SECTION 4
SPILLWAY ADEQUACY

Introduction

The determination of spillway adequacy is an essential aspect of studies involving the addition of hydropower units to an existing structure. A spillway acts as a safety valve to protect a dam from being overtopped, an occurrence which, especially in the case of an earthfill or rockfill dam, can have catastrophic consequences. Only a cursory inspection and evaluation are warranted until a reconnaissance estimate of hydro-electric energy resources indicates favorable economics and further study. A long history of trouble-free operation is not a certain indication of a safe spillway design. Spillway adequacy is a function of spillway capacity, storage capacity of the reservoir, physical condition and reservoir operation procedures. If operation for hydropower requires that a reservoir be kept at higher levels than intended by the original design, a presently adequate spillway may become inadequate. Also, the addition of power facilities at the toe of a dam will, in some cases, require that a spillway be relocated so that it does not discharge in the vicinity of the powerhouse.

Two main topics are addressed in this Section. The first deals with hydraulic characteristics of outlet structures, and the second pertains to criteria for spillway adequacy and techniques for calculating the Spillway Design Flood.

The overall strategy for evaluating spillway adequacy is outlined below:

Reconnaissance

- Determine if a State or Federal safety report is available.
- Determine spillway crest type.
- Obtain physical dimensions, number of piers, and type of abutments; and relative crest elevation and top of dam elevation.
- Estimate discharge coefficients from experience or appropriate references.
- Determine evaluation criteria for size and hazard classification (Table 4-3).
- Estimate spillway design discharge from regional envelope curves, if data are available.
- Estimate maximum water surface and remaining freeboard or depth of probable overtopping.

Feasibility

- Same as reconnaissance but with greater accuracy.
- Determine spillway design flood hydrograph from probable maximum precipitation and watershed runoff model.
- Route hydrograph through reservoir surcharge storage and spillway.
- Determine freeboard adequacy or required modifications.
- Evaluate location adequacy of stilling basin relative to power plant site.

Hydraulic Characteristics of Spillways

A spillway is a hydraulic passageway designed to conduct flood flows safely past a dam. Some dams are designed with two spillways - a service spillway to discharge floods likely to occur fairly frequently, and an auxiliary or an emergency spillway to handle larger, infrequent flows. The latter type of spillway is frequently constructed with unpaved channels; hence, maintenance costs associated with erosion of the structure, and possibly with downstream deposition, may be incurred following periods of operation.

The configuration of a spillway is tailored to a particular dam site and is dependent on the type of dam and intended operation and on the economic tradeoff between spillway capacity and dam height. Types of spillway are overflow, chute, shaft, side-channel, and siphon.

Overflow. An overflow spillway is a portion of a dam designed for water to pass over. Many overflow spillways are designed with a shape that closely approximates the shape of the lower nappe of flow over a sharp-crested weir, because a profile of this shape produces near-maximum discharge efficiency at the design head. The curved shape of the nappe is found to be a function of the head on the weir, the slope of the front side of the weir, and the velocity of approach. Consequently these three characteristics affect the magnitude of discharge coefficients for various weir shapes and heads.

Chute. A chute spillway is basically an open channel designed to convey water from a control section to the downstream river channel. Chute spillways are commonly used with earthfill dams. Flow down steep chutes is rapid and unstable, and a chute must be carefully designed for safe and proper operation. Chutes may be constructed along the abutment of a dam, down the face of a dam, or down a saddle at some distance from the dam.

Shaft. A shaft spillway typically consists of a vertical shaft which makes a 90-degree bend into a horizontal tunnel that passes through or under a dam or abutment. This type of spillway is often used where space or site conditions preclude the use of other types of spillway. Because the inlet for a shaft spillway is often a funnel-shaped overflow crest, the name "morning glory" is commonly applied to a shaft spillway. Under low heads, the overflow crest will act as a control. However, as the head increases, control will shift to the spillway "throat", and finally full pipe flow will occur. Under full pipe flow conditions, discharge will vary in proportion to the square-root of the head, in which case there is little increase in capacity with increasing head. Vortex formation at the intake, surging in the vertical shaft, and erosion of concrete in the vicinity of the vertical elbow are problems associated with shaft spillways.

Side-Channel. A side-channel spillway is one in which water enters a channel by passing over an overflow crest that parallels the channel. Side channel spillways are sometimes used in narrow canyons where there is insufficient width for a suitable crest length for an overflow or chute spillway.

Siphon. A siphon spillway is sometimes used where it is desirable to develop full discharge capacity quickly, for example in the event of a turbine shut-down. Siphon spillways are capable of providing automatic regulation of reservoir levels within fairly narrow limits. However, the siphon spillway, like the shaft spillway, cannot handle flows much greater than the design flow, because under high flow conditions discharge is proportional to the square-root of the head. Inability to pass ice and debris, and potential for surging are also disadvantages of the siphon spillway.

Although from the operational viewpoint an uncontrolled spillway is often desirable, there are many situations where the advantages of having a gated spillway outweigh the disadvantages. Gates are used when sufficient crest length for an uncontrolled spillway cannot be developed or when sufficient head cannot be developed for the required discharge capacity. Gates are also used where it is necessary to initiate spillway releases when the reservoir is below the normal full pool elevation. Numerous gate types are in use, including rectangular lift gates, roller gates, radial gates, and drum gates. In some instances flashboards or stoplogs may be utilized. For information on the hydraulics of gated spillways, the reader is referred to *Hydraulic Design of Spillways* (U.S. Army Corps of Engineers, 1965), *Design of Small Dams* (USBR, 1977), and *Handbook of Applied Hydraulics* (Davis and Sorenson, 1969).

Discharge Over an Ogee Spillway Crest

Overflow spillways behave as weirs and, therefore, if the spillway is not submerged the flow will pass through critical depth over the crest. One of the most common crest shapes is the "ogee". The discharge over an uncontrolled ogee crest is given by the following equation:

$$Q = C \times L \times H_e^{3/2} \qquad (4\text{-}1)$$

where
Q = discharge
C = variable discharge coefficient
L = effective crest length
H_e = total head on the crest, including velocity of approach head, h_a

The discharge coefficient, C, is influenced by the following factors: (1) the depth of approach, (2) relation of crest shape to "ideal" nappe shape, (3) slope of upstream weir face, (4) downstream apron interference, and (5) submergence if it occurs downstream. Where the design of the approach channel results in appreciable additional losses, they must be added to He to determine reservoir elevations that correspond to discharges determined with equation (4-1).

As the approach depth of the flow to a weir decreases the approach velocity increases, thus affecting the discharge coefficient. The discharge coefficient for a vertical-faced ogee crest ranges between 3.8 and 3.9 for values of P/H_0 ranging from .5 to 3.0, where P is the weir height and H_0 is the design head for the spillway (USBR, 1977). The discharge coefficient will also vary for heads other than the design head. The ratio of discharge coefficient to design-head discharge coefficient varies from 0.85 to 1.07 as the ratio of head to design head varies from 0.2 to 1.6 (USBR, 1977). The reader is referred to *Design of Small Dams* (USBR, 1977), *Hydraulic Design of Spillways* (U.S. Army Corps of Engineers, 1965), and *Hydraulic Design Criteria* Chart 111-3/3 WES 2-72 (U.S. Army Corps of Engineers, 1968), for relationships for discharge coefficients, including adjustments for downstream apron interference and downstream submergence.

Piers and abutments cause side contraction of the overflow and therefore decrease the effective crest length of a spillway. These effects may account for a reduction of 1 to 5 percent, depending on pier and abutment types and spacing. The references sited above should be consulted for details.

Sources of Criteria for Determining Spillway Discharge Characteristics

The previous paragraphs briefly reference criteria that are applicable for estimating discharge characteristics for an ogee spillway crest. Detailed criteria for ogee crests and for the other spillway types mentioned previously may be found in a number of technical publications. Some of these are *Design of Small Dams* (USBR, 1977), *Handbook of Applied Hydraulics* (Davis and Sorenson, 1969), *Handbook of Hydraulics* (King and Brater, 1963), *Hydraulic Design of Spillways* (U.S. Army Corps of Engineers, 1965), and *Hydraulic Design Criteria* (U.S. Army Corps of Engineers, 1968). Computer programs are available as an aid to determining spillway ratings. Exhibit II, program 7 is one such program available through U.S. Army Corps of Engineers, Hydrologic Engineering Center, 609 Second Street, Davis, California 95616.

Although many past physical model studies provide a good insight to the range of values for coefficients and transition losses, large investment costs in major spillway and stilling basin modifications can justify consideration of site specific physical model studies.

Hydraulic Characteristics of Conduits and Outlet Works

Outlet works are a means of controlling the release of water from a reservoir. They are used to control downstream flows for a variety of purposes, such as irrigation, water supply, recreation, fisheries, and water quality control. Outlet works may be used during flood control regulation to augment spillway discharges or to evacute storage in anticipation of a flood event. They are also used to empty an impoundment for inspection and repair.

