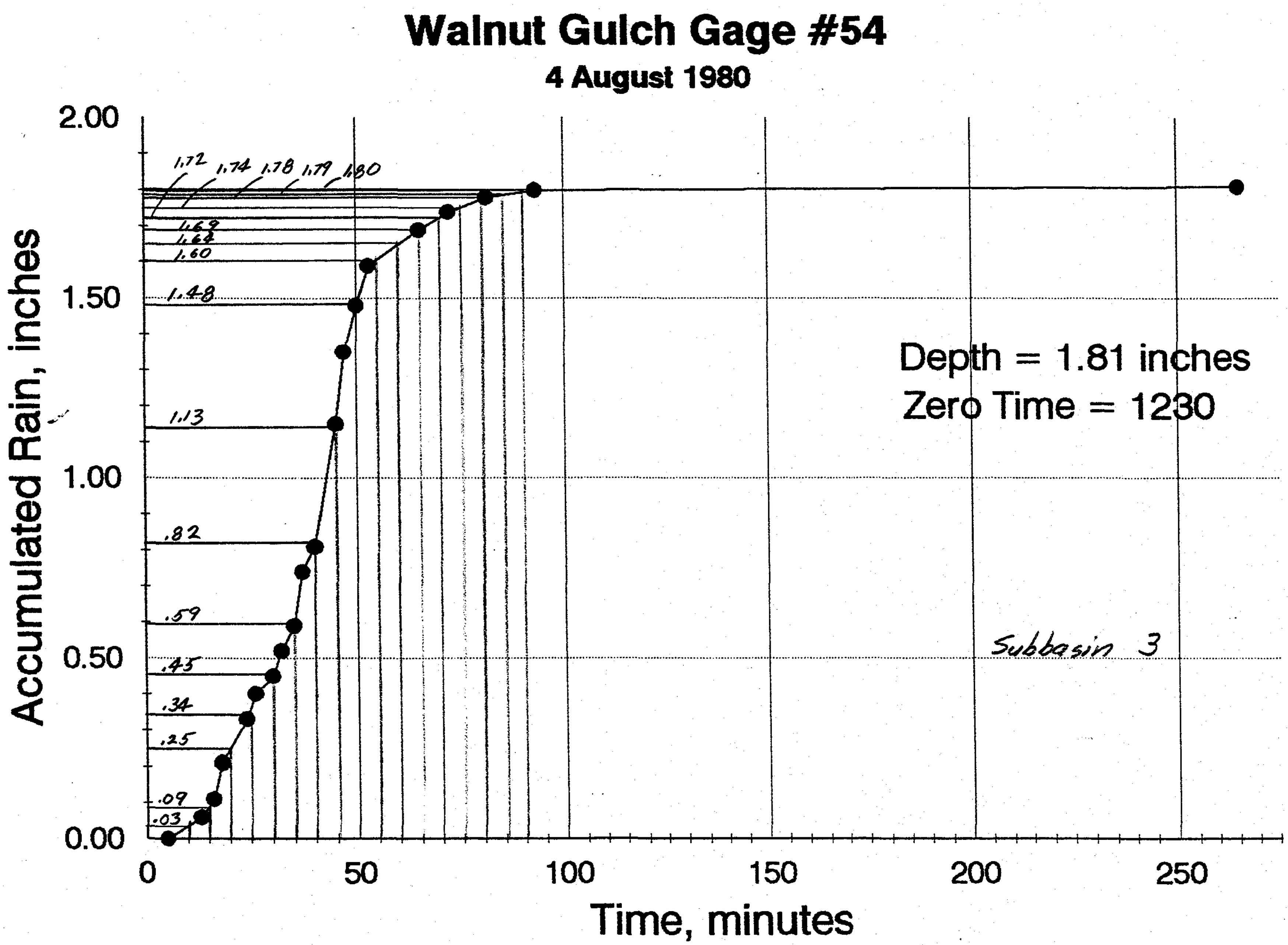

Walnut Gulch Gage #54
4 August 1980
Accumulated Rain, inches
Time, minutes
Depth = 1.81 inches
Zero Time = 1230
Subbasin 3
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
1.72
1.74
1.78
1.79
1.80
1.69
1.64
1.60
1.48
1.13
.82
.59
.45
.34
.25
.09
.03

FIGURE 6

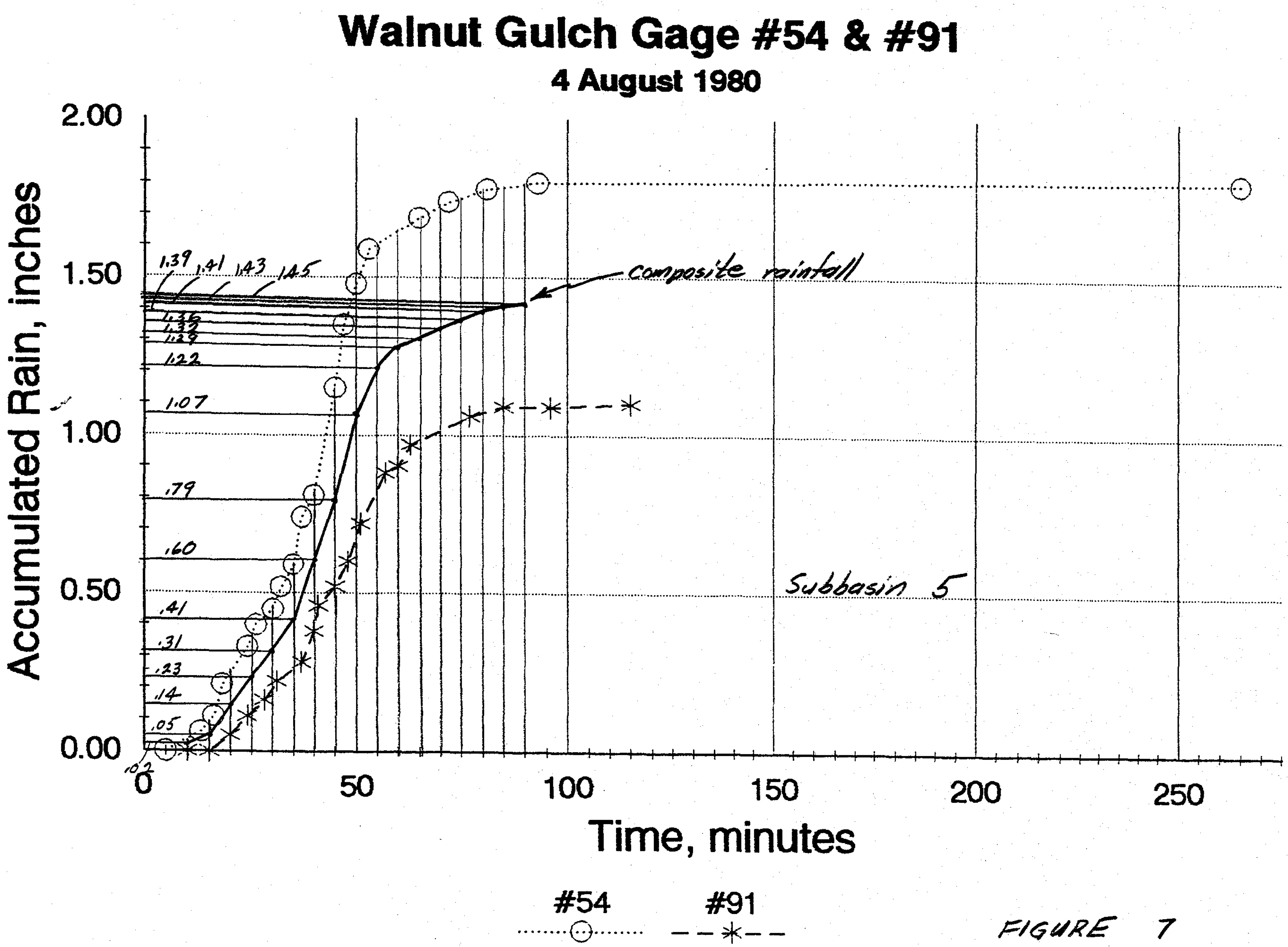
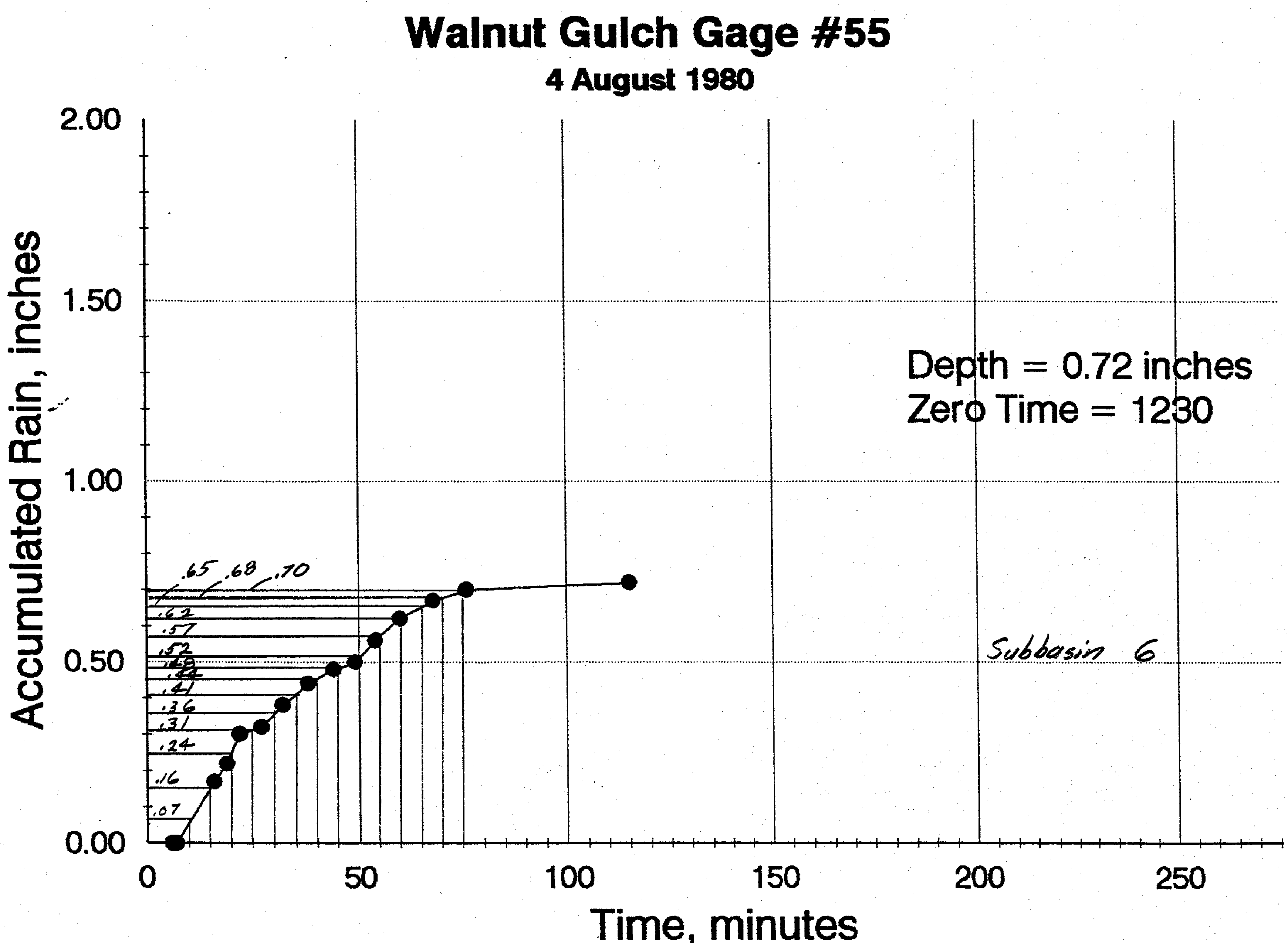
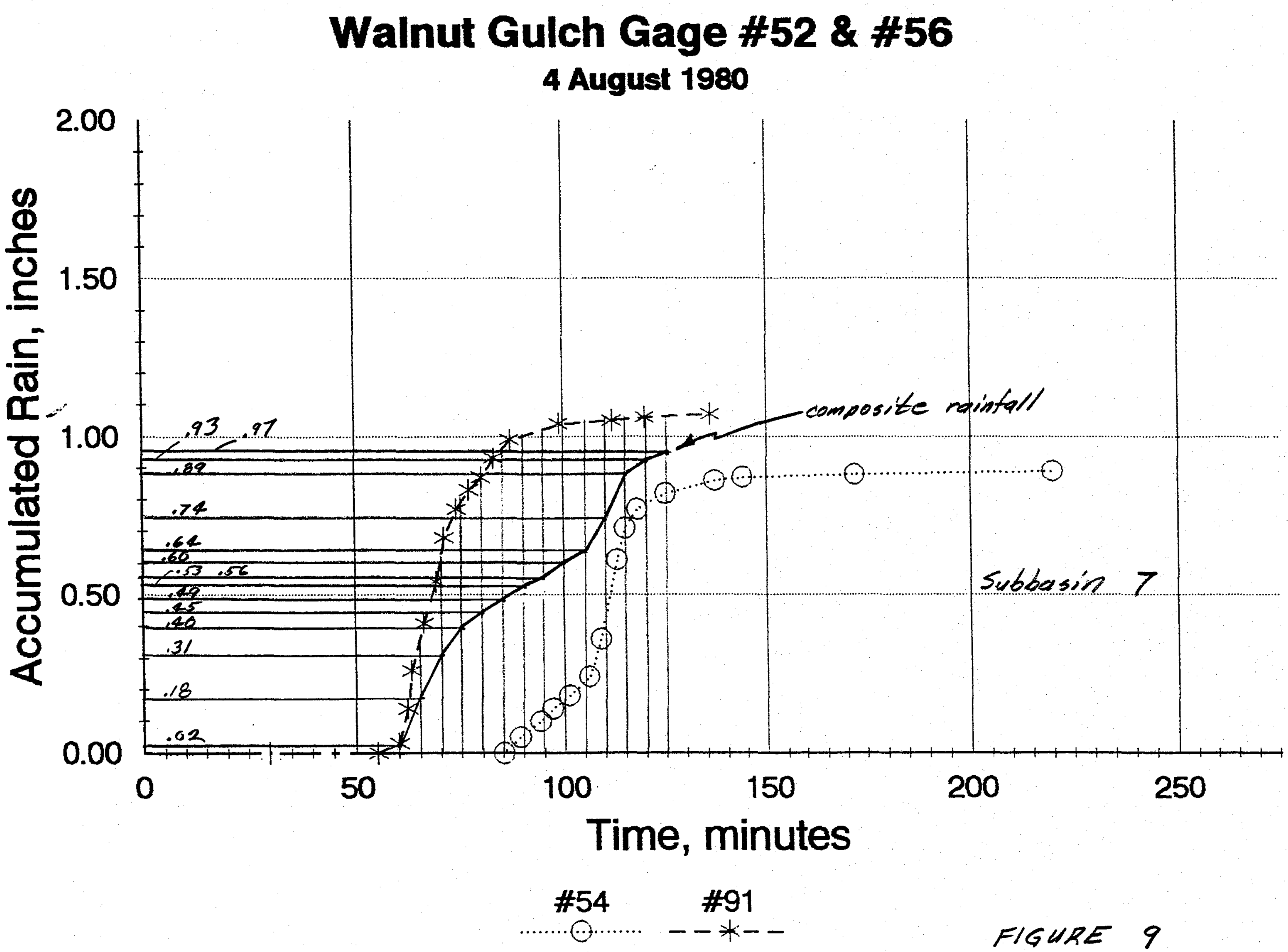

Walnut Gulch Gage #54 & #91
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
Time, minutes
0
50
100
150
200
250
1.39
1.41
1.43
1.45
1.36
1.33
1.29
1.22
1.07
.79
.60
.41
.31
.23
.14
.05
.02
composite rainfall
Subbasin 5
#54
#91

FIGURE 7



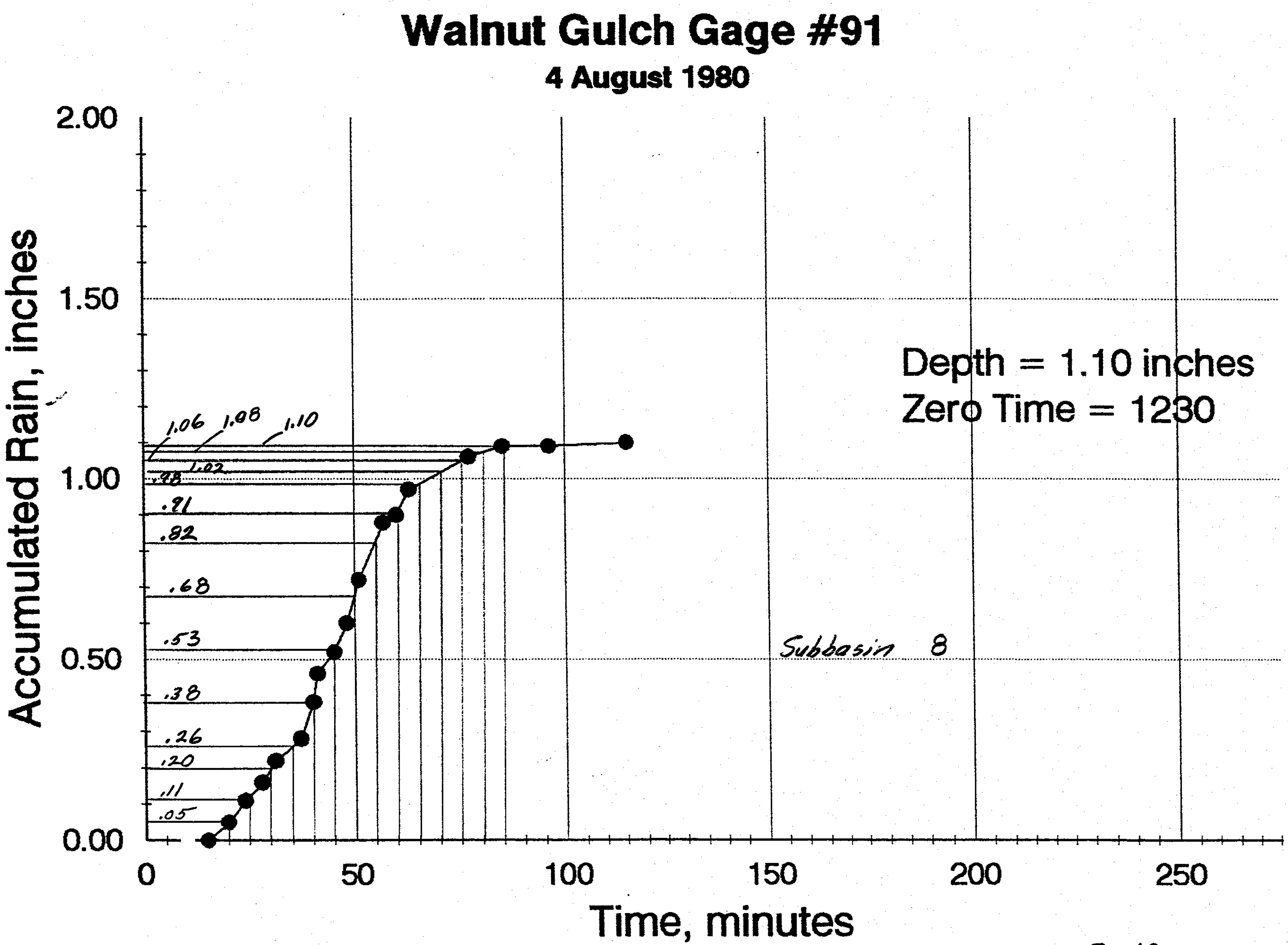
Walnut Gulch Gage #55
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
Time, minutes
Depth = 0.72 inches
Zero Time = 1230
Subbasin 6
.65
.68
.70
.62
.57
.52
.41
.36
.31
.24
.16
.07

FIGURE 8



Walnut Gulch Gage #52 & #56
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
Time, minutes
0
50
100
150
200
250
.93
.97
.89
.74
.64
.60
.53
.56
.49
.45
.40
.31
.18
.02
composite rainfall
Subbasin 7
#54
#91

FIGURE 9



Walnut Gulch Gage #91
4 August 1980
2.00
1.50
1.00
0.50
0.00
Accumulated Rain, inches
0
50
100
150
200
250
Time, minutes
Depth = 1.10 inches
Zero Time = 1230
1.06
1.08
1.10
1.02
.98
.91
.82
.68
.53
.38
.26
.20
.11
.05
Subbasin 8

FIGURE 10

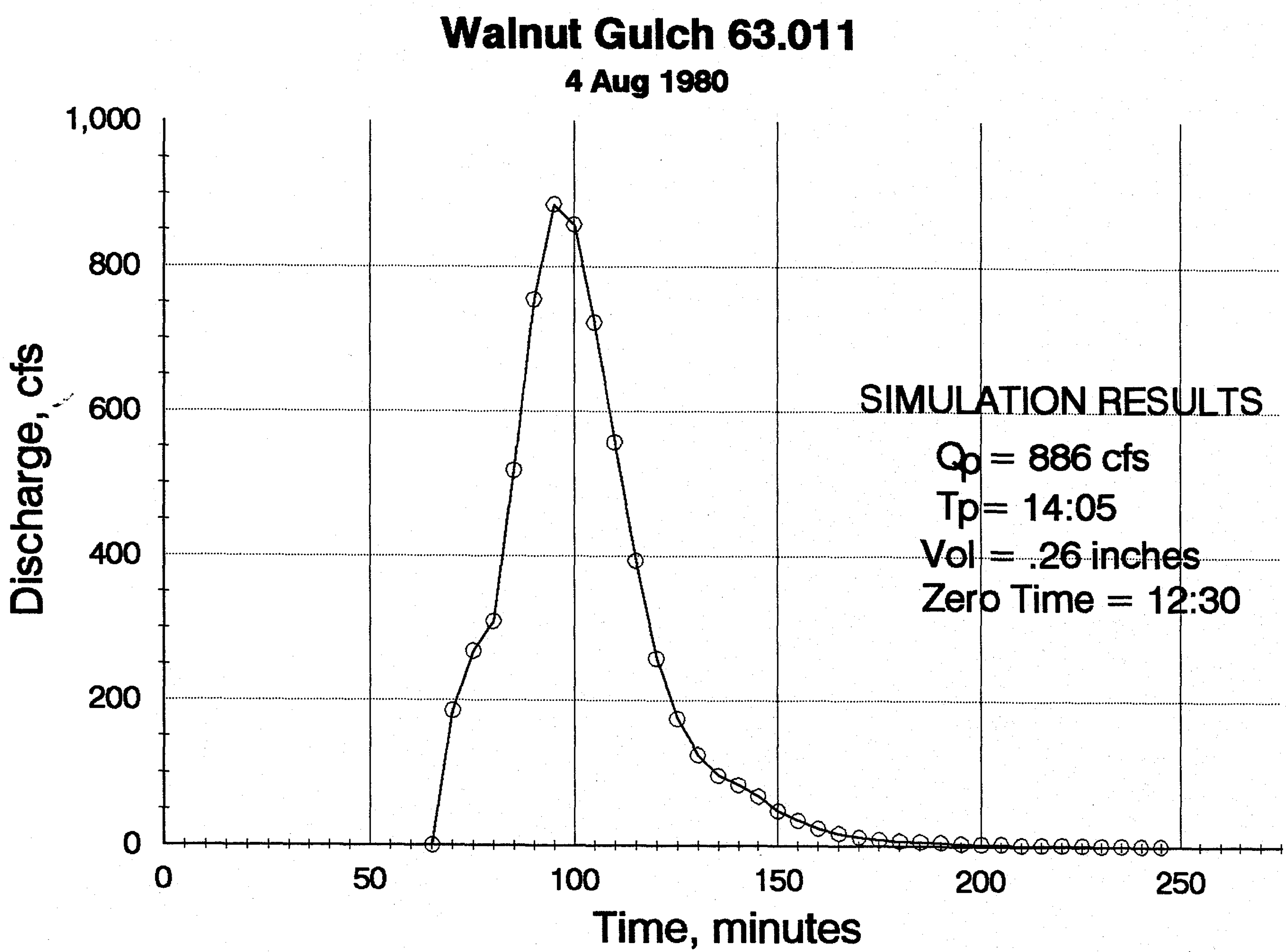

Walnut Gulch 63.011
4 Aug 1980
1,000
800
600
400
200
0
Discharge, cfs
0
50
100
150
200
250
Time, minutes
SIMULATION RESULTS
Qp = 886 cfs
Tp= 14:05
Vol = .26 inches
Zero Time = 12:30

FIGURE 11

APPENDIX

ARS Data

```
********************************************************************************
  WALNUT GULCH - FOUR RAINFALL/RUNOFF EVENTS FROM SUBWATERSHED 11
********************************************************************************
```

