Chapter 13
Analysis of Storm Events

13-1. Introduction

This chapter is concerned with the application of event-type simulation models for flood-runoff analysis. Such models are commonly used with frequency-based hypothetical storms to develop discharge-frequency estimates or with standard project or probable maximum storms to develop associated flood estimates. The chapter begins with a discussion of initial development of a simulation model. This is followed by consideration of methods for calibration/verification of the model. Applications issues associated with design storms are the focus of the remainder of the chapter.

13-2. Model Development

Steps in the initial development of a simulation model are as follows: assess data requirements and availability; acquire and process data; develop subbasin configuration of model; and develop initial estimates for model parameters.

a. Assessment of data requirements and availability.

(1) It is essential that the model developer be fully aware of the study objectives and requirements, including the intended use of modeling products. Types of data required for model development include

(a) historical precipitation and streamflow data;

(b) runoff-parameter data from past studies;

(c) data associated with watershed characteristics such as drainage areas, soil types, and land use;

(d) characteristics of rivers and other drainage-system (natural or artificial) features; and

(e) existence and characteristics of storage elements such as lakes, detention basins, etc.

(2) A field reconnaissance of the study basin should be performed. Information acquired from field observations can significantly enhance one's understanding of the runoff-response characteristics of the watershed and perhaps enable recognition of important watershed features that might otherwise be overlooked.

b. Acquisition and processing of data. Aspects of data management are treated in Chapter 17. Much precipitation and streamflow data are stored on electronic media, which can greatly facilitate data acquisition. It is generally desirable to place data in a data base and review it with graphics software. As the study proceeds, simulation results can also be stored in the data base, and utility software can be used to produce graphs, tables, etc. of key information. A careful review should be made of past studies and of the basis for all of the data being acquired.

c. Development of subbasin configuration.

(1) For most studies, it is necessary to divide a basin into subbasins to enable development of information at locations of interest and to better represent spatially variable runoff characteristics. A subbasin outlet should be located:

(a) at each stream location where discharge estimates are required,

(b) at each stream gauge, and

(c) at dams and other significant hydraulic structures.

(2) Nondistributed models use lumped (spatially averaged) values for precipitation and loss (infiltration) parameters. Subbasins should be sufficiently small so that spatial-averaging of this information is reasonable. Basin subdivision may also be performed to tailor rainfall-runoff transformations to particular land-use conditions. For example, rural and urban portions of a basin might be represented separately. If flood-damage or other model-dependent analyses are to be performed, subbasin delineation should be coordinated with the users of model results. There may be reasons other than hydrologic that affect the choice of locations of subbasin outlets.

d. Development of initial estimates for parameter values.

(1) After defining the subbasin configuration, a skeleton input file can be developed which contains all required information (such as drainage areas, subbasin linkages, etc.) except values for runoff parameters. Such parameters might be required for defining unit hydrograph, kinematic wave, loss-rate, base-flow, or routing relationships. At this point, initial estimates of values for runoff parameters can be made and entered into the input file. Estimates can be derived from

(a) past studies,

(b) application of previously developed regional relationships, and

(c) physical characteristics of the subbasins.

(2) If there were no streamflow data available for the basin, there may be little basis for improving the initial estimates. However, generally, there are some streamflow data for locations in or near the basin which can be used in a calibration process to improve the initial estimates.

13-3. Model Calibration

Calibration here refers to the process of using historical precipitation and streamflow data to develop values for runoff parameters. Verification refers to the testing of calibrated values, generally with data not used for calibration. Topics in this section pertain to calibration strategy, selection of historical events, calibration techniques, and model verification.

a. Calibration strategy. Calibration of simulation models must be done carefully with due consideration for the reliability of historical data and for the simplistic nature of model components used to represent complex physical processes in heterogeneous basins. The insight that an experienced analyst brings to bear in accommodating these factors is, in many cases, the single most important element of the calibration process.

