CALIBRATING THE PROPOSED ORANGE COUNTY EMA UNIT HYDROGRAPH PROCEDURE (A STUDY OF BASIN FACTOR AND COASTAL S-GRAPH)

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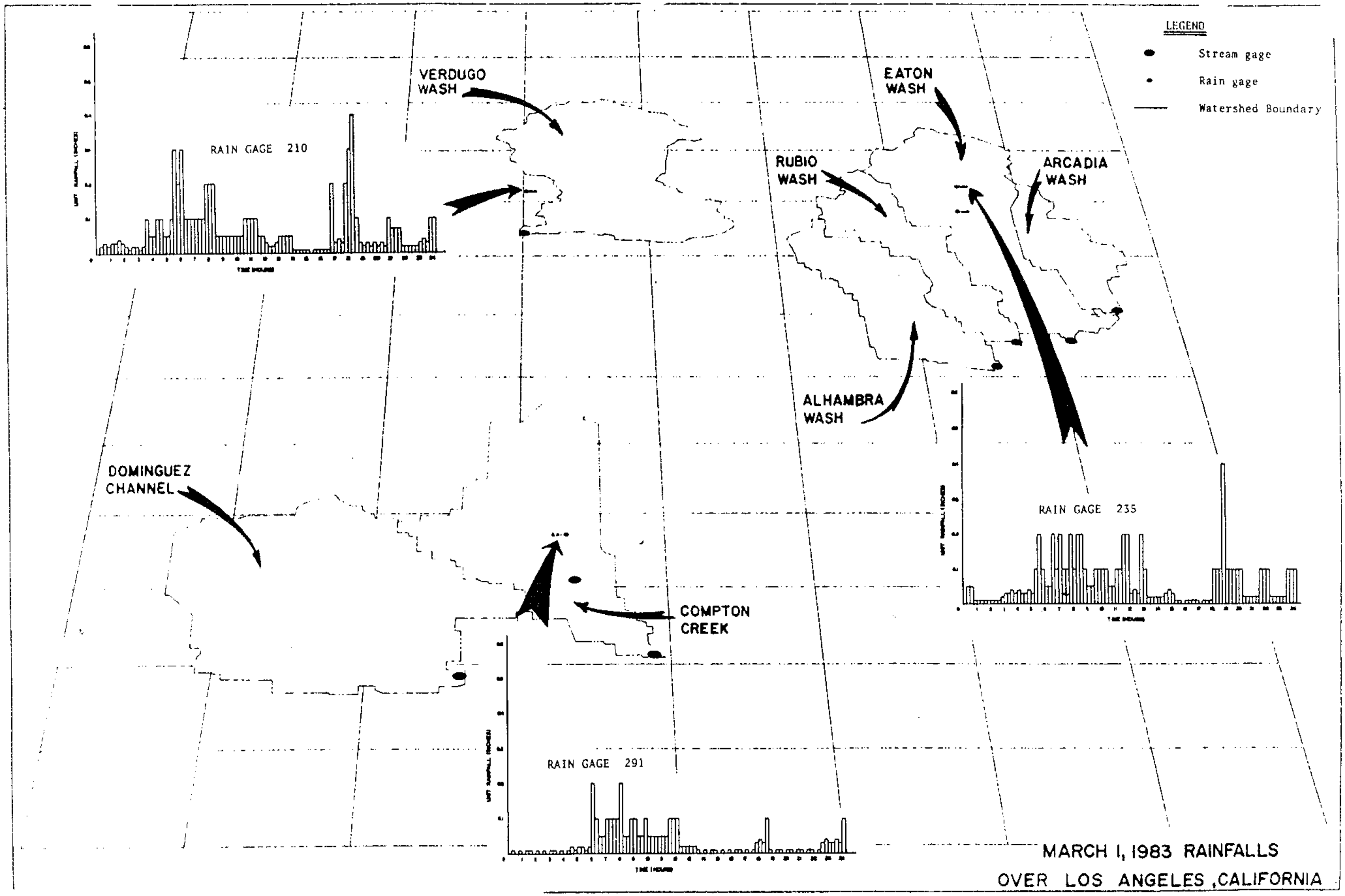
Prepared for

ORANGE COUNTY ENVIRONMENTAL MANAGEMENT AGENCY

PUBLIC WORKS/FLOOD PROGRAM DIVISION

December 1985



MARCH 1, 1983 RAINFALLS
OVER LOS ANGELES, CALIFORNIA

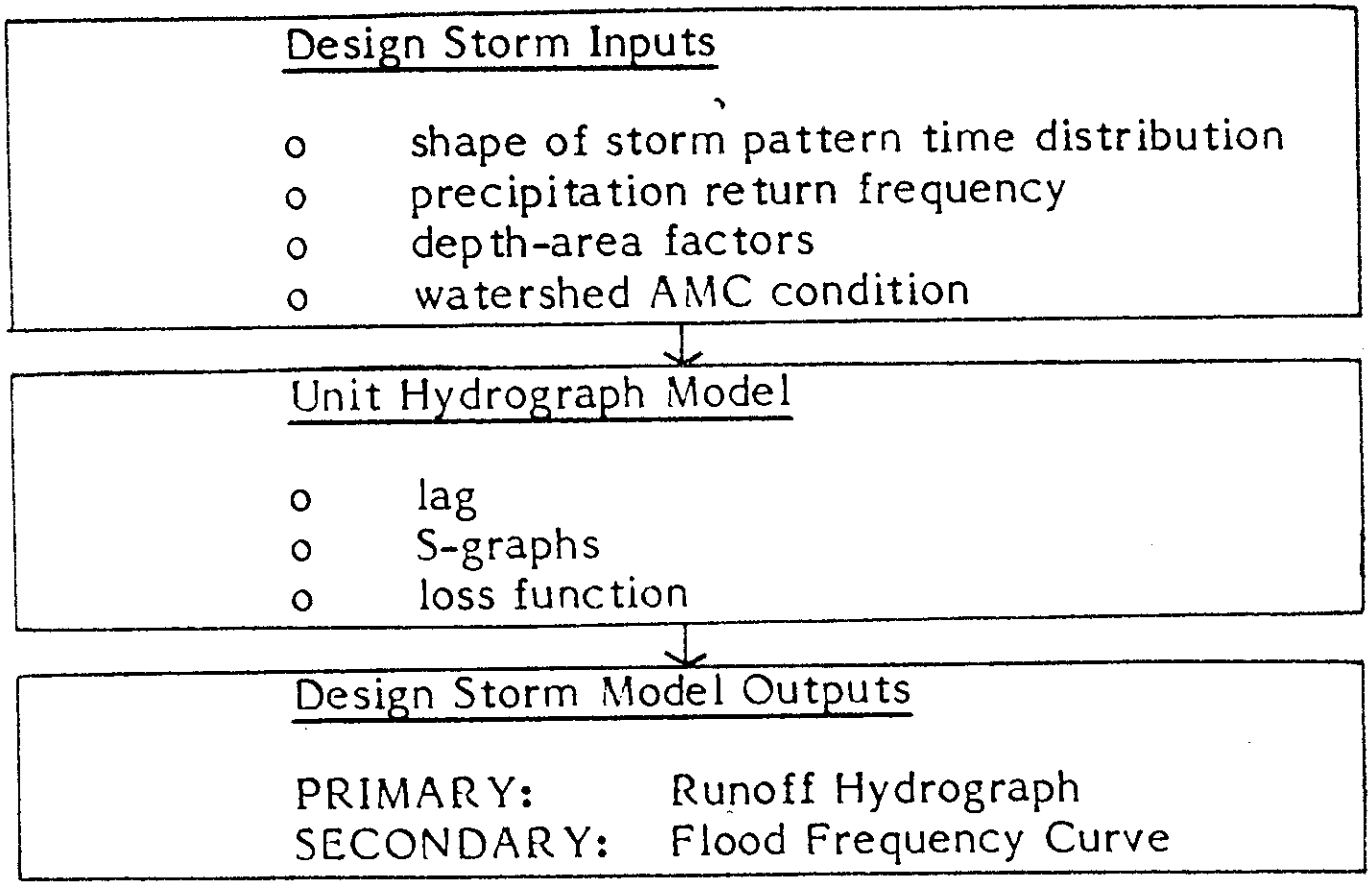


FIG. 1 The Design Storm Modeling Approach

From Fig. 1, it is seen that two of the input parameters used in the design storm approach are the storm pattern shape and the storm pattern's rainfall magnitudes (or return frequency).

TABLE I.1
STATISTICAL ESTIMATION OF Q_p
(CATEGORY I)

APPLICABILITY/ADVANTAGES

Procedures are based on accepted statistical methods.

Procedures are available for most of the country.

Reliability of the prediction equations is known for gaged areas used in derivation.

Estimates are reliable for hydrologically similar basins as those used in the derivation.

Once developed, the procedure is quick and easy to use.

Permits direct calculation of specific peak flood flow frequency estimates that are individually and statistically derived.

Procedures may be used in conjunction with other procedures such as to provide calibration relationships for simulation models.

Provides a quick check for reasonableness for situations requiring use of other procedures.

Easy to use and may be used where other methods are more appropriate.

LIMITATIONS/DISADVANTAGES

- Requires knowledge of both statistics and hydrology in derivation and utilization.
- Procedures require numerous regression analyses and are time consuming to develop.
- Only provides estimates of specific peak flood flow frequency relationships.
- Cannot evaluate effects resulting from modifications in the system (physical works and alternative land use patterns).
- Procedures are often misused by application for areas with different stream patterns and other hydrologic characteristics from the gaged locations used in the derivation.
- Cannot adequately evaluate hydrologically unique areas in the region.
- Derivation requires several hydrologically similar gaged basins in the region.
- Does not assume a distribution; hence reliability confidence limits cannot be calculated.

TABLE I.2
STATISTICAL ESTIMATION OF Q_p
(CATEGORY II)

APPLICABILITY/ADVANTAGES	LIMITATIONS/DISADVANTAGES
Procedures are based on accepted statistical methods.	o Requires knowledge of both statistics and hydrology in derivation and utilization.
The entire frequency function is developed from the three moments; means, standard derivation, and skew.	o Procedure requires regression analysis for the two or three moments of the frequency.
Reliability of the prediction equations is known for gaged areas used in derivation.	o May be time consuming to develop.
Estimates are reliable for hydrologically similar basins as those used in derivation.	o Does not calculate specific flood flow frequency events.
Once developed, the procedure is quick and easy to use.	o Only provides estimates of peak flood flow frequency relationships.
Procedures may be used in conjunction with other procedures such as to provide calibration results for simulation models.	o Cannot evaluate effects resulting from modifications in the system (physical works and alternative land use patterns).
Provides a quick check for reasonableness for situations requiring use of other procedures.	o Cannot adequately evaluate many complex river systems.
	o Cannot evaluate hydrologically unique areas in the region.
	o Ease of use may result in improper application.
	o Derivation requires several hydrologically similar gaged basins in the region.

TABLE I.3
INDEX FLOOD ESTIMATE
(CATEGORY III)

APPLICABILITY/ADVANTAGES	LIMITATIONS/DISADVANTAGES
Procedure is easier to develop than other statistical methods, and has only one regression analysis.	o Procedure yields same variance (slope of frequency curve) for all applications.
Procedures are commonly used and based on accepted statistical methods.	o Probably least accurate of the statistical procedures.
Reliability of prediction equation for index flood is known for derivation.	o Requires knowledge of both statistics and hydrology in derivation and utilization.
Estimates are reliable for hydrologically similar basins as those used in derivation.	o May be time consuming to develop.
Once developed, the procedure is quick and easy to use.	o Only provides estimates of peak flood flow frequency relationships.
Procedures may be used in conjunction with other procedures such as to provide calibration results for simulation models.	o Cannot evaluate effects resulting from modifications in the system (physical works and alternative land use patterns).
Provides a quick check for reasonableness for situations requiring use of other procedures.	o Cannot adequately evaluate many complex river systems.
	o Cannot evaluate hydrologic unique areas in the region.
	o Ease of use may result in improper application.
	o Derivation requires several hydrologically similar gaged basins in the region.

TABLE I.4
TRANSFER METHODS
(CATEGORY IV)

APPLICABILITY/ADVANTAGES	LIMITATIONS/DISADVANTAGES
(WRC Transfer of Qp)	**(WRC Transfer of Qp)**
Procedure is easy and quick to use.	o Procedure ease of use may result in improper application.
Provides reliable estimates immediately upstream and downstream of gage location if hydrologic characteristics are consistent.	o Can only be utilized immediately upstream and downstream of gaged area where hydrologic characteristics are consistent.
Procedure is commonly used and generally acceptable.	
(Direct Transfer)	**(Direct Transfer)**
Provides quick estimate where time constraints are binding and other proedures are not applicable.	o Estimates are not accurate enough for most analysis requirements.
Can readily be used as a check for reasonableness of results from other procedures.	o Cannot be used for modified basin conditions.
Provides valuable insight as to the regional slope characteristics of the flood flow frequency relationships.	o Can only be used as check in areas where hydrologic characteristics are nearly similar and with drainage areas within the same order of magnitude.

TABLE I.5
EMPIRICAL EQUATIONS
(CATEGORY V)

APPLICABILITY/ADVANTAGES	LIMITATIONS/DISADVANTAGES
Provides quick means of estimating peak discharge frequency for small areas.	o Generally are not applicable for areas greater than one square mile.
Concepts can be understood by non-hydrologists.	o Estimate only the peak discharge frequency relationships.
Suitable for many types of municipal engineering analyses (storm sewers, culverts, small organization impacts, etc.)	o Cannot be used to design storage facilities.
Familiarity of procedures and use has led to politically acceptable solutions f small areas.	o Cannot adequately evaluate complex systems where timing and combining of flood hydrographs are important.
Can be used as a check for reasonableness of more applicable procedures in small areas.	

TABLE I.6
SINGLE EVENT SIMULATION
(CATEGORY VI)

APPLICABILITY/ADVANTAGES	LIMITATIONS/DISADVANTAGES
Generates other hydrologic information rather than peak discharges (volumes, time of peak, rate of rise, etc.)	o Balanced flood concept is difficult to understand.
Generates balanced floods as opposed to historically generated events which may be biased.	o Modeling requires more time, data, resources (costs) than statistical procedures.
Enables evaluation of complex systems and modifications to the watersheds.	o Hydrologists must understand the concepts utilized by the model.
Provides good documentation for quick future use.	o Requires calibration to assure rainfall frequency that approximates runoff frequency.
Uses fewer parameters than most continuous simulation models.	o Unit hydrograph assumes linear relationship with runoff.
Approximates the hydrologic runoff process as opposed to statistical methods.	o Requires data processing capabilities.
Procedures are more economical than continuous simulation procedures.	o Procedures greatly simplify the hydrologic process.
Calibration procedures are easier than continuous simulation models.	o Parameters are difficult to obtain for existing and modified conditions.
Models may be calibrated to either simple or complex systems.	o Difficult to obtain antecedent moisture conditions.
	o Depth-area of rainfall varies with drainage area size.

TABLE I.7
MULTIPLE DISCRETE EVENTS
(CATEGORY VII)

APPLICABILITY/ADVANTAGES

Concepts are easier to understand than those associated with hypothetical frequency events.

Antecedent moisture conditions are determined.

Depth-area precipitation problems are eliminated.

Evaluates fewer events than continuous simulation models.

Enables evaluations of complex systems and physical modifications in the watershed.

Uses fewer parameters than continuous simulation models.

Approximates hydrologic process as opposed to statistical methods.

Provides good documentation for future use.

LIMITATIONS/DISADVANTAGES

- Requires numerous storm analyses and subsequent event analyses.
- Important events may be overlooked.
- Results may be biased by historic records.
- Procedures use simplified hydrologic process.
- Requires data processing capabilities.
- Parameters are difficult to obtain.
- Unit hydrograph assumes linear relationship with runoff.
- Requires calibration which is more time consuming than single event due to the large number of events that are processed.
- Procedure is significantly more expensive than single event modeling.
- Procedures generally not feasible for small study areas, short time constraints, etc.

TABLE I.8
CONTINUOUS RECORDS
(CATEGORY VIII)

APPLICABILITY/ADVANTAGES

Concepts are easily understood.

Concepts are more physically based than other procedures.

Antecedent moisture conditions are automatically accounted for.

Can be used in unique basins where other procedures such as statistical procedures are not applicable.

Process analyses in single computer runs as opposed to handling numerous discrete events.

Can automatically determine annual peak floods at various locations even if their frequencies are different.

Can model the effects of complex systems and physical works.

LIMITATIONS/DISADVANTAGES

o The calibration process is extensive and generally must be performed by qualified experienced hydrologists.

o Procedures are expensive and time consuming to use, impractical for moderate or small resources allocated projects.

o The results may be biased by the use of historic rainfall data.

o The procedures require large analytical processing capabilities.

o The models typically require a large amount of data to properly define the parameters.

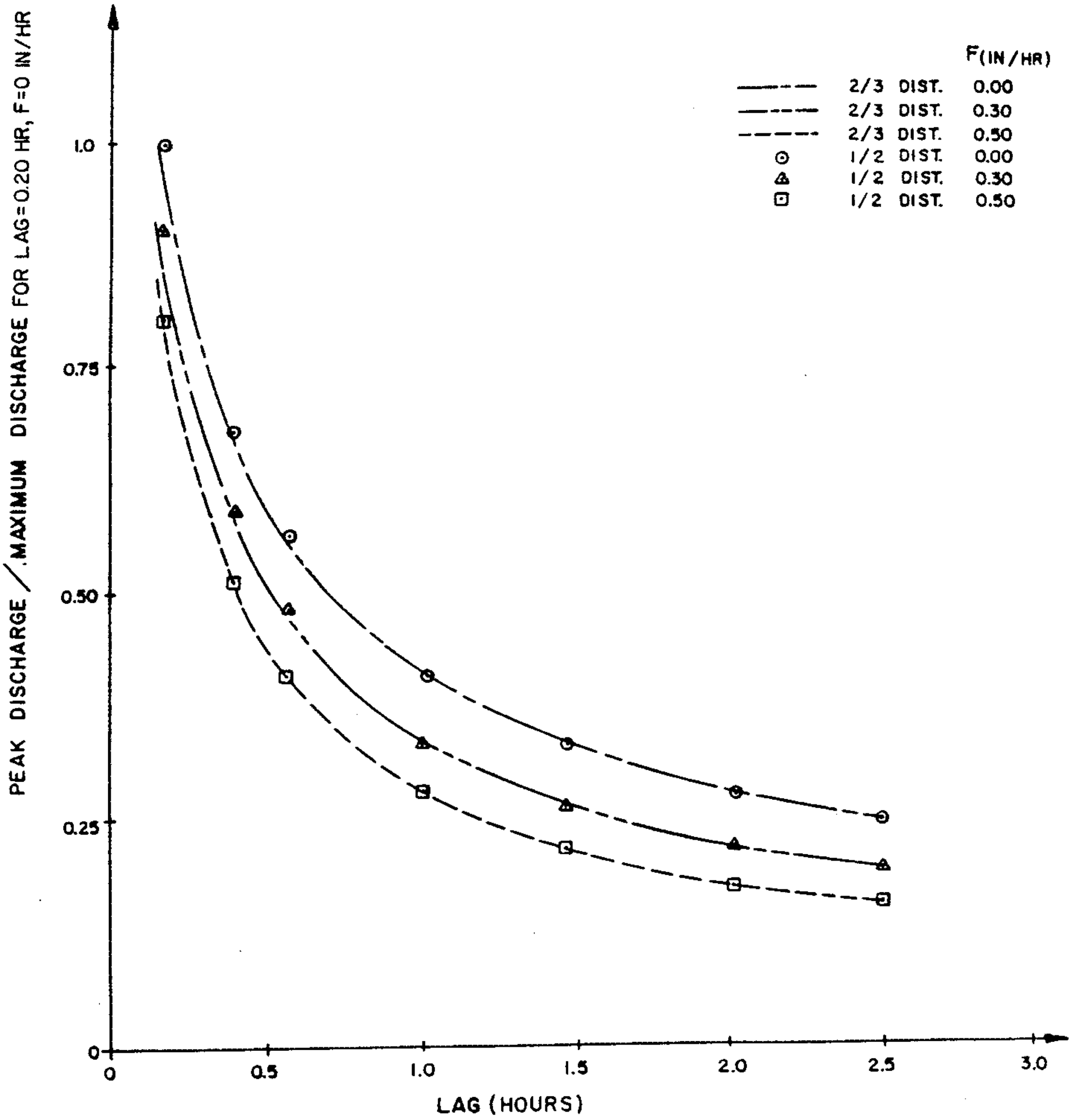


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FIGURE 2

COMPARISON OF RUNOFF RESULTS (2/3,1/3 DISTRIBUTION OF STORM PATTERN NESTED UNIT RAINFALL COMPARED TO (1/2,1/2 DISTRIBUTION).

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The following watershed parameters are used:

L	=	7500 feet
L_{ca}	=	3750 feet
Elevation drop	=	75 feet
Lag	=	varied from 0.2 to 2.5 hours
A	=	640 acres (1 sq. mile) and 25,600 acres (40 sq. miles)
F	=	0.30 inch/hour
F*	=	0.30 (in percentage)

Valley S-hydrograph was used

The various Australian storm pattern rainfalls and the design storm pattern are compared in Table 1 for the case of the 1 square mile watershed where depth-area adjustment is negligible.

TABLE 1: AUSTRALIAN VS. AGENCY'S STORM PATTERN RAINFALLS (INCHES) (1 SQUARE MILE WATERSHED)

Duration of Rainfall	30-Min. Storm		1-Hour Storm		3-Hour Storm		6-Hour Storm	
	Agency	Aus.	Agency	Aus.	Agency	Aus.	Agency	Aus.
5 Min.	0.51	0.40	0.51	0.37	0.51	0.36	0.51	0.32
30 Min.	1.13	1.13	1.13	1.10	1.13	0.95	1.13	0.88
1 Hour	---	---	1.53	1.53	1.53	1.40	1.53	1.30
3 Hour	---	---	---	---	2.48	2.48	2.48	2.40
6 Hour	---	---	---	---	---	---	3.37	3.37

The Australian storm patterns are fixed patterns of unit rainfalls where each unit rainfall is a given percentage of the total storm rainfall. For example, Table 1 shows that for a 30-minute storm the peak 5-minute unit rainfall is, on the average, about 35 percent of the total storm (30-minute) rainfall. In comparison, the 3-hour storm pattern assigns 14.5 percent of the total 3-hour storm rainfall to the peak 5-minute unit rainfall.

From Table 1 it is seen that the peak 5-minute unit rainfall value drops as the storm duration increases. Typically, the critical duration of storm rainfall for a watershed is strongly dependent upon the watershed size. That is, a 1 square mile watershed would most likely achieve the highest peak flowrate using the most severe 30-minute or 1-hour Australian storm patterns. However, for a 40 square mile watershed, the critical Australian storm pattern most likely would be the 1-hour or 3-hour pattern. But for these longer storm patterns, the peak 5-minute unit rainfalls are less than the 30-

