CHAPTER 3

FLOOD FREQUENCY ANALYSIS

3-1. Introduction.

The procedures that federal agencies are to follow when computing a frequency curve of annual flood peaks have been published in Guidelines for Determining Flood Flow Frequency, Bulletin 17B (46). As stated in Bulletin 17B, "Flood events ... do not fit any one specific known statistical distribution." Therefore, it must be recognized that occasionally, the recommended techniques may not provide a reasonable fit to the data. When it is necessary to use a procedure that departs from Bulletin 17B, the procedure should be fundamentally sound and the steps of the procedure documented in the report along with the frequency curves.

This report contains most aspects of Bulletin 17B, but in an abbreviated form. Various aspects of the procedures are described in an attempt to clarify the computational steps. The intent herein is to provide guidance for use with Bulletin 17B. The step by step procedures to compute a flood peak frequency curve are contained in Appendix 12 of Bulletin 17B and are not repeated herein.

3-2. Log-Pearson Type III Distribution.

a. General. The analytical frequency procedure recommended for annual maximum streamflows is the logarithmic Pearson type III distribution. This distribution requires three parameters for complete mathematical specification. The parameters are: the mean, or first moment, (estimated by the sample mean, X); the variance, or second moment, (estimated by the sample variance, S^2); the skew, or third moment, (estimated by the sample skew, G). Since the distribution is a logarithmic distribution, all parameters are estimated from logarithms of the observations, rather than from the observations themselves. The Pearson type III distribution is particularly useful for hydrologic investigations because the third parameter, the skew, permits the fitting of non-normal samples to the distribution. When the skew is zero the log-Pearson type III distribution becomes a two-parameter distribution that is identical to the logarithmic normal (often called log-normal) distribution.

b. Fitting the Distribution.

(1) The log-Pearson type III distribution is fitted to a data set by calculating the sample mean, variance, and skew from the following equations:

$$\overline{X} = \frac{\sum X}{N} \qquad (3\text{-}1)$$

$$S^2 = \frac{\sum x^2}{N-1} = \frac{\sum (X-\overline{X})^2}{N-1} \qquad (3\text{-}2a)$$

$$= \frac{\sum X^2 - (\sum X)^2/N}{N-1} \tag{3-2b}$$

$$G = \frac{N(\sum x^3)}{(N-1)(N-2)S^3} = \frac{N(\sum (X-\overline{X})^3)}{(N-1)(N-2)S^3} \tag{3-3a}$$

$$= \frac{N^2(\sum X^3) - 3N(\sum X)(\sum X^2) + 2(\sum X)^3}{N(N-1)(N-2)S^3} \tag{3-3b}$$

in which:

$\overline{X}$ = mean logarithm

X = logarithm of the magnitude of the annual event

N = number of events in the data set

S^2 = unbiased estimate of the variance of logarithms

x = $X-\overline{X}$, the deviation of the logarithm of a single event from the mean logarithm

G = unbiased estimate of the skew coefficient of logarithms

The precision of the computed values is more sensitive to the number of significant digits when Equations 3-2b and 3-3b are used.

(2) In terms of the frequency curve itself, the mean represents the general magnitude or average ordinate of the curve, the square root of the variance (the standard deviation, S) represents the slope of the curve, and the skew represents the degree of curvature. Computation of the unadjusted frequency curve is accomplished by computing values for the logarithms of the streamflow corresponding to selected values of percent chance exceedance. A reasonable set of values and the results are shown in Table 3-1. The number of values needed to define the curve depends on the degree of curvature (i.e., the skew). For a skew value of zero, only two points would be needed, while for larger skew values all of the values in the table would ordinarily be needed.

(3) The logarithms of the event magnitudes corresponding to each of the selected percent chance exceedance values are computed by the following equation:

$$\log Q = \overline{X} + KS \tag{3-4}$$

where $\overline{X}$ and S are defined as in Equations 3-1 and 3-2 and where log Q = logarithm of the flow (or other variable) corresponding to a specified value of percent chance exceedance

K = Pearson type III deviate that is a function of the percent chance exceedance and the skew coefficient.

c. Example Computation.

(1) As shown in the following example, Equation 3-4 is solved by using the computed values of $\bar{X}$ and S and obtaining from Appendix V-3 the value of K corresponding to the adopted skew, G, and the selected percent chance exceedance (P). An example computation for P=1.0, where $\bar{X}$, S and G are taken from Table 3-1, is:

$$\log Q = 3.3684 + 2.8236\,(.2456)$$
$$= 4.0619$$
$$Q = 11500 \text{ cfs}$$

(2) It has been shown (36) that a frequency curve computed in this manner is biased in relation to average future expectation because of uncertainty as to the true mean and standard deviation. The effect of this bias for the normal distribution can be eliminated by an adjustment termed the expected probability adjustment that accounts for the actual sample size. This adjustment is discussed in more detail in Section 3-4. Table 3-1 and Figure 3-1 shows the derived frequency curve along with the expected probability adjusted curves and the 5 and 95 percent confidence limit curves.

Table 3-1. Computed Frequency Curve and Statistics.

-FREQUENCY CURVE- 01-3735 FISHKILL CREEK AT BEACON, NEW YORK

FLOW, CFS COMPUTED	FLOW, CFS EXPECTED PROBABILITY	PERCENT CHANCE EXCEEDANCE	CONFIDENCE LIMITS 0.05 LIMIT	CONFIDENCE LIMITS 0.95 LIMIT
19200.	28300.	0.2	39100.	12300.
14500.	19000.	0.5	26900.	9740.
11500.	14100.	1.0	20100.	8080.
9110.	10500.	2.0	14800.	6640.
7100.	7820.	4.0	10800.	5380.
4960.	5210.	10.0	6850.	3950.
3650.	3740.	20.0	4710.	2990.
2190.	2190.	50.0	2650.	1790.
1440.	1420.	80.0	1760.	1110.
1200.	1170.	90.0	1490.	884.
1040.	1010.	95.0	1320.	746.
841.	791.	99.0	1100.	568.

FREQUENCY CURVE STATISTICS		STATISTICS BASED ON	
MEAN LOGARITHM	3.3684	HISTORIC EVENTS	0
STANDARD DEVIATION	0.2456	HIGH OUTLIERS	0
COMPUTED SKEW	0.7300	LOW OUTLIERS	0
GENERALIZED SKEW	0.6000	ZERO OR MISSING	0
ADOPTED SKEW	0.7000	SYSTEMATIC EVENTS	24

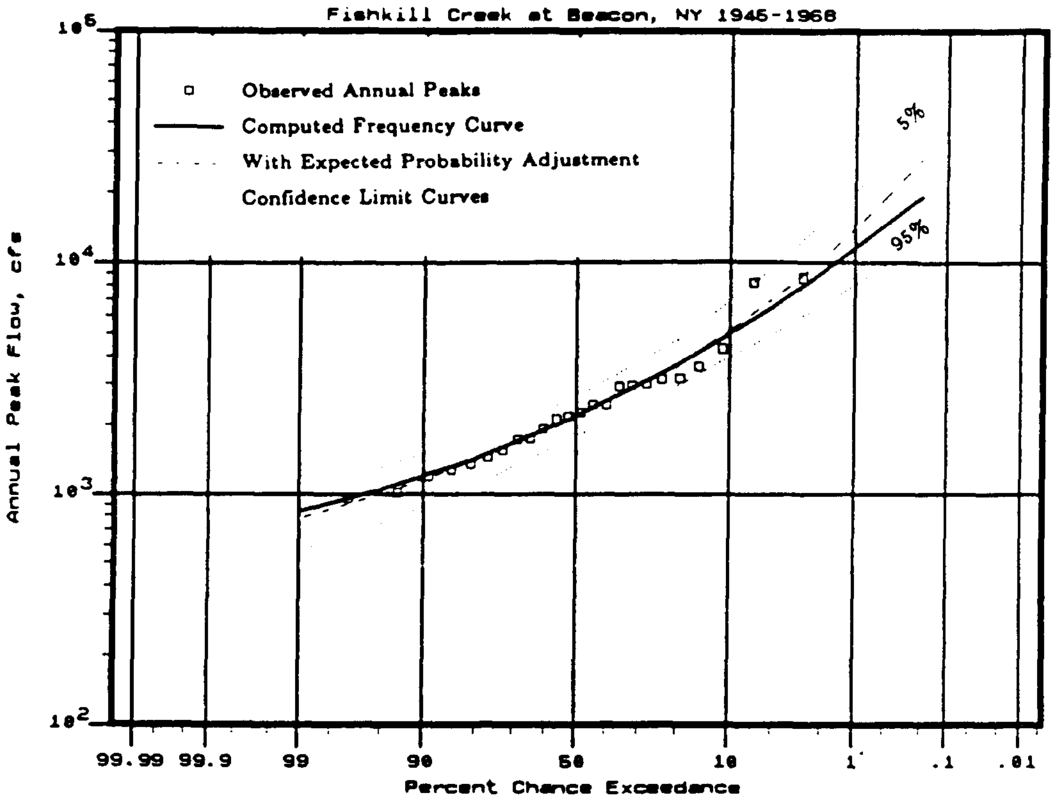


Figure 3-1. Annual Frequency Curve.

d. Broken Record. A broken record results when one or more years of annual peaks are missing for any reason not related to the flood magnitude. In other words, the missing events were caused by a random occurrence. The gage may have been temporarily discontinued for budgetary or other reasons. The different segments of the record are added together and analyzed as one record, unless the different parts of the record are considered non-homogeneous. If a portion of the record is missing because the gage was destroyed by a flood or the flood was too low to record, then the observed events should be analyzed as an incomplete record.

e. Incomplete Record. An incomplete record can result when some of the peak flow events were either too high or too low. Different analysis procedures are recommended for missing high events and for missing low events. Missing high events may result from the gage being out of operation or the stage exceeding the rating table. In these cases, every effort should be made to obtain an estimate of the missing events. Missing low floods usually result when the flood height is below the minimum reporting level or the bottom of a crest stage gage. In these cases, the record should be analyzed using the conditional probability adjustment described in Appendix 5 of Bulletin 17B and Section 3-6 of this report.

f. Zero-flood years. Some of the gaging stations in arid regions record no flow for the entire year. A zero flood peak precludes the normal statistical analysis because the logarithm of zero is minus infinity. In this case the record should be analyzed using the conditional probability adjustment described in Appendix 5 of Bulletin 17B and Section 3-6 of this report.

g. Outliers.

(1) Guidance. The Bulletin 17B (46) defines outliers as "data points which depart significantly from the trend of the remaining data." The sequence of steps for testing for high and low outliers is dependent upon the skew coefficient and the treatment of high outliers differs from that of low outliers. When the computed (station) skew coefficient is greater than +0.4, the high-outlier test is applied first and the adjustment for any high outliers and/or historic information is made before testing for low outliers. When the skew coefficient is less than -0.4, the low-outlier test is applied first and the adjustment for any low outlier(s) is made before testing for high outliers and adjusting for any historic information. When the skew coefficient is between -0.4 and +0.4, both the high- and low-outlier tests are made to the systematic record (minus any zero flood events) before any adjustments are made.

