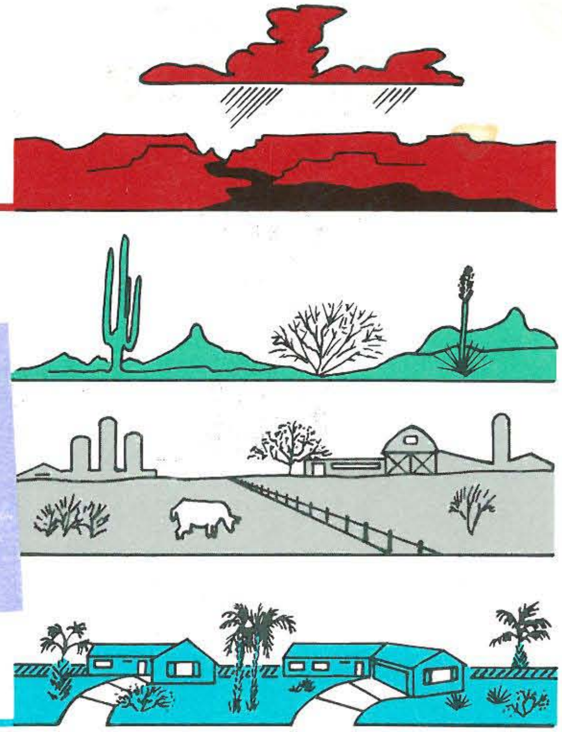

FLOOD CONTROL
DISTRICT OF
MARICOPA COUNTY

Addendum/Erratum
to

A HYDROLOGIC ANALYSIS OF THE POWERLINE F.R.S.

VOLUME I

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

/89

Addendum/Erratum
to

A HYDROLOGIC ANALYSIS OF THE POWERLINE F.R.S.

Prepared by

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Hydrology Division
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June 1989

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to

A HYDROLOGIC ANALYSIS
OF THE
POWERLINE F.R.S.

VOLUME I

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

I. INTRODUCTION

The purpose of this addendum/erratum to the Powerline F.R.S. analysis was to discuss the areal reduction procedures in further detail, to address the comments by the Soil Conservation Service (SCS), and to correct errors made in the original analysis.

This report was written in an issue and solution format in response to comments concerning the original Powerline F.R.S. analysis.

II. POWERLINE F.R.S. SPILLWAY

Two questions related to modeling of the spillways were:

Where did the stage-storage rating curve for the Powerline spillway originate and was it validated?

> The stage-storage rating curve for the Powerline F.R.S. spillways up to the elevation of 1590.1 feet amsl was taken from the 1985, SCS Weekes Wash Study (Reference 5). The values were verified against the "Area and Capacity Curve for Powerline F.R.S." found in the Flood Control District's (FCD) Structures Book (Reference 3) (See Figure D-1 of Appendix D for the curves) and found to be accurate.
>
> In the original analysis, an error was detected in the estimated storage volume for 1590.6 feet amsl, which caused an error in the estimate of the increase of height in the dam to retain the full PMF without overtopping of the structure. To correct the error, the Area & Capacity curve for Powerline F.R.S. found in the FCD's Structures Book was used to extrapolate storage volumes over the height of the structure. (See Figure D-1 in Appendix D for the rating curve).
>
> Correcting the rating curve resulted in an increase of the proposed height (5 feet) in which the structure would need to be raised in order to alleviate overtopping of the structure by the full PMF.

Why was the FCD outflow discharge rating curve different from the HEC-2 run of APP-419?

> The initial stage-discharge relationship was interpolated from the HEC-2 run (App-419) to enable the stage, storage, and discharge to be dependent relationships. To eliminate future confusion on this point the curve was changed in the HEC-1 to match the HEC-2 results. (See Volume II Addendum for the HEC-2 output).

As a point of clarification, the stage elevation was assumed to be the energy gradeline (EG) instead of the water surface elevation (WSE). The EG provided a conservative elevation for the higher outflows. (See Table 2.1 for a comparison of the EG and the WSE).

No change in the results occurred by separating the stage, storage, and discharge values.

Table 2.1

Stage-Discharge Rating Curve
Energy Gradeline vs. Water Surface Elevation
in the HEC-2 Analysis
for the Powerline F.R.S. Emergency Spillway

Discharge	Water Surface Elevation	Energy Gradeline
500	1583.89	1583.90
1000	1584.18	1584.19
5000	1585.52	1585.61
10,000	1586.63	1586.85
15,000	1587.51	1587.86
20,000	1588.30	1588.76
25,000	1588.99	1589.57
30,000	1589.63	1590.32
35,000	1590.22	1591.04
40,000	1590.79	1591.72
50,000	1591.82	1592.97
60,000	1592.79	1594.16
70,000	1593.70	1595.28
80,000	1594.55	1596.34

The larger flows caused the HEC-2 program to extend some of the cross-sections vertically to increase the capacity of the channel. Table 2.2 shows the height in feet that the levees would need to be extended to prevent the emergency spills from flowing along the toe of the structure.

Table 2.2

Powerline F.R.S. Analysis
Emergency Spillway Levee Extension

Flows (cfs)	X-Section	1	2	3	4	4.5	4.7	5	6	7
	Levee Extension (feet)									
15,000							.1			
20,000							.85			
25,000							1.5	.5	.5	.4
30,000		.3				.4	2.1	1.1	1.1	1.0
35,000		.7	.4			.9	2.7	1.7	1.7	1.6
40,000		1.1	.9	.5	.3	1.4	3.2	2.3	2.3	2.2
50,000		2.0	1.7	1.3	1.1	2.2	4.2	3.3	3.3	3.2
60,000		2.7	2.4	2.0	1.8	3.1	5.2	4.2	4.2	4.2
70,000		3.5	3.2	2.8	2.6	3.8	6.0	5.1	5.1	5.1
80,000		4.2	3.9	3.5	3.3	4.6	6.9	5.9	6.0	6.0

III. PRECIPITATION

3.1 100 Year Precipitation

In-house discussions lead to the direction to use areal reduction in the Powerline analysis:

> The decision was made to incorporate areal reduction using the NOAA Technical Memorandum NWS HYDRO-40: Depth-Area Ratios in the Semi-Arid Southwest United States (Reference 7). Independent determinations for the values were conducted to determine accurate ratios. The depth-area ratios, the related areas, and the precipitation depths for the Powerline F.R.S. analysis are listed below in Table 3.1. (See Figure C-1 in Appendix C for the Depth-Ratio Curve).

Table 3.1

Depth-Area Ratios for Central Arizona

Area (sq. mi.)	Ratio	Precipitation Depth (inches)
0.01	1.00	3.85
10	.95	3.72
20	.92	3.63
30	.90	3.53
40	.89	3.42
50	.88	3.39

By using the depth-area ratios extracted from HYDRO-40 in the simulation, the peak inflow to Powerline F.R.S. was just under 11,800 cfs compared to the peak inflow of 14,310 cfs for no areal reduction. The peak outflow for the 100 year, 24 hour storm using areal reduction was 470 cfs with a peak elevation of 1583.8 feet amsl. With no areal reduction the peak outflow for the 100 year, 24 hour storm was 1610 cfs with a peak elevation of 1584.4 feet amsl.

Areal reduction as described in HYDRO-40 resulted in a decrease of approximately 1140 cfs of the outflow from the Powerline F.R.S..

What was the affect of the emergency spills during the 100 year event on areas downstream of the Powerline F.R.S.?

> The 100-year storm with areal reduction only produced a peak outflow of 470 cfs. The Powerline F.R.S. was assumed to protect the downstream area from the 100-year event. Although there were emergency spills during this event, the Powerline Floodway, directly downstream of the Powerline Emergency Spillway, has the capacity to accept all of the flows from the principal and emergency spillways.
>
> The Vineyard F.R.S. and the Rittenhouse F.R.S. also drain into the Powerline Floodway. It was assumed that if there was a single storm over the three watersheds that areal reduction of the storm would be significant enough to not cause a problem at the Powerline Floodway.

3.2 Probable Maximum Precipitation

The Probable Maximum Precipitation (PMP) for the 72 hour and the 6 hour durations were determined incorrectly.

> The PMPs were re-computed indicating the total PMP of 17.91 inches for the 72 hour duration and 10.91 inches for the 6 hour duration. See Appendix B for the graphs and tables used to determine the General and Local PMP (Reference 6).
>
> The PMP 6-hour duration and the PMP 72-hour duration storms were compared to determine the storm that produced the higher peak stage at the structure. The 6-hour storm produced a peak stage of 1590.4 feet amsl while the 72-hour storm only produced a peak stage of 1589.7 feet amsl. Since the 6-hour storm produced the higher peak stage, it was used for our analysis.
>
> Following ADWR's Dam Safety criteria (Reference 1), the design flood magnitude for the Powerline F.R.S. was the 1/2 Probable Maximum Flood (PMF), which was generated from 1/2 the volume produced from the PMP (6-hour duration). The peak stage for the 1/2 PMF was 1588.7 feet amsl.
>
> By correcting the PMP total rainfall values the 72 hour duration storm increased in peak inflow, peak stage, and peak outflow at the Powerline F.R.S.. The effect of decreasing the total inches for the 6 hour duration storm was a decrease in the peak inflow, peak stage, and peak outflow at the Powerline F.R.S.. The 6 hour duration storm created higher peaks than the 72 hour duration storm, therefore the 6 hour duration storm will be the design storm.

What affect would future downstream growth and land use conditions have on the inflow design flood magnitude?

The peak stage for the full PMF under current modeling conditions was 1590.4 feet amsl and resulted in 1.3 feet of flows overtopping the structure.