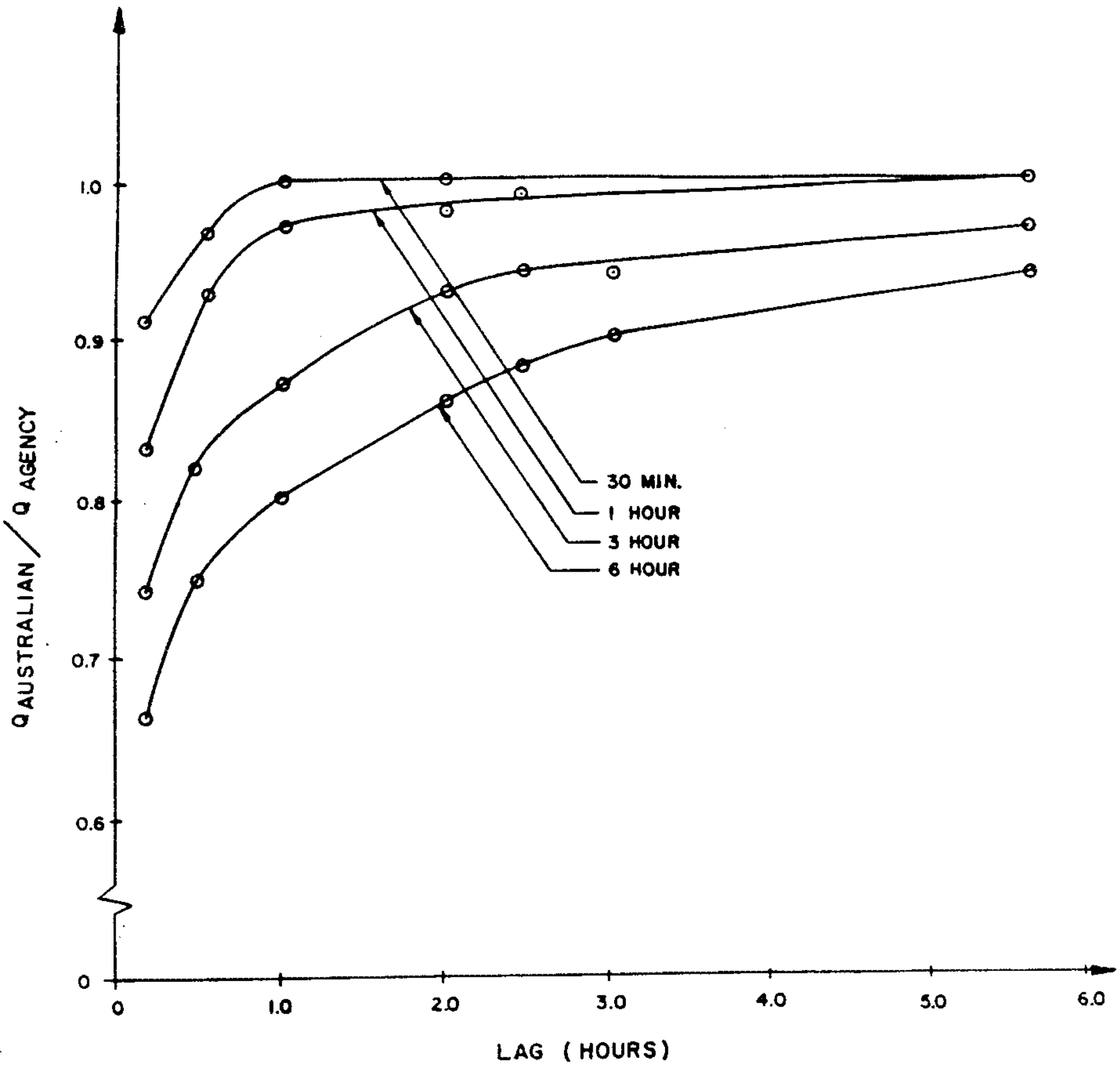

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FIGURE 3a

COMPARISON OF RUNOFF RESULTS FROM 1 SQ. MILE DRAINAGE AREA - AGENCY STORM PATTERN VERSUS AUSTRALIAN STORM PATTERN

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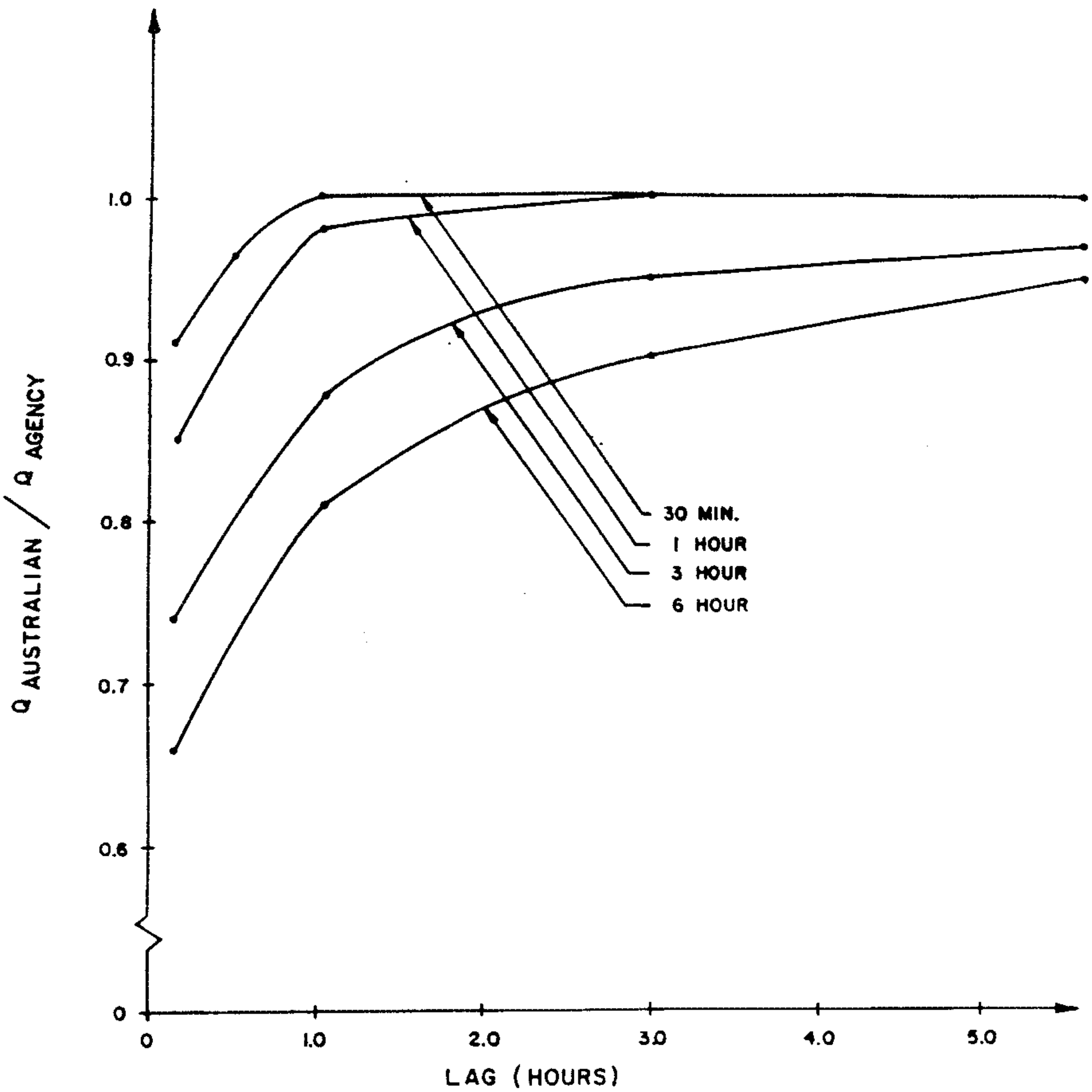

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FIGURE 3b

COMPARISON OF RUNOFF RESULTS FROM 40 SQ. MILE DRAINAGE AREA- AGENCY STORM PATTERN VERSUS AUSTRALIAN STORM PATTERN

SCALE
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FIELD BOOK
DATE 1-30-84
JOB NO.

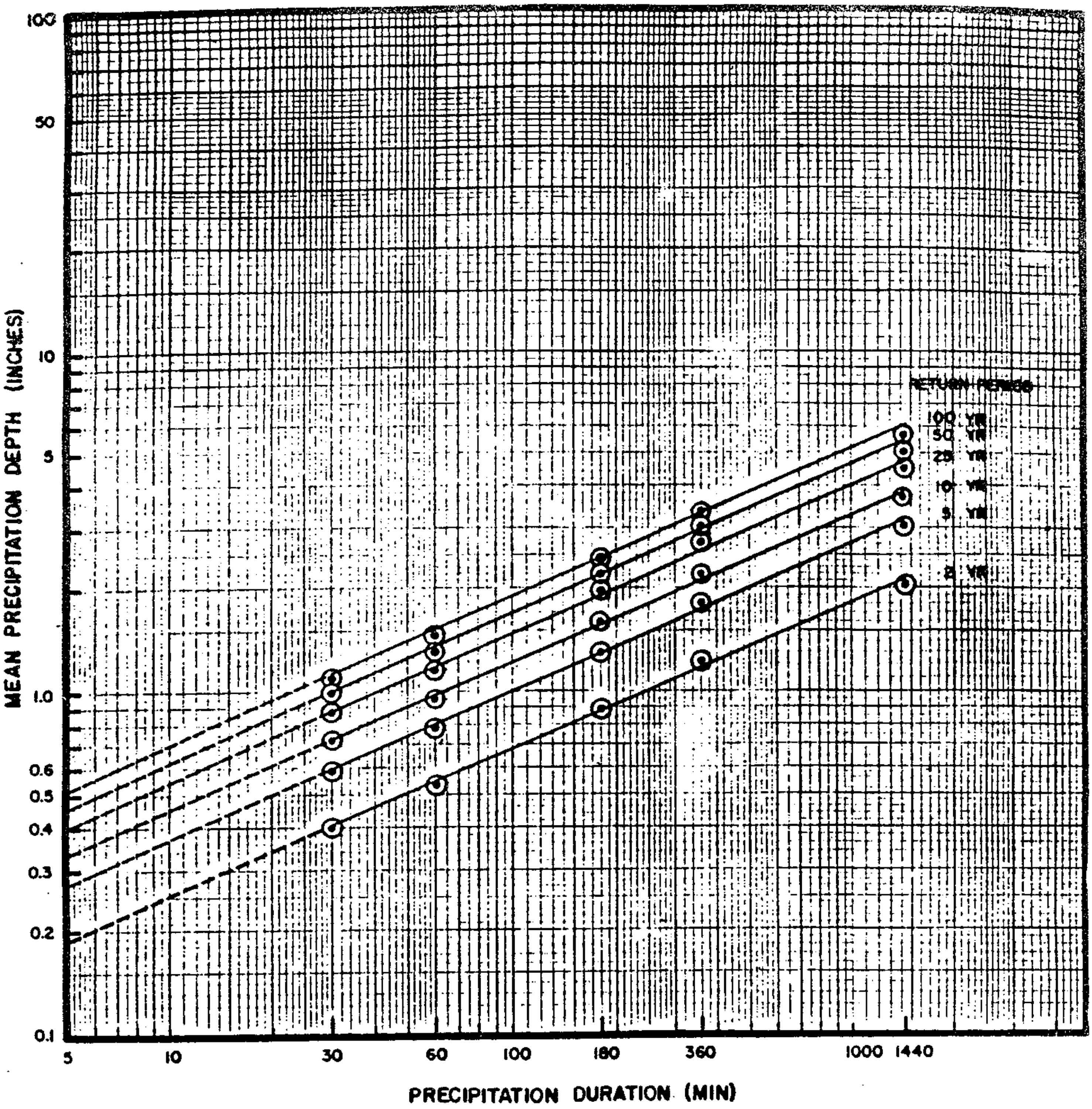


Fig. 4. Mean Depth-Duration Relationships for Orange County Non-Mountainous Areas (see Appendix E).

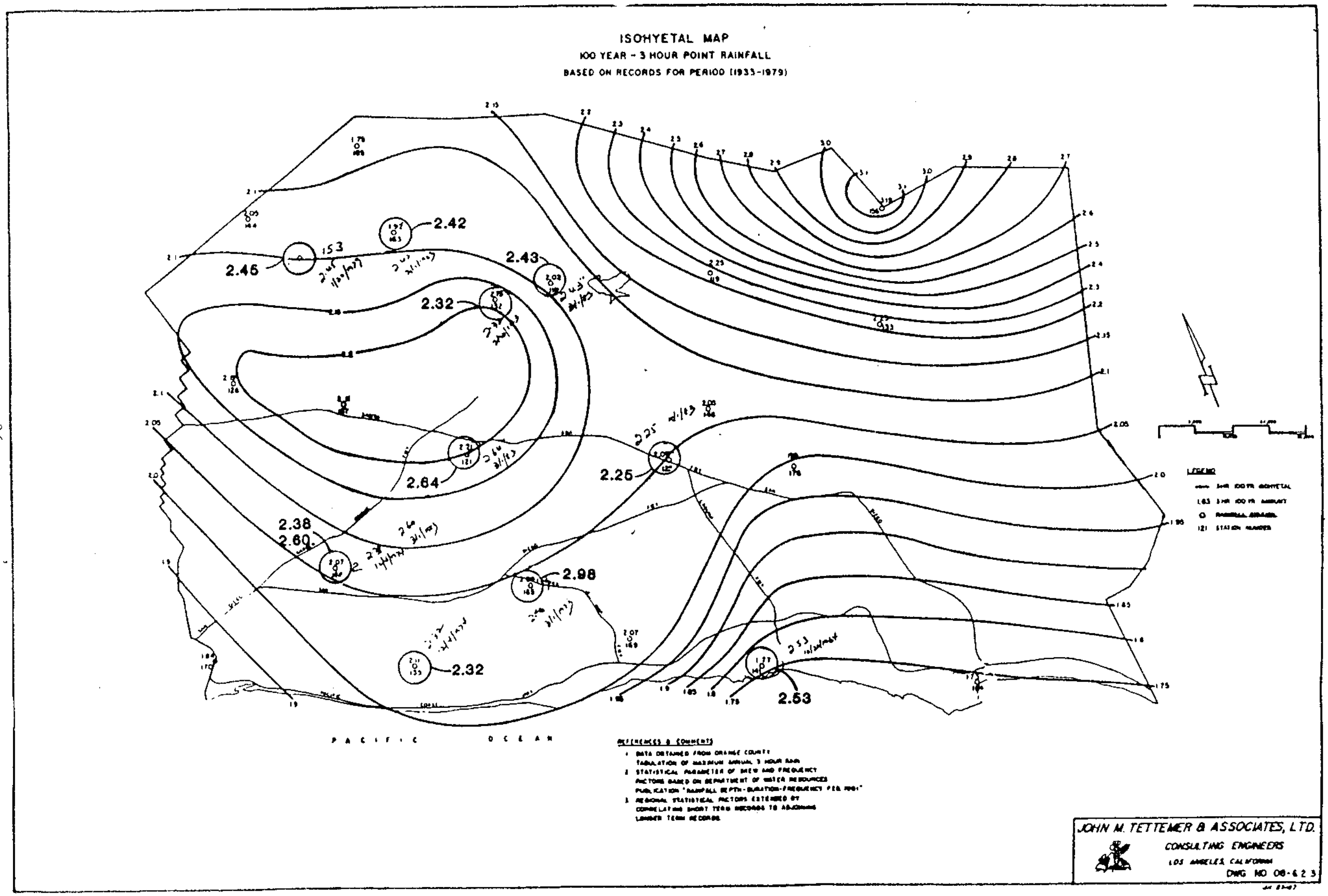
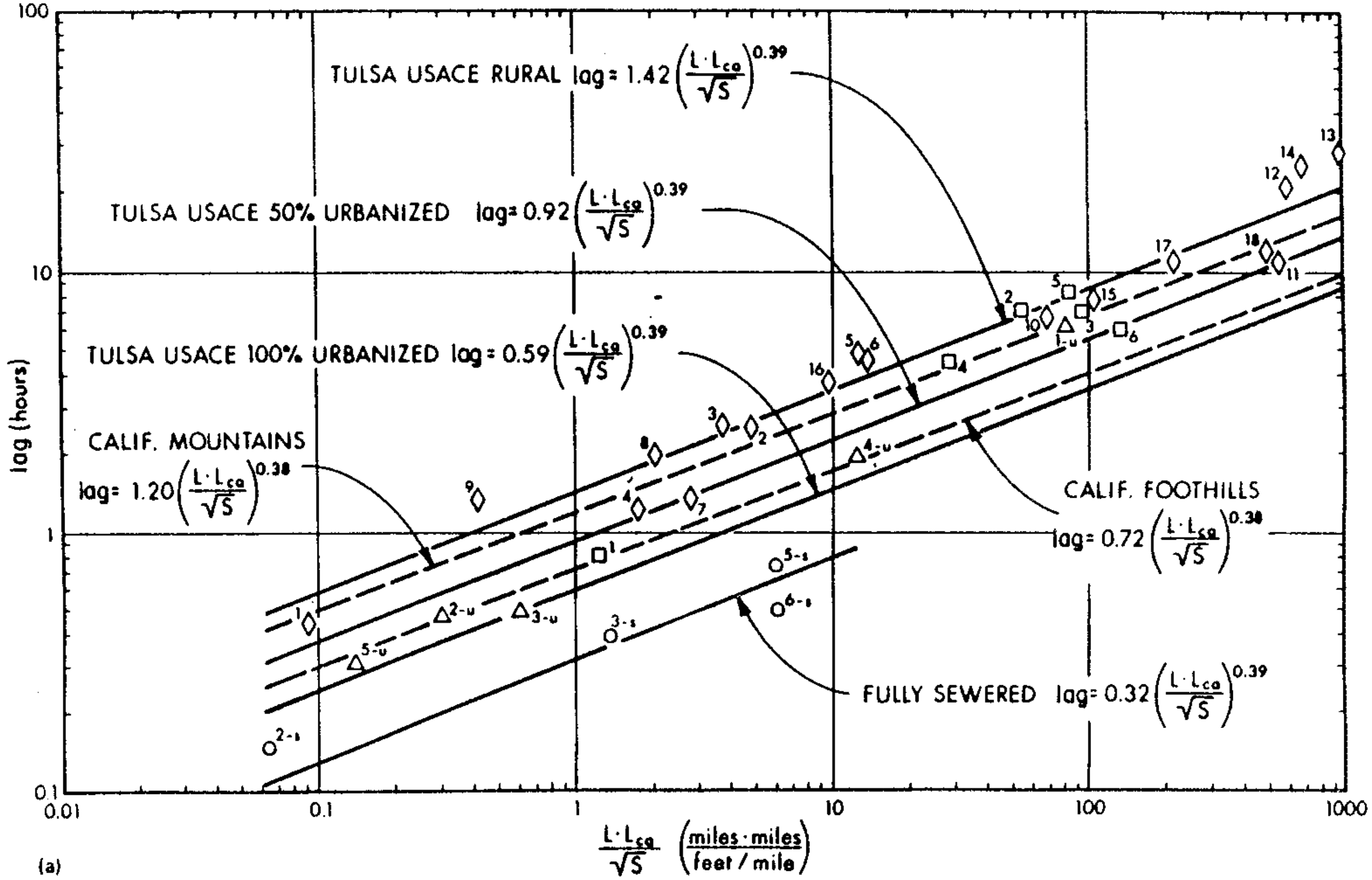


Fig 5. Record Exceedances of 3-Hour Point Rainfalls of a Considered Isohyetal "100-Year" 3-Hour Map.



(a)

Figure 6 (a) lag vs. LL_{ca} $/\sqrt{S}$ for Synthetic Unit Hydrograph Procedure.

OKLAHOMA, RURAL (by Tulsa, USACE)

		A sq. mile	S ft/mile	L miles	L_{ca} miles
◊1	Little Dry Ck., Alex, OK	88	82.1	1.4	0.6
◊2	Worley Ck., Tuttle, OK	11.2	17.4	6.3	3.2
◊3	West Beaver, Orland, OK	13.9	23.8	6.4	2.9
◊4	Canyon View, Geary, OK	11.8	19.4	4.4	1.9
◊5	Dry Creek, Kendrick, OK	69	10.9	10.7	3.9
◊6	Elm Creek, Foraker, OK	18.2	17.5	9.4	5.8
◊7	Rock Creek, Snider, OK	9.1	35.8	5.4	3.1
◊8	Adams Ck., Beggs, OK	5.9	32.1	4.4	2.7
◊9	Corral Ck., Yale, OK	2.89	53.3	2.4	1.3
◊10	Big Hill Ck., Cherryvale, KS	37	11.1	21.8	10.6
◊11	Bird Creek, Avant, OK	364	5.8	52.4	25.8
◊12	L. Caney R., Copan(Upper), OK	424	5.1	50.4	26.7
◊13	L. Caney R., Copan(Lower), OK	502	4.1	60.5	33.0
◊14	Hominy Ck., Shiatook, OK	340	5.5	55.2	29.0
◊15	Polecat Ck., Heyburn Dam	133	9.0	25.8	12.2
◊16	Council Ck., Stillwater, OK	31	12.1	8.6	4.0
◊17	Pryor Ck., Pryor, OK	229	5.3	36	14
◊18	Sand Ck., Okesa, OK	139	13.5	60	31

OKLAHOMA, URBAN (by Tulsa, USACE)

			A sq. mile	S ft/mile	L miles	L_{ca} miles
▲1	Deep Fork R., Arcadia, OK.	(30% Urbanized)	108	10.3	25.8	10.0
▲2	Bluff Ck., Okla. City, OK	(60% ")	1.64	62.7	2.18	1.14
▲3-u	Deep Fork Ck., Okla. City, OK.	(100% ")	2.98	44.9	2.88	1.44
▲4-u	Deep Fork Ck., Eastern Ave. Okla City, OK	(100% ")	20.3	19.2	11.4	4.8
▲5-u	Crutch Ck., Trib. Okla. City, OK	(60% ")	0.47	49.1	1.45	0.7

TEXAS AND ILLINOIS, URBAN

		I percent	A sq. mile	S ft/mile	L miles	L_{ca} miles	COMMENTS
□1	Boneyard, Illinois	37.4	4.45	9.5	2.8	1.3	storm sewers, no channel improvements
□2	Brays Bayour, Houston, TX	40.0	88.4	4.07	23.3	10.4	storm sewers and channel improvements
□3	Greens Bayou, Houston, TX	25.0	67.5	6.65	21.6	10.0	agricultural & urban, no storm sewers
□4	Halls Bayou, Houston, TX	30.0	26.2	7.08	13.5	5.7	some storm sewers & channel improvements
□5	Simms Bayou, Houston, TX	30.0	63.0	3.38	18.0	9.7	some storm sewers & channel improvements
□6	White Oak Bayou, Houston, TX	35.0	92.0	5.02	21.1	12.8	storm sewers & channel improvements

KENTUCKY, FULLY SEWERED

		I percent	A sq. mile	S* ft/mile	L miles	L_{ca} miles
○2-s	17th St., Louisville, KY	83	.22	20.06	.93	.31
⊙3-s	NW Trunk, Louisville, KY	50	1.90	6.34	3.03	1.13
⊙5-s	Southern Outfall, Louisville KY	48	6.44	7.23	6.44	2.52
⊙6-s	S.W. Outfall, Louisville, KY	33	7.51	7.76	6.48	2.68

*Slope used is the Weighted Sewer Slope

(b)

Figure **6** (b) Explanation of data points.

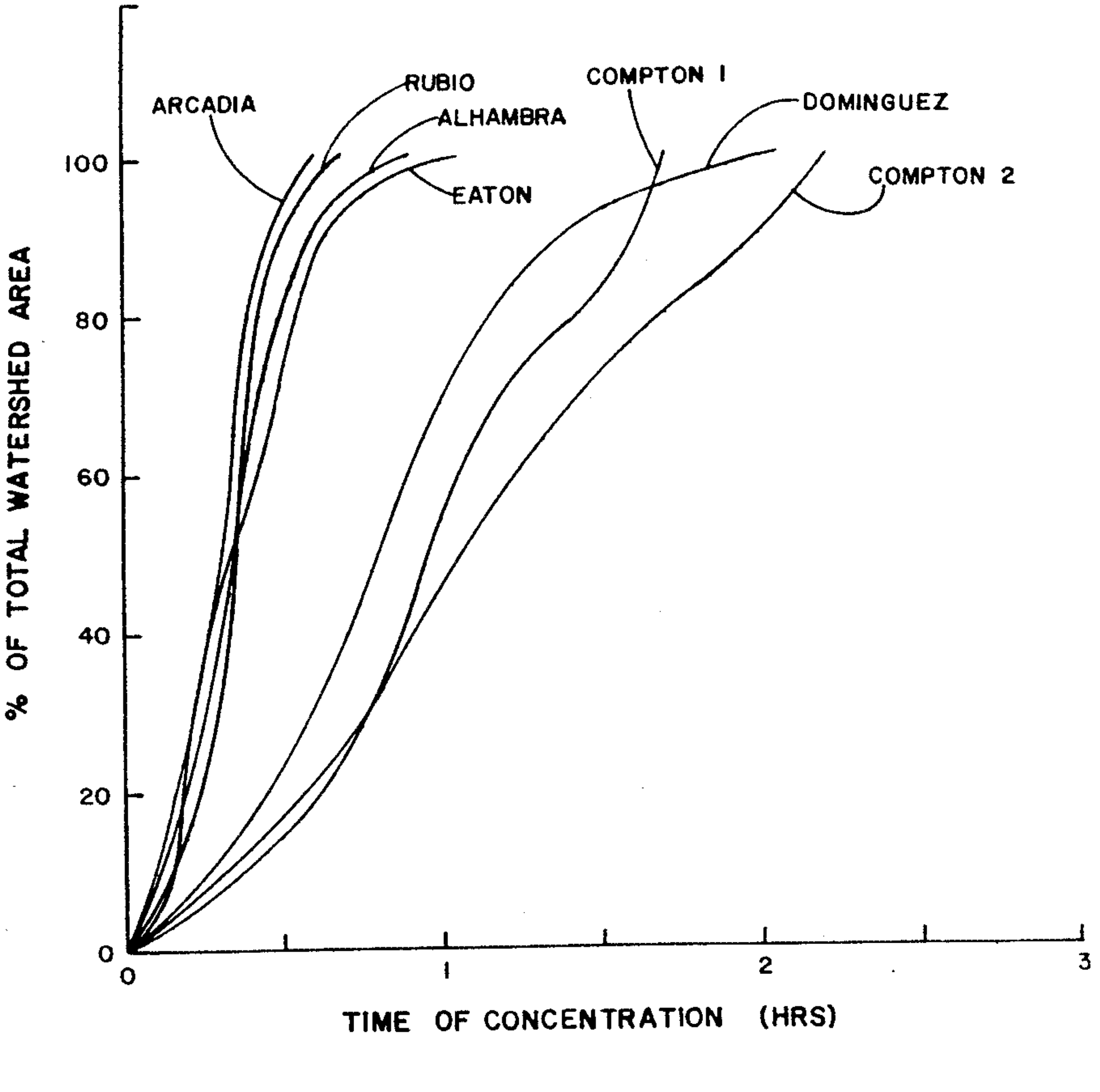


Figure 7. Time-Area Diagrams

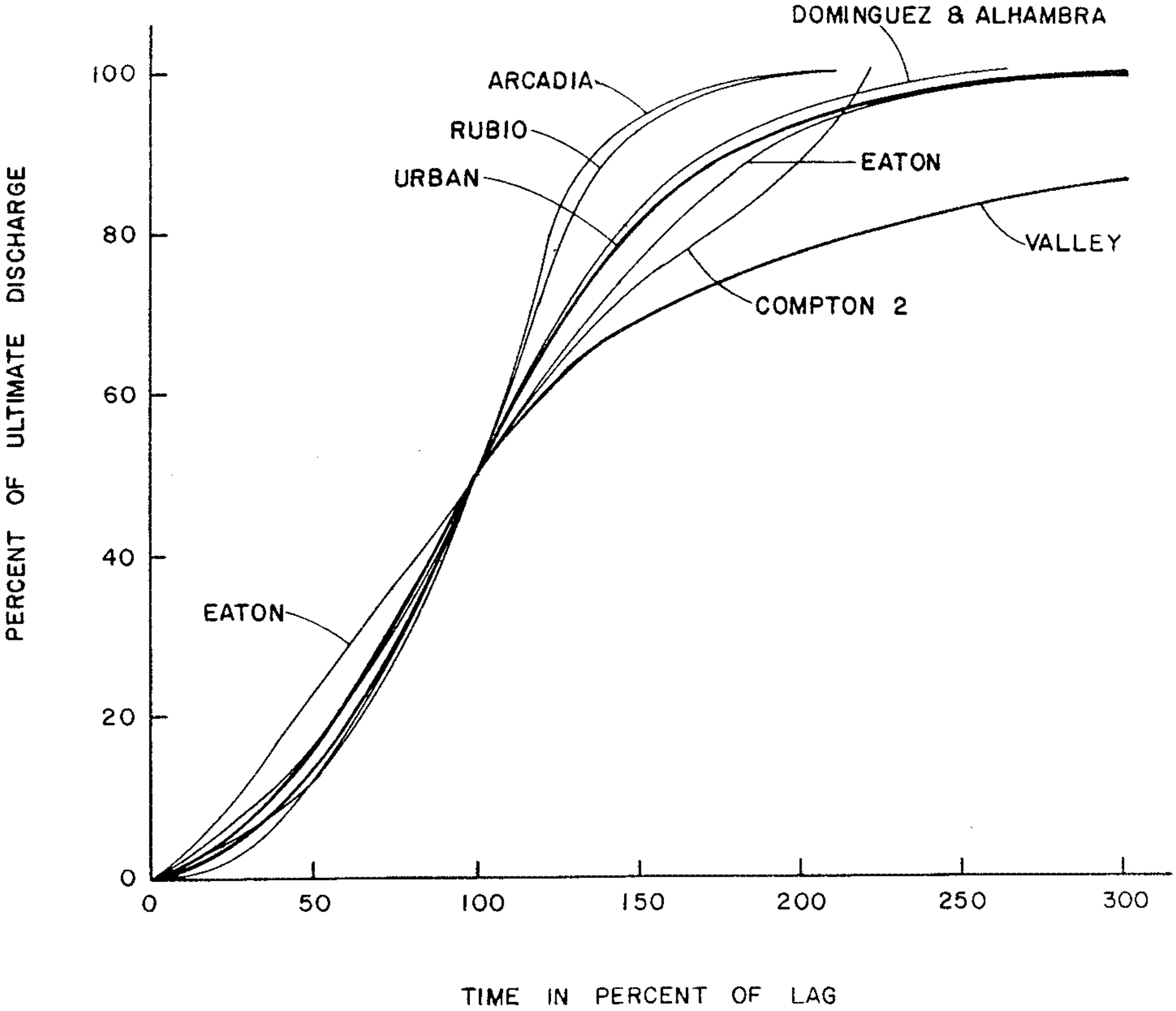


Figure 8. Comparison of S-Graph to Time-Area Diagrams
(Normalized with Respect to Lag)