(2) Equation. The following equation is used to screen for outliers:

$$\overline{X}_o = X \pm K_N S \tag{3-5}$$

where:

X_o = outlier threshold in log units

$\overline{X}$ = mean logarithm (may have been adjusted for high or low outliers, and/or historical information depending on skew coefficient)

S = standard deviation (may be adjusted value)

K_N = K value from Appendix 4 of Bulletin 17B or Appendix F, Table 11 of this report. Use plus value for high-outlier threshold and minus value for low-outlier threshold

N = Sample size (may be historic period (H) if historically adjusted statistics are used)

(3) High Outliers. Flood peaks that are above the upper threshold are treated as high outliers. The one or more values that are determined to be high outliers are weighted by the historical adjustment equations. Therefore, for any flood peak(s) to be weighted as high outlier(s), either historical information must be available or the probable occurrence of the event(s) estimated based on flood information at nearby sites. If it is not possible to obtain any information that weights the high outlier(s) over a longer period than that of the systematic record, then the outlier(s) should be retained as part of the systematic record.

(4) Low Outliers. Flood peaks that are below the low threshold value are treated as low outliers. Low outliers are deleted from the record and the frequency curve computed by the conditional probability adjustment (Section 3-6). If there are one or more values very near, but above the threshold value, it may be desirable to test the sensitivity of the results by considering the value(s) as low outlier(s).

h. Historic Events and Historical Information.

(1) Definitions. Historic events are large flood peaks that occurred outside of the systematic record. Historical information is knowledge that some flood peak, either systematic or historic, was the largest event over a period longer than that of the systematic record. It is historical information that allows a high outlier to be weighted over a longer period than that of the systematic record.

(2) Equations. The adjustment equations are applied to historic events and high outliers at the same time. It is important that the lowest historic peak be a fairly large peak, because every peak in the systematic record that is equal to or larger than the lowest historic peak must be treated as a high outlier. Also a basic assumption in the adjusting equations is that no peaks higher than the lowest historic event or high outlier occurred during the unobserved part of the historical period. Appendix D in this manual is a reprint of Appendix 6 from Bulletin 17B and contains the equations for adjusting for historic events and/or historical information.

3-3. Weighted Skew Coefficient.

a. General. It can be demonstrated, either through the theory of sampling distributions or by sampling experiments, that the skew coefficient computed from a small sample is highly unreliable. That is, the skew coefficient computed from a small sample may depart significantly from the true skew coefficient of the population from which the sample was drawn. Consequently, the skew coefficient must be compared with other representative data. A more reliable estimate of the skew coefficient of annual flood peaks can be obtained by studying the skew characteristics of all available streamflow records in a fairly large region and weighting the computed skew coefficient with a generalized skew coefficient. (Chapter 9 provides guidelines for determining generalized skew coefficients.)

b. Weighting Equation. Bulletin 17B recommends the following weighting equation:

$$G_W = \frac{MSE_{\overline{G}}(G) + MSE_G(\overline{G})}{MSE_{\overline{G}} + MSE_G} \qquad (3\text{-}6)$$

where:

G_W = weighted skew coefficient

G = computed (station) skew

$\overline{G}$ = generalized skew

$MSE_{\overline{G}}$ = mean-square error of generalized skew

MSE_G = mean-square error of computed (station) skew

c. Mean Square Error.

(1) The mean-square error of the computed skew coefficient for log-Pearson type III random variables has been obtained by sampling experiments. Equation 6 in Bulletin 17B provides an approximate value for the mean-square error of the computed (station) skew coefficient:

$$MSE_G \approx 10^{\{A-B[\log_{10}(N/10)]\}} \quad (3\text{-}7a)$$

$$\approx 10^{A+B}/N^B \quad (3\text{-}7b)$$

$$A = -0.33 + 0.08\,|G| \text{ if } |G| \leq 0.90$$

$$= -0.52 + 0.30\,|G| \text{ if } |G| > 0.90$$

$$B = 0.94 - 0.26\,|G| \text{ if } |G| \leq .50$$

$$= 0.55 \qquad \text{if } |G| > 1.50$$

where:

$|G|$ = absolute value of the computed skew

N = record length in years

Appendix F-10 provides a table of mean-square error for several record lengths and skew coefficients based on Equation 3-7a.

(2) The mean-square error (MSE) for the generalized skew will be dependent on the accuracy of the method used to develop generalized skew relations. For an isoline map, the MSE would be the average of the squared differences between the computed (station) skew coefficients and the isoline values. For a prediction equation, the square of the standard error of estimate would approximate the MSE. And, if an arithmetic mean of the stations in a region were adopted, the square of the standard deviation (variance) would approximate the MSE.

3-4. Expected Probability.

a. The computation of a frequency curve by the use of the sample statistics, as an estimate of the distribution parameters, provides an estimate of the true frequency curve. (Chapter 8 discusses the reliability and the distribution of the computed statistics.) The fact of not knowing the location of the true frequency curve is termed uncertainty. For the normal distribution, the sampling errors for the mean are defined by the t distribution and the sampling errors for the variance are defined by the chi-squared distribution. These two error distributions are combined in the formation of the non-central t distribution. The non-central t-distribution can be used to construct curves that, with a specified confidence (probability), encompass the true frequency curve. Figure 3-2 shows the confidence limit curves around a frequency curve that has the following assumed statistics: N=10, $\overline{X}$=0., S=1.0.

b. If one wished to design a flood protection work that would be exceeded, on the average, only one time every 100 years (one percent chance exceedance), the usual design would be based on the normal standard deviate of 2.326. Notice that there is a 0.5 percent chance that this design level may come from a "true" curve that would average 22 exceedances per 100 years. On the other side of the curve, instead of the expected one exceedance, there is a 99.5 percent chance that the "true" curve would indicate 0.004 exceedances. Note the large number of exceedances possible on the left side of the curve. This relationship is highly skewed towards the large exceedances because the bound on the right side is zero exceedance. A graph of the number of possible "true" exceedances versus the probability that the true curve exceeds this value, Figure 3-3, provides a curve with an area equal to the average (expected) number of exceedances.

c. The design of many projects with a target of 1 exceedance per 100 years at each project and assuming N=10 for each project, would actually result in an average of 2.69 exceedances (see Appen ix F-8).

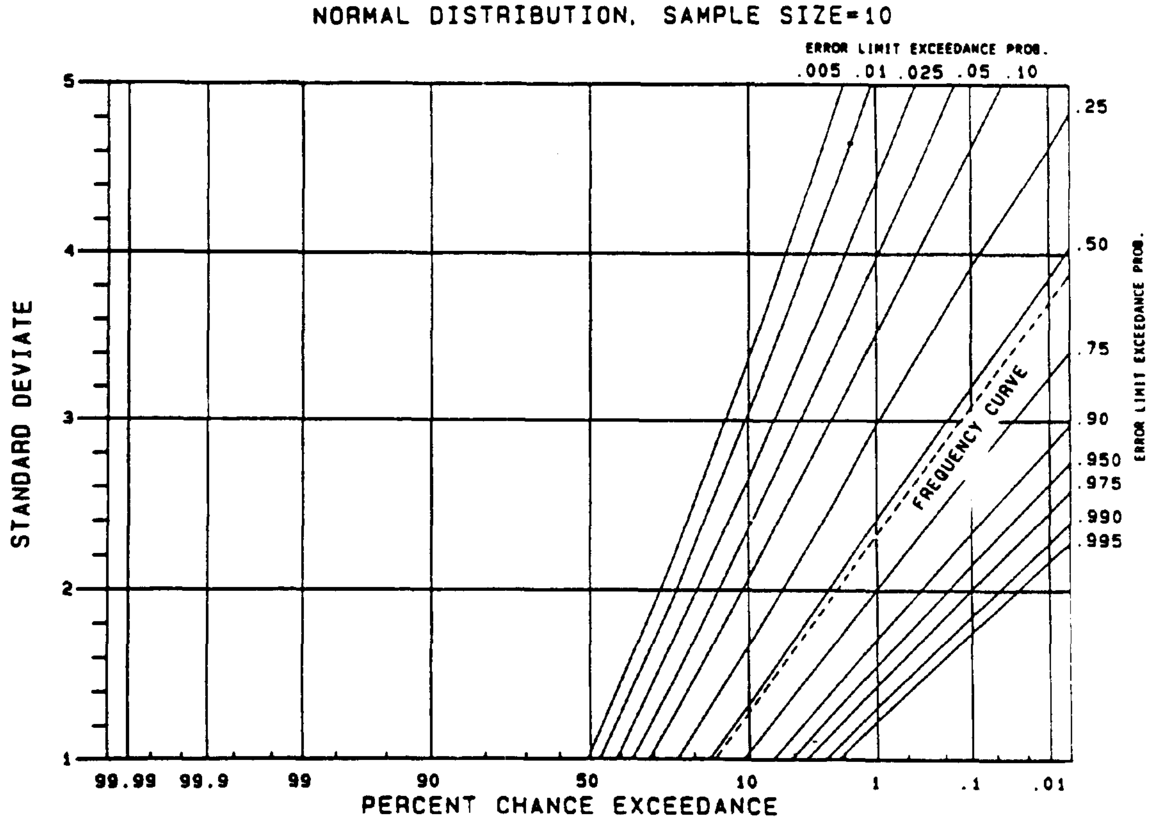


Figure 3-2. Confidence Limit Curves based on the Non-central t Distribution.