(1) The calculation of a discharge hydrograph at a location in a basin may be a function of few or many runoff parameters. A headwater subbasin is one for which there are no subbasins upstream. The simulation of runoff from a headwater subbasin is a function of parameters associated solely with that subbasin. Calculated runoff for the outlet of a downstream subbasin is a function not only of the parameters of the subbasin, but also of those for all upstream subbasins and routing reaches. For this reason, the calibration of values of parameters for gauged headwater subbasins is often more direct and reliable than calibration associated with downstream gauges.

(2) In a multisubbasin model, subbasins with streamgauges at the outlet are generally a small proportion of the total number of subbasins. Hence, the general approach is to first calibrate parameter values for all gauged headwater subbasins and to use the results as an aid in setting or adjusting values for all other subbasins. The next (and generally most difficult) step is to review calculated versus simulated results at all downstream gauges and to manually adjust key parameter values to provide basin-wide simulations that are as reasonable and consistent as possible.

b. Selection of historical events. Model components that employ unit hydrographs or other linear entities produce outputs proportional to inputs. Because watersheds do not respond in a truly linear manner, the events chosen for calibration and verification should, if possible, be consistent in magnitude with the magnitude of hypothetical events to which the model will be applied. In many cases, this is not feasible because the hypothetical events are more rare than those that have been experienced historically. Nevertheless, the largest historical events for which data are available generally provide the best basis for calibration/verification.

(1) In addition to the size of a historical event, the state of the basin at the time of occurrence is significant. The model must represent land-use and other conditions consistent with the time of occurrence of the historical event. If existing basin conditions are of primary interest and a historical event occurred when the basin conditions were markedly different, the event may be of little value for calibration.

(2) Also important are the amount and quality of data associated with historical events. If precipitation data are lacking or if only daily values are available and a model with small subbasins is being calibrated, an event may be of limited value for calibration.

(3) In general, it is desirable to use several events (say, four to six) for calibration. It is also desirable to reserve a couple of events for verification. Sometimes the amount of useful data is limited so that there are few events for calibration and no events for verification.

c. Calibration techniques for gauged headwater basins. Computer software can be used for automated calibration of parameter values for gauged headwater subbasins. Figure 7-7 shows in simple terms the procedure that may be used. As may be noted, it is necessary to specify initial values for the parameters to be optimized. The simulation is performed with these values and the results compared with the observed discharge hydrograph. The quantitative measure of goodness of fit, the *objective function*, is often defined in terms of a root mean square error, where error is the difference between computed and observed discharge ordinates. For flood-runoff analysis, the errors may be weighted with a function that gives more weight to higher flows than lower flows, as illustrated in Equation 7-18 in paragraph 7-3*e*.

(1) Parameter values are adjusted in automatic calibration to minimize the magnitude of the objective function. Because of interdependence between parameters and other factors, a global minimum is not always achieved, which results in suboptimal values for parameters. Another aspect of calibration is that constraints on acceptable parameter values are often imposed. For example, negative loss rates would be unreasonable. Parameter values obtained by calibration should be reviewed carefully; values that are unreasonable or inconsistent should be rejected. Generally, the quality of fit between the observed and computed hydrographs is best judged by reviewing plots of the hydrographs and associated rainfall and rainfall-excess hyetographs, rather than simply looking at statistical measures of the fit.

(2) The analyst should thoroughly understand the optimization procedure being implemented and have sufficient output information to enable verification of its performance. Suboptimal results can sometimes be improved by reoptimization with different initial conditions, restricting the optimization region, or other means.

(3) The parameter values optimized for each historical event will be unique. Criteria are required for choosing a single set of values to represent the runoff characteristics of the subbasin. Consideration should be given to factors such as

(a) the quality of fit between the observed and computed hydrographs,

(b) the magnitude of the event, and

(c) the quality of the precipitation and streamflow data for the event.