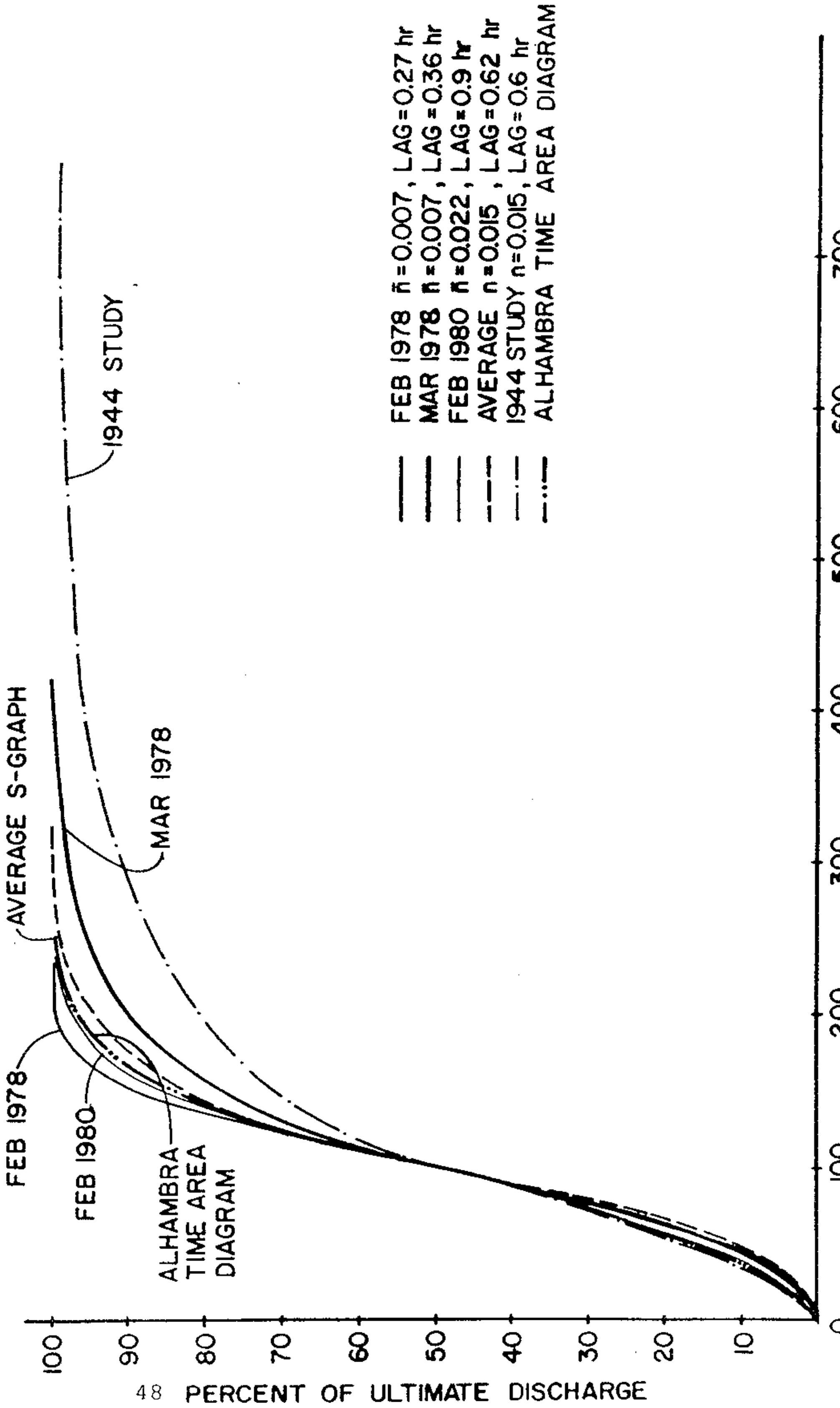


Figure 9. Comparison of Alhambra Time-Area Diagram and S-Graphs.