Future downstream land use conditions would require the design flood magnitude to increase to the full PMF. An analysis was conducted to determine the additional height needed to allow the structure to comply with the Dam Safety Criteria for future land use. Our analysis indicated that an additional 5 feet of height was required to contain the full PMF. The value is only an estimate because we did not take into account the possibility of building an emergency spillway at the northwest end of the structure to provide for increased flows. This kind of analysis would need to be part of a more detailed study if and when the decision was made to modify the structure.

IV. OTHER COMMENTS

The SCS included a comment about the Weekes Wash Breakout at Junction Drive in their letter to the FCD (See Appendix A, SCS letter comment #2). They recommended the conservative approach, and assume that the total flow from Weekes Wash be included in the design of the Powerline F.R.S.

In our analysis, we do have the 100 year, 24 hour flows being contained in the road crossing and eventually reaching the structure. We feel that this was a conservative assumption, because most wash road crossings, by Maricopa County Highway Department standards, are only sized for the 50 year peak event. Thus, there theoretically could be some flows breaking out of the Powerline watershed during the 100 year event.

To assume that all the flows at Junction Drive would remain in the Powerline watershed during the PMF would be unreasonable. Therefore, the flows from the PMF were divided such that all the flows from the 100 year, 24 hour storm remained in the Powerline watershed, and those flows above the 100 year event would breakout at the same rate as occurred before the modification to the road crossing.

The SCS suggested that the curve numbers be adjusted upward for shorter duration storms based on Donald E. Woodward's paper entitled, "Runoff Curve Numbers for Semi-arid Range and Forest Conditions" (Reference 8).

Although the paper showed an increase in curve numbers for shorter durations in any given event, there did not appear to be a general trend for different events with the same frequency. There was no information about the trend of curve numbers for different events and different durations. (See Table F-1 in Appendix F for the Summary Table of curve numbers).

To address the SCS comment on increasing the curve numbers for the shorter duration storm (although the event was much larger) we increased the curve numbers. Woodward's paper included a graph which related curve numbers to durations. This graph was used as an approximate value of increase. (See Figure F-1 in Appendix F for the graph).

The increase in curve numbers did not significantly increase the peak inflow, peak stage, peak storage, or peak outflow at Powerline F.R.S.. See Table 4.1, Plan 3 for the values. Since there was not a significant change in the peak stage, Plan 3 was not used for planning purposes.

The SCS commented on what appeared to be a typographical error on Page 31 of the original report. The phrase was "...raise the design flood magnitude to 1/2 PMF - PMF".

Actually, this was what was actually written in the Dam Safety Guidelines. Since the design flood magnitudes are only guidelines, Dam Safety can chose the value to be used. In this case the design flood could range from the 1/2 PMF up to and including the full PMF. In our analysis, because of the eventual great potential for losses in the downstream area, we assumed the worst case situation and modeled the future design flood magnitude as the full PMF.

Table 4.1

Powerline F.R.S. Analysis
Comparison of Plans

Storm	Plan	Peak Inflow (cfs)		Peak Stage (feet msl)		Peak Storage (acre-feet)		Peak Outflow (cfs)	
100-year 24-hour*	1		11,770		1583.8		4467		467
PMP 72-Hour	1	19,680	38,320	1588.3	1589.7	6840	7660	17,350	38,225
	2	19,680	38,320	1588.3	1591.3	6840	8660	17,350	36,940
		Half	Full	Half	Full	Half	Full	Half	Full
PMP 6-Hour	1	40,900	80,830	1588.7	1590.4	7110	8140	19,670	76,330
	2	40,900	80,830	1588.7	1593.7	7110	10,130	19,670	56,270
	3	42,730	84,175	1589.0	1590.5	7260	8180	21,290	80,160
	4	42,730	84,175	1589.0	1594.1	7260	10,350	21,310	59,415

* Aerial Reduction Used

Plans

1 - Orginal Analysis
2 - Powerline F.R.S. Raised by 5.0 feet
3 - Original Analysis with Increased Curve Numbers
4 - Increased Curved Numbers and the Powerline F.R.S. Raised by 5.0 feet

V. CONCLUSIONS

Further analysis of the Powerline F.R.S. did not substantially change the conclusions from the original report.

The Powerline F.R.S. was designed to protect the downstream areas from the 100-year peak runoff event, which was the 24-hour duration storm. Although there are emergency spills during this flood, the Powerline Floodway has the capacity to contain the flows and alleviate flooding downstream.

The recommended spillway design flood for the current downstream land use, using future land use conditions upstream was the 1/2 Probable Maximum Flood. The analysis showed that the peak stage for the 1/2 PMF was 1588.7 feet amsl which does not exceed the top of dam, an elevation of 1589.1 feet amsl.

According to downstream land use trends, the area will develop rapidly within the next ten years. This condition would change the hazard category according to Dam Safety Criteria to "High", which in turn would raise the design flood magnitude to the full PMF. We recommend that the full PMF be used as the design flood magnitude for the Powerline F.R.S. to allow for the maximum protection of future downstream development. The Flood Control District currently has the financial ability to bring Powerline F.R.S. to future dam safety standards. It is not known if in five to ten years from now we will still be in the same financial situation.

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1. Flood Control District of Maricopa County, Data on Structures, November 1985.

2. Lawrence, Dan Roger, Guidelines for the Determination of Spillway Capacity Requirements, Arizona State Department of Water Resources, Division of Dam Safety, February, 1985

3. U.S. Arny Corps of Engineers, Hydrologic Engineering Center, HEC-1 Hydrograph Package, Revised February 6, 1987, Davis, California.

4. U.S. Army Corps of Engineers, Hydrologic Engineering Center, HEC-2 Water Surface Profiles, Version of September 1988, Davis, California.

5. U.S. Department of Agriculture, Soil Conservation Service, Weekes Wash - Powerline Dam Analysis, November 1985.

6. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, Hydrometeorological Report No. 49 - Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages, 1977.

7. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, NOAA Technical Memorandum NWS HYDRO-40, Depth-Area Ratios in the Semi-Arid Southwest United States, Office of Hydrology Silver Spring, MD., August 1984.

8. Woodward, Donald E., Runoff Curve Numbers for Semiarid Range and Forest Conditions, Regional Technical Service Center, Soil Conservation Service, U.S. Department of Agriculture, Portland Oregan, June 1973.

APPENDIX A

SCS LETTER

United States Department of Agriculture

Soil Conservation Service

201 E. Indianola Ave.
Suite 200
Phoenix, AZ 85012

April 18, 1989

Dan E. Sagramoso, P.E.
Maricopa Co. Flood Control District
3335 West Durango St.
Phoenix, Arizona 85009

2

Dear Mr. Sagramoso:

We have reviewed the FCD's "Hydrologic Analysis of the Powerline FRS", and found the report to be well written and easy to follow. A thorough analysis of the Powerline FRS has been made by your staff, and conclusions appear to be reasonable, except as noted below:

1. Page 9, Table 2.2, the aerial reduction ratios listed under the "Osburn method" appear to be too low for the 24-Hour Storm. We reference Figure 15, page 29 of "NOAA Technical Memorandum NWS HYDRO-4., Depth-Area Ratios in the Semi-Arid Southwest United States."

2. Page 14, "Weeks Wash Breakout at Junction Drive." We recognize that the breakout is an existing condition, but can this be assured under all future conditions. We would recommend the conservative approach, and assume that the total flow from Weeks Wash is included in the design of the Powerline FRS.

3. Page 17, Curve Number Comparison, Table 4.1. The curve numbers shown are for the 24-hour duration storms. We understand that these were also used for the PMP and 1/2 PMP storms, which are 6-hour durations. We suggest the CN's be adjusted upward for the shorter duration storms based on Donald E. Woodward's paper entitled, "Runoff Curve Numbers for Semiarid Range and Forest Conditions" (See attached copy).

4. Page 23, Table 4.2, FCD outflow discharge is different than HEC-2 run of APP-419.

5. Page 27, Comparison of Alternatives, Table 6.1. In Comparing Alternatives G and B, it does not appear reasonable that the "Peak Flood Reservoir Water Surface Elevation" would rise only 0.02 of a foot (1590.47 to 1590.49) when the outflow discharge is decreased from 76,200 CFS to 36,500 CFS. We would suggest that the

calculations and assumptions be checked for these two runs.

6. Page 31, there appears to be a typo in sentence two of the second paragraph from the bottom, the phrase:"....raise the design flood magnitude to 1/2 PMF - PMF" should read "....raise the design flood magnitude from 1/2 PMF to PMF."

7. Page 32, we agree with the suggestion for a meeting between the SCS, ADWR and MCFCD to clarify the design requirements, and to discuss possible alternatives to correct the deficiencies in the Powerline FRS.

We are looking forward to continue working with you and your staff on this structure.

Sincerely,

CHARLES R. ADAMS
State Conservationist

Enclosure 1

cc:
Ralph Arrington, SCE, SCS, Phoenix, AZ
Barton E. Ambrose, ASTC (P), SCS, Phoenix, AZ

APPENDIX B

PROBABLE MAXIMUM PRECIPITATION CALCULATIONS

APPENDIX B-I

PMP - GENERAL STORM

Table B-1
General-Storm PMP Computations

Table 6.1.--General-storm PMP computations for the Colorado River and Great basin

Drainage Power Line Dam Area 47.1 mi^2 (km^2)

Latitude 33°26', Longitude 111°30' of basin center

Month Sept.