Generally, estimates based on the larger events would be given more weight if the calibrated model is intended for application to rare events. Once a set of parameter values has been adopted, the historical events should be rerun with these values. Further refinement may be needed to achieve the best compromise in matching available data.

d. Calibration techniques for downstream gauges. The calibration process for downstream locations involves simulating runoff at each streamgauge and ascertaining what parameter-value adjustments, if any, should be made for upstream subbasins and/or routing reaches. The calibration should be performed starting at the upstream gauges and working downstream. Adjustments should generally not be tailored to any one event. Rather, the model performance should be judged for all calibration events. When a consistent bias is noted, for example if the timing of runoff is consistently too early or too late, the most likely cause of the bias should be sought and the model adjusted accordingly. Often, poor results are due to erroneous definition of precipitation or other data problems. If the problems cannot be reconciled, the data should be rejected for calibration purposes. Numerous simulations may be required to determine a final set of parameter values that are most reasonably consistent with knowledge of the basin and the data associated with the calibration events.

e. Verification. Verification enables assessment of the reliability of the calibrated model. It is performed by simulating historical events not used for calibration. With an event-type model, there is always uncertainty associated with loss rates, and they are critical in their impact on runoff volumes. For purposes of verification, the antecedent rainfall-runoff conditions should be assessed and loss rates chosen that are consistent with similar antecedent conditions associated with calibration events. Adjustment of the loss rates may be required to obtain reasonable agreement with the observed runoff volumes. Once this agreement has been achieved, a critical assessment of the simulated results can be made. Good agreement between simulated and observed hydrographs engenders confidence in the model performance, at least for events similar in magnitude to those simulated. If results are poor, reasons for such results should be ascertained, if possible. Parameter-value modifications required to produce reasonable simulations of the verification events should be determined. If such modifications can be made without significant degradation of the results obtained for the calibration events, the modifications can be adopted. If degradation of calibration results would occur, it may be appropriate to redo the calibration with incorporation of the verification events. In either case, the poor results are cause for associating a higher level of uncertainty with model application.

13-4. Simulation of Frequency-Based Design Floods

Event-type models are commonly used with frequency-based hypothetical storms for the development of discharge-frequency estimates. Issues discussed in this section include design-storm definition, depth-area adjustments, and association of runoff frequency with rainfall frequency. Other issues such as transfer of frequency information from gauged to ungauged locations, conversion of nonstationary to stationary peak discharges, and development of future-condition frequency estimates are discussed in Chapter 17.

a. Design-storm definition.

(1) The NOAA has published generalized rainfall criteria for the United States. Appendix A lists a number of these publications. The criteria consist of maps with isopluvial lines of point precipitation for various frequencies and durations. Generally, the maps for mountainous regions are substantially more detailed because of orographic effects.

(2) The rainfall depths obtained from NOAA criteria are point values commonly assumed to apply up to 10 square miles. For larger areas, the average precipitation over the area is less than the value for a point, and adjustments are required. Figure 13-1 shows adjustment criteria provided in NOAA publications.

(3) The rainfall depths from NOAA criteria are based on a *partial duration series*. If value of the annual series is desired, adjustment factors are applied to recurrence intervals of 10 years or less. No adjustment is applied for larger recurrence intervals larger than 10 years, as the two series essentially merge at that recurrence interval.

(4) The NOAA criteria do not contain specific guidance for establishing the temporal distribution of design-storm rainfall. A common approach is to arrange the rainfall to form a *balanced* hyetograph; that is, the depth associated with each duration interval of the storm satisfies the relation between depth and duration for a given frequency. For example, for a 1 percent-chance (100-year) 24-hr storm, the depths for the peak 30-min, 1-hr, 2-hr, ..., 24-hr durations would each equal the 1 percent-chance depth for that duration. Although such storms do not preserve the random character of natural storms, use of a balanced storm ensures an appropriate depth (in terms of frequency), regardless of the time-response characteristics of a particular river basin.