RAINFALL DATA FOR ALL GAGES ON A GIVEN WATERSHED FOR A GIVEN EVENT ARE IN A SINGLE FILE NAMED AFTER THE DATE OF OCCURRENCE. THE FILES ARE IN KINEROS INPUT FORMAT AND INCLUDE TEXT MAKING THEM MOSTLY SELF-EXPLANATORY. NOTE: UNITS ARE ENGLISH AS FOLLOWS

1. RAINGAGE COOR. (FT) - ARIZONA STATE COOR.
2. SI (DIMENSIONLESS) - RELATIVE SOIL SATURATION
3. RAINGAGE TIME (MIN) - DEPTH (INCH) PAIRS

RUNOFF DATA IS GIVEN FOR A SINGLE STATION FOR ALL EVENTS CORRESPONDING TO THE RAINFALL FILES LISTED FOR THAT STATION. THE SEQUENCE OF EVENTS IN THE RUNOFF FILE WILL BE AS PER THE LISTS THAT FOLLOW. THE STRUCTURE OF THE FILE IS REPEATED BLOCKS OF DATA, THE FIRST RECORD BEING A SUMMARY LINE:

NUMBER OF TIME-DISCHARGE PAIRS; TIME TO PEAK (MIN); PEAK DISCHARGE (IPH); RUNOFF VOLUME (IN); START TIME (MILITARY); BASIN AREA (SQ.FT)

THIS IS FOLLOWED BY THE TIME, DISCHARGE PAIRS (MIN, CFS)

SUBWATERSHED 11

04AUG80.PPT
20SEP83.PPT
26SEP83.PPT
15JUL86.PPT

RUNOFF IN WGS11.HYD

Runoff Hydrograph

51 20.00 0.568401 0.227573 1327. 67949984.00

start time

raintall excess, inches

time	Q, cfs
0.00	0.000000
1.00	70.000504
2.00	225.996521
3.00	292.324066
4.00	330.451477
4.50	364.210297
5.50	402.910217
7.50	439.383789
9.00	485.930420
10.50	567.204407
11.00	620.579956
13.00	707.798401
15.50	809.803345
16.00	859.253418
17.50	890.205200
20.00	894.046509
22.50	841.378906
23.00	805.664551
24.00	705.627380
25.00	503.782440
26.00	386.059540
27.00	322.243622
28.50	237.499817
30.00	194.254791
31.50	157.684799
32.50	138.595413
34.00	113.828285
36.50	165.814636
37.50	180.443054
39.50	186.836975
42.50	176.720795
44.50	163.625214
46.00	140.093353
48.00	120.860344
49.50	105.739456
51.00	94.523460
53.00	87.044075
55.00	72.409500
62.50	48.498035
65.50	37.946865
68.00	31.481146
74.50	23.667288
86.50	16.068886
96.50	10.175661
106.00	5.119636
111.50	2.766040
122.00	1.099850
133.50	0.416548
146.50	0.133377
161.00	0.034883
181.50	0.000000

Rainfall Data

KINEROS Rainfall Input Data

```
#
*****************************
     Gage Network Data    USDA-ARS WS # 63  EVENT OF 8/ 4/80
*****************************
# NUMBER OF    MAX. NO. OF TIME-DEPTH   SIMULATION  EARLIEST RAINGAGE
  RAINGAGES  DATA PAIRS FOR ALL GAGES      TIME       MILITARY START
  (NGAGES)            (MAXND)             (TFIN)          TIME
  --------            -------             ------          ----
     10                  23               450.0           1235
#  SEQUENTIAL      INITIAL SOIL
   GAGE NUMBER     MOISTURE (SI)        GAGE X         GAGE Y
   -----------     -------------        ------         ------
        1              0.098          557250.44      275284.66
        2              0.117          554911.00      272545.19
        3              0.124          558018.94      272479.38
        4              0.105          563500.19      275357.97
        5              0.103          560932.06      273393.13
        6              0.104          569052.81      274078.03
        7              0.106          566271.56      272031.06
        8              0.126          563899.13      269954.56
        9              0.106          557921.81      268783.94
       10              0.138          553538.25      268724.94
#
*****************************
      Rainfall Data
*****************************
 There must be NGAGES sets of rainfall data. Repeat lines from * to *
 for each gage inserting a variable number of TIME-DEPTH data pairs
 (see example in User Manual).
```

* WALNUT GULCH GAGE # 89 = SEQUENTIAL GAGE NUM. 1
# THE STARTING DATE & TIME OF RAINFALL AT GAGE 89 IS 8/ 4 1300

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
1	22

PRECIPITATION IS RAIN
There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).
THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
25	0.00
29	0.05
30	0.13
31	0.30
32	0.37
34	0.45
35	0.69
36	0.71
37	0.83
43	1.12
44	1.29
47	1.53
50	1.58
53	1.65
64	1.76
69	1.82
81	1.87
128	1.87
168	1.88
187	1.89
370	1.89

*
* WALNUT GULCH GAGE # 51 = SEQUENTIAL GAGE NUM. 2
# THE STARTING DATE & TIME OF RAINFALL AT GAGE 51 IS 8/ 4 1257

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
2	16

PRECIPITATION IS RAIN
There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).
THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
22	0.00
27	0.05
33	0.31
37	0.36
43	0.51
48	0.56
55	0.62
58	0.71
63	0.77
69	0.83
82	0.83
106	0.84
146	0.85
175	0.86
370	0.86

*

* WALNUT GULCH GAGE # 90 = SEQUENTIAL GAGE NUM. 3
# THE STARTING DATE & TIME OF RAINFALL AT GAGE 90 IS 8/ 4 1300

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
3	23

PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
25	0.00
28	0.05
31	0.13
32	0.20
33	0.34
35	0.45
36	0.59
39	0.72
41	0.86
44	0.99
45	1.05
49	1.20
51	1.24
60	1.48
64	1.52
67	1.57
72	1.61
81	1.65
93	1.66
169	1.67
189	1.68
370	1.68

*
* WALNUT GULCH GAGE # 54 = SEQUENTIAL GAGE NUM. 4
# THE STARTING DATE & TIME OF RAINFALL AT GAGE 54 IS 8/ 4 1235

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
4	21

PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
8	0.06
11	0.11
13	0.21
19	0.33
21	0.40
25	0.45
27	0.52
30	0.59
32	0.74
35	0.81
40	1.15
42	1.35
45	1.48
48	1.59
60	1.69
67	1.74
76	1.78
88	1.80
260	1.81

* WALNUT GULCH GAGE # 88 = SEQUENTIAL GAGE NUM. 5
# THE STARTING DATE & TIME OF RAINFALL AT GAGE 88 IS 8/ 4 1300

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
5	20

\# PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

\# THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
25	0.00
35	0.04
38	0.20
39	0.36
41	0.39
44	0.52
45	0.64
47	0.71
49	0.85
52	0.92
54	1.00
57	1.06
59	1.16
63	1.22
70	1.25
78	1.29
84	1.31
112	1.33
370	1.33

*

* WALNUT GULCH GAGE # 55 = SEQUENTIAL GAGE NUM. 6
# THE STARTING DATE & TIME OF RAINFALL AT GAGE 55 IS 8/ 4 1236

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
6	16

\# PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

\# THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
1	0.00
10	0.17
13	0.22
16	0.30
21	0.32
26	0.38
32	0.44
38	0.48
43	0.50
48	0.56
54	0.62
62	0.67
70	0.70
109	0.72
370	0.72

*

```
*
* WALNUT GULCH GAGE # 52 = SEQUENTIAL GAGE NUM.  9
#  THE STARTING DATE & TIME OF RAINFALL AT GAGE  52 IS  8/ 4  1315
  SEQ. GAGE NUM.     NUM. OF DATA PAIRS (ND)
  --------------     -----------------------

        9                        17
#                                   PRECIPITATION IS RAIN
 There must be ND pairs of time-depth (T D) data: NOTE: The last time
 must be greater than TFIN (the total computational time).
# THERE ARE    0 BREAKPOINTS ESTIMATED
   TIME     ACCUM. DEPTH
  ------   -------------

     0         0.00
    40         0.00
    44         0.05
    49         0.10
    52         0.14
    56         0.18
    61         0.24
    64         0.36
    68         0.61
    70         0.71
    73         0.77
    80         0.82
    92         0.86
    99         0.87
   127         0.88
   175         0.89
   370         0.89
*
* WALNUT GULCH GAGE # 44 = SEQUENTIAL GAGE NUM. 10
#  THE STARTING DATE & TIME OF RAINFALL AT GAGE  44 IS  8/ 4  1320
  SEQ. GAGE NUM.     NUM. OF DATA PAIRS (ND)
  --------------     -----------------------

       10                        14
#                                   PRECIPITATION IS RAIN
 There must be ND pairs of time-depth (T D) data: NOTE: The last time
 must be greater than TFIN (the total computational time).
# THERE ARE    0 BREAKPOINTS ESTIMATED
   TIME     ACCUM. DEPTH
  ------   -------------

     0         0.00
    45         0.00
    50         0.06
    56         0.11
    60         0.17
    66         0.21
    89         0.23
    94         0.25
   103         0.27
   118         0.28
   146         0.29
   157         0.31
   184         0.32
   370         0.32
*
```

* WALNUT GULCH GAGE # 91 = SEQUENTIAL GAGE NUM. 7

THE STARTING DATE & TIME OF RAINFALL AT GAGE 91 IS 8/ 4 1240

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
7	20

PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
5	0.00
10	0.05
14	0.11
18	0.16
21	0.22
27	0.28
30	0.38
31	0.46
35	0.52
38	0.60
41	0.72
47	0.87
50	0.90
53	0.97
67	1.06
75	1.09
86	1.09
105	1.10
370	1.10

*

* WALNUT GULCH GAGE # 56 = SEQUENTIAL GAGE NUM. 8

THE STARTING DATE & TIME OF RAINFALL AT GAGE 56 IS 8/ 4 1300

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
8	18

PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
25	0.00
30	0.03
32	0.14
33	0.26
36	0.41
39	0.54
41	0.68
44	0.77
47	0.83
50	0.87
53	0.93
57	0.99
69	1.04
82	1.05
90	1.06
106	1.07
370	1.07

*

* WALNUT GULCH GAGE # 52 = SEQUENTIAL GAGE NUM. 9

\# THE STARTING DATE & TIME OF RAINFALL AT GAGE 52 IS 8/ 4 1315

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
9	17

\# PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

\# THERE ARE 0 BREAKPOINTS ESTIMATED

TIME	ACCUM. DEPTH
0	0.00
40	0.00
44	0.05
49	0.10
52	0.14
56	0.18
61	0.24
64	0.36
68	0.61
70	0.71
73	0.77
80	0.82
92	0.86
99	0.87
127	0.88
175	0.89
370	0.89

*

* WALNUT GULCH GAGE # 44 = SEQUENTIAL GAGE NUM. 10

\# THE STARTING DATE & TIME OF RAINFALL AT GAGE 44 IS 8/ 4 1320

SEQ. GAGE NUM.	NUM. OF DATA PAIRS (ND)
10	14

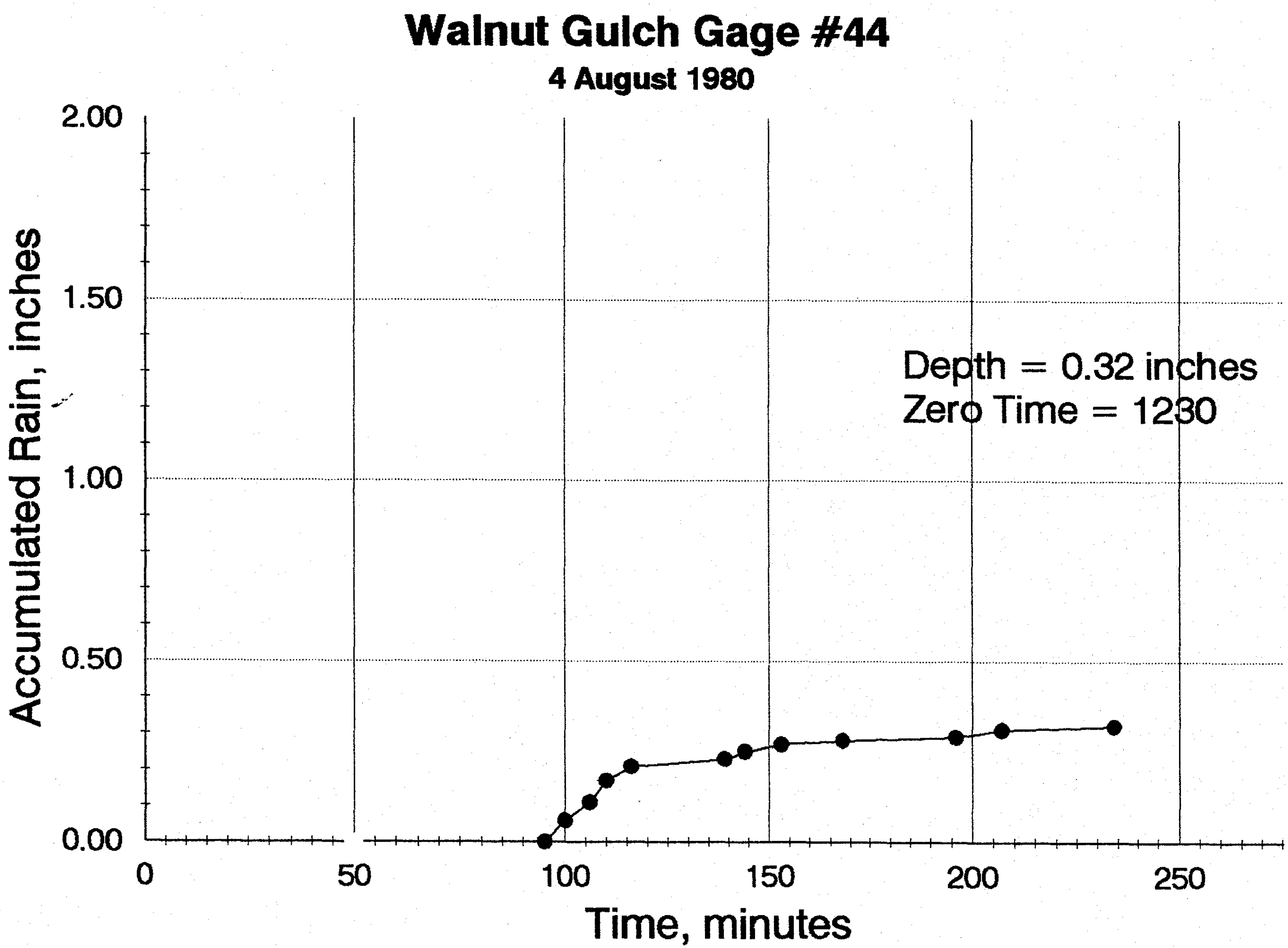
\# PRECIPITATION IS RAIN

There must be ND pairs of time-depth (T D) data: NOTE: The last time must be greater than TFIN (the total computational time).

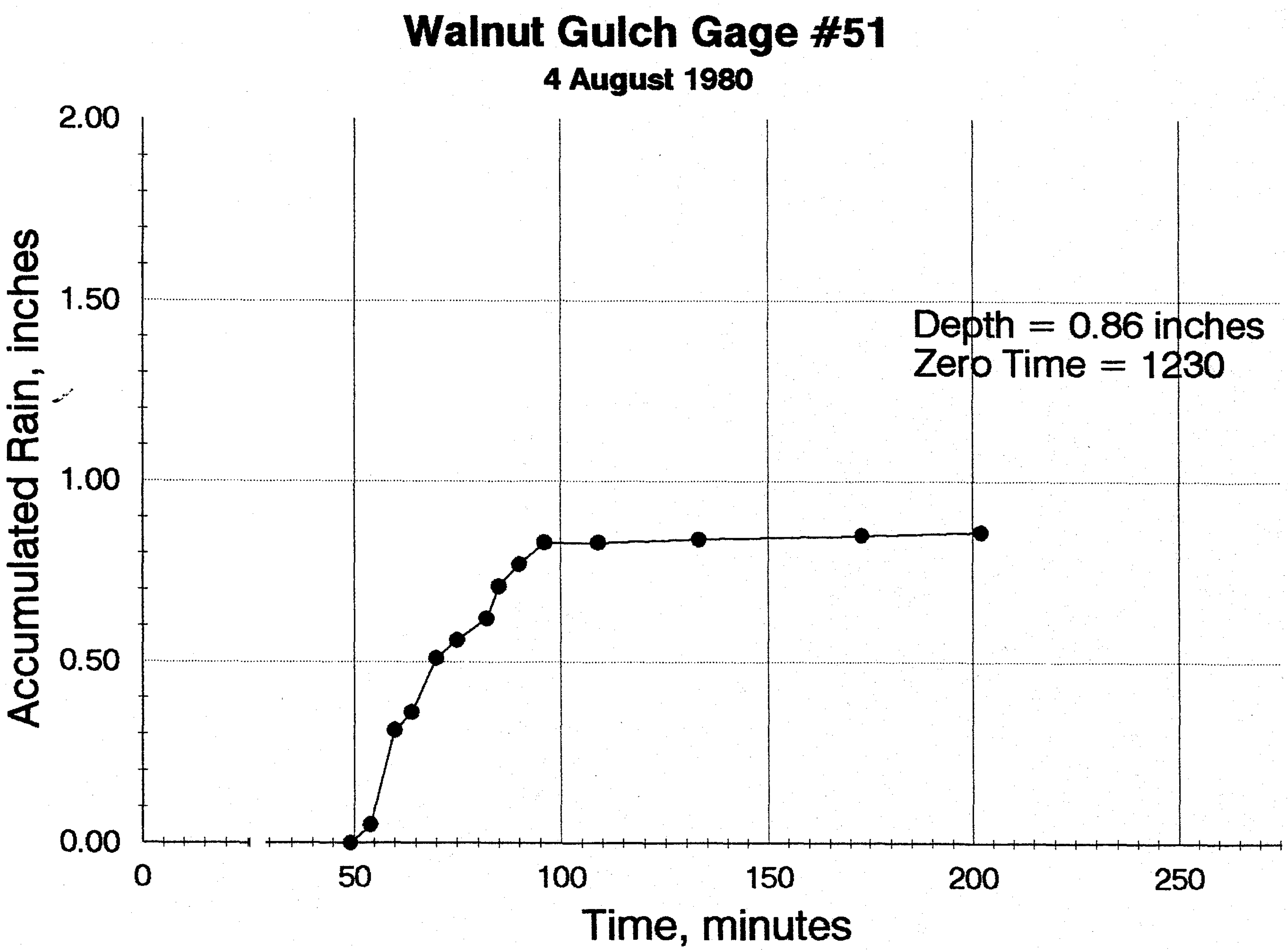
\# THERE ARE 0 BREAKPOINTS ESTIMATED

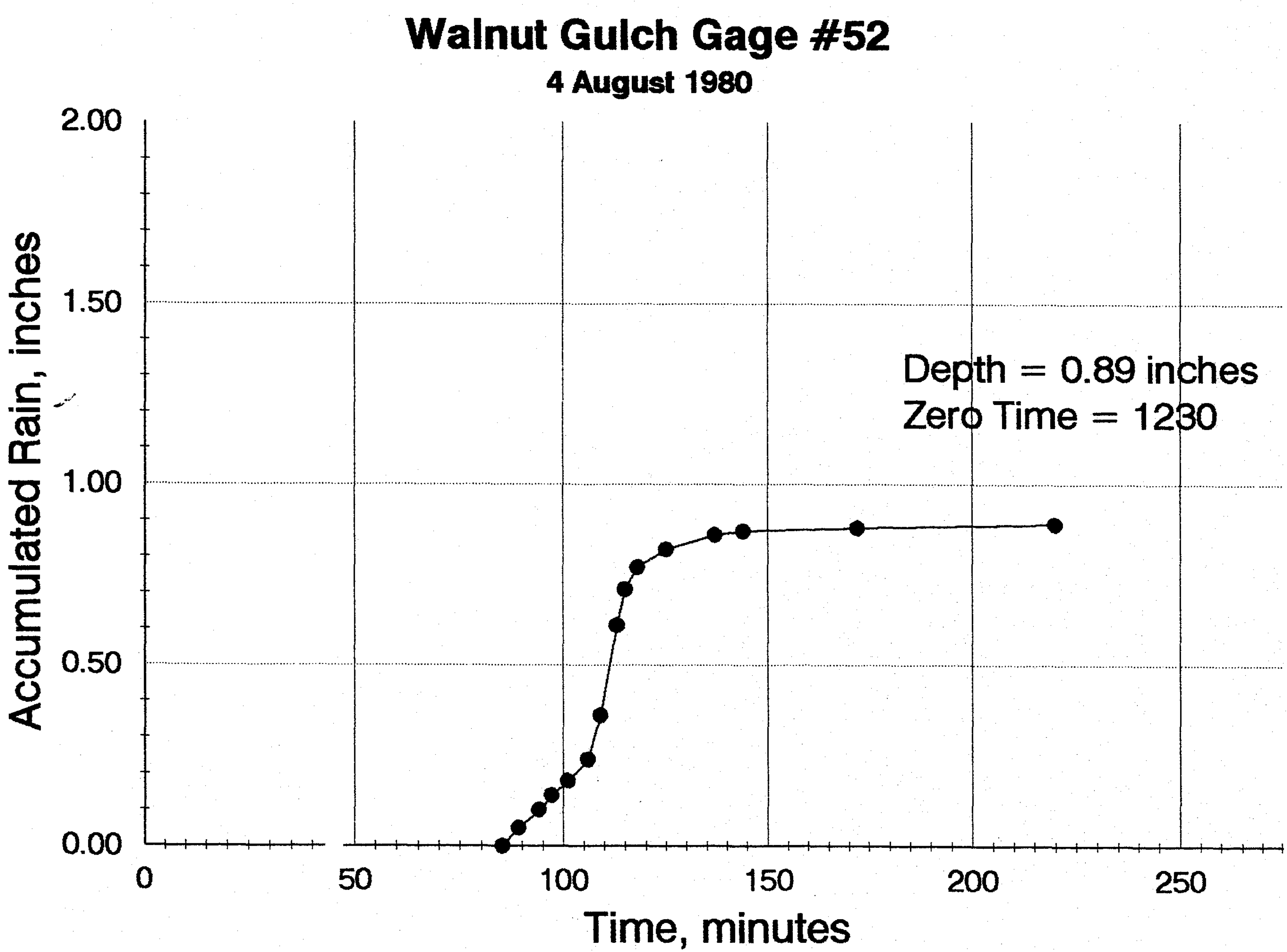
TIME	ACCUM. DEPTH
0	0.00
45	0.00
50	0.06
56	0.11
60	0.17
66	0.21
89	0.23
94	0.25
103	0.27
118	0.28
146	0.29
157	0.31
184	0.32
370	0.32