Step		6	12	18	24	48	72	
A. Convergence PMP								
1. Drainage average value from one of figures 2.5 to 2.16	13.9 in. (mm)							
2. Reduction for barrier-elevation [fig. 2.18]	80 %							
3. Barrier-elevation reduced PMP [step 1 X step 2]	11.12 in. (mm)							
4. Durational variation [figs. 2.25 to 2.27 and table 2.7].		76	90	76	100	111	115	%
5. Convergence PMP for indicated durations [steps 3 X 4]		8.45	10.0	10.68	11.12	12.34	12.79	in. (mm)
6. Incremental 10 mi^2 (26 km^2) PMP [successive subtraction in step 5]		8.45	1.55	.68	.44	1.22	.45	in. (mm)
7. Areal reduction [select from figs. 2.28 and 2.29]		89	97	98	99	100	100	%
8. Areally reduced PMP [step 6 X step 7]		7.52	1.50	.67	.44	1.22	.45	in. (mm)
9. Drainage average PMP [accumulated values of step 8]		7.52	9.02	9.69	10.13	11.35	16.80	in. (mm)
B. Orographic PMP								
1. Drainage average orographic index from figure 3.11a to d.	(REVISED) 4.0 in. (mm)							
2. Areal reduction [figure 3.20]	96 %							
3. Adjustment for month [one of figs. 3.12 to 3.17]	100 %							
4. Areally and seasonally adjusted PMP [steps 1 X 2 X 3]	3.84 in. (mm)							
5. Durational variation [table ~~3.6~~ 3.9]		36	63	84	100	140	159	%
6. Orographic PMP for given durations [steps 4 X 5]		1.38	2.42	3.23	3.84	5.38	6.11	in. (mm)
C. Total PMP								
1. Add steps A9 and B6		8.9	11.44	12.92	13.97	16.73	17.91	in. (mm)

2. PMP for other durations from smooth curve fitted to plot of computed data.

3. Comparison with local-storm PMP (see sec. 6.3).

Figure B-1
1000mb 24 hour Convergence PMP
for September

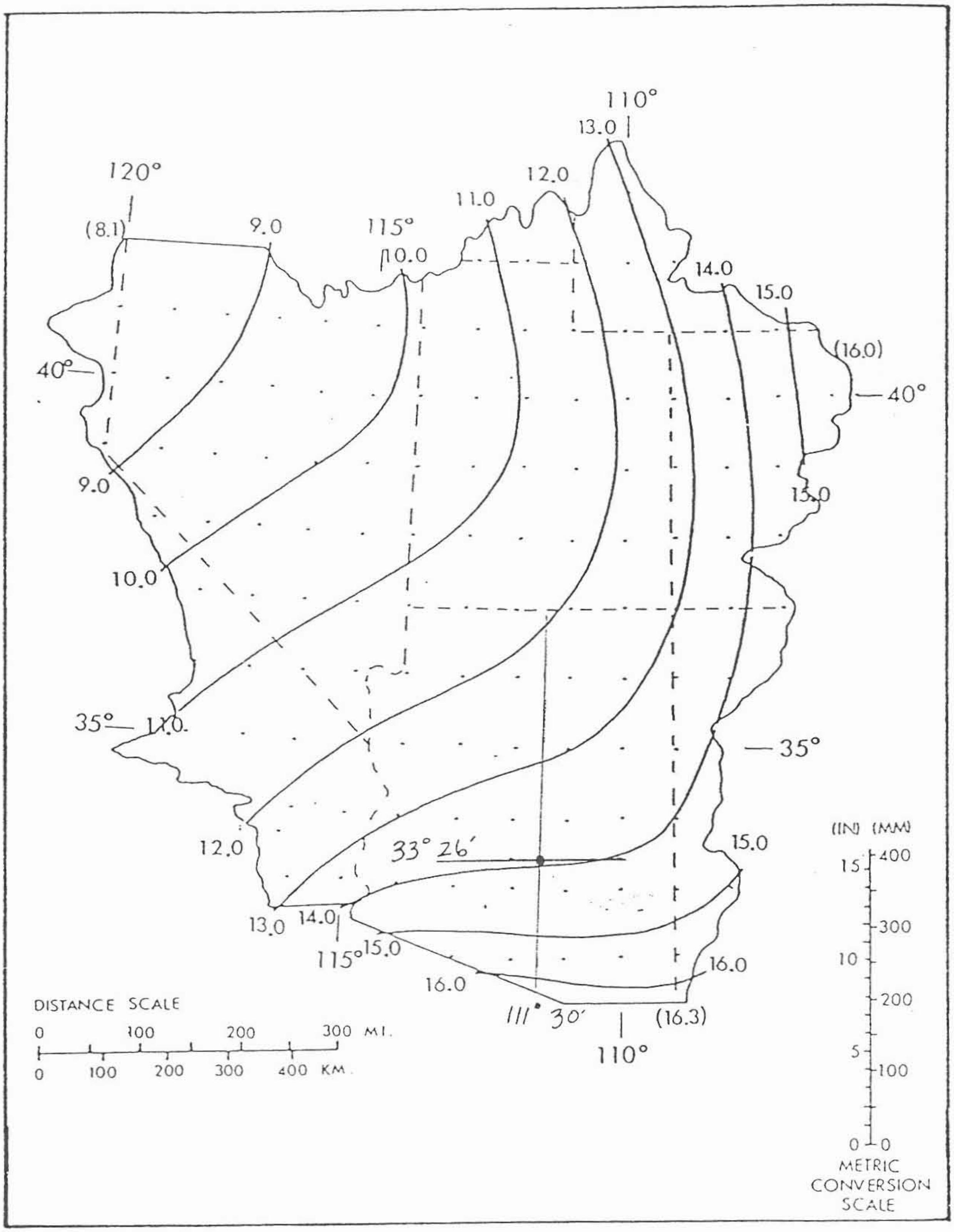


Figure 2.13.--1000-mb (100-kPa) 24-hr convergence PMP (inches) for 10 mi² (26 km²) for September. Values in parentheses are limiting values and are to facilitate extrapolation beyond the indicated gradient.

Figure B-2
Percent of 1000mb Convergence PMP

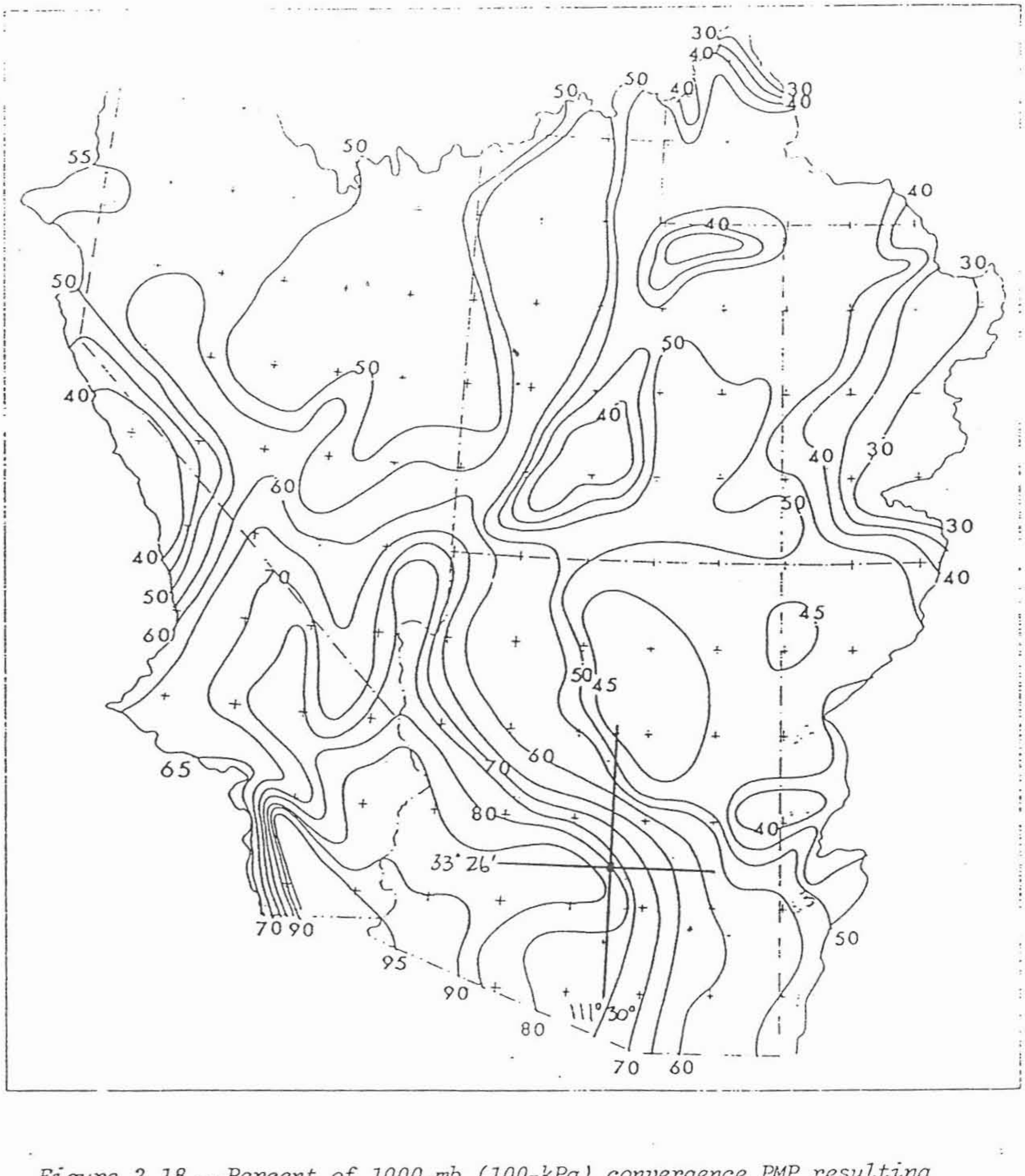


Figure 2.18.--Percent of 1000-mb (100-kPa) convergence PMP resulting from effective elevation and barrier considerations. Isolines drawn for every five percent.