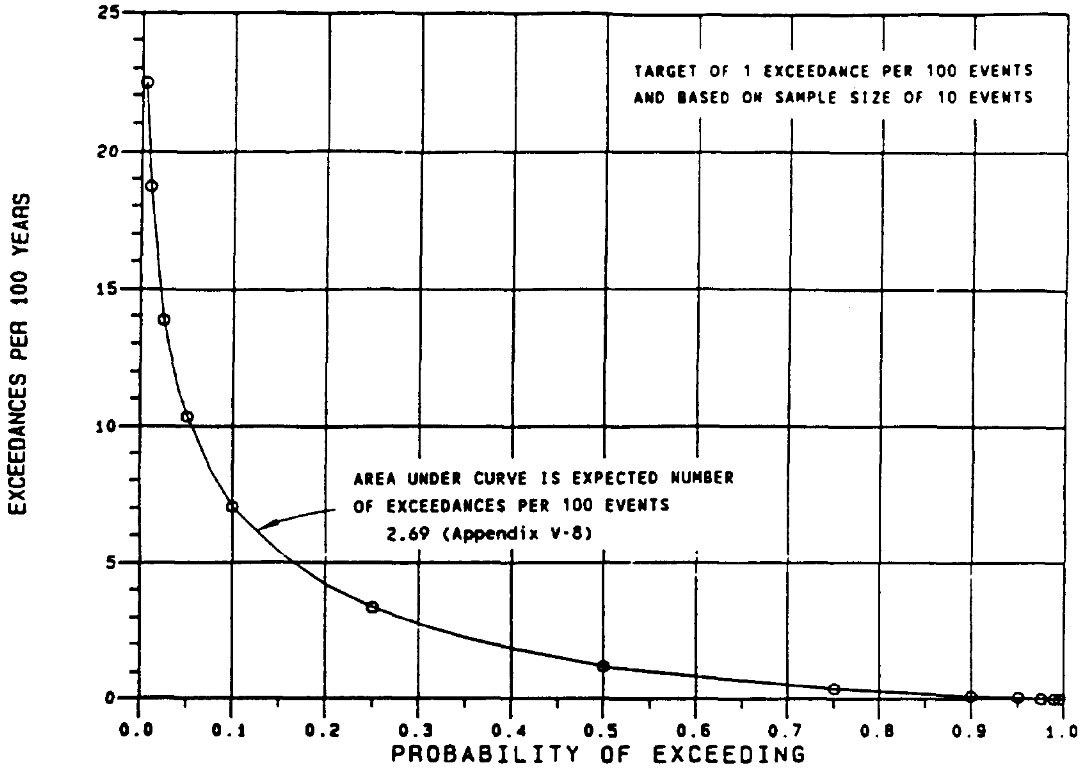


Figure 3-3. Cumulative Probability Distribution of Exceedances per 100 Years.

d. There are two methods that can be used to correct (expected probability adjustment) for this bias. The first method, as described above, entails plotting the curve at the "expected" number of exceedances rather that at the target value, drawing the new curve and then reading the adjusted design level. Appendix F-8 provides the percentages for the expected probability adjustment.

e. The second method is more direct because an adjusted deviate (K value) is used in Equation 3-4 that makes the expected probability adjustment for a given percent chance exceedance. Appendix F-7 contains the deviates for the expected probability adjustment. These values may be derived from the t-distribution by the following equation:

$$K_{P,N} = t_{P,N-1} \left[(N+1)/N\right]^{1/2} \tag{3-8}$$

where:

P = exceedance probability (percent chance exceedance divided by 100)

N = sample size

K = expected probability adjusted deviate

t = Student's t-statistic from one-tailed distribution

f. For a sample size of 10 and a 1% percent chance exceedance, the expected probability adjusted deviate is 2.959 as compared to the value of 2.326 used to derive the computed frequency curve.

g. As mentioned in the first paragraph, the non-central t distribution, and consequently the expected probability adjustment, is based on the normal distribution. The expected probability adjustment values in Appendices F-7 and F-8 are considered applicable to Pearson type III distributions with small skew coefficients. The phrase "small skew coefficients" is usually interpreted as being between -0.5 to +0.5. Note also that the uncertainty in the skew coefficient is not considered. In other words, the skew coefficient is treated as if it were the population skew coefficient.

h. The expected probability adjustment can be applied to frequency curves derived by other than analytical procedures if the equivalent worth (in years) of the procedure can be computed or estimated.

3-5. Risk.

a. Definition. The term risk is usually defined as the possibility of suffering loss or injury. In a hydrologic context, risk is defined as "the probability that one or more events will exceed a given flood magnitude within a specified period of years" (46). Note that this narrower definition includes a time specification and assumes that the annual exceedance frequency is exactly known. Uncertainty is not taken into account in this definition of risk. Risk then enables a probabilistic statement to be made about the chances of a particular location being flooded when it is occupied for a specified number of consecutive years. The percent chance of the location being flooded in any given year is assumed to be known.

b. Binomial Distribution. The computation of risk is accomplished by the equation for the binomial distribution:

$$R_I = \frac{N!}{I!(N-I)!} P^I(1-P)^{N-I} \tag{3-9}$$

where:

R_I = risk (probability) of experiencing exactly I flood events

N = number of years (trials)

I = number of flood events (successes)

P = exceedance probability, percent chance exceedance divided by 100, of the annual event (probability of success)

(The terms in parentheses are those usually used in statistical texts)

When I equals zero (no floods), Equation 3-9 reduces to:

$$R_0 = (1-P)^N \qquad (3\text{-}10a)$$

and the probability of experiencing one or more floods is easily computed by taking the complement of the probability of no floods:

$$R_{(1 \text{ or more})} = 1-(1-P)^N \qquad (3\text{-}10b)$$

c. Application.

(1) Risk is an important concept to convey to those who are or will be protected by flood control works. The knowledge of risk alerts those occupying the flood plain to the fact that even with the protection works, there could be a significant probability of being flooded during their lifetime. As an example, if one were to build a new house with the ground floor at the 1% chance flood level, there is a fair (about one in four) chance that the house will be flooded before the payments are completed, over the 30-year mortgage life. Using Equation 3-10b:

$$\begin{aligned} R_{(1 \text{ or more})} &= 1-(1-.01)^{30} \\ &= 1-.99^{30} \\ &= 1-.74 \\ &= .26 \text{ or } 26\% \text{ chance} \end{aligned}$$

(2) Appendix F-12 provides a table for risk as a function of percent chance exceedance, period length and number of exceedances. This table could also be used to check the validity of a derived frequency curve. As an example, if a frequency curve is determined such that 3 observed events have exceeded the derived 1% chance exceedance level during the 50 years of record, then there would be reason to question the derived frequency curve. From Appendix F-12, the probability of this occurring is 0.0122 or about 1%. It is possible for the situation to occur, but the probability of occurring is very low. This computation just raises questions about the validity of the derived curve and indicates that other validation checks may be warranted before adopting the derived curve.

3-6. Conditional Probability Adjustment. The conditional probability adjustment is made when flood peaks have either been deleted or are not available below a specified truncation level. This adjustment will be applied when there are zero flood years, an incomplete record or low outliers. As stated in Appendix 5 of Bulletin 17B, this procedure is not appropriate when 25 percent or more of the events are truncated. The computation steps in the conditional probability adjustment are as follows:

1. Compute the estimated probability ($\tilde{P}$) that an annual peak will exceed the truncation level:

$$\tilde{P} = N/n \qquad (3\text{-}11a)$$

where N is the number of peaks above the truncation level and n is the total number of years of record. If the statistics reflect the adjustments for historic information, then the appropriate equation is

$$\tilde{P} = \frac{H - WL}{H} \qquad (3\text{-}11b)$$

where H is the length of historic period, W is the systematic record weight and L is the number of peaks truncated.

2. The computed frequency curve is actually a conditional frequency curve. Given that the flow exceeds the truncation level, the exceedance frequency for that flow can be estimated. The conditional exceedance frequencies are converted to annual frequencies by the probability computed in Step 1:.

$$P = \tilde{P}\, P_d \qquad (3\text{-}12)$$

where P is the annual percent chance exceedance and P_d is the conditional percent chance exceedance.

3. Interpolate either graphically or mathematically to obtain the discharge values (Q_p) for 1, 10 and 50 percent chance exceedances.

4. Estimate log-Pearson type III statistics that will fit the upper portion of the adjusted curve with the following equations:

$$G_s = -2.50 + 3.12 \frac{\log (Q_1/Q_{10})}{\log (Q_{10}/Q_{50})} \qquad (3\text{-}13)$$

$$S_s = \frac{\log (Q_1/Q_{50})}{K_1 - K_{50}} \qquad (3\text{-}14)$$

$$X_s = \log (Q_{50}) - K_{50}\, S_s \qquad (3\text{-}15)$$

where G_s, S_s and $\overline{X}_s$ are the synthetic skew coefficient, standard deviation and mean, respectively; Q_1, Q_{10} and Q_{50} the discharges determined in Step 3; and K_1 and K_{50} are the Pearson Type III deviates for percent change exceedances of 1 and 50 and skew coefficient G_s.

5. Combine the synthetic skew coefficient with the generalized skew by use of Equation 3-6 to obtain the weighted skew.

6. Develop the computed frequency curve with the synthetic statistics and compare it with the plotted observed flood peaks.

3-7. Two-Station Comparison.

a. Purpose.

(1) In most cases of frequency studies of runoff or precipitation there are locations in the region where records have been obtained over a long period. The additional period of record at such a nearby station is useful for extending the record at a short record station provided there is reasonable correlation between recorded values at the two locations.

(2) It is possible, by regression or other techniques, to estimate from concurrent records at nearby locations the magnitude of individual missing events at a station. However, the use of regression analysis produces estimates with a smaller variance than that exhibited by recorded data. While this may not be a serious problem if only one or two events must be estimated to "fill in" or complete an otherwise unbroken record of several years, it can be a significant problem if it becomes necessary to estimate more than a few events. Consequently, in frequency studies, missing events should not be freely estimated by regression analysis.

(3) The procedure for adjusting the statistics at a short-record station involves three steps: (1) computing the degree of correlation between the two stations, (2) using the computed degree of correlation and the statistics of the longer record station to compute an adjusted set of statistics for the shorter-record station, and (3) computing an equivalent "length of record" that approximately reflects the "worth" of the adjusted statistics of the short-record station. The longer record station selected for the adjustment procedure should be in a hydrologically similar area and, if possible, have a drainage area size similar to that of the short-record station.

b. Computation of Correlation. The degree of correlation is reflected in the correlation coefficient R^2 as computed through use of the following equation:

$$R^2 = \frac{[\sum XY - (\sum X \sum Y)/N]^2}{[\sum X^2 - (\sum X)^2/N]\,[\sum Y^2 - (\sum Y)^2/N]} \qquad (3\text{-}16)$$

where:

R^2 = the determination coefficient

Y = the value at the short-record station

X = the concurrent value at the long-record station

N = the number of years of concurrent record

For most studies involving streamflow values, it is appropriate to use the logarithms of the values in the equations in this section.

c. Adjustment of Mean. The following equation is used to adjust the mean of a short-record station on the basis of a nearby longer-record station:

$$\overline{Y} = \overline{Y}_1 + (\overline{X}_3 - \overline{X}_1)\, R\, (S_{Y_1}/S_{X_1}) \qquad (3\text{-}17)$$

where:

$\overline{Y}$ = the adjusted mean at the short-record station

$\overline{Y}_1$ = the mean for the concurrent record at the short-record station

$\overline{X}_3$ = the mean for the complete record at the longer-record station

$\overline{X}_1$ = the mean for the concurrent record at the longer-record station

R = the correlation coefficient

S_{Y_1} = the standard deviation for the concurrent record at the short-record station

S_{X_1} = the standard deviation for the concurrent record at the longer-record station

All of the above parameters may be derived from the logarithms of the data where appropriate, e.g., for annual flood peaks. The criterion for determining if the variance of the adjusted mean will likely be less than the variance of the concurrent record is:

$$R^2 > 1/(N_1 - 2) \qquad (3\text{-}18)$$

where N_1 equals the number of years of concurrent record. If R^2 is less than the criterion, Equation 3-17 should not be applied. In this case just use the computed mean at the short-record station or check another nearby long-record station. See Appendix 7 of Bulletin 17B for procedures to compare the variance of the adjusted mean against the variance of the entire short-record period.