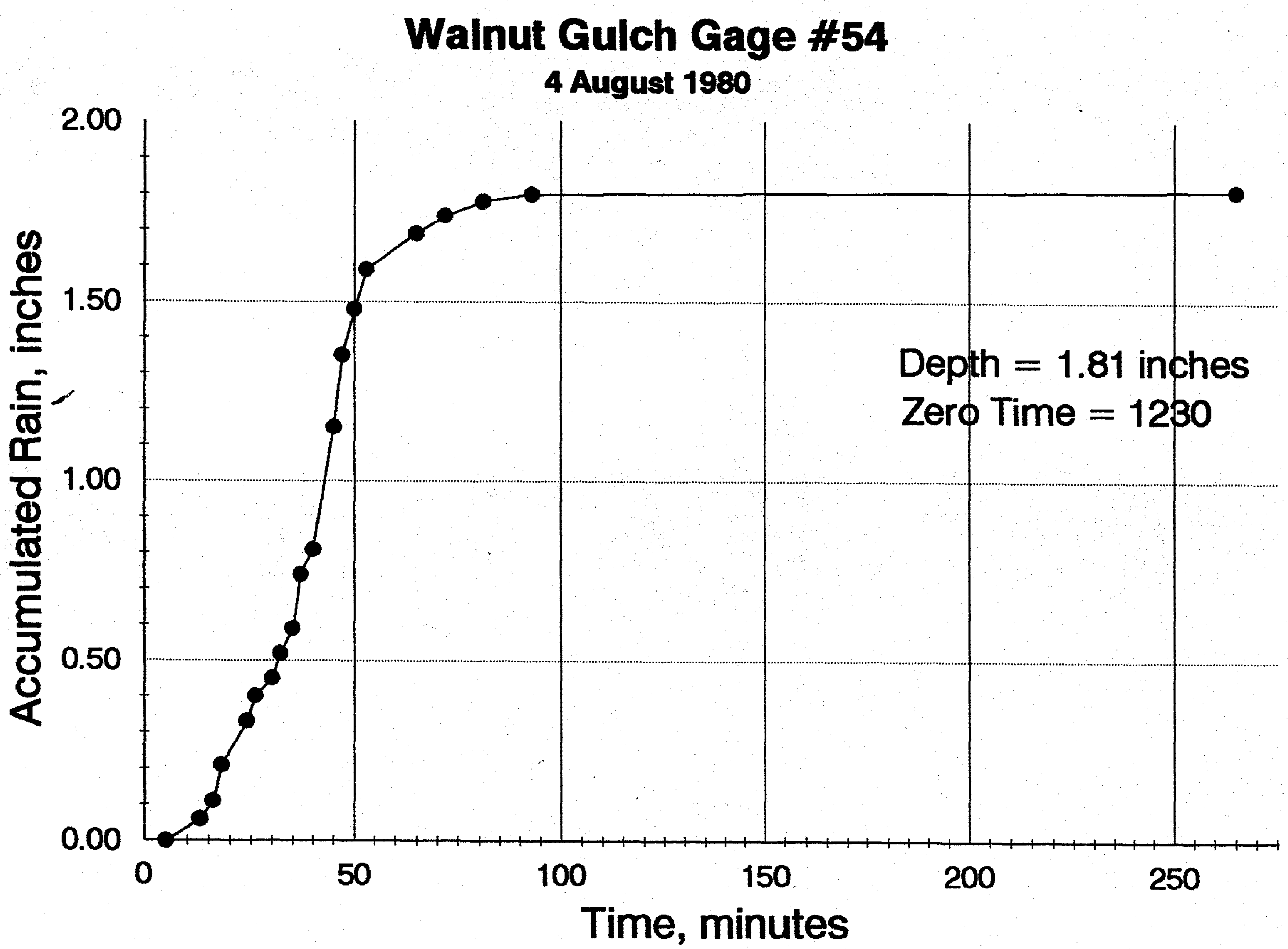
*



Walnut Gulch Gage #44
4 August 1980
2.00
1.50
1.00
0.50
0.00
Accumulated Rain, inches
Depth = 0.32 inches
Zero Time = 1230
0
50
100
150
200
250
Time, minutes

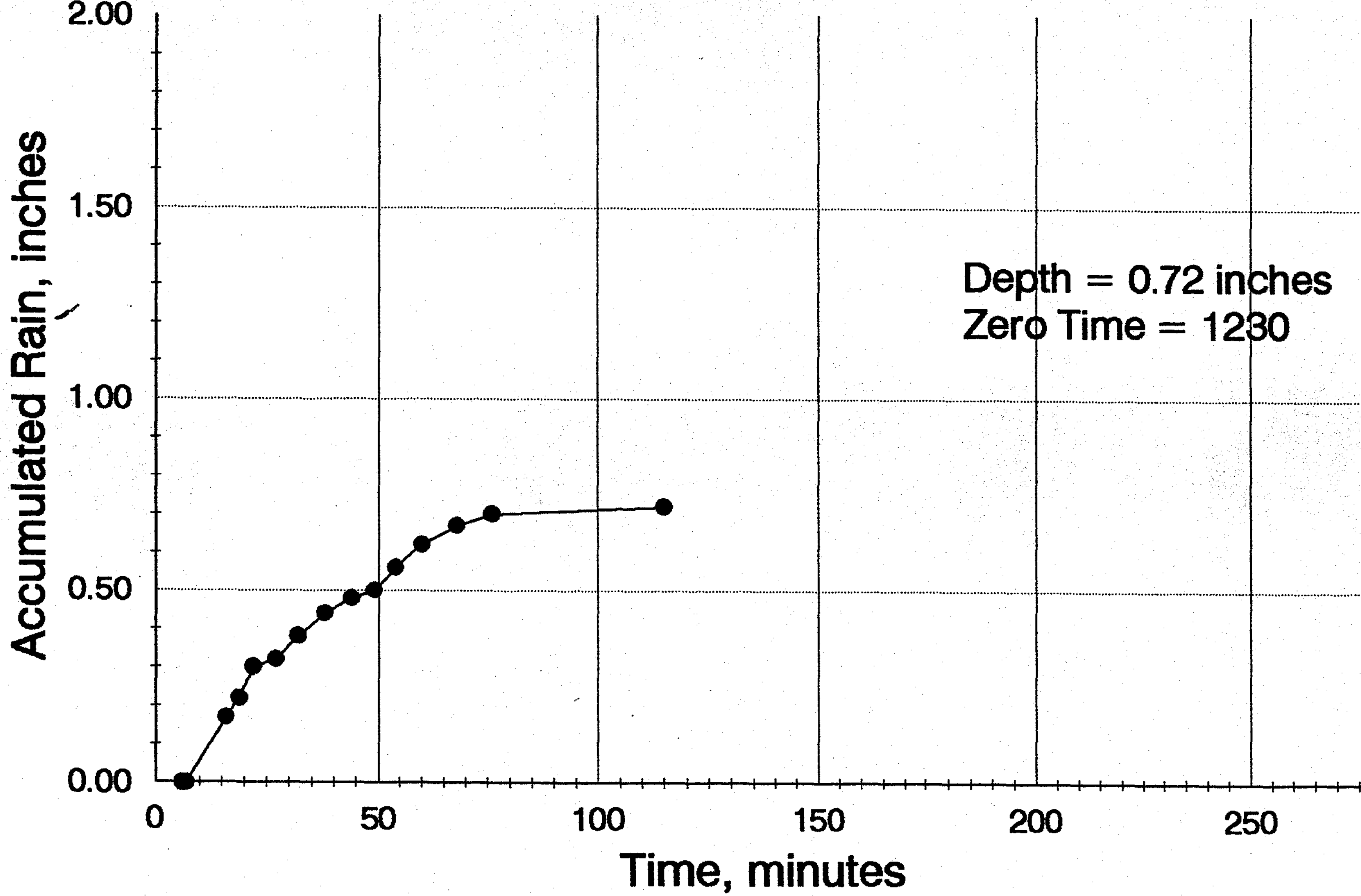

Walnut Gulch Gage #51
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
Time, minutes
Depth = 0.86 inches
Zero Time = 1230

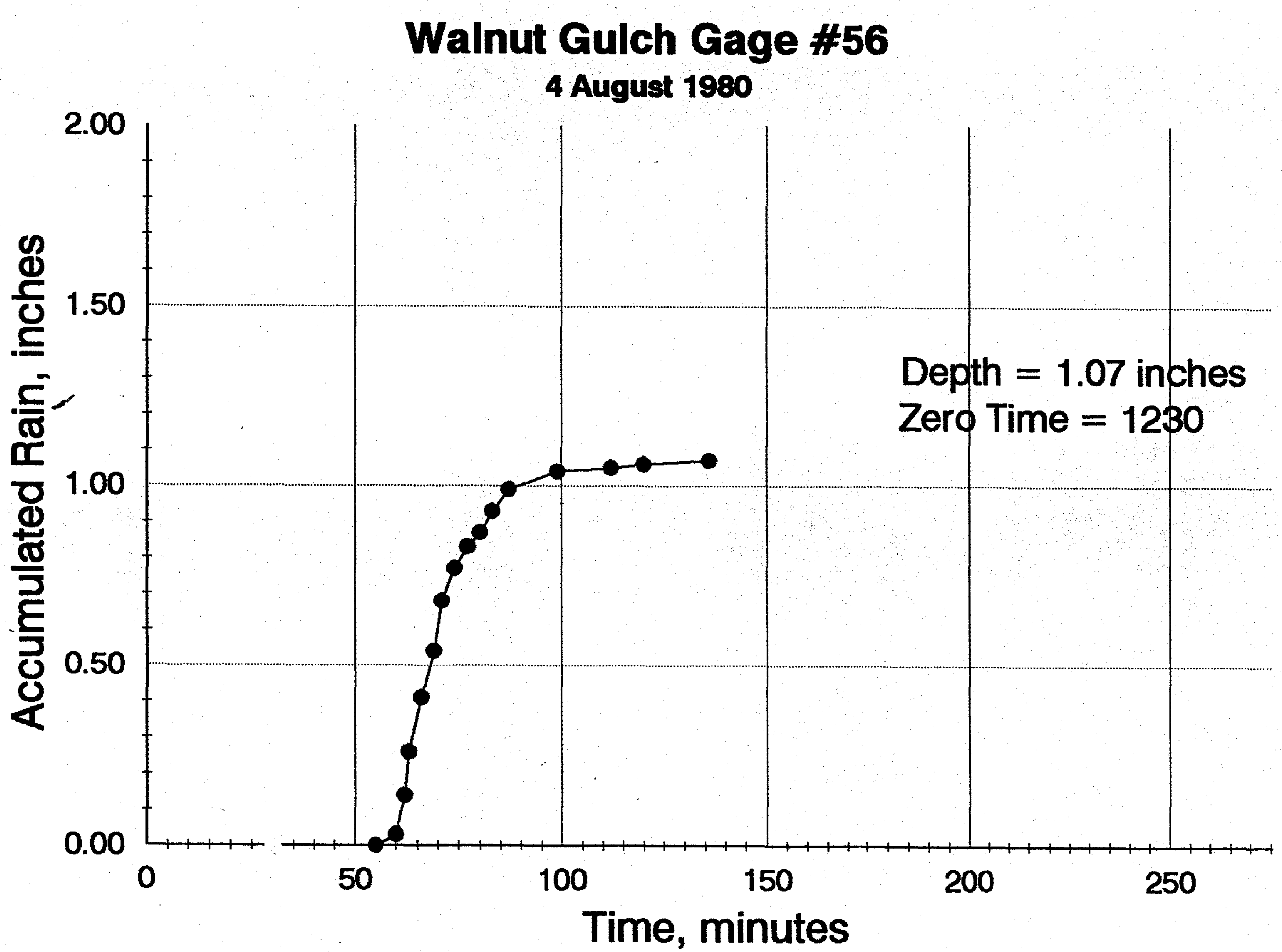

Walnut Gulch Gage #52
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
Time, minutes
Depth = 0.89 inches
Zero Time = 1230


Walnut Gulch Gage #54
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
Time, minutes
Depth = 1.81 inches
Zero Time = 1230

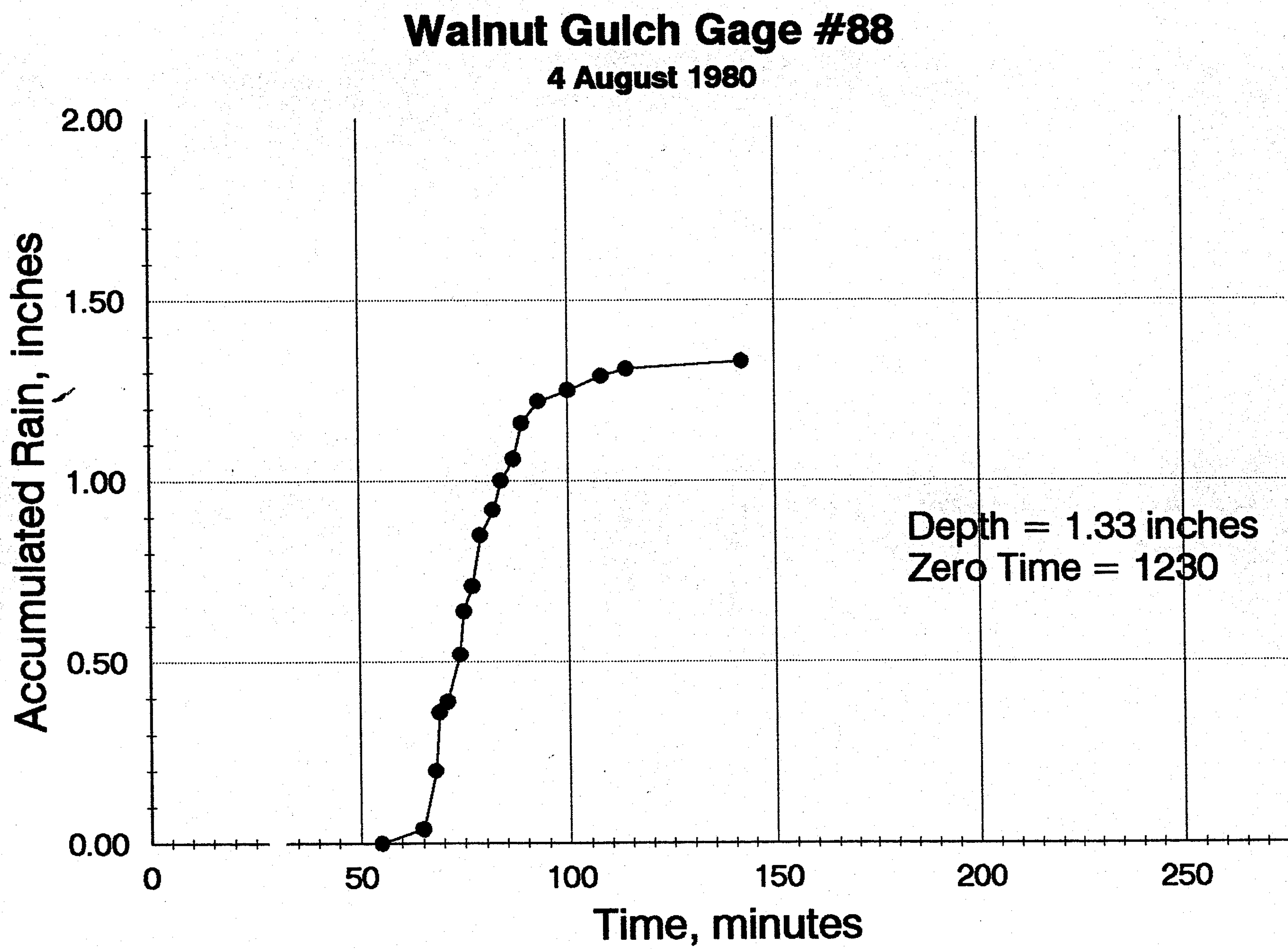
Walnut Gulch Gage #55

4 August 1980





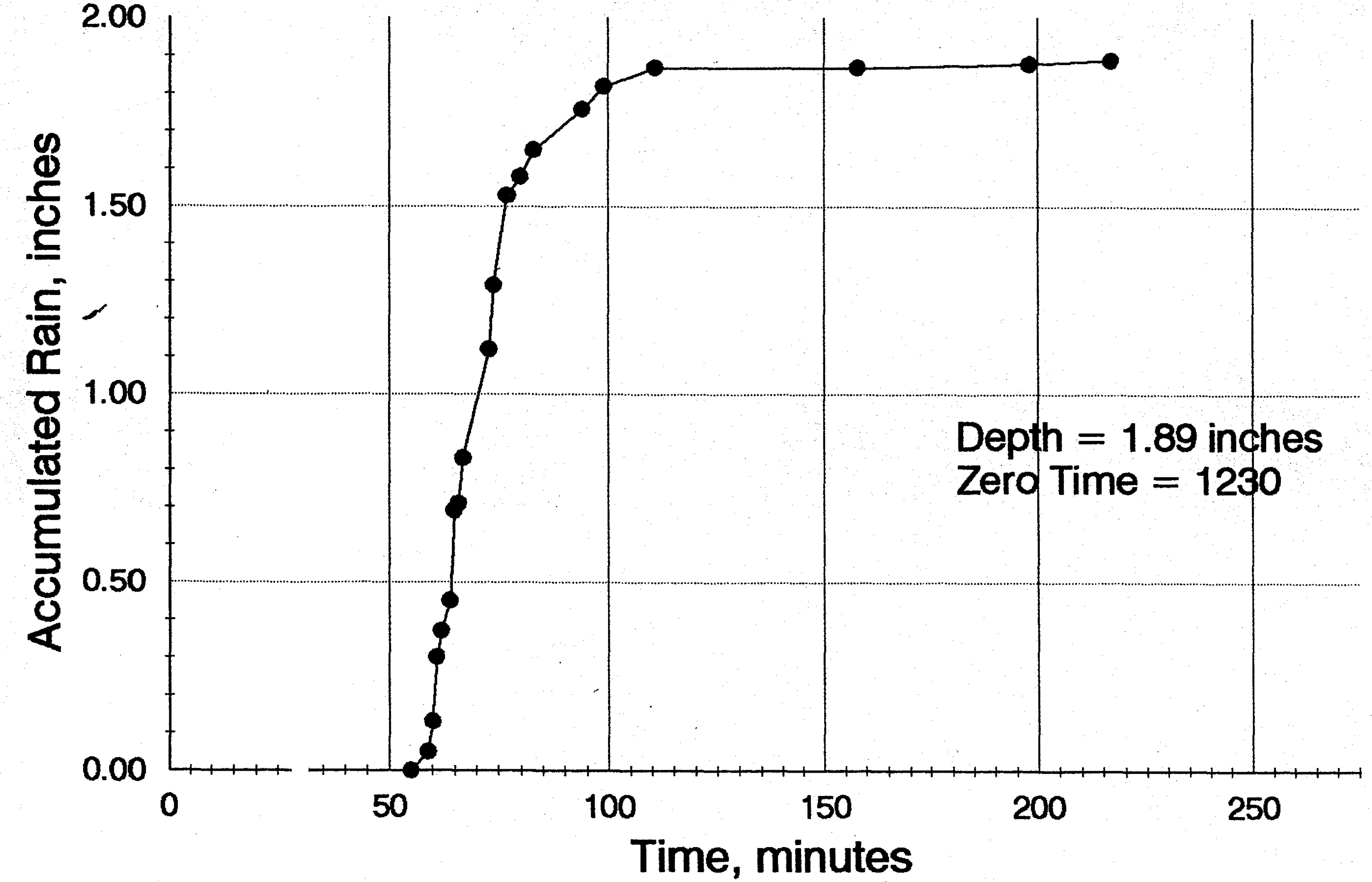
Walnut Gulch Gage #56
4 August 1980
2.00
1.50
1.00
0.50
0.00
Accumulated Rain, inches
0
50
100
150
200
250
Time, minutes
Depth = 1.07 inches
Zero Time = 1230

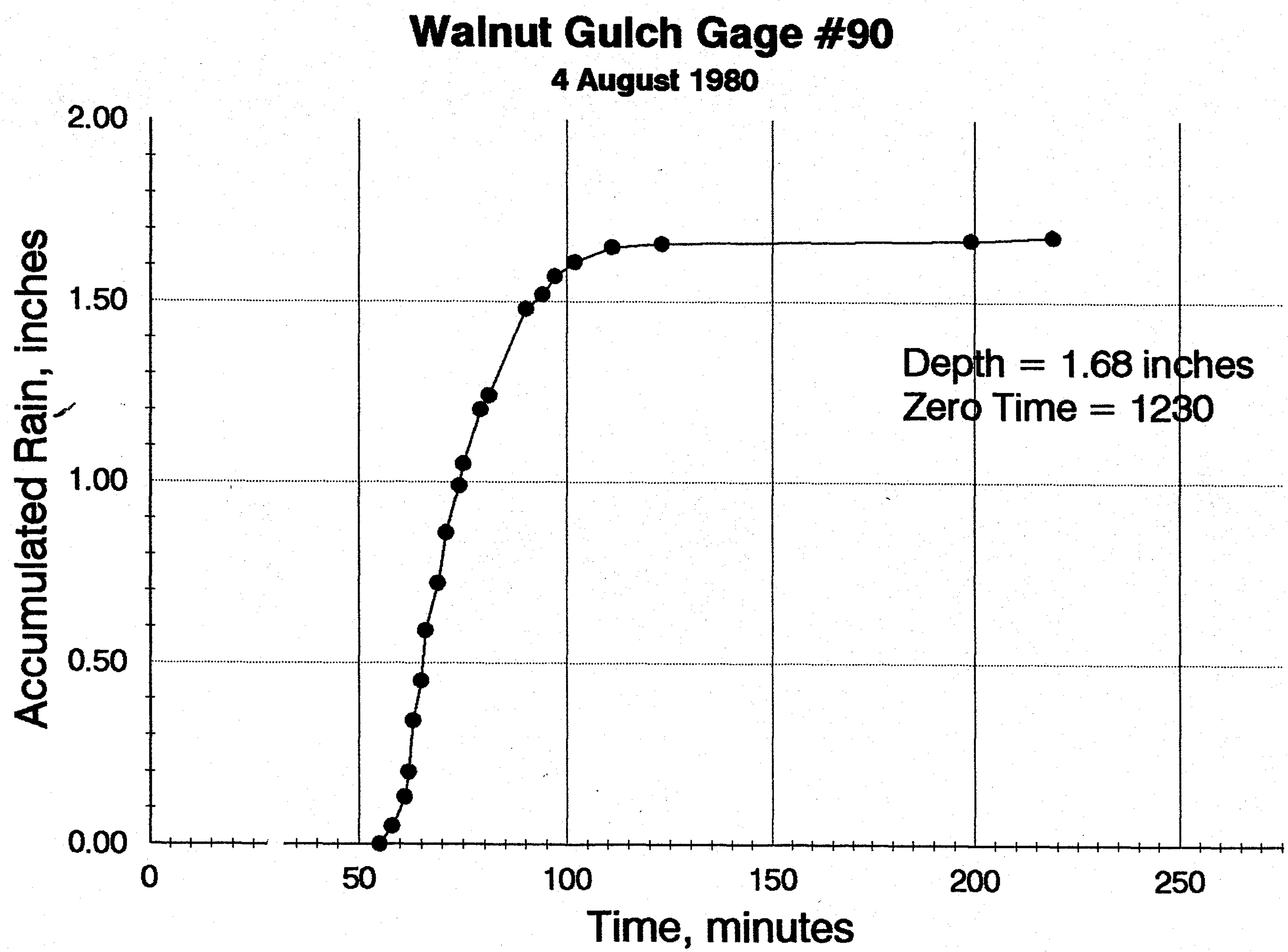


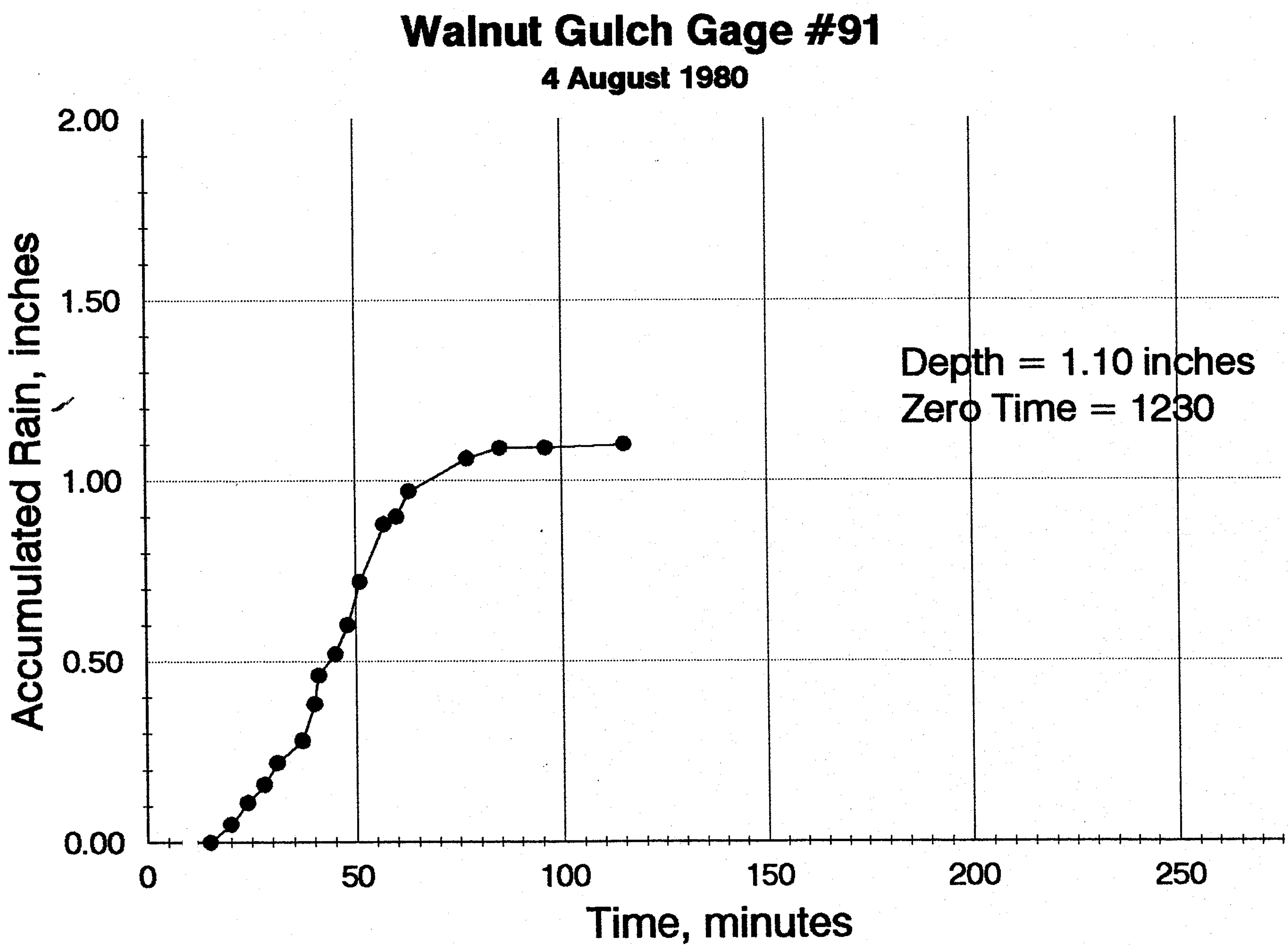
Walnut Gulch Gage #88
4 August 1980
Accumulated Rain, inches
0.00
0.50
1.00
1.50
2.00
0
50
100
150
200
250
Time, minutes
Depth = 1.33 inches
Zero Time = 1230

Walnut Gulch Gage #89

4 August 1980




Walnut Gulch Gage #90
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
Time, minutes
Depth = 1.68 inches
Zero Time = 1230


Walnut Gulch Gage #91
4 August 1980
Accumulated Rain, inches
2.00
1.50
1.00
0.50
0.00
0
50
100
150
200
250
Time, minutes
Depth = 1.10 inches
Zero Time = 1230

George V. Sabol Consulting Engineers, Inc.
1351 EAST 141st AVENUE
BRIGHTON, COLORADO 80601
(303) 457-4015

FLOOD CONT... RE...
SEP 23 1991
CH ENG | P & PM
DC | 1 | HYDRO
ADMIN | LMGT
FINANCE | FILE
C & O | 3 | SDW
ENGR | 2 | JJT
REMARKS
MB

19 September 1991

Mr. Stephen D. Waters
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Subject: Rational Method - documentation on C frequency factor for the revised Manual

Dear Steve:

In your phone call of 17 September 1991, you requested documentation on the factors to be used to correct the Rational Method C for various return periods. The factors that are to be recommended in the revision to the Manual are provided in several references, but I cannot track the original reference or who is responsible.