Figure B-3
Regional Variation of 6/24-hr ratios

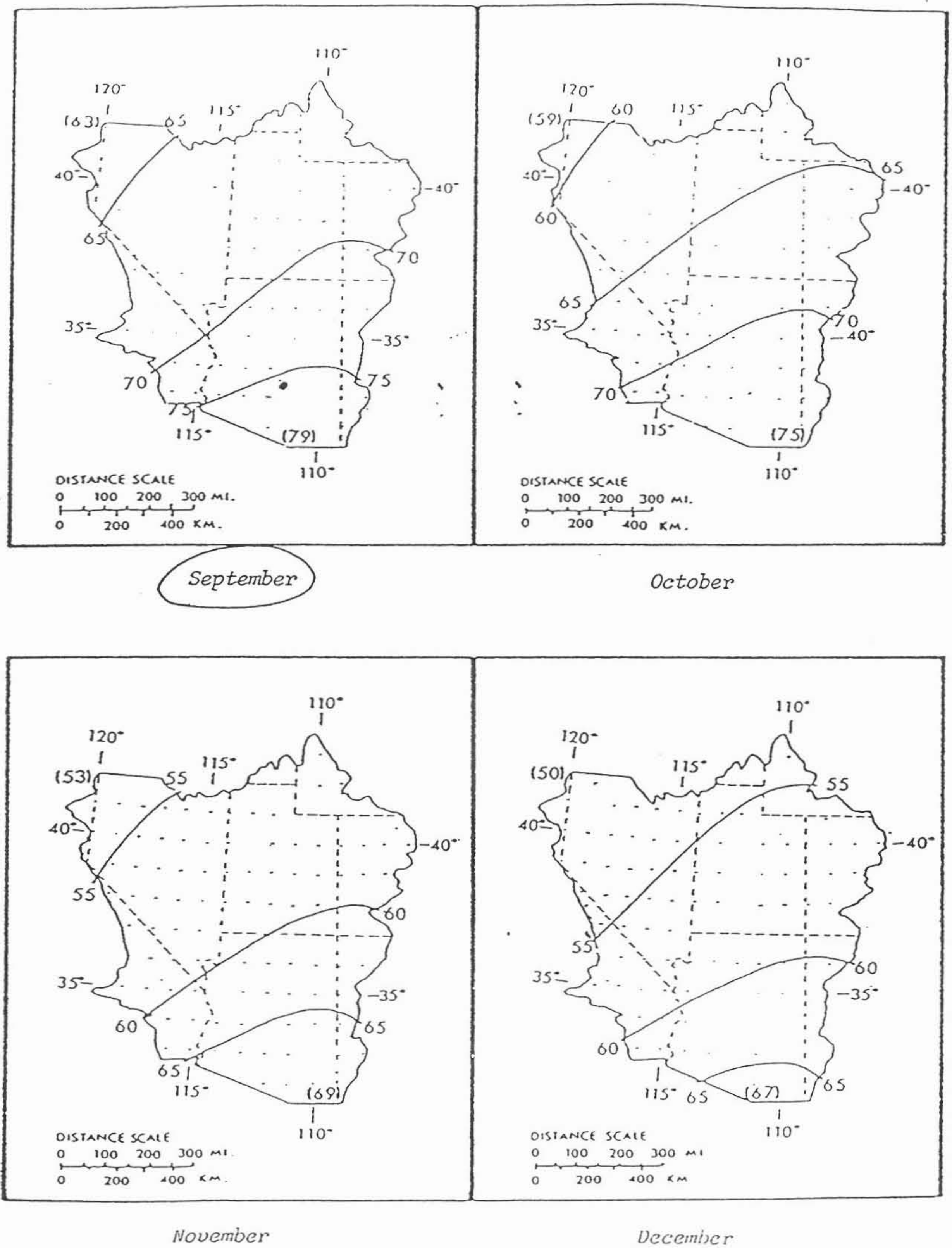


Figure 2.27.--Regional variation of 6/24-hr ratios by month (percent). Values in parentheses are limiting values and are to facilitate extrapolation beyond the indicated gradient.

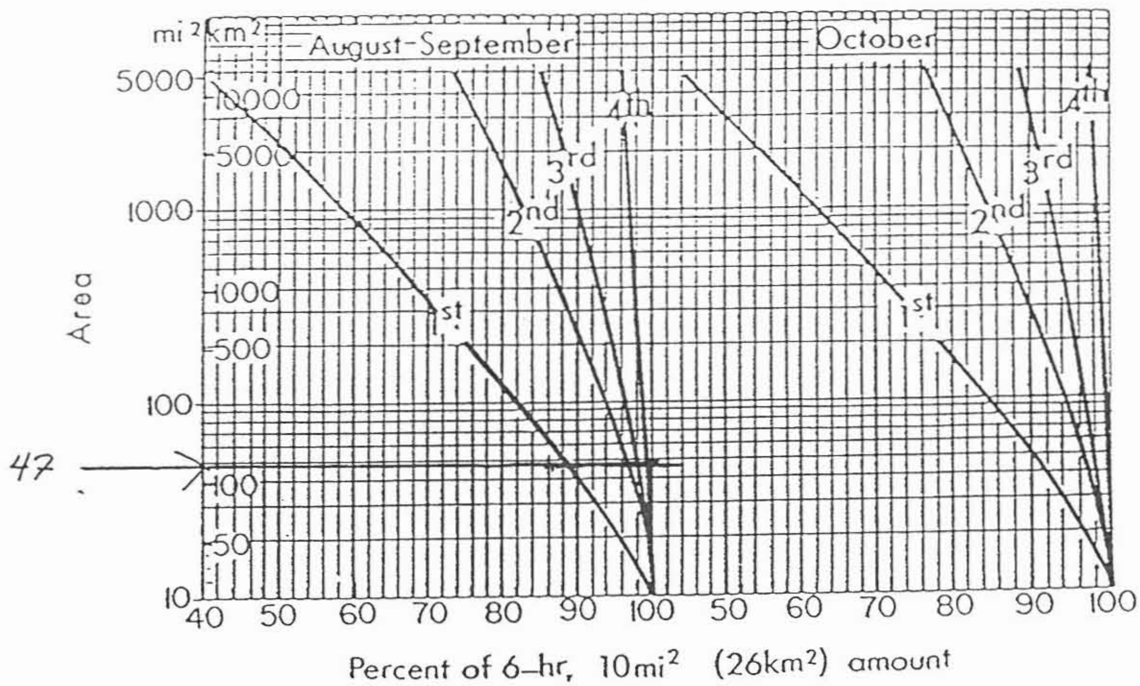
Table B-2
Durational Variation of Convergence PMP

Table 2.7.--Durational variation of convergence PMP (in percent of 24-hr amount).

Duration (Hrs)					
6	12	18	24	48	72
50	76	90	100	129	150
51	77	90	100	128	148
52	77	90	100	127	146
53	77	91	100	127	144
54	78	91	100	126	142
55	78	91	100	125	140
56	79	91	100	124	138
57	79	92	100	123	137
58	80	92	100	122	135
59	80	92	100	121	134
60	81	92	100	120	132
61	81	92	100	120	131
62	82	93	100	119	129
63	82	93	100	118	128
64	83	93	100	117	126
65	84	93	100	117	125
66	84	93	100	116	124
67	85	94	100	116	123
68	85	94	100	115	122
69	86	94	100	115	121
70	87	94	100	114	120
71	87	95	100	114	119
72	88	95	100	113	118
73	88	95	100	113	118
74	89	95	100	112	117
75	89	96	100	112	116
76	90	96	100	111	115
77	90	96	100	110	114
78	91	96	100	110	114
79	92	97	100	109	113
80	92	97	100	109	113

Note: For use, enter first column (6 hr) with 6/24-hr ratio from figures 2.25 to 2.27.

Figure B-4
Depth-Area Variation for Convergence PMP



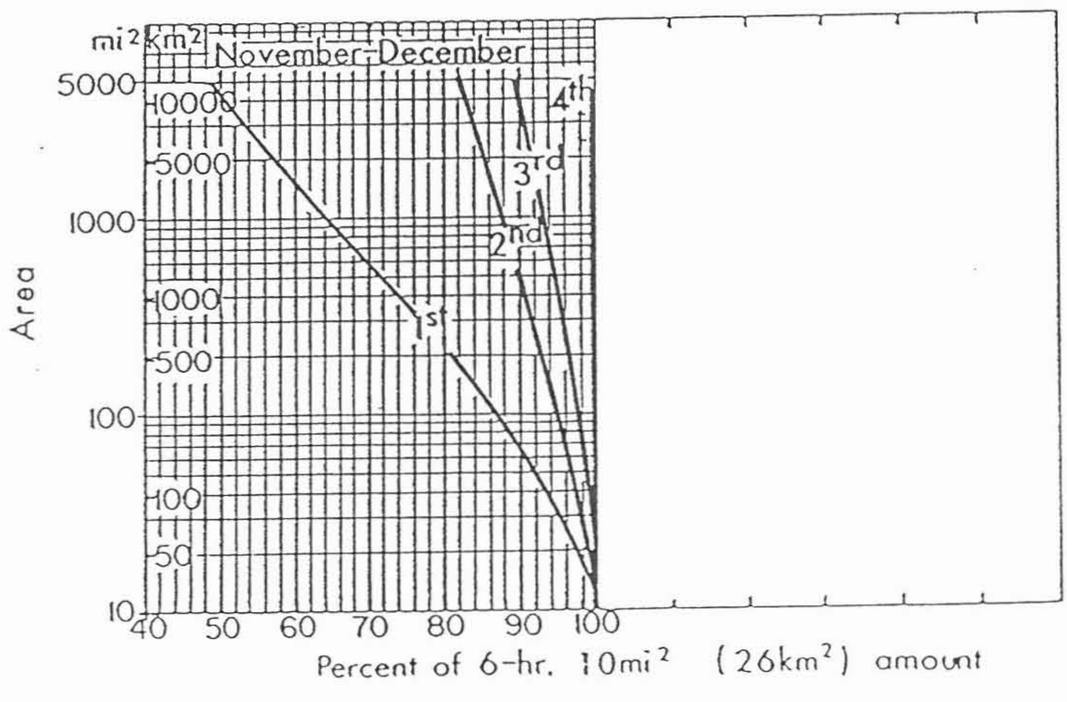


Figure 2.29.--Depth-area variation for convergence PMP for first to fourth 6-hr increments.