d. Adjustment of Standard Deviation. The following equation can be used to adjust the standard deviation:

$$S_Y^2 = S_{Y_1}^2 + (S_X^2 - S_{X_1}^2)\, R^2 (S_{Y_1}^2/S_{X_1}^2) \qquad (3\text{-}19)$$

(approximate)

where:

S_Y = the adjusted standard deviation at the short-record station

S_{Y_1} = the standard deviation for the period of concurrent record at the short-record station

S_X = the standard deviation for the complete record at the base station

S_{Y_1} = the standard deviation for the period of concurrent record at the base station

R^2 = the determination coefficient

All of the above parameters may be derived from the logarithms of the data where appropriate, e.g., for annual flood peaks. This equation provides approximate results compared to Equation 3-19 in Appendix 7 of Bulletin 17B, but in most cases the difference in the results does not justify the additional computations.

e. Adjustment of Skew coefficient. There is no equation to adjust the skew coefficient that is comparable to the above equations. When adjusting the statistics of annual flood peaks either a weighted or a generalized skew coefficient may be used depending on the record length.

f. Equivalent Record Length. The final step in adjusting the statistics is the computation of the "equivalent record length" which is defined as the period of time which would be required to establish unadjusted statistics that are as reliable (in a statistical sense) as the adjusted values. Thus, the equivalent length of record is an indirect indication of the reliability of the adjusted values of Y and S_Y. The equivalent record length for the adjusted mean is computed from the following equation:

$$N_Y = \frac{N_{Y_1}}{1 - [(N_X - N_{Y_1})/N_X]\,[R^2 - (1 - R^2)/(N_{Y_1} - 3)]} \qquad (3\text{-}20)$$

where:

N_Y = the equivalent length of record of the mean at the short-record station

N_{Y_1} = the number of years of concurrent record at the two stations

N_X = the number of years of record at the longer-record station

R = the adjusted correlation coefficient

Figure 3-4 shows the data and computations for a two-station comparison for a short record station with 21 events and a long record station with 60 systematic events. It can be seen that the adjustment of the frequency statistics provides an increased reliability in the mean equivalent to having an additional 17 years of record at the short-record station.

Year	Flow: Chattooga River	Flow: Tallulah River
1915	12600	
1917	14000	
1918	5900	
1919	16000	
1920	8200	
1921	4100	
1922	6200	
1923	5300	
1924	9200	
1925	3900	
1926	6200	
1927	3600	
1928	20100	
1929	11400	
1940	29000*	
1941	7530	
1942	6870	
1943	6870	
1944	3840	
1945	2930	
1946	6650	
1947	6440	
1948	12400	
1949	13900	
1950	4740	
1951	5220	
1952	13400	
1953	6020	
1954	6230	
1955	5820	
1956	5820	
1957	5820	
1958	5620	
1959	5620	
1960	6440	
1961	7310	
1962	9660	
1963	5420	
1964	9880	
1965	27200	7440
1966	13400	5140
1967	15400	2800
1968	5620	3100
1969	14700	2470
1970	3480	2010
1971	3290	976
1972	7440	2160
1973	19600	8500
1974	6400	4660
1975	6340	2410
1976	18500	6530
1977	13000	3580
1978	7850	4090
1979	14800	6240
1980	10900	2880
1981	4120	1600
1982	5000	1960
1983	7910	3260
1984	4810	2000
1985	4740	1010

* Historic information, peak largest since 1915.

STATISTICS OF DATA

	Long Record: Total	Long Record: Concurrent	Long Record: Not Concurrent	Short Record
Systematic Events	60	21	39	21
Historic Period	71	0	0	0
Log Mean	3.8866	3.9310	3.8650	3.4796
Standard Deviation	0.2322	0.2678	0.2140	0.2609
Skew Computed	0.5075	0.1588	0.7340	-0.0811
Skew Generalized	-	-	-	-
Skew Adopted	-	-	-	-0.1000

COMPUTATIONS FOR TWO-STATION COMPARISON:

Slope: $b = 0.790495$
Correlation Coefficient: $R = 0.811351$ (See Figure 9-01)

From equations 19, 21, and 22:

$$Y_1 = 3.4796 + 0.790495\ (3.8866 + 3.9310)$$
$$Y_1 = 3.4445$$

$$S_Y^2 = [(0.2322)^2 - (0.2678)^2](0.8114)^2(0.2609/0.2678)^2 + (0.2609)^2$$
$$S_Y = 0.2386$$

$$N_{Y_1} = \frac{21}{1 - \frac{71-2}{71}\left(0.6583 - \frac{1 - 0.6583}{21 - 3}\right)} = 38$$

$\bar{G} = 0.0 \quad MSE_{\bar{G}} = 0.302 \quad G = -0.0811 \quad MSE_G = 0.142$

$$G_W = \frac{(0.302)(-0.0811) + (0.0142)(0.0)}{0.302 + 0.142}$$

$G_W = -0.055 \Rightarrow -0.1$

FREQUENCY CURVE, TALLULAN RIVER NEAR CLAYTON, GA

FLOW, CFS: COMPUTED	FLOW, CFS: EXPECTED PROBABILITY	PERCENT CHANCE EXCEEDANCE	CONFIDENCE LIMITS: .05 LIMIT	CONFIDENCE LIMITS: .95 LIMIT
12700	14200	.2	18800	9580
10900	11900	.5	15600	8410
9590	10300	1.0	13400	7540
8350	8800	2.0	11300	6680
6760	6990	5.0	8810	5550
5590	5710	10.0	7040	4680
4430	4480	20.0	5370	3780
2810	2810	50.0	3260	2420
1760	1740	80.0	2060	1450
1370	1340	90.0	1640	1080
1110	1070	95.0	1360	847
745	686	99.0	957	524

Figure 3-4. Two-Station Comparison Computations.

(Figure 9-1 shows the computations for b and R and Figure 9-2 shows the Tallulah River annual peaks plotted against the Chattooga River peaks.) Figure 3-5 shows the resulting unadjusted and adjusted frequency curves based on the computed and adjusted statistics in Figure 3-4. Although N_Y is actually the equivalent years of record for the mean, the value is used as an estimate equivalent record length in the computation of confidence limits and the expected probability adjustment.

g. Summary of Steps. The procedure for computing and adjusting frequency statistics using a longer-record station can be summarized as follows:

(1) Arrange the streamflow data by pairs in order of chronological sequence.

(2) Compute $\overline{Y}_1$ and S_{Y_1} for the entire record at the short-record station.

(3) Compute $\overline{X}$ and S_X for the entire record at the longer-record station.

(4) Compute $\overline{X}_1$ and S_{X_1} for the portion of the longer-record station which is concurrent with the short-record station.

(5) Compute the correlation coefficient using Equation 3-16.

(6) Compute $\overline{Y}$ and S_Y for the short-record station using Equations 3-17 and 3-18.

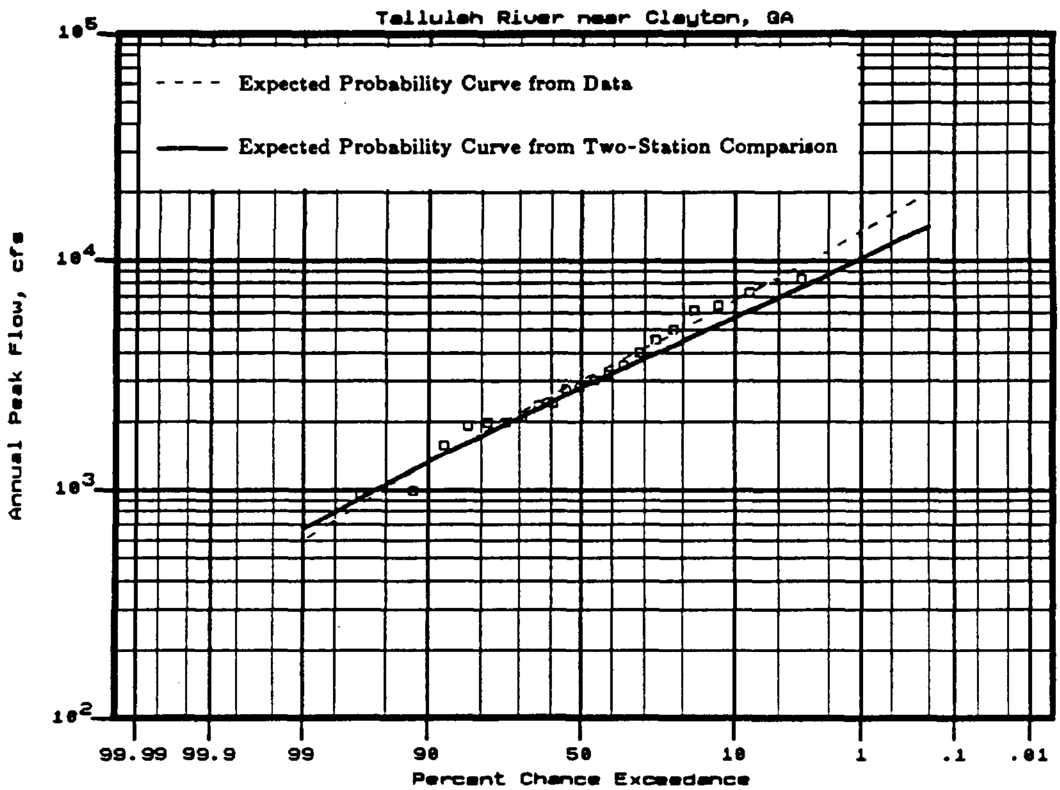


Figure 3-5. Observed and Two-Station Comparison Frequency Curves.

(7) Calculate the equivalent length of record of the mean for the short-record station using Equation 3-20.

(8) Compute the frequency curve using adjusted values of $\overline{Y}$ and S in Equation 3-4 and K values from Appendix F-2 corresponding to the adopted skew coefficient.

(9) Compute the expected probability adjustment and the confidence limits.

3-8. Flood Volumes.

a. Nature and Purpose. Flood volume frequency studies involve frequency analysis of maximum runoff within each of a set of specified durations. Flood volume-duration data normally obtained from the USGS WATSTORE files consists of data for 1, 3, 7, 15, 30, 60, 90, 120, and 183 days. These same values are the default values in the HEC computer program STATS (Table 3-2). Runoff volumes are expressed as average flows in order that peak flows and volumes can be readily compared and coordinated. Whenever it is necessary to consider flows separately for a portion of the water year such as the rain season or snowmelt season, the same durations (up to the 30-day or 90-day values) are selected from flows during that season only. Flood volume-duration curves are used primarily for reservoir design and operation studies, and should generally be developed in the design of reservoirs having flood control as a major function.