I performed extensive research on this topic in the preparation of the ADOT Hydrology Manual. Technical Memorandum No. 6 was prepared on this topic for ADOT. The District received copies of my ADOT Technical Memorandums and these should be on file at the District. You may want to refer to that memorandum for additional information. If you cannot locate a copy of that memorandum, let either me or Joe Rumann know. A copy of all of the ADOT memorandums are available in the Phoenix office and Joe would help you in obtaining copies of any of these. The following should be adequate to answer the District's concern on this topic. The ADOT memorandum will provide additional information, if desired.

First, the Maricopa County C coefficients (Table 3.2) are basically the ASCE table (1960, revised in 1969) with some minor deviations and a few additional categories that were not included in the original ASCE table. A copy of the ASCE table is included. Notice that at the bottom of that table is the following statement:

> "The coefficients in these two tabulations are applicable for storms of 5- to 10-yr frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff."

However, no guidance is provided as to how the coefficients are to be adjusted upward for use with less frequent storms.

Somewhere along the way, and as of yet, an unknown source applied the following frequency coefficients to the ASCE table:

Return Period, yrs	Frequency Coefficient
2-10	1.00
25	1.10
50	1.20
100	1.25

Those same factors are suggested to be added to the revised Maricopa County Manual.

Several other hydrology manuals have been found to use the ASCE table along with the above frequency coefficients. Included in this group are:

City of Gilette, WY, Drainage Criteria Manual
U.S. Dept. of Trans., Hydrology for Transportation Eng., 1980
Road and Trans. Assoc. of Canada, Drainage Manual
City of Stillwater, OK, Drainage and Flood Control Criteria Manual, 1988
AASHTO, Model Drainage Manual, 1988.

A more extensive search would probably find many more references in this group.

Another group of manuals were found to contain C tables for which C is listed according to return period (with larger Cs for less frequent storms). Included in this group are:

Austin, TX, Drainage Criteria Manual, 1987
Denver, CO, Drainage Criteria Manual, 1984
Colorado Springs, CO, Drainage Criteria Manual, 1986.

Again, a more extensive search would identify more in this group.

My conclusions are:

1. A frequency correction, of some sort, should be used with the Rational Method.
2. The C coefficients in the Maricopa County Manual are for 2- to 10-yr return period and should be adjusted for other return periods.
3. The frequency coefficients that are recommended are generally accepted in the profession.
4. Other methods can be used to achieve the desired result (such as a C table with C as a function of return period), but the method presented in the Maricopa County Manual (with the revision as noted) is adequate.

Please contact me if you need additional information or support on this.

I suggest that this letter be made a part of the Documentation for the Manual.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo Sabol

George V. Sabol

Attachment: Copy of ASCE table of C coefficients

Copy: Mr. J.M. Rumann, GVSCE, Phoenix

DESIGN AND CONSTRUCTION OF SANITARY AND STORM SEWERS

Prepared by a
Joint Committee of the American Society of Civil Engineers
and
the Water Pollution Control Federation

ASCE
345 East 47th St.
New York, NY. 10016

WPCF
2626 Pennsylvania Avenue, N.W.
Washington, D.C. 20037

1986
(First Printing 1969)

HISTORY OF THE MANUAL

The original manual was the result of seven years' work by a joint committee of ASCE and WPCF (then FSIWA) members and was copyrighted and published in 1960. The members of the full committee on the original manual included:

ASCE	WPCF
Richard R. Kennedy	C. Gordon Gaither
Joseph C. Lawler	H. Sidwell Smith
Ray E. Lawrence	Leland L. Spahr
Raymond R. Ribal	Charles R. Velzy
Bernal H. Swab,* *Chairman*	Samuel I. Zack

In addition, 35 task force members assisted the full committee on the original manual and an editing committee, composed of the following, reviewed and arranged the material for publication:

H. H. Benjes	J. C. Lawler
S. W. Jens	H. S. Smith
R. R. Kennedy	A. L. Tholin

B. H. Swab,* *Chairman*

The original manual was well received and both ASCE and WPCF valued it as one of their most important publications.

In 1964 after the results of an extensive poll indicated that it would be worthwhile to consider revisions to the original manual, both WPCF and ASCE approved the formation of a Joint Committee on Revision of Manual of Practice of Design and Construction of Sanitary and Storm Sewers. The following were members of the full committee:

Roy Aaron	Richard D. Pomeroy
Paul L. Andrews	Lincoln W. Ryder
John S. Autry	Bernal H. Swab *
David G. Chase	Royal C. Thayer
Clarence E. Cuyler	Charles R. Velzy
Glenn E. Hands	Cay G. Weinel, Jr.
Paul A. Kuhn	Lloyd W. Weller
Cecil M. Pepperman	

Joseph C. Lawler, *Chairman*

storm, is common. The range of coefficients, classified with respect to the general character of the tributary area reported in use, is:

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

It often is desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. This procedure often is applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Coefficients with respect to surface type currently in use are:

Character of Surface	Runoff Coefficients
Pavement	
Asphaltic and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The coefficients in these two tabulations are applicable for storms of 5- to 10-yr frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

(*c*) **Coefficients Varying with Time.**—Figure 11 shows the variation of the runoff coefficient with respect to length of time of prior wetting,

RESEARCH PROJECT NO. HPR-PL-1(31)281

ADOT HIGHWAY DRAINAGE DESIGN MANUAL

HYDROLOGY

GEORGE V. SABOL CONSULTING ENGINEERS, INC.
PHOENIX, ARIZONA
and
BRIGHTON, COLORADO

SUBCONTRACTOR TO:

NBS/LOWRY ENGINEERS & PLANNERS

PHOENIX, ARIZONA

5 November 1991

WORKING PAPER NO. 5

RATIONAL METHOD

PREPARED FOR

ARIZONA TRANSPORTATION RESEARCH CENTER

PHOENIX, ARIZONA

RATIONAL METHOD

INTRODUCTION

General Discussion

The Rational Method, as presented herein, can be used to estimate peak discharges, the runoff hydrograph shape, and runoff volume for small, uniform drainage areas that are not larger than 160 acres in size. The method is usually used to size drainage structures for the peak discharge of a selected return period. An extension of the basic method is provided to estimate the shape of the runoff hydrograph if it is necessary to design retention/detention facilities and/or to design drainage facilities that will require routing of the runoff hydrograph through the structure.

The Rational Method is based on the equation

$$Q = CiA \tag{5-1}$$

where Q is the peak discharge, in cfs, of selected return period,
C is the runoff coefficient,
i is the average rainfall intensity, in in/hr, of calculated rainfall duration for the selected rainfall return period, and
A is the contributing drainage area, in acres.

PROCEDURE

General Considerations

1. Depending on the intended application, the runoff coefficient (C) should be selected based on the character of the existing land surface or the projected land surface under future development conditions. In some situations, it may be necessary to estimate C for both existing conditions and for future conditions.
2. Land-use must be carefully considered because the evaluation of land-use will affect both the estimation of C and also the estimation of the watershed time of concentration (T_c).
3. The peak discharge (Q) is generally quite sensitive to the calculation of T_c and care must be exercised in obtaining the most appropriate estimate of T_c.
4. Both C and the rainfall intensity (i) will vary if peak discharges for different flood return periods are desired.

5. Since the T_c equation is a function of rainfall intensity (i), T_c will also vary for different flood return periods.

Applications and Limitations

1. The total drainage area must be less than or equal to 160 acres.
2. The land-use of the contributing area must be fairly consistent over the entire area; that is, the area should not consist of a large percentage of two or more land-uses, such as 50 percent commercial and 50 percent undeveloped. This will lead to inconsistent estimates of T_c (and therefore i) and errors in selecting the most appropriate C coefficient.
3. The contributing drainage area cannot have drainage structures or other facilities within the area that would require flood routing to correctly estimate the discharge at the point of interest.
4. Drainage areas that do not meet the above conditions will require the use of an appropriate rainfall-runoff model (the HEC-1 Program) to estimate flood discharges.

Estimation of Area (A)

An adequate topographic map of the drainage area and surrounding land is needed to define the drainage boundary and to estimate the area (A), in acres. The map should be supplemented with aerial photographs, if available, especially if the area is developed. If the area is presently undeveloped but is to undergo development, then the land development plan and maps should be obtained because these may indicate a change in the drainage boundary due to road construction or land grade changes. If development plans are not available, then land-use should be based on current zoning of the area.

The delineation of the drainage boundary needs to be carefully determined. The contributing drainage area for a lower intensity storm does not always coincide with the drainage area for more intense storms. This is particularly true for urban areas where roads can form a drainage boundary for small storms but more intense storm runoff can cross roadway crowns, curbs, etc. resulting in larger contributing area. Floods on alluvial fans (active and inactive) and in distributary flow systems can result in increased contributing drainage areas during larger and more intense storms. It is generally prudent to consider the largest reasonable drainage area in such situations.

Estimation of Rainfall Intensity (i)

The intensity (i) in Equation 5-1 is the average rainfall intensity in inches/hour for the period of maximum rainfall of a specified return period (frequency) having a duration equal to the time of concentration (T_c) for the drainage area. The frequency is usually specified according to a design criteria or standard for the intended application. The rainfall intensity (i) is obtained from an intensity-duration-frequency (I-D-F) graph. Two methods can be used for obtaining I-D-F information; 1) two generalized I-D-F graphs are provided that can be used for any site in Arizona, and 2) a site specific I-D-F graph can be developed, if desired. The two generalized I-D-F graphs are shown in Figures 5-1 for Zone 6, and Figure 5-2 for Zone 8. The delineation of Zone for Arizona is shown in Figure ____ of the RAINFALL SECTION. Procedures for developing a site specific I-D-F graph are described in the RAINFALL SECTION.

The intensity (i) in Equation 5-1 is the average intensity from the I-D-F graph of the selected return period for the rainfall duration that is equal to the time of concentration (T_c) as calculated according to the procedure described below. A minimum rainfall duration of 10 minutes is to be used if the calculated T_c is <u>less</u> than 10 minutes. The Rational Method should not be used if the calculated T_c is greater than 60 minutes.

Estimation of Time of Concentration (T_c)

Time of concentration (T_c) is to be calculated by Equation 5-2:

$$T_c = 11.4\ L^{0.5}\ K_b^{0.52}\ S^{-0.31}\ i^{-0.38} \qquad (5\text{-}2)$$

where T_c is the time of concentration, in hours,
L is the length of the longest flow path, in miles,
K_b is the watershed resistance coefficient,
S is the slope of the longest flow path, in ft/mile, and
i is the average rainfall intensity, in in/hr, for a duration of rainfall equal to T_c (the same i as Equation 5-1) unless

T_c is less than 10 minutes, in which case the i of Equation 5-1 is for a 10-minute duration).

The longest flow path will be estimated from the best available map and the length (L) measured from the map.

The slope (S) will be calculated by one of two methods: If the longest flow path has a uniform gradient with no appreciable grade breaks, then the slope is calculated by Equation 5-3;

$$S = \frac{H}{L} \qquad (5\text{-}3)$$

where H is the change in elevation, in feet, along L, and
L is as defined in Equation 5-2.

If the longest flow path does not have a uniform gradient or has distinct grade breaks, then the slope is calculated by Equation 5-4:

$$S = 5{,}280 \left(\frac{l}{I}\right)^2 \qquad (5\text{-}4)$$

where $l = 5{,}280\ (L)$,

$$I = \sum \left(\frac{l_i^3}{H_i}\right)^{1/2}$$

and l_i is an incremental change in length, in <u>feet</u>, along the longest watercourse, and

H_i is an incremental change in elevation, in feet, for each length segment, l_i.

The resistance coefficient (K_b) is selected from Table 5-1. Use of Table 5-1 requires a classification as to the landform and the determination of the nature of runoff; whether in a defined drainage network of rills, gullies, channels, etc., or predominantly as overland flow.

The solution of Equation 5-2 is an iterative process since the determination of i requires the knowledge of the value of T_c. Therefore, Equation 5-2 will be solved by a trial-and-error procedure. After L, K_b, and S are estimated and after the appropriate I-D-F graph is selected or prepared, a value for T_c will be guessed (a trial value) and i will be read from the I-D-F graph for the corresponding value of duration = T_c. That i will be used in Equation 5-2 and T_c will be calculated. If the calculated value of T_c does not equal the trial value of T_c, then the process is repeated until the calculated and trial values of T_c are acceptably close (a difference of less than 10 percent should be acceptable).

Selection of Runoff Coefficient (C)

The runoff coefficient (C) is selected from Figures 5-3 through 5-8 depending on the classification of the nature of the watershed. It may be required to select the appropriate C value for existing conditions and another C value for anticipated future conditions, if the watershed is undergoing development.

Note: Estimation of peak discharges for various conditions of development in the drainage area or for different return periods will also require separate estimates of T_c for each existing or assumed land-use condition and for each flood return period.

Estimation of Hydrograph Shape

This procedure is to be used if a runoff hydrograph is needed for the design of a detention basin, pump station, or for any other purpose where routing of the storm inflow through the drainage structure is desired. The procedure is based on synthesizing a hydrograph from the peak discharge estimated by the Rational Method and by the use of some dimensionless hydrograph shapes from TR-55 (Soil Conservation Service, 1986). Two sets of dimensionless hydrographs are provided; one set is for use with urbanized watersheds (Table 5-2), and the other set is for use with undeveloped watersheds (Table 5-3). Both sets of dimensionless unit hydrographs are functions of T_c.

INSTRUCTIONS

A. For estimating peak discharge:

1. Determine the size of the contributing drainage are (A), in acres.
2. Decide whether the generalized I-D-F graphs will be used or whether a site-specific I-D-F graph will be developed.
   a.) If the generalized I-D-F graphs are to be used, determine the Zone from Figure ____ of the RAINFALL SECTION. Use the I-D-F graph of Figure 5-1 if the watershed is in Zone 6, and use Figure 5-2 if the watershed is in Zone 8.
   b.) If a site-specific I-D-F graph is to be used, develop the I-D-F graph by procedures in the RAINFALL SECTION.
3. Select the desired return period(s).
4. Determine the 1-hour rainfall depth (P_1) for each return period. Note: P_1 = 1-hr rainfall intensity times 1 hour.
5. Estimate the time of concentration (T_c), for each return period, by Equation 5-2.
6. Select the rainfall intensity (i) from the I-D-F graph at a duration equal to T_c which is the value of i used in the solution of Equation 5-2 (but not less than 10 minutes).
7. Estimate C:
   a.) If the watershed is developed, use Figure 5-3. This will require an appraisal of development type and percent <u>total</u> impervious area. C is selected as a function of P_1 and type of development.
   b.) If the watershed is undeveloped, use Figures 5-4 through 5-8. This will require an appraisal of Hydrologic Soil Group (HSG), A through D, from Soil conservation Service (SCS) soils reports, and an estimate of percent vegetation cover. C is selected as a function of P_1, and HSG-percent vegetation cover.
8. Calculate the peak discharge by Equation 5-1.

B. For estimating a runoff hydrograph:

1. Calculate Q according to the above instructions.
2. Select the appropriate dimensionless hydrograph coordinates to use from Table 5-2 or Table 5-3. The selection is based on T_c (round to the nearest T_c value in the tables) and on whether the drainage area is urbanized or undeveloped.

3. Read the maximum unit peak discharge, $q_{t_{max}}$ for the selected dimensionless hydrograph and computed T_c value in either Table 5-2 or Table 5-3.

4. Calculate

$$K = Q/q_{t_{max}} .$$

5. Tabulate the time and q_t values from either Table 5-2 or Table 5-3 and multiply each q_t by K

$$q = Kq_t .$$

6. Plot the hydrograph discharge (q) versus time.

7. Draw a smooth hydrograph. This may require extending the rising limb of the hydrograph to intersect the 0 discharge axis.

REFERENCES

Soil Conservation Service, 1986, Urban Hydrology for Small Watersheds: Technical Release No. 55.

TABLE 5-1

Resistance Coefficient (K_b) for use with the Rational Method T_c Equation

Description of Landform	K_b with defined drainage network	K_b overland flow, only
Mountain, with forest and dense ground cover (overland slopes – 50% or greater)	0.15	0.30
Mountain, with rough rock and boulder cover (overland slopes – 50% or greater)	0.12	0.25
Foothills (overland slopes – 10% to 50%)	0.10	0.20
Alluvial fans, Pediments and Rangeland (overland slopes – 10% or less)	0.05	0.10
Irrigated Pasture [a]	—	0.20
Tilled Agricultural Fields [a]	—	0.08
Urban		
Residential, L less than 1,000ft [b]	0.04	—
Residential, L greater than 1,000ft [b]	0.025	—
Grass; parks, cemetaries, etc. [a]	—	0.20
Bare ground; playgrounds, etc. [a]	—	0.08
Paved; parking lots, etc. [a]	—	0.02

Notes: a – No defined drainage network.
b – L is length in the T_c equation. Streets serve as drainage network.

TABLE 5-2

Urban Watershed – Coordinates (q_t) of Dimensionless Hydrograph to be used with the Rational Method

Values in csm/inch runoff

HYDROGRAPH TIME (HOURS)

T_c hours	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
.17	24	34	53	334	647	1010	623	217	147	123	104	86	76	66	57	51	46	42	38	34	32	29	26	23	21	20	19	18	15	13	12	0
.18 – .25	23	31	47	209	403	739	800	481	250	166	128	102	86	70	61	54	49	44	40	35	33	30	27	24	21	20	19	18	16	13	12	0
.26 – .35	20	28	41	118	235	447	676	676	459	283	196	146	114	80	66	57	51	46	42	37	33	31	28	24	22	20	19	18	16	13	12	0
.36 – .45	18	25	36	77	141	271	468	592	574	431	298	216	163	104	77	63	55	49	44	38	34	31	28	25	22	21	20	18	16	14	12	0
.46 – .62	17	23	32	57	94	170	308	467	529	507	402	297	226	140	96	74	61	53	47	41	36	32	29	26	23	21	20	19	16	14	12	0
.63 – .88	13	18	24	36	46	68	115	194	294	380	424	410	369	252	172	123	93	74	61	49	41	35	31	27	24	22	20	19	17	15	12	0
.89 – 1.12	11	15	20	29	35	47	72	112	168	231	289	329	357	313	239	175	133	103	83	63	50	40	33	29	26	23	21	20	17	15	12	0
1.13 – 1.38	10	13	18	25	29	38	54	81	118	163	213	256	284	311	266	212	163	129	104	78	61	47	37	31	27	24	22	20	18	16	12	1
1.39 – 1.75	9	11	15	21	25	31	41	58	82	112	147	184	216	255	275	236	198	159	129	98	76	57	43	35	30	25	23	21	18	16	12	1
1.76 – 2.5	7	9	12	16	18	21	27	36	49	64	82	104	127	171	201	226	208	193	171	132	105	79	58	45	36	30	26	23	20	17	13	3

Note: The maximum unit peak discharge, $q_{t\,max}$) is highlighted for each hydrograph.

Reference: TR-55 (1986), Exhibit 5-II for $I_a/P = 0.10$ and Travel Time = 0.0

TABLE 5-3

Undeveloped Watershed – Coordinates (qt) of Dimensionless Hydrograph to be used with the Rational Method

Values in csm/inch runoff

Tc hours	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
.17	0	0	0	0	70	539	377	196	171	154	134	117	108	99	89	83	77	72	67	61	59	56	51	46	43	42	40	38	34	30	28	0
.18–.25	0	0	0	0	7	98	371	322	221	182	158	137	120	104	94	86	80	74	69	62	60	57	52	47	44	42	40	39	35	30	28	0
.26–.35	0	0	0	0	1	25	151	299	277	219	187	162	141	113	100	90	84	78	72	65	61	58	53	48	44	42	41	39	35	31	28	0
.36–.45	0	0	0	0	0	7	59	168	245	257	213	186	163	128	109	96	88	81	75	67	62	58	54	50	45	43	41	39	35	31	28	0
.46–.62	0	0	0	0	0	2	26	89	170	217	229	200	179	144	119	104	93	85	78	70	64	59	55	51	46	43	41	40	36	32	28	0
.63–.88	0	0	0	0	0	0	2	16	45	92	137	166	185	170	146	125	110	98	89	79	70	63	58	53	48	44	42	41	37	33	28	0
.89–1.12	0	0	0	0	0	0	1	7	21	42	71	101	126	160	154	138	123	110	100	87	77	67	60	55	50	46	43	41	38	34	28	1
1.13–1.38	0	0	0	0	0	0	1	5	13	26	44	68	91	125	142	142	128	117	107	94	83	72	63	57	52	47	44	42	38	34	28	2
1.39–1.75	0	0	0	0	0	0	0	3	8	16	27	42	59	92	116	128	130	121	112	100	90	78	67	60	55	50	46	43	39	35	29	4
1.76–2.5	0	0	0	0	0	0	0	1	4	8	13	20	28	51	73	92	104	111	112	106	97	86	75	66	60	54	49	46	41	37	30	7

Note: The maximum unit peak discharge, ($q_{t\,max}$) is highlighted for each hydrograph.