Definition of the discharge characteristics for outlet works typically involves both open channel and pressure flow computations. For the situation where free flow occurs over the crest at the outlet works, the weir flow equation, equation 4-1, is applicable. When open channel outlet flows are controlled by partly opened surface gates or radial gates, or sluice flows are controlled by partly opened surface gates or radial gates, sluice flow will result. Discharge for sluice flow may be calculated with the equation:

$$Q = 2/3 \times \sqrt{2 \times g} \times C \times L \times (H_1^{3/2} - H_2^{3/2}) \tag{4-2}$$

where
- Q = discharge
- g = gravitational acceleration
- C = discharge coefficient
- L = width of gate
- H_1 = total head to overflow crest
- H_2 = total head to bottom of gate

The discharge coefficient in equation 4-2 will vary with gate type and as a function of flow conditions upstream and downstream of the gate. Typically "C" ranges from .65 to .85.

For the case where the control opening is either partly or entirely submerged, discharges are calculated with submerged orifice or tube flow relations such as:

$$Q = C \times A \times \sqrt{2 \times g \times H}$$

where
- Q = discharge
- A = area of opening
- g = gravitational acceleration
- H = difference between the upstream and downstream water levels
- C = discharge coefficient for submerged orifice or tube flow

Coefficients for various conditions and orifice configurations are found in *Design of Small Dams* (USBR, 1977), and *Hydraulic Design of Reservoir Outlet Structures* (U.S. Army Corps of Engineers, 1963).

Discharge through an outlet conduit that is flowing full may be determined by application of the Bernonilli equation and estimation of loss coefficients for the various components of loss, which may include trashrack, entrance, bend, expansion, contraction, gate, and exit losses in addition to friction losses. Friction losses are commonly estimated with the Darcy-Weisbach formula. Loss coefficients and friction factors are provided in numerous textbooks and publications which deal with pipe flow. See, for example, *Hydraulic Design of Reservoir Outlet Structures* (U.S. Army Corps of Engineers, 1963), or *Handbook of Hydraulics* (King and Brater, 1963). The former publication contains a detailed example computation of the discharge rating curve for outlet works.

Spillway Design Floods

The determination of a standard against which to base judgment on the spillway adequacy is not a clear cut decision upon which the engineering profession has fully agreed. State and Federal agencies have varying standards and an owner of any dam of sufficient height and storage to come within the approval requirements of the State within which it is located, must obtain approval from the appropriate State agency. In addition, Federal licensing agency (Federal Energy Regulatory Commission) review and approval are required for those structures associated with hydropower installations. The mere fact that the dam has operated "safe and sound" for a significant period of time does not in itself assure an adequate hydrologic design.

Inspection Standards. The occurrence of dam failures during the past ten years resulted in the passage of Public Law 92-367, 92nd Congress, House Resolution 15951, 8 August 1972, wherein the Secretary of the Army, acting through the Chief of Engineers, Corps of Engineers, was directed to carry out a national program of inspection of dams. Appendix D of the Secretary's report to the Congress, "National Program of Inspection of Dams", 1975, contains recommended guidelines for inspection of existing dams. The guidelines are not intended as appropriate standards for the design of new facilities. However, the guidelines provide a satisfactory basis for evaluating existing projects for a reasonable degree of safety under existing conditions. The following three tables have been copied from the above reference.

TABLE 4-1
SIZE CLASSIFICATION

Category	Impoundment Storage (Ac-Ft)	Height (Ft)
Small	≥ 50 and < 1000	≥ 25 and < 40
Intermediate	≥ 1000 and < 50,000	≥ 40 and < 100
Large	≥ 50,000	≥ 100

TABLE 4-2
HAZARD POTENTIAL CLASSIFICATION

Category	Loss of Life (Extent of Development)	Economic Loss (Extent of Development)
Low	Nonexpected (no permanent structures for human habitation)	Minimal (Undeveloped to occasional structures or agriculture)
Significant	Few (no urban developments and no more than a small number of inhabitable structures)	Appreciable (Notable agriculture, industry or structures)
High	More than few	Excessive (Extensive community, industry or agriculture)

TABLE 4-3
HYDROLOGIC EVALUATION GUIDELINES
RECOMMENDED SPILLWAY DESIGN FLOODS

Hazard	Size	Spillway Design Flood (SDF)
Low	Small	50 to 100-yr freq
	Intermediate	100-yr to 1/2 PMF[1]
	Large	1/2 PMF to PMF
Significant	Small	100-yr to 1/2 PMF
	Intermediate	1/2 PMF to PMF
	Large	PMF
High	Small	1/2 PMP to PMF
	Intermediate	PMF
	Large	PMF

1/ PMF = (probable maximum flood); this represents the flood that may be expected from the most severe combination of meteorologic and hydrologic conditions that are considered to be *reasonably possible* in the geographical region encompassing the basin under study.

Design Standards. Whenever overtopping and failure of a dam could cause significant increases in downstream flow and stage, there may be hazards to life and property. Therefore, reservoir projects modified to include hydropower facilities must also provide reasonable security against flood flows. If inspection and evaluation of the project reveals that the spillway capacity is deficient, according to the Corps of Engineers "Recommended Guidelines", plans for improvements to the project spillway capacity must be included with the proposed powerplant design. Because of the potential for future development downstream from hydropower projects, all projects that could significantly increase downstream flooding hazards should be designed to safely pass at least one half of the probable maximum flood (PMF) hydrograph. When potential hazard to life is a major consideration, FERC may require the project to safely pass the full PMF hydrograph.

Indirect PMF Estimates. Many small dams which will be the likely sites for small hydro development will have had recent State or Federal safety inspections as a result of PL 92-367. These reports may have developed PMF estimates which are available. An approximation of the magnitude of PMF inflow can be obtained by means of an envelope curve of drainage area versus PMF discharges at other Federal and FERC licensed public and private sites in the same general region.

A source of PMF data at existing dams is contained in the Nuclear Regulatory Commission's (NRC) Regulatory Guide 1.59, August 1977. Envelope curves for the eastern region of the U.S. were developed by Nunn, Snyder, and Associates for the NRC and are presented in R.G. 1.59 referenced above. These figures are reproduced herein in Figure 4-1 through 4-6 for easy reference. These estimates are intentionally high, but, if a spillway at a site in the region of the map coverage is able to safely pass this discharge, further detailed estimates can probably be delayed until final design studies are warranted.

Probable Maximum Precipitation (PMP)

Well defined procedures have been developed for obtaining PMP estimates for the eastern part of the United States. Various Hydrometeorological Reports have been prepared by the Department of Commerce, National Oceanic and Atmospheric Administration, Office of Hydrology. These reports, listed in the reference section of this volume, address specific large river basins and regions of the country. The most recent version of PMP derivation for basins east of 105th meridian is contained in (HR 51, 1978). These reports are available from the Superintendent of Documents, Washington, D.C. HR 51 is applicable to river basins 10 to 20,000 square miles in size and for storm durations 6 to 72 hours. There are 30 figures which are used to obtain storm depths for a range of durations and sizes. An example of the use of these PMP charts from HR 51 follows:

1. Determine the geographic location and size of the drainage areas under study.
2. From the PMP maps, tabulate the average PMP depths for the basin location. Generally 3 or 4 areal sizes bracketing the size of the study basin are adequate. Tabulate depths for each duration 6, 12, 24, 48, and 72 hours.
3. Plot the PMP depths on semilog paper (depth versus area) and draw smooth curves through the plotted points.
4. From the depth-area-duration graph of step 3, determine the PMP depths at the basin size for each duration.
5. Plot these values on cartesian grid or semilog grid and interpolate for intermediate durations, if required.

Example PMP. Compute PMP values for a 1,200 square mile basin located in central Arkansas, Latitude 35° N, Longitude 93° W. From maps like Figure 4-7, we obtain the following values.

	(A)	(B)	(C)	(D)	(E)
Duration (hr)	6	12	24	48	72
200 sq. mi. Depth (in)	22.0	26.8	30.5	34.0	36.8
1,000 sq. mi Depth	16.3	20.8	25.0	28.5	30.3
5,000 sq. mi. Depth	9.5	13.5	17.0	21.1	23.7

These values are plotted in Figure 4-8, and the adopted 1,200 square mile depth-duration curve is constructed as curve F extrapolated to 3 hours. Values at 3 or 6-hour intervals can be taken from curve F.

PMP West of 105th Meridian. Estimates for basins west of 105 degrees longitude are complicated by the strong influence of the high mountain ranges. The loss of moisture on the windward sides of the ranges and the desiccating effect of the subsidence of the air mass on the leeward side of the range are examples of this influence.