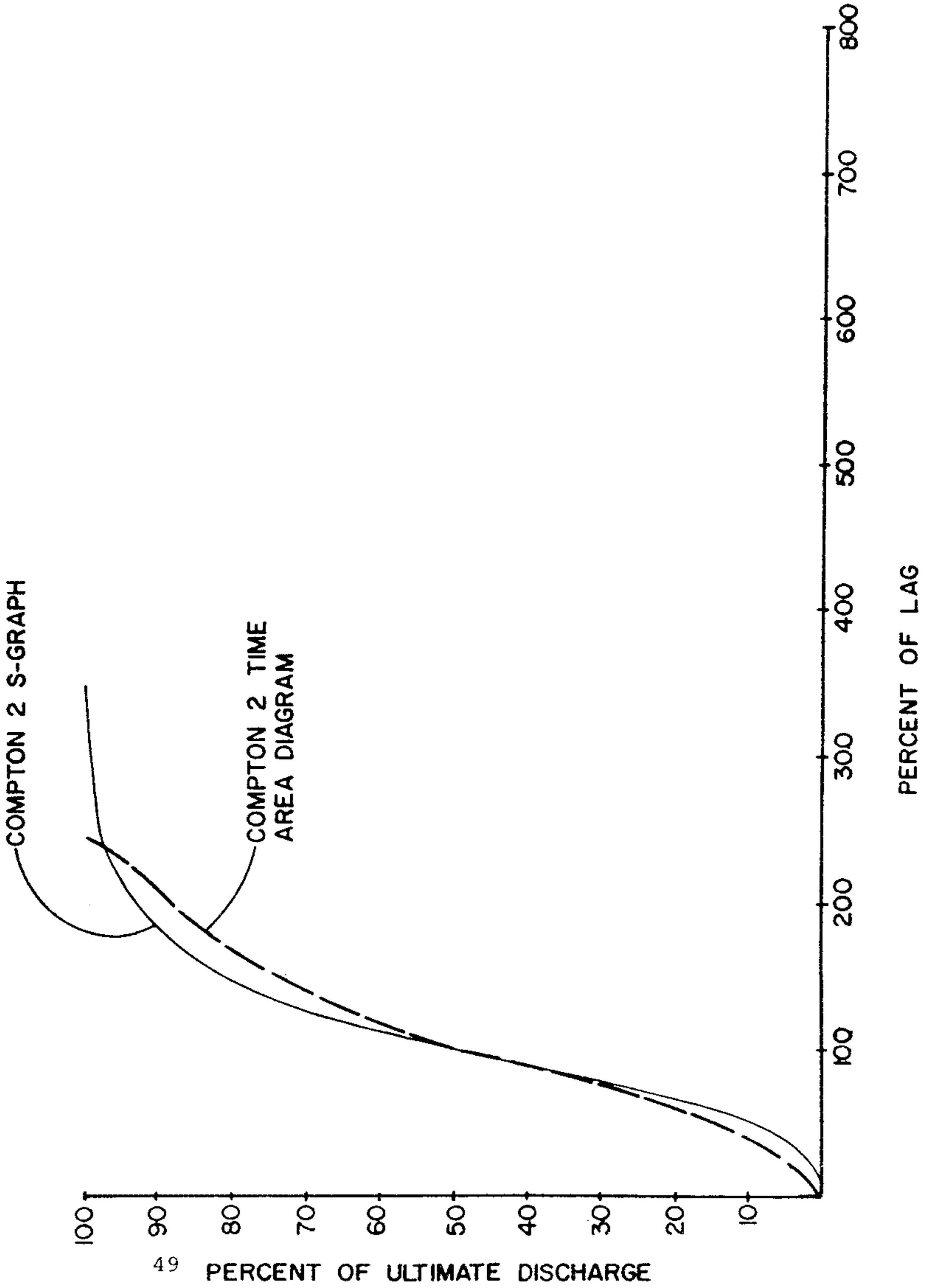


Figure 10. Comparison of Compton Creek (Compton 2) Time-Area Diagram and S-Graph

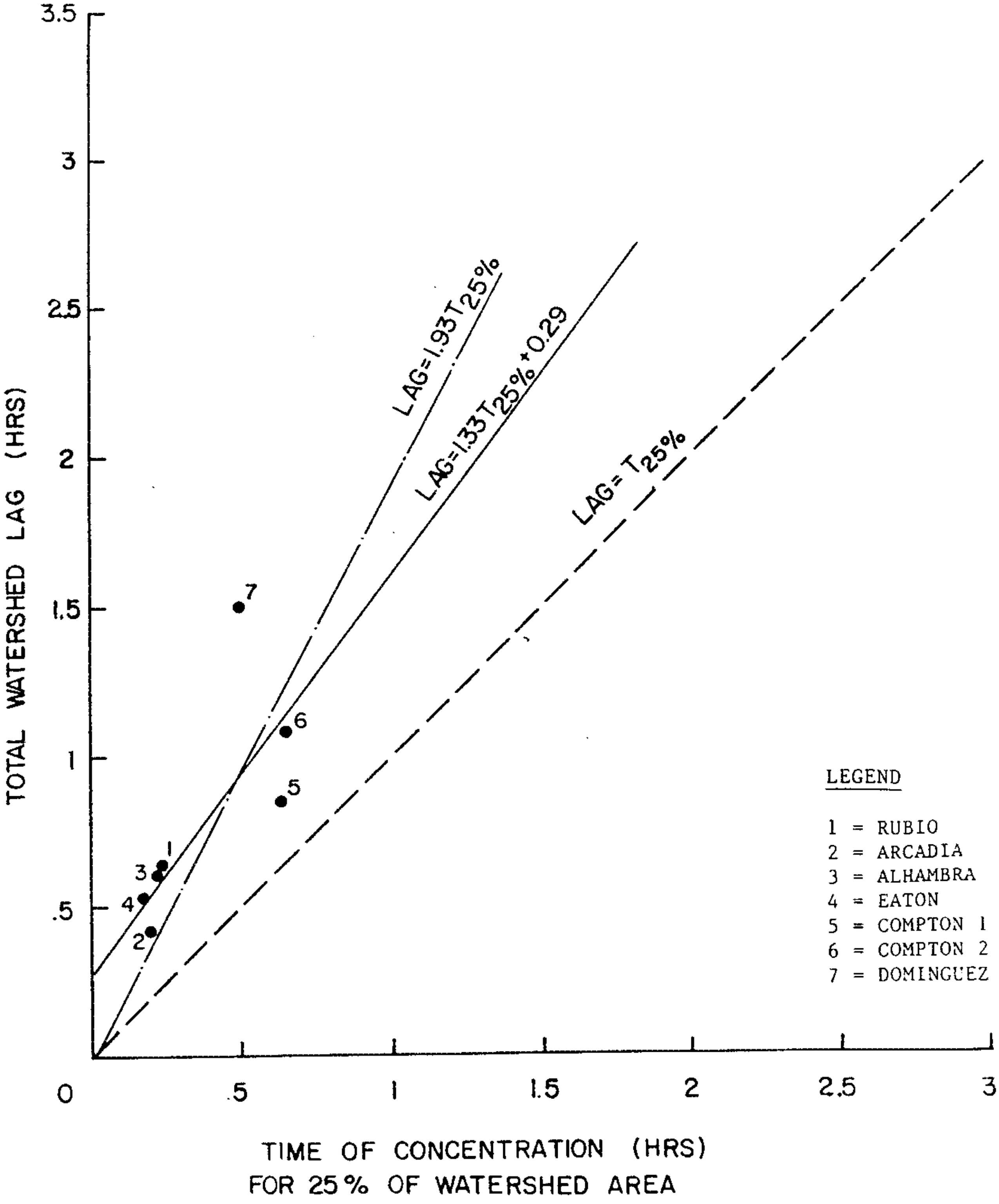


Figure 11. Lag as a Function of $T_{25\%}$

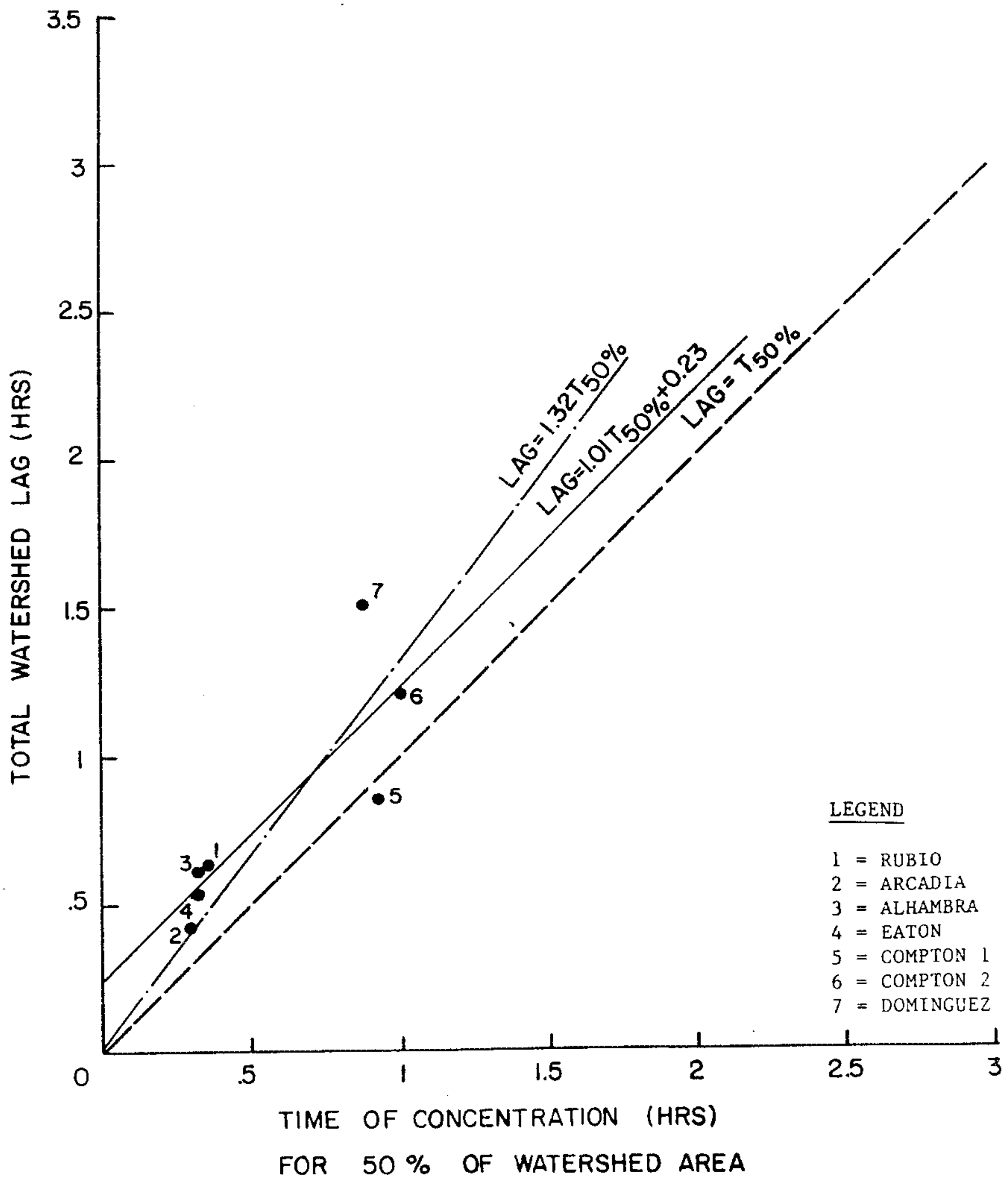


Figure 12. Lag as a Function of $T_{50\%}$

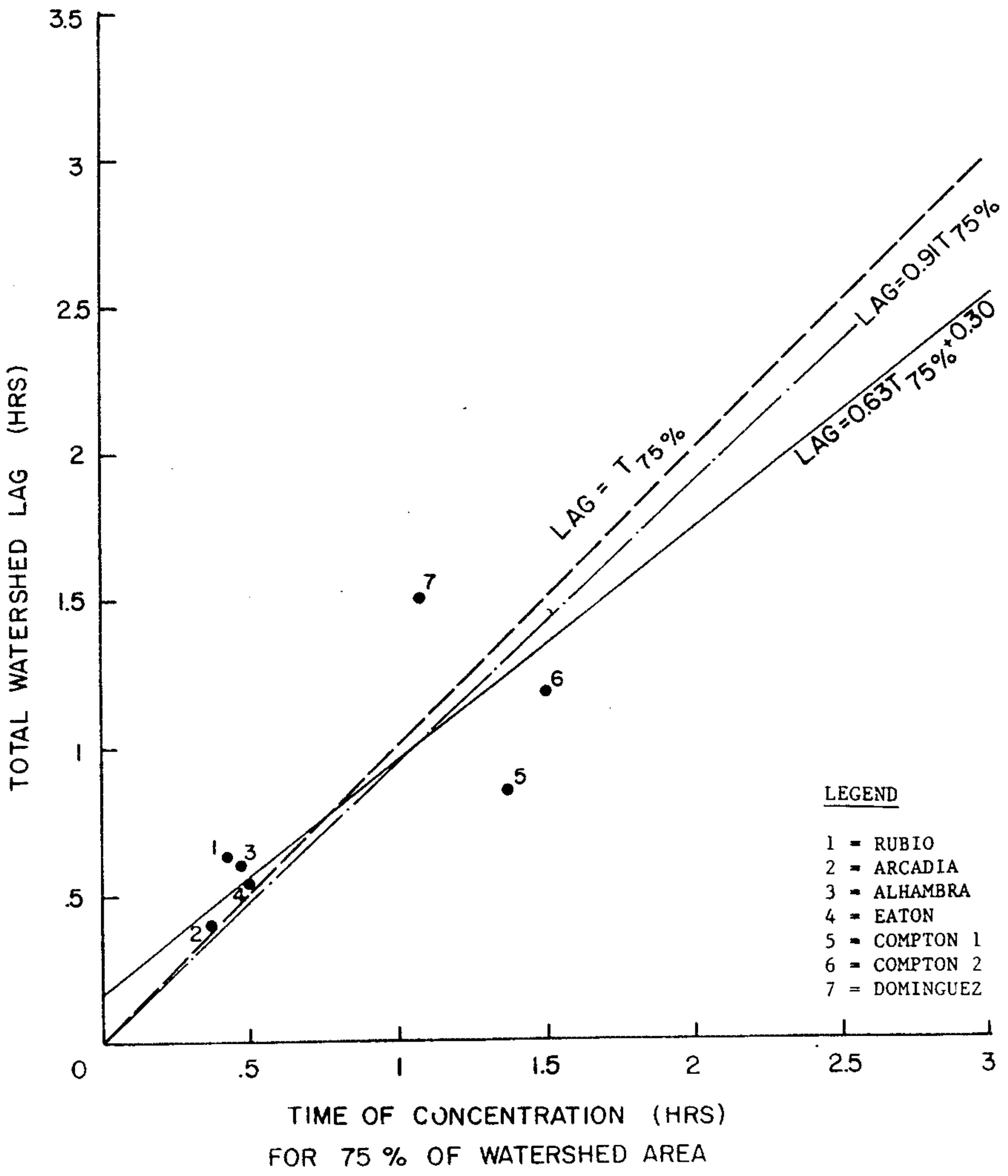


Figure 13. Lag as a Function of $T_{75\%}$

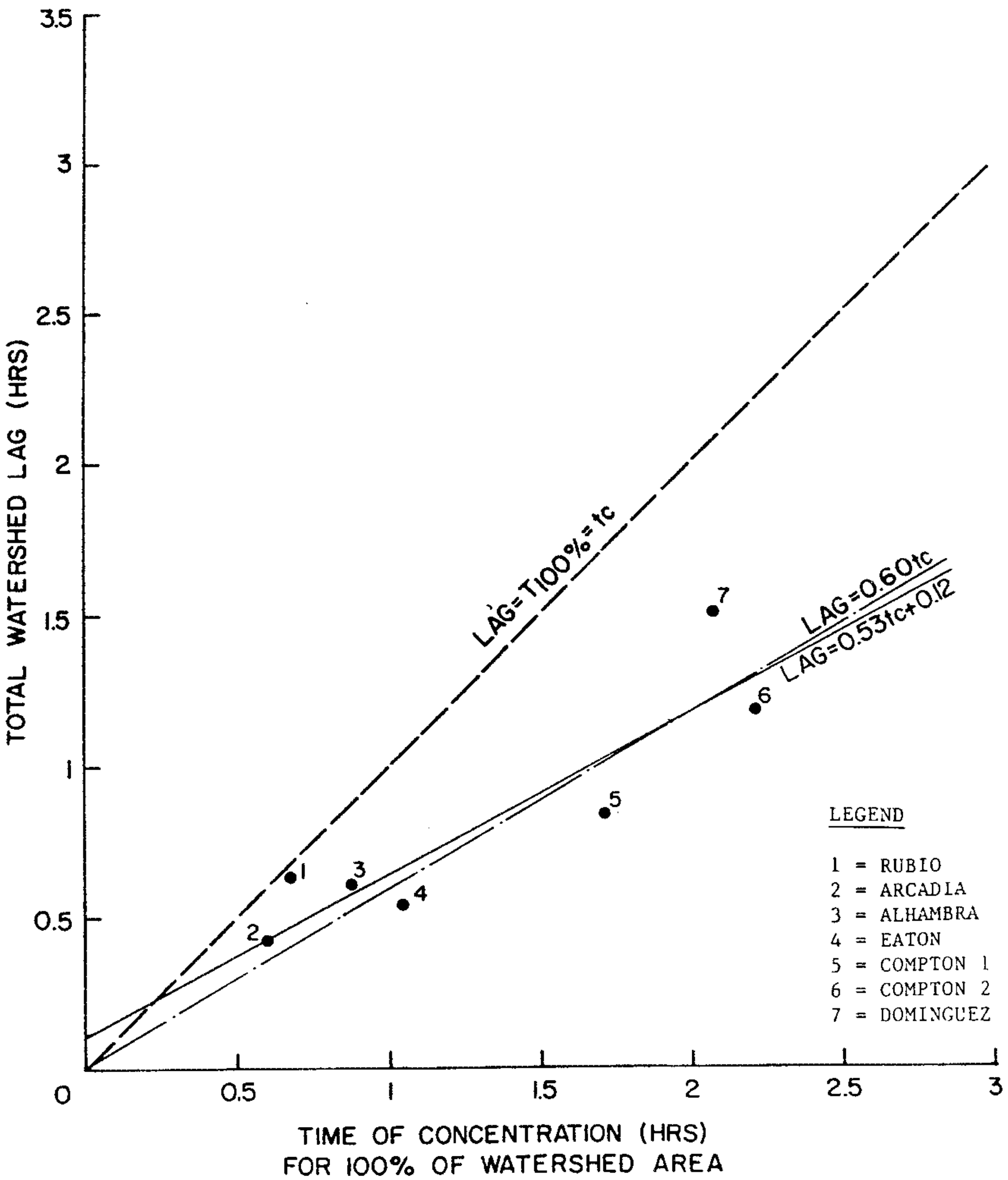


Figure 14. Lag as a Function of t_c

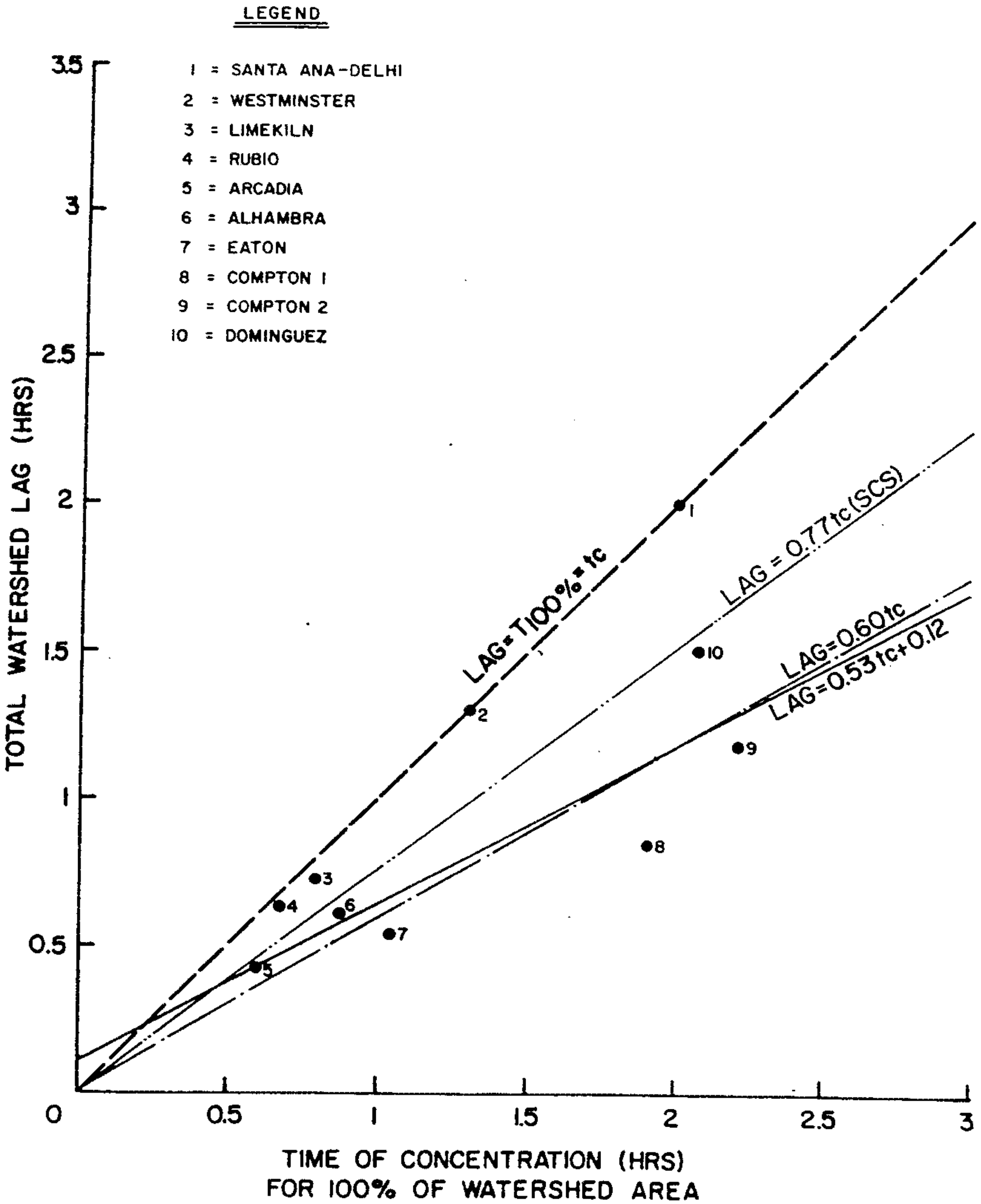


Figure 15. SCS Relationship Between Lag and t_c

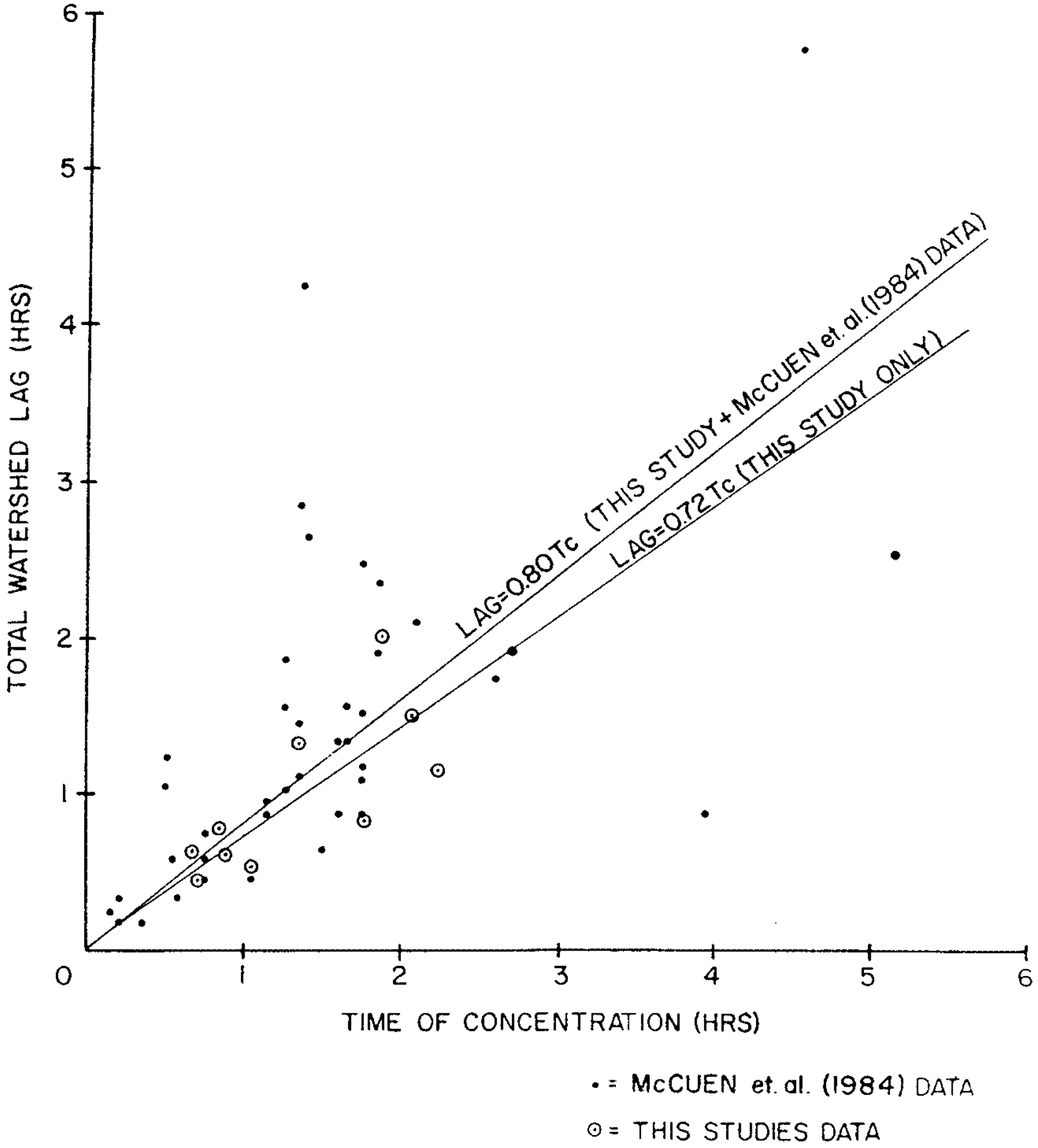


Figure 16. S.C.S and this Study Correlations Between Lag and t_c

Hicks (1944) suggests using maximum depression storage depths of 0.02 in. for sand, 0.15 in. for loam, and 0.10 in. for clay soils. The Denver Regional Council of Governments, (Wright-McLaughlin Engineers, 1969) has compiled (see Table 3) suggested depression and detention depths which are similar to those by Hicks. While the values of surface depression and detention are reported only for use in the Colorado unit hydrograph procedure, they are in general agreement with accepted ASCE (1970) values of 1/16 in. for impervious areas and 1/4 in. for pervious areas.

TABLE 3: TYPICAL DEPRESSION AND DETENTION RAINFALL STORAGE VALUES FOR VARIOUS LAND COVERS

Land Cover	Depression and Detention (inches)	Recommended (inches)
Impervious		
Large paved areas	0.05 - 0.15	0.1
Roofs, flat	0.1 - 0.3	0.1
Roofs, sloped	0.05 - 0.1	0.05
Pervious		
Lawn grass	0.2 - 0.5	0.3
Wooded areas and open fields	0.2 - 0.6	0.4

IV.4 INFILTRATION

The topic of infiltration has been the subject of numerous publications, offering a variety of equations expressing infiltration as functions of time, soil permeability, capillary suction, or soil storage capacity. However, estimates of any of the proposed infiltration parameters in most cases remain largely guess-work.

Even at a site at which infiltration tests have been performed, a change in ion concentration due to major rainfall or runoff events or due to some form of surface pollution may alter the soil permeability drastically. Rose (1966) provides one of the most thorough descriptions of soil physical and chemical factors which may affect infiltration rates; however, no specific or typical infiltration rates can be found in his otherwise highly instructive book.

Horton Equation

The best-known and most widely used infiltration equation is the one developed by Horton (1940) and is shown in Figure 17:

$$f = f_c + (f_o - f_c) \exp(-kt)$$

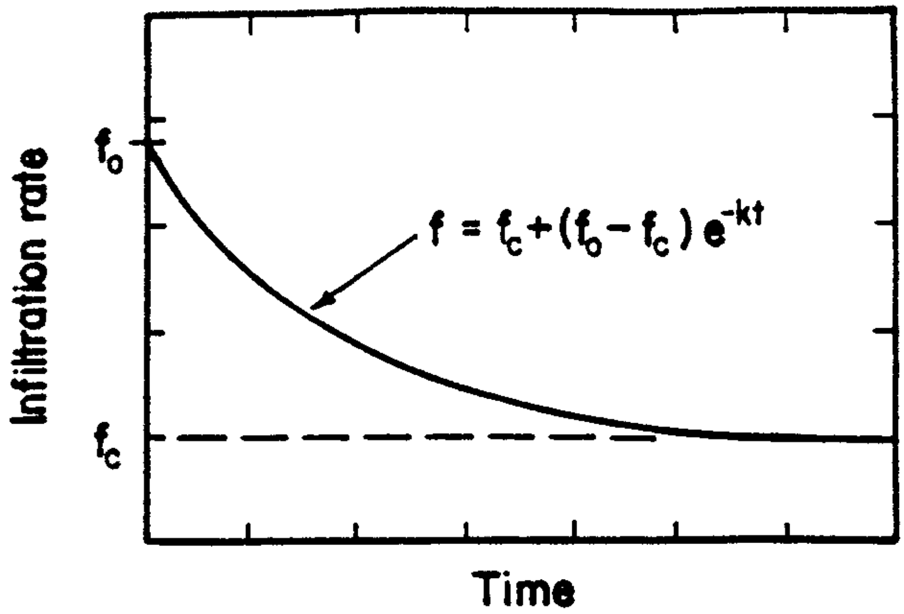


Fig. 17. Characteristic Horton infiltration rate.

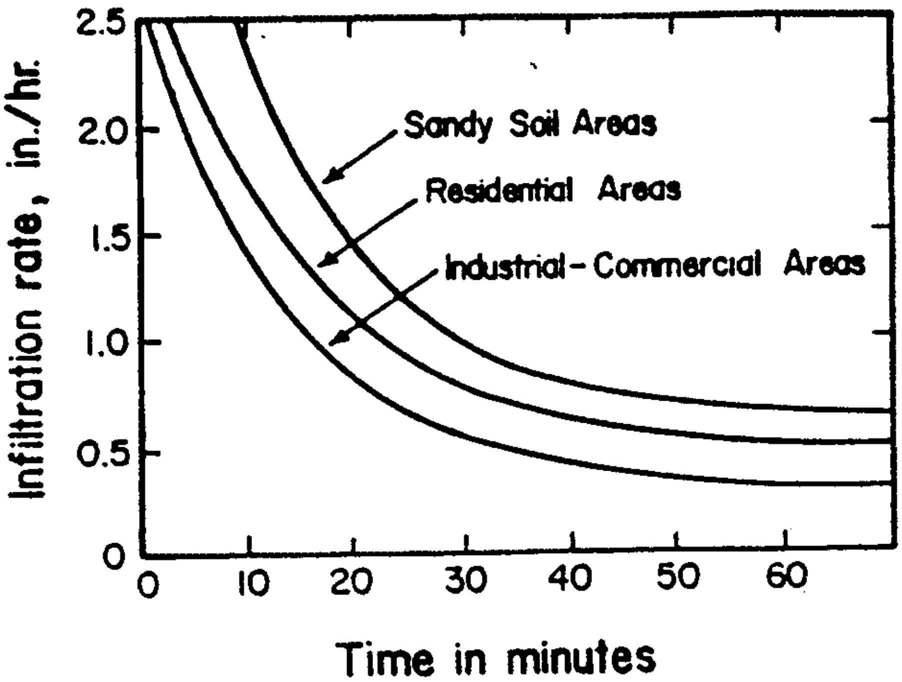


Fig. 18. Recommended typical infiltration rates [ASCE, 1970].

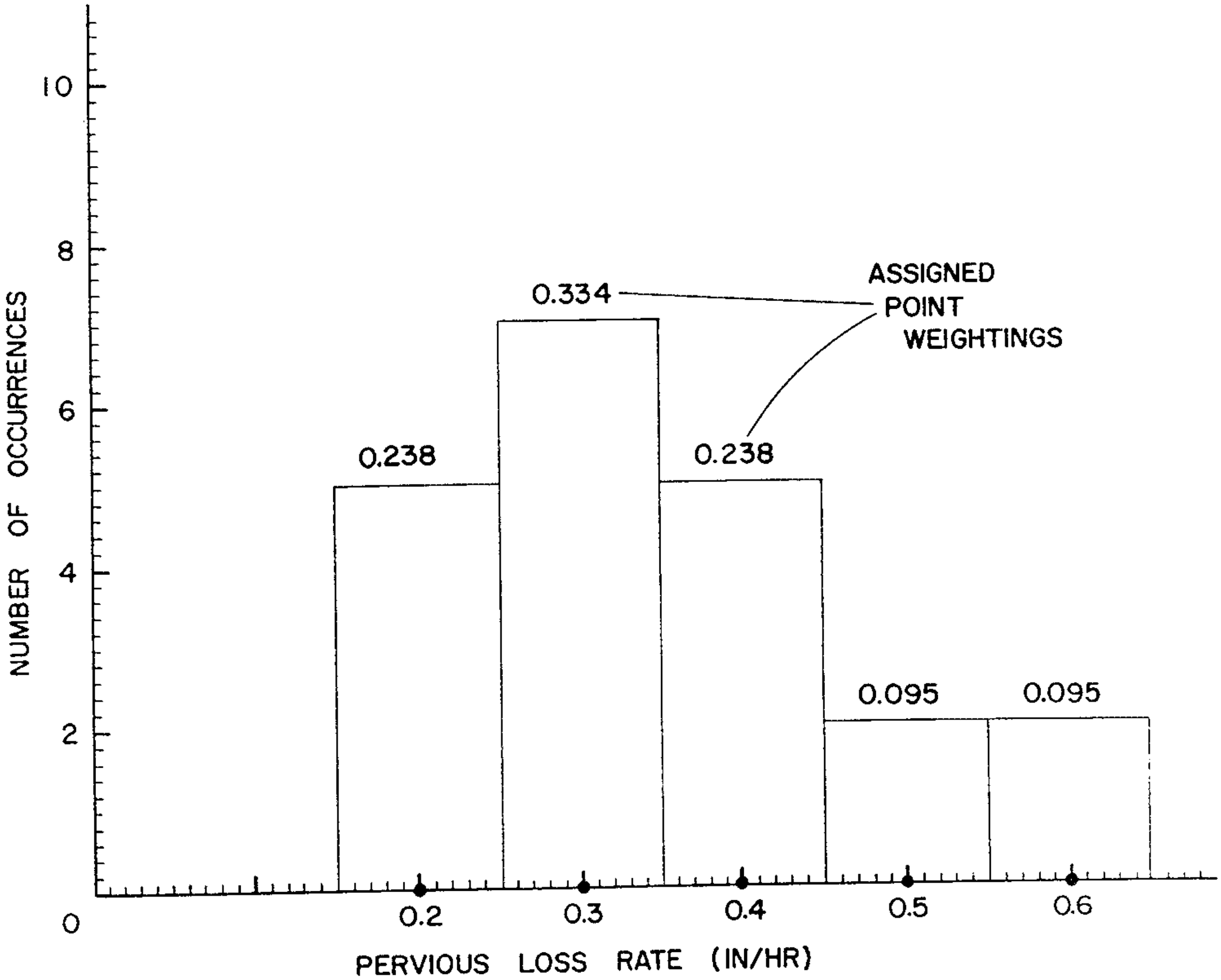


Fig. 19. Pervious Loss Rate Occurance Distribution

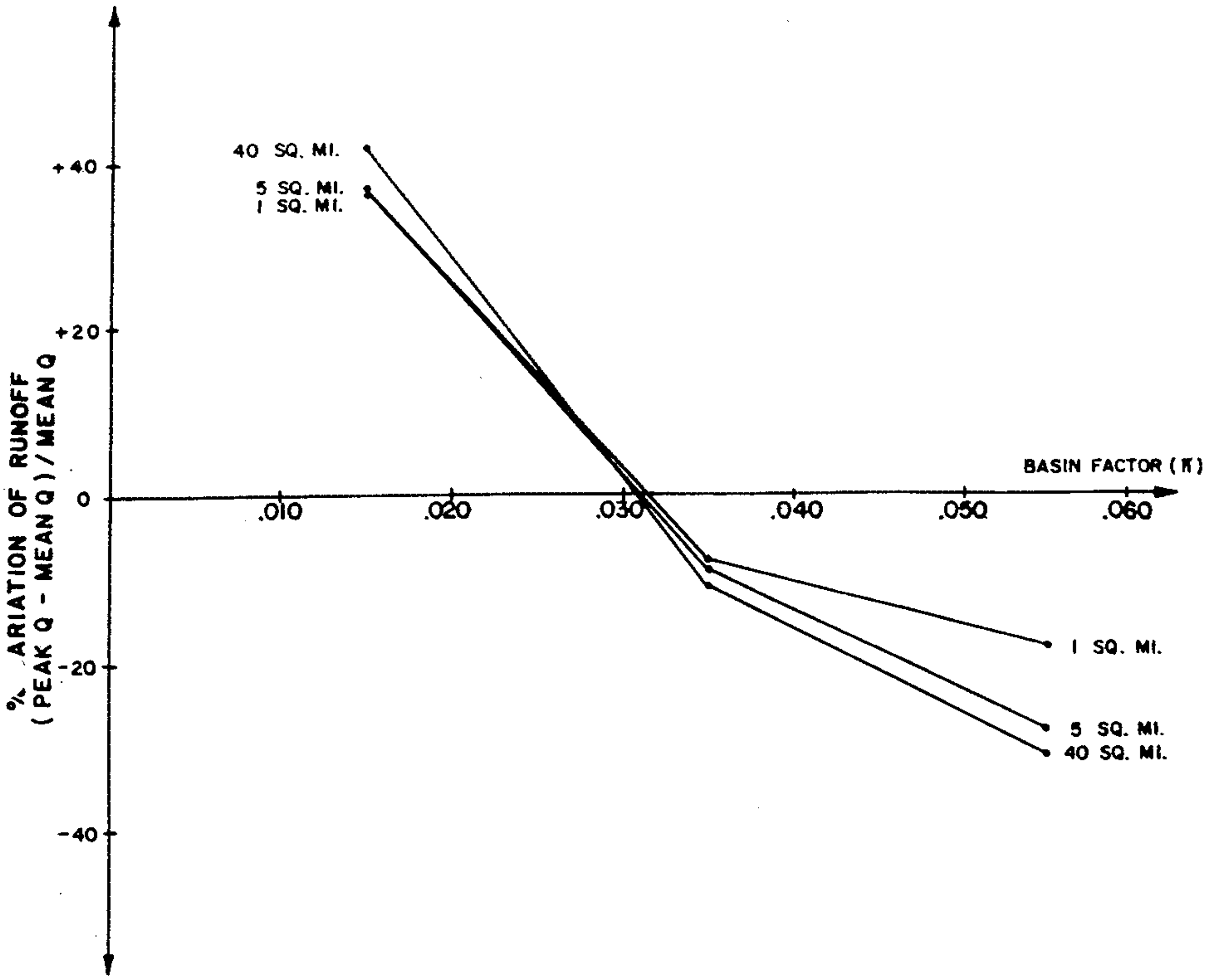


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FIGURE 20

VARIATION OF RUNOFF RESULTS FOR DIFFERENT VALUES OF BASIN FACTORS (n̄) FOR VARIOUS DRAINAGE AREA SIZES

SCALE	
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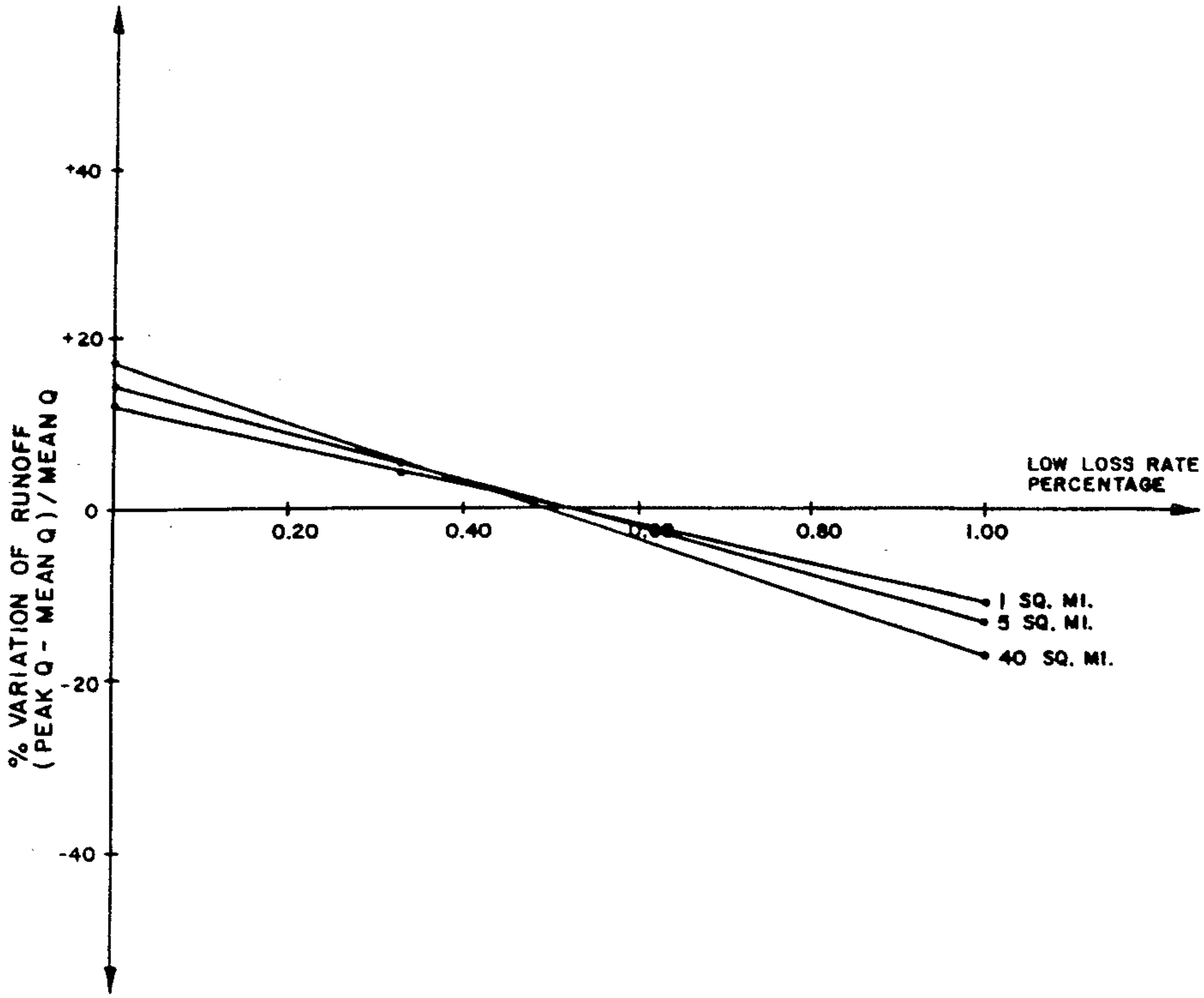

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FIGURE 21A

VARIATION OF RUNOFF RESULTS FOR DIFFERENT VALUES OF LOW LOSS RATE PERCENTAGE FOR VARIOUS DRAINAGE AREA SIZES

SCALE	
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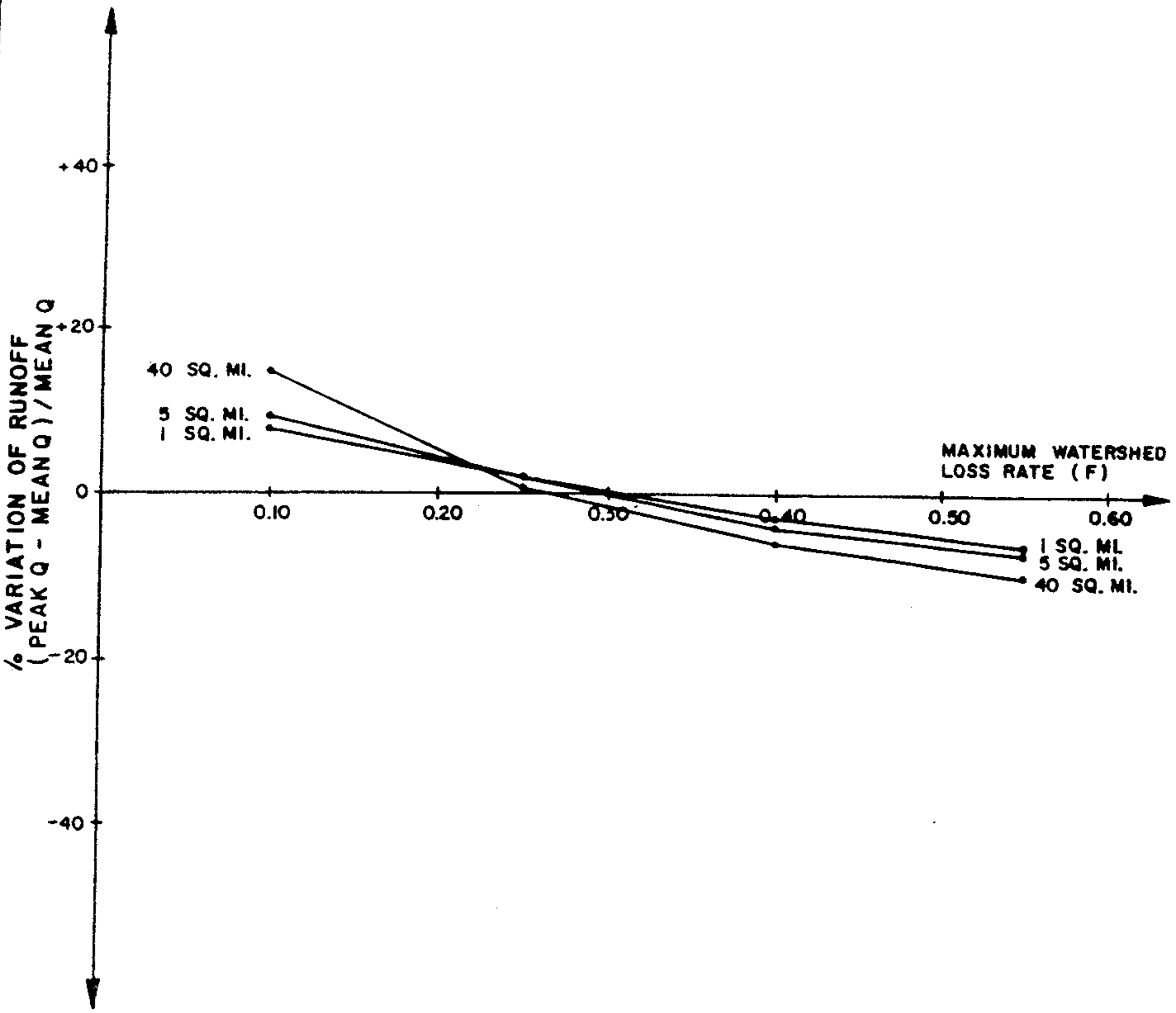

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FIGURE 21B

VARIATION OF RUNOFF RESULTS FOR DIFFERENT VALUES OF MAXIMUM WATERSHED LOSS RATE (Fm) FOR VARIOUS DRAINAGE AREAS SIZES

SCALE	
DRAWN BY	RLS
SURVEYED BY	
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FIELD BOOK	
DATE	1-30-84
JOB NO.	

VI. AN EVALUATION OF A COASTAL S-GRAPH (F01 WATERSHED)

VI.1 DISCUSSION

This 11,000-acre Santa Ana-Delhi (F01) watershed located in Orange County was recently studied (Bickel, 1984) to evaluate the severe storm hydrologic response using the Agency model.