Reference: TR-55 (1986), Exhibit 5-II for $I_a/P = 0.50$ and Travel Time = 0.0

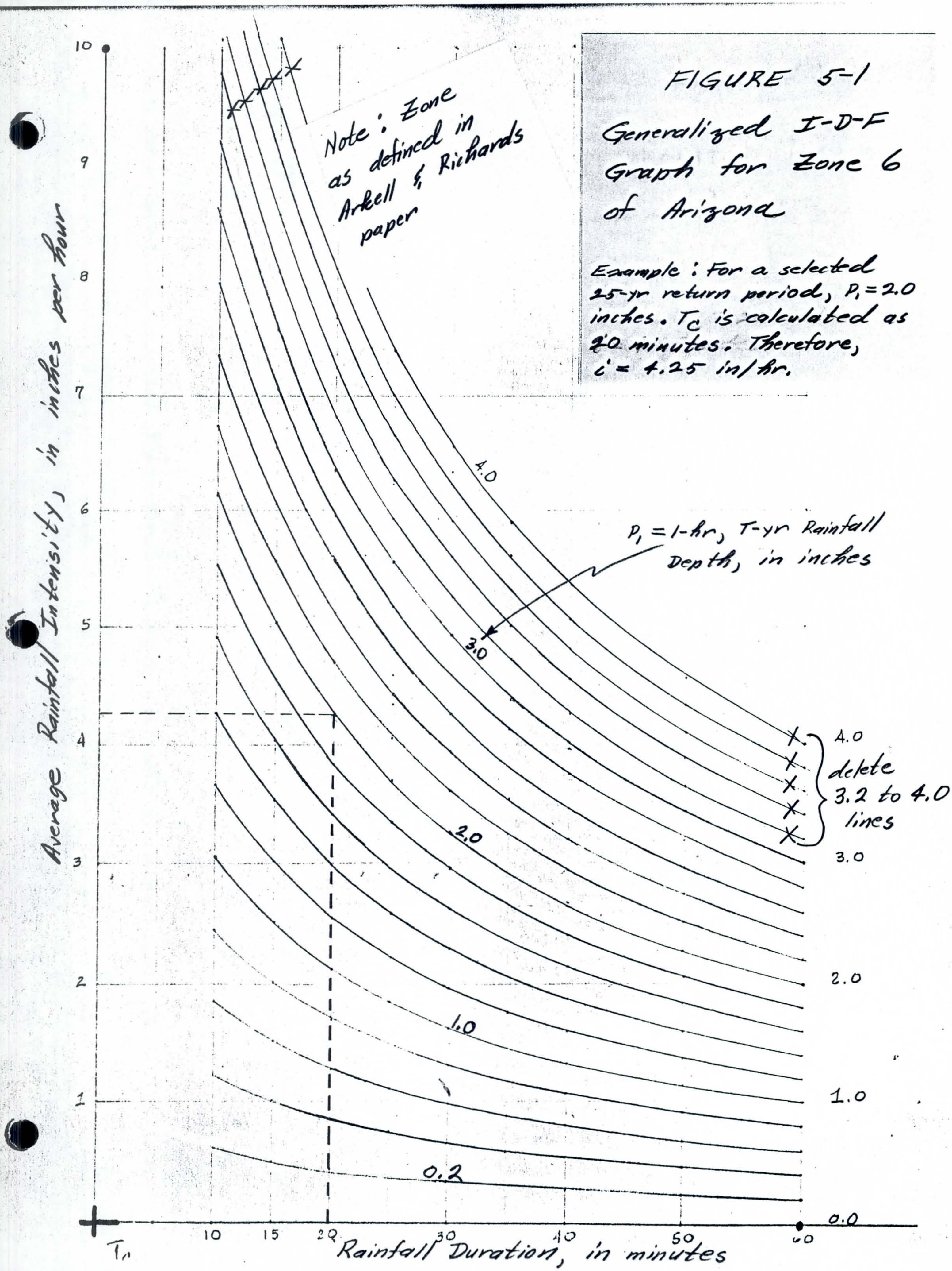


FIGURE 5-1
Generalized I-D-F Graph for Zone 6 of Arizona
Example : For a selected 25-yr return period, P1 = 2.0 inches. TC is calculated as 20 minutes. Therefore, i = 4.25 in/hr.
Note : Zone as defined in Arkell & Richards paper
P1 = 1-hr, T-yr Rainfall Depth, in inches
delete 3.2 to 4.0 lines
4.0
3.0
2.0
1.0
0.2
4.0
3.0
2.0
1.0
0.0
Average Rainfall Intensity, in inches per hour
10
9
8
7
6
5
4
3
2
1
Tn
10
15
20
30
40
50
60
Rainfall Duration, in minutes

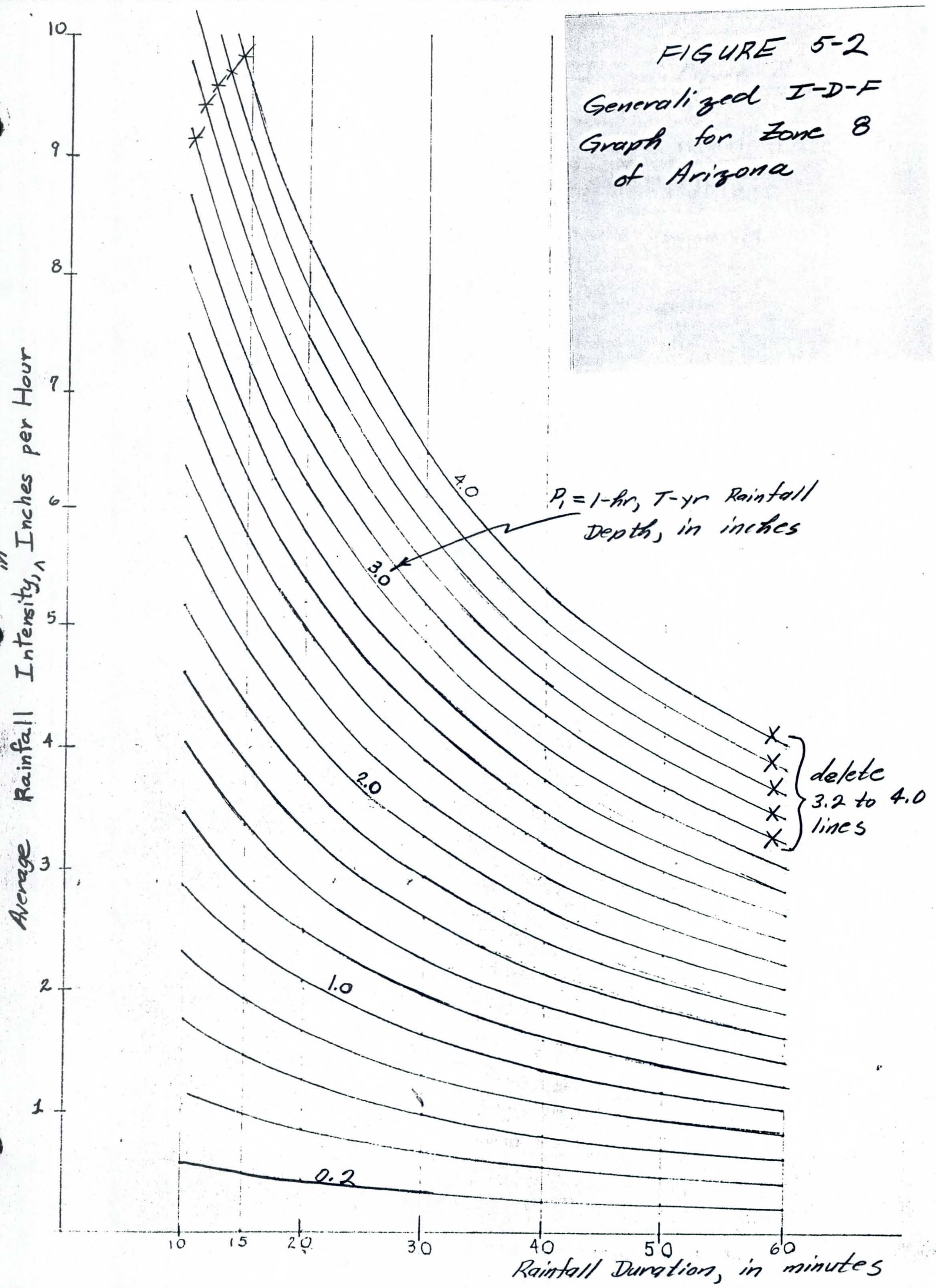
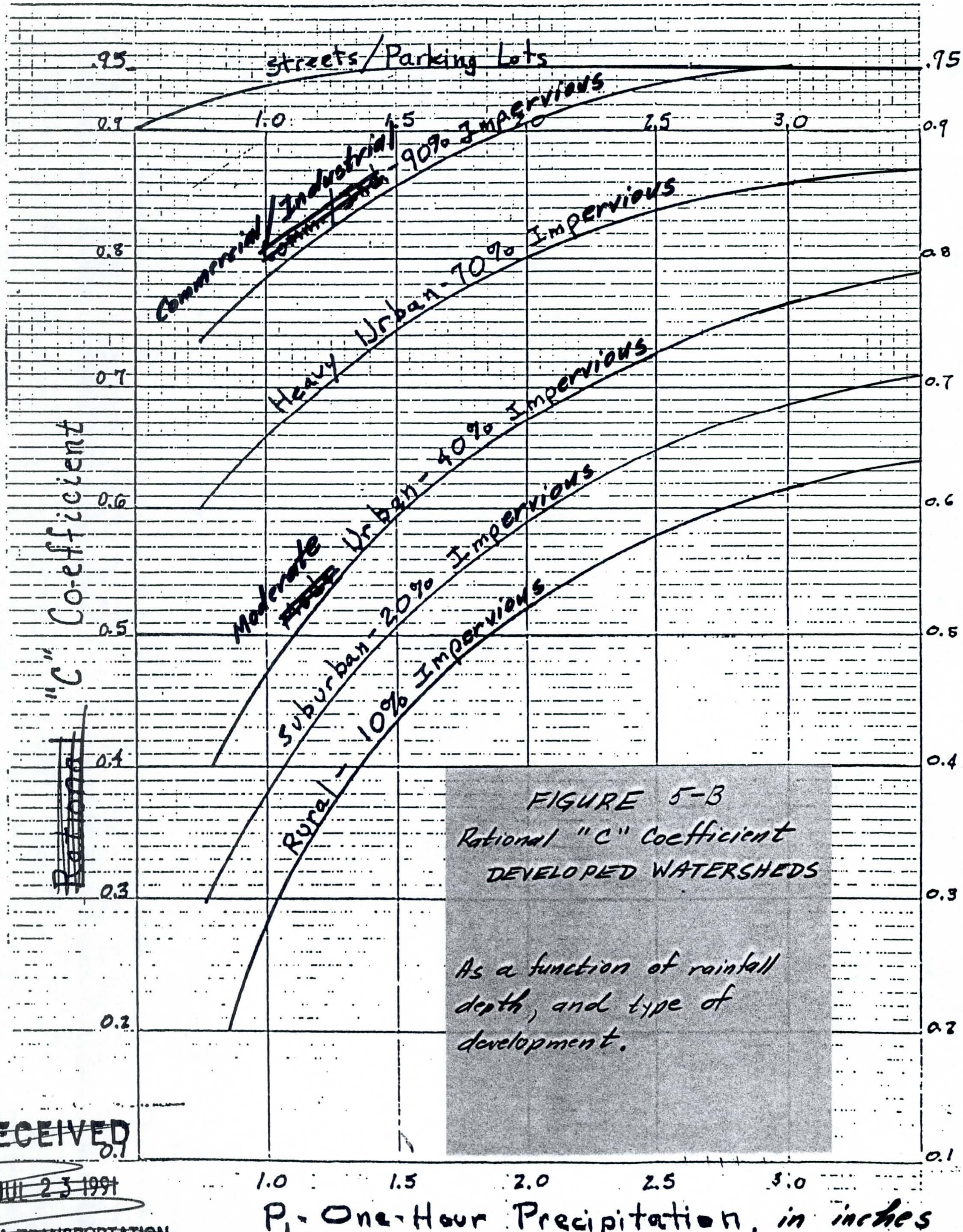


FIGURE 5-2
Generalized I-D-F Graph for Zone 8 of Arizona
Average Rainfall Intensity, in Inches per Hour
Raintall Duration, in minutes
P_1 = 1-hr, T-yr Rainfall Depth, in inches
4.0
3.0
2.0
1.0
0.2
delete 3.2 to 4.0 lines
10
9
8
7
6
5
4
3
2
1
10
15
20
30
40
50
60



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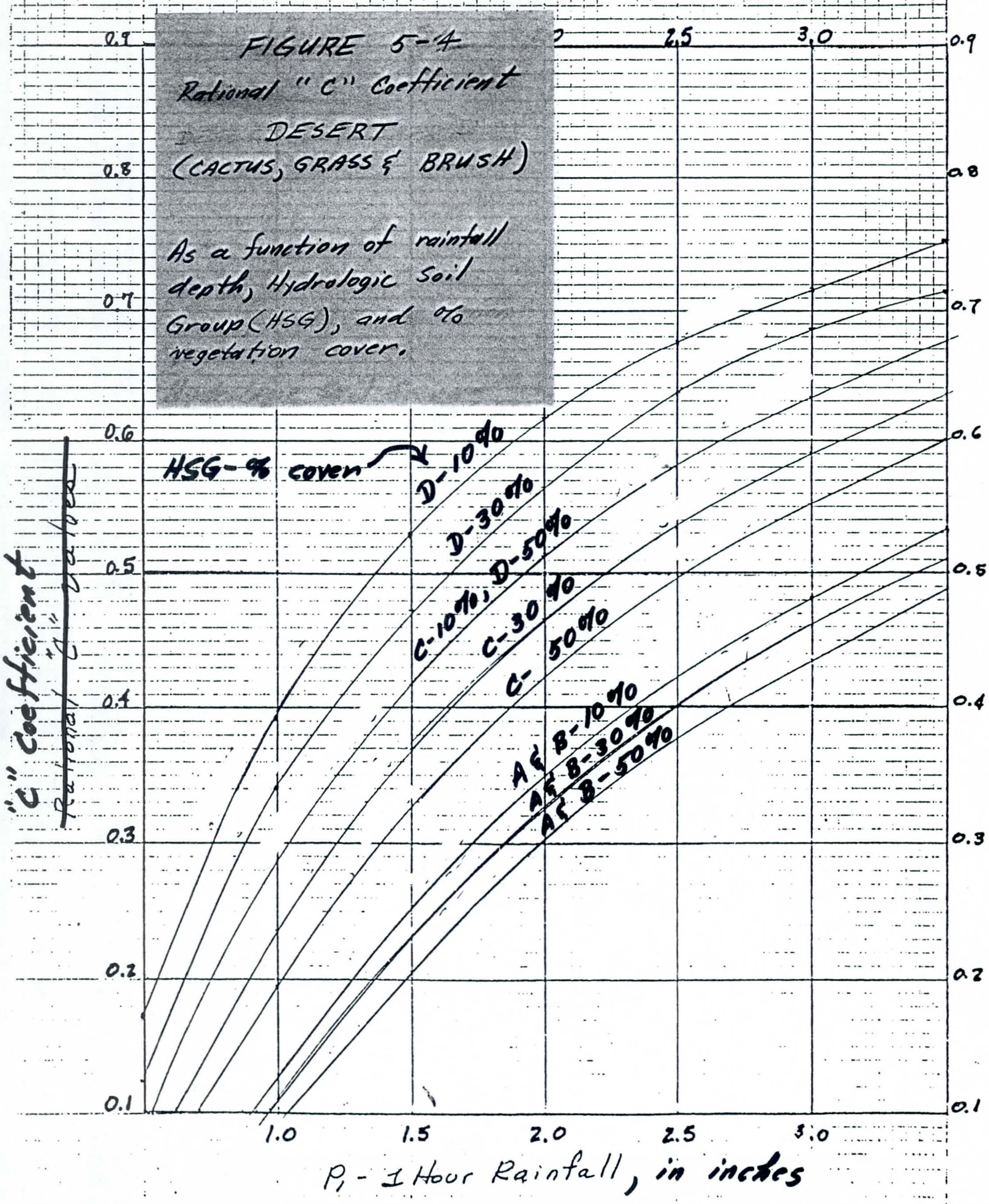
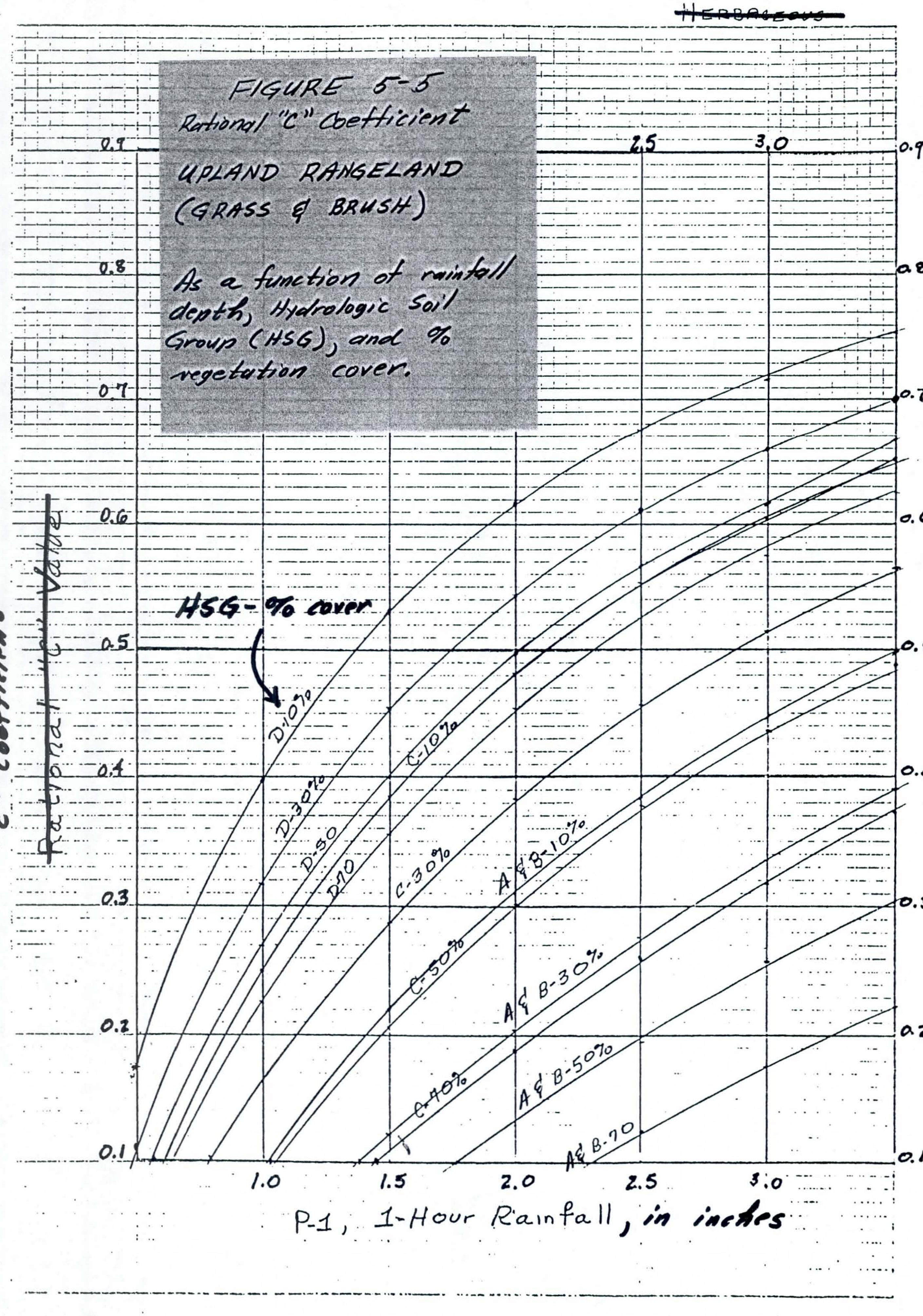
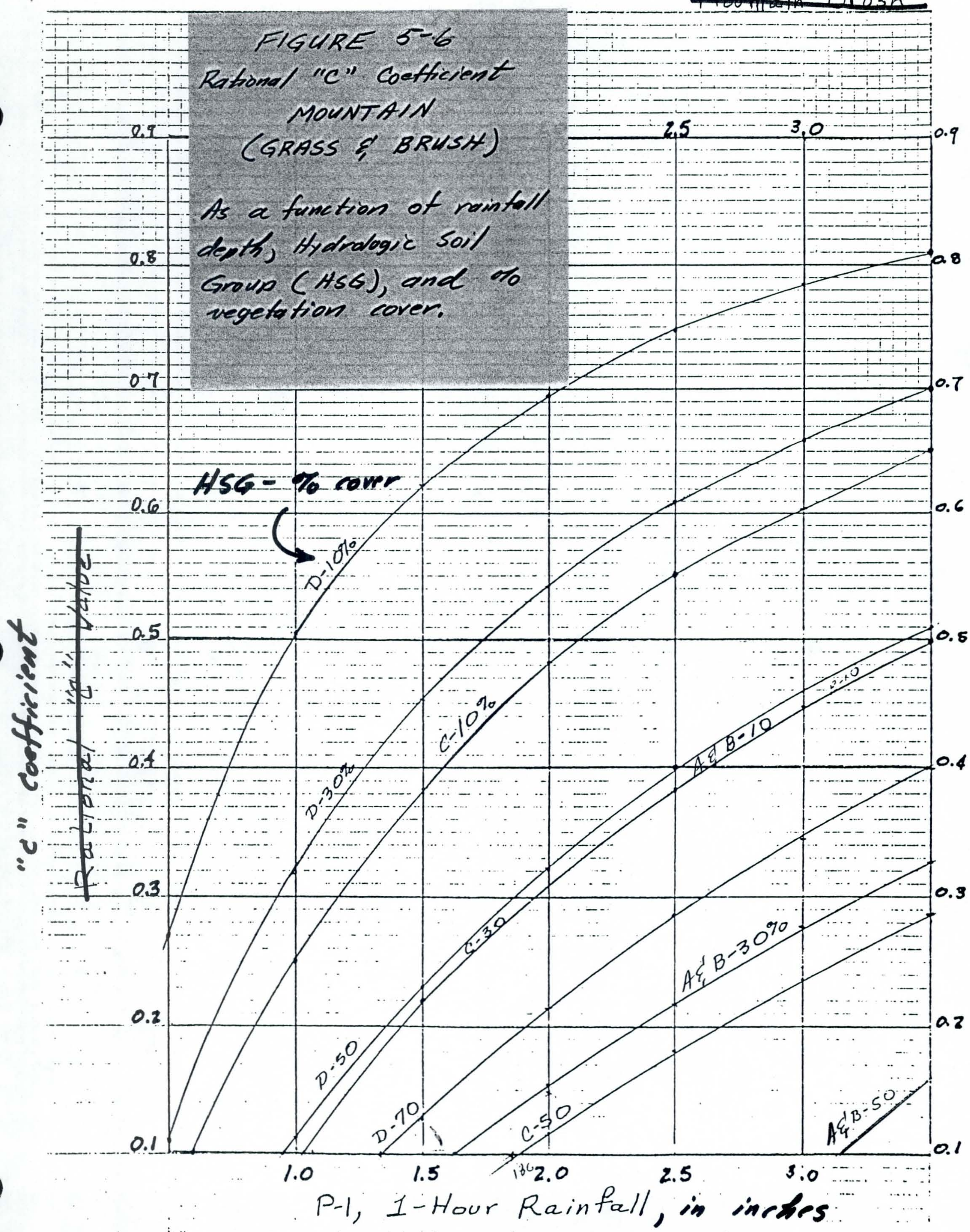


FIGURE 5-4
Rational "C" Coefficient
DESERT
(CACTUS, GRASS & BRUSH)
As a function of rainfall depth, Hydrologic Soil Group (HSG), and % vegetation cover.
HSG-% cover
D-10%
D-30%
D-50%
C-10%, D-50%
C-30%
C- 50%
A & B-10%
A & B-30%
A & B-50%
"C" Coefficient
Rational "C" Values
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.5
2.0
2.5
3.0
P_1 - 1 Hour Rainfall, in inches



~~HERBACEOUS~~
FIGURE 5-5
Rational "C" Coefficient
UPLAND RANGELAND
(GRASS & BRUSH)
As a function of rainfall depth, Hydrologic Soil Group (HSG), and % vegetation cover.
HSG - % cover
D-10%
D-30%
D-50
D-70
C-10%
C-30%
C-50%
C-70%
A & B-10%
A & B-30%
A & B-50%
A & B-70
"C" Coefficient
~~Rational "C" Value~~
P-1, 1-Hour Rainfall, in inches
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.5
2.0
2.5
3.0