Both general and local type storm genesis are characteristic of the west. The general storms have two components that cause air mass lifting and consequent precipitation. These are orographic and convergence. These two components are considered separately and then combined to develop the storm total precipitation.

Orographic precipitation is defined as that which results from the lifting effect of a topographic feature on a flow of air passing over it. The induced vertical motions in the flow are primarily due to the ground slope, but may also be related to the narrowing of the terrain, such as a constricted valley (e.g., northern Sacramento Valley). Orographic precipitation includes

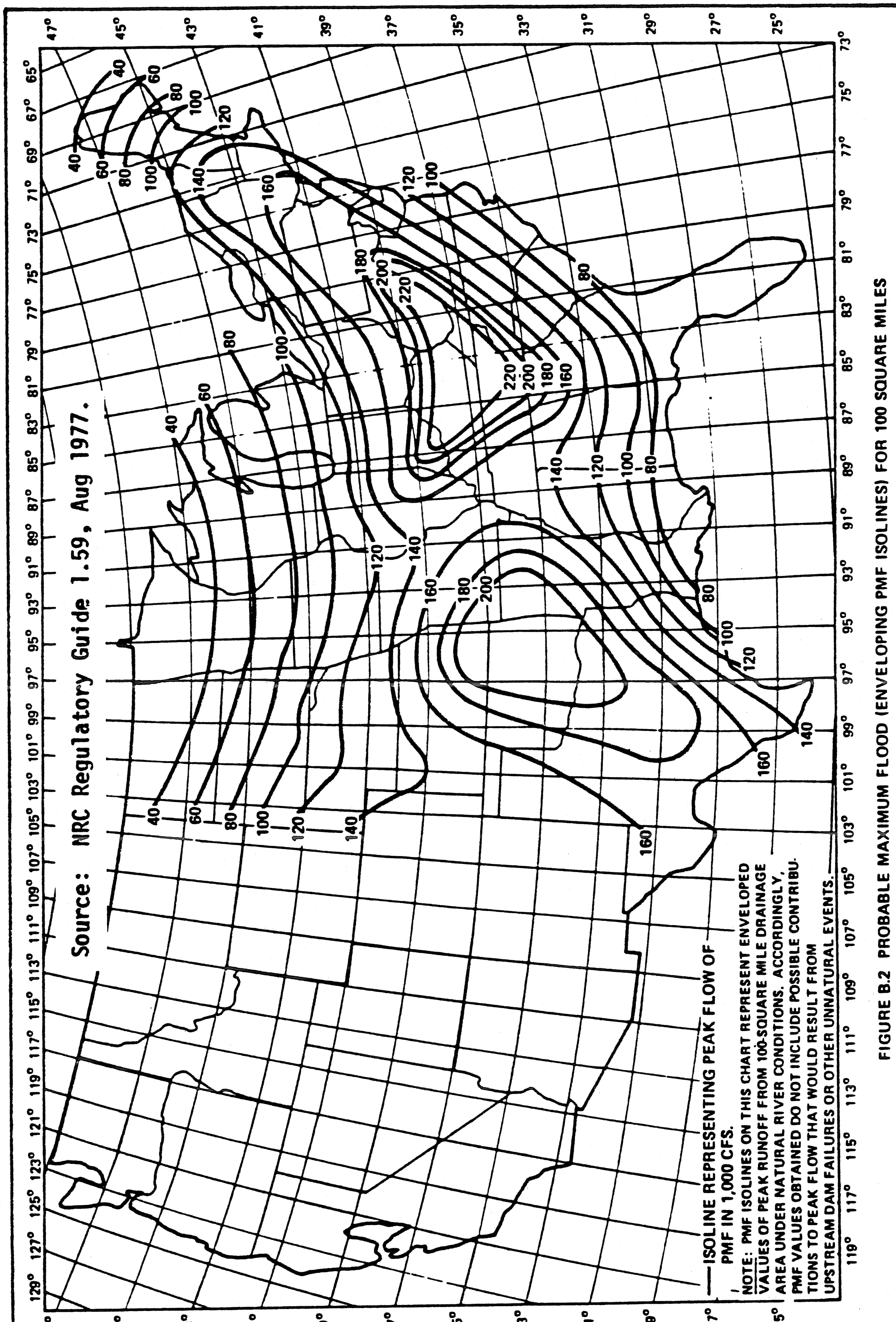

Figure 4-1. Probable Maximum Flood (100 sq. mi.)

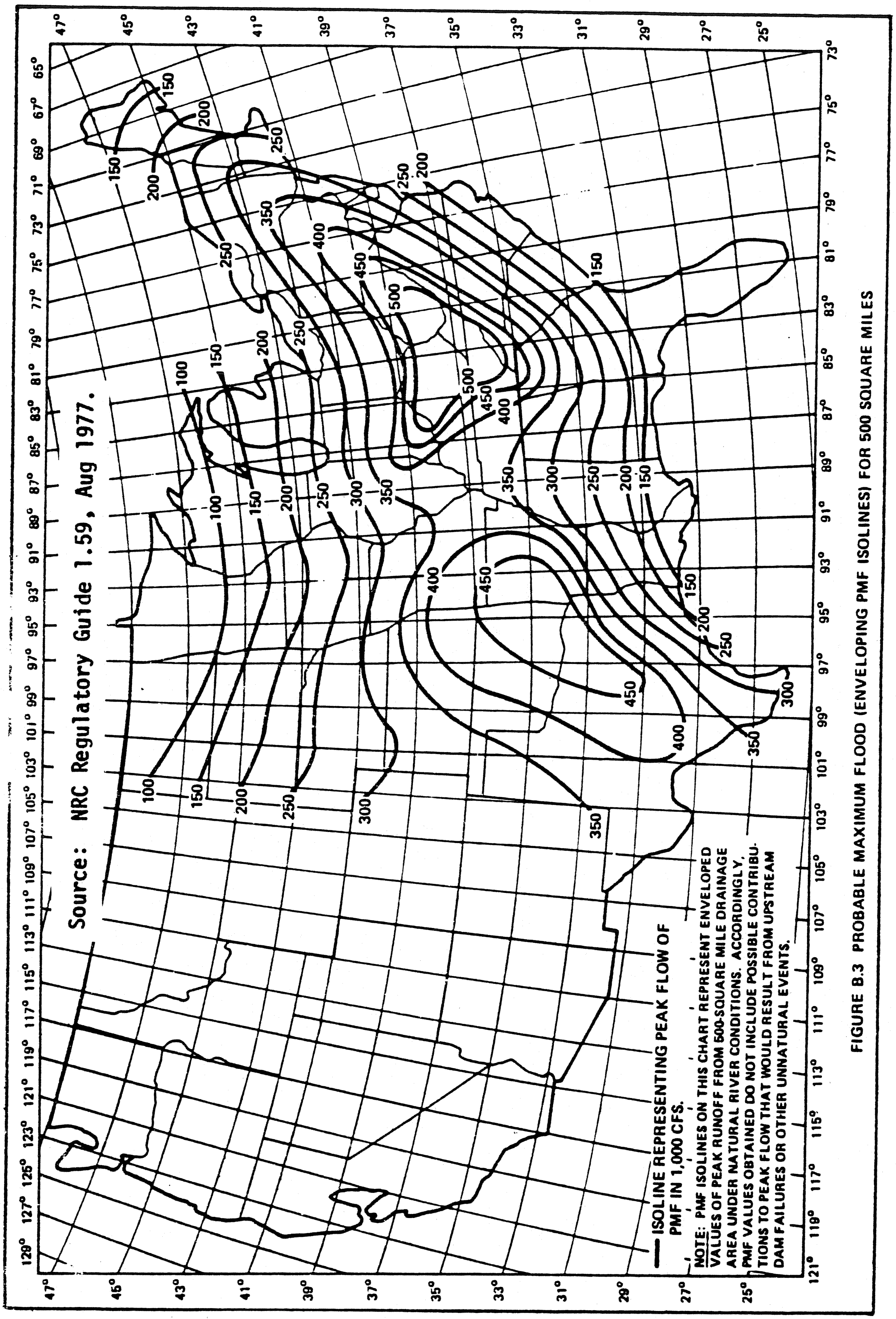

FIGURE B.3 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 500 SQUARE MILES

Figure 4-2. Probable Maximum Flood (500 sq. mi.)