Figure B-5
24-hr Orographic PMP - Index Map

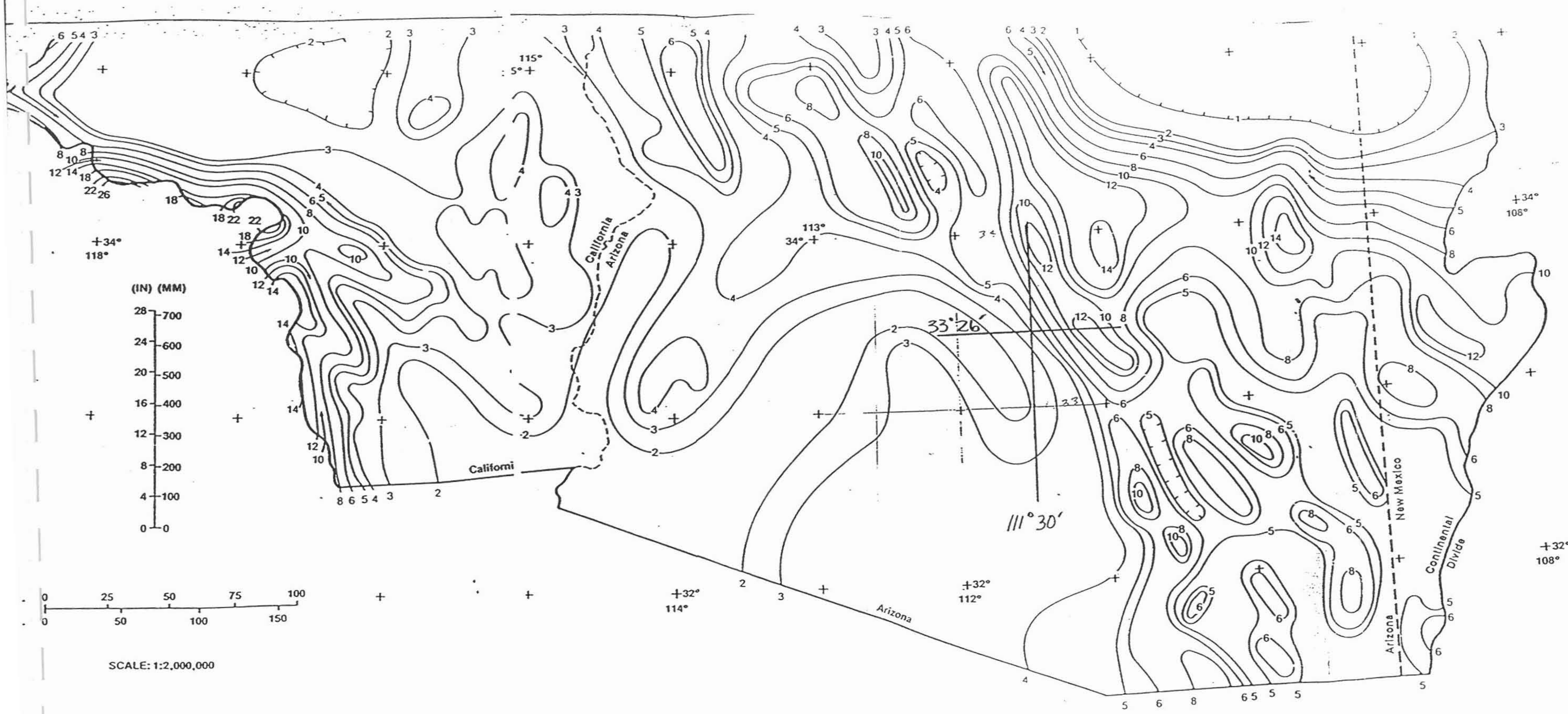


FIGURE 3.11d (Revised) — 10-mi² (26-km²) 24-hr orographic PMP index map (inches), southern sectio

Figure B-6
Variation of Orographic PMP with Basin Size

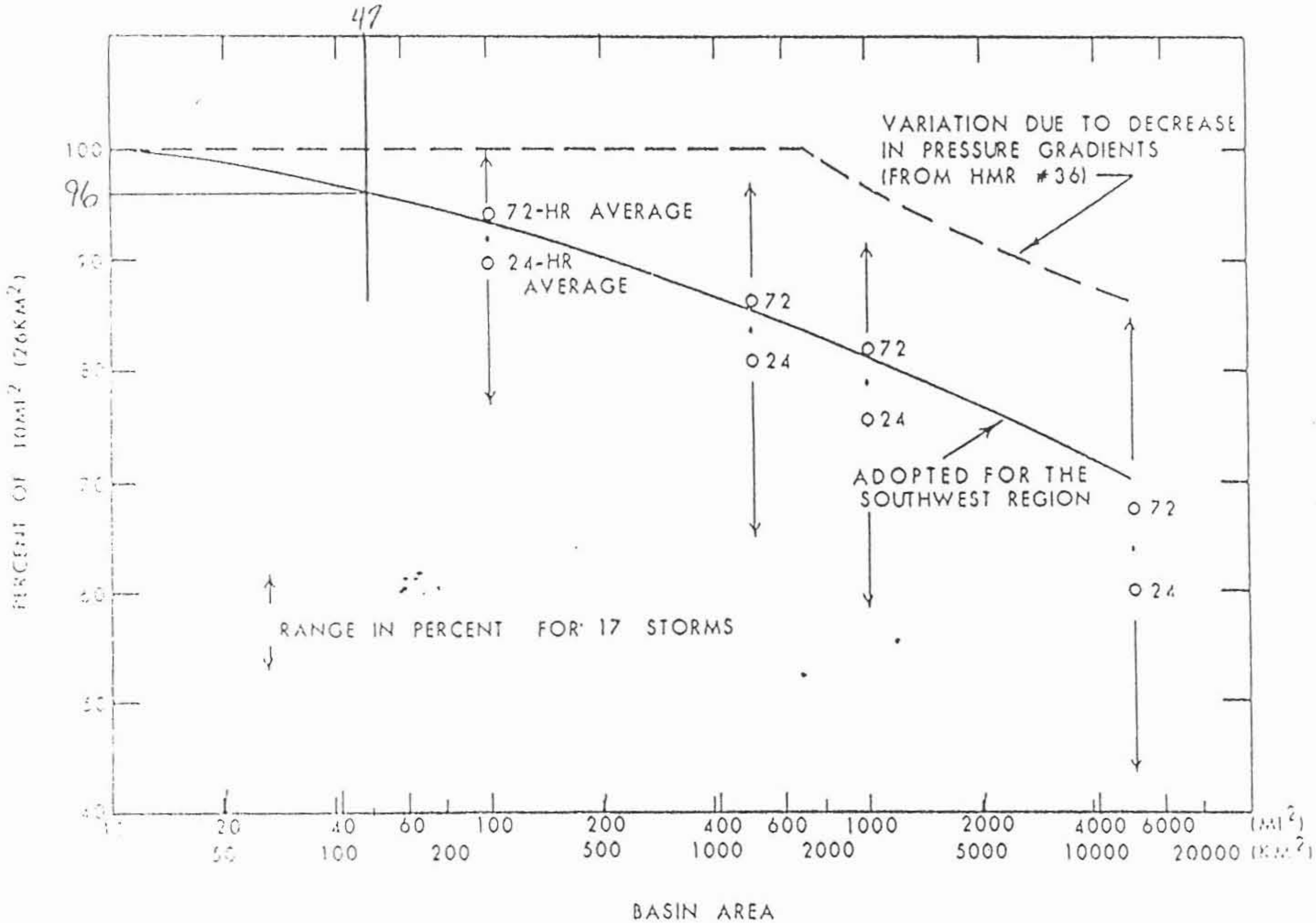
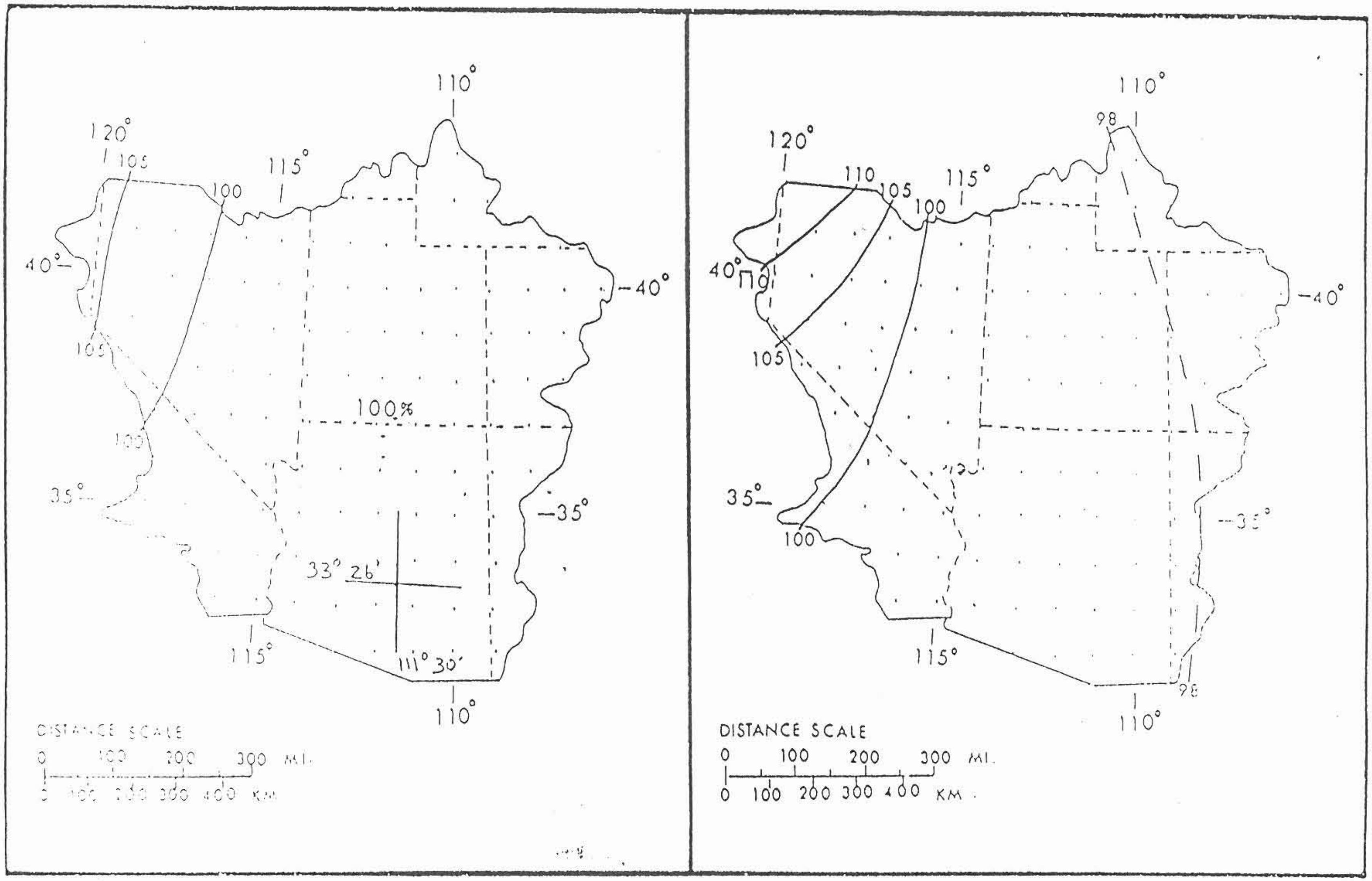