(5) The SCS has developed four 24-hr synthetic rainfall distributions (USDA 1986) from available National Weather Service duration-frequency data. Types I and IA represent the Pacific maritime climate with wet winters and dry summers. Type III represents Gulf of Mexico and Atlantic coastal areas where tropical storms bring large 24-hr rainfall amounts. Type II represents the rest of the country. Other approaches for defining the temporal distribution of design storms are reported in the literature. If none of the synthetic distributions are applicable to the area being modeled, the hydrologist should look at historical information, as well as regional data, to develop an adequate temporal distribution.

b. Depth-area considerations.

(1) The area-adjustment criteria of Figure 13-1 have a nonlinear effect on storm hyetographs. That is, a balanced hyetograph for one storm size is not a simple proportion of a balanced hyetograph for a different storm size. Each storm size will have its own unique depth and temporal distribution. This creates a problem in situations where it is desired to develop a consistent set of frequency estimates for numerous sites in a basin. It would be necessary to develop a unique storm hyetograph for every location. For a basin with many subbasins and stream junctions, the computational requirements could be substantial.

(2) An approach for dealing with this situation is based on calculating *index* discharge hydrographs at each location of interest from a set of *index* hyetographs for storm areas that encompass the full range of drainage areas from the area of the smallest subbasin to the total basin area. The hydrograph for a given location is obtained by interpolating, based on drainage area, between two index hydrographs for that location. This is illustrated in Figure 13-2.

(3) A semi-logarithmic interpolation equation (used in computer program HEC-1) is as follows:

$$Q = Q1 \left(\frac{\log \frac{A2}{Ax}}{\log \frac{A2}{A1}} \right) + Q2 \left(\frac{\log \frac{Ax}{A1}}{\log \frac{A2}{A1}} \right) \qquad (13\text{-}1)$$