Table 3-2. High Flow Volume-Duration Data

- VOLUME-DURATION DATA - FISHKILL CR AT BEACON, NY - DAILY FLOWS

	HIGHEST MEAN VALUE FOR DURATION, FLOW,CFS								
YEAR	1	3	7	15	30	60	90	120	183
1945	2080.0	1936.7	1714.3	1398.7	1106.8	752.3	742.2	649.4	559.2
1946	1360.0	1180.3	923.0	837.3	657.8	605.3	476.2	451.5	379.9
1947	1800.0	1616.7	1159.1	820.5	687.1	611.9	558.5	485.8	396.4
1948	2660.0	2430.0	2322.9	1641.7	1145.1	862.0	706.2	638.1	512.7
1949	2900.0	2346.7	1715.7	1358.9	888.9	680.7	586.8	522.4	422.4
1950	1050.0	909.7	746.9	639.7	588.1	455.9	423.0	387.2	335.1
1951	2160.0	1886.7	1744.3	1248.1	872.9	832.1	781.2	689.8	568.9
1952	2870.0	2266.7	1557.6	1186.5	1032.8	925.1	854.1	732.6	692.9
1953	2850.0	2233.3	1644.3	1317.2	1145.5	994.6	831.1	794.4	654.5
1954	1520.0	1096.7	811.7	620.4	482.9	397.0	405.7	372.7	348.1
1955	6970.0	4536.7	2546.1	1360.0	758.2	608.0	494.0	463.1	478.7
1956	6760.0	5456.7	3354.3	1959.7	1572.8	1080.9	767.7	635.8	641.7
1957	1230.0	1117.3	1037.7	758.9	524.2	408.8	363.3	373.4	324.4
1958	2130.0	1916.7	1587.1	1354.5	1128.1	972.0	848.2	777.8	654.1
1959	1670.0	986.7	782.1	586.6	517.6	466.7	437.5	398.8	346.2
1960	2080.0	1770.0	1374.3	1046.9	712.3	605.5	530.5	515.1	468.4
1961	3440.0	2966.7	2155.7	1590.2	1152.3	845.2	759.5	656.2	491.4
1962	2570.0	2070.0	1547.7	1105.0	857.7	600.9	461.3	429.4	325.0
1963	1730.0	1616.7	1309.0	1216.0	900.8	569.1	438.0	370.8	305.9
1964	1300.0	1106.7	945.3	737.8	541.2	514.8	486.6	450.1	368.3
1965	900.0	826.3	652.6	455.7	375.8	303.3	275.7	235.0	175.0
1966	930.0	774.7	693.3	546.5	445.7	352.5	296.2	272.5	209.0
1967	1520.0	1416.7	1247.1	1023.5	906.8	701.3	581.4	521.1	436.8
1968	3500.0	2810.0	1934.3	1328.5	878.7	611.7	609.5	567.3	460.3

Note - Data based on water year of October 1 of preceeding year through September 30 of given year.

b. Data for Comprehensive Series. Data to be used for a comprehensive flood volume-duration frequency study should be selected from nearly complete water year records. Unless overriding reasons exist, the durations in Table 3-2 should be used in order to assure consistency among various studies for comparison purposes. Maximum flood events should be selected only for those years when recorder gages existed or when the maximum events can be estimated by other means. Where a minor portion of a water year's record is missing, the longer-duration flood volumes for that year can often be estimated adequately. If upstream regulation or diversion is known to have an effect, care should be exercised to assure that the period selected is the one when flows would have been maximum under the specified (usually natural) conditions.

c. Statistics for Comprehensive Series.

(1) The probability distribution recommended for flood volume-duration frequency computations is the log-Pearson type III distribution; the same as that used for annual flood peaks. In practice, only the first two moments, mean and standard deviations are based on station data. As discussed in Section 3-3, the skew coefficient should not be based solely on the station record, but should be weighted with information from regional studies. To insure that the frequency curves for each duration are consistent, and especially to prevent the curves from crossing, it is desirable to coordinate the variation in standard deviation and skew with that of the mean. This can be done graphically as shown in Figure 3-6. For a given skew coefficient, there is a maximum and minimum allowable slope for the standard deviation-versus-mean relation which prevents the curves from crossing within the established limits. For instance, to keep the curves from crossing within 99.99 and .01 percent chance exceedances with a skew of 0., the slope must not exceed .269, nor be less than -.269, respectively. The value of this slope constraint is found by stating that the value of one curve (X_A for curve A) must equal or exceed the value for a second curve (X_B for curve B) at the desired exceedance frequency. Each of these values can be found by substitution into Equation 3-4 (the K for zero skew and 99.99 percent chance exceedance is -3.719):

$$X_A \geq X_B$$

$$\overline{X}_A + (-3.719)\,S_A \geq \overline{X}_B + (-3.719)\,S_B$$

$$3.719\,(S_B - S_A) \geq \overline{X}_B - \overline{X}_A$$

$$\frac{(S_B - S_A)}{(\overline{X}_B - X_A)} \geq 0.269$$

where:

X_A = Value of frequency curve A at 99.99 percent chance exceedance

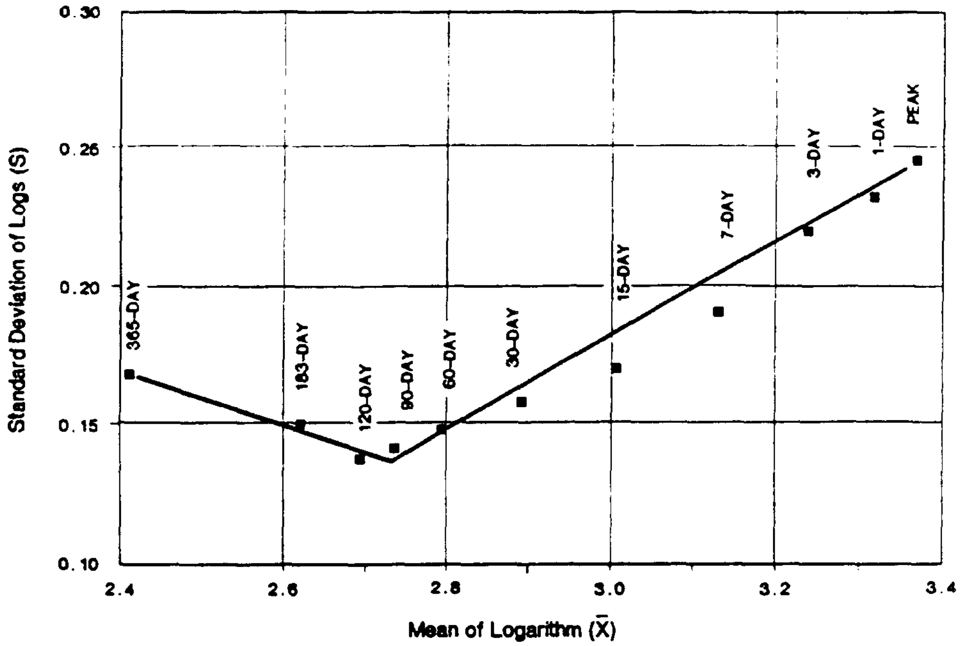
X_B = Value of frequency curve B at 99.99 percent chance exceedance

$\overline{X}_A$ = Mean of frequency curve A

$\overline{X}_B$ = Mean of frequency curve B

S_A = Standard deviation of frequency curve A

S_B = Standard deviation of frequency curve B



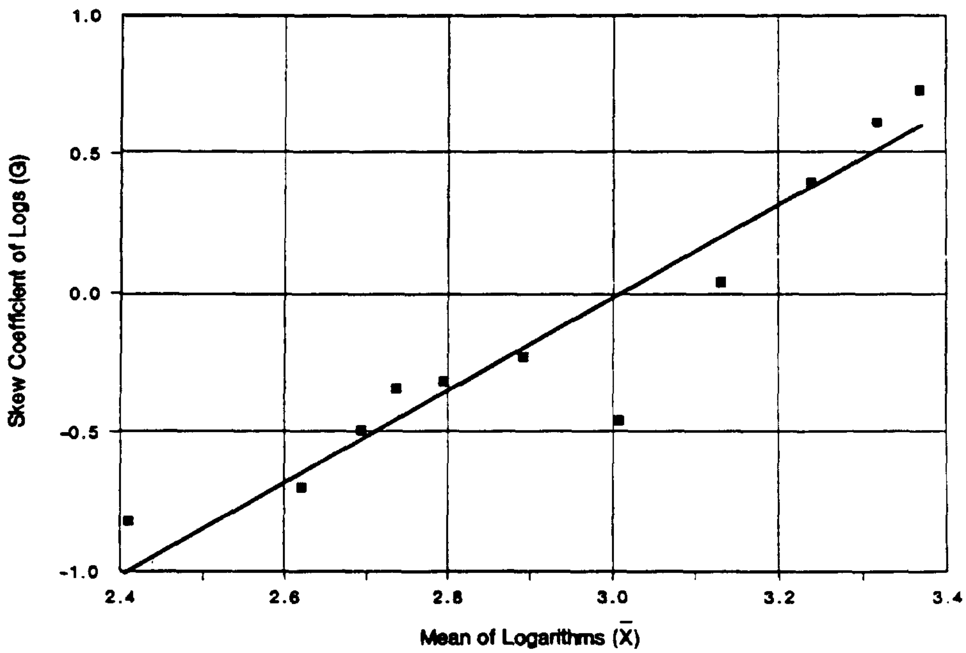


Figure 3-6. Coordination of Flood-Volume Statistics.

(2) When the skew changes between durations, it is probably easiest to adopt smoothed relations for the standard deviation and skew and input the statistics into a computer program that computes the ordinates. The curves can then be inspected for consistency.

(3) If the statistics for the peak flows have been computed according to the procedures in Bulletin 17B, the smoothing relations should be forced through those points. The procedure for computing a least-squares line through a given intersection can be found in texts describing regression analyses.

d. Frequency Curves for Comprehensive Series.

(1) General Procedure. Frequency curves of flood volumes are computed analytically using general principles and methods of Chapters 2 and 3. They should also be shown graphically and compared with the data on which they are based. This is a general check on the analytic work and will ordinarily reveal any inconsistency in data and methodology. The computed frequency curves and the observed data should be plotted on a single sheet for comparison purposes, Figure 3-7.