Figure 3.20.--Variation of orographic PMP with basin size.

Figure B-7
Seasonal Variation



September *October*

Figure 3.16.--Seasonal variation in 10-mi^2 (26-km^2) 24-hr orographic PMP for the study region (in percent of values in figure 3.11).

Table B-3
Durational Variation of Orographic PMP

Table 3.9.--Durational variation of orographic PMP

Latitude °N	Percent of 24-hr value					
	6 hr	12	18	24	48	72
42	28	55	79	100	161	190
41	29	56	79	100	160	189
40	30	57	80	100	159	187
39	30	57	80	100	157	185
38	31	58	81	100	155	182
37	32	59	81	100	152	177
36	33	60	82	100	149	172
35	34	61	82	100	146	167
34	35	62	83	100	143	162
→ 33	36	63	84	100	139	157
32	37	64	84	100	135	152
31	39	66	85	100	132	146
33.43	36	63	84	100	141	159

APPENDIX B-II

PMP - LOCAL STORM

Table B-4
Local-Storm PMP Computations

Table 6.3A.--Local-storm PMP computation, Colorado River, Great Basin and California drainages. For drainage average depth PMP. Go to table 6.3B if areal variation is required.

Drainage Powerline F.R.S. Area 47.1 mi^2 (km^2)
Latitude 33°2 ' Longitude 111°30 Minimum Elevation ____ ft (m)

Steps correspond to those in sec. 6.3A.

1. Average 1-hr 1-mi^2 (2.6-km^2) PMP for drainage [fig. 4.5]. 11.5 in. (mm)

2. a. Reduction for elevation. [No adjustment for elevations up to 5,000 feet (1,524 m): 5% decrease per 1,000 feet (305 m) above 5,000 feet (1,524 m)]. 100 %

   b. Multiply step 1 by step 2a. 11.5 in. (mm)

3. Average 6/1-hr ratio for drainage [fig. 4.7]. 1.30

	Duration (hr)									
	1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio of step 3 [table 4.4].	74	89	95	100	114	121	125	128	130	%
5. 1-mi^2 (2.6-km^2) PMP for indicated durations [step 2b X step 4].	8.51	10.24	10.93	11.5	13.11	13.92	14.38	14.72	14.95	in. (mm)
6. Areal reduction [fig. 4.9].	48	56	58	61	66	68	71	72	73	%
7. Areal reduced PMP [steps 5 X 6].	4.08	5.73	6.34	7.02	8.65	9.47	10.20	10.60	10.91	in. (mm)

8. Incremental PMP [successive subtraction in step 7]. 7.02 1.63 .82 .73 .40 .31 in. (mm)

4.08 1.65 .61 .68 } 15-min. increments

9. Time sequence of incremental PMP according to:

Hourly increments [table 4.7]. .40 .73 7.02 1.63 .82 .31 in. (mm)

Four largest 15-min. increments [table 4.8]. 4.08 1.65 .68 .61 in. (mm)

Figure B-8
Local-Storm PMP for 1 mi.^{2} , 1 hr

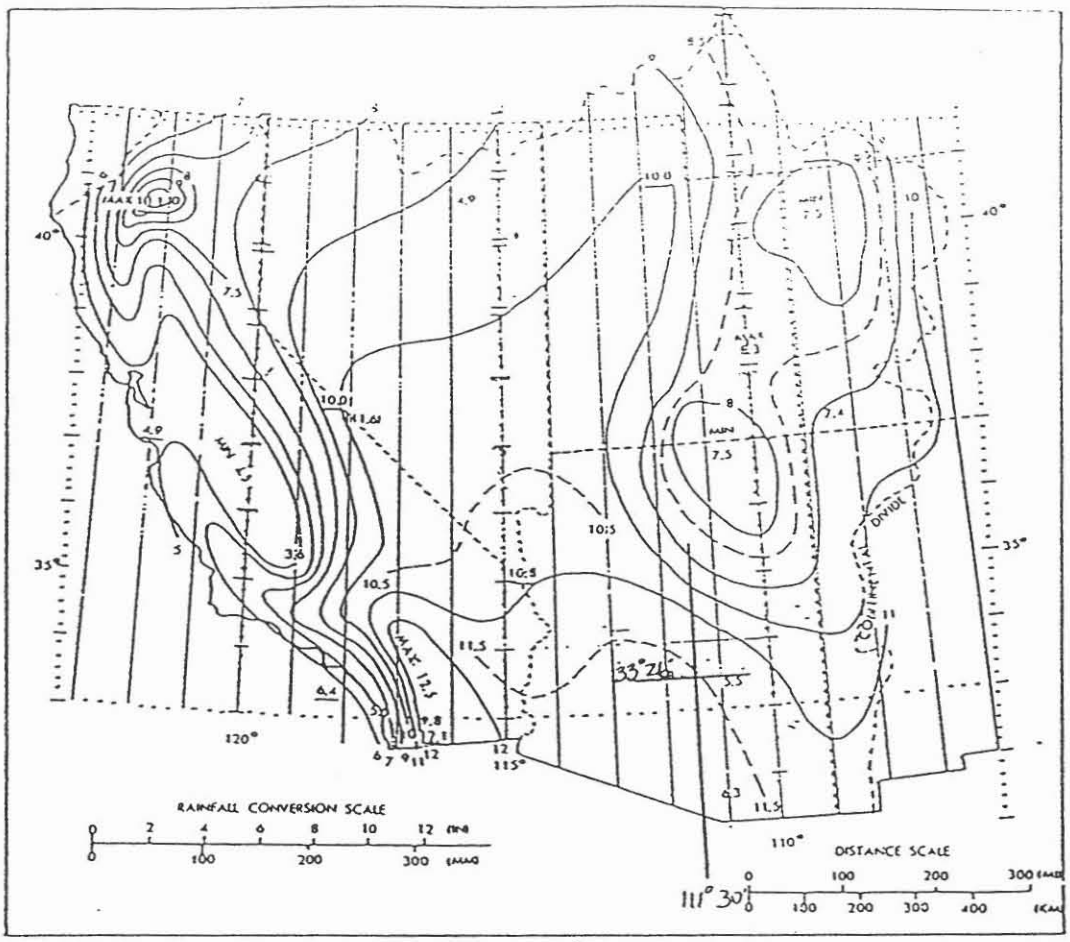


Figure 4.5--Local-storm PMP for 1 mi^2 (2.6 km^2) 1 hr. Directly applicable for locations between sea level and 5000 ft (1524 m). Elevation adjustment must be applied for locations above 5000 ft.