In the referenced study, detailed attention was paid to the evaluation of the watershed S-graph. Using a severe storm of November, 1982, a unit hydrograph (and corresponding S-graph) was developed and subsequently used for hydrologic analysis.

In the study, a basin factor of n = 0.025 was assumed which produced a lag of 80 minutes. Based on this lag, runoff flowrates were generated and compared to the 1982 storm runoff hydrograph. The measure peak flowrate (1982 storm) was 1,300 cfs whereas the model (lag of 80 minutes) produced 1,911 cfs. The referenced report concluded that the model overpredicted flowrates, and that the Valley S-graph may be inappropriate.

In this chapter, the F01 S-graph of Bickel (1984) is re-examined with respect to the definition of watershed lag used by the COE. In this analysis it is seen that the reconstituted S-graph from the November 1982 storm actually closely follows the Valley S-graph currently in use by the Agency, but the watershed lag corresponding to this unit hydrograph is 2 hours rather than the 80-minute lag used by Bickel (1984).

VI.2 EVALUATION OF LAG

Fortunately, the Bickel (1984) report included an S-graph developed from the 1982 storm (Figure 22). From the S-graph, the lag is determined immediately to be 2-hours rather than the 80-minute value assumed in Bickel (1984). This lag corresponds to a basin factor (using the COE lag formula) of n = 0.0375 which agrees well with a recent COE estimate of n = 0.040. Table 14 summarizes the several lag estimates, including the rational method estimate for a time-of-concentration of 1.83 hours (OCEMA F01 1962 report).

TABLE 4: LAG ESTIMATES FOR F01 STREAMGAGE

Lag (Hours)	Basin n	Reference
1.33	0.0250	Bickel (1984) report
2.00	0.0375	1982 storm S-graph (See Fig. 22)
2.13	0.0400	COE estimate
t_c=1.83	-----	1962 OCEMA rational method

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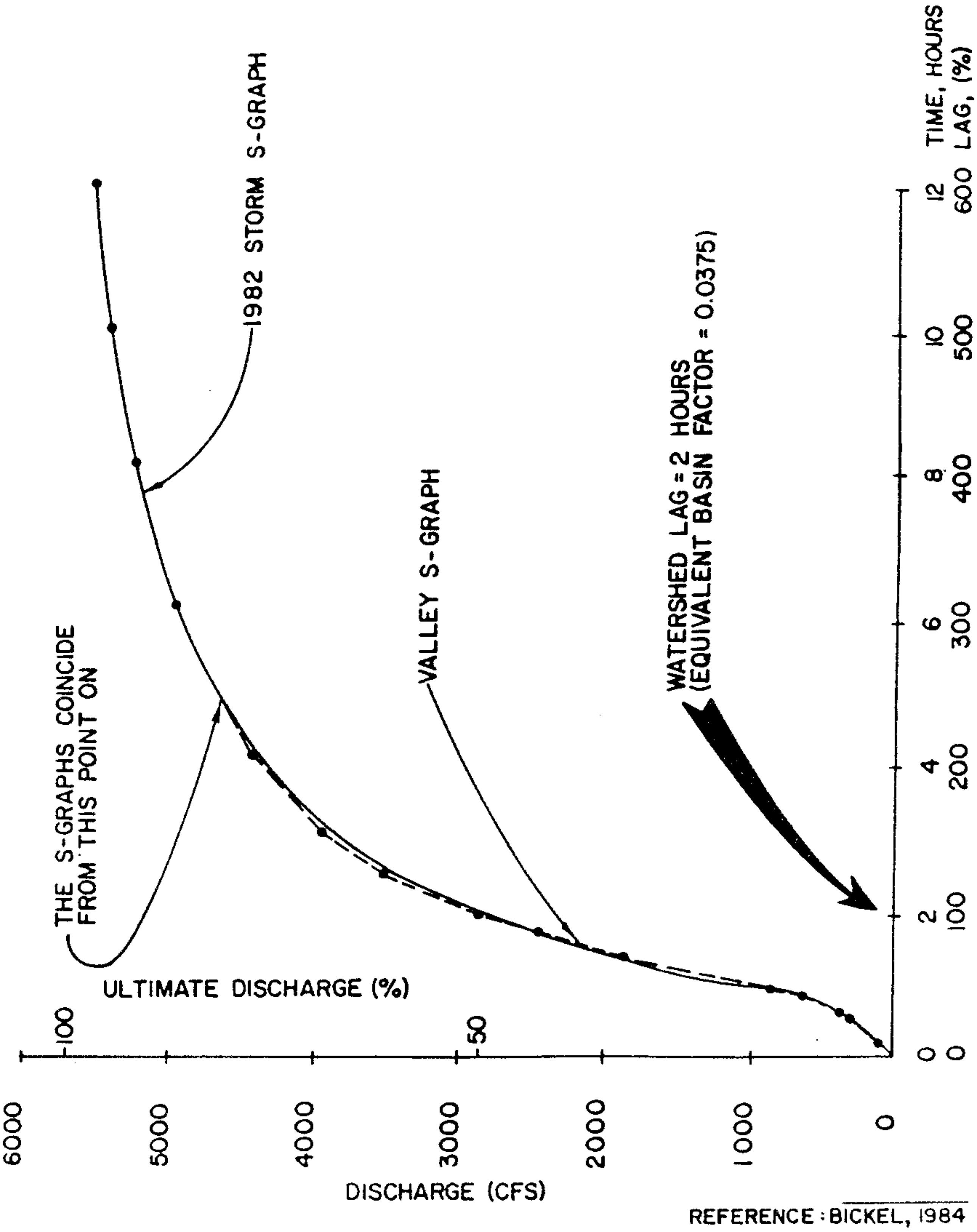

Figure 22. Santa Ana-Delhi Watershed S-Graph
From November, 1982 Storm

Table 5 illustrates the variation in peak flowrate predictions and the variation in modeled time-to-peak to measured time-to-peak for the 1982 storm. Table 6 shows the percentage variation in peak Q and time-to-peak estimates as compared to the corresponding measured values. In the provided results, the November 1982 storm pattern is directly used and the Valley S-graph is used to develop the F01 unit hydrograph for the various values of lag.

TABLE 5: FO1 WATERSHED MODEL ESTIMATES

Peak Q (cfs)	Variation in time-to-peak (minutes)	Comments
1,300	0	1982 storm
1,911	0	lag=1.33 hrs (n=0.025)
1,508	19	lag=2 hrs (from 1982 storm S-graph, (n=0.038, Fig. 22)
1,460	27	lag=2.13 hrs (n=0.04, COE)

TABLE 6: MODEL PERFORMANCE ON 1982 STORM FOR VARIOUS BASIN FACTOR ESTIMATES

(Model Q)/(GAGE Q)	(Error in time-to-peak)/lag	Comments
+47	0	n=0.025 (Bickel)
+16	+15.8	n=0.038 (S-graph, Fig. 22)
+12	+22.5	n=0.040 (COE)

From the above tables, it is seen good modeling results are achieved when using the Valley S-graph with the proper value of watershed lag. A 22 percent error in flowrate prediction is well within the accuracy of any hydrologic model. The 19-minute offset in time-to-peak is also a good estimate due to the fact that the measured 1982 storm hydrograph produced runoff quantities that are within 5-percent of the instantaneous peak flow rate for a time period of about 20-minutes, and also due to the uncertainty in synchronization between raingages and streamgages.

VI.3 EVALUATION OF THE 1982 STORM S-GRAPH

Figure 22 includes the S-graph developed from the 1982 storm. Superimposed on the figure is the Valley S-graph used by the Agency. From the figure, the ultimate discharge is 5,700 cfs. Thus, 100% lag occurs at 2,850 cfs which corresponds to a lag of 2 hours.

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From the figure, the Valley S-graph closely duplicates the developed S-graph from the F01 channel stream gage. Consequently, the F01 study provides good support for using the Valley S-graph in watersheds that are hydrologically similar to the F01 watershed. This result conforms to recent COE (telephone communication with Mr. John Pederson, Los Angeles office) conclusions that the Valley S-graph is appropriate, but that watershed lag is being estimated inaccurately when simply using the basin factor selected from a generalized tabulation.

It is noted that the rational method tc value of 1.83 hours compares well to the lag of 2.00 hours. This data is included in the correlation of lag to tc contained in Chapter III, where the recommended lag = 0.77 tc relationship would result in the estimate for the FO1 channel of lag = (0.77)(1.83 hours)=1.41 hours.

VI.4 EVALUATION OF Q_{100}

By re-examining the Q_{100} estimates from the F01 report of Bickel (1984), it is seen that the Agency's 100-year design storm model performs well in estimating a Q_{100} when using the Valley S-graph and a watershed lag of 2-hours (see Table 7).

TABLE 7: Q_{100} ESTIMATES FOR SANTA ANA-DELHI (FOI) CHANNEL

Q_{100}(cfs)	NOTES
7,930	Bickel (1984)
7,990	lag=2-hours (Fig. 22), Valley S-graph

VI.5 CONCLUSIONS

From the results presented in this chapter, the following conclusions are forwarded:

(1) The Valley S-graph may be appropriate for use in watersheds hydrologically similar to the current FO1 system, when lag is properly estimated.

(2) The current Valley S-graph closely matches the November 1982 storm S-graph of Bickel (1984).

(3) The Valley S-graph may be appropriate for use as a Coastal S-graph for flat, urbanized drainage areas hydrologically similar to the F01 drainage area.

(4) Further study is needed to verify whether the Valley S-graph should be used as a standard Coastal S-graph.

VII. VERIFICATION OF THE AGENCY UNIT HYDROGRAPH RAINFALL-RUNOFF MODEL

VII.1 DISCUSSION

The procedures of watershed model testing by "split sampling", where the model is calibrated to one set of rainfall runoff data and then "verified" by application to another set of rainfall-runoff data are used in this report.

The Agency watershed model is essentially calibrated to rainfall-runoff data developed by the COE from the ongoing LACDA study and other previous studies. The results of the LACDA calibration study is based on reconstitution analysis of recent storm events in the neighboring Los Angeles watersheds for storms prior to 1983. This COE study suggests a new Urban S-graph, and a Foothill S-graph. Chapter VI suggests using the existing Valley S-graph for coastal watersheds which are hydrologically similar to the FO1 watershed system. During the design storm's peak rainfalls, Chapter IV recommends a mean previous loss rate of 0.30 inch/hour. The low loss rate precentage is calibrated to match the SCS 24-hour storm yields as a function of the curve number (CN). Finally, the important watershed lag parameter is calibrated in Chapter III to be computed, on the average, as a proportion of the Agency's rational method tc estimate rather than using a generalized parameter table and an associated lag formula.

In this chapter, the unit hydrograph model is verifed by testing its application to the March 1, 1983 storm event in Los Angeles. Again, the March 1 storm was not considered as a calibration sample storm and, consequently, these model tests use only the parameters and procedures developed from the calibration storm set.

In this section are contained the results of using the Agency's procedure for estimating March 1, 1983 storm runoff quantities recorded for five watersheds in Los Angeles, California. These watersheds are listed in Table 8.

TABLE 8: LOS ANGELES WATERSHEDS
(MARCH 1, 1983 STORM TESTS)

Name	LACFCD Hydrologic Report (1982) Page Number
Rubio Wash @ Glendora	137
Arcadia Wash @ Grand	197
Eaton Wash @ Loftus	199
Dominguez Channel @ Vermont	223
Alhambra Wash @ Klingerman	135

inch/hour and an assumed impervious area fraction of 50 percent, for all test watersheds.

VII.3 MARCH 1, 1983 STORM

The March 1, 1983 storm produced stream gage record runoff at several of the considered watersheds. Table 9 itemizes the recorded stream gage peak flow rates.

TABLE 9: MARCH 1, 1983 STORM PEAK FLOWS

Watershed	Area (Square Miles)	Flow (cfs)
Rubio Wash	10.9	3760
Arcadia Wash	8.5	4110
Eaton Wash	22.8	5430
Dominguez Channel	37.3	9822
Alhambra Wash	15.2	7010

In Appendix C are contained on a watershed-by-watershed basis the following information:

1. LACFCD streamgage and watershed information
2. Comparison plots of modeled and measured streamflows for the March 1, 1983 storm event.

Contained in Appendix D are the March 1, 1983 unit rainfalls for the 24-hour distribution. In examining the results contained in Appendices C and D, it is seen that the rainfall-runoff correlations are weak for the Rubio, Arcadia and Eaton Washes for the storm hours of 15-24.

Raingage stations 235 and 449 are both contained within the Eaton Wash watershed and should apply to the Alhambra, Rubio, Arcadia, and Eaton Wash streamgages. The measured streamflow data indicated heavy runoff during the first 12-hours with less runoff during the second 12-hours of March 1. However, a comparison of raingages 235 and 449 indicates substantially heavier rainfall for gage 235 than that recorded at gage 449 (see Appendix D for raingage data).

To model these watersheds, raingage 235 was used as recorded, ignoring the data from raingage 449. In order to aid in compensating for excessively high rainfalls distributed over the watersheds by using only gage 235, the low loss rate percentage F* was increased to provide runoff volume yields assuming only F* applies. However, Fm (or f) was maintained as a constant 0.15 inch/hour which corresponds to 50 percent pervious area (estimated by the COE) and a pervious loss rate of 0.30 inch/hour (see Chapter IV).

Because of the rainfall pattern shown by gage 235, it would be expected that hour 19 would result in a high runoff value. The model does show high runoff values for hour 19, but this information contradicts the recorded streamgage data. Consequently, the available rainfall and runoff data indicate a weak correlation for storm hours 15-24.

Dominguez Channel is based on the data of raingage 291. Good correlation was achieved between rainfall and runoff for this watershed.

It is noted that for all watersheds, good correlation in timing of peak flows is evident. Because of the several peak flows shown in the recorded hydrographs, the model results are compared to streamgage data for each of the peak flows. These comparisons are summarized in Table 10. Because of the weak correlation between rainfall and runoff for hours 16-24, only hours 0-16 are considered in Table 10 for Rubio, Arcadia, and Eaton Washes.

TABLE 10: MARCH 1, 1983 PEAK FLOW ESTIMATES

Watershed	Recorded Q	Modeled Q	Relative Error
Rubio Wash	1500	2500	+ 67
	3750	4050	+ 8
	2500	3550	+ 42
Arcadia Wash	1350	2000	+ 48
	2850	3650	+ 28
	4100	2700	- 34
Eaton Wash	1800	2600	+ 44
	2850	3300	+ 16
	3500	5400	+ 54
Alhambra Wash	2300	3000	+ 30
	7000	4500	- 36
	1700	2050	+ 21
	5300	4100	- 23
	5250 [1]	5600	+ 7
Dominguez Channel	9800	8200	- 16
	6200	5500	- 11
	2400 [1]	2700	+ 13
	1700 [1]	2050	+ 21

Note [1]: flowrate occurred during storm hours 16-24

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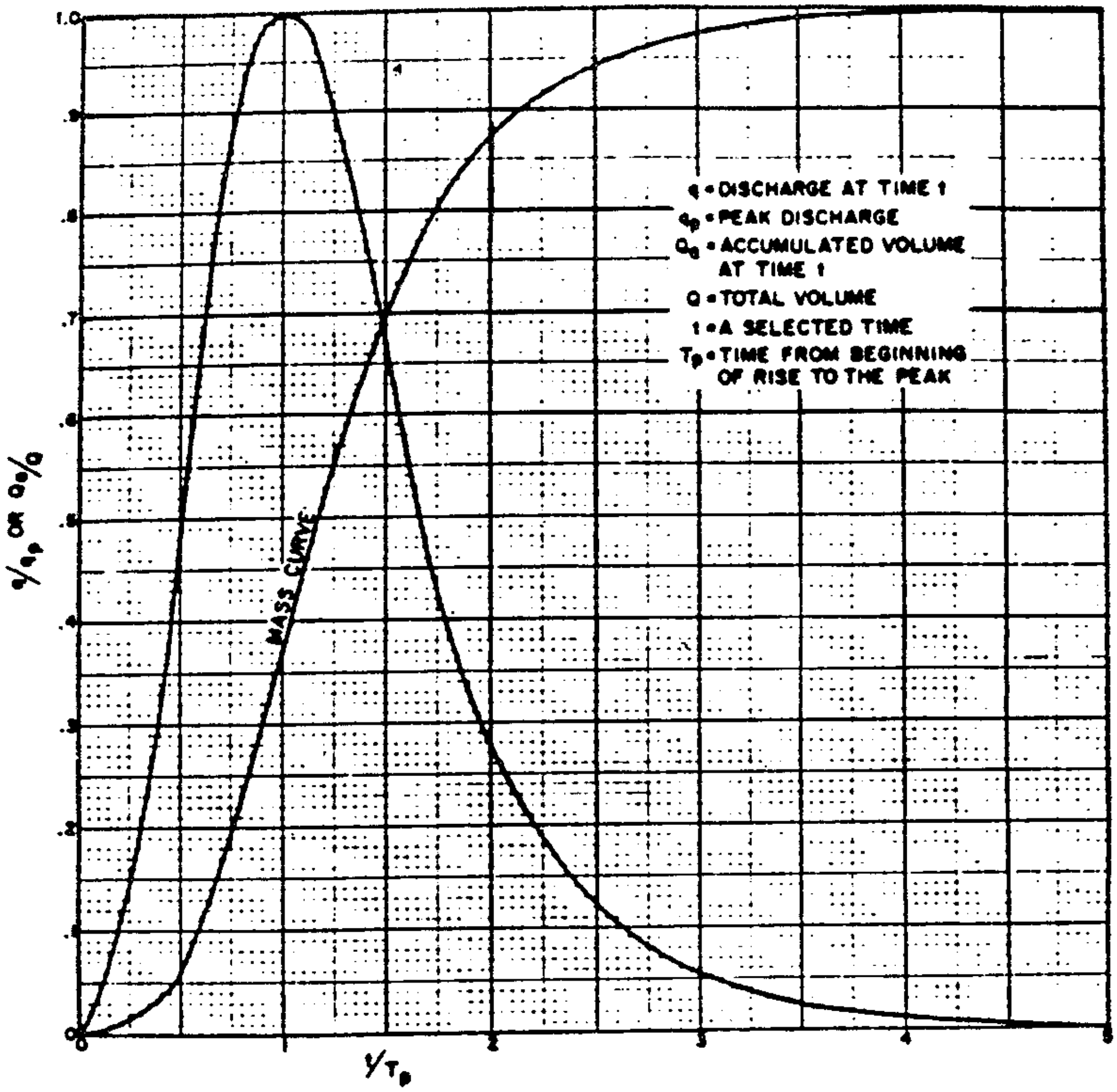


Figure 23. Unit Hydrograph (SCS)

