Prediction of Peak Flows for Culvert Design on Small Watersheds in Oregon

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ABSTRACT

Forest engineers must frequently make flood frequency estimates for very small watersheds when designing culvert installations. Empirical formulae and simplified rainfall runoff models, the most commonly used techniques to predict floods from very small watersheds, require considerable engineering judgement to give reasonable results. As an alternative to such methods, this study presents equations to predict peak flows on small watersheds in Oregon. The equations were developed from 80 watersheds ranging in size from 0.21 to 10.60 square miles.

Oregon was divided into six physiographic regions based on previous flood frequency studies. In each region, annual peak flow data from gaging stations with more than 20 years of record were analyzed using four flood frequency distributions (Gumbel, two-parameter log-normal, three-parameter log-normal, log Pearson type III). The log Pearson type III distribution was found to be suitable for use in all regions of the state, based on the chi-square goodness of fit test. Flood magnitudes having recurrence intervals of 10, 25, 50, and 100 years were related to physical and climatic indices of drainage basins by multiple regression analysis. Drainage basin area (A) was the most important variable in explaining the variation of flood peaks (Q_t) in all regions. Mean basin elevation (E) and mean annual precipitation (P) were also significantly related to flood peaks in two regions in western Oregon. The following equations to predict the 25-year flood were developed for each physiographic region in Oregon: (1) Willamette region $Q_{25} = 156A^{.80}$ (2) Coast region $Q_{25} = 6.31A^{1.01}E^{.51}$ (3) Cascade region $Q_{25} = .032A^{.44}P^{1.97}$ (4) Rogue-Umpqua region $Q_{25} = 163A^{.77}$ (5) Blue-Wallowa region $Q_{25} = 67.6A^{.47}$

(6) Klamath region $Q_{25} = 41.9A^{.79}$. Average percent error for all developed regression equations ranged from 16.1 to 64.1 percent, the smaller errors being associated with the more humid regions. Confidence limits developed for the regression equations provide the engineer with estimates of prediction uncertainties over the range of design flows. These prediction equations provide a better basis for culvert design on small forested watersheds than rules of thumb or empirical methods.

FOREWORD

The Water Resources Research Institute, located on the Oregon State University Campus, serves the State of Oregon. The Institute fosters, encourages and facilitates water resources research and education involving all aspects of the quality and quantity of water available for beneficial use. The Institute administers and coordinates statewide and regional programs of multidisciplinary research in water and related land resources. The Institute provides a necessary communications and coordination link between the agencies of local, state and federal government, as well as the private sector, and the broad research community at universities in the state on matters of water-related research. The Institute also coordinates the inter-disciplinary program of graduate education in water resources at Oregon State University.

It is Institute policy to make available the results of significant water-related research conducted in Oregon's universities and colleges. The Institute neither endorses nor rejects the findings of the authors of such research. It does recommend careful consideration of the accumulated facts by those concerned with the solution of water-related problems.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

Prediction of Peak Flows for Culvert Design
on Small Watersheds in Oregon

I. INTRODUCTION

Need for Investigation

Construction of forest road systems frequently requires installation of culverts at crossings of small streams. The Forestry Practices Act for Oregon states that culvert installations must be designed to accommodate floods having a return period of 25 years. The Siuslaw National Forest currently designs its culverts for a 25-year return interval event.[1] Overdesign of an installation results in needless expense, while underdesign may result in failure of the road, increased sediment in the stream system, and possible damage to aquatic habitat downstream of the installation. In order to design the most efficient installation possible, the engineer must have some means of estimating the magnitude and frequency of floods that can be expected from these small drainage areas.

The most common methods currently used for determining flood magnitudes from small drainage areas are empirical formulas and statistical frequency analysis. Numerous studies (Burnham, 1980; Dodge, 1972; Hetherington, 1974; Hiemstra and Reich, 1967) have discussed the strengths and weaknesses of empirical formulas. Empirical formulas do not utilize peak flow records from gaged watersheds and generally require considerable engineering judgment in order to produce acceptable results (Hiemstra and Reich, 1967).

Statistical frequency analyses use actual streamflow data and accepted statistical procedures to estimate the magnitude and frequency of floods. In addition, these methods require less engineering

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[1]Reim, J. Forest Hydrologist. Siuslaw National Forest. Personal Communication, July 8, 1981.

judgment to produce consistently reliable results. Previous flood frequency studies in Oregon have used streamflow data from all sizes of watersheds in the analysis. Whether equations developed in these studies can be used to accurately estimate peak flows from small watersheds is uncertain. There have been no studies in Oregon aimed specifically at the needs of the forest engineer for use in designing culvert installations which have used actual streamflow records from small drainage basins.

Objectives

The objectives of this study were twofold: first, to determine which of the many statistical frequency distributions now in use are most appropriate for small watersheds in Oregon; second, using the appropriate frequency distribution to develop equations to predict peak flows from small drainage basins in Oregon. The procedure developed to predict peak flows should meet the following criteria:

Accuracy. The method developed should yield the highest accuracy possible while still meeting the criteria of reproducibility and practicality.

Reproducibility. The procedure should be such that results obtained through its use can be consistently reproduced by a group of professionals.

Practicality. The procedure developed should be such that peak flows can be easily and routinely determined.

Procedure

Gaging station records were compiled for all watersheds smaller than five square miles and more than 50 percent forested, and having at least ten years of record. It was found that this data base involved too few stations to meet the needs of this study. As a result 12 stations over five square miles and nine stations over 50 percent forested were included in the study. After reviewing the type and amount of data available a review of the literature was made to determine which technique of flood estimation for ungaged watersheds would best suit the objectives of the study and the limitations of the data base.

Following selection of a technique, the data was organized into homogeneous regions and an analysis of the long-record stations was made. Long-record stations were defined as having more than 20 years of record. A minimum of five long-term stations in each hydrologic region was used in this analysis. In several regions there were not five stations with 20 years of record. When this was the case the smallest stations for which 20 years of record could be found were used in the analysis. Four frequency distributions were considered for use: type I extremal (Gumbel); 2 parameter log-normal; 3 parameter log-normal; and log Pearson type III with station and regionalized skew coefficient. A chi-square test was used to determine the goodness of fit of each distribution.

The chosen distribution was then used to calculate the magnitude of the 10, 25, 50, and 100 year flood events for each gaged station in the region. Specific watershed characteristics that were expected to correlate with the calculated flood peaks were then tabulated. Regression analysis was used to determine equations relating flood peaks of selected recurrence interval with specific watershed parameters. As part of the regression analysis an error analysis was made to determine the accuracy of the prediction equations. The equations relating flood peaks to watershed characteristics are the final result of this study and are in the form

$$Q_t = b_o X_1^{b_1} X_2^{b_2} \ldots X_n^{b_n}$$

where, Q_t = peak flow having a probability of occurring one time in t years

b_o = regression constant

$b_1 \ldots b_n$ = exponents determined by regression

$X_1 \ldots X_n$ = independent variables

Recommendations for use and application of the results are outlined.

II. REVIEW OF LITERATURE

The literature abounds with procedures for determining the magnitude and frequency of floods for ungaged watersheds. A recent, comprehensive review by McCuen et al. (1977) evaluated over 240 studies. Although the number of studies is voluminous, the types of procedures used to determine flood flow frequency estimates at ungaged locations can be classified into four general categories: (1) statistical methods; (2) empirical equations; (3) transfer methods; and (4) simulation models. The procedures can be further expanded into eight categories: (1) statistical estimation of Q_t; (2) statistical estimation of moments; (3) index flood technique; (4) empirical methods; (5) estimation by transfer of Q_t; (6) single storm event; rain frequency is proportional to runoff frequency; (7) multiple discrete events; and (8) continuous record simulation (McCuen et al., 1977).