Figure B-9
Average 6/1-hr Ratios for Drainage

3. Avereage 6/1-hr ratio for drainage - 1.32 inches

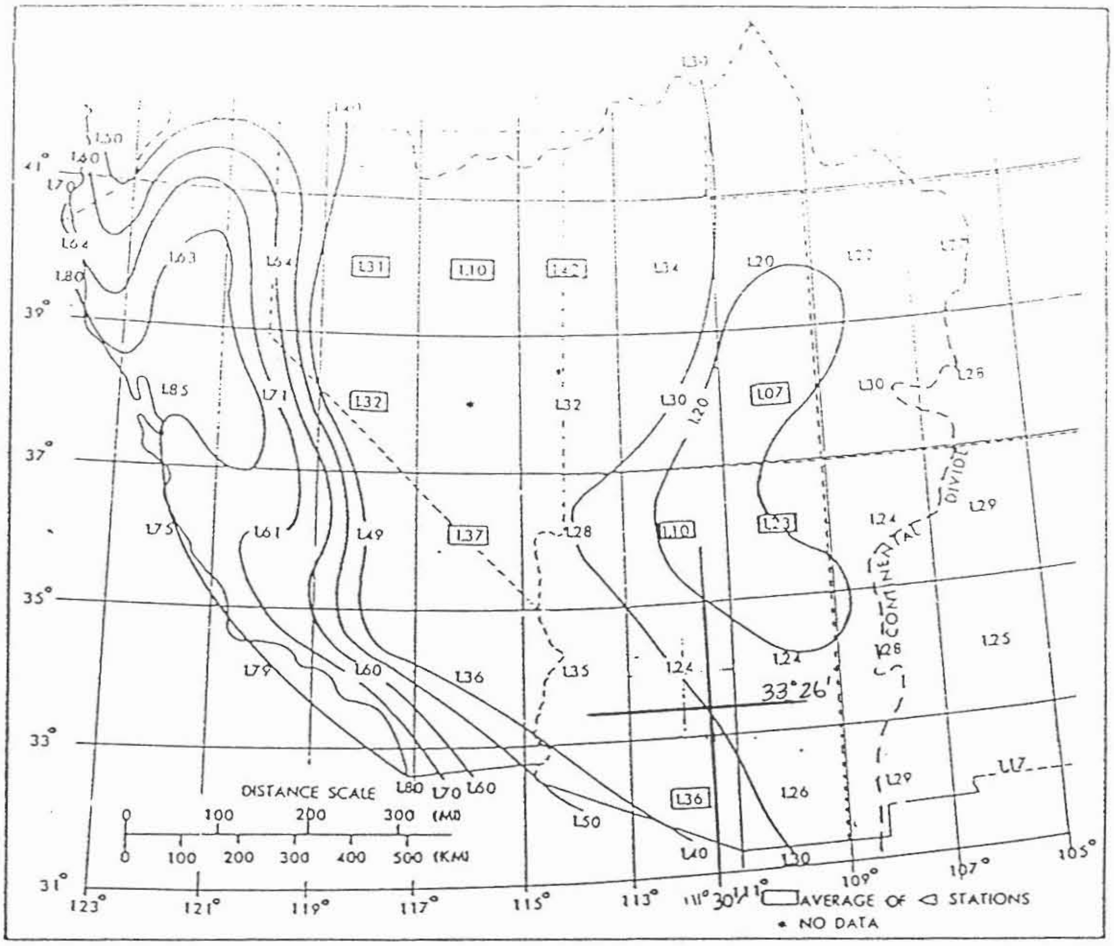


Figure 4.7.--Analysis of 6/1-hr ratios of averaged maximum station data (plotted at midpoints of a 2° latitude-longitude grid).

Table B-5
Durational Variation of 1-mi.^{2} Local-Storm PMP

Table 4.4.--Durational variation of 1-mi^2 (2.6-km^2) local-storm PMP in percent of 1-hr PMP (see figure 4.3)

6/1-hr ratio	Duration (hr) 1/4	1/2	3/4	1	2	3	4	5	6
1.1	86	93	97	100	107	109	110	110	110
1.2	74	89	95	100	110	115	118	119	120
* 1.3	74	89	95	100	114	121	125	128	130
1.4	63	83	93	100	118	126	132	137	140
1.5	63	83	93	100	121	132	140	145	150
1.6	43	70	87	100	124	138	147	154	160
1.8	43	70	87	100	130	149	161	171	180
2.0	43	70	87	100	137	161	175	188	200

Figure B-10
Areal Reduction for Local-Storm PMP

6. Aerial Reduction 48 56 58 61 66 68 71 72 73

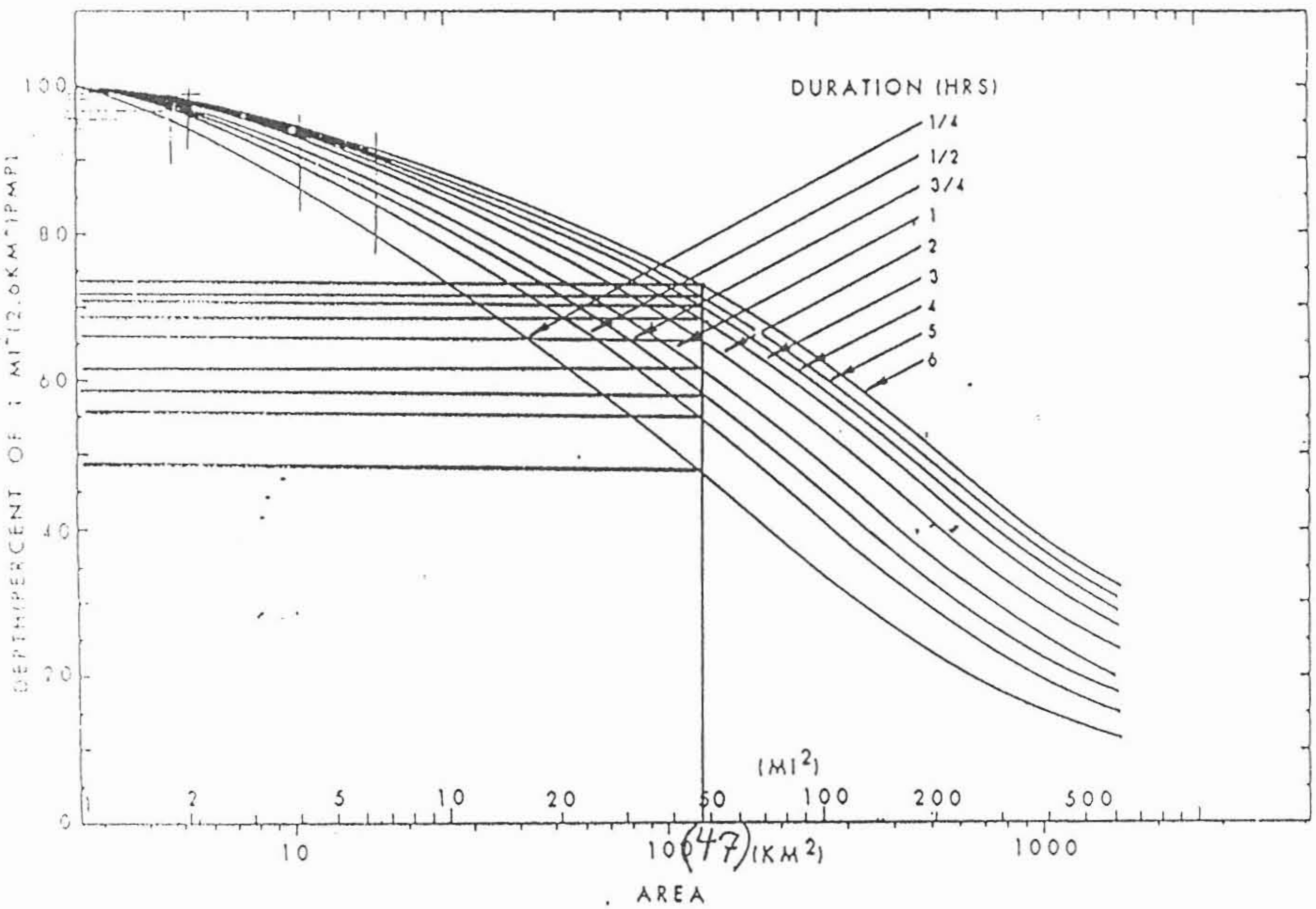


Figure 4.9.--Adopted depth-area relations for local-storm PMP.

Table B-6
Time Sequence for Hourly Incremental PMP
in 6-hr Storm

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Table 4.7.--Time sequence for hourly incremental PMP in 6-hr storm

Increment	HMR No. 5[1]	EM1110-2-1411[2]
	Sequence Position	
Largest hourly amount	Third	Fourth
2nd largest	Fourth	Third
3rd largest	Second	Fifth
4th largest	Fifth	Second
5th largest	First	Last
least	Last	First

[1]U. S. Weather Bureau 1947.
[2]U. S. Corps of Engineers 1952.

Table B-7
Time Sequence for 15-min. Incremental PMP
within 1 hour

Table 4.8.--Time sequence for 15-min incremental PMP within 1 hr.

Increment	Sequence Position
Largest 15-min amount	First
2nd largest	Second
3rd largest	Third
least	Last

APPENDIX C

AREAL REDUCTION for 100 Year Event

Figure C-1
Areal Reduction for 100 Year Event

Figure 14.—$\bar{X}'_L$ (2.54-yr depth-area ratio, see sec. 4.3) for 3-, 6-, 12-, and 24-hr in southeast Arizona. Dashed lines are 3-hr and 24-hr Chicago $\bar{X}'_L$ (from TR 24)