Mountain Brush



~~Juniper / Grass~~

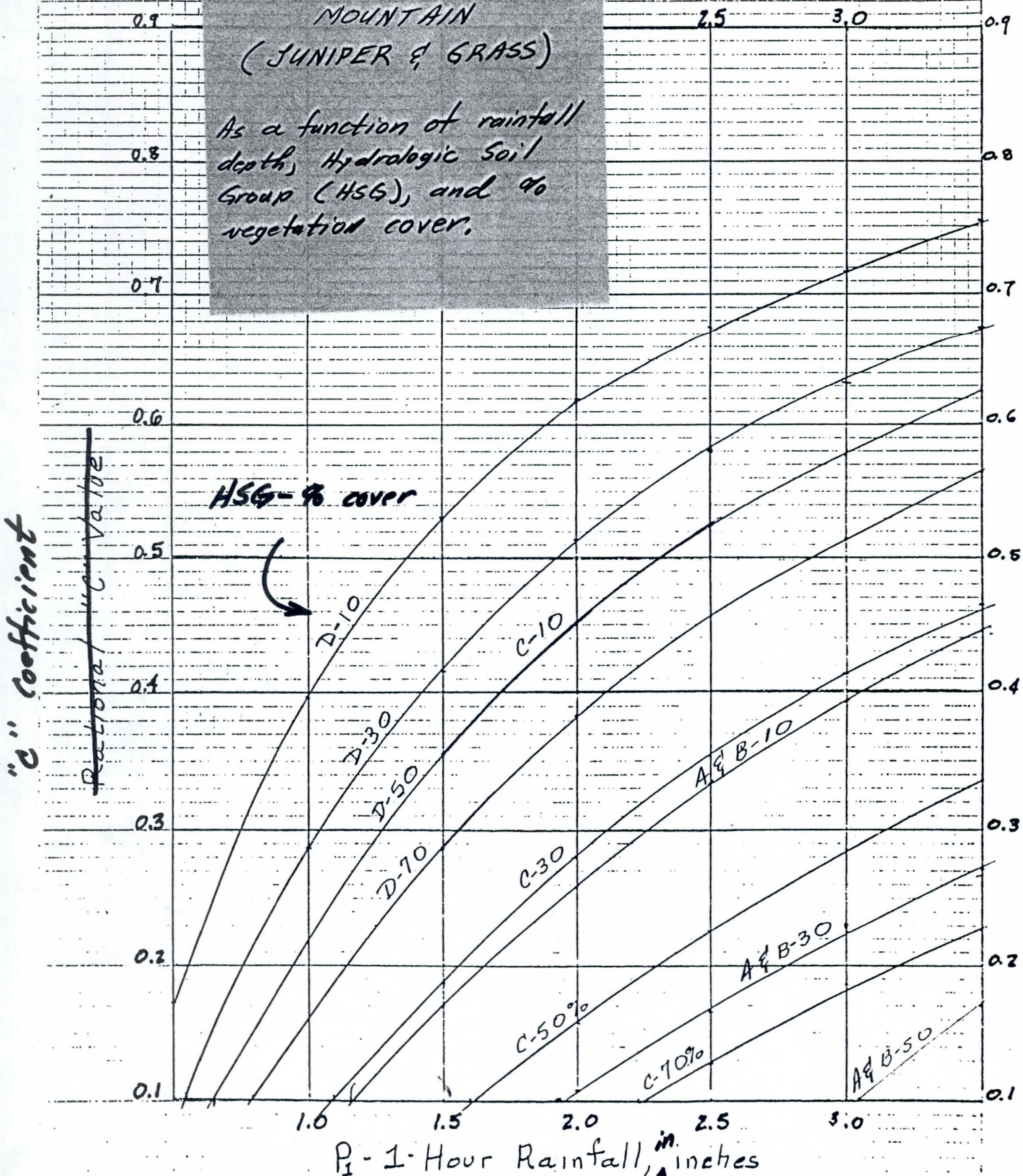
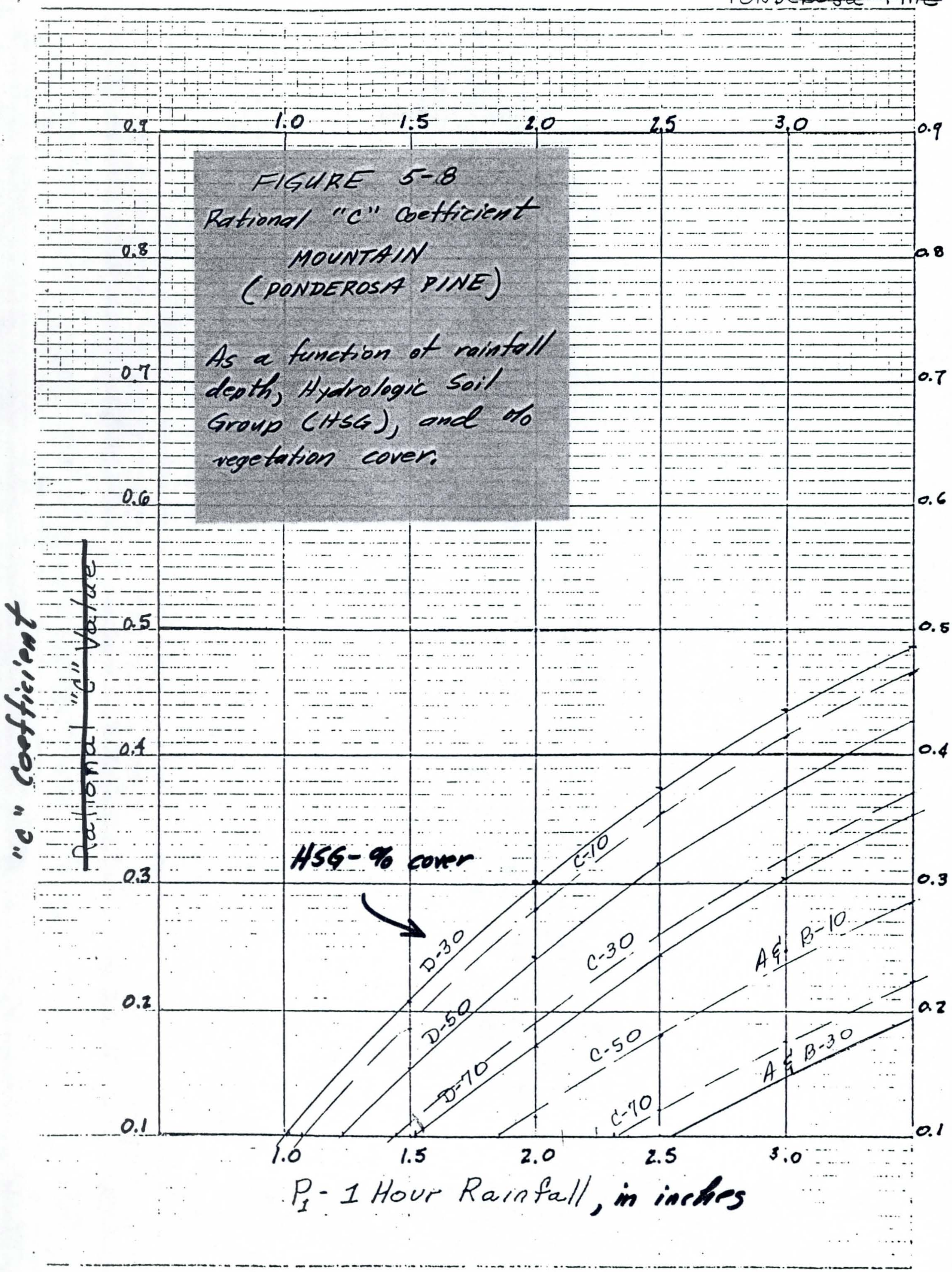


FIGURE 5-J
Rational "C" coefficient
MOUNTAIN
(JUNIPER & GRASS)

As a function of rainfall depth, Hydrologic Soil Group (HSG), and % vegetation cover.

~~Ponderosa Pine~~



VEG COVER

EXAMPLE: Calculate the 100-yr peak discharge and estimate the runoff hydrograph for a 60 acre, single-family residential (about 20% impervious) watershed in Phoenix. The following are the watershed characteristics:

$A = 60$ acres

$S = 25$ ft/mi

$L = 0.7$ mile

The following were obtained for the watershed:

$P_1 = 2.5$ inches from the NOAA Atlas

$K_b = .025$ from Table 5-1

$C = .65$ from Figure 5-3

This example is solved using A) a site-specific I-D-F graph, and B) using the generalized I-D-F graph.

A) Using the site-specific I-D-F graph (shown):

Solve for T_C:

$$T_C = 11.4 L^{.5} K_b^{.52} S^{-.31} i^{-.38}$$

$$T_C = 11.4(.7^{.5})(.025^{.52})(25^{-.31})\, i^{-.38}$$

$$= .52\, i^{-.38}$$

Trial T_C, hr	i, in/hr	Calculated T_C, hr
.75	3.0	.34
.30	5.4	.27
.27	5.8	.27 OK

Calculate Q:

$$Q = c\,i\,A$$
$$= (.65)(5.8)(60)$$
$$= 226 \text{ cfs}$$

B) Using the generalized I-D-F graph (Figure 5-2 for Zone 8):

Solve for T_C:

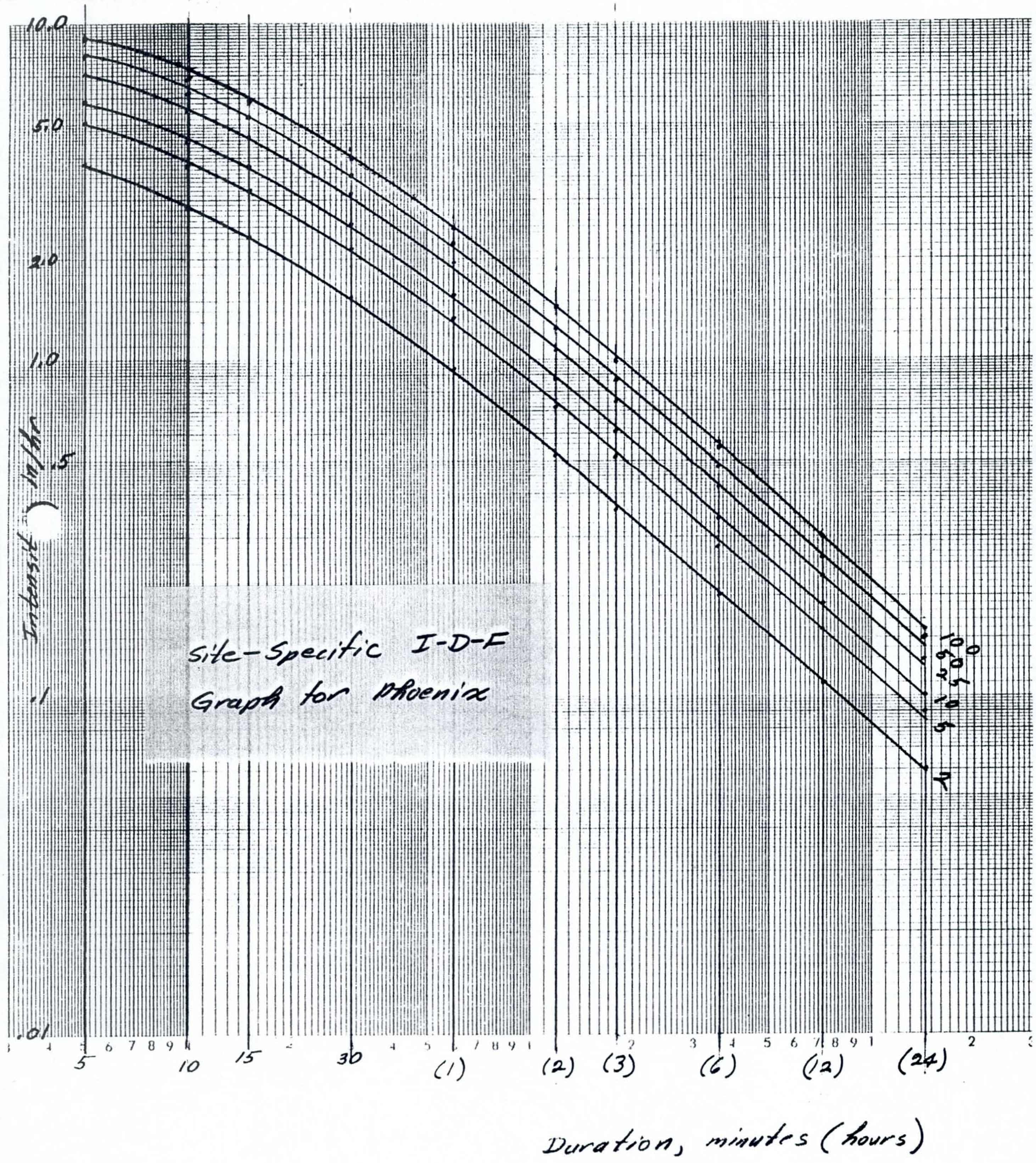
$$T_C = .52\,i^{-.38}$$

Trial T_C, hr	i, in/hr	Calculated T_C, hr
.33 (20 minutes)	5.2	.28
.27 (16 minutes)	6.3	.26 OK

Calculate Q:

$$Q = c\,i\,A$$
$$= (.65)(6.3)(60)$$
$$= 246 \text{ cfs}$$

The hydrograph shape is calculated using the Q that was calculated using the site-specific I-D-F graph.



Logarithmic 3 X 3 Cycles

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PROJECT ______ DATE ______

SUBJECT ______ BY ______

PROJECT NO. ______

Estimate the hydrograph shape:
Use the urban, dimensionless hydrograph from Table 5-2 for $T_C = .26$ to $.35$ hr.

$$q_{t_{max}} = 676$$

$$K = Q / q_{t_{max}} = 226/676 = .33$$

Tabulated Hydrograph		Runoff Hydrograph	Volume Calculation	
Time	q_t	$q = K q_t$	$\bar{q}_t$	$\bar{q}(\Delta t)$
11.0	20	7	8	2.4
11.3	28	9	12	3.6
11.6	41	14	27	8.1
11.9	118	39	59	5.9
12.0	235	79	114	11.4
12.1	447	149	188	18.8
12.2	676	226	226	22.6
12.3	676	226	190	19.0
12.4	459	153	124	12.4
12.5	283	95	81	8.1
12.6	196	66	58	5.8
12.7	146	49	44	4.4
12.8	114	38	33	6.6
13.0	80	27	25	5.0
13.2	66	22	21	4.2
13.4	57	19	18	3.6
13.6	51	17	16	3.2
13.8	46	15	14	2.8
14.0	42	14	13	3.9
14.3	37	12	12	3.6
14.6	33	11	10	4.0
15.0	31	10	10	5.0
15.5	28	9	8	4.0
16.0	24	8	8	4.0
16.5	22	7	7	3.5
17.0	20	7	6	3.0
17.5	19	6	6	3.0
18.0	18	6	6	6.0
19.0	16	5	4	4.0
20.0	12	4	4	8.0
22.0	12	4	2	8.0
26.0	0	0	0	
				207.9 cfs-hr
				(17.2 ac-ft)

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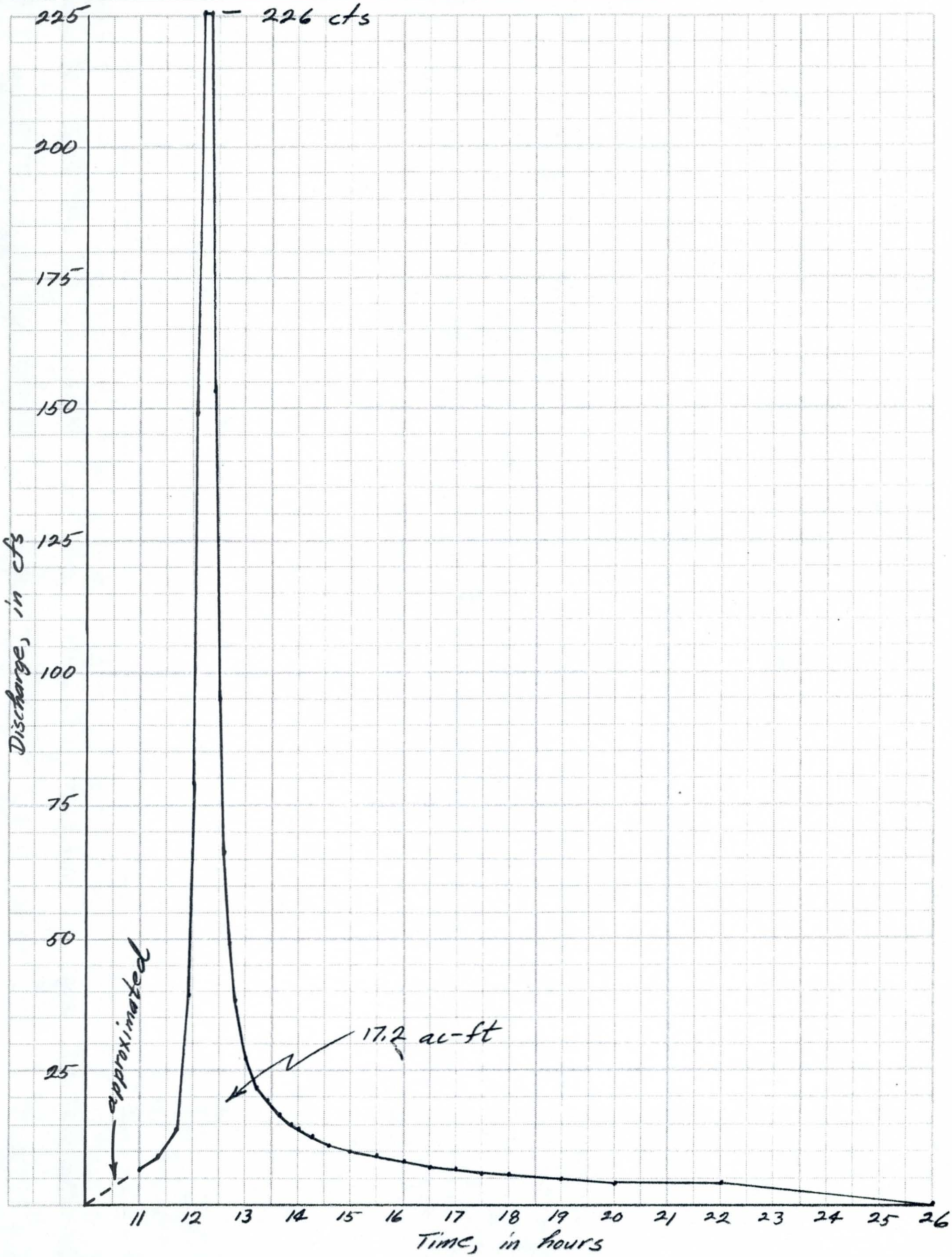
PROJECT NO. ______



new City of Austin table

TABLE 2-2

RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

Runoff Coefficient (C)

Character of Surface	Return Period 2 Years	5 Years	10 Years	25 Years	50 Years	100 Years	500 Years
DEVELOPED							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/Roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass Areas (Lawns, Parks, etc.)							
Poor Condition (grass cover less than 50 percent of the area)							
Flat, 0-2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2-7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, Over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair Condition (grass cover on 50 to 75 percent of the area)							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, Over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good Condition (grass cover larger than 75 percent of the area)							
Flat, 0-2 %	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2-7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, Over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
UNDEVELOPED							
Cultivated Land							
Flat, 0-2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2-7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, Over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/Range							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, Over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/Woodlands							
Flat, 0-2%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2-7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, Over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

Source: 1. Rossmiller, R.L. "The Rational Formula Revisited."
2. City of Austin, Watershed Management Division.

1988 version of City of Austin C table

Austin Drnge Criteria Manual

Austin Dept of T and Pub Services 1988

TABLE 2-2. RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

Runoff Coefficient (C)

Character of Surface	Return Period 2 Years	5 Years	10 Years	25 Years	50 Years	100 Years	500 Years
DEVELOPED							
Asphaltic	0.73	0.77	0.81	0.86	0.90	0.95	1.00
Concrete/Roof	0.75	0.80	0.83	0.88	0.92	0.97	1.00
Grass Areas (Lawns, Parks, etc.)							
Poor Condition (grass cover less than 50 percent of the area)							
Flat, 0-2%	0.32	0.34	0.37	0.40	0.44	0.47	0.58
Average, 2-7%	0.37	0.40	0.43	0.46	0.49	0.53	0.61
Steep, Over 7%	0.40	0.43	0.45	0.49	0.52	0.55	0.62
Fair Condition (grass cover on 50 to 75 percent of the area)							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, Over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Good Condition (grass cover larger than 75 percent of the area)							
Flat, 0-2 %	0.21	0.23	0.25	0.29	0.32	0.36	0.49
Average, 2-7%	0.29	0.32	0.35	0.39	0.42	0.46	0.56
Steep, Over 7%	0.34	0.37	0.40	0.44	0.47	0.51	0.58
UNDEVELOPED							
Cultivated Land							
Flat, 0-2%	0.31	0.34	0.36	0.40	0.43	0.47	0.57
Average, 2-7%	0.35	0.38	0.41	0.44	0.48	0.51	0.60
Steep, Over 7%	0.39	0.42	0.44	0.48	0.51	0.54	0.61
Pasture/Range							
Flat, 0-2%	0.25	0.28	0.30	0.34	0.37	0.41	0.53
Average, 2-7%	0.33	0.36	0.38	0.42	0.45	0.49	0.58
Steep, Over 7%	0.37	0.40	0.42	0.46	0.49	0.53	0.60
Forest/Woodlands							
Flat, 0-2%	0.22	0.25	0.28	0.31	0.35	0.39	0.48
Average, 2-7%	0.31	0.34	0.36	0.40	0.43	0.47	0.56
Steep, Over 7%	0.35	0.39	0.41	0.45	0.48	0.52	0.58

1987 version of City of Austin C table

TABLE 2 - 3

RATIONAL METHOD RUNOFF COEFFICIENTS BY LAND USE TYPE FOR USE IN Q = CiA

Runoff Coefficient (C)

Land Use*	I.C.** %	Slope	2 Years	5 Years	10 Years	25 Years	50 Years	100 Years	500 Years
			Return Period						
Rural Resi-		0-2%	0.38	0.42	0.44	0.48	0.52	0.56	0.68
dential (RR)	25	2-7%	0.39	0.43	0.46	0.50	0.54	0.58	0.71
		7% +	0.40	0.44	0.47	0.52	0.56	0.60	0.73
Single Family		0-2%	0.44	0.47	0.51	0.55	0.59	0.64	0.76
(SF-1)	40	2-7%	0.46	0.49	0.53	0.57	0.61	0.66	0.79
		7% +	0.47	0.51	0.54	0.59	0.62	0.68	0.80
Multifamily		0-2%	0.46	0.49	0.53	0.58	0.62	0.66	0.78
(MF-1)	45	2-7%	0.48	0.51	0.55	0.60	0.64	0.68	0.81
		7% +	0.49	0.53	0.56	0.61	0.65	0.70	0.82
Neighborhood		0-2%	0.49	0.52	0.56	0.60	0.64	0.69	0.81
Commercial(LR)	50	2-7%	0.50	0.54	0.57	0.62	0.66	0.71	0.83
		7% +	0.51	0.55	0.58	0.63	0.68	0.72	0.84
General		0-2%	0.53	0.57	0.61	0.65	0.69	0.74	0.86
Office (GO)	60	2-7%	0.54	0.59	0.62	0.67	0.71	0.76	0.88
		7% +	0.55	0.60	0.63	0.68	0.72	0.77	0.89
Multifamily		0-2%	0.58	0.62	0.66	0.71	0.75	0.79	0.91
(MF-4)	70	2-7%	0.59	0.63	0.67	0.72	0.76	0.81	0.93
		7% +	0.60	0.64	0.68	0.73	0.77	0.82	0.94
Limited Ind.		0-2%	0.63	0.68	0.71	0.76	0.80	0.85	0.97
Service (LI)	80	2-7%	0.64	0.69	0.72	0.77	0.81	0.86	0.98
Major Ind.(MI)		7% +	0.65	0.69	0.73	0.78	0.82	0.87	0.99
Commercial		0-2%	0.72	0.76	0.80	0.85	0.89	0.93	1.00
Service (CS)	95	2-7%	0.72	0.77	0.80	0.85	0.89	0.94	1.00
		7% +	0.72	0.77	0.81	0.86	0.90	0.94	1.00

* For specific land use and corresponding maximum impervious coverage values check the zoning ordinance.

**I.C. (Impervious Cover) For impervious cover values not listed in the table, interpolate those values given.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

PROJECT HDM PAGE ____ OF ____

DETAIL TABLE 3.3 COMPUTED SDW DATE 11/26/91

CHECKED BY ____ DATE ____

Rec from GVS 12/18/91

TABLE 3.3
C COEFFICIENTS FOR USE WITH THE RATIONAL METHOD

LAND USE		2-10 yr (1)	25 yr (2)	50 yr (3)	100 yr (4)
			RETURN PERIOD		
STREETS and ROADS					
~~Asphaltic~~ Paved	.75-.85	~~0.70-0.95~~	0.77-0.95	0.84-0.95	0.88-0.95
~~Concrete~~		~~0.80-0.95~~	0.88-0.95	0.95	0.95
Gravel roadways & shoulders		0.40-0.60	0.44-0.66	0.48-0.72	0.50-0.75
INDUSTRIAL AREAS					
~~Flat Commercial (about 90% impervious)~~		~~0.80~~	~~0.88~~	~~0.95~~	~~0.95~~
Heavy	.70-.80	~~0.60-0.90~~	0.66-0.95	0.72-0.95	0.75-0.95
Light	.60-.70	~~0.50-0.80~~	0.55-0.88	0.60-0.95	0.63-0.95
BUSINESS AREAS					
Downtown	.75-.85	~~0.70-0.95~~	0.77-0.95	0.84-0.95	0.88-0.95
Neighborhood	.55-.65	~~0.50-0.70~~	0.55-0.77	0.60-0.95	0.63-0.95
RESIDENTIAL AREAS					
Lawns - flat	.10-.25	~~0.05-0.15~~	0.06-0.17	0.06-0.18	0.06-0.19
- steep	.25-.40	~~0.15-0.35~~	0.17-0.39	0.18-0.42	0.19-0.44
Suburban	.30-.45	~~0.25-0.40~~	0.28-0.44	0.30-0.48	0.31-0.50
Single Family	.45-.55	~~0.30-0.50~~	0.33-0.55	0.36-0.60	0.38-0.63
Multi Unit	.50-.60	~~0.40-0.60~~	0.44-0.66	0.48-0.72	0.50-0.75
Apartment	.60-.70	~~0.50-0.70~~	0.55-0.77	0.60-0.84	0.63-0.88
PARKS, CEMETERIES		0.10-0.25	0.11-0.28	0.12-0.30	0.13-0.31
PLAYGROUNDS	.40-.50	~~0.20-0.30~~	0.22-0.33	0.24-0.36	0.25-0.38
AGRICULTURAL AREAS		0.10-0.20	0.11-0.22	0.12-0.24	0.13-0.25
BARE GROUND		0.20-0.30	0.22-0.33	0.24-0.36	0.25-0.38
UNDEVELOPED DESERT		0.30-0.40	0.33-0.44	0.36-0.48	0.38-0.50
MOUNTAIN TERRAIN (SLOPES > 10%)		0.60-0.80	0.66-0.88	0.72-0.95	0.75-0.95

NOTE: C values in columns 2, 3, and 4 were derived using frequency adjustment factors of 1.10, 1.20, and 1.25 respectively, with an upper limit of 0.95.