Statistical Methods

Statistical estimation of Q_t

This procedure uses observed data at gaged stations to derive prediction equations for selected peak frequency discharges. Data from regionally based streamflow gages are first fitted to an appropriate frequency distribution. The frequency distribution is then used to determine the magnitude of the peak discharges to be evaluated. The values of Q_t are then regressed against selected meteorological and physical basin characteristics. The end result is a series of equations relating peak discharges at specific recurrence intervals to selected watershed parameters.

The first statistical analysis of streamflow records in Oregon was by Lystrom (1970). A log Pearson type III frequency distribution was used in the analysis. The data base consisted of 304 gaging stations having records longer than ten years. Stations used in the analysis ranged in size from 0.21 square miles to 11,300 square miles. The state was divided into two regions along the crest of the Cascades. Equations were developed to predict the discharges of the 2, 5, 10, 25, and 50 year return interval events. In western Oregon the most significant variables were watershed area, percent of area in lakes and ponds, elevation of the watershed, mean annual precipitation, precipitation intensity, mean minimum January temperature and a soils index. In eastern Oregon the most significant variables were watershed area, percent forest cover, mean annual precipitation, and a soils index. Standard errors of estimate ranged from 40 to 46 percent in western Oregon and from 56 to 60 percent in eastern Oregon.

Harris et al. (1979) used this technique to develop equations relating the magnitude of the 2, 5, 10, 25, 50, and 100 year flood events to various basin characteristics in western Oregon. The data base consisted of 239 stations ranging in size from 0.21 to 7,820 square miles. The data were fitted to a log Pearson type III distribution which was then used to calculate the value of the various peak flows. Western Oregon was divided into four hydrologically homogeneous regions for the purpose of regionalizing the data base (Fig. 1). A step backward regression technique was used to determine the most significant variables for inclusion in the regional equations. Variables included in the final equations were drainage area, two year 24-hour precipitation, percent of area in lakes and ponds, and percent forest cover. Standard errors of the estimate ranged from 32 to 72 percent. The results of this study are shown in Table I.

A study by Cummans et al. (1975) in Washington used essentially the same technique as Harris to develop prediction equations for 12 homogeneous regions in the state. The data base consisted of streamflow records from 452 gaging stations. The basins included in the study varied in size from 0.15 to 3,550 square miles. A step forward regression analysis was used to determine the most significant variables for inclusion in the regional equations. In western Washington drainage area and mean annual precipitation were found to be the most

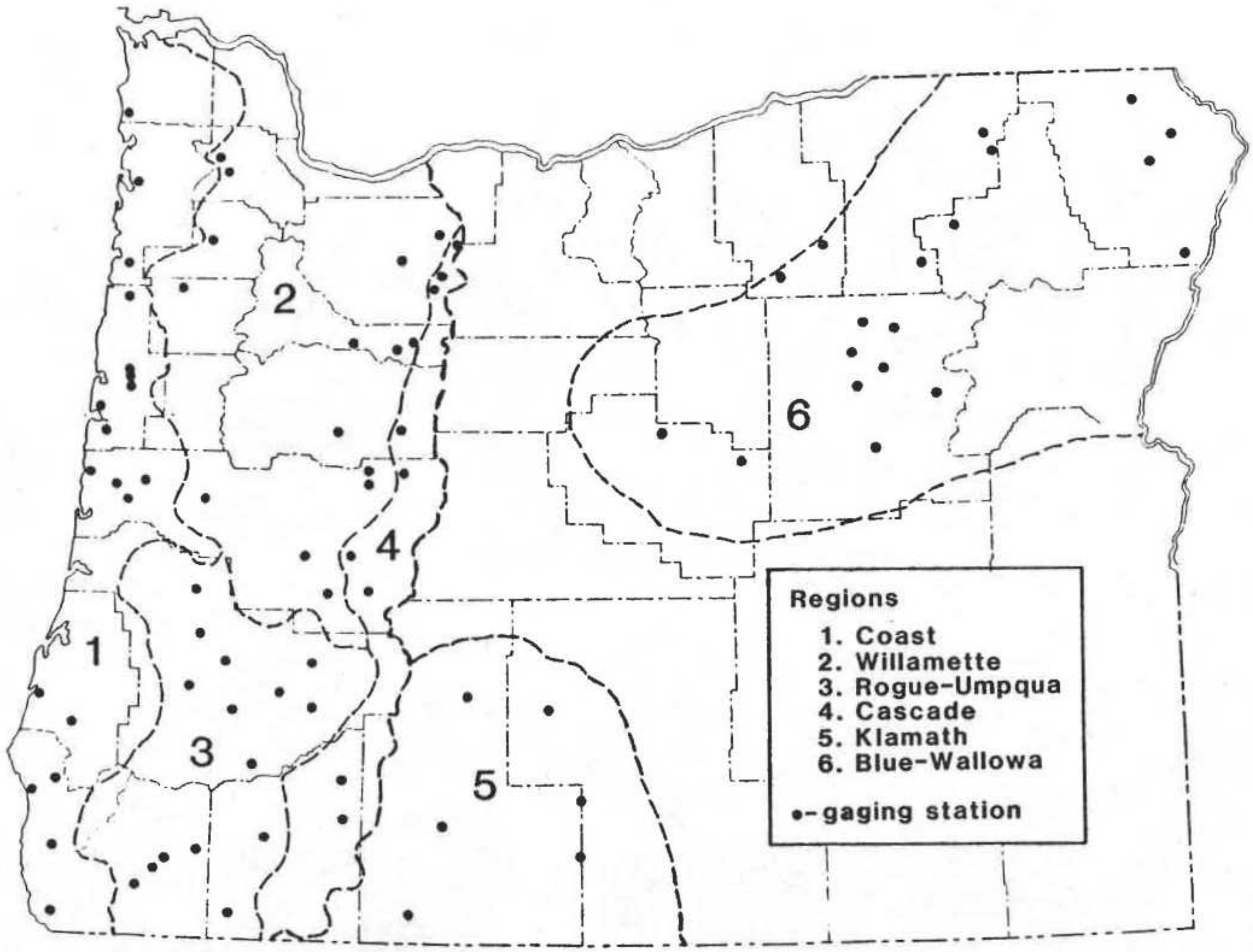

Figure 1. Flood-frequency regions of western Oregon. Regions 1-4 are used by Harris et. al. (1979). Regions 5,6 were added for this study.

TABLE I. Flood prediction equations for western Oregon. From Harris et al. (1979).

General form of equation $Q_T = KA^a(ST+1)^b(101-F)^c I^d$ where

Q_T=discharge for selected exceedance probability,

K = regression constant,

A = drainage area, in square miles,

ST = area of lakes and ponds, in percent,

F = forest cover, in percent, and

I = precipitation intensity, in inches.

(When the functions of F and ST are not significant, the factors $(ST+1)^b$ and $(101-F)^c$ are omitted from the equation.)