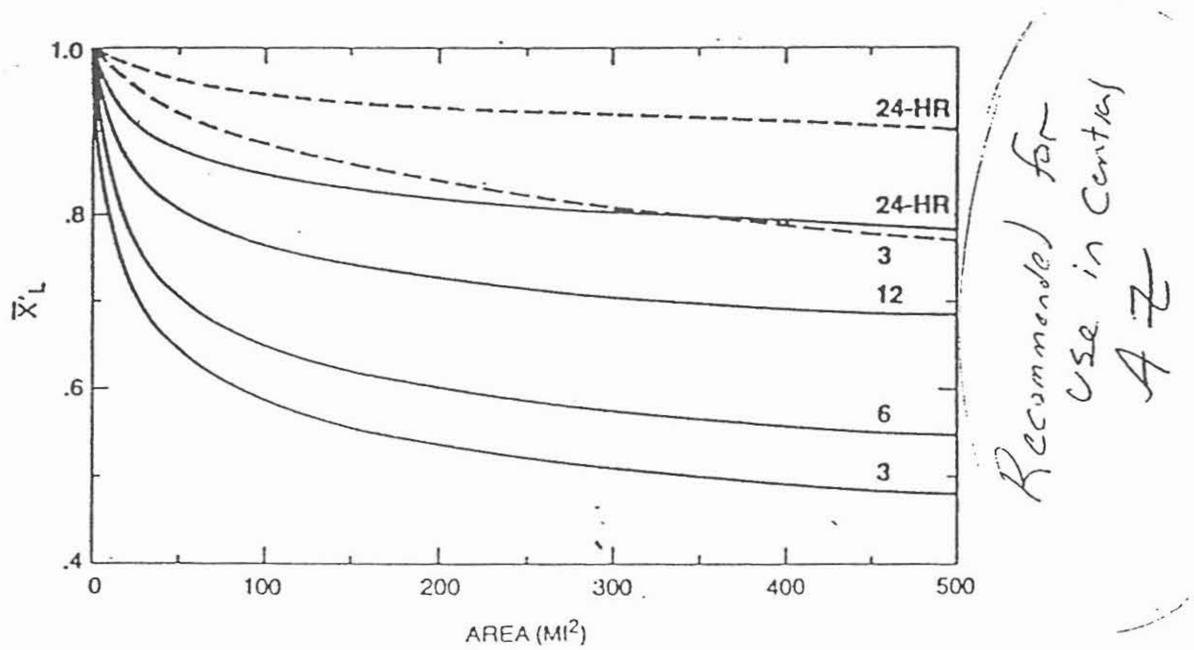
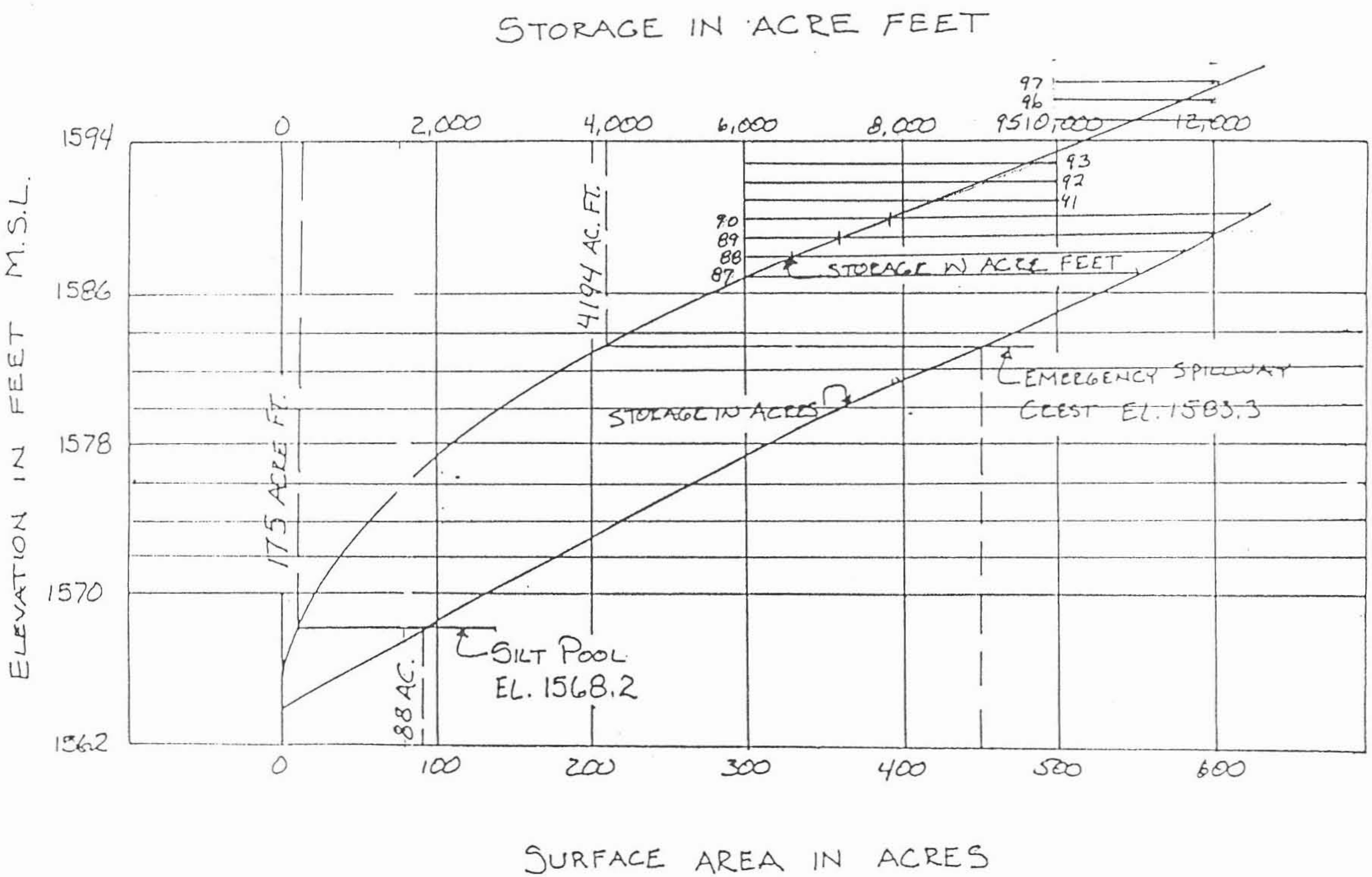


Figure 15.—Same as figure 14, but for central Arizona.

APPENDIX D

SPILLWAY CAPACITIES
FOR
POWERLINE F.R.S.

Figure D-1
Area & Capacity Curves - Powerline F.R.S.



AREA & CAPACITY CURVES — POWERLINE F.R.S.

APPENDIX E

POWERLINE FLOODWAY SPECIFICATIONS

Table E-1
Powerline Floodway Specifications

PROJECT TITLE: Powerline Floodway

WATERSHED AND RELATIONSHIP TO OTHER STRUCTURES:

Apache Junction-Gilbert Watershed

The floodway starts near the principle outlet of Powerline FRS and crosses the CAP then south westerly across southern end of General Motors Proving Grounds and along northern edge of Williams Air Force Base to join the RWCD Floodway at the north west corner of the base.

LOCATION: Township, range, section; description from well known physical feature; how to get there:

T1S, R7E, Sections 19, 20, 21, 22, 23, 24, 25, 27, 28, 29, and 30
T1S, R8E, Sections 8, 9, 17, 18, and 19

AUTHORIZATION:
e.g., Watershed Protection and Flood Prevention Act, Public Law 566

FEDERAL SPONSOR: Soil Conservation Service

LOCAL SPONSOR(S):

Flood Control District of Maricopa County
Board of Supervisors of Pinal County
East Maricopa Natural Resource Conservation District

DOCUMENTATION:

CONTRACTOR: The Ashton Company, Tucson, Arizona

DATE OF CONSTRUCTION AWARD: May 31, 1967

DATE OF FINAL ACCEPTANCE: June 1968

FUNCTIONAL DESCRIPTION:

Impounded water behind Rittenhouse FRS, Vineyard FRS and Powerline FRS drain into a common channel that discharges into the Powerline Floodway. The floodway then crossed the Central Arizona Project Canal and flows south westerly across the southern end of the General Motors Proving Ground and west to the RWCD Floodway.

The floodway picks up additional runoff water throughout its entire length. This water enters the floodway by way of side channel inlets, weir inlets and side drain inlet pipes.

PROJECT FEATURES:

Length	= 8.75 miles 6.93 miles lined 1.82 miles unlined
Width-Lined section	= 6' to 8'
Depth-Lined section	= 4.75' to 6.5'

Table E-1
Powerline Floodway Specifications

MAINTENANCE RESPONSIBILITIES

Structure: Powerline Dam 6A310

Bank Protection - Riprap		acres
Bridges - Pedestrian		each
Bridges - Vehicle		each
Culverts, Box		each
Culverts, Pipe		each
Drainage Channel - Lined		feet
Drainage Channel - Unlined	0.8	miles
Drop Structure		each
Embankment	80	acres
Embankment, Soil Cement		acres
Fencing		feet
Floodway - Lined		feet
Floodway - Unlined		acres
Gated Outlet	1	each
Gates	2	each
Gutters, Concrete		feet
High Flow	94	acres
Landscape		acres
Low Flow	47	acres
Manholes		each
Meter Houses		each
Outlet Structure	1	each
Pilot Channel		miles
Pool Area	456	acres
Principal Outlet	65	feet
Ramps, Concrete		feet
Retaining Wall		feet
Right-of-Way		acres
River Clearing		acres
Roads - Asphalt		miles
Roads - Dirt	8.8	miles
Side Inlet		each
Spillway - Earth	38	acres
Spillway - Lined		feet
Stilling Basins	1	each
Stormdrain Pipe		feet
Trash Racks	1	each
Vegetative Drains	1	each

APPENDIX F

RUNOFF CURVE NUMBERS FOR SEMIARID
RANGE AND FOREST CONDITIONS

Donald D. Woodward

Table F-1
Curve Numbers - Event vs. Duration

TABLE 6

SUMMARY OF RCNDV-II

SAFFORD W-II

Event	Runoff curve numbers for various distributions						
	Thunderstorm					Type IIA	Type II
	30 min.	1 hr.	2 hr.	3 hr.	6 hr.	24 hr.	24 hr.
100	95.4	93.4	90.7	88.5	93.2	88.6	89.7
50	95.2	93.2	90.3	87.9	93.3	89.7	90.5
25	94.9	92.7	89.3	86.5	93.2	90.7	90.5
10	94.4	91.9	88.0	84.9	94.0	91.4	91.2
5	93.7	90.7	86.0	85.9	93.8	91.9	91.9
2	91.6	86.9	77.1	78.0	94.5	93.2	92.9
Average	94.2	91.5	86.9	85.2	92.8	90.9	91.1

Figure F-1
Curve Number vs. Duration