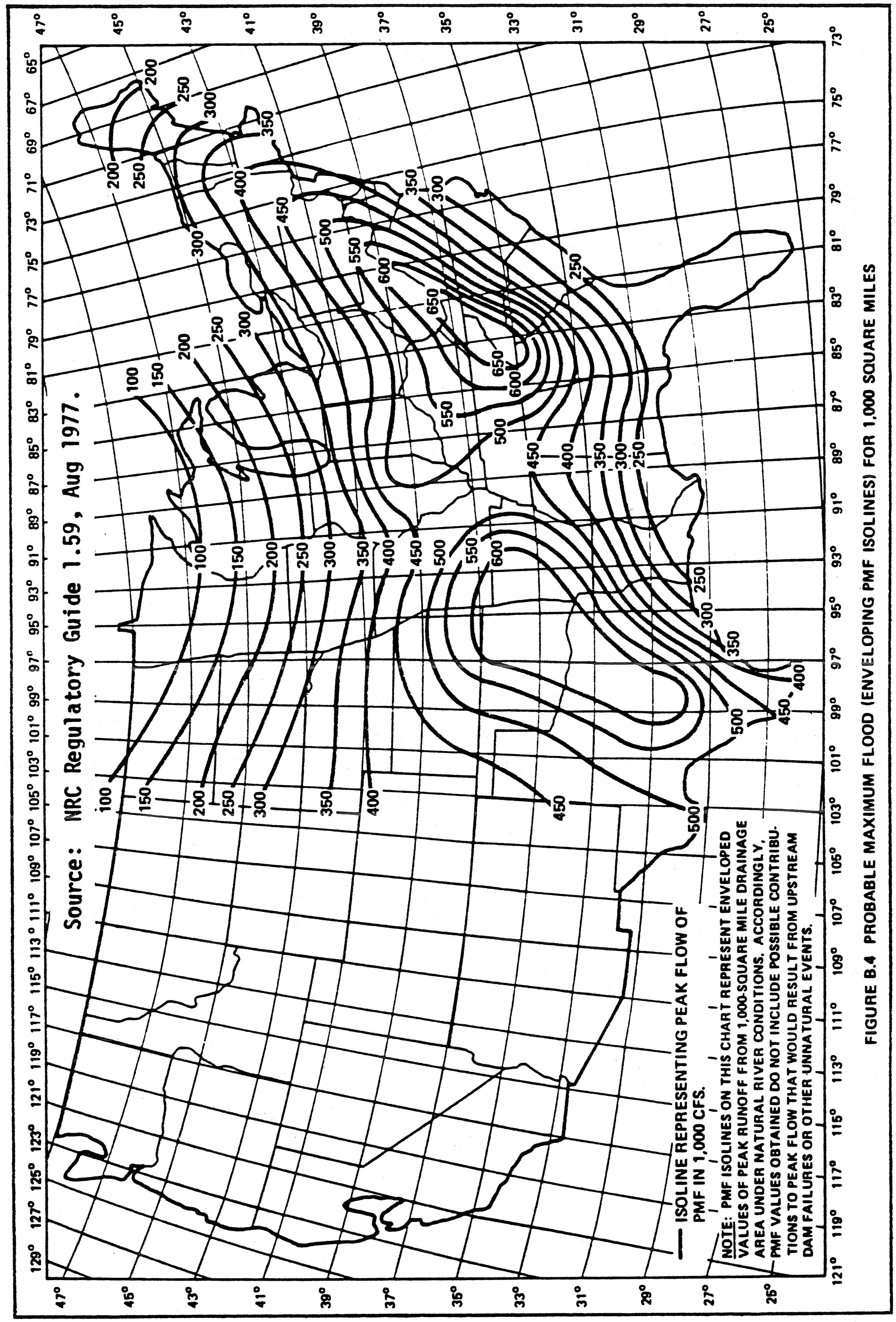

Figure 4-3. Probable Maximum Flood (1,000 sq. mi.)

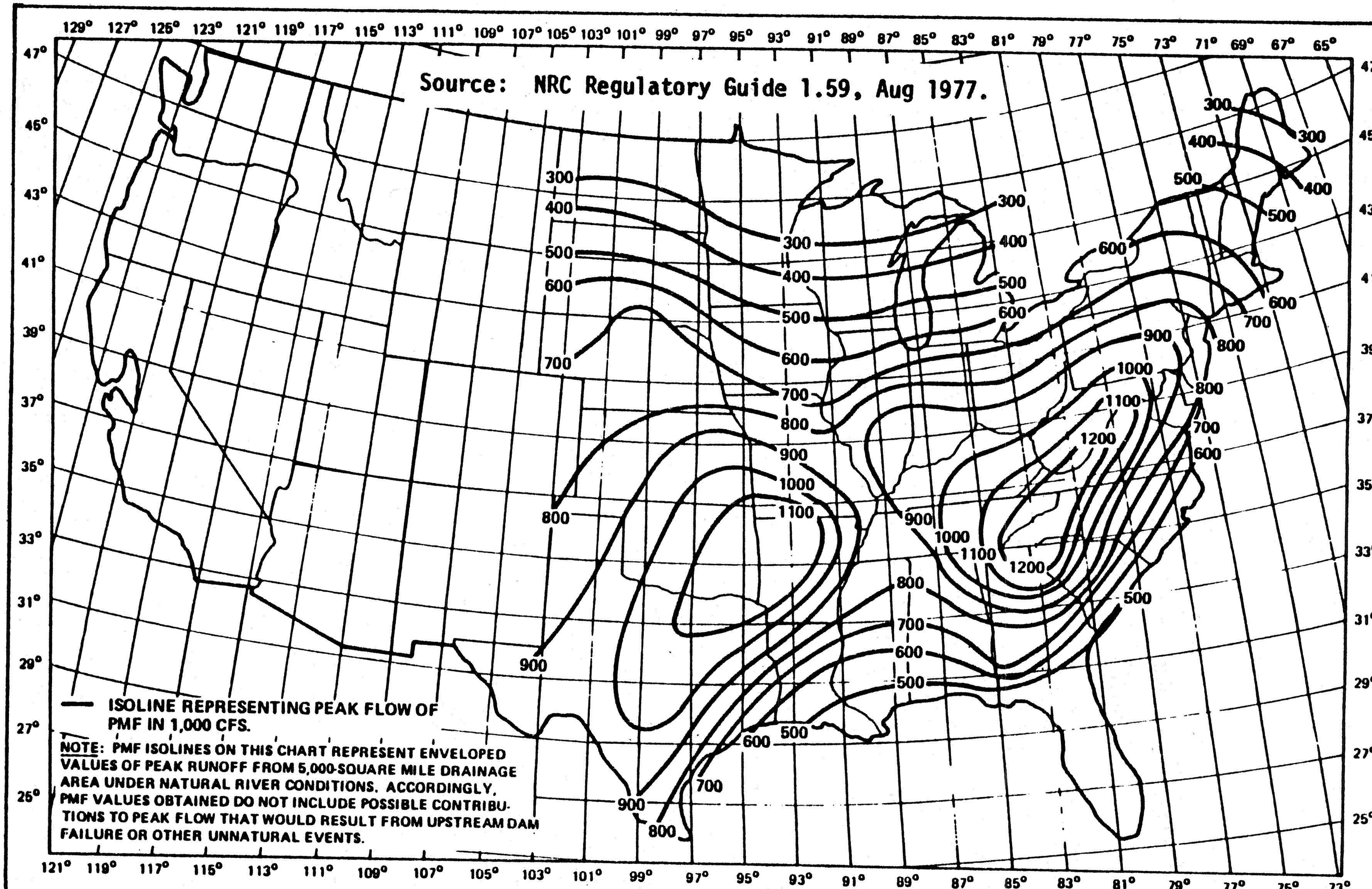

FIGURE B.5 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 5,000 SQUARE MILES

Figure 4-4. Probable Maximum Flood (5,000 sq. mi.)