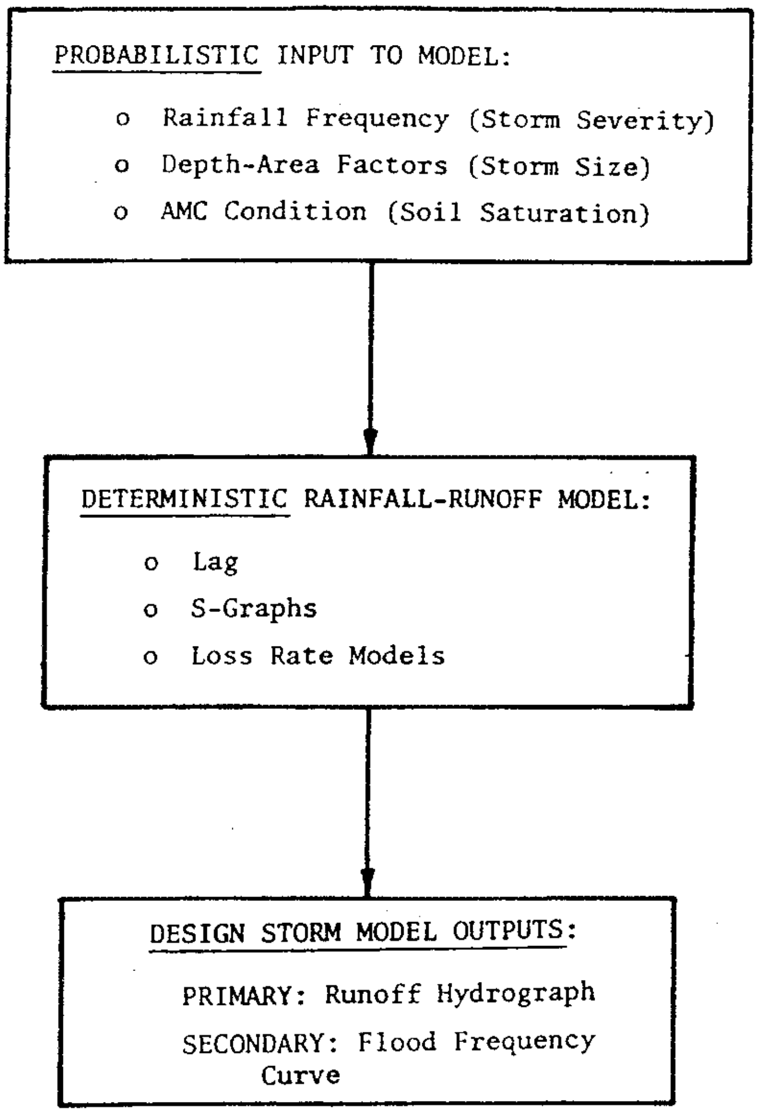


Figure 24. Design Storm Model Schematic

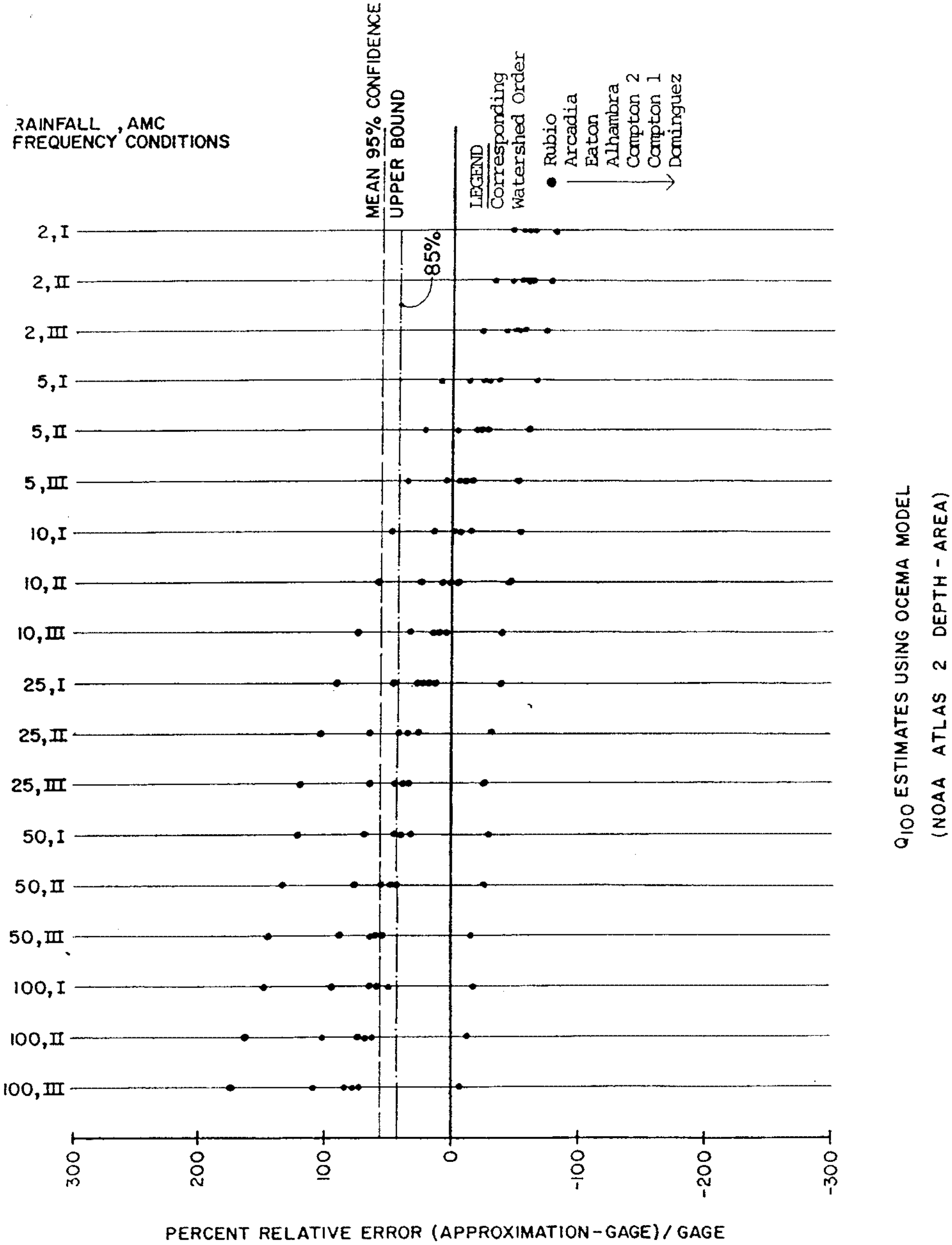


Figure 25. 100-Year Flow Estimates

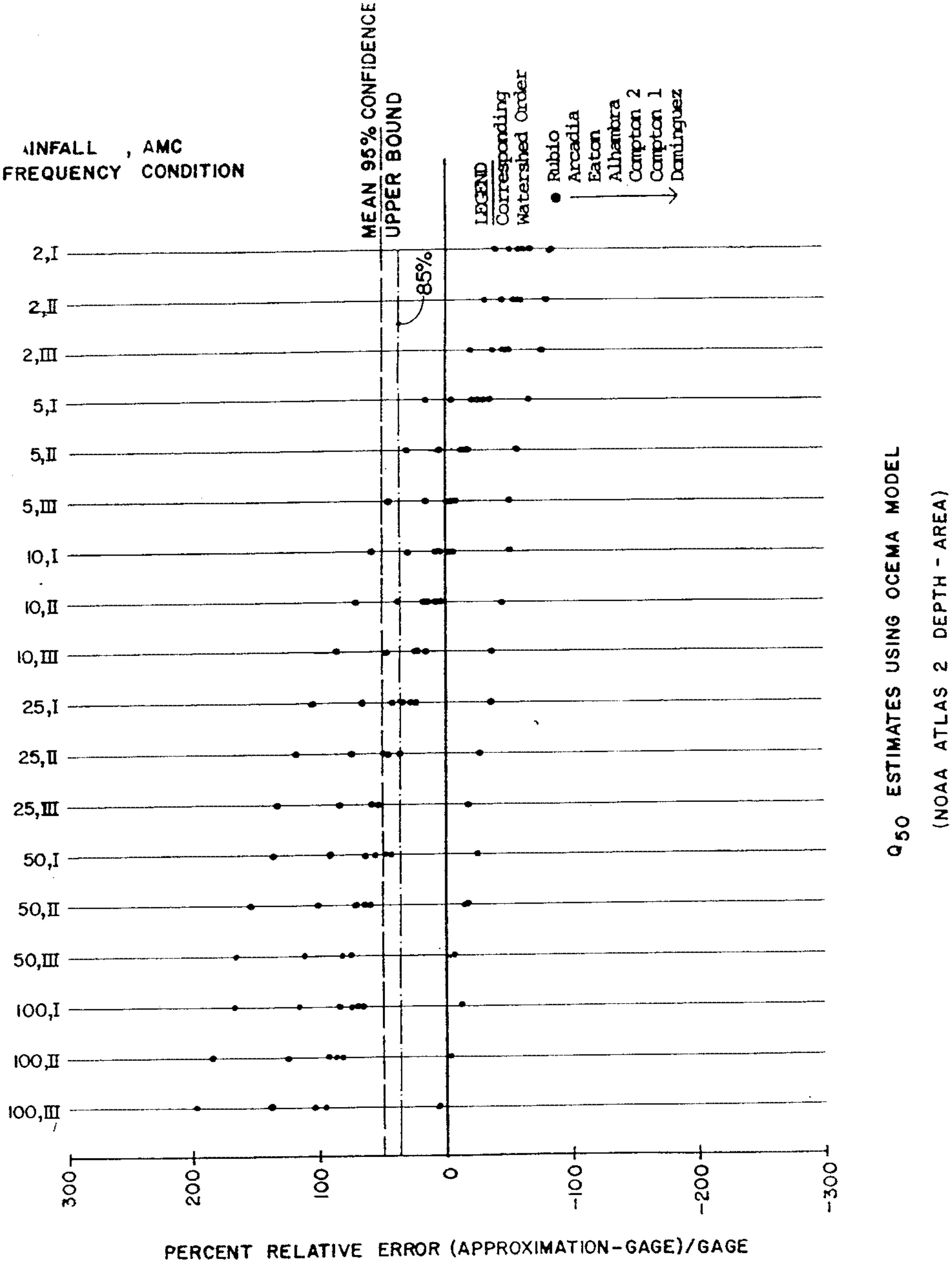


Figure 26. 50-Year Flow Estimates

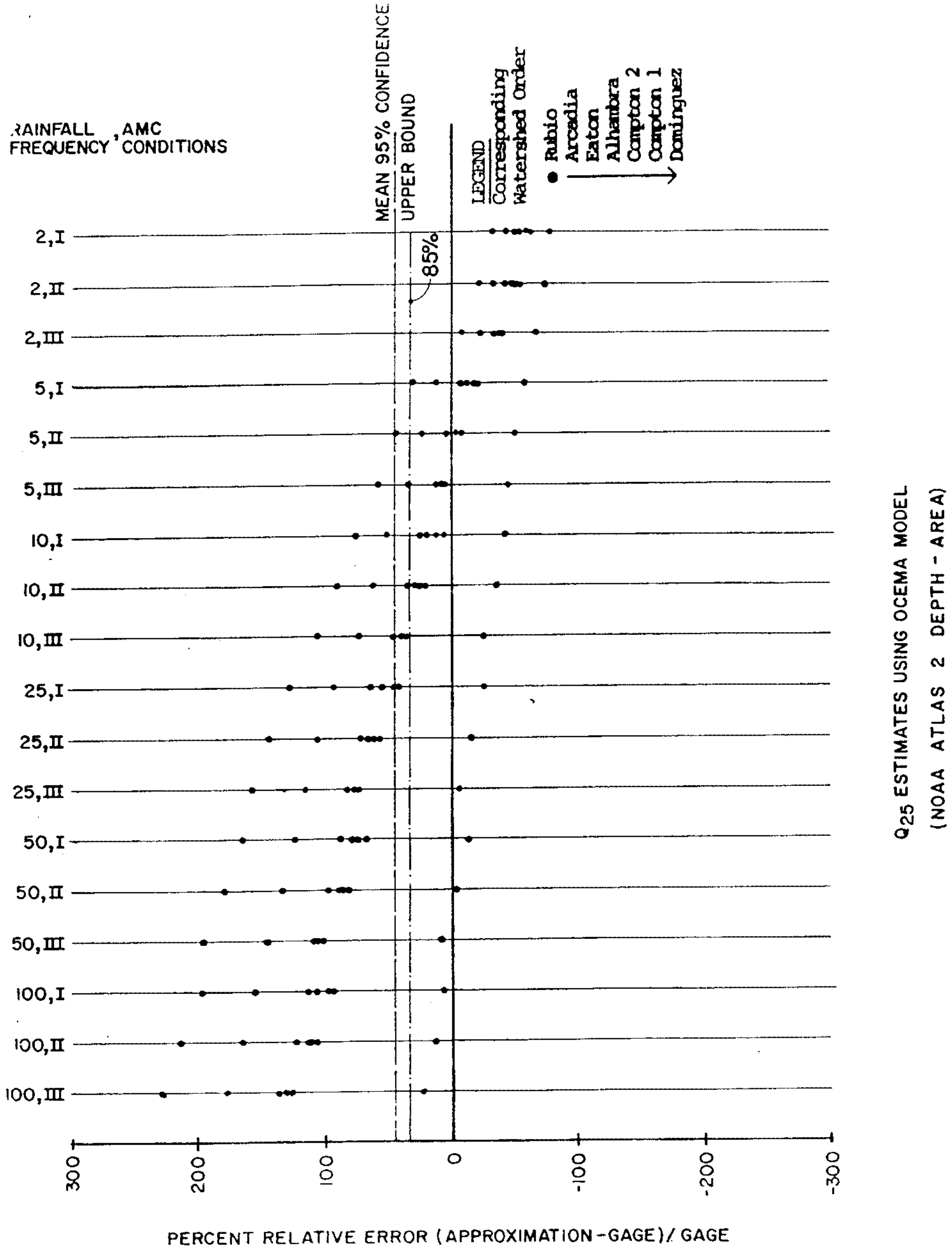


Figure 27. 25-Year Flow Estimates

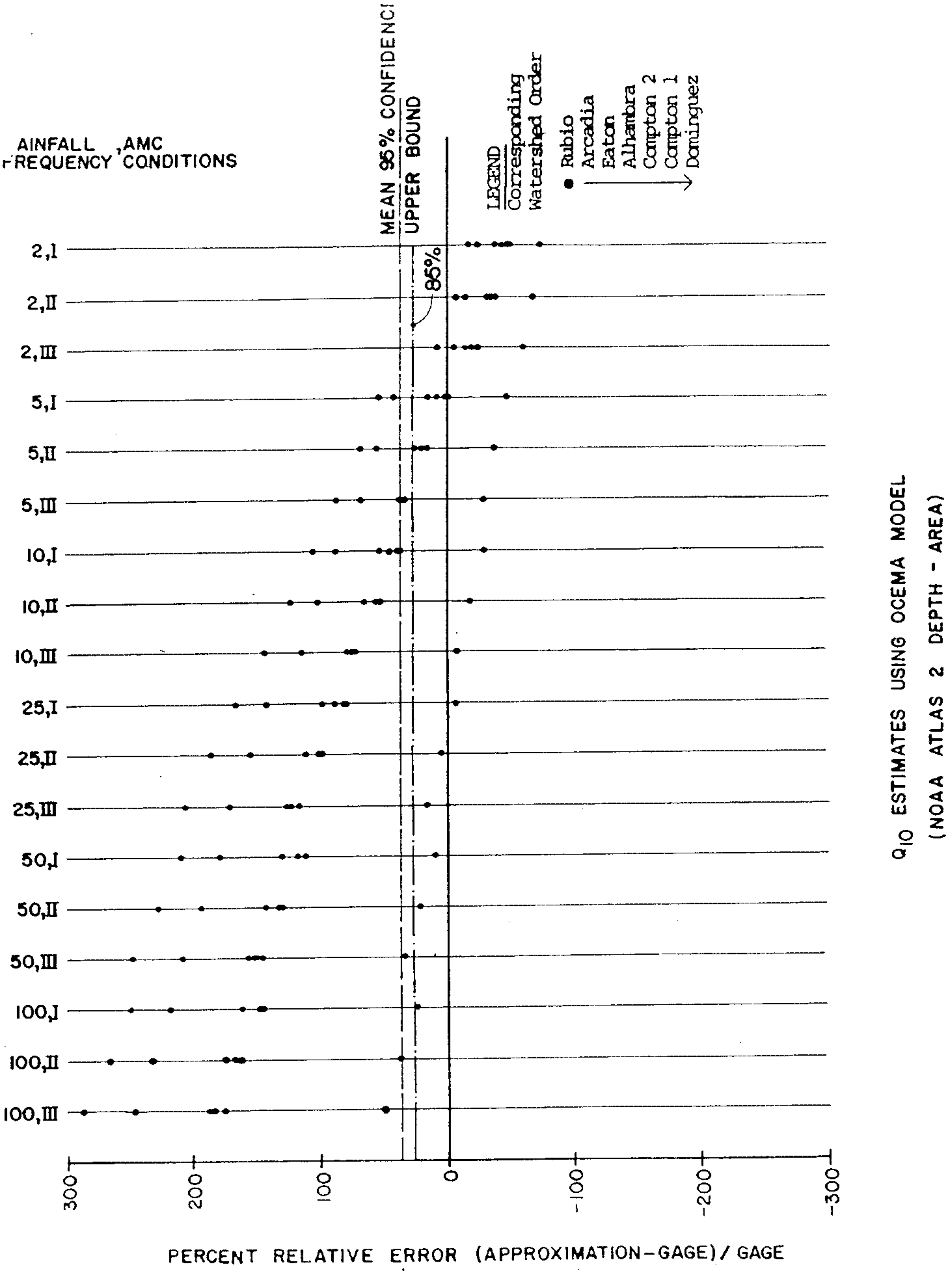


Figure 28. 10-Year Flow Estimates

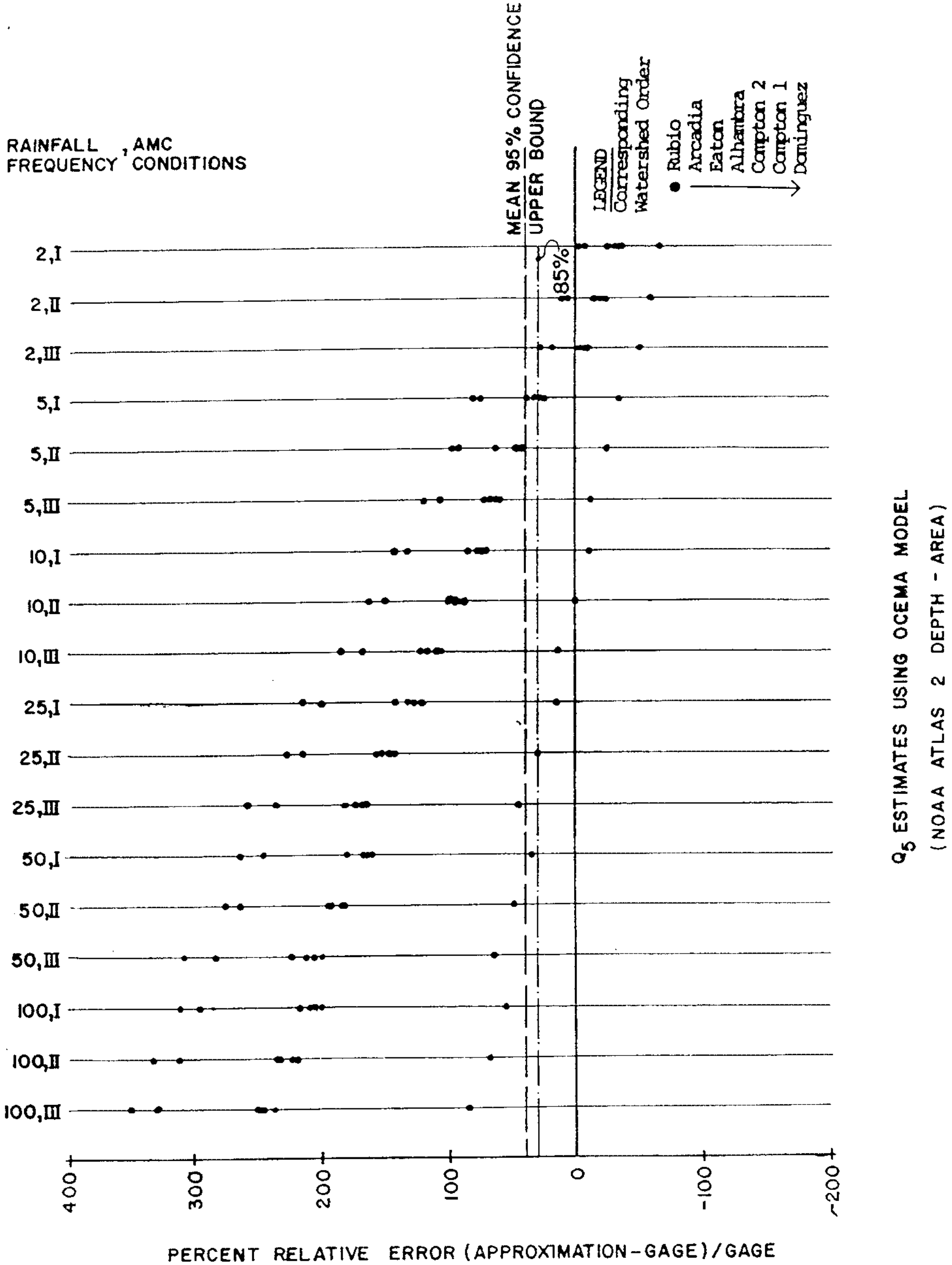


Figure 29. 5-Year Flow Estimates

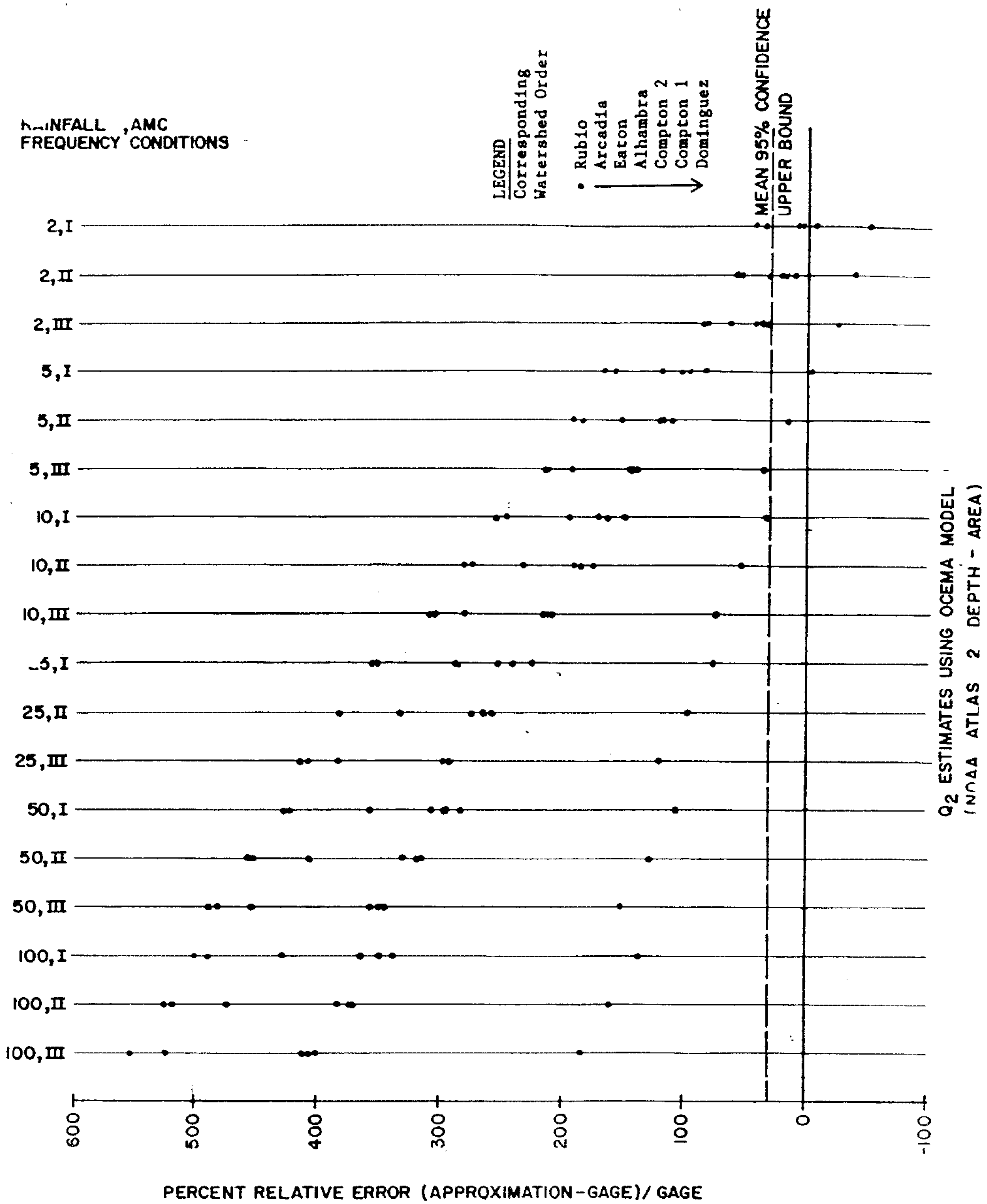


Figure 30. 2-Year Flow Estimates

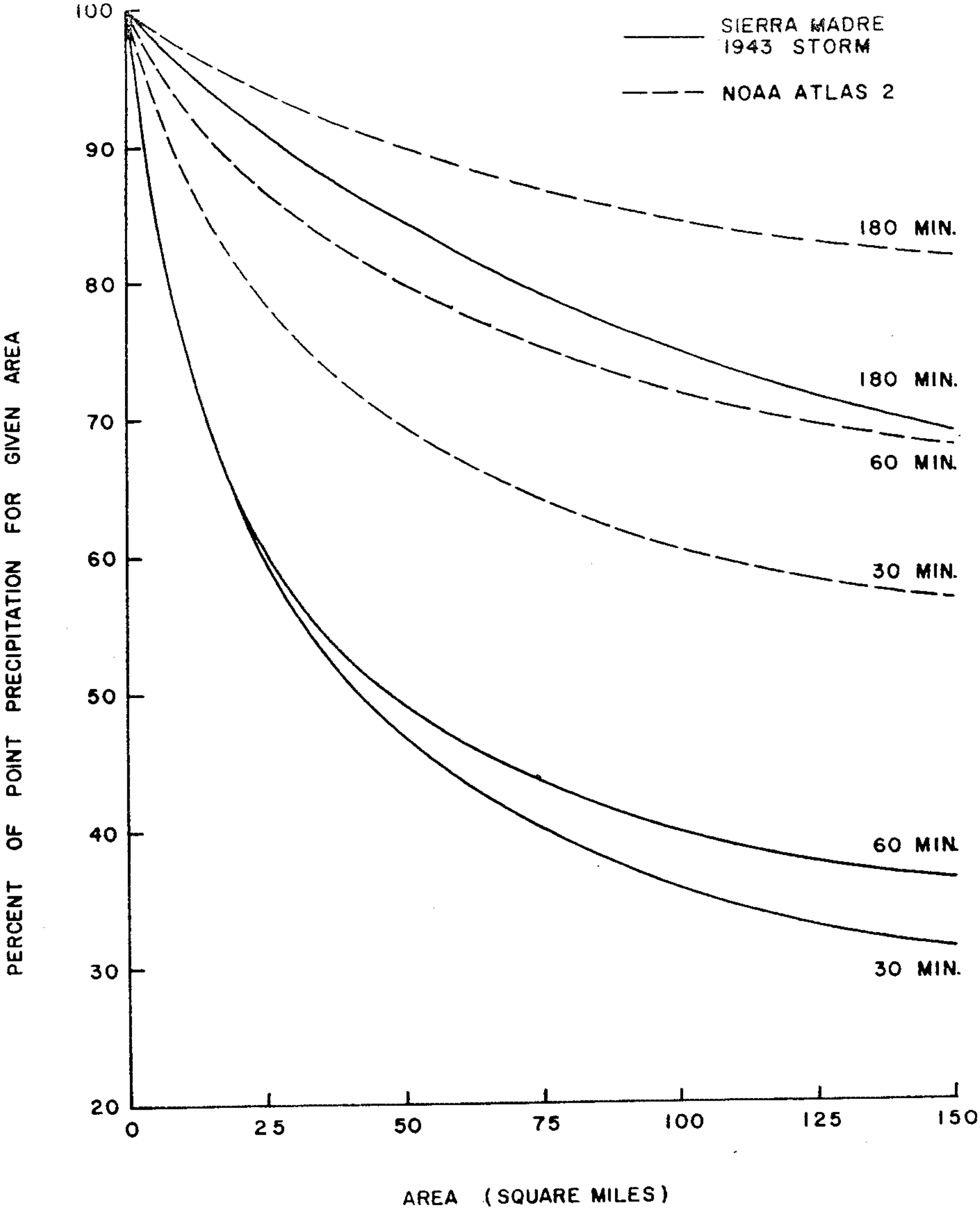


Figure 31. Comparison of Depth-Area Curves

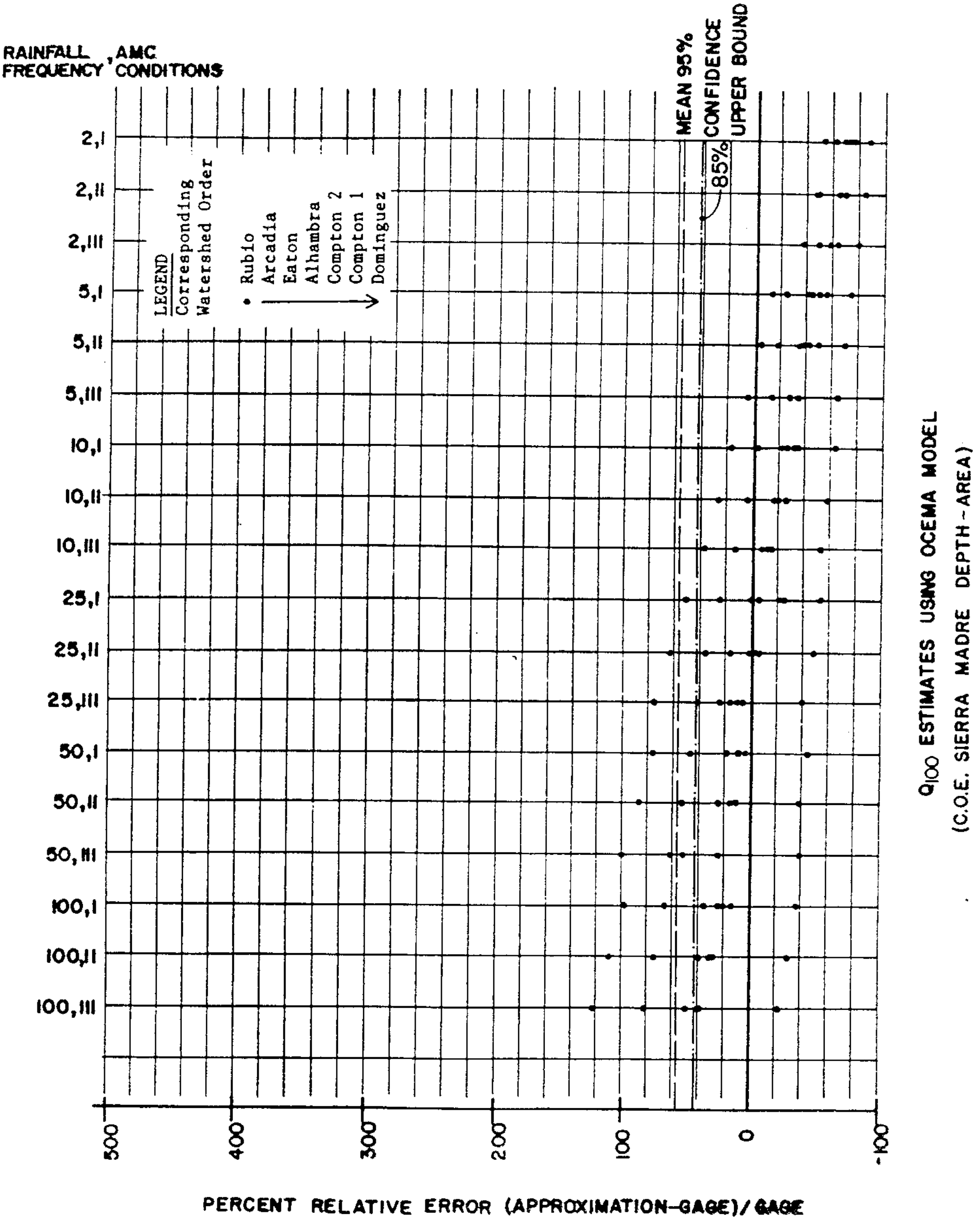


Figure 32. 100-Year Flow Estimates

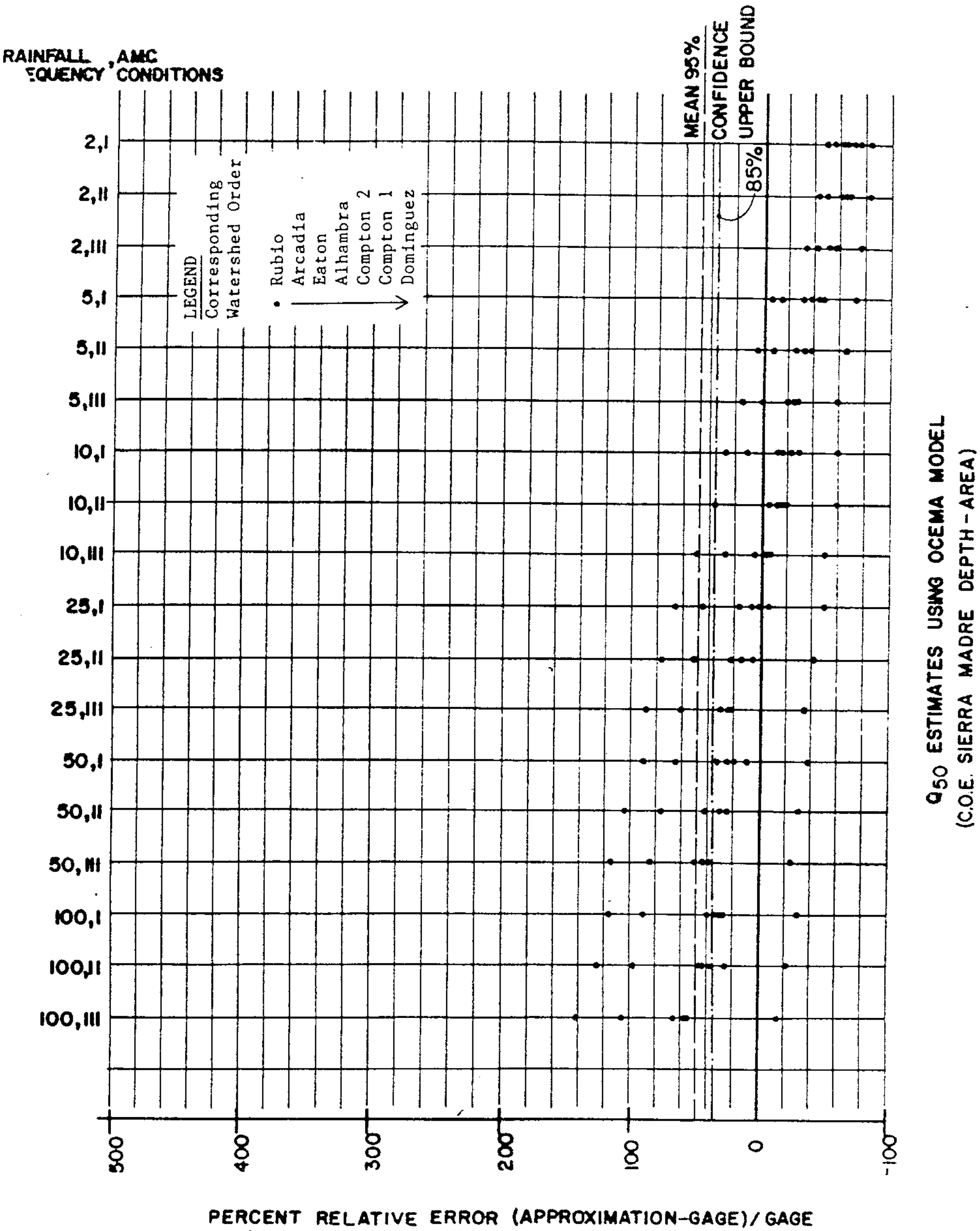


Figure 33. 50-Year Flow Estimates

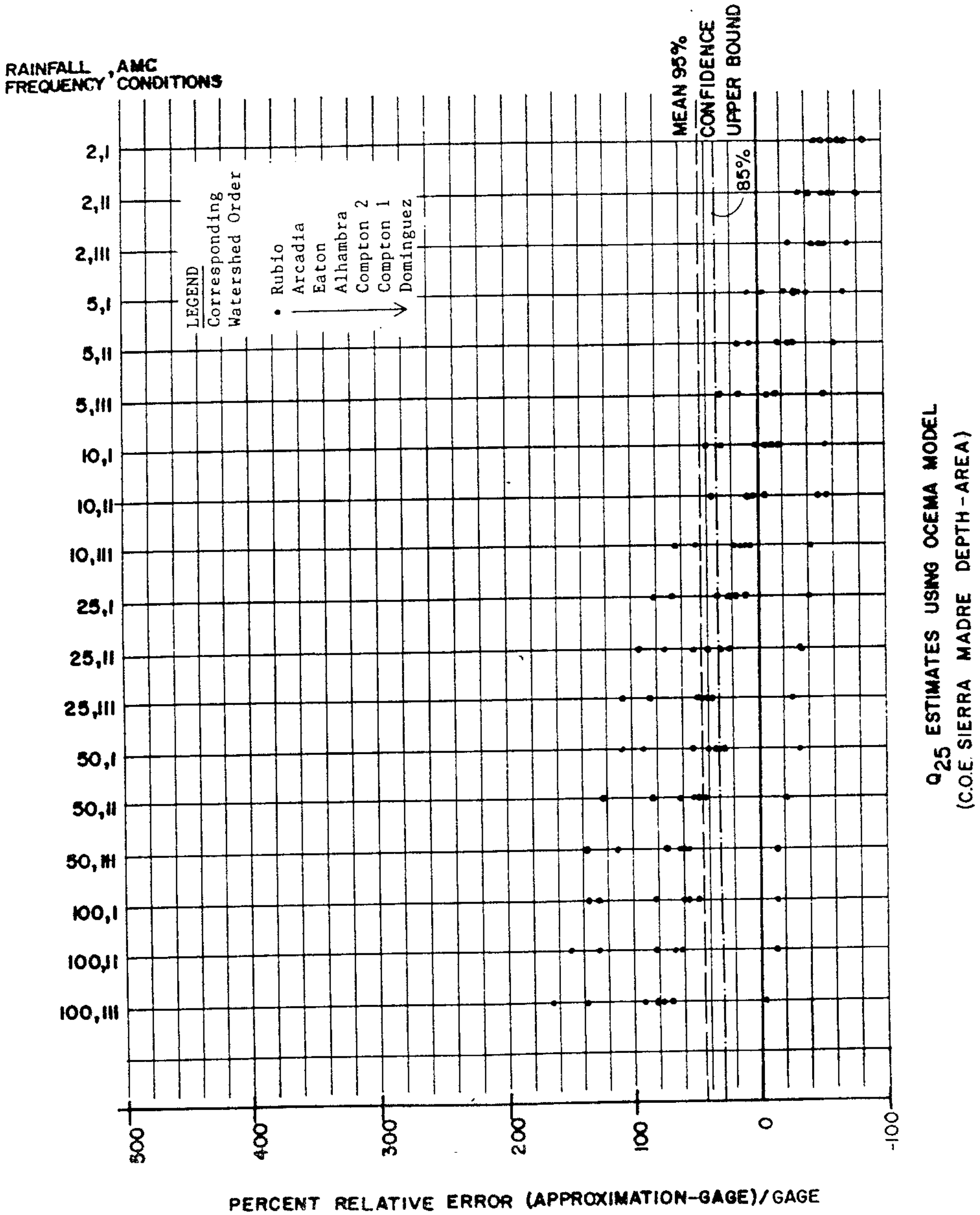


Figure 34. 25-Year Flow Estimates

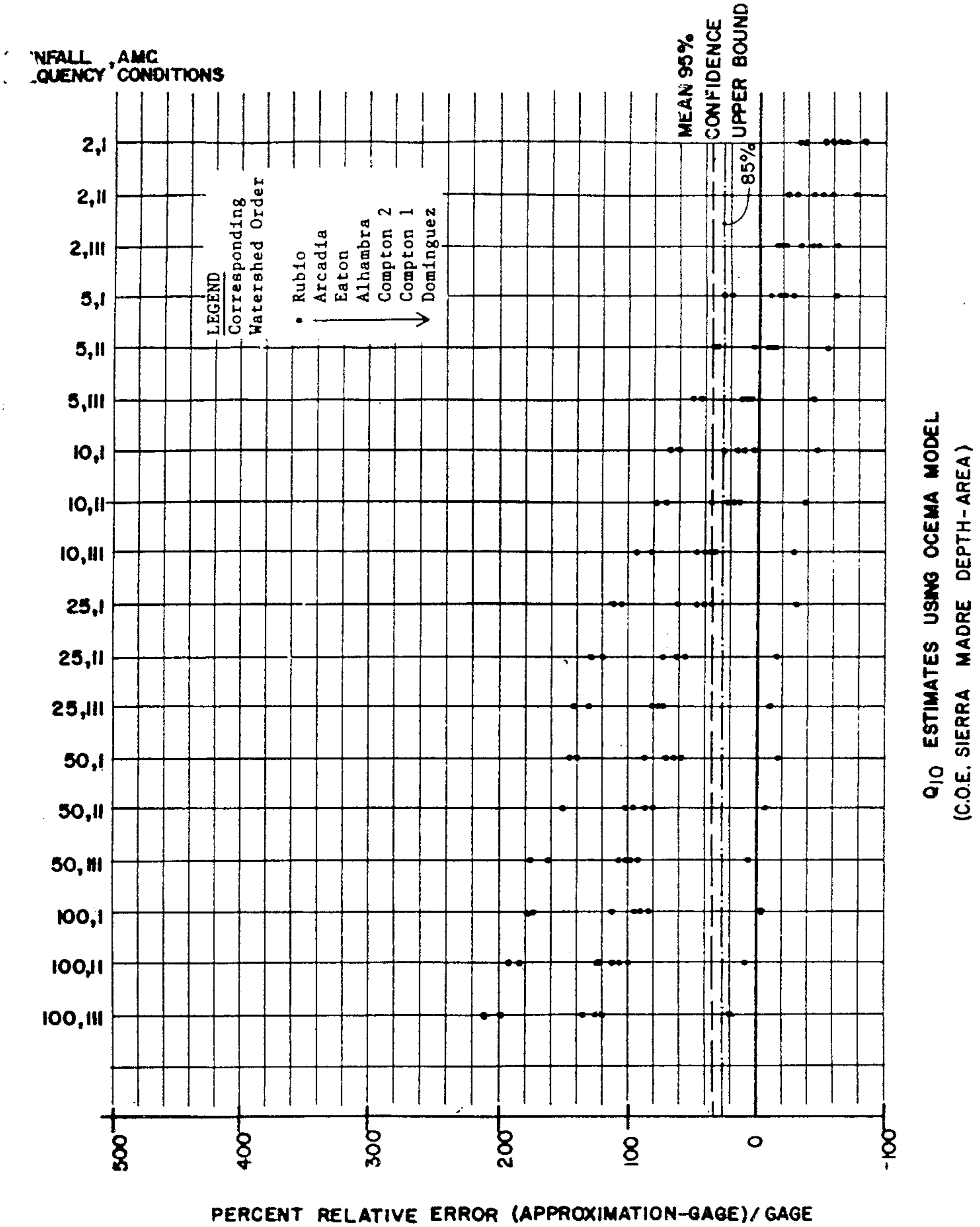


Figure 35. 10-Year Flow Estimates

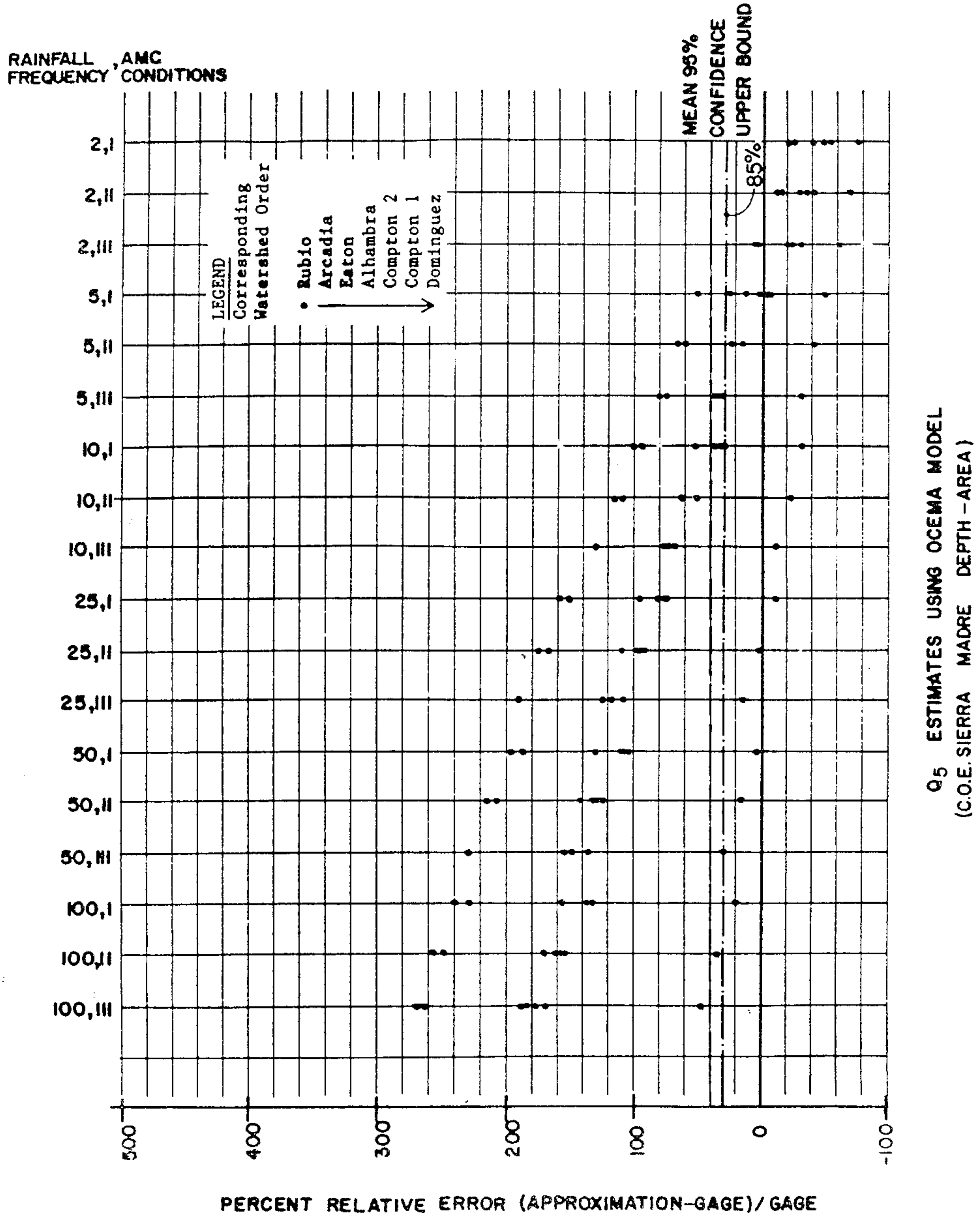


Figure 36. 5-Year Flow Estimates

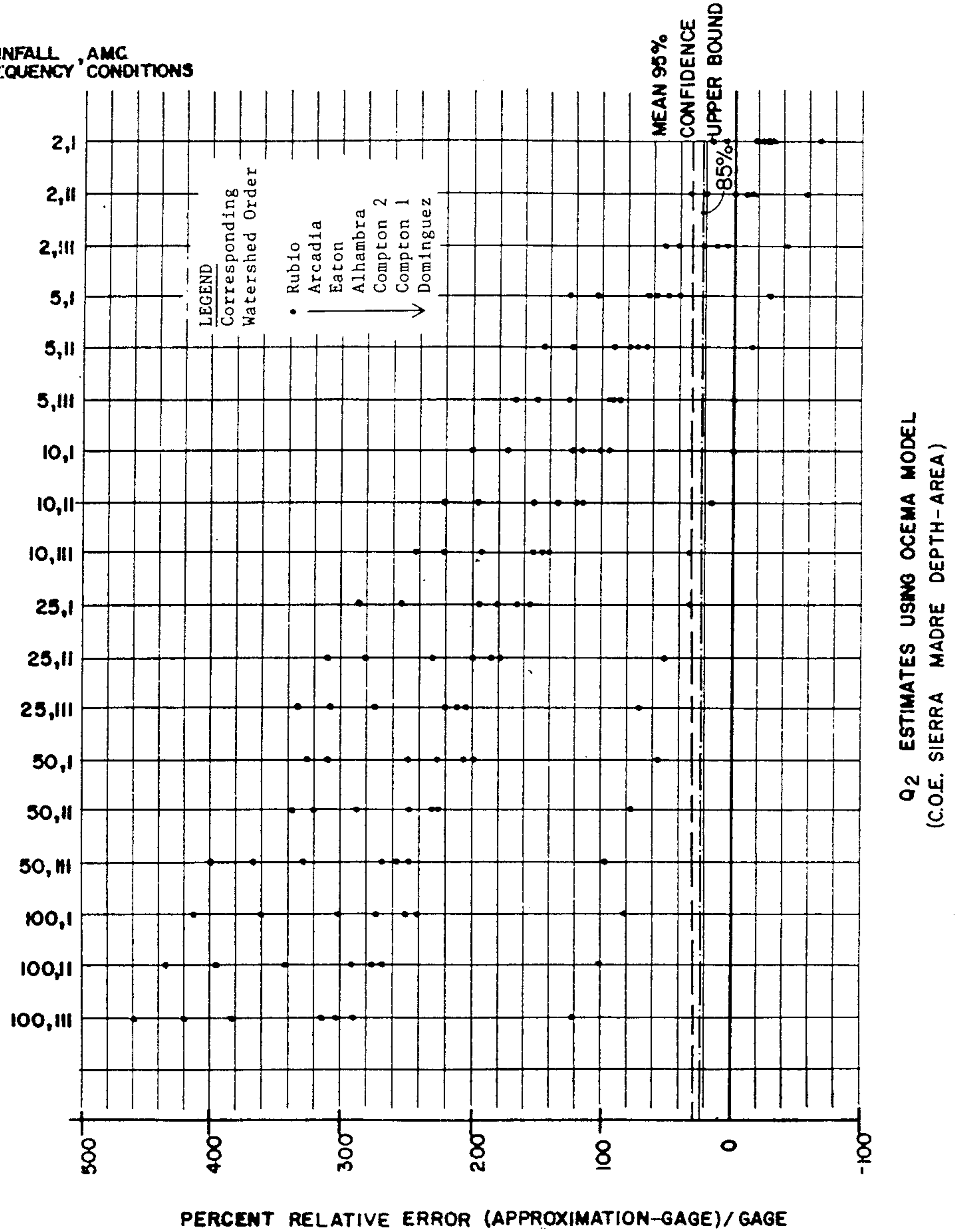


Figure 37. 2-Year Flow Estimates

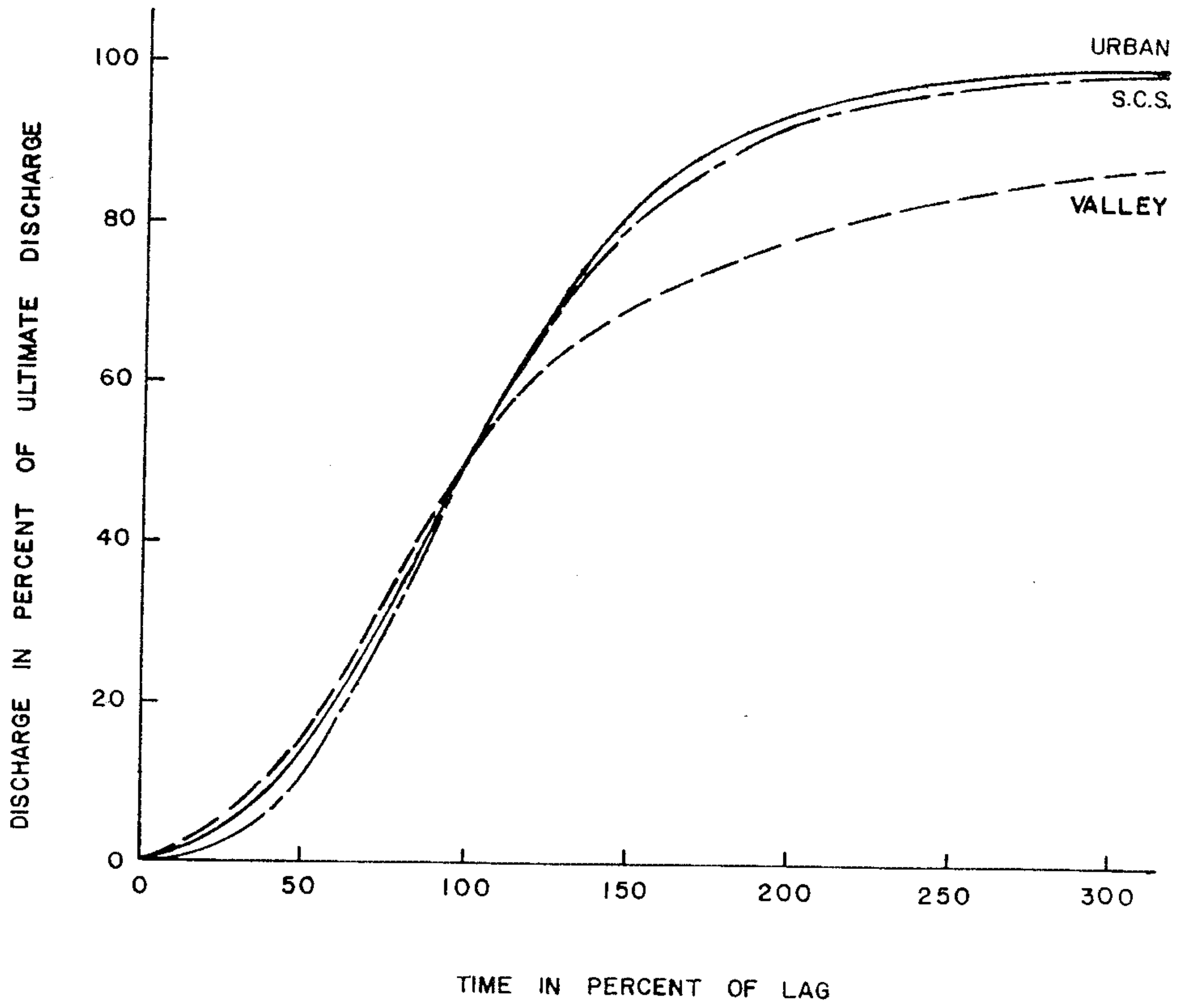


Figure 38. Comparison of S-Graphs

Table 12 summarizes the results of the study leading to Figs. 32 through 37:

TABLE 12: UNBIASED T-YEAR FLOODFLOW MODEL CONDITIONS

Runoff Return Frequency (years)	Rainfall Return Frequency (years)	AMC
2	2	II
5	5	I
10	5	III
25	10	II
50	10	III
100	25	II