Exceedance probability (RI)1/	Equation	Percent standard error
	(1) COAST REGION (40 stations)	
$Q_{0.5}(2)$	$= 4.59A^{0.96}(ST+1)^{-0.45}I^{1.91}$	33
$Q_{0.2}(5)$	$= 6.27A^{0.95}(ST+1)^{-0.45}I^{1.95}$	32
$Q_{0.1}(10)$	$= 7.32A^{0.94}(ST+1)^{-0.45}I^{1.97}$	33
$Q_{0.04}(25)$	$= 8.71A^{0.93}(ST+1)^{-0.45}I^{1.99}$	34
$Q_{0.02}(50)$	$= 9.73A^{0.93}(ST+1)^{-0.44}I^{2.01}$	35
$Q_{0.01}(100)$	$= 10.7A^{0.92}(ST+1)^{-0.44}I^{2.02}$	37
	(2) WILLAMETTE REGION (111 stations)	
$Q_{0.5}(2)$	$= 8.70A^{0.87}I^{1.71}$	33
$Q_{0.2}(5)$	$= 15.6A^{0.88}I^{1.55}$	33
$Q_{0.1}(10)$	$= 21.5A^{0.88}I^{1.46}$	33
$Q_{0.04}(25)$	$= 30.3A^{0.88}I^{1.37}$	34
$Q_{0.02}(50)$	$= 38.0A^{0.88}I^{1.31}$	36
$Q_{0.01}(100)$	$= 46.9A^{0.88}I^{1.25}$	37
	(3) ROGUE-UMPQUA REGION (60 stations)	
$Q_{0.5}(2)$	$= 24.2A^{0.86}(ST+1)^{-1.16}I^{1.15}$	44
$Q_{0.2}(5)$	$= 36.0A^{0.88}(ST+1)^{-1.25}I^{1.15}$	43
$Q_{0.1}(10)$	$= 44.8A^{0.88}(ST+1)^{-1.28}I^{1.14}$	44
$Q_{0.04}(25)$	$= 56.9A^{0.89}(ST+1)^{-1.31}I^{1.12}$	46
$Q_{0.02}(50)$	$= 66.7A^{0.90}(ST+1)^{-1.33}I^{1.10}$	49
$Q_{0.01}(100)$	$= 77.3A^{0.90}(ST+1)^{-1.34}I^{1.08}$	51
	(4) HIGH CASCADES REGION (28 stations)	
$Q_{0.5}(2)$	$= 4.75A^{0.90}(ST+1)^{-0.62}(101-F)^{0.11}I^{1.17}$	55
$Q_{0.2}(5)$	$= 8.36A^{0.86}(ST+1)^{-0.81}(101-F)^{0.08}I^{1.30}$	50
$Q_{0.1}(10)$	$= 11.3A^{0.85}(ST+1)^{-0.92}(101-F)^{0.07}I^{1.37}$	53
$Q_{0.04}(25)$	$= 15.4A^{0.83}(ST+1)^{-1.03}(101-F)^{0.05}I^{1.46}$	59
$Q_{0.02}(50)$	$= 18.8A^{0.82}(ST+1)^{-1.10}(101-F)^{0.04}I^{1.52}$	66
$Q_{0.01}(100)$	$= 22.6A^{0.81}(ST+1)^{-1.17}(101-F)^{0.03}I^{1.57}$	72

1/ Numbers in parentheses refer to recurrence intervals in years.

significant variables. In eastern Washington, drainage area, mean annual precipitation, and percent forest cover were found to be significant. Standard errors of the estimate ranged from 24.6 to 60.7 percent in western Washington and from 41.7 to 129 percent in eastern Washington.

Waananen and Crippen (1977) used the statistical estimation technique to develop peak flow equations for California. As in the previous two studies cited, a log Pearson type III distribution was used to determine the magnitude of selected peak flow events. Two of the regions developed in this study, the North Coast region, and the Northeast region are immediately adjacent to the Oregon border. The data base for the North Coast region consisted of 141 stations ranging in size from 0.05 to 3,113 square miles. In the Northeast region the data base consisted of 31 stations ranging in size from 0.06 to 24.8 square miles. A step forward regression analysis technique was used to determine the regional equations. Drainage area, mean annual precipitation, and an altitude index were found to be significant in the North Coast region. The altitude index was computed as the average of the altitudes at the 10 and 85 percent points along the main stream channel, in thousands of feet. Drainage area was found to be the only significant variable in the Northeast region, which included only stations under 25 square miles. Standard errors of the estimate ranged from 0.24 to 0.26 $\log_{10}$ units in the North Coast region and from 0.38 to 0.46 $\log_{10}$ units in the Northeast region.

The statistical estimate of Q_t permits direct calculation of specific peak flows that are individually and statistically derived. This procedure is quick and easy to use, requiring little engineering judgment to make reasonable predictions. The primary disadvantage of this procedure is that, because of its ease of use, the equations are often misused by application to areas with different hydrologic characteristics than those used in developing the prediction equations.

Statistical estimation of moments

The statistical estimation of moments procedure is similar to the statistical estimation of Q_t procedure except that instead of correlating specific peak flows against basin characteristics, the moments (mean, standard deviation, and skew) of the frequency distribution for each gaged station used in the analysis are correlated with selected basin characteristics. The end results of this technique are prediction equations which define the moments of the estimated frequency curve at the ungaged location in terms of drainage basin characteristics.

The ability to derive the frequency curve directly is the major advantage over a statistical estimation of Q_t, if the entire range of flood flow frequencies is required in the analysis. Most practicing professionals feel that it is impossible to relate the skew coefficient, and often the standard deviation in a statistically meaningful manner to basin characteristics (Burnham, 1980). Other than this, the advantages and disadvantages are the same as for the statistical estimation of Q_t.

Index flood method

The index flood method described by Dalrymple (1960) uses procedures similar to the previous statistical methods except that only the mean annual flood is regressed against basin characteristics. The final product is composed of two parts. First, a dimensionless frequency curve for each region relating the ratio of selected flow events and mean annual flood to return interval. Second, a graph or equation relating mean annual flood to basin characteristics.

The first step in applying this technique is to determine flood flow frequency estimates at gaged locations. As originally developed the type I extremal (Gumbel) distribution was fitted graphically to peak flow records (Dalrymple, 1960). However, any appropriate frequency distribution may be used in this technique. Flow frequency data for each station are made dimensionless by dividing selected discharges computed from the frequency distribution by the mean annual flood (Q_a) determined from the frequency distribution. The median ratios of (Q_t/Q_a) for selected discharges (Q_t) are plotted against return period in years. A curve is fitted through these plotted data which represents the frequency curve for the region. Regression analysis is then used to determine a relationship between the mean annual flood and selected basin characteristics.

To make an estimate for an ungaged watershed using the index flood method the derived prediction equation is used to estimate the mean annual flood. The regional frequency curve is then used to determine the ratio of mean annual flood to the desired flood. An example would be, $Q_{10} = 1.7Q_a$ and $Q_{100} = 3.5Q_a$.

Wheeler (1971) developed 29 regional frequency curves covering 26 drainage basins in Oregon using this technique. The data base consisted of more than 120 recording and crest stage gaging station records. Most of the records were longer than 20 years. Frequency curves relating the mean annual flood to basin area are given along with a series of multipliers to be used to determine the flood magnitude with return intervals up to 100 years. Wheeler states that the results of the study are only applicable for stations under 100 square miles in area. No estimate of the accuracy of the results is given.

Morgan (1962) used the index flood technique to develop a method for predicting the discharge of the 50-year flood event in Oregon. The type I extremal distribution was used to determine the magnitude of the 50-year flood at gaged locations. The 50-year flood event for each site was then plotted against drainage area for that station. A family of lines was fitted by eye to the mass of points, each of which was greater than the line below it by a factor of 1.44. Each line was assigned an arbitrary index number in multiples of ten. The index numbers for all sites were then plotted on a map of Oregon and isopleths of equal index numbers were drawn. To predict the

50-year event, the drainage area and index number are entered into the graph, relating area, index number and flood magnitude. The 50-year discharge is then read from the graph. Stations used in the study ranged in size from 0.21 to 11,300 square miles. Standard error of the estimate ranged from 50 to 113 percent (Jenkins, 1968).

The advantages and disadvantages of the index flood method are the same as for the other statistical estimation techniques. Cruff and Rantz (1965) found the index flood technique to be less suitable than statistical estimation of Q_t for use in coastal California. The index flood method is probably the least accurate of the statistical methods (Burnham, 1980).

Empirical Equations

Empirical equations are widely used by practicing engineers to design culverts. In terms of procedures actually being used, empirical equations probably outnumber all other methods. The ease of using these equations, and lack of alternative procedures, is no doubt responsible for their great popularity. The most frequently used empirical equations for culvert design are the rational formula, Manning's formula, BPR method, and Talbot's formula.

The rational formula

The rational formula is probably the most common empirical method used to predict peak flows from small watersheds. The rational formula was introduced into the United States by Emil Kuichling in 1889 for analysis of sewer discharge in municipal areas. The rational formula is:

$$Q_p = CIA$$

where, Q_p = the peak runoff rate (cfs)

C = a runoff coefficient (assumed to be dimensionless)

I = the average rainfall intensity (in/hr) lasting for a critical period of time, t_c

t_c = time of concentration of the watershed (hrs)

A = size of the drainage area (acres)

Numerous assumptions are necessary for use of the rational formula.