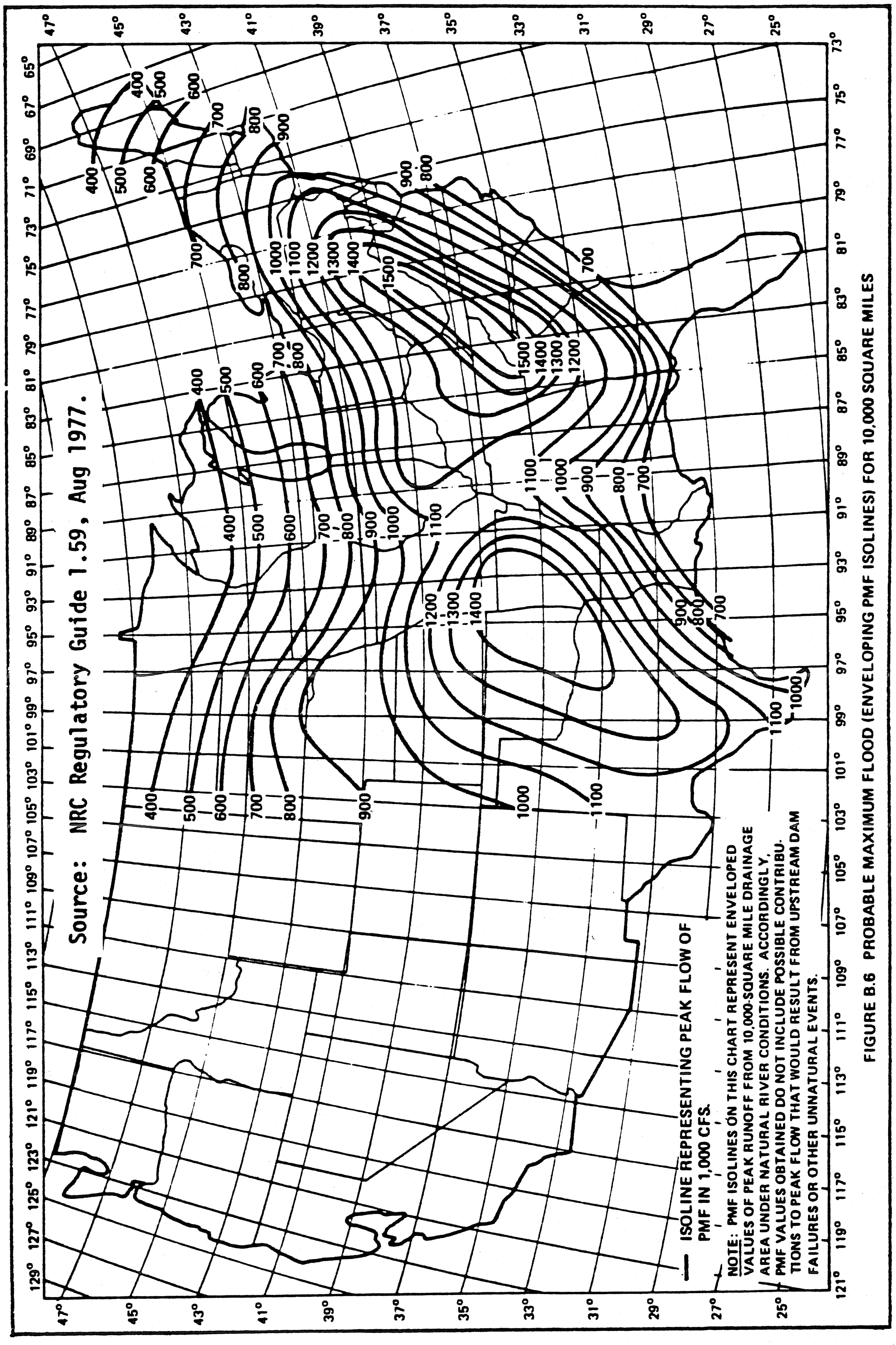

Figure 4-5. Probable Maximum Flood (10,000 sq. mi.)

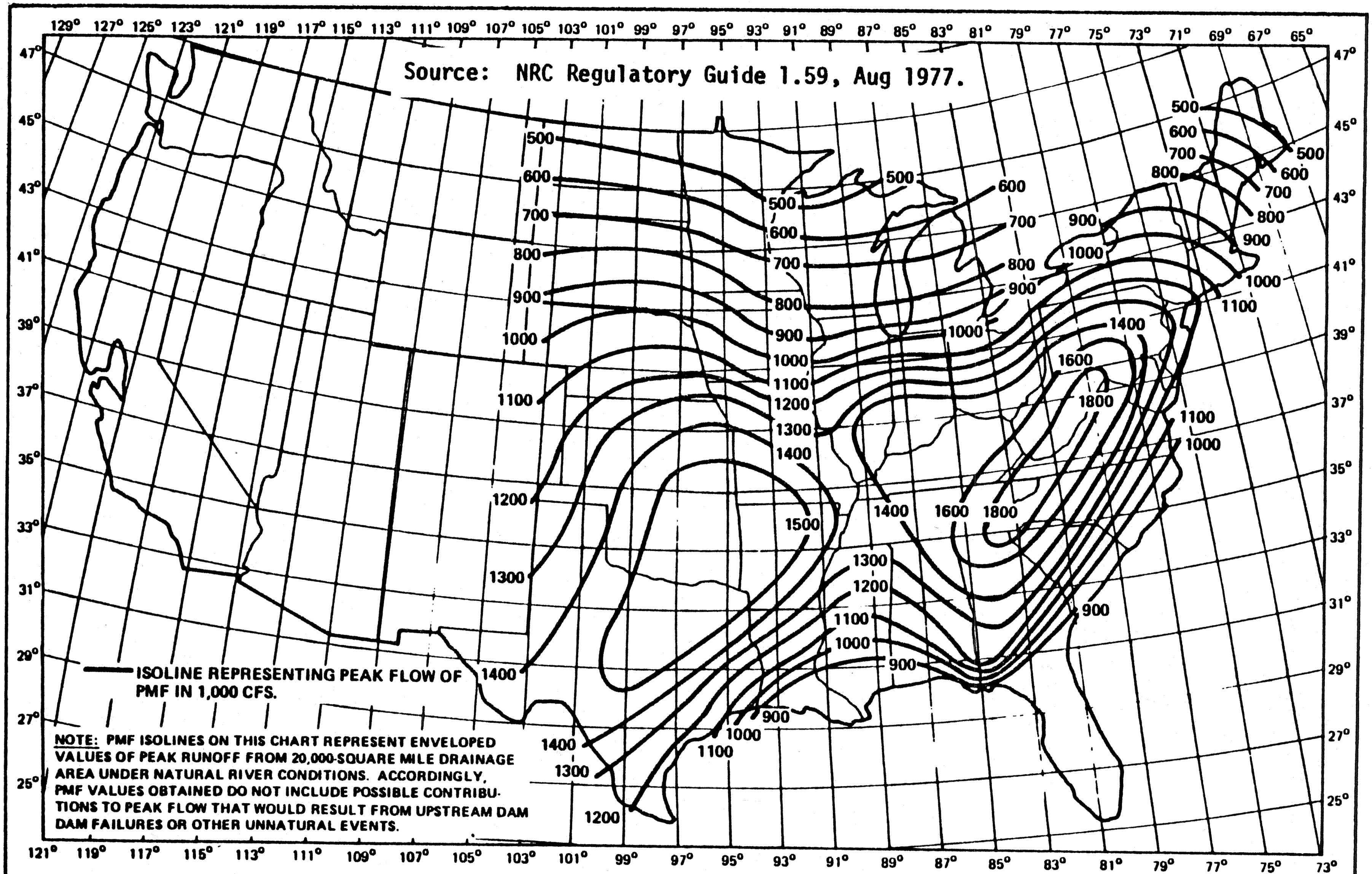

FIGURE B.7 PROBABLE MAXIMUM FLOOD (ENVELOPING PMF ISOLINES) FOR 20,000 SQUARE MILES

Figure 4-6. Probable Maximum Flood (20,000 sq. mi.)