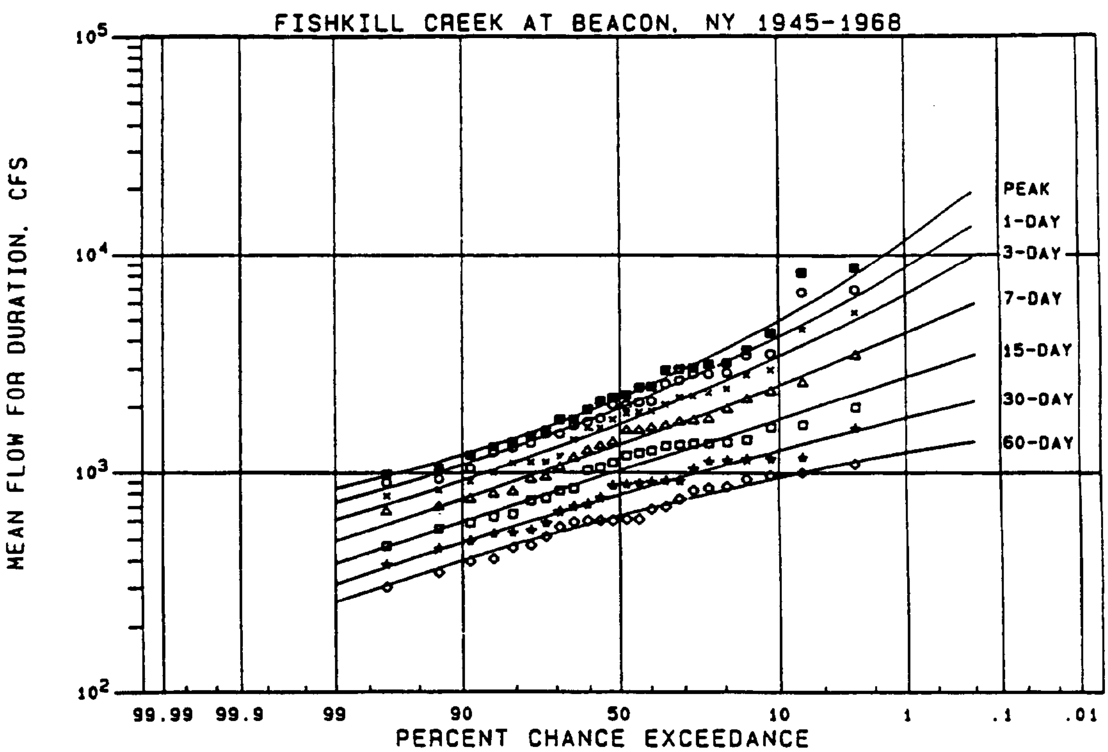


Figure 3-7. Flood-Volume Frequency Curves.

(2) Interpolation Between Fixed Durations. The runoff volume for any specified frequency can be determined for any duration between 1-day and 365-days by drawing a curve on logarithmic paper relating mean discharge (or volume) to duration for that specified frequency (see Figure 3-8a). When runoff volumes for durations shorter than 24 hours are important, special frequency studies should be made. These could be done in the same manner as for the longer durations, using skew coefficients interpolated in some reasonable manner between those used for peak and 1-day flows.

e. Applications of Flood Volume-Duration Frequencies.

(1) Volume-duration Curves. The use of flood volume-duration frequencies in solving reservoir planning, design, and operation problems usually involves the construction of volume-duration curves for specified frequencies. These are drawn first on logarithmic paper for interpolation purposes, as illustrated on Figure 3-8a. The mean discharge values are multiplied by appropriate durations to obtain volumes and are then replotted on an arithmetic grid as shown on the Figure 3-8b. A straight line on this grid represents a constant rate of flow. The straight line represents a uniform flow of 1,500 cfs, and the maximum departure from the 2% chance exceedance curve demonstrates that a reservoir capacity of 16,000 cfs-days (31,700 acre-feet) is required to control the indicated runoff volumes by a constant release of 1,500 cfs. The curve also indicates that a duration of about 8 days is critical for this project release rate and associated flood-control storage space.

(2) Application to a Single Reservoir. In the case of a single flood-control reservoir located immediately upstream of a single damage center, the volume frequency problems are relatively simple. A series of volume-duration curves, similar to that shown on Figure 3-8, corresponding to selected exceedance frequencies should first be drawn. The project release rate should be determined, giving due consideration to possible channel deterioration, encroachment into the flood plain, and operational contingencies. This procedure can be used not only as an approximate aid in selecting a reservoir capacity, but also as an aid in drawing filling-frequency curves.

(3) Application to a Reservoir System. In solving complex reservoir problems, representative hydrographs at all locations can be patterned after one or more past floods. The ordinates of these hydrographs can be adjusted so that their volumes for the critical durations will equal corresponding magnitudes at each location for the selected frequency. A design or operation scheme based on regulation of such a set of hydrographs would be reasonably well balanced. Some aspects of this problem are described in Section 3-9g.

3-9. Effects of Flood Control Works on Flood Frequencies.

a. Nature of the Problem. Flood control reservoirs are designed to substantially affect the frequency of flood flows (or flood stages) at various downstream locations. Many land use changes such as urbanization, forest clearing, etc. can also have significant effects on downstream flood flows (see Section 3-10). Channel improvements (intended to reduce stages) and levee improvements (intended to confine flows) at specified locations can substantially affect downstream flows by eliminating some of the natural storage effects. Levees can also create backwater conditions that affect river stages for a considerable distance upstream. The degree to which flows and stages are modified by various flood control works or land use changes can depend on the timing, areal distribution and magnitude of rainfall (and snowmelt, if pertinent) causing the flood. Accordingly, the studies should include evaluations of the effects on representative flood

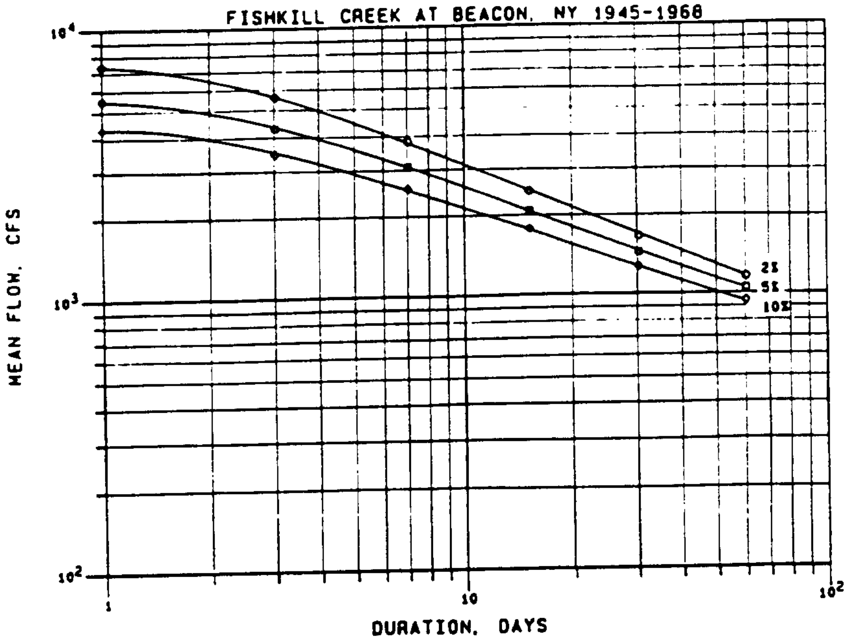


Figure 3-8a. Flood-Volume Frequency Relations.

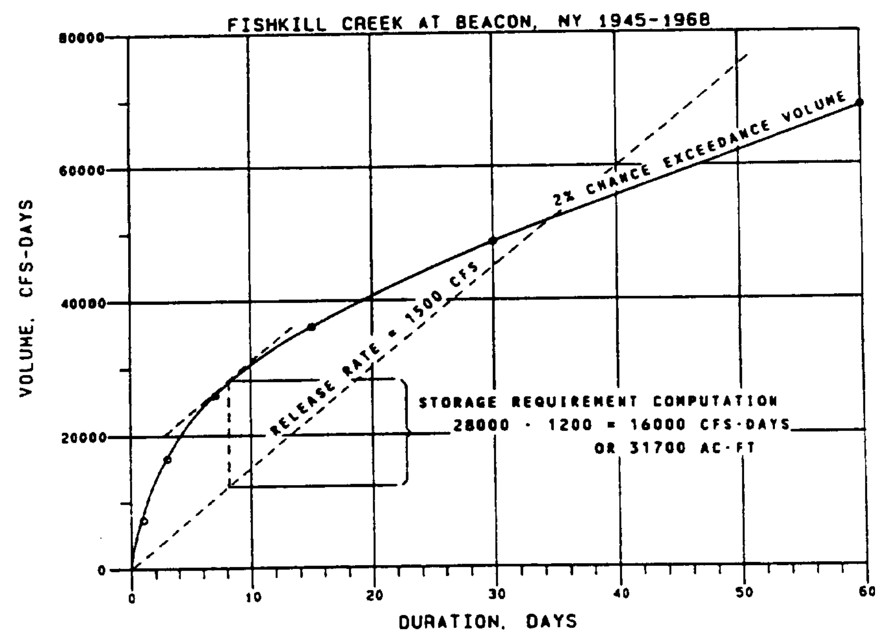


Figure 3-8b. Storage Requirement Determination.

events, with careful consideration given to the effects of different temporal and areal distributions.

b. Terminology.

(1) Natural Conditions. Natural conditions in the drainage basin are defined as hydrologic conditions that would prevail if no regulatory works or other works of man were constructed. Natural conditions, however, include the effects of natural lakes, swamp areas, etc.

(2) Present Conditions. Present or base conditions are defined as the conditions that exist as of the date of the study or some specified time.

(3) Without-Project Conditions. Without-project conditions are defined as the conditions that would exist without the projects under consideration, but with all existing projects and may include future projects whose construction is imminent.

(4) With-Project Conditions. With-project conditions are defined as the conditions that will exist after the projects under consideration are completed.

c. Reservoir-Level Frequency Computation.

(1) Factors to be Considered. Factors affecting the frequency of reservoir levels include historical inflow rates and anticipated future inflow rates estimated by volume-frequency studies, the storage-elevation curves, and the plan of reservoir regulation including location and size of reservoir outlets and spillway. A true frequency curve of annual maxima or minima can only be computed when the reservoir completely fills every year. Otherwise, the events would not be independent. If there is dependence between annual events, the ordinate should be labeled "percent of years exceeded" for maximum events and "percent of years not exceeded" for minimum events.

(2) Computation and Presentation of Results. A frequency curve of annual maximum reservoir elevations (or stages) is ordinarily constructed graphically, using procedures outlined in Section 2-4. Observed elevations (or stages) are used to the extent that these are available, if the reservoir operation will remain the same in the future. Historical and/or large hypothetical floods may also be routed through the reservoir using future operating plans. A typical frequency curve is illustrated on Figure 6-4. Elevation-duration curves are constructed from historical operation data or from routings of historical runoff in accordance with procedures discussed in Section 2-2, Figure 3-9. Such curves may be constructed for the entire period of record or for a selected wet period or dry period. For many purposes, particularly recreation uses, the seasonal variation of reservoir elevation (stages) is important. In this case a set of frequency or duration curves for each month of the year may be valuable. One format for presenting this information is illustrated on Figure 3-10.