1. The rate of runoff equals the rate of supply (rainfall excess) if t_{rain} is greater than or equal to t_c.
2. Maximum discharge occurs when entire area is contributing runoff simultaneously.
3. At equilibrium conditions the duration of rainfall at intensity I is $t = t_c$.
4. Rainfall is uniformly distributed over the basin.
5. Return period of Q_p is the same as the frequency of occurrence of rainfall intensity I.
6. The runoff coefficient is constant between storms and during a given storm and is determined solely by basin surface conditions.

Klingeman[2] notes eight limitations of the rational formula.

1. Actual rainfall is not uniform in intensity or in distribution over a basin.
2. Time distribution of actual rainfall may not be constant.
3. Only applicable to very small basins. Viessman et al. (1972) gives an upper limit of applicability of one square mile.
4. The assumption that "rate of runoff equals rate of supply when t_{rain} is greater than or equal to t_c" is only true for areas of less than a few acres. For larger basins, overland storage and channel storage affect the peak discharge but the

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[2] Study Book for Engineering Hydrology, Class CE 411, Dept. of Civil Engineering, Oregon State University.

formula does not account for this.

5. The formula is not valid if rainfall duration is less than the time of concentration.

6. There are many limitations on C.

   a) It does not include the varying affects of overland flow and channel storage.

   b) It varies from storm to storm.

   c) It changes as the land becomes saturated during a storm.

   d) It changes for a basin as land use changes over the years.

7. The value of Q_p is no better than the estimated value of C.

8. The frequency of occurrence of Q_p is not necessarily the same as the frequency of occurrence of I, since rainfall and runoff magnitudes do not always correlate well.

Heimstra and Reich (1972) tested the rational formula against 60 recorded flood events on 45 agricultural research station watersheds. Area of the watersheds used in the study ranged from 0.12 to 8.16 square miles. The rational formula overestimated the peak flows by an average factor of 2.01. Heimstra and Reich had difficulty in defining the runoff coefficient C. Dodge (1972) recommends that the rational formula not be used to predict peak flows from natural pervious watersheds but may provide reasonable estimates of peak flows from small impervious areas.

Manning's formula

Manning's formula developed in 1889 for flow in open channels is sometimes used for peak flow prediction. Manning's formula is

$$Q = \frac{1.49}{n} A \, R^{2/3} \, S^{1/2}$$

where, Q = peak discharge (cfs)

A = cross-sectional area of the stream (ft^2)

S = slope of the water surface

n = roughness coefficient of the streambed

R = hydraulic radius, A/WP (ft)

WP = wetted perimeter of the stream (ft)

The assumption for use of Manning's equation is that of uniform steady flow. It is doubtful that this condition ever exists in natural streams, especially in relatively high gradient forested watersheds. Area and wetted perimeter are determined by field observation of high water marks. The flow calculated by Manning's equation has a return period corresponding to the flow from which A is determined. The difficulty in determining the high water mark makes this method impractical for any flows with return periods longer than about five years.

Bureau of Public Roads (BPR) method

The BPR method was developed by Potter (1961) specifically for rural watersheds smaller than 25 square miles. The method is the result of a study of 96 gaged watersheds in the U.S., all located east of the 105th meridian. Potter's method groups all watersheds in the study region into four physiographic zones based on geologic similarity.

To apply Potter's method in estimating the ten- and 50-year flood peaks a topographic index T is first determined as:

$$T = \frac{0.3L}{\sqrt{S_1}} + \frac{0.7L}{\sqrt{S_2}}$$

where, L = total length of the stream channel from the point where the channel begins, to the outlet (miles)

S_1 = average slope of the upper third of the main stream channel (ft/mile)

S_2 = average slope of the lower third of the main stream channel (ft/mile)

A precipitation index, P, is defined as the ten-year one-hour rainfall (in inches), and determined from precipitation frequency maps or rainfall-intensity-duration frequency curves. An initial trial value for Q_{10} is determined from a set of graphs relating area, precipitation index, and topographic index. A test is then conducted to determine if the watershed is topographically similar to the basins studied by Potter. If the difference between the actual and ideal topographic index is less than 30 percent, the trial value of Q_{10} is accepted. If the difference between indices is greater than 30 percent, the ratio of actual topographic index to ideal topographic index is used to determine a compensating multiplier, C. The trial value of Q_{10} is then multiplied by C to obtain the ten-year peak flow. The 50-year peak flow is determined by a graph relating the ten-year flood to the 50-year flood. Potter suggests that peak discharges for other recurrence intervals be obtained by plotting Q_{10} and Q_{50} on extremal (Gumbel) probability paper and drawing a straight line through the plotted points.

Heimstra and Reich (1967) found Potter's method to yield unbiased estimates and had less variability than other empirical methods. On the average, Potter's method overpredicted floods by a factor of 1.54 which Heimstra and Reich felt was an acceptable safeguard against underdesign. The main advantage of Potter's method compared to other empirical methods tested by Heimstra and Reich was that all factors used in the method are incorporated into charts and no evaluations of coefficients, runoff curve numbers, or infiltration capacities are necessary. Judgment plays a much smaller role in applying this method than in the other empirical methods, hence reproducibility of estimates is very good.

The main disadvantage of Potter's method is that the method assumes that the same watershed factors are important in all areas of the country. Numerous studies (Thomas and Benson, 1969; Benson, 1964; Benson, 1962; Reich, 1971; Crosby, 1975; Harris et al., 1979) have shown the importance of different factors as flood predictors in various parts of the United States. On the basis of these studies it would be unreasonable to assume that Potter's method can be applied everywhere.

Talbot's formula

One of the simplest empirical formulas used for predicting peak flows from small watersheds is Talbot's formula

$$A = C\ M^{.75}$$

where, A = area of culvert opening (ft^2)

C = Talbot's constant

M = drainage area (acres)

This formula, developed from very high intensity storm data in the midwestern U.S., assumes a given rainfall intensity of four inches per hour (Hetherington, 1974). The uncertainty associated with the runoff coefficient C is the major limitation of Talbot's formula. The runoff coefficient includes all of the topographic and meteorologic variability of the watershed and so could be expected to vary from one watershed to another and from storm to storm. To correctly evaluate the runoff coefficient for any particular watershed would require considerable judgment and experience.

Estimation by Transfer of Q_t

The U.S. Water Resources Council (1978) defines this method as transferring selected peak flow values immediately upstream or downstream of a gaged location to an ungaged location. An adjustment is made in the peak flow values based on drainage areas. Burnham (1980) broadened this category to include transfer of peak flow values from a gaged location to a nearby ungaged location, not necessarily in the same watershed. This procedure is quick and easy to use and may provide a quick check on other methods for reasonableness of use.

The determination of adequate correlation factors is very difficult as these factors vary from basin to basin. Generally the estimates made by transfer procedures are not accurate enough for most analysis requirements but do serve a useful purpose as a quick check on other methods (Burnham, 1980).

Simulation Models

Single storm event: Rainfall frequency proportional to runoff frequency

Single storm event methods predict the peak flow from a watershed by developing relationships between rainfall frequency and runoff frequency. The peak flow is generally calculated from a generalized hydrograph. The final result is a runoff frequency hydrograph for a selected rainfall event. The most widely used method of this type for culvert design purposes is that developed by the Soil Conservation Service (1972).