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FLOOD CONTROL DISTRICT
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ADMIN		LMGT
FINANCE		FILE
C & O		
ENGR		

REMARKS

9 December 1991

Mr. Stephen D. Waters
Flood Control District of Maricopa County
2801 West Durango
Phoenix, Arizona 85009

Subject: Rational Method - documentation of the source of the frequency factor for C

Dear Steve:

At your request, I conducted further "research" into the source of the frequency factor that occasionally appears in the literature and manuals in regard to adjusting the C coefficient for return periods larger than 10-yr. I had previously reported on this subject in my letter to you of 19 September 1991. I have, I believe, discovered the source of the frequency factors.

I had several discussions with Mr. Ben Urbonas and Mr. Kevin Stewart of the Urban Drainage and Flood Control District (UD&FCD) in Denver. Mr. Urbonas has been with UD&FCD throughout the development and evolution of its Drainage Criteria Manual. That manual was first released in 1969 and was prepared by Wright-McLaughlin Engineers of Denver under the direction of Mr. Ken Wright. The 1969 version of the manual contains the same frequency factor table that I show in my letter of 19 September. Mr. Urbonas indicated that those factors were selected by Mr. Wright, probably based on some generalized results of data that had been collected on rainfall and runoff for small, urban watersheds in Denver. Therefore, the "source" of these frequency factors appears to be the 1969 version of the UD&FCD manual.

The UD&FCD revised parts of its manual on several occasions. There are 1984 and 1989 modifications to the Rational Method and related C tables. I have enclosed pages from all three (1969, 1984, and 1989) versions of the manual that discuss and show the C coefficient tables. Notice that the 1969 version has a C table that is stated to be applicable for flood return periods of 5- to 10-yr, and these are to be used <u>with</u> the shown frequency factors for longer return periods.

The 1984 version provides a C table that is a function of return period, negating the need for a separate table of frequency factors. This is the approach that you are recommending and I believe that it is preferable in lieu of the separate frequency factor table. The 1989 version is the same as the 1984 version with the exception of the modification of some of the C values in the table.

I believe that this solves the mystery of the unknown source for the C frequency factor table.

In searching this out, I came upon an interesting piece of literature concerning the Rational Method by Dr. Ronald L. Rossmiller. Pages 11-13 contain a discussion of the variation of C with rainfall intensity. I think that this does a particularly good job of illustrating why C should increase with return period. I hope that this is of use to you.

Contact me if you have questions, but I trust that this should answer your questions on this topic.

Sincerely yours,
George V. Sabol Consulting Engineers, Inc.

Geo Sabol

George V. Sabol

Enclosures: 1. Copies of portions of various UD&FCD manuals
2. Copy of paper by R.L. Rossmiller

3.4 Continued

value is calculated from the velocity of flow as given by the Manning Formula for hydraulic conditions prevailing in the pipes.

The inlet time can be estimated by calculating the various overland distances and flow velocities taken from the most remote point. A common mistake is to assume velocities that are too small for the areas near the collectors. Another common error is to not review the runoff from only a part of the basin which is sometimes greater than that computed for the whole basin. This error is most often encountered in long basins, or a basin where the upper portion contains grassy park land and the lower developed urban land. Often the remote areas have flow that is very shallow and in this case the velocities cannot be calculated by "channel" equations such as Manning's but special overland flow analysis must be considered (11). Figure 3-1 can be used to help estimate time of surface flow.

When studying proposed subdivision land do not necessarily take the overland flow path perpendicular to the contours since the land will be graded and swales will often intercept the natural contour and conduct the water to the streets thus cutting down on the time of concentration.

3.5 Intensity

The intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration.

After the design storm frequency has been selected, a graph should be made showing rainfall intensity versus time. The procedure for obtaining the local data and drawing the graph is explained and illustrated by Example 3 in the Rainfall Part of this Manual.

3.6 Runoff Coefficient

The runoff coefficient, C, is the variable of the Rational Method least susceptible to precise determination and requires judgment and understanding on the part of the engineer (10). Its use in the formula implies a fixed ratio for any given drainage area. In reality this is not the case. The coefficient represents the integrated effects of infiltration, detention storage, evaporation, retention, flow routing, and interception which all affect the time distribution and peak rate of runoff.

Table 3-1 presents recommended ranges for C values.

3.6 Continued

TABLE 3-1 (8)

RATIONAL METHOD RUNOFF COEFFICIENTS

Description of Area	Runoff Coefficients
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single-family areas	0.35 to 0.50
Multi units, detached	0.40 to 0.60
Multi units, attached	0.60 to 0.75
Residential (1/2 acre lots or more)	0.30 to 0.45
Apartment dwelling areas	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area. This procedure is often applied to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Suggested coefficients with respect to surface type are given in Table 3-2. See the Storm Sewers Part of this Manual for a discussion of the use of the Rational Method in conjunction with the use of on site ponding and roof ponding.

3.6 Continued

TABLE 3-2 (8)

RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

Character of Surface	Runoff Coefficients
Streets:	
Asphaltic	0.70 to 0.95
Concrete	0.80 to 0.95
Drives and Walks	0.75 to 0.85
Roofs	0.75 to 0.95
Lawns, Sandy Soil:	
Flat, 2%	0.05 to 0.10
Average, 2 to 7%	0.10 to 0.15
Steep, 7%	0.15 to 0.20
Lawns, Heavy Soil:	
Flat, 2%	0.15 to 0.20
Average, 2 to 7%	0.20 to 0.25
Steep, 7%	0.25 to 0.35

The coefficients in these two tabulations are applicable for storms of 5-year to 10-year frequencies. Less frequent higher-intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff, as given in the following section.

3.7 Adjustment for Infrequent Storms

The adjustment of the Rational Method for use with major storms can be made by multiplying the right side of the Rational Formula by a frequency factor C_f, which is used to account for antecedent precipitation conditions. The Rational Formula now becomes:

$$Q = CIAC_f \qquad (3\text{-}2)$$

The following table of C_f values can be used. The product of C times C_f should not exceed 1.0.

3.7 Continued

TABLE 3-3

FREQUENCY FACTORS FOR RATIONAL FORMULA

Recurrence Interval (years)	C_f
2 to 10	1.0
25	1.1
50	1.2
100	1.25

3.8 Application of the Rational Method

The first step in applying the Rational Method is to obtain a good topographic map and define the boundaries of all of the relevant drainage basins. Basins to be defined include all basins tributary to the area of study and subbasins in the study area. A field check and possibly field surveys should be made for each basin. At this stage of planning, the possibility of the diversion of transbasin waters should be investigated.

Transbasin diversions out of the study area should also be kept in mind. The engineer should be very cautious when reducing a design flow due to a transbasin export, particularly for the major storm analysis. See Colorado Drainage Law in this Manual for comments on liability concerning transbasin waters.

The major storm drainage basin does not always coincide with the minor storm drainage basin. This is often the case in urban areas where a low flow will stay next to a curb and follow the lowest grade, but when a large flow occurs the water will be deep enough so that part of the water will overflow street crowns and flow into a new subbasin.

For an example of how to apply the Rational Method refer to the Storm Sewer Part of this Manual.

3.9 Major Storm Analysis

When analyzing the major runoff occurring on an area that has a storm sewer system sized for the initial storm, care must be used when applying the Rational Method. Normal application of the Rational Method

The travel times are then accumulated in a downstream direction to calculate the time of concentration at each successive downstream design points.

3.5 Intensity

The intensity, I, is the average rainfall rate in inches per hour for the period of maximum rainfall of a given frequency having a duration equal to the time of concentration.

After the design storm frequency has been selected, a graph should be made showing rainfall intensity versus time. The procedure for obtaining the local data and drawings the graph is explained and illustrated by Example 4.2 of the RAINFALL part of this Manual.

3.6 Runoff Coefficient

The runoff coefficient, C, represents the integrated effects of infiltration, evaporation, retention, flow routing, and interception, all which effect the time distribution and peak rate of runoff. Its determination requires judgement and understanding on the part of the engineer (10). Table 3-1 presents the recommended values of C for the various recurrence frequency storms (42). The values are presented for different surface characteristics as well as for different aggregate land uses.

Table 3-1 provides runoff coefficients that vary with recurrence frequency. The coefficients were developed using the available rainfall and runoff information in the Denver region and were designed to work in conjunction with the time of concentration recommendations in 3.4. Use of these coefficients and procedure outside of the semi-arid climates found in areas such as the Denver region may not be valid. However, because the coefficients vary with frequency, no further adjustments are needed for large storms as was described in the earlier versions of the Manual.

See the STORM SEWERS part of this Manual for further discussion of the use of the Rational Method.

3.7 Application of the Rational Method

The first step in applying the Rational Method is to obtain a good topograpic map and define the boundaries of all of the relevant drainage basins. Basins to be defined include all basins tributary to the area of

TABLE 3-1 (42)

RECOMMENDED RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS

LAND USE OR SURFACE CHARACTERISTICS	PERCENT IMPERVIOUS	FREQUENCY 2	5	10	100
Business:					
Commercial Areas	95	.87	.87	.88	.89
Neighborhood Areas	70	.60	.65	.70	.80
Residential:					
Single-Family	*	.40	.45	.50	.60
Multi-Unit (detached)	50	.45	.50	.60	.70
Multi-Unit (attached)	70	.60	.65	.70	.80
1/2 Acre Lot or Larger	*	.30	.35	.40	.60
Apartments	70	.65	.70	.70	.80
Industrial:					
Light Areas	80	.71	.72	.76	.82
Heavy Acres	90	.80	.80	.85	.90
Parks, Cemetaries:	7	.10	.10	.35	.60
Playgrounds:	13	.15	.25	.35	.65
Schools:	50	.45	.50	.60	.70
Railroad Yard Areas	40	.40	.45	.50	.60
Undeveloped Areas:					
Historic Flow Analysis- Greenbelts, Agricultural	2	(See "Lawns")			
Offsite Flow Analysis (when land use not defined)	45	.43	.47	.55	.65
Streets:					
Paved	100	.87	.88	.90	.93
Gravel	13	.15	.25	.35	.65
Drive and Walks:	96	.87	.87	.88	.89
Roofs:	90	.80	.85	.90	.90
Lawns, Sandy Soil	0	.00	.01	.05	.20
Lawns, Clayey Soil	0	.05	.10	.20	.40

NOTE: These Rational Formula coefficients may not be valid for large basins.

*See Figure 2-1 for percent impervious.

TABLE 3-1 (42)

RECOMMENDED RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS

LAND USE OR SURFACE CHARACTERISTICS	PERCENT IMPERVIOUS	FREQUENCY 2	5	10	100
Business:					
Commercial Areas	95	.87	.87	.88	.89
Neighborhood Areas	70	.60	.65	.70	.80
Residential:					
Single-Family	*	.40	.45	.50	.60
Multi-Unit (detached)	50	.45	.50	.60	.70
Multi-Unit (attached)	70	.60	.65	.70	.80
1/2 Acre Lot or Larger	*	.30	.35	.40	.60
Apartments	70	.65	.70	.70	.80
Industrial:					
Light Areas	80	.71	.72	.76	.82
Heavy Acres	90	.80	.80	.85	.90
Parks, Cemetaries:	7	.10	.18	.25	.45
Playgrounds:	13	.15	.20	.30	.50
Schools:	50	.45	.50	.60	.70
Railroad Yard Areas	20	.20	.25	.35	.45
Undeveloped Areas:					
Historic Flow Analysis- Greenbelts, Agricultural	2	(See "Lawns")			
Offsite Flow Analysis (when land use not defined)	45	.43	.47	.55	.65
Streets:					
Paved	100	.87	.88	.90	.93
Gravel (Packed)	40	.40	.45	.50	.60
Drive and Walks:	96	.87	.87	.88	.89
Roofs:	90	.80	.85	.90	.90
Lawns, Sandy Soil	0	.00	.01	.05	.20
Lawns, Clayey Soil	0	.05	.15	.25	.50

NOTE: These Rational Formula coefficients may not be valid for large basins.

*See Figure 2-1 for percent impervious.

A Review

of the

Rational Formula

by

Dr. Ronald L. Rossmiller

April 1980

THE RATIONAL FORMULA

Introduction

Many new methodologies involving complex computer programs have been proposed in recent years for the planning and design of urban stormwater management systems. These systems include storm sewers, detention areas and overflow facilities and take into account both the quantity and quality of urban runoff. However, until these emerging methods come into more general use, the smaller storm sewers in the system will continue to be designed using the rational method. Thus, a review of its orgins and present-day interpretations is in order so that designers are reminded of what it is and of what it is not, of its limitations and its many interpretations.

The rational formula consists of four variables: a runoff coefficient, rainfall intensity, drainage area and time of concentration. The definitions of these variables have been expressed in various ways and these have led to some widely-held misconceptions. These misconceptions and the assumptions and limitations of the rational formula are each discussed in turn.

The two variables subject to the widest intrepretation are the runoff coefficient and the time of concentration. Presently, a designer can select values for C for a watershed which differ by two or three times from each other simply by using tables recommended by various agencies and texts. A new formula is proposed for the runoff coefficient which should reduce the present variability in the estimates of C. The same variability exists for estimates of the time of concentration, t_c. Using the same data and presently available equations, estimates of t_c can range from 5 to 35 minutes.

Each of the four variables is discussed in turn and comments are made on the usefulness and shortcomings of several of the tables, equations and figures presently used to estimate these variables. Following this, some examples of the use of the rational formula are given along with some advice on how the rational method should be applied to the design of storm sewers.

As originally conceived, the rational formula yields only a peak discharge rate. However, some engineers also use the rational formula to develop a hydrograph for detention basin design. Two such methods are discussed along with the problems and uncertainties inherent in using the rational formula for hydrograph development. Examples of these two methodologies are given and are compared with the results obtained from using the method contained in TR55 of the Soil Conservation Service.[1]

History of the Formula

The rational formula had its beginnings about 130 years ago. In 1851, T. J. Mulvaney, an Irish engineer, published a paper entitled "On the Use of the Self-registering Rain and Flood Guages in Making Observations on the Relation of Rainfall and Flood Discharges in a Given Catchment" in the Transactions of the Institution of Civil Engineers, Ireland.[2] Though not stated as such, the underlying principles of the rational formula, including the concept of the time of concentration, were definitely implied in his paper.

However, this paper was largely ignored and not until 1889 did the rational formula begin to come into general use. In that year Emil Kuichling, the city engineer of Rochester, New York, presented a paper entitled "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts" before the American Society of Civil Engineers.[3] He indicated that

> "in drainage areas of moderate size, the heaviest discharge always occurs when the rain lasts long enough at its maximum intensity to enable all portions of the area to contribute to the flow."

He concluded

> "that there must be some definite relation between these fluctuations of discharge and the intensity of the rain, also between the magnitude of the drainage area and the time required for the floods to appear and subside."

The rational formula was introduced into England in 1906 by David Ernest Lloyd-Davies in his paper "The Elimination of Storm-Water from Sewerage Systems" before the Institution of Civil Engineers.[4] Thus, in England, the rational formula is known as the Lloyd-Davies formula.

In the next few decades several writers sought to estimate the time of concentration (t_c), runoff coefficient (C) and rainfall intensity (i) more accurately. Some success was achieved with rainfall intensity through the development of intensity-duration-frequency (I-D-F) curves. However, work on t_c and C met with much less success. In the last 40 years, there have been few if any improvements in the use of the rational formula; what has occurred is a proliferation of methods to estimate the various factors in the form of equations, graphs and tables. This movement towards simplicity has resulted generally in some widely-held misconceptions and mediocrity in the use of the formula.

Hydrologic Cycle

Any formula or methodology which estimates a peak discharge rate and/or flood hydrograph must, to a greater or lesser extent, incorporate the several portions of the hydrologic cycle. Thus, a review is necessary to determine to what extent the rational formula meets this test.

The hydrologic cycle consists of an unending sequence of events. Water vapor in the atmosphere is lifted by some mechanism and then falls to the earth's surface as one of several forms of precipitation. In the rational formula, we are only concerned with precipitation which falls as rain. Some rain is intercepted by foliage and structures before it reaches the earth's surface. That which reaches the ground first gets everything wet and then begins to fill the innumerable surface depressions. Only after this depression storage volume is satisfied and if the rainfall intensity is greater than the infiltration rate at that point in time does surface runoff begin. This surface runoff flows overland, then in channels of ever-increasing size until the runoff reaches the ocean. Evaporation from land and water surfaces adds water vapor to the atmosphere and the cycle continues.

At some point in some channel we can measure a runoff hydrograph, the peak of which is estimated by the rational formula. Two other portions of the hydrologic cycle, evapotranspiration and groundwater flow, play an insignificant role in the short time spans, small drainage areas and channels with which the rational formula is concerned and can be neglected.

Definitions of Variables

The rational formula is usually expressed as

$$Q_T = Ci_TA \tag{1}$$

where Q_T is the estimate of the peak rate of runoff in cubic feet per second for some recurrence interval, T; C is the fraction of rainfall, expressed as a dimensionless decimal, that appears as surface runoff from the tributary area (the ratio of surface runoff to rainfall); i_T is the average rainfall intensity in inches per hour during a period of time equal to t_c for some recurrence interval, T; A is the watershed area in acres tributary to the point of design; and t_c is the rainfall intensity averaging time in minutes, usually referred to as the time of concentration, equal to the time required for water to flow from the hydraulically most distant point in the watershed to the point of design.

Precipitation in the hydrologic cycle is included in the rational formula by using the average rainfall intensity over some time period. By default, all other portions of the hydrologic cycle must be contained in the runoff coefficient, C. Therefore, C includes interception, depression storage, infiltration, evaporation and groundwater flow. The variables needed to estimate C should include soil type, land use, degree of imperviousness, watershed slope, surface roughness, antecedent moisture condition, duration of rainfall and the intensity of rainfall as reflected by the recurrence interval. The fewer of these variables used to estimate C, the less accurately will the rational formula reflect the hydrologic cycle.

The peak discharge rate is assumed to vary directly with the magnitude of the drainage area. This assumption makes the equation essentially dimensionally accurate since 1.0 acre-inch per hour is equal to 1.0083 cubic feet per second.

The next logical step would be to discuss each of the variables in the rational formula in detail. However, by first discussing some of the assumptions, limitations and misconceptions of the rational formula, it is hoped that the reader will have a better appreciation for the ensuing discussion of the above variables.

Assumptions and Misconceptions

Assumptions and misconceptions are grouped together because an assumption used in the rational formula might in itself be a misconception or could be a conclusion based on some misconception. Several assumptions are listed below with each followed by a brief discussion.

The peak rate of runoff at some point is a direct function of the tributary drainage and the average rainfall intensity during the time of concentration to that point. This is the rational formula stated in words and is the basis (the basic assumption) of Kuichling's 1889 paper.[3] Sufficient data, both rainfall and runoff records, have not been available to either prove or disprove this hypothesis.

The method assumes that the frequency (recurrence interval) of the peak discharge rate is the same as the frequency of the average rainfall intensity. This is not always the case due to watershed-related variations. However, this assumption is used in many methodologies for estimating peak flows or runoff hydrographs.

The runoff frequency curve is parallel to the rainfall frequency curve. This implies that the same value of the runoff coefficient C is used for all recurrence intervals. However, work done by Schaake, Geyer and Knapp indicates that the two curves tend to converge at the rarer frequency rainfall events.[5]

Each of the variables (C,i,A) is independent of each other and each is estimated separately. This is one of the major misconceptions. There is some interdependency among the variables. Present procedure is to estimate each variable separately from an equation, graph, map or table. A close look at these aids indicates, in most cases, a lack of recognition of any interdependency between these variables.

The time of concentration t_c is the time required for water to flow from the hydraulically most remote point in the watershed to the point of design. Rather than an assumption, the foregoing statement is usually given as the definition of t_c. However, Schaake, Geyer and Knapp have stated that there is no known way to determine t_c, either from measurements in the field during storms or from records of rainfall and runoff and[5]

> "except for steady state conditions, which rarely, if ever, are reached during a thunderstorm, there is no good reason to believe that the time of flow from the farthest point in a drainage area should necessarily be the best rainfall averaging time to use in the Rational Method."

The rainfall intensity remains constant during the time period equal to t_c. Based on rainfall records, this assumption is true for short periods of time, such as a few minutes. However, as the time period increases, this assumption becomes less and less realistic.

The above assumption has led to another assumption: the definition of i in the rational formula. A common definition of i is the rainfall intensity in inches per hour of a storm whose "duration" is equal to the time of concentration of the basin. This definition evolved from current practice or current practice evolved from this definition.

"Duration" has been placed in parentheses because the interpretation placed on "duration" has led to the worst misconception of all. The common interpretation is that the duration of the storm is equal to t_c. This assumption is totally false and misleading. It is, of course, theoretically possible, since rainfall is a random event; however, the much more common case is that the total storm duration is considerably longer than t_c. Of equal importance is the concept that t_c (rainfall intensity averaging time) can occur during any segment of the total storm duration - at the beginning, before, during or after the middle portion or near the end.

This concept also has implications for the runoff coefficient C and how well the rational formula mirrors the hydrologic cycle. If t_c occurs at the beginning of the storm, then the antecedent moisture conditions become important. If t_c occurs near the end of a long storm, then the ground may be saturated and the depressions already filled with water when t_c begins.

Another assumption and misconception is that the area to be used is the total area tributary to the point of design. Kuichling recognized this possibility when he stated that[3]

> "the conclusion is accordingly irresistible that the rates of rainfall adopted in computing the dimensions of a main sewer must correspond to the time required for the concentration of the drainage waters from the whole tributary area when small, or from so much thereof as will produce an absolute maximum discharge when the area is very large."

Time of concentration formulas estimate t_c. Unfortunately, many times this assumption is just not true. T_c consists of an inlet time plus flow time. Inlet time consists of the time required for water flowing overland to reach established surface drainage channels, such as ditches and street gutters, plus travel time through them to the point of inlet to a storm sewer. Flow time is the time of flow through the storm sewer to the point of design. Even though many equations purportedly yield t_c, some estimate only overland flow time or inlet time.

The rational method assumes that runoff is linearly related to rainfall. If rainfall is doubled; runoff is doubled. This is not really accurate, for many variables interact.

One last major misconception is that the runoff coefficient C is a constant. C is a variable and during the design of a storm sewer system, it should take on several different values for the various pipe segments, rather than retain a constant value throughout the entire design, even though the land use remains the same.

Limitations of the Formula

The most outstanding limitation is that the only product of the method is a peak discharge. The method provides only an estimate of a single point on the runoff hydrograph.

Another limitation is that the results are usually not replicable from user to user. There are considerable variations in interpretation and methodology in the use of the formula. The simplistic approach permits and requires a wide latitude of subjective judgement in its application. Each firm or agency has its favorite t_c formula, its favorite table for determining C, its own method for determining the tributary area and its own set of criteria for determining which recurrence interval is to be used in certain situations.

The average rainfall intensities used in the method bear no time sequence relation to the actual rainfall pattern during a storm. The intensity - duration - frequency (I-D-F) curves prepared by the Weather Bureau are not time sequence curves of precipitation. The maximums of the several durations as used in the method are not necessarily in their original sequential order; and the resulting tabulations of maximums ordered by size or duration may bear little resemblence to the original storm pattern. In many, if not most, cases, the intensities on the same frequency curve for various durations are not from the same storm.

The method assumes that the rainfall intensity is uniform over the entire watershed during the "duration" of the storm. This assumption is true only for small watersheds and time periods, thus limiting the use of the rational formula to small watersheds. Whether "small" means 20 acres or 200 acres is still being discussed.