where

Q = instantaneous discharge for the interpolated hydrograph

Ax = drainage area represented by the interpolated hydrograph

$A1$ = index drainage area that is closest to, but smaller than, Ax

$A2$ = index hydrograph closest to, but larger than, Ax

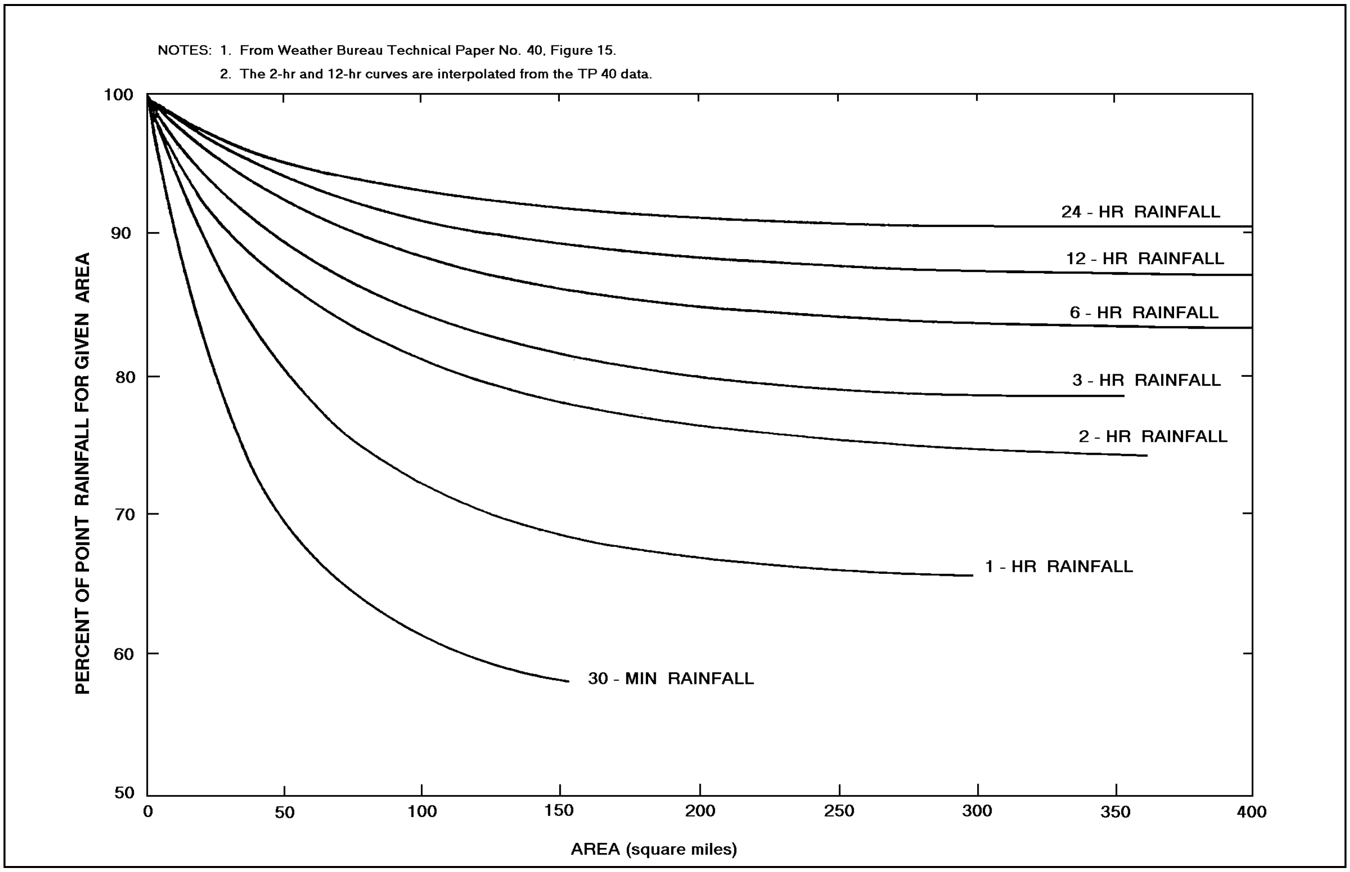

Figure 13-1. Area-adjustment of point rainfall

$Q1$ = instantaneous discharge for the index hydrograph corresponding to $A1$

$Q2$ = instantaneous discharge for the index hydrograph corresponding to $A2$

An illustration of this approach is given in the HEC-1 User's Manual.

c. Association of runoff frequency with rainfall frequency. Although the NOAA rainfall criteria associate frequency with depth, it does not follow that the same frequencies should be associated with the design storms or the calculated flood-runoff.

(1) In addition to rainfall, runoff is a function of loss rates and base flow, the magnitudes of which vary with time and antecedent moisture conditions. A very dry antecedent condition associated with a 100-year storm might produce runoff with a significantly smaller recurrence interval.

(2) Because of the uncertainty of the frequency of design-storm runoff, it is best to utilize statistically based frequency information (for locations with at least 10 years of streamflow data) wherever possible to 'calibrate' the exceedance frequency to associate with particular combinations of design storms and loss rates. This important concept is discussed further in Chapter 17.

13-5. Simulation of Standard Project and Probable Maximum Floods

Standard project and probable maximum floods are used as design events and also as reference events for comparison with flood magnitudes developed by other means. They are generally developed by simulation (with an event-type model) of runoff from design storms. The events represent very rare occurrences, generally well beyond the range of events for which reliable frequency estimates (from statistically based frequency curves) could be made. This section defines each design flood and discusses issues associated with their derivation.