The method is based on a dimensionless unit hydrograph having 37.5 percent of the volume on the rising side. A triangular approximation to the unit hydrograph is used to derive an equation relating the peak of the hydrograph to drainage area of the basin and runoff. The equation for the peak of the hydrograph is:

$$q_p = \frac{484AQ}{T_p}$$

where, q_p = peak runoff rate (cfs)
A = drainage area (mi^2)
Q = direct runoff (in)
T_p = time to peak of the hydrograph (hrs)

Any change in the dimensionless unit hydrograph reflecting a change in the percent of volume under the rising limb would cause a corresponding change in the constant, 484. This constant has been known to vary from about 600 in steep terrain to 300 in very flat, swampy country (Mockus), 1972). Input requirements for the SCS method include length of the longest watercourse in miles, elevation difference between the highest point in the watershed and the gaging station in feet, area of the watershed in square miles, soil type and vegetation types by area, and rainfall intensity for either the 25-year or 50-year six-hour duration event, depending on the peak flow being estimated.

The first step in using this method is to classify the soils into hydrologic groups (A, B, C, or D) on an area weighted average. Soils classed as A have high infiltration rates while those classed as D have the lowest infiltration rates. The SCS publishes a list of over 4000 soils in the United States and their hydrologic soil group. The classifications are based on the judgment of soil scientists and assume that surfaces are bare, maximum swelling had taken place, and rainfall rates exceeded surface intake rates. Next, a "runoff curve number" (CN) is determined from a table using soil class and predominant vegetation characteristics. The runoff curve number is a function of land use and treatment, hydrologic condition, soil group, and antecedent moisture condition. The runoff curve number is defined as:

$$CN = \frac{1000}{10+S}$$

where S is the storage capacity of the soil (inches). The value of S depends on the antecedent moisture conditions (AMC). Antecedent moisture condition is classed either I, II, or III and depends on the amount of rainfall in the five-day period preceding the design storm. The value of S includes an initial abstraction, I_a, which is the combined interception, infiltration, and surface storage occurring before runoff begins. The initial abstraction is defined as

$$I_a = 0.2S \quad .$$

This relation for I_a was empirically derived from rainfall-runoff records on experimental watersheds under ten acres in size. A graph relating design storm precipitation and runoff curve number to direct runoff (Q) is used to determine the amount of runoff. The graph was developed from the equation

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

where P is the design storm precipitation (inches). The next step is to calculate the time of concentration as,

$$T_c = \frac{11.9L^3}{H}$$

where, T_c = time of concentration (hours)

L = length of the longest watercourse (miles)

H = change in watershed elevation between highest point in the watershed and outlet (feet)

This relationship was empirically derived from experimental agricultural watersheds. The time to peak (T_p) is then calculated as,

$$T_p = \sqrt{T_c} + 0.6T_c$$

The peak discharge is then calculated as,

$$q_p = \frac{484AQ}{T_p}$$

which is the equation for the peak of a triangular hydrograph having an assumed shape as discussed earlier.

There are numerous limitations to use of the SCS method of flood prediction, particularly in the forest environment. The hydrologic soil classification is at best a coarse estimate of the infiltration rate of a soil and should be viewed as a qualitative, rather than quantitative estimate. In the forest environment the presence of root channels, decayed organic matter, and animal burrows tend to increase the infiltration rate of a soil. A forest soil that is classed in group C could have a greater infiltration rate than an agricultural soil classed B or A. Until on-site measurements of infiltration rates for specific soils become available, use of the SCS hydrologic soil classes in forested areas should be viewed with skepticism. The equation relating the initial abstraction to storage capacity was empirically derived from agricultural watersheds. It is unlikely that a forested area would have the same relationship between interception, surface storage, and storage capacity as these very small agricultural watersheds. The data used to develop this relationship show a very large amount of scatter (Mockus, 1972). The constant used in the peak flow equation depends on the shape of the flood hydrograph. Even though this constant is known to vary from 300 to about 600, the value of 484 has become accepted because information about the true shape of the flood hydrograph is generally not available for ungaged watersheds. Finally, it is uncertain that the 25- and 50-year six-hour rainfall events produce the 25- and 50-year flood events.

Heimstra and Reich (1967) tested the SCS method on 64 separate storms on Agricultural Research Station watersheds and found that it underestimated the flood peaks 86 percent of the time by an average of 36 percent. In forested terrain Williams (1971) tested the SCS method in an attempt to predict the flood event of June, 1965 on Duck Creek, near Brockway, Montana. Williams found that the SCS method predicted a peak flow more than twice as large as the recorded peak.

Single event simulation models for specific areas of the country have been developed by Chie and Bittler (1969), Reich (1971), and Rogowski (1974). These models are similar in that they all assume some underlying flood hydrograph shape, which is then related to precipitation and other basin characteristics. Reich (1971) developed both a single event simulation model and a statistical estimation of Q_t

procedure for small watersheds in Pennsylvania and concluded that the statistical estimation technique was superior.

Multiple discrete events and continuous record simulation models

These two methods are discussed together because they share most of the same advantages and disadvantages. The multiple discrete events method uses historical rainfall runoff event data and single event simulation models. The results are maximum peak annual hydrographs for each year of the period of record. These values are then used to develop corresponding frequency curves. The continuous record method provides a continuous simulation of the hydrologic rainfall runoff process for a specified period of record. Continuous historic rainfall records are typically used in this method.

These mathematically elegant models generally have input requirements beyond what is available or practical to collect for small watersheds. The Stanford Watershed model IV utilizes three physical parameters, eleven land surface parameters, and six channel system parameters (Crawford and Linsley, 1966). The SSAR model developed by the U.S. Army Corps of Engineers requires a soil moisture index, base infiltration index, evapotranspiration index and a snow melt coefficient (Army Corps of Engineers, 1972). Such models are ideally suited to comprehensive evaluations of larger river systems, but their input and calibration requirements make them infeasible for use on small watersheds for culvert design since they must be applied on a site-by-site basis. The advantages and disadvantages of these techniques are outlined by Burnham (1980).

Selection of a Frequency Distribution

All statistical frequency analyses require use of a statistical distribution to determine the return periods of recorded events of known magnitude and to estimate the magnitude of design events beyond the range of recorded data. The sample data are used as an estimate of an unknown population to calculate the parameters of the selected probability distribution. The fitted probability distribution is then used to calculate the magnitude of events having return periods greater than those of the recorded events. The question of which frequency distribution is most appropriate for analyzing flood series is the source of much controversy among hydrologists (Kite, 1977).

Numerous frequency distributions have been advocated by hydrologists including the normal, two parameter log-normal, three parameter log-normal, Pearson type III, log Pearson type III, type I extremal (Gumbel), and, most recently, the Wakeby distribution. In 1968 the U.S. Water Resources Council adopted the log Pearson type III distribution for use by federal agencies (Benson, 1968). The council could give no statistical grounds for selection of this distribution over any others except that as a three parameter distribution it offers considerable flexibility, when the skew coefficient equals zero it reduces to the two parameter log-normal.

Numerous studies have compared distributions with varying results. Spence (1973) compared the normal, two parameter log-normal, type 1 extremal, and log type 1 extremal for use on the Canadian prairies and found that the two parameter log-normal was the best fitting distribution. In a comparison of six distributions in California, Cruff and Rantz (1965) found the Pearson type III to be the best distribution. Santos (1970) found the two parameter log-normal distribution better than the Pearson type III. Cicioni (1973) tested the two parameter log-normal, three parameter log-normal, Pearson type III, and extremal type I distributions on 108 watersheds in Italy and found the two parameter log-normal to be the most suitable. Stolte and Dumontier (1977) tested the two parameter log-normal, three parameter log-normal, Pearson type III, log Pearson type III, type I extremal, and log type I extremal distributions for use in mountainous and prairie watersheds in Saskatchewan. They found that the three parameter log-normal distribution was the most suitable for evaluation of long record stations but that for stations with records from 1950 to 1977 the two parameter log-normal distribution gave the best estimate of the 100-year flood. They also found that the type I extremal distribution tended to underpredict flood peaks and that the log Pearson type III tended to overpredict the flood peaks. Hutchinson et al. (1977) tested the two parameter log-normal, three parameter log-normal, type I extremal, and log Pearson type III distributions on long record stations in New Brunswick. They found that the three parameter log-normal and log Pearson type III distributions gave very similar results but adopted the three parameter log-normal distribution because the standard error of the estimate was slightly lower. In contrast to Stolte (1977), Hutchison found that in certain cases the log Pearson type III distribution underpredicted high return period events. A survey by Reich (1973) found that most practicing hydrologists and engineers felt the log Pearson type III distribution gave better results than either the type I extremal or log type I extremal distributions. In a study of 100 long-record gaging stations, Benson (1962) found that no one distribution gave consistently better results.