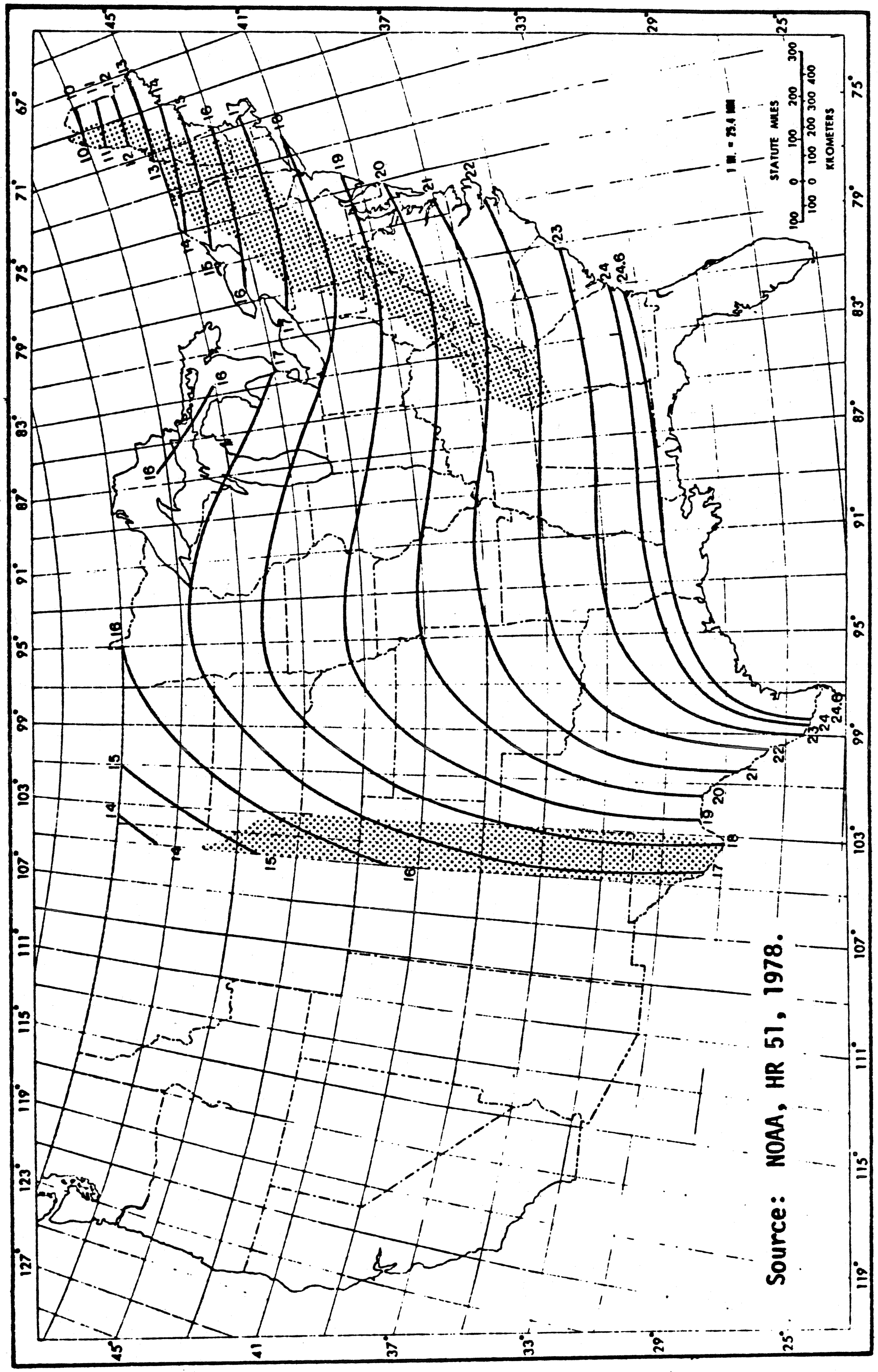

Figure 4-7. All season PMP for 6 hrs, 200 sq. mi.

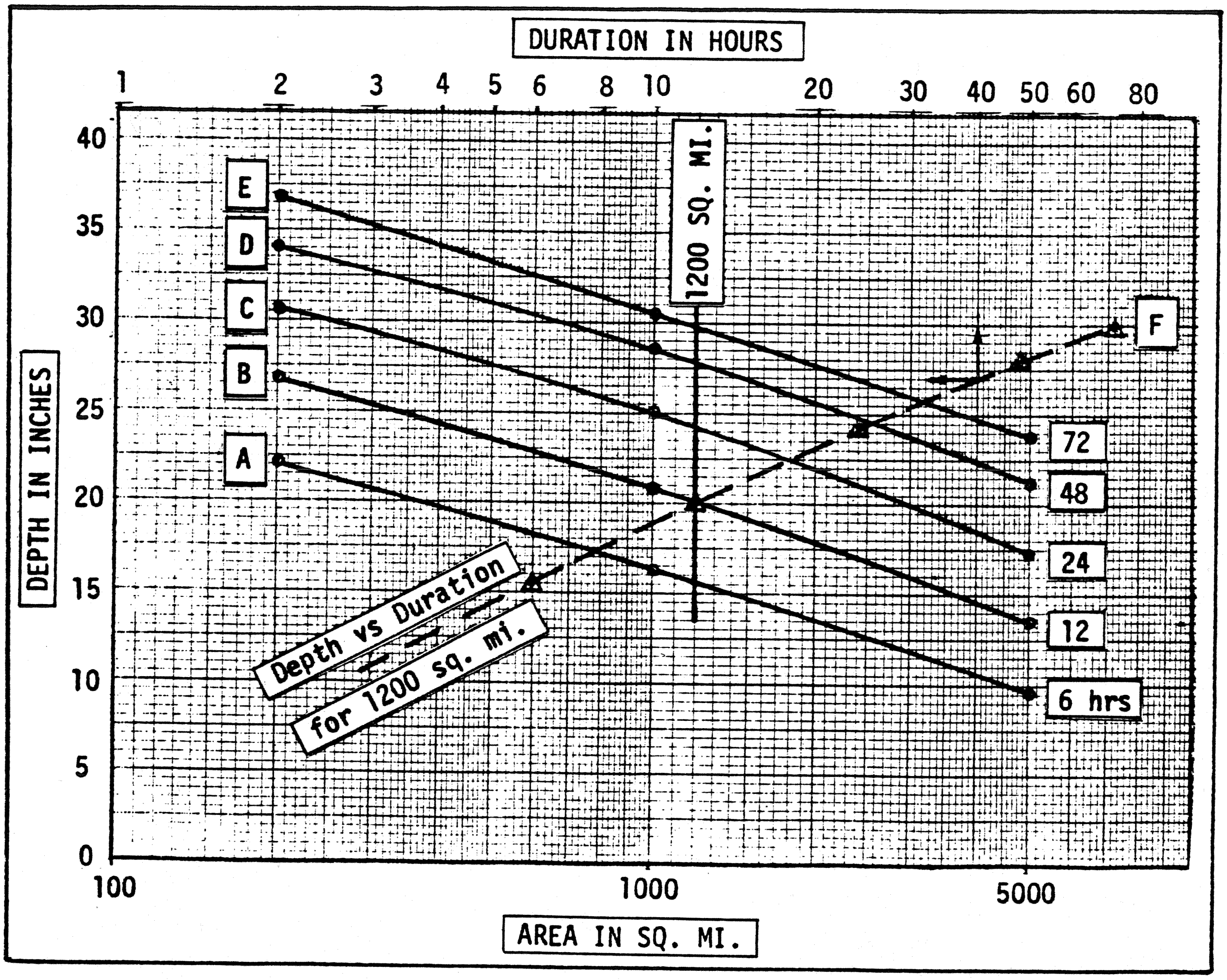

Figure 4-8. Example PMP estimate using HR 51, NOAA, 1978.

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both that falling on the windward slope and that blown over the barrier (called spillover).

Convergence precipitation includes precipitation resulting from lifting induced by atmospheric processes other than orographic. These are mainly horizontal convergence, frontal lifting, and instability. In mountainous areas these occur simultaneously.

Details for the steps used to estimate the PMP for general storms in the west are contained in various Hydrometeorological Reports (HR) and Technical Papers (TP), as follows:

1. *California*

(1) HR 36, "Interim Report, Probable Maximum Precipitation in California," presents basic steps and charts for making general storm estimates. The report is currently approved for general application in the computation of probable maximum flood hydrographs for drainage basins less than about 5,000 square miles within the area covered by generalized charts, subject to such additional special analyses as may be warranted in special cases. The procedures used include approximate allowances for basin shape, elevations, and orientation, so that no additional adjustments in rainfall quantities are called for. However, in view of the unusually complex problems involved in preparing generalized estimates that are applicable to the region indicated, flood estimates based on the criteria contained in HR 36 should be utilized with utmost care and judgment by experienced hydrologic engineers.

(3) Technical Paper 38, "Generalized Estimates of Probable Maximum Precipitation for the U.S. West of the 105th Meridian for Areas Less Than 400 Square Miles and Durations to 24 Hours," was prepared by the Cooperative Studies Section of the National Weather Service for the Soil Conservation Service. A different, but reasonable, approach to PMP was used in this

report. TP 38 includes small-area intense local warm-season storms while HR 36 is restricted to cool season storms. Also, a single 6/24-hour duration ratio was adopted in TP 38 since local storms are most likely to control for small areas.

2. *Hawaii.* HR 39, "PMP in the Hawaiian Islands," gives generalized rainfall criteria for computation of probable maximum floods for all sized basins encountered in the Hawaiian Islands. Effects of topography, wind exposures, and other pertinent factors are reflected in the estimates. Guide criteria for estimating hyetographs are also included.

3. *Northwest.* HR 43, "PMP, Northwest States," contains generalized PMP rainfall estimates 1 to 72 hours for basins of 10 to 5,000 square miles west of the Cascade Divide, and 10 to 1,000 square miles east of the Divide, in the Columbia River Basin. Seasonal variations are given by months, October through June. Various optional storm patterns are included (see Figure 5 of this material). HR 42, "Meteorological Conditions for the PMF on the Yukon River above Rampart, Alaska," is typical of a number of basin specific reports prepared by the HMB, NWS.

4. *Southwest.* HR 49, "PMP Estimates, Colorado and Great Basin Drainages", gives general-storm PMP estimates for durations between 6 and 72 hours and for area sizes between 10 and 5,000 square miles for the Colorado River and Great Basin. Probable Maximum Thunderstorm Precipitation Estimates, Southwest States, West of Continental Divide, are also included in this report. It covers the areas of California, Nevada, Utah, Arizona, Wyoming, Colorado, and New Mexico, and durations 15 minutes to 6 hours and areas 1 to 500 square miles. Isohyetal patterns are included.