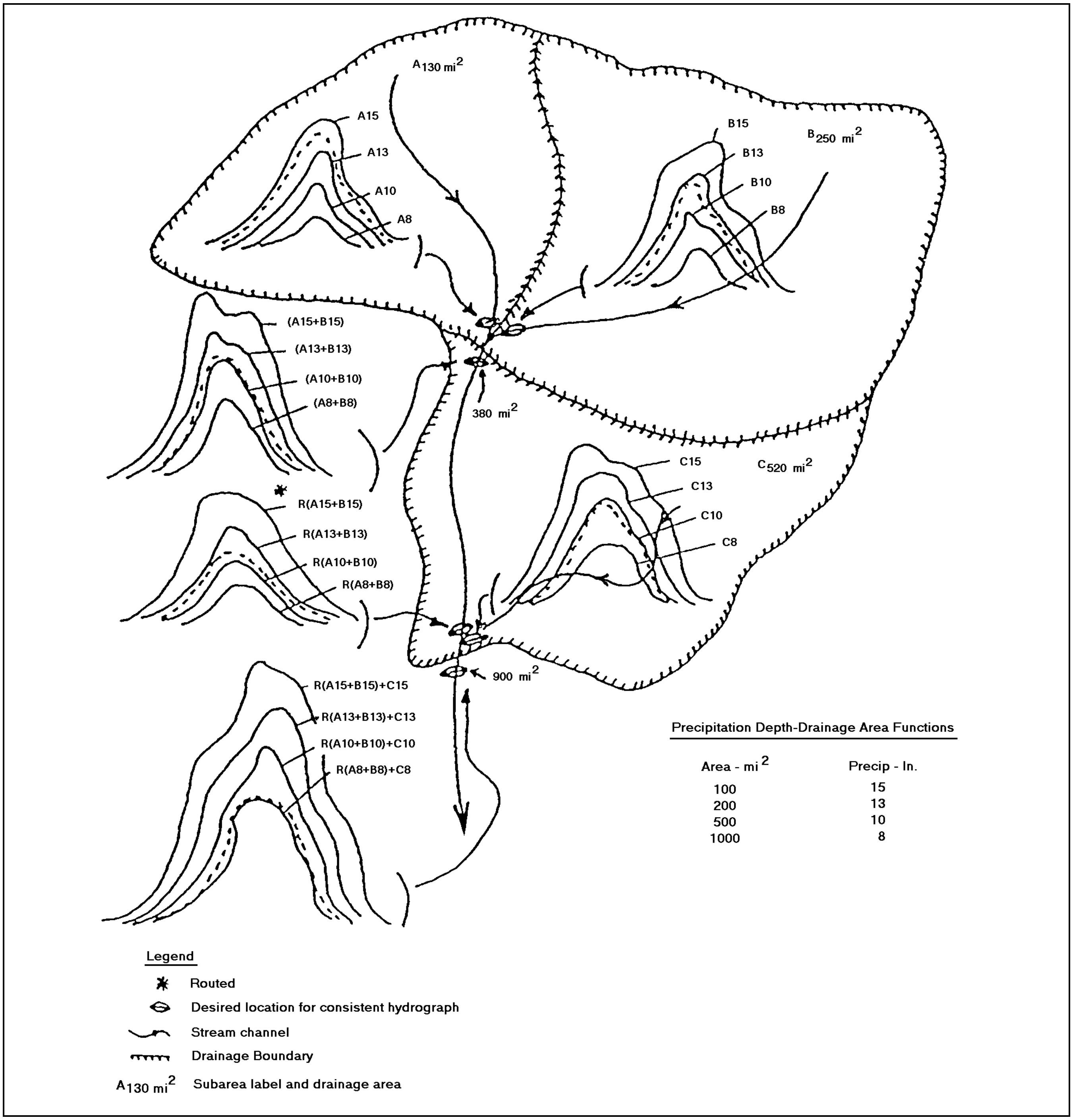

Figure 13-2. Index and interpolated hydrographs

a. Standard project flood. The standard project flood (SPF) is the flood that can be expected from the most severe combination of meteorologic and hydrologic conditions that are considered *reasonably characteristic* of the region in which the study basin is located. The SPF is generally based on analysis (and transposition) of major storms that have occurred in the region and selection of a storm magnitude and temporal distribution that is as severe as any of the transposed storms, with the possible exception of any storm or storms that are exceptionally larger than others and are considered to be extremely rare. Studies compiled in the United States indicate that SPF peak discharges are usually of the order of 40 to 60 percent of probable maximum peak discharges.