Selection of an appropriate frequency distribution must depend on the conditions of the study. Goodness of fit of a frequency distribution with the recorded data is a necessary but not sufficient condition for acceptance (Kite, 1977). Both parametric and nonparametric goodness of fit tests, such as Chi-square, Kolmogorov-Smirnov, Cramer-Von Mises and Anderson-Darling tests have been used to help determine the best-fitting frequency distribution (Cicioni et al., 1973). The U.S. Water Resources Council work group concluded that a frequency distribution could not be selected on the basis of statistical grounds alone (Benson, 1968).

Although statistical tests cannot by themselves be used to determine the best frequency distribution for use, they can, in many cases, provide reasons why certain distributions should not be used.

Both the Pearson type III and log Pearson type III distributions require estimation of the coefficient of skewness from the sample data. The variability of estimates of the coefficient of skew tends to be very large particularly for short records. This may be sufficient reason to prefer another distribution (Matalas et al., 1975; Houghton, 1978). The Water Resources Council recommends that a regionalized value of the skew coefficient be used for stations having less than 25 years of record. However, use of a two parameter distribution implies a fixed value of skew for all stations which is also questionable. The two parameter distribution is only satisfactory when the skew of the logarithms of the data is close to zero. The Gumbel distribution will only be suitable if the coefficient of skew of the recorded events is very close to 1.13. The three parameter log-normal distribution may be satisfactory only if the coefficient of skew of the reduced data ($\ln(x-a)$) is close to zero.

There are four techniques that can be used to estimate the parameters of a distribution. In order of increasing efficiency they are graphical, least squares, method of moments, and method of maximum likelihood (Kite, 1977). Because of the length of computations required in finding solutions by the method of maximum likelihood, the method of moments has become the standard fitting technique. Benson (1962) states that mathematical fitting of a probability distribution is preferable to graphical fitting because

1. mathematical fitting is theoretically better.
2. mathematical fitting eliminates the subjectivity of individual judgment.

However, the very lack of subjectivity of mathematical fitting may sometimes be a disadvantage. The inclusion or exclusion of one or two events may cause large changes in the resulting frequency function. The individual drawing in a curve by eye may elect to exclude certain events in order to gain a better fit to the rest of the data. For stations with limited data, Reich (1981) recommends graphical fitting of two parameter distributions on probability paper. Because of the skew coefficient the log Pearson type III distribution cannot be linearized on one type of graph paper, making graphical fitting of a straight line on probability paper impossible. Reich states that simply because mathematical fitting procedures produce uniform results does not mean they are the best procedures for use if those results are uniformly inaccurate.

III. METHODOLOGY

Selection of a Procedure

The statistical estimation procedure was chosen as the most applicable technique for this study. This procedure fills the requirements of accuracy, reproducibility, and practicality as outlined in the objectives for this study. Of the three statistical estimation procedures, statistical estimation of Q_t was selected as being the most appropriate technique. Engineers designing culvert installations are primarily concerned with the magnitude of individual flood events and are only secondarily interested in the frequency curve for a site. For this reason a statistical estimation of Q_t was selected over a statistical estimation of moments procedure. Cruff and Rantz (1965) have concluded that a statistical estimation of Q_t is superior to the index flood method.

The data for this study is in the form of maximum yearly discharges, thus a rainfall-runoff model would not make full use of the data available. Empirical methods and transfer methods fail to meet the criteria of accuracy and the more sophisticated simulation models fail to meet the criteria of reproducibility and practicality as outlined in the objectives.

Regionalization of Data

Oregon was divided into six regions in order to try and organize the data into hydrologically homogeneous areas. In western Oregon the regions developed by Harris et al. (1977) were used. These regions were developed by fitting all of the data used in their study to a log Pearson type III distribution and determining regression equations for the entire state. The residuals of these equations were plotted on a map of Oregon. The residual plot and topographic maps were evaluated to delineate the boundaries of the four flood frequency regions used in their analysis. These regions were used in this study because our data base was not large enough to adequately define homogeneous regions based on residual analysis. The regions developed by Harris follow general topographic outlines and appear to offer good estimates of hydrologically homogeneous areas. In eastern Oregon the limitations of the data base precluded objective regionalization of the data. The data were basically grouped by location into two areas. The regions and stations used in this study are shown on the included map of Oregon.

In the Willamette, Coast, and Rogue-Umpqua regions floods are caused primarily by frontal storms coming from the Pacific Ocean. Peak flows in streams draining from the Cascade range are often caused by runoff from snowmelt in conjunction with direct rainfall. In the higher elevations of the Cascades peak flows are predominately caused by snowmelt in conjunction with rainfall. In eastern Oregon floods can be caused by several factors, including frontal storms in late fall and winter, snowmelt, rain on snow, and extreme convective storms.

Data Base for Study

The data used in this study consisted of maximum yearly discharge from 73 gaging stations in Oregon and seven gaging stations in northern California. The majority of the stations were crest stage gages. The watersheds used in the regression analysis ranged in size from 0.21 to 10.6 square miles. Only 12 of the watersheds used in the regression analysis are larger than five square miles, most of which are located in eastern Oregon where the number of small gaged watersheds is limited. All stations used in the analysis had at least ten years of record. Stations used in this study, length of record, and magnitude and date of maximum recorded discharge are given in Appendix A.

When analyzing long-record stations for the purpose of choosing an appropriate frequency distribution, it became necessary to use some stations that were larger than those used in the regression analysis. This was necessary because there were not five stations with more than 20 years of record in every region. It was assumed that the best fitting frequency distribution for these stations would also be appropriate for all of the stations in the region. These stations are marked with an asterisk in Appendix A. These additional stations were not used in the regression analysis from which the prediction equations were developed.

Frequency Distributions Considered for Use in This Study

Four frequency distributions were tested for appropriateness of use in each region. The frequency distributions were tested on stations having at least 20 years of record with the chi-square goodness of fit test. The frequency distributions considered for use were the two parameter log-normal (LN2), three parameter log-normal (LN3), type I extremal (T1E), log Pearson type III with station skew (LP3), and log Pearson type III with regional skew (LP3/reg). These frequency distributions were chosen because they are the most widely used flood frequency distributions in common usage.

For any distribution, Chow (1964) has shown that the T-year event can be computed from a general equation of the form:

$$Q_t = u + k\sigma \qquad (1)$$

where, u and σ are sample estimates of the population mean and standard deviation, respectively. The factor k is a frequency factor specific to the particular distribution. For any distribution the frequency factor can be computed from the sample size and sample parameters. A brief description of the flood frequency distributions used in this study follows.

Two parameter log-normal

The two parameter log-normal distribution is very attractive for use in flood analysis because of the ease of calculating the parameters of the distribution and its use of the well understood normal distribution. A theoretical justification for use of the log normal distribution is given by Yevjevich (1972).

For the two parameter log-normal distribution the magnitude of the T-year event was calculated as

$$Q_t = 10^{(\bar{x}_n + tS_n)} \qquad (2)$$

Where $\bar{x}_n$ and S_n are the mean and standard deviation of the logarithms of the recorded events, and t is the standard normal deviate corresponding to the desired probability of exceedance P. The mean and standard deviation are estimated by the equations.

$$\bar{x}_n = \sum_{i=1}^{n} \log x_i / n \qquad (3)$$

and

$$S_n = \sqrt{\frac{\sum_{i=1}^{n} (\log x_i - \bar{x}_n)^{2/n}}{n}} \qquad (4)$$

where n is the number of recorded events.