Hyetographs for Use in Computing PMF Hydrographs

The chronological sequence of probable maximum precipitation and snowmelt, associated infiltration losses, and rainfall excess quantities by incremental time intervals of 6 hours or less should be presented in tabular form for each basin sub-area selected for the hydrograph computation. One or more representative hyetographs should be shown graphically by plotting of associated PMF hydrographs, in proper time relationship, with sufficient explanatory notes to show clearly the correlations between PMP increments and runoff.

The sequential arrangements of 24-hour incremental values of PMP, and sub-divisions of these into 6-hour increments (or less) for computation of PMF hydrographs must be compatible with meteorological characteristics affecting the specific basin. Guidance in determining the time sequences may be obtained from observed storms. The following guidelines are acceptable for the development of hyetographs representing PMP sequences derived from generalized enveloping depth-area-duration curves east of 105° longitude.

(1) Group the four heaviest 6-hour increments of PMP in a 24-hour sequence, the next highest four increments in a 24-hour sequence, etc.

(2) For the maximum 24-hour sequence, arrange the four 6-hour increments ranked 1, 2, 3, and 4 (maximum to minimum) in the order 4, 2, 1, 3

(3) Arrange the 24-hour sequences so that the highest period is near the end of the storm, and the second, third, etc., are distributed in a manner similar to (2) above.

(4) Subdivide the most intense 6-hour quantity during the maximum 24-hour PMP series into 1, 2, or 3-hour intervals, if trial computations show such subdivisions would produce significantly higher PMF hydrograph peaks at locations of interest in the studies involved. One-hour increments should equal 10, 12, 15, 38, 14, and 11 percent, respectively, of the maximum 6-hour PMP quantity. These one-hour percentages may be combined into successive two-hour or three-hour increments, if studies show these subdivisions give satisfactory results in hydrograph computations. Rainfall intensities may be assumed as uniform during all other 6-hour increments of the PMP series.

The Southwestern Division of the Corps of Engineers, Dallas, Texas, has studied a large number of storms and concluded that storms in that region have significantly greater intensity than is reflected by the above hourly percentages of 6-hour tota. Distributions made for basins in Texas, Oklahoma, New Mexico, and Southern Kansas may be obtained from hydrologic engineers at the Dallas office, Corps of Engineers. Typical distribution patterns suggested in HR 43 for the northwestern states are presented in Figure 4-9.

Other Aspects of PMP Derivation. The computation of probable maximum flood hydrographs for fairly large drainage basins usually requires determination of runoff from a number of basin subareas. The subareas selected should conform with requirements for unit hydrograph determinations, flood routing computations, and/or other hydrologic-hydraulic evaluations. They should be numbered for convenient reference, depicted on basin maps, and tabulated with areas in square miles and other pertinent data. Various combinations of the following reasons may influence selections of subareas for large basin studies:

1. Physical features of the drainage basin, such as topography, drainage patterns, exposure to air mass movements that affect PMP, and large variations in infiltration characteristics, natural hydraulic efficiencies of principal stream channels and floodways, valley storage capacities during major floods, and other conditions;

2. Major lakes and reservoirs that are capable of impounding or releasing large quantities of water during major floods;

3. Major channel improvements, lock and dam projects, levee systems, flow diversion structures, and other man-made works that influence movements of flows through principal drainage channels; and

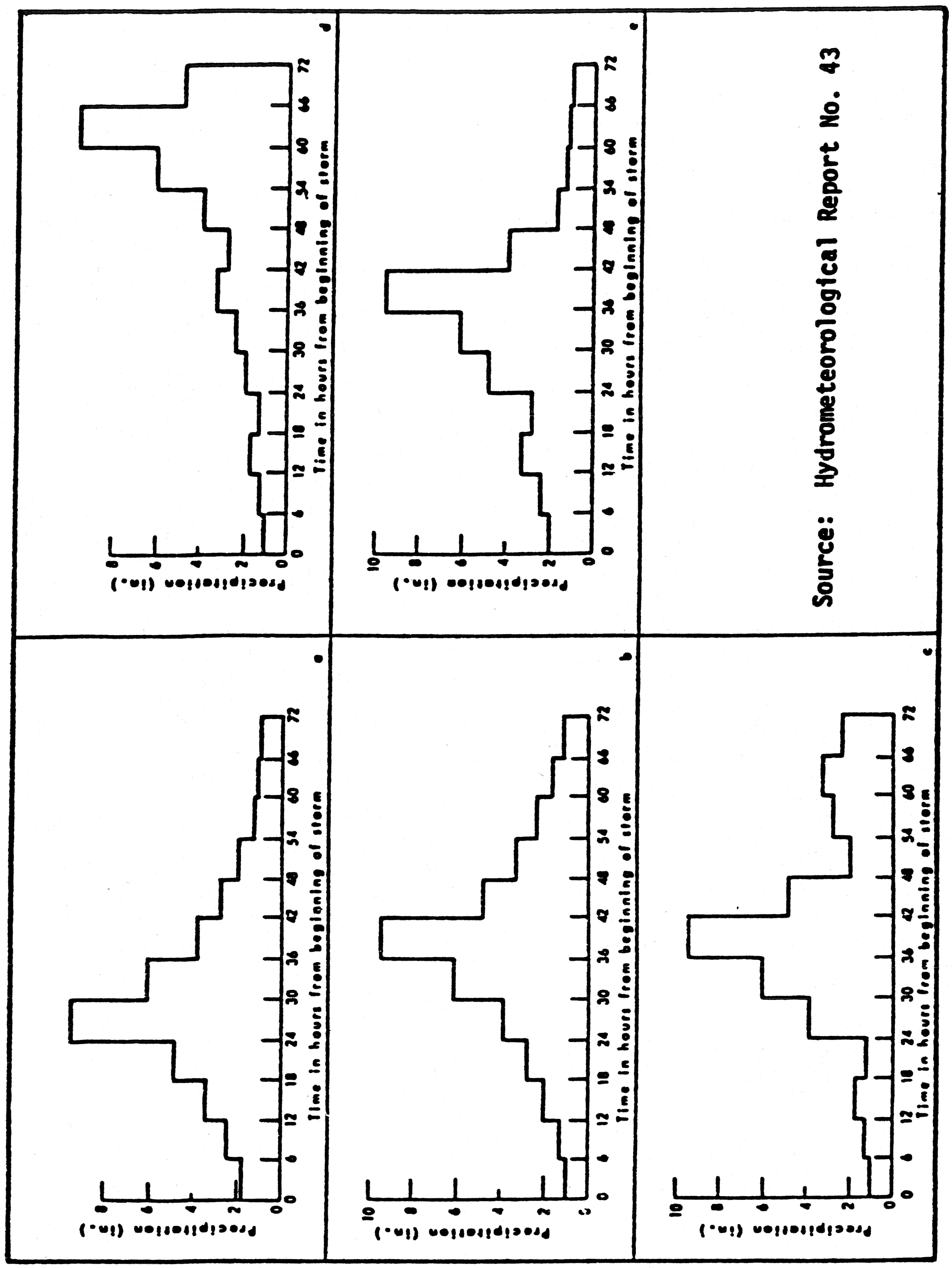

Figure 4-9. Typical PMP Time Distribution.

4. Locations along principal stream channels that have special significance in specific studies, such as cities, industrial centers, and proposed projects.

Areal Distribution of PMP in Small Basins. In utilizing generalized PMP estimates for drainage areas less than approximtely 1,000 square miles, it may be appropriate to assume a uniform depth over the entire drainage basin. However, if a breakdown by major basin subdivision is desirable in specific cases, higher concentrations of PMP intensities may be estimated by meteorologically sound procedures. For example, if there are valid reasons for assuming higher concentrations of PMP over, say, a 400 square mile subdivision of a 1,000 square mile area, computations could be as follows:

1. Determine total-storm PMP for the 1,000-square mile area from Report 51, as explained earlier, expressed in inches depth. In the same manner, determine the value for a 400-square mile area.

2. Convert the values obtained in step (1) to "inch-square miles," subtract to obtain the difference, and divide this by 600 to obtain the average total-storm PMP in inches depth over the 600-square mile subbasin.

Areal Distribution of PMP in Large Drainage Basins. Determinations of critical distributions of PMP quantities over large drainage areas (e.g., exceeding approximately 1,000 square miles) are more difficult to establish, and vastly more significant in estimating PMF hydrographs than in small basins. In large drainage basins located where infiltration capacities of the ground are high, variations in the areal distribution of precipitation during successive 6-hour periods of a total storm may reduce or increase estimates of total net runoff volumes by more than twofold, simply by changing opportunities for infiltration. The critical location of successive 6-hour PMP increments can also change the concentration of runoff at a particular location significantly. The existence of major reservoirs and other runoff controls in some drainage basins may have major influences on characteristics of PMF hydrographs associated with various areal distributions of PMP.