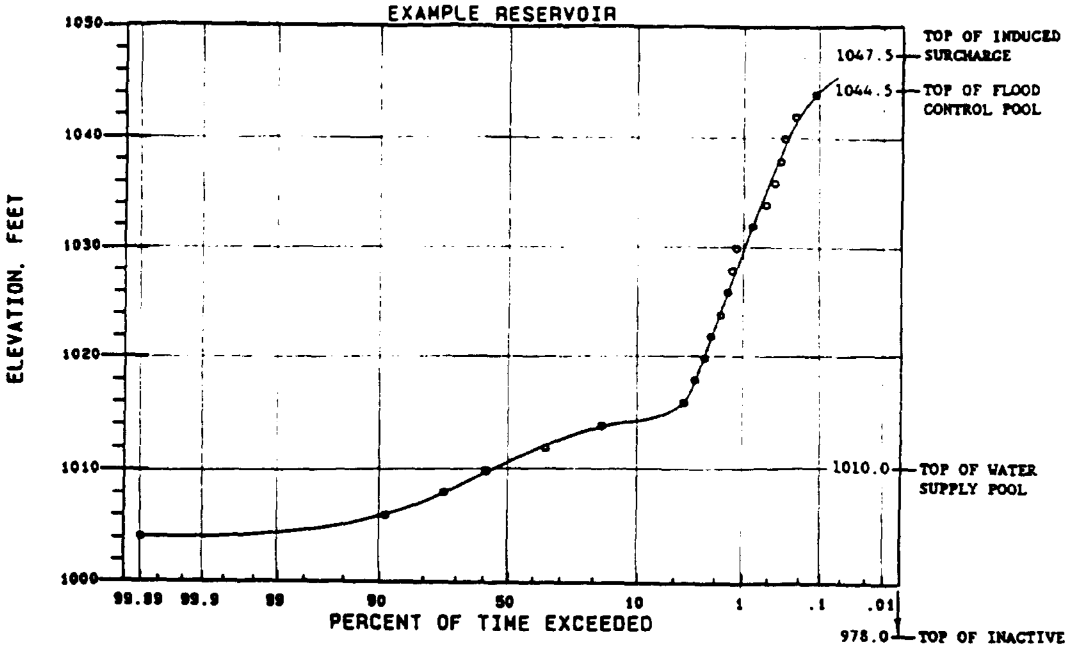


Figure 3-9. Daily Reservoir Elevation-Duration Curve.

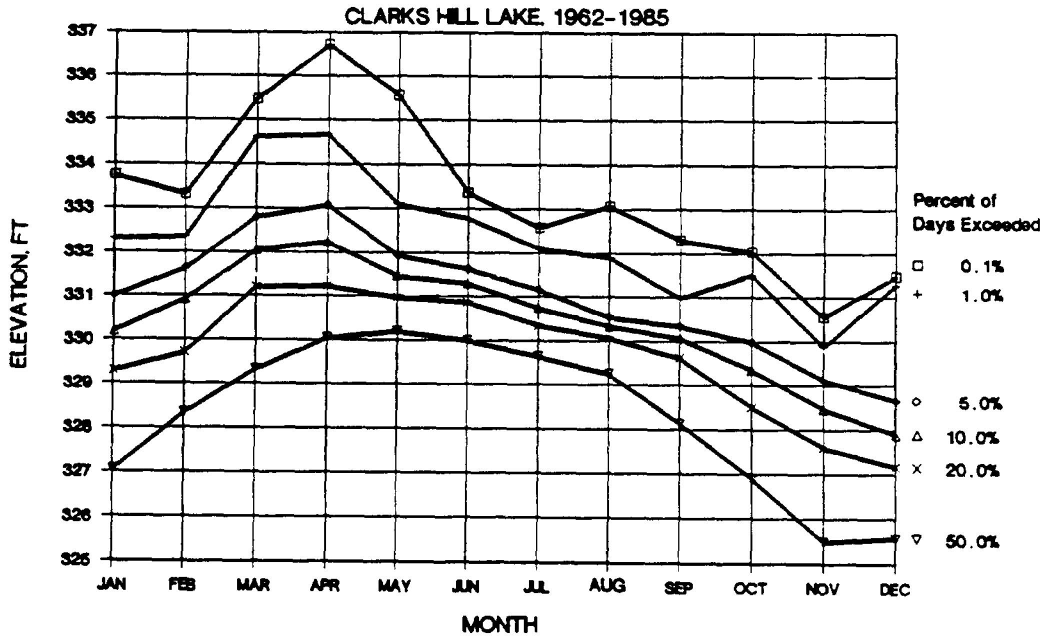


Figure 3-10. Seasonal Variation of Elevation-Duration Relations.

d. Effects of Reservoirs on Flows at Downstream Points.

(1) Routing for Period of Record. The frequency of reservoir outflows or of flows at a downstream location can be obtained from routings of the period-of-record runoff by the following methods:

(a) Determine the annual maximum flow at each location of interest and construct a frequency curve of the regulated flows by graphical techniques (Section 2-4).

(b) Construct a graph of with-project versus without-project flows at the location of interest and draw a curve relating the two quantities as illustrated on Figure 3-11. The points should be balanced in the direction transverse to the curve, but factors such as flood volume of the events and reliability of regulation must be considered in drawing the curve. This curve can be used in conjunction with a frequency curve of without-project flows to construct a frequency curve of with-project flows as illustrated on Figure 3-12. This latter procedure assures consistency in the analysis and gives a graphical presentation of the variability of the regulated events for a given unregulated flow.

(2) Use of Hypothetical-Flood Routings. Usually recorded values of flows are not large enough to define the upper end of the regulated frequency curve. In such cases, it is usually possible to use one or more large hypothetical floods (whose frequency can be estimated from the frequency curve of unregulated flows) to establish the corresponding magnitude of regulated flows. These floods can be multiples of the largest observed floods or of floods computed from rainfall; but it is best not to multiply any one flood by a factor greater than two or three. The floods are best selected or adjusted to represent about equal severity in terms of runoff frequency of peak and volumes for various durations. The routings should be made under reasonably conservative assumptions as to initial reservoir stages.

(3) Incidental Control by Water Supply Space. In constructing frequency curves of regulated flows, it must be recognized that reservoir operation for purposes other than flood control will frequently provide incidental regulation of floods. However, the availability of such space cannot usually be depended upon, and its value is considerably diminished for this reason. Consequently, the effects of such space on the reduction of floods should be estimated very conservatively.

(4) Allowance for Operational Contingencies. In constructing frequency curves of regulated flows, it should be recognized that actual operation is rarely perfect and that releases will frequently be curtailed or diminished because of unforeseen operation contingencies. Also, where flood forecasts are involved in the reservoir operation, it must be recognized that these are subject to considerable uncertainty and that some allowance for uncertainty will be made during operation. In accounting for these factors, it will be found that the actual control of floods is somewhat less than could be expected if full release capacities and downstream channel capacities were utilized efficiently and if all forecasts were exact.

(5) Runoff from Unregulated Areas. In estimating the frequency of runoff at a location that is a considerable distance downstream from one or more reservoir projects, it must be recognized that none of the runoff from the intermediate areas between the reservoir(s) and the damage center will be regulated. This factor can be accounted for by constructing a frequency curve of the runoff from the intermediate area, and using this curve as an indicator of the lower limit for the curve of regulated flows. Streamflow

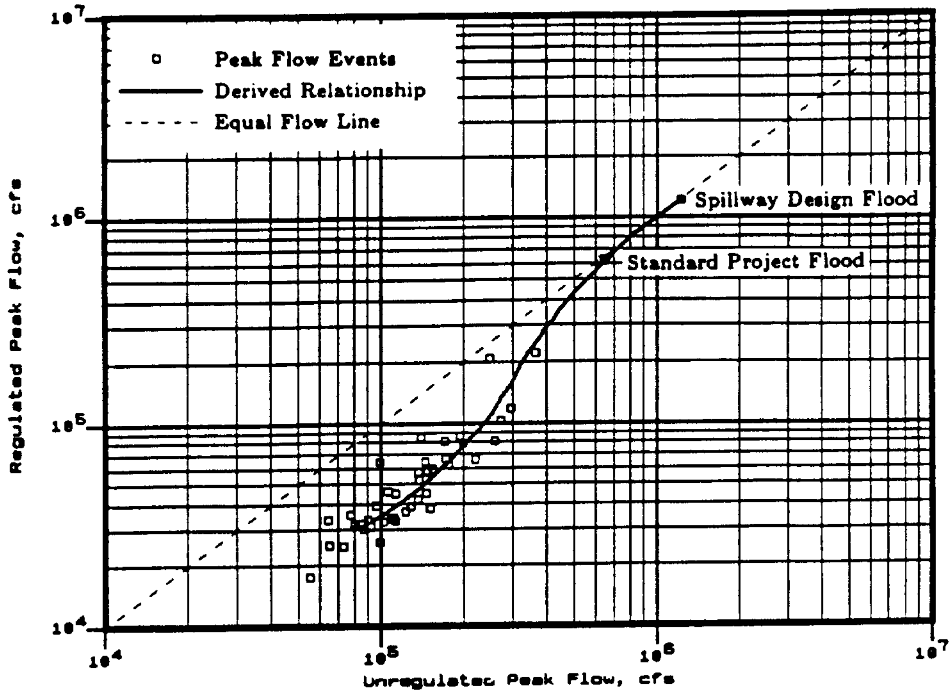


Figure 3-11. Example With-Project versus Without-Project Peak Flow Relations.

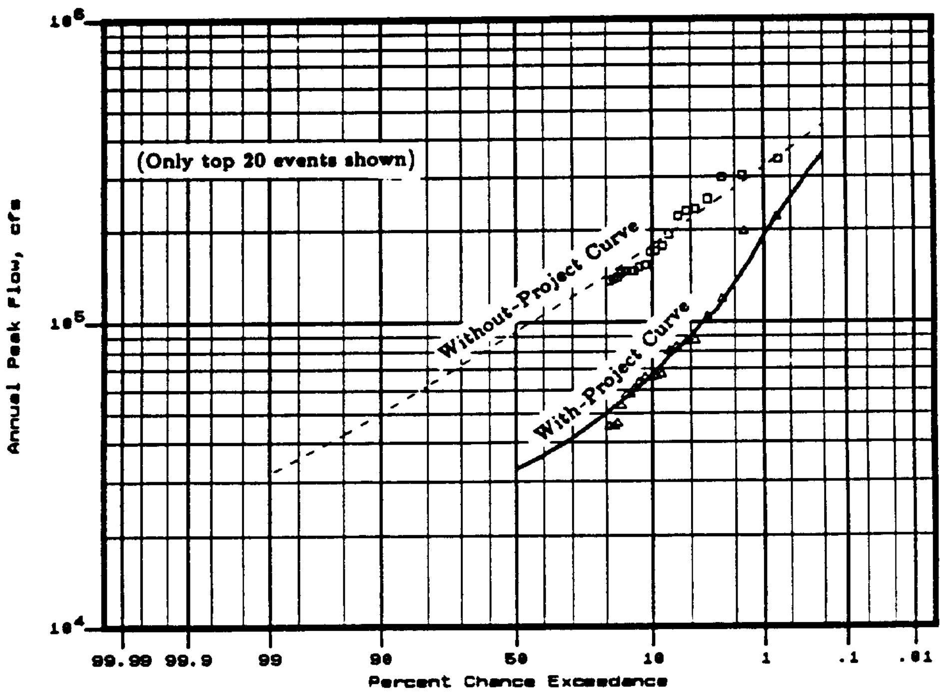


Figure 3-12. Example Without-Project and With-Project Frequency Curves.

routing and combining of both the flows from the unregulated area and those from the regulated area is the best procedure for deriving the regulated frequency curve.

e. Effects of Channel, Levee and Floodway Improvements. The effect of channel, levee and floodway improvements on river stages at the project location and on river discharges downstream from the project location can generally be evaluated by routing several typical floods through the reaches of the improvement and the upstream reaches affected by backwater. The stages or discharges thus derived can be plotted against corresponding without- project values, and a smooth curve drawn. This curve could be used in conjunction with a frequency curve of without-project values to construct a frequency curve of with-project values as discussed in Paragraph 3-09d(1)b. Corresponding stages upstream from the selected control point can be estimated from water-surface profile computations.

f. Changes in Stage-Discharge Relationships. Changes in stage-discharge relations due to channel improvements, levee construction or flow obstructions can best be evaluated by computing theoretical water surface profiles for each of a number of discharges. The resulting relationships for modified conditions can be used to modify routing criteria to enable evaluation of the downstream effects of these changes.

g. Effects of Multiple Reservoir Systems.