Type I extremal (Gumbel)

This distribution was first proposed for the analysis of flood flows by Gumbel in 1941. Gumbel considered the daily flow as a statistical variable unlimited to the positive end of the distribution, and defined a flood as being the largest value of the 365 daily flows. The flood flows are therefore the largest values of flows. According to the theory of extreme values the annual largest values of number of years of record will approach a definite pattern of distribution when the number of observations each year is large. Thus, the annual maximum floods constitute a series which can be fitted in the theoretical extremal distribution of type I. It has been questioned (Benson, 1962) whether the number of observations in a year is large enough for the asymptotic distribution to be approached. However, practical applications have shown satisfaction with the use of this distribution for flood analysis (Dalrymple, 1960; Reich, 1973). For the type I extremal distribution the value of the T-year event was calculated as

$$Q_t = \bar{x} + \left[\frac{-\ln(-\ln(P)) - u_y}{\sigma_y}\right] S \tag{5}$$

where P is defined as the probability of exceedance, and $\bar{x}$ and S are the mean and standard deviation of the recorded events, respectively. The parameters u_y and σ_y are defined as the mean and standard deviation of the plotting positions, which are functions of the sample size only and have been tabulated (Gumbel, 1958).

Three parameter log-normal

The three parameter log-normal distribution is analogous to the two parameter log-normal distribution except for the inclusion of a lower bound, a. If the population of floods conforms to the three parameter log-normal distribution then there exists a constant, a, such that the series ln (x-a) is normally distributed with mean u and standard deviation σ. The equation used to calculate the T-year event is:

$$Q_t = a + \exp(\bar{x}_{na} + tS_{na}) \tag{6}$$

where a is the lower boundary of the distribution and $\bar{x}_{na}$ and S_{na} are the mean and standard deviation of the logarithms of the distribution (x-a).

In order to fit the three parameter log-normal distribution, it is necessary to estimate the lower bound, a. Both the method of maximum likelihood and method of moments estimates of the lower bound were calculated. Yevchevich (1972) gives the equation for the maximum likelihood estimate of the lower bound as

$$\sum_{i=1}^{n} \frac{1}{x_i - a} \left\{ \frac{1}{n} \sum_{i=1}^{n} \ln^2 (x_i - a) - \left[\frac{1}{n} \sum_{i=1}^{n} \ln (x_i - a) \right]^2 - \frac{1}{n} \sum_{i=1}^{n} \ln (x_i - a) \right\} + \sum_{i=1}^{n} \frac{\ln (x_i - a)}{(x_i - a)} = 0 \qquad (7)$$

This equation can only be solved by an iterative procedure. A computer program developed by Kite (1977) was used for fitting the three parameter log-normal distribution. The mean and standard deviation are then calculated from the equations

$$\bar{x}_{na} = \sum_{i=1}^{n} \frac{\ln (x_i - a)}{n} \qquad (8)$$

and

$$S_{na} = \sqrt{\frac{\sum_{i=1}^{n} \left[\ln (x_i - a) - \bar{x}_{na} \right]^2}{n}} \qquad (9)$$

It is possible that a maximum likelihood solution for the lower bound does not exist. When this occurred the method of moments was used to estimate the parameters of the distribution as:

$$a = u_x (1 - Z_1/Z_2) \qquad (10)$$

where u_x is the mean of the recorded events and Z_1 and Z_2 are the coefficients of variation of the distributions x and x-a, defined as

$$Z_1 = \sigma_x / u_x \qquad (11)$$

$$Z_2 = \frac{\sigma_x}{u_x - a} \tag{12}$$

The value of Z_1 can be computed directly from the observed events. The value of Z_2 is obtained from the relationship

$$\gamma = 3Z_2 + Z_2^{\ 3} \tag{13}$$

where γ is the coefficient of skew of the recorded events, x. The solution of this equation is:

$$Z_2 = \frac{1 - w^{2/3}}{w^{1/3}} \tag{14}$$

where

$$w = \frac{-\gamma + (\gamma^2 + 4)^{1/2}}{2} \tag{15}$$

The procedure is to compute the mean u_x, standard deviation σ_x, and skew coefficient γ, of the observed events, x. Equation 14 is then used to compute Z_2; a is computed from equation 10 and $\bar{x}_{na}$ and S_{na} are computed from the equations

$$S_{na} = \left[\ln(Z_2^2 + 1)\right]^{1/2} \tag{16}$$

and

$$\bar{x}_{na} = \ln\,(\sigma_x/Z_2) - 1/2\,\ln(Z_2^2 + 1) \tag{17}$$

When the lower bound of the three parameter log-normal distribution equals zero the distribution reduces to the two parameter log-normal distribution.

Log Pearson type III

The log Pearson type III distribution as adopted by the U.S. Water Resources Council consists of fitting the Pearson type III distribution to the logarithms of the flow events by the method of moments. The Pearson type III distribution was first presented for use in analysis of flood series by Foster (1924). Although the method of maximum likelihood gives theoretically better estimates of the parameters of the log Pearson type III distribution, numerous computational difficulties in obtaining maximum likelihood estimates of the parameters for this distribution have been found (Matalas and Wallis, 1973). The method of moments was used to estimate the parameters of the log Pearson type III distribution in this study. The equation used to determine the magnitude of the T-year event is:

$$Q_t = 10^{(\bar{x}_n + kS_n)} \quad (18)$$

where k is a frequency factor whose value depends on the skew of the data and the return period of the calculated event. The skew coefficient γ is calculated as

$$\gamma = \frac{n^2\ (\Sigma x^3) - 3n\ (\Sigma x)(\Sigma x^2) + 2\ (\Sigma x)^3}{n(n-1)(n-2)S_n^3} \quad (19)$$

An approximate value of the frequency factor, k, is given by the U.S. Water Resource Council (1977) as

$$k = \frac{2}{\gamma}\left[\left[(t - \frac{\gamma}{6})\ \frac{\gamma}{6} + 1\right]^3 - 1\right] \quad (20)$$

Where t is the standard normal deviate corresponding to the desired probability of exceedance. In practice, k is not calculated but rather taken from tables of k coefficients such as those prepared by the U.S. Water Resources Council (1977).

The U.S. Water Resources Council recommends that for stations with less than 25 years of record a regional estimate of the skew coefficient be used because of the variability associated with this parameter when estimated from small samples.

Special Situations

In flood frequency analysis studies three situations often occur that may require special handling because of their effect on the statistical parameters computed from the data. These situations are outlying data points, missing data, and zero flood years.

Outliers

Ourliers are data points which depart significantly from the trend of the balance of the data. The retention, modification, or deletion of outliers can significantly effect the statistical parameters computed from the data. Beard (1974) tested four techniques for analysis of flood series with outlying data. In this study an outlier was defined as an extreme value whose ration to the second most extreme value was more than the ratio of the second most extreme value to the eighth most extreme value. The techniques tested for handling outliers were

1. keep the value as is.
2. reduce the value to the product of the second largest event and the ratio of the second largest to eighth largest event.
3. reduce the value to the product of the second largest event and the square root of the ratio of the second largest event to eighth largest event.
4. discard the value.

Beard used split record testing on long record stations to test each method and concluded that keeping the value as is was the most logical and justifiable method to use. This method was adopted in this study.

Missing data

Annual peaks for some years may be missing because of reasons not related to flood magnitude, such as removal of gage. When this occurred the data was analyzed as one continuous record with length equal to the sum of both records. This is the method adopted by the U.S. Water Resources Council (1977). An attmpt to correlate stations with missing data with nearby stations with complete records was abandoned because of the poor correlations obtained between flood peaks.

Zero flood years

For several streams in eastern Oregon there were years in which no flow was recorded for the entire water year. This situation precludes analysis with frequency distributions that use logarithms of the flow series because the logarithm of zero is negative infinity. Several solutions to this problem have been proposed including,

1. add 1.0 to all data.
2. add small positive value to all data (i.e., 0.1, 0.01, 0.001, etc.).
3. substitute 1.0 in place of all zero values.
4. substitute small positive value in place of all zero values.
5. ignore all zero values.