(1) The SPF is intended as a practicable expression of the degree of protection to be considered for situations where protection of human life and high-valued property is required, such as for an urban levee or floodwall. It also provides a basis of comparison with the recommended protection for a given project. Although a specific fre quency cannot be assigned to the SPF, a return period of a few hundred to a few thousand years is commonly associated with it.

(2) Because the standard project storm (SPS) is not widely used outside the USACE, only a limited number of publications describe its derivation and use. EM 1110-2-1411 describes SPS derivation for the United States east of the 105° longitude. Computer program HEC-1 contains an option for automatically applying this criteria. SPS development for the remainder of the United States is based on various published and unpublished Corps reports and procedures. Sometimes the SPF is developed as a proportion (e.g., 50 percent) of the probable maximum flood.

(3) Associated with SPF simulation is the choice of loss rate and base flow parameter values and perhaps antecedent snowpack and related information. Loss rates and base flow should be commensurate with values considered reasonably likely to occur during storms of such magnitude. They should be estimated on the basis of rates observed in floods that have occurred in the basin or in similar areas. EM 1110-2-1406 is a source of information relating to snowpack and snowmelt assumptions to associate with an SPF.

b. Probable maximum flood. The probable maximum flood (PMF) is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are *reasonably possible* in the region. It is used in the design of projects for which virtually complete security from flood-induced failure is desired. Examples are the design of dam height and spillway size for major dams and protection works for nuclear power plants.

(1) The PMF is calculated from a probable maximum storm (PMS), generally with an event-type model. The PMS is based on probable maximum precipitation (PMP), criteria developed by the Hydrometeorological Branch of the Office of Hydrology, NWS. Figure 13-3 shows regions of the contiguous United States for which generalized PMP criteria have been developed. The hydrometeorological reports shown in the figure are listed in the references. Hydrometeorological Report (HMR) No. 52 (U.S. Department of Commerce 1982) provides criteria for developing a PMS based on PMP criteria from Reports No. 51 and 55 (U.S. Department of Commerce 1978 and 1988) (for the United States east of longitude 105°). A computer program (USACE 1984b) has been developed to apply the criteria in Report No. 52. Hydrometeorological criteria are being updated for various areas of the country. A check should be made for the most recently available criteria prior to performing a study. In regions where there are strong orographic influences, it is sometimes desirable for basin-specific criteria to be developed. Such studies require considerable time and dollar resource commitments, and their need should be well established. The Hydrometeorological Branch of the NWS is partially funded by the USACE and is available to serve in a consulting capacity.

(2) The technical basis for PMP estimation is described in the various hydrometeorological reports. NOAA Technical Report NWS 25 contains maps indicating storms of record that produced rainfall within 50 percent of PMP. (Other maps show the ratio of point PMP to 100-year values.) Such information shows that PMP values are consistent with reasonable extrapolation of the major storms of record; in some cases, the extrapolation is less than about 10 percent.

(3) Ground conditions that affect losses during the PMS should be the most severe that can reasonably exist in conjunction with such an event. The lowest loss rates that have been developed for historical storms might be used if there is reasonable assurance that such storms represent severe conditions. Where it is possible for the ground to be frozen at the start of a rain flood or snowmelt flood, it can be concluded that zero or near-zero loss rates should be used. If there is a seasonal variation in minimum loss rates, the values selected should be representative of extreme conditions for the season for which the PMF is being developed.