The method also assumes that the runoff rate reaches a maximum at a time equal to t_c. This assumption is true only when equilibrium conditions exist, which seldom occur during a thunderstorm, except over small areas, again limiting the usefulness of the rational formula.

Discussion of the Variables

With the preceding as background, each of the four variables in the rational formula is discussed in turn: runoff coefficient, rainfall intensity, rainfall intensity averaging time and tributary drainage area. While the rainfall intensity averaging time t_c does not appear in the formula, it must be estimated in order to estimate the rainfall intensity.

Runoff Coefficient C

Various writers have used one or more variables to estimate C. A compilation of these variables yields the following list.

1. percentage of impervious surface
2. character of soil (soil type)
3. duration of rainfall
4. intensity of rainfall
5. shape of tributary drainage area
6. antecedent moisture conditions
7. slope of watershed
8. design frequency (recurrence interval)
9. nature of the surface (land use)
10. surface storage (pondage)
11. interception
12. roof drainage - is it connected directly to the storm sewer, directed to a driveway or directed onto a pervious surface?

<u>Variation of C with i_T.</u> As indicated above, some writers state that C varies with rainfall intensity. As the rainfall intensity increases, the value of C also increases. This is logical since after interception and depression storage are satisfied and the infiltration rate has been reduced to some constant minimum value, any increase in the rainfall rate must be accompanied by an increase in the rate of runoff. From the first portion of the hydrologic cycle, the following equation can be written.

$$P = F + I_a + SRO \tag{2}$$

where P is precipitation, F is infiltration, I_a is initial abstraction which includes interception and depression storage and SRO is surface runoff, all measured in inches. Also,

$$P = i_T \times \text{time} \tag{3}$$

For simplicity, assume the following conditions: the soil is saturated prior to the beginning of the storm, the minimum infiltration capacity of the soil is 1.27 cm/hr (0.5 in./hr), the initial abstraction is 1.27 cm/hr (0.5 in.) the storm duration is 1.0 hr and the watershed contains no impervious area. The surface runoff for various rainfall intensities and resulting values of C are shown in Table 1, Variation of C with i_T. These results are based on the following equations.

$$SRO = P - F - I_a \tag{4}$$

$$C = SRO/P \tag{5}$$

Note that the values of C range from 0.00 to 0.83, hardly a constant value, as shown in Figure 1, Variation of C with i_T.

TABLE 1

VARIATION OF C WITH i_T

Average Intensity in./hr.	P in.	F in.	I_a in.	SRO in.	C
0.0	0.0	0.0	0.0	0.0	0.00
0.5	0.5	0.5	0.0	0.0	0.00
1.0	1.0	0.5	0.5	0.0	0.00
1.5	1.5	0.5	0.5	0.5	0.33
2.0	2.0	0.5	0.5	1.0	0.50
2.5	2.5	0.5	0.5	1.5	0.60
3.0	3.0	0.5	0.5	2.0	0.67
3.5	3.5	0.5	0.5	2.5	0.71
4.0	4.0	0.5	0.5	3.0	0.75
4.5	4.5	0.5	0.5	3.5	0.78
5.0	5.0	0.5	0.5	4.0	0.80
5.5	5.5	0.5	0.5	4.5	0.82
6.0	6.0	0.5	0.5	5.0	0.83

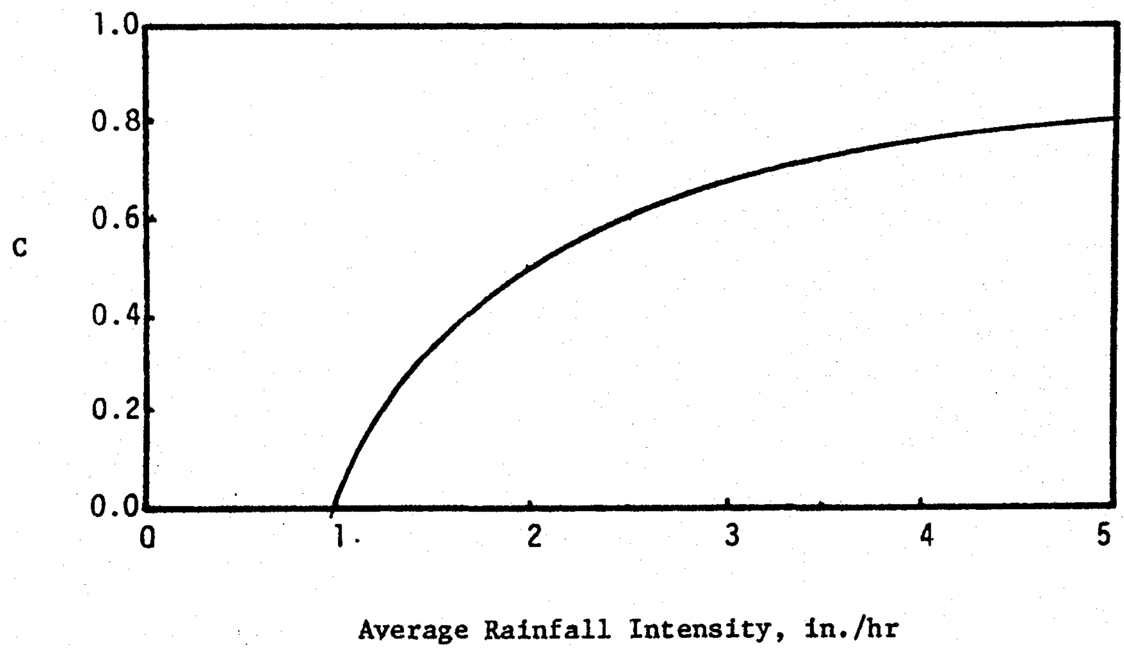

FIGURE 1 VARIATION OF C WITH i_T

Tables of C values published by various authors. Several tables have been published which enable users to estimate a value for C. The values can range from zero to 1.0, or more if rain falls on frozen ground, from no runoff to all rainfall becoming runoff. In the following tables, note that some include a range of values, but no directions are given to indicate what other parameters should be used to determine if the user should be at the low or high end of the range for his or her particular watershed. Note also the number and types of variables used in the tables. Table 2, Runoff Coefficients for Various Areas, was taken from a 1970 Concrete Pipe Design Manual.[6] Table 3, Coefficients of Runoff to be Used in the Rational Formula, was obtained from a highway engineering text by Ritter and Paquette.[7] Table 4, Runoff Coefficient C, came from a 1958 Concrete Pipe Handbook.[8]

TABLE 2

RUNOFF COEFFICIENTS FOR VARIOUS AREAS

Description of Areas	Runoff Coefficients
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single-family areas	0.30 to 0.50
Multi units, detached	0.40 to 0.60
Multi units, attached	0.60 to 0.70
Residential (suburban)	0.25 to 0.40
Apartment dwelling areas	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemetaries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

TABLE 3

COEFFICIENTS OF RUNOFF TO BE
USED IN THE RATIONAL FORMULA

Type of Drainage Areas	Runoff Coefficients
Concrete and bituminous pavements	0.70 to 0.95
Gravel or macadam surfaces	0.40 to 0.70
Impervious soil	0.40 to 0.65
Impervious soils, with turf*	0.30 to 0.55
Slightly pervious soils*	0.15 to 0.40
Pervious soils*	0.05 to 0.10
Wooded areas (depending on slope and cover)	0.05 to 0.20

*For slopes from 1 to 2 percent.

TABLE 4

RUNOFF COEFFICIENT C

Type of Surfaces	C Values
USE FOR A CULVERT DESIGN	
Impervious surfaces	0.90 - 0.95
Steep barren surfaces	0.80 - 0.90
Rolling barren surfaces	0.60 - 0.80
Flat barren surfaces	0.50 - 0.70
Rolling meadow	0.40 - 0.65
Deciduous timberland	0.35 - 0.60
Conifer timberland	0.25 - 0.50
Orchard	0.15 - 0.40
Rolling farmland	0.15 - 0.40
Flat farmland	0.10 - 0.30
USE FOR AN AIRPORT DRAINAGE DESIGN	
Watertight roof surfaces	0.75 - 0.95
Asphalt runway pavements	0.80 - 0.95
Concrete runway pavements	0.70 - 0.90
Gravel or macadam pavements	0.35 - 0.75
Impervious soils (heavy)*	0.40 - 0.64
Impervious soils w/ turf*	0.30 - 0.55
Slightly pervious soils*	0.15 - 0.40
Slightly pervious soils w/ turf*	0.10 - 0.30
Moderately pervious soils*	0.05 - 0.20
Moderately pervious soils w/ turf*	0.00 - 0.10
USE FOR A STORM SEWER IN AN URBAN AREA	
Watertight surfaces, roofs & pavements	0.70 - 0.90
Block pavements w/ open joints	0.50 - 0.70
Macadam pavements	0.25 - 0.60
Gravel surfaces	0.15 - 0.30
Parks, cultivated lands, lawns, etc., dependent on slopes and character of soil	0.05 - 0.30
Wooded areas	0.01 - 0.02

*For slopes from 1 to 2 percent

Table 5, Average Runoff Coefficient for Use in the Rational Formula, is the table of runoff coefficients which appears in Manual No. 37 of the ASCE.[9] A footnote to this table indicates that these coefficients are applicable for storms of 5 to 10 year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground is frozen. However, no instructions are given in the table as to how much higher the coefficients should be when a 25-, 50- or 100-yr storm is used for design.

Table 6, Runoff Coefficients for Use in the Rational Formula, was taken from the drainage manual of Erie and Niagara Counties in New York.[10] Table 7, Rational Method Runoff Coefficients for Composite Analysis, was obtained from the drainage manual for the City of Austin, Texas.[11] Note that additional variables have been added to these two tables: slope, soil type and frequency of occurence. With the addition of these three new variables, the runoff coefficient obtained from either of these two tables should more nearly reflect the hydrologic cycle.

Rather than a table, Ordon has presented a figure, reproduced here as Figure 2, Runoff Coefficient vs. Rainfall Intensity, to estimate C.[12] In his figure, C varies with rainfall intensity and land use. The family of curves drawn by Ordon are similar to the curve shown in Figure 1. While his curves are intuitively correct, he gives no details on how they were derived, except to say that they are based on data assembled from the literature. Recurrence interval is reflected somewhat in the rainfall intensity, but soil type and slope do nor appear in his curves. In his article, he did comment that his curves were based on low permeability soils with a high potential for runoff.

TABLE 5

AVERAGE RUNOFF COEFFICIENT FOR
USE IN THE RATIONAL FORMULA

Description of Use	Runoff Coefficients
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single family areas	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.70
Residential (suburban)	0.25 to 0.70
Apartment dwelling units	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemetaries	0.10 to 0.25
Playgrounds	0.20 to 0.40
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surface in the drainage area.

Character of Surface	Runoff Coefficients
Streets:	
Asphaltic	0.70 to 0.95
Concrete	0.80 to 0.95
Brick	0.70 to 0.85
Drives and walks	0.85 to 0.85
Roofs	0.75 to 0.95
Lawns; Sandy soil:	
Flat, 2%	0.05 to 0.10
Average, 2% to 7%	0.10 to 0.15
Steep, 7%	0.15 to 0.20
Lawns; Heavy soil:	
Flat, 2%	0.13 to 0.17
Average, 2% to 7%	0.18 to 0.22
Steep, 7%	0.25 to 0.35

TABLE 6

RUNOFF COEFFICIENTS FOR USE IN THE RATIONAL FORMULA

Land Use	Hydrologic Soil Group and Slope Range											
	A			B			C			D		
	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +	0-2%	2-6%	6% +
Industrial	0.67[1]	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85[2]	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
High Density[3] Residential	0.47	0.49	0.50	0.48	0.50	0.52	0.49	0.51	0.54	0.51	0.53	0.56
	0.58	0.60	0.61	0.59	0.61	0.64	0.60	0.62	0.66	0.62	0.64	0.69
Medium Density[4] Residential	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Low Density[5] Residential	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.28	0.35
	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Agricultural	0.08	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Open Space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Freeways and Expressways	0.57	0.59	0.60	0.58	0.60	0.61	0.59	0.61	0.63	0.60	0.62	0.64
	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78

1 Lower runoff coefficients for use with storm recurrence intervals less than 25 years

2 Higher runoff coefficients for use with storm recurrence intervals of 25 years or more

3 High Density Residential - greater than 15 dwelling units per acre

4 Medium Density Residential - 4 to 15 dwelling units per acre

5 Low Density Residential - 1 to 4 dwelling units per acre

TABLE 7

RATIONAL METHOD RUNOFF COEFFICIENTS FOR COMPOSITE ANALYSIS

Character of Surface	Runoff Coefficient Design Coefficient for Storm Frequency of		
	5 - 10 Years	25 Years	100 Years
Streets			
Asphaltic	.80	.88	.95
Concrete	.85	.93	.95
Drives and Walks, Concrete	.85	.93	.95
Roofs	.85	.93	.95
Lawns, Sandy Soil			
Flat, 2%	.07	.08	.09
Average, 2-7%	.12	.13	.15
Steep, 7%	.17	.19	.21
Lawns, Clay Soil			
Flat, 2%	.18	.20	.22
Average, 2-7%	.22	.24	.27
Steep, 7%	.30	.33	.37
Undeveloped Woods & Pasture			
Sandy Soil			
Flat, 2%	.12	.13	.15
Average, 2-7%	.20	.22	.25
Steep, 7%	.30	.33	.37
Clay Soil			
Flat, 2%	.30	.33	.37
Average, 2-7%	.40	.44	.50
Steep, 7%	.50	.55	.62

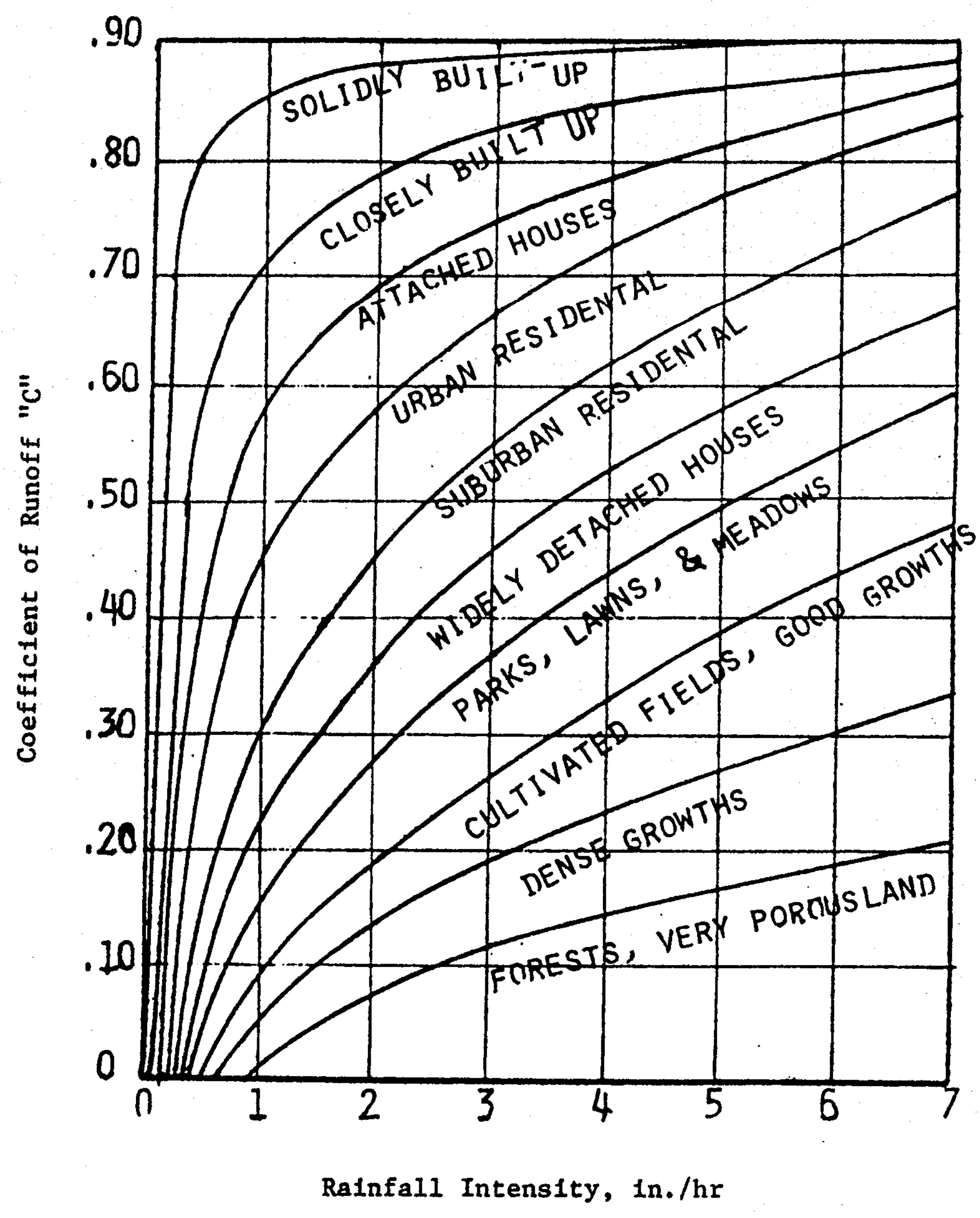

FIGURE 2 RUNOFF COEFFICIENT VS. RAINFALL INTENSITY

Rossmiller's equation for the runoff coefficient C. Each of the preceeding tables has one or more shortcomings: some do not include essential variables, some do not explain how to select a particular value from a given range of values, some do not include particular land uses. While some writers have tried to solve these deficiencies as shown in Tables 6 and 7, there is still a lack of agreement for a certain set of conditions.

A number of variables should be used to estimate C. These include land use, soil type, antecedent moisture condition, recurrence interval, imperviousness of the watershed, rainfall intensity, watershed slope and surface roughness. Each of the variables, acting in concert with some of the others, affects the portion of rainfall which will appear as runoff. As an aid to more uniform estimation, the following empirical equation is proposed for estimating the runoff coefficient C.

$$C = 7.2(10)^{-7} CN^{3} RI^{.05} ((.01CN)^{.6})^{-S^{.2}} (.001CN^{1.48})^{.15-.1I} ((Imp+1)/2)^{.7} \qquad (6)$$

where C is the runoff coefficient, a dimensionless decimal between zero and 1.00; CN is the SCS curve number, a dimensionless integer between zero and 100; RI is the recurrence interval in years; S is the average land slope of the watershed in percent, i.e., for a 4% slope, S = 4; I is the rainfall intensity in inches per hour; and Imp is the watershed imperviousness, a dimensionless decimal between zero and 1.00, i.e., for 20% imperviousness, Imp = 0.20. The SCS curve number is calculated from equation (7).

$$CN = 98Imp + X(1-Imp) \qquad (7)$$

where X is a dimensionless integer which varies with the SCS hydrologic soils group (HSG) as shown in Table 8, Variation of X with the SCS Soils Group.

TABLE 8

VARIATION OF X WITH THE SCS SOILS GROUP

HSG	X
A	39
B	61
C	74
D	80

The first two terms in equation (6) yield a basic runoff coefficient. The next three terms adjust this basic value for the effects of frequency, slope and rainfall intensity, respectively. As these variables increase, the value of C also increases. The form of the fourth and fifth terms takes into account the tendency for the effect of increased slope and rainfall intensity to be less and less as the runoff potential of the surface becomes greater and greater. The last term takes into account the surface roughness. As the imperviousness of the watershed increases, the surface becomes smoother, thus increasing the amount of runoff. Also, as imperviousness increases, more and more of this area becomes interconnected which allows more water to reach the point of design.

The formula yields values which range from 0.04 to 0.95 and is based on the assumption that the rain falls on ground which is not frozen.

Rainfall Intensity i_T

As stated before, a common definition of i_T has led to many misconceptions: the rainfall intensity in inches per hour of a storm whose duration is equal to the time of concentration of the watershed. This intensity is assumed to be uniform over the time period equal to t_c.

Current practice is to compute t_c by some method, then from an I-D-F curve prepared by the Weather Bureau for the design location, pick off a rainfall intensity for some desired frequency and a "duration" equal to t_c. What has been lost sight of in present-day use of the rational formula is that the intensities taken from an I-D-F curve are simply maximum average intensities over some time periods and bear no relation to sequential rainfall in an actual storm. Also, I-D-F curves yield <u>average</u> intensities. The actual intensities may have varied considerably during the "duration" shown on an I-D-F curve. This is due to the manner in which the I-D-F curves were derived. The following explanation of the development of I-D-F curves was taken from Hjelmfelt and Cassidy.[13]

1. Precipitation also varies with time within each particular storm, and the duration (total time during which rain falls) varies from storm to storm; therefore, analysis of precipitation at a point must involve both the amount (depth) of rain that falls and the elapsed time (duration) during which that amount fell. This is called intensity-duration analysis and proceeds in the following manner. The rainfall record from a recording rain gage is listed in Table 9, Precipitation Data in Inches. A particular duration is selected and the maximum rainfall for this time is determined. The maxima for all storms are listed in order of descending magnitude. Table 10, Frequency Analysis of Exceedence Values, is an example of an analysis of a 10-minute duration rainfall for Chicago, Illinois; column 1 is the order number m, column 2 is the rainfall in the most intense 10 minutes y, and column 3 is the return period assigned to each rainfall T_r. This is a partial-duration series; therefore, the return period is given by the formula $T_r = N/m$, N = years of record.

2. Next, the same type of analysis is carried out for a different duration, say 30 minutes. The 30-minute values may or may not include the 10-minute values of the preceding analysis. A frequency distribution is constructed from the 30-minute values, and the process is continued for other durations. The manner in which the precipitation data is reported has changed through the years, and modification of the record may be needed to put all the data on the same basis.

TABLE 9

PRECIPITATION DATA IN INCHES

Date	Year	Duration in Minutes													
		5	10	15	20	25	30	35	40	45	50	60	80	100	120
July 14	1913	0.16	0.29	0.40	0.50	0.59	0.67	0.74	0.79						
Aug. 7		0.31	0.37												
Aug. 7-8		0.30	0.44	0.56											
Aug 18		0.28	0.49	0.63	0.67										
Apr. 27	1914	0.27													
May 27		0.18	0.33	0.41	0.49										
Jun 4		0.21	0.35	0.40											
Jul 16		0.33	0.66	0.79	0.97	1.21	1.48	1.61							
Aug. 9		0.35	0.62	0.83	0.91										
Aug. 13		0.19	0.36	0.50	0.60	0.68									
Sept. 1		0.14	0.27	0.38	0.40										
May 15	1915	0.17	0.25	0.32	0.40	0.48									
Jun 12		0.18	0.31	0.46	0.56	0.76	0.82	0.89	0.92	0.98					
Data from July 7, 1915 through July 12, 1947 were listed and analyzed but are not shown here.															
Jul 13	1947	0.31	0.44	0.57	0.62	0.64	0.66	0.68	0.70	0.72	0.73	0.75	0.81	0.84	0.84
Aug. 29		0.36	0.60	0.72	0.77	0.81	0.83	0.85	0.87	0.89	0.91				
Sept 11		0.25	0.50	0.72	0.77	0.77	0.78								
Sept 21		0.13	0.23	0.33	0.42	0.50	0.57	0.62	0.67	0.72	0.77	0.85	0.98	1.13	1.24
Oct. 26		0.19	0.29	0.38	0.43	0.45	0.46	0.48	0.49	0.49	0.50	0.53	0.55	0.56	

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Mike Nissen	Nissen Engineering	325 I Avenue Nevada, IA
Jerry P. Oliver	CEC, Inc.	1233 8th St. West Des Moines, IA 50265
Bob Payer	Otto & Culver	1711 N. Lake Avenue Storm Lake, IA 50588
Gary Petersen	Shive-Hattery & Assoc.	800 First St. NW Cedar Rapids, IA 52405
*Herbert G. Poertner	Engr. & Research Consultant	Bolingbrook, IL
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Wayne A. Reed	W. G. Jaques Co.	2183 NE 86th St. Des Moines, IA 50322
Ralph V. Reiner	Wilson Concrete Co.	Box 108 Washington, IA 52353
*Stan Ring	Iowa State University	Ames
*Dr. Robert B. Robinson	Iowa State University	Ames
William H. Robinson	Robinson Engineering Co.	428 East 4th St. Waterloo, IA 50703
*Ron Rossmiller	Iowa State University	Ames
Michael J. Ryan	Brice, Petrides & Assoc.	191 W. 5th St. Waterloo, IA 50701
Hollis E. Ryken	Ryken Engineering Co.	711 Park Avenue Ackley, IA 50601

John R. Sommers	Associated Engineering, Inc.	3421 Mt. Vernon Road Cedar Rapids, IA 52403
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*Bruce Thorson	City of W Des Moines	318 5th Street West Des Moines, IA 50265
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*Paul Wiegand	City of Ames	Depot Annnex Ames, IA 50010
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