(1) Representative events. When more than one reservoir exists above a damage center, the problem of evaluating reservoir stages and downstream flows under project conditions becomes increasingly complex. Whenever practicable, it is best to make complete routings of five to ten historic flood events and a large event that has been developed from a hypothetical rainfall pattern. If necessary, it is possible to supplement these events by using multiples of the flow values. Care muse be exercised in selecting events that have representative flood volumes, timings, and areal distributions. Also, there should be a balance of events caused by particular climatic factors, i.e. snowmelt, tropical storm, thunderstorm, etc. Furthermore, the flood-volume-duration characteristics of the hypothetical events should be similar to the recorded events (see Section 3-8). Hypothetical events must be used with caution, however, because certain characteristics of atypical floods may be responsible for critical flooding conditions. Accordingly, such studies should be supplemented by a critical examination of the potential effects of atypical floods.

(2) Computer Program. It is generally impossible to make all of the flood routings necessary to evaluate the effect of a reservoir system by hand computations. Computer programs have been developed to route floods through a reservoir system with complex operational criteria (55).

3-10. Effects of Urbanization.

a. General Effects. Urbanization has two major effects on the watershed which influence the runoff characteristics. First, there is a substantial increase in the impervious area, which results in more water entering the stream system as direct runoff. Second, the drainage system collecting the runoff is generally more efficient and tends to concentrate the water faster in the downstream portion of the channel system. It is important to keep these two effects in mind when considering the changes in the flood peak frequency curve caused by increasing urbanization.

b. Effect on Frequency Relations. A general statement can be made about the effects of urbanization on flood-peak frequency relations. The usual effect on the frequency relation is to cause a significant increase in the magnitude of the more frequent events, but a lesser increase in the less frequent events. This results in an increase in the mean of the annual flood peaks, a decrease in the standard deviation and an unpredictable effect on the skew coefficient (see Figure 3-13). The resulting frequency relation may not fit any of the standard theoretical distributions. Graphical techniques should be applied if a good fit is not possible by an analytical distribution.

c. Other Considerations. The actual effect of urbanization at a specific location is dependent on many factors. Some of the factors that must be considered are basin slope, basin shape, previous land use and ground cover, number of depressional areas drained, magnitude and nature of urban development and the typical flood source (snowmelt, thunderstorm, hurricane, or frontal storm).It is possible for urbanization to cause a decrease in the flood peaks at a particular site. For instance, consider an area downstream of two tributary areas of such size and shape that the large floods are caused by the addition of the nearly coincident peaks from the two tributaries. Urbanization in one of the tributary areas will likely cause the contribution from this area to arrive downstream earlier. This change in the timing of the peaks would result in lower downstream peaks. Of course, when both areas have become equally urbanized, the flood peaks may coincide again. The construction of bridges or other encroachments can reduce the flood peak downstream, but causes backwater flooding upstream.

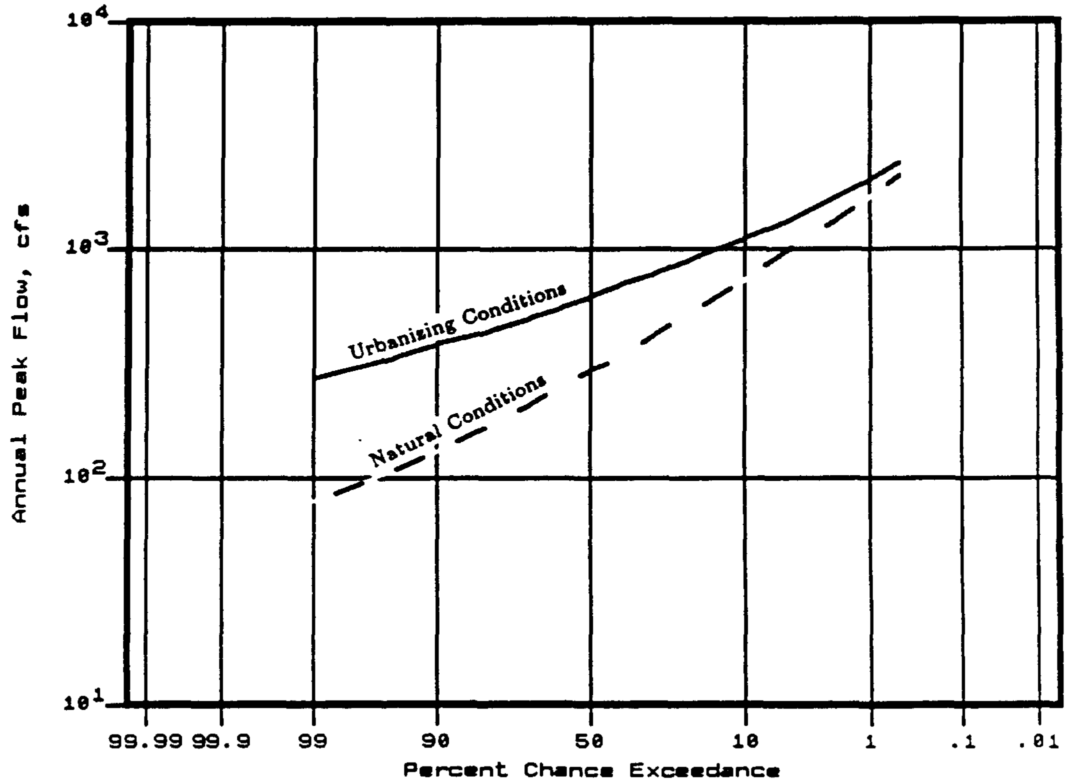


Figure 3-13. Typical Effect of Urbanization on Flood Frequency Curves.

d. Adjustment of a Series of Nonstationary Peak Discharges. When the annual peak discharges have been recorded at the outlet of a basin which has been undergoing progressive urbanization during the period of record, the peak discharges are nonstationary because of the varying basin condition. It is generally necessary to adjust the discharges to a stationary series representative of existing conditions. One approach to adjusting the peaks to a stationary series is as follows:

(1) Develop and calibrate a rainfall-runoff model for existing basin conditions and for conditions at several other points in time during the period of record.

(2) Develop a hypothetical storm for the basin using generalized rainfall criteria, such as that contained in Weather Bureau Technical Paper 40 (14). Select the magnitude of the storm, e.g., a 25-year recurrence interval, to be used. The recurrence interval is arbitrary as it is not assumed in this approach that runoff frequency is equal to rainfall frequency. The purpose of adopting a specific magnitude is to establish a base storm to which ratios can be applied for subsequent steps in the analysis.

(3) Apply several ratios (say 5 to 8) to the hypothetical storm developed in the previous step such that the resulting calculated peak discharges at the gage will cover the range desired for frequency analysis. Input the balanced storms to the rainfall-runoff model for each of the basin conditions selected in step (1), and determine peak discharges at the gaged location.

(4) From the results of step (3), plot curves representing peak discharge versus storm ratio for each basin condition (or point in time).

(5) Use the curves developed in step (4) to adjust the observed annual peak discharges. For example, an observed annual peak discharge that occurred in 1975 is adjusted by entering the "1975" curve (or interpolating) with that discharge, locating the frequency of that event, and reading the magnitude of the adjusted peak from the base-condition curve for the same frequency. The adjusted peak thus obtained is assumed to be the peak discharge that would have occurred for the catchment area and development at the base condition. It is not necessary to adjust to natural conditions. A stationary series could be developed for one or more points in time.

(6) A conventional frequency analysis can be performed on the adjusted peak discharges determined in the preceding step. If the data represent natural conditions, Bulletin 17B procedures would be applicable. If the basin conditions represent significant urbanization, graphical analysis may be appropriate.

e. Development of Frequency Curves at Ungaged Sites. There are several approaches that can be taken to develop frequency curves at ungaged sites that have been subject to urbanization. In order of increasing difficulty, they are: 1) application of simple transfer procedures (e.g., Q = CIA); 2) application of available region-specific criteria, e.g., USGS regression equations; 3) application of rainfall-runoff models to hypothetical storm events; 4) application of simple and detailed rainfall-runoff models with observed storm events and 5) complete period-of-record simulation. As approaches (3) and (4) are often applied, the computational steps are presented in some detail.

(1) Hypothetical Storm Approach

(a) Develop peak-discharge frequency curve for specific land use conditions from available gaged data and/or regional relationships.

(b) Develop balanced storms of various frequencies using data from generalized criteria, a nearby gage or the equivalent.

(c) Develop rainfall-runoff model for the specific watershed with the adopted land-use conditions. Calibrate runoff and routing parameters by reproducing observed hydrographs occurring under natural conditions.

(d) Input balanced storms (from b) to rainfall-runoff model (from c). Determine exceedance probabilities to associate with balanced storms from adopted specific land-use conditions peak discharge frequency curve (from a) with computed peak discharges.

(e) Modify parameters of rainfall-runoff model to reflect future urban runoff characteristics. Input balanced storms to the urban- conditions model.

(f) Plot results assuming frequency of each event is the same for both the adopted land use and the future urban conditions.

(2) "Simple" and Detailed Simulation of Historic Events

(a) Simulate all major historic events with a relatively simple model to establish the ranking of events and an approximate peak discharge for each. The approximate peaks could be developed by using a multiple linear regression approach, by using a very simple rainfall-runoff model, or by any other approach that will capture the hydrologic response of the basin.

(b) Perform a conventional frequency analysis of the approximate peaks obtained in step a.

(c) Make detailed simulations of selected events and correlate the more precise peaks with the approximate peaks.

(d) Use the relationship developed in step c to determine the desired frequency curve. The same approach can be followed for both existing and future conditions.