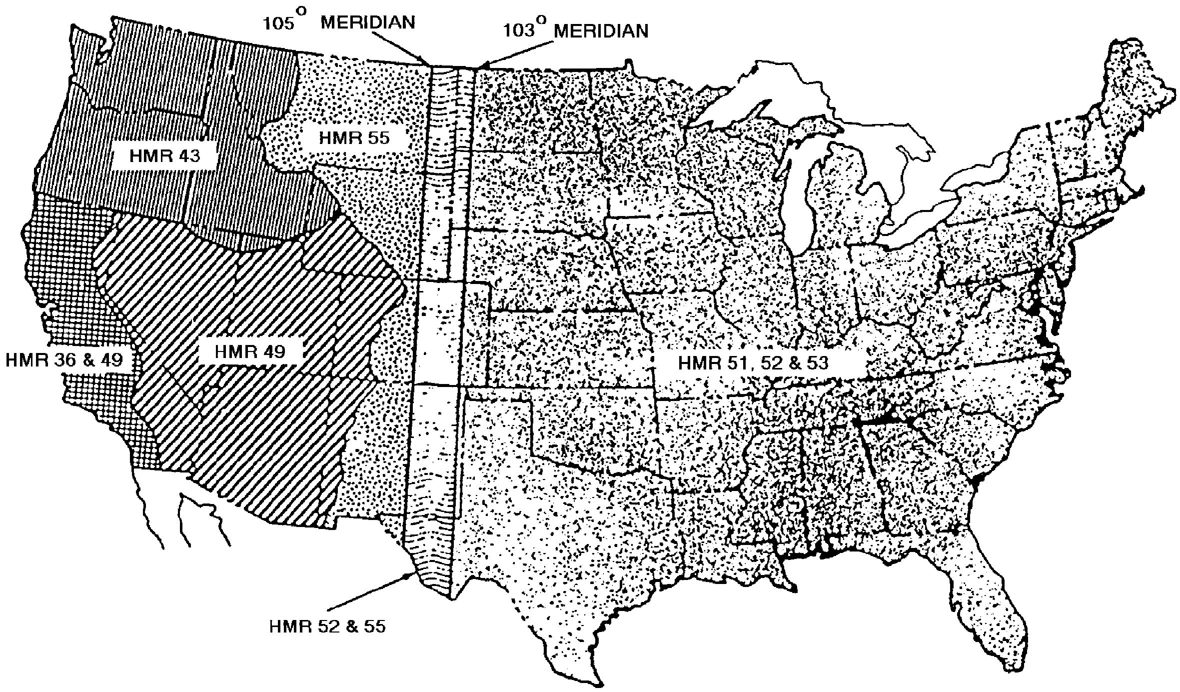

Figure 13-3. Regions covered by generalized PMP studies

(4) For situations where snowpack/snowmelt is a factor, it is generally not feasible to compute maximum snowpack accumulation from winter precipitation, temperature, and snowmelt losses. Rather, a probable maximum snowpack for floods that are primarily snowmelt floods can be estimated by extrapolation of historical snowpack data. In the case of rain floods that have some snowmelt contribution, snowpack used for probable maximum rain-flood computation should be the maximum that can contribute to 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 rain-flood 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 temperature sequences for snowmelt simulation are discussed in EM 1110-2-1406.

(5) Runoff parameter values used for the transformation of rainfall/snowmelt to runoff should be appropriate for the magnitude of the event being simulated. Travel times tend to be significantly shorter during major events. Indices of travel time, such as values for unit hydrograph parameters and routing coefficients, are frequently adjusted downward from their magnitudes based on historical events to reflect the severe conditions. In applications for spillway design, allowance should be made for the acceleration effect of a reservoir in relation to the stream reaches that are inundated.

(6) In spillway design applications, flood conditions that precede the PMF may have substantial influence on the regulatory effect of the reservoir. In such cases, it is appropriate to precede the PMF with a flood of major magnitude at a time interval that is consistent with the causative meteorological conditions. While a special meteorological study is desirable for this purpose, it is often assumed that the PMF is preceded by a SPF 4 or 5 days earlier.