Accordingly, generalized estimates of PMP depth-area-duration relations for specific regions or drainage basins must be supplemented by appropriate analyses to establish critical areal distribution patterns of PMP from the beginning to the end of the overall storm. Normally a breakdown of areal distribution patterns by 6-hour intervals is suitable, in consideration of accuracy limitations in other facets of the computations. Shorter intervals may be needed in special cases.

Some PMP estimates for large drainage basins are based on "transportation" of major storms of record in the region, with certain adjustment in rainfall amounts to represent PMP quantities. In such cases, it is usual practice to retain generally the same isohyetal pattern in the transposed position, and the same chronological sequence of rainfall amounts at individual rainfall stations. These data provide a basis for estimating PMP amounts for any drainage basin subdivisions of interest in the specific studies, with a disaggregation by chronological increments of time needed for PMF hydrograph computations. This "storm transportation" technique provides the most convenient method of accounting for areal distributions of PMP, and the chronological sequence of occurrence.

Rainfall Excess Estimates. For computation of hydrographs of runoff from maximum precipitation, estimates of rainfall amounts exceeding infiltration and other losses are required for successive increments of time. These rainfall excess estimates should be related to basin subareas, in order to account for significant variations in areal distribution of PMP and any differences in infiltration characteristics of the areas. Loss factors tend to vary during successive periods of a rainfall sequence. Ground conditions that affect losses during the probable maximum storm should be the most severe that can reasonably exist in conjunction with maximum probable precipitation. Lowest loss rates that have been observed might be used if there is reasonable assurance that the entire range of possible losses has been experienced. However, loss estimates are subject to major uncertainties, and there are cases where negative loss rates are computed simply because of inadequate precipitation data. Accordingly, some allowance must be made for this uncertainty, and loss rates that are conservatively low should be selected for probable maximum flood computation. Where it is possible for the ground to be frozen at the start of a rainflood or snowmelt flood, it can be concluded that zero or near-zero loss rates should be used for probable maximum flood computation. There may exist a seasonal variation in minimum loss rates, in which case rates selected should be those representative of the most extreme conditions for the season for which probable maximum runoff is being computed. Typical values throughout the United States are in the range of .10 to 1.0 inch initial loss followed by a uniform rate of .05 to .15 inch per hour.

Probable Maximum Snowpack and Snowmelt. Guidance on probable maximum snow conditions is contained in U.S. Army Corps of Engineers Manual 1110-2-1406, "Runoff from Snowmelt", available from OCE Publications Depot, 890 South Pickett Street, Alexandria, VA 22304. Computations of probable maximum snowpack accumulation from winter precipitation, temperatures, and snowpack losses should be estimated from observed snowpack data and should exceed maximum observed accumulations. Adjustments to historical maximum snowpack to obtain probable maximum snowpack should generally be obtained from the National Weather Service.

In the case of rainfloods that have some snowmelt contribution, snowpack used for probable maximum rainflood computation should be the maximum that can contribute toward the peak flow and runoff volume of the flood without inhibiting the direct runoff from rainfall.

The critical snowpack in mountainous regions will ordinarily be located at elevations where most of the rainflood runoff originates. Snowpack is ordinarily greater at higher elevations and less at lower elevations, and hence critical snowpack will not exist at all elevations. Factors to be considered in selecting melt factors for the probable maximum snowmelt are discussed in the referenced EM 1110-2-1406 and in HR 43.

Probable Maximum Base Flow. Base flow for the probable maximum flood is not a critical item because the peak flow, which is not greatly affected by base flow, is the primary characteristic of interest in the probable maximum flood. Nevertheless, it is prudent to adopt a base flow value that is more severe than that which would be used for lesser floods.

Probable Maximum Flood Computation. Rainfall-runoff factors should be selected as the most severe that are reasonably consistent with the storm and flood conditions, and should be more severe than those that have been historically observed. Channel routing coefficients should likewise be modified toward greater translation speed and less storage effects because of the more efficient hydraulic flow conditions during larger floods.

In application of the probable maximum flood for spillway design associated with a large lake surface area, allowance should be made for the accelerating effect of a reservoir in relation to the stream reaches that are inundated, and the reservoir level at the start of the flood should be the highest level reasonable, consistent with probable maximum flood conditions.

Antecedent Conditions. In many spillway design applications, flood conditions that precede the probable maximum flood may have substantial influence on the regulatory effect that the reservoir has on the probable maximum flood. In such cases, it is appropriate to precede the probable maximum flood with a flood of major magnitude at a minimum time interval that is consistent with the causative meteorological conditions. While a special meteorological study is desireable where possible for this purpose, it is often considered that the start of a probable maximum flood reasonable be preceded by the start of a flood of 30 to 60 percent of the PMF by a period of 4 or 5 days.

Computer Programs. Many private and governmental engineering organizations have developed precipitation-runoff models which can be used to convert complex rainfall-snowmelt into discharge hydrographs. Some of the more well known models are:

- Stanford Watershed Model IV (Stanford University)
- Kentucky Watershed Model (University of Kentucky)
- ILLUDAS (Illinois State Water Survey)
- HSP (Hydrocomp International)
- MITCAT (Resource Analysis Inc.)
- SSAAR (Corps of Engineers)
- HEC-1 (Corps of Engineers)
- TR-20 (Soil Conservation Service)
- NWSRFS (National Weather Service)

Users of any of these models are cautioned to obtain proper guidance from experienced hydrologic engineers.

Discharge Exceedence Frequency Estimates

The standard for the "small" size, "low to significant" hazard category of dams allows a minimum design flood having an average return period of 50 to 100 years. More properly stated, this flood would have a 0.02 to 0.01 probability of exceedence in a given year. For reasons previously discussed, design standards will usually exceed this. Estimates of the magnitude of these events are determined by an analysis of the annual observed maximum events during the period of stream flow record at or near the damsite or from regional analysis of recorded events in hydrologically similar basins.

The United States Water Resources Council conducted an investigation of procedures and techniques for analyzing flood events in estimating flood flow frequencies at gaged, essentially unregulated, sites (Bulletin 17A Rev. June 1977). There are numerous other publications available on the subject as well as standard college text books in hydrology (IHD vol. 3, 1975), (Chow, 1964). The recommended distribution to use in the analylsis is the log Pearson Type III with a regional skew. The method of moments is used to determine the statistical parameters of the distribution from the observed annual series. Techniques for identifying and handling high and low outliers are also addressed in Bul. 17A. Computer programs designed to follow procedures recommended in Bul. 17A are available from HEC (Exhibit II), or from the U.S. Geological Survey (USGS), Reston, VA. Data on annual observed maxima at most stream gaging locations throughout the United States are published annually by the State Water agencies and in 5 year summary reports available at USGS offices, Corps of Engineer District libraries, and most State and large university libraries. The WATSTORE data storage system of the USGS is also a readily available source of annual peak discharge data.

Generalized Regional Procedures. The U.S. Geological Survey (USGS) in cooperation with State and other Federal water resources agencies have prepared regional procedures for estimating discharge frequency curves for ungaged areas. These may be in the form of an "index flood" method, regression equations, or other similar methods. These methods can be used for reconnaissance estimates but should be used with caution and considerable hydrologic judgment.

At run-of-river sites, these peak discharge estimates are used directly for spillway hydraulic design evaluation. If reservoir storage is significant, a total design hydrograph must be developed or a reduction of peak inflow to peak outflow can be based on engineering judgement to account for storage effects.

Reservoir Routing

After developing a reservoir inflow hydrograph representing the spillway design flood, the hydrograph must be routed through reservoir storage and spillway. Usually the "storage-indication", also called "modified Puls" method is used to accomplish this. This involves the basic continuity equation: Inflow - Outflow = Change in Storage, and a relationship between outflow and storage. Discussions of this procedure may be found in most hydrology textbooks and in a previously cited reference (USBR, 1977).

All established operating criteria are utilized in this routing, making use of all available outlets. The starting elevation for this flood routing should be normal pool level or, if there is authorized flood control space, additional studies should be made to evaluate the likelihood of a higher starting elevation.

Freeboard Allowance

Freeboard as used herein refers to the vertical height between some reference lake levels and the elevation of some specified feature of the dam (normally the top of the main non-overflow section).

Guidance on minimum freeboard allowance is not within the scope of this volume. Since the primary emphasis of this manual is on existing facilities, it is obvious that some criteria were applied at the time of design and construction. Federal and State agencies have generally not adopted a uniform criterion for "freeboard" requirements. However, parameters that should be considered include: (1) duration of sustained high water levels in the reservoir, (2) effective wind fetch and velocity, (3) reservoir depth, (4) embankment slope and roughness, (5) ability of the dam to resist erosion from overtopping, (6) hazard classification, and (7) reference level. Additional discussion and guidance on this aspect of dam safety can be found in *Design of Small Dams*, USBR, 1977.