From Table 12, the Agency may elect to modify the unbiased modeling input hydrologic conditions in order to provide a safety factor. For example, the unbiased 100-year peak flowrate estimate may be provided by Table 12 conditions, resulting in one-half of all watersheds (on the average) being overprotected and one-half of the watersheds being underprotected. By a cost-to-benefit analysis, it may prove prudent to provide a higher level of flood protection in order to reduce drainage deficiency design over 50 percent of the watersheds.

Table 13 summarizes the study results leading to flood flowrate estimates which, on the average, lie on the upper 85 percent confidence limit obtained from the flood frequency curve analysis.

TABLE 13: 85% CONFIDENCE LEVEL T-YEAR FLOODFLOW MODEL CONDITIONS

Runoff Return Frequency (years)	Rainfall Return Frequency (years)	AMC
2	2	III
5	5	II
10	10	II
25	25	II
50	50	II
100	100	III

Another approach to evaluate the rainfall-runoff modeling success in achieving a specificed level of flood protection is to weight the modeling relative errors by the watershed area (i.e., $((Q_m - Q_f)/Q_f\,(\%) = \Sigma(A_i/\Sigma A_i)((Q_{m_i} - Q_{f_i})/Q_{f_i})(100))$ where Q_{m_i} is the model estimate for gage i, Q_{f_i} is the flood frequency curve estimate for gage i, and A_i is the area of gage i. Figures 39 and 40 show the corresponding weighted relative error plots for the NOAA Atlas and COE depth-area relationships, respectively.

Tables 14 and 15 provide the necessary model inputs giving the unbiased and 85 percent confidence in model estimates using the area-weighted estimation procedure and the COE depth-area factors.

TABLE 14: UNBIASED T-YEAR FLOODFLOW MODEL CONDITIONS (AREA-WEIGHTED)

COE Depth-Area Adjustment

Runoff Return Frequency (years)	Rainfall Return Frequency (years)	AMC
2	2	II
5	5	II
10	10	I
25	10	III
50	25	II
100	50	I

TABLE 15: 85% CONFIDENCE T-YEAR FLOODFLOW MODEL CONDITIONS (AREA-WEIGHTED)

COE Depth-Area Adjustment

Runoff Return Frequency (years)	Rainfall Return Frequency (years)	AMC
2	2	III
5	10	I
10	25	I
25	50	I
50	100	I
100	100 to 150	III

Included in Figs. 39 and 40 are the average upper 85% and 95% confidence limits for the 100-year peak flow estimates using flood frequency curve analysis procedures (for example, compare to Figs. 32-37). Table 15 incorporates these lower confidence intervals for the more frequent storm events. It is noted that the results of Figs. 39 and 40 are only intended to relate the design storm input variables of return frequency and watershed loss AMC to the estimated flood frequency curve values. Consequently, these figures should not be construed to mean that the return frequency of

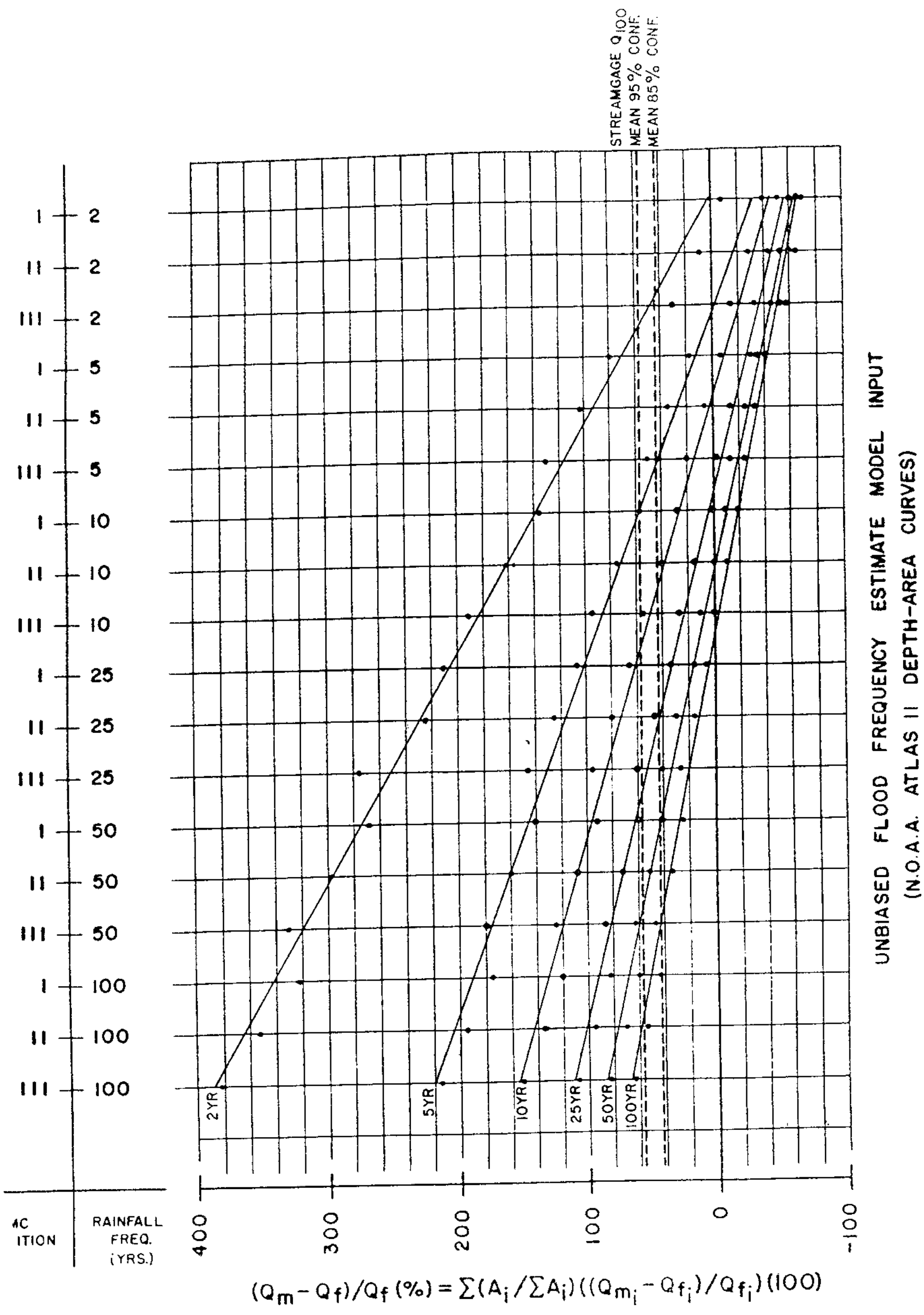


Figure 39. Area-Weighted Model Runoff Frequency Estimates (NOAA Atlas 2 Depth-Area)

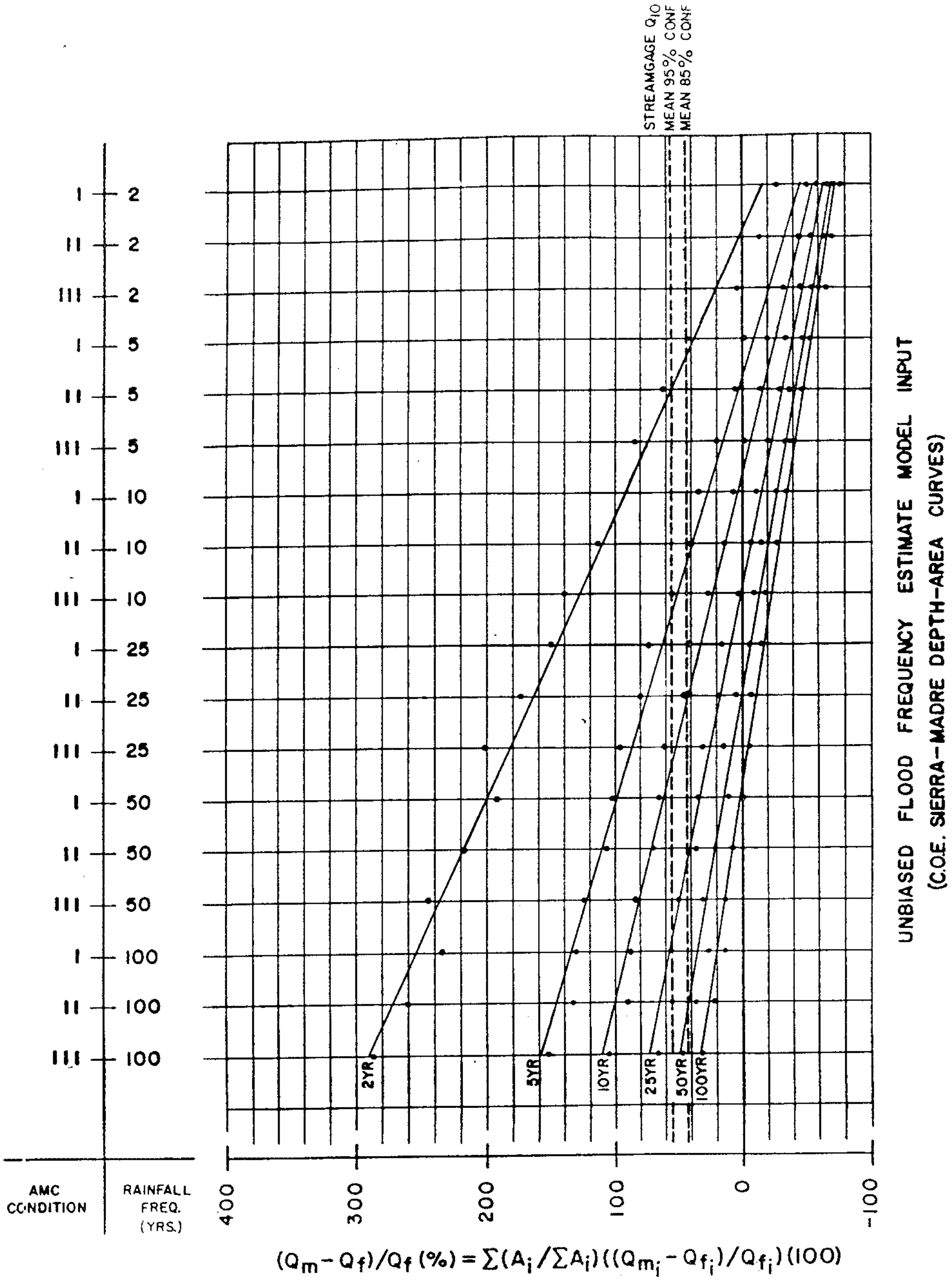


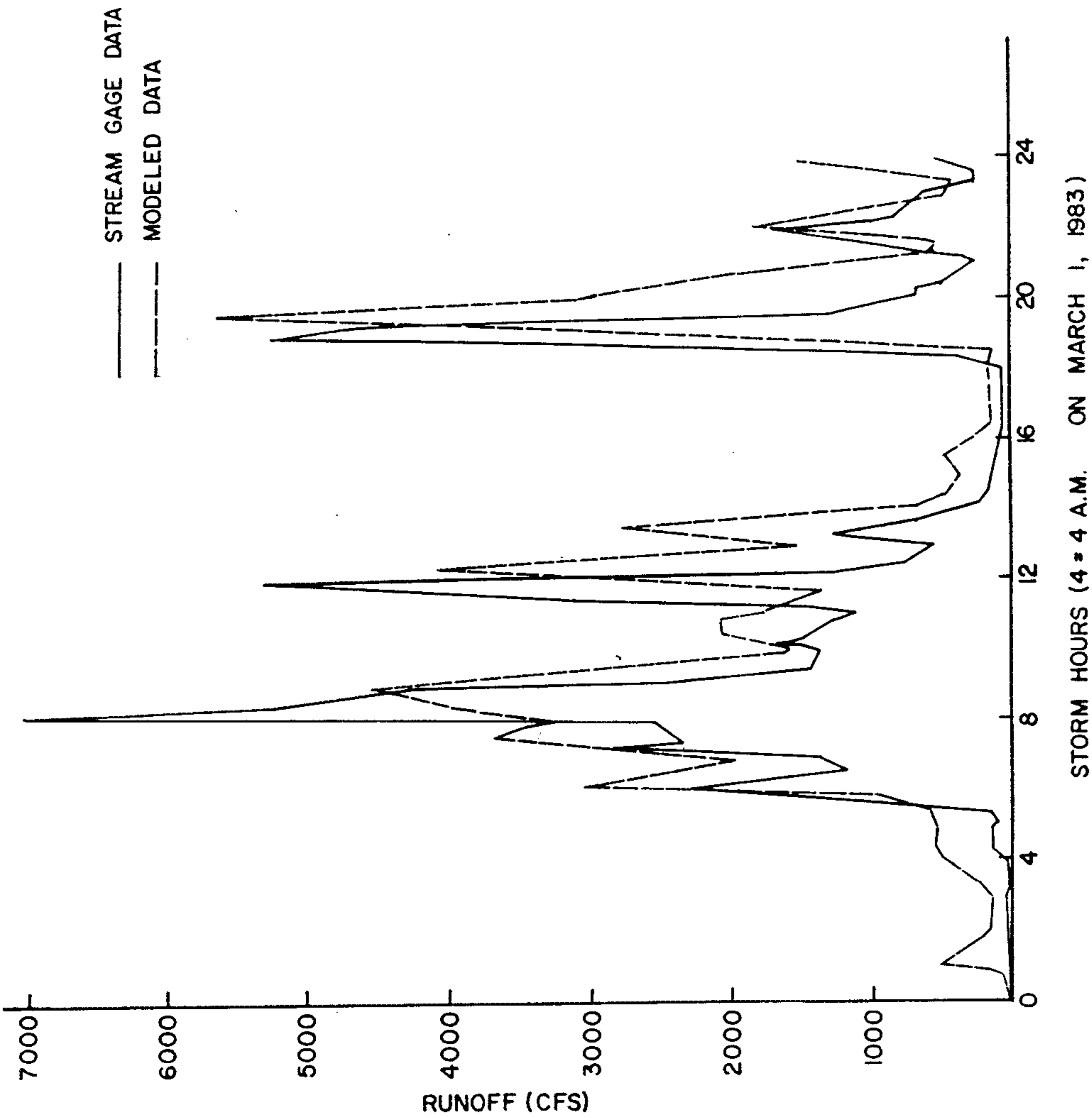
Figure 40. Area-Weighted Model Runoff Frequency Estimates (COE Depth-Area)

TABLE A.1— Preliminary flood-frequency analyses results (urbanization not accounted for).

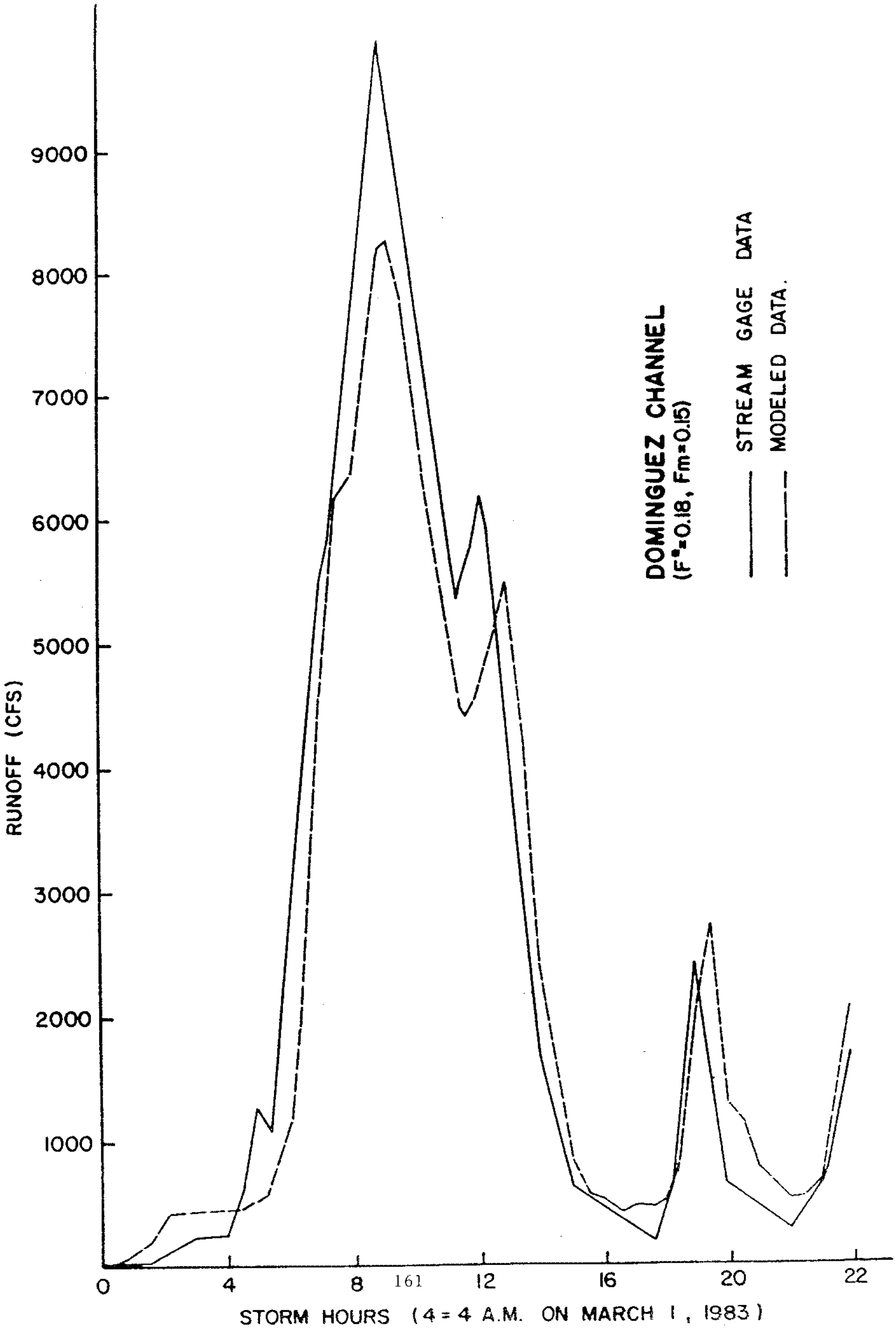
Streamgage	Years analyzed[1]	Missing peaks estimated[1]	Outliers[1]	Computed peaks (cfs) 2-year	5-year	10-year	25-year	50-year	100-year	Percentage difference in Q_{100}[2]	Q_{100} cfs/acre
Dominguez Channel at Vermont Ave.	1967-1982	--	--	6,680	10,200	12,500	15,400	17,500	19,600	--	0.83
Alhambra Wash near Klingerman St.	1930-1983	--	--	2,760	4,090	4,990	6,120	6,950	7,780	--	0.80
Rubio Wash at Glendon Way	1930-1983	--	--	1,950	2,790	3,290	3,870	4,260	4,620	--	0.66
Eaton Wash at Loftus Dr.	1957-1984	1969-1973	--	2,370	3,450	4,190	5,160	5,910	6,670	+1%	0.46
Arcadia Wash below Grand Ave.	1957-1984	1968-1973	--	1,480	2,250	2,790	3,500	4,040	4,600	+13%	0.85
Compton Creek at 120th St.	1952-1978	1974	--	2,080	3,050	3,740	4,630	5,320	6,030	+2%	0.65
Compton Creek near Greenleaf Dr.	1928-1984 missing 1938	1938 not estimated	1928, low	2,430	4,050	5,060	6,240	7,030	7,760	--	0.54
Verdugo Wash at Estelle Ave.	1929-1983	1934	1931, low	1,630	3,610	5,080	6,930	8,250	9,480	+7%	0.55

1 Water years

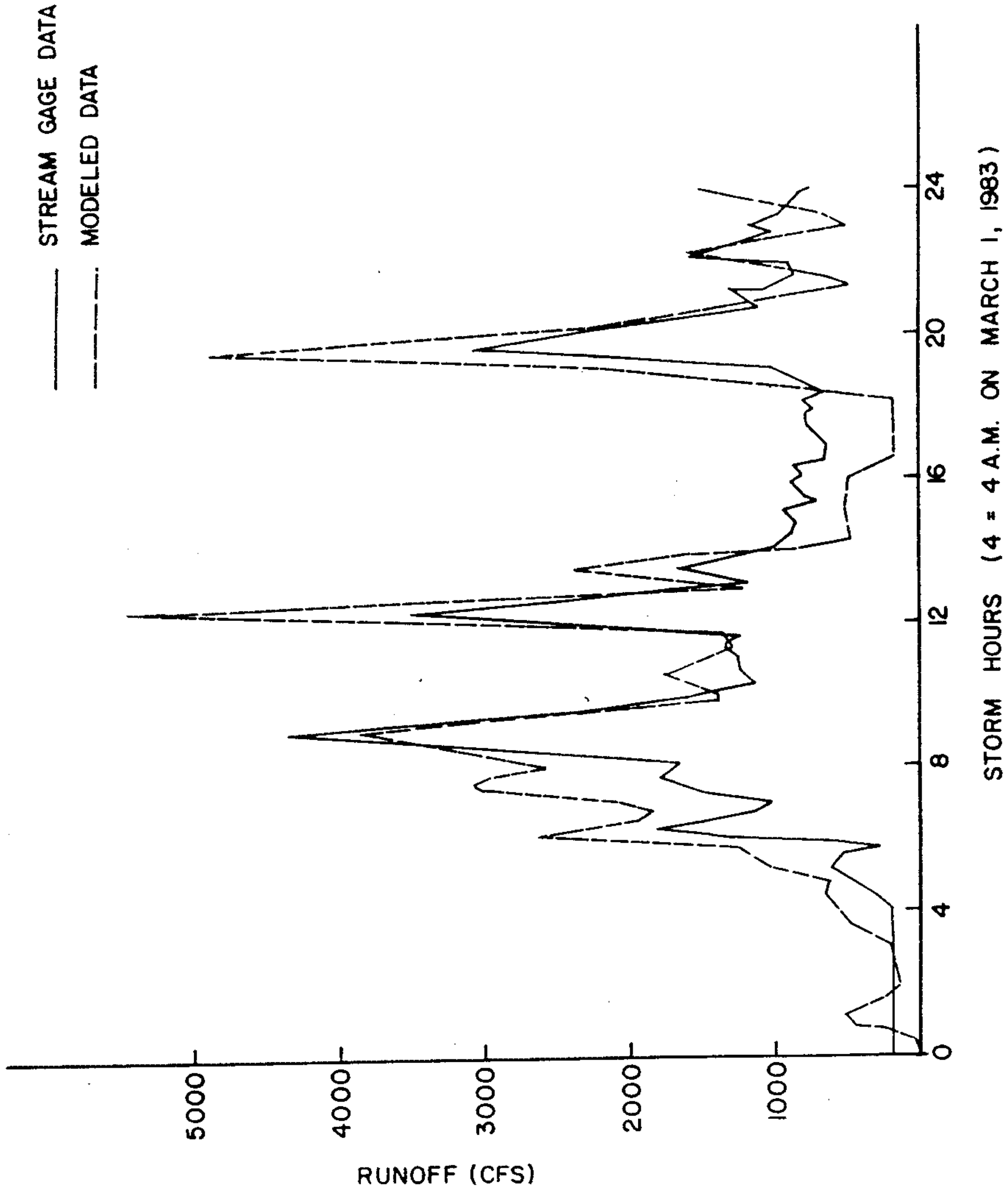
2 Percentage difference = $\frac{Q_{100}\text{ (excluding missing/adjustment)} - Q_{100}\text{ (including estimate/adjustment)}}{Q_{100}\text{ (including estimate/adjustment)}} \times 100$, Q_{100} = 100-year peak



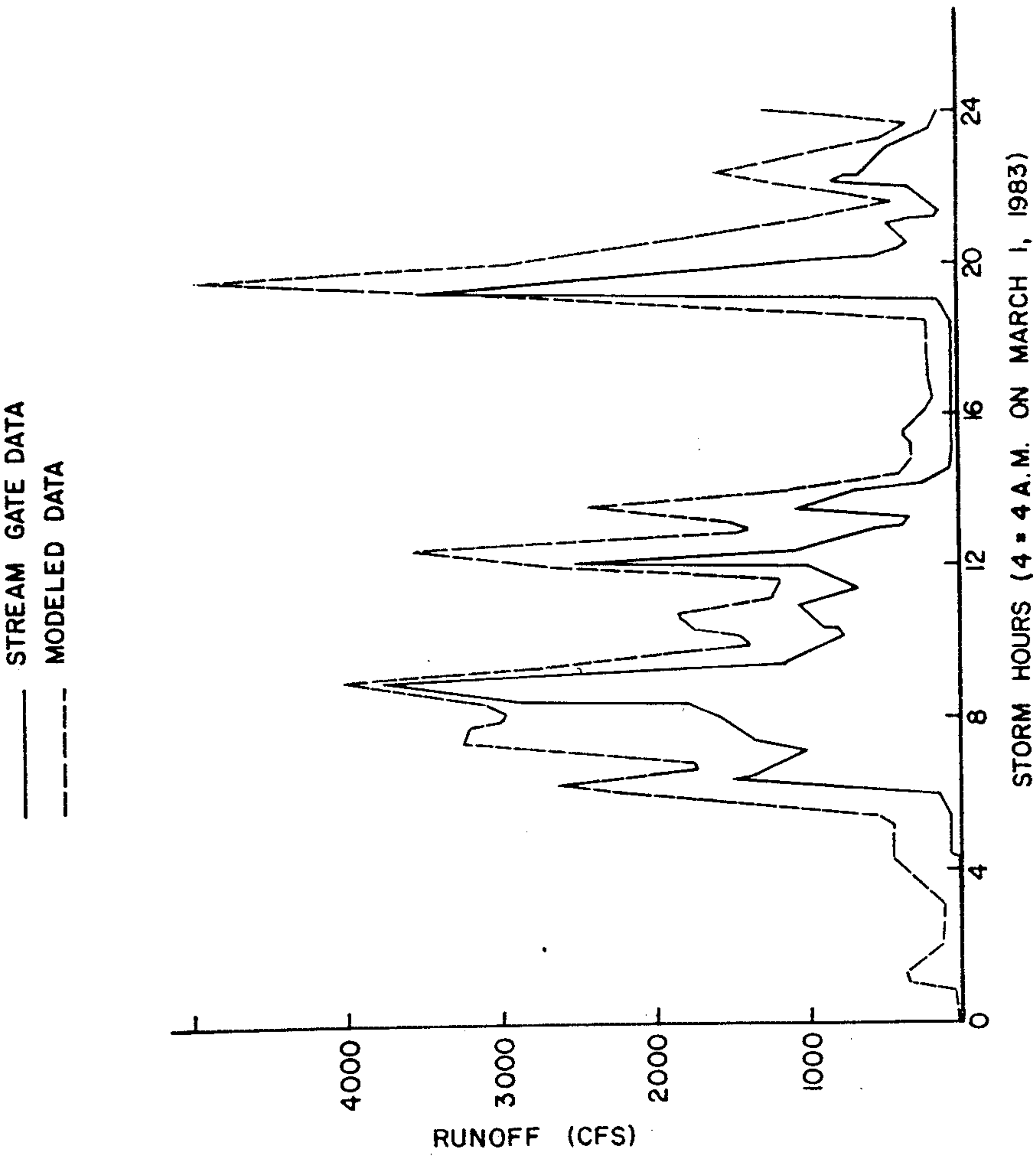
ALHAMBRA WASH
(F* =0.55, Fm = 0.15)



DOMINGUEZ CHANNEL
(F*=0.18, Fm=0.15)
STREAM GAGE DATA
MODELED DATA.
RUNOFF (CFS)
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
4
8
12
16
20
22
STORM HOURS (4 = 4 A.M. ON MARCH 1, 1983)



EATON WASH
(F*0.37, Fm=0.22)



RUBIO WASH

(F^{*}=0.60 , Fm=0.15)

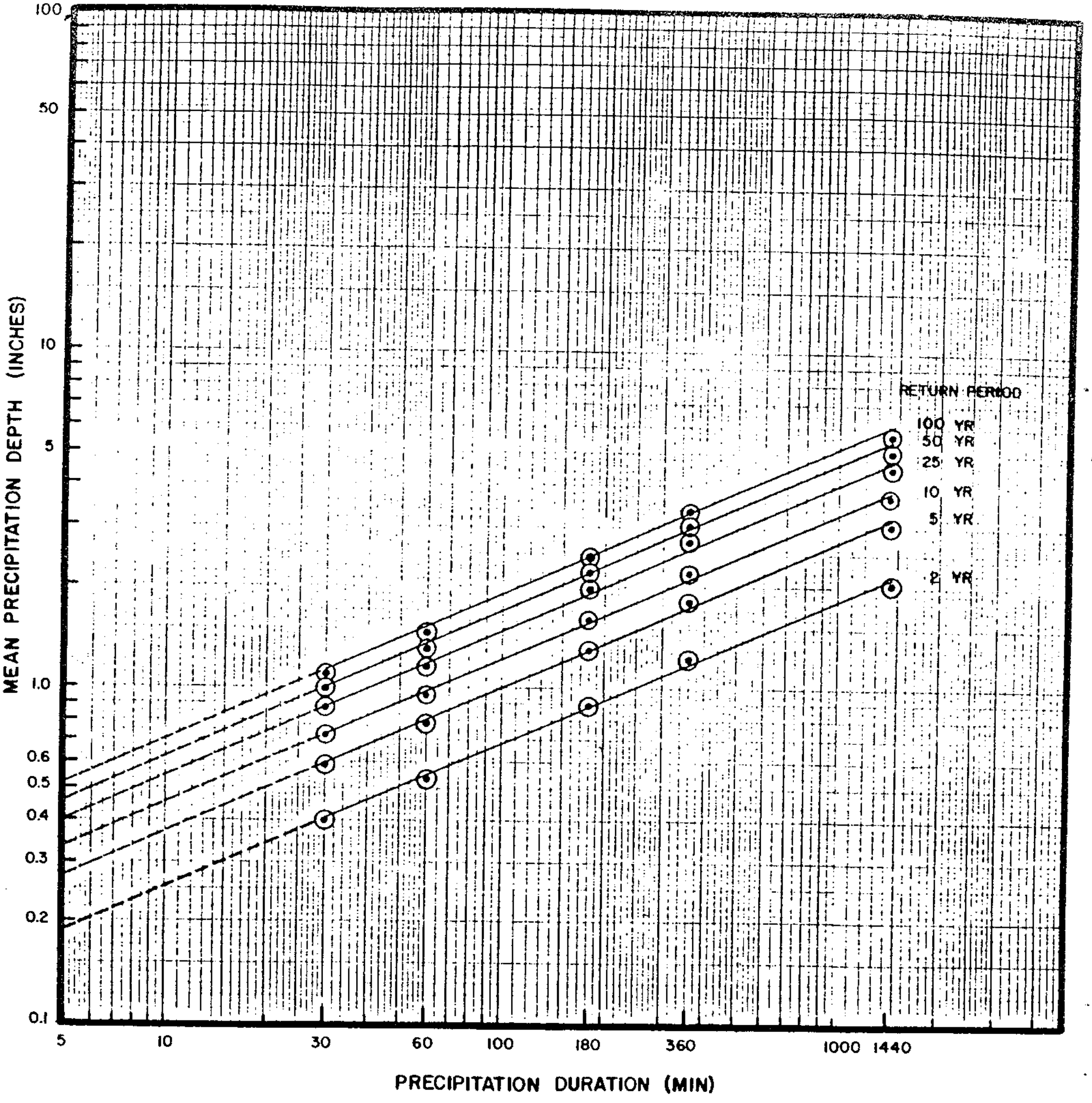


Figure E.1 Mean Depth-Duration Relationships for Orange County Non-Mountainous Areas Per DWR