All of these techniques affect the parameters of the distribution. Techniques one and two affect the mean, and techniques three, four, and five affect both the mean and variance. Substituting a value of less than one should be avoided because of the large effect this has on the parameters of the distribution when the value is logarithmically transformed (i.e., the $\log_{10}$ of 0.001 is -3). Kilmartin (1972) found that adding one to all values was the best simple solution to this problem since the logarithm of one is zero. This technique was used for analysis of records with some years of zero flows in this study.

Goodness of Fit Test

The chi-square goodness of fit test was used to help determine the most appropriate distribution for use in each region. Cicioni et al. (1973) found this test to be better than either the Kolmogorov-Smirnov, Cramer-Von Mises, or Anderson-Darling tests of goodness of fit. The difficulty of obtaining meaningful results from goodness of fit tests alone was recognized by Matalas and Wallis (1973) who state that current goodness of fit tests do not seem to be powerful enough to distinguish among similarly skewed distributions. Goodness of fit tests do provide a means of assessing the fit of frequency distributions with the recorded data.

The statistic

$$\chi^2 = \sum_{j=1}^{k} \frac{(O_j - E_j)^2}{E_j} \tag{21}$$

is distributed asymptotically as chi-square with k-m-1 degrees of freedom, in which k is the number of class intervals, m is the number of parameters estimated from the sample for the particular distribution, O_j is the observed class frequency, and E_j is the expected class frequency. If classes of equal probability are used then E_j is equal to n/k, where n is the sample size. Equation 21 then reduces to

$$\chi^2 = \frac{k}{n} \sum_{j=1}^{k} O_j^2 - n \tag{22}$$

according to Kendall (1966). The class intervals were computed from the frequency factor equations for each distribution corresponding to each probability of exceedance. Seven class intervals were used to compute χ^2 for each long-record station.

Estimation of Q_t for Gaged Sites

The chosen flood frequency distribution in each region was used to calculate the magnitude of selected peak flows. Flows corresponding to the 10, 25, 50, and 100 year return interval events were calculated. Historical flood information was used in this analysis particularly in regard to the flood event of December, 1964. For most of the gaging stations used in this analysis this event was the largest on record. The return period of this event varied with the location of each particular station but in most cases was considerably longer than the period of record. In the Willamette basin the December, 1964 flood event had a return period of between 80 and 100 years. In the Rogue and Umpqua basins this event had a probable recurrence interval of 50 to 100 years. The 1964 flood had a recurrence interval of about 50 years for Coastal Oregon Streams (Army Corps of Engineers, 1966). Calculated values of Q_{50} and Q_{100} were compared with the magnitude of the December, 1964 flood as a check on reasonableness of these estimates.

Regression Analysis

A step forward linear regression technique was used to derive prediction equations relating peak flows to selected watershed characteristics. In this technique the most significant independent variable (basin characteristic) is added at each step. Variables were added until the F statistic was not significant at the 95 percent probability level. In addition to the F-test, evaluation of the coefficient of determination R^2 was used to help determine the best set of equations.

Log-linear regression was used to derive the prediction equations. Preliminary analysis using linear regression indicated that the variance of the residuals was not constant for the regression line. When log-linear regression was used the variance of the residuals was essentially constant. All prediction equations developed in this study are in the form

$$Q_t = b_o X_1^{b_1} X_2^{b_2} \ldots X_n^{b_n} \qquad (23)$$

where, Q_t = peak flow with return period t (cfs)

b_o = regression constant

$b_1 .. b_n$ = exponents determined from regression

$X_1 .. X_n$ = independent variables

By using the logarithms of all variables and linear regression a prediction equation is produced which has the form

$$\log Q_t = \log b_o + b_1 \log X_1 + b_2 \log X_2 + \ldots + b_n \log X_n \qquad (24)$$

Taking the anti-log of equation 24 produces the final form of the prediction equation. In this study the $\log_{10}$ of all variables was used.

Error analysis

After obtaining the results of the regression analysis it was evident that some method of assessing the accuracy of the regression equations was needed. The error analysis took two forms: calculation of the average percent error to indicate the overall fit of the regression equations and calculation of the standard error of the estimate for determining the confidence intervals of a single predicted value.

The percent error for a flood value on a particular watershed is defined as:

$$e_i = \frac{Q_a - Q_p}{Q_a} \times 100 \qquad (25)$$

where, e_i = percent error for a particular watershed
Q_a = actual flood for a specific recurrence interval
Q_p = predicted flood for a specific recurrence interval

Similarly, E is defined as the average percent error in the region for the recurrence interval under consideration as:

$$E = \frac{1}{n} \sum_{i=1}^{n} |e_1| \qquad (26)$$

The average percent error values are a measure of the accuracy of the regression equations in predicting the estimated flood values for a particular region. They are not a measure of the real accuracy with which the estimated values themselves are determined. The estimated values for each recurrence interval were determined from the log Pearson type III distribution and so are only the best estimate of the actual flood values. In short, the average percent error values represent a measure of how well the regression equations fit the input data which consisted of the estimated recurrence interval floods (dependent variables), and the measured watershed parameters (independent variables).

In order to determine the confidence interval for a particular value of Q_p it is necessary to determine the standard error of the estimate of the regression equation. The equation used to construct a confidence interval for a particular estimate of Q_p is

$$Q_a = Q_p \pm t_{(1-\alpha/2,n-p)} \, S.E. \sqrt{1 + \frac{1}{n} + \frac{(Q_p - \bar{Q}_p)^2}{\sum_{i=1}^{n} (Q_p - \bar{Q}_p)^2}} \qquad (27)$$

where, $t(1-\alpha/2,n-p)$ = the t-statistic for n-p degrees of freedom and the $1-\alpha/2$ confidence level.

p = the number of independent variables in the equation, plus one.

S.E. = the standard error of the estimate for a particular estimate of Q_p.

Q_p = flood peak calculated from the prediction equation.

$\bar{Q}_p$ = average of all Q_p in a region.

n = the number of stations in a region.

The primary requirement for the use of this equation is that the variance of the fitted data be constant along the regression line. This is true when using the $\log_{10}$ values of Q_p and Q_a, however when using the prediction equations in the form of equation 23, the variance is not constant along the regression line but increases with increasing values of Q_p. The standard error for any particular value of Q_p in arithmetic units (cfs) is

$$S.E._{Q_p} = Q_p \; S.E._{\log_{10}} \qquad (28)$$

where, $S.E._{Q_p}$ = the standard error for a particular Q_p value (cfs)

Q_p = flood peak calculated from the prediction equation (cfs)

$S.E._{\log_{10}}$ = standard error of the estimate (constant for a specified recurrence interval) ($\log_{10}$ units)

The confidence limits for any predicted value of Q_p are thus larger for larger values of Q_p, a result which seems intuitively reasonable.

Drainage basin characteristics

Floods in Oregon are caused by excessive rainfall or snowmelt. The initial causes of either rainfall or snowmelt floods are meteorologic. After precipitation reaches the ground in some form and varying magnitude, the conversion to runoff is affected mainly by the physical characteristics of the basin. Some of the basin characteristics, such as drainage area and land slope, are stable over time; others, such as land use, are variable.

The meteorologic and basin characteristics together are the hydrologic variables that affect flood peaks, and both must be considered when trying to relate flood peaks to environmental factors. All factors that could be expected to relate to flood peaks should be examined in a study. In reality the availability of data greatly restricts the number of factors that can be considered. The factors should be simple, expressible quantitatively, and exhibit no interdependence.

Hydrologic factors that are totally independent of each other are generally not available in flood hydrology. Intuitively, the most important factor is the size of the drainage area. In general, the larger the drainage area the greater the volume of water that will fall on a watershed, and the greater the discharge. After the selection of drainage area as an independent variable most other factors will be related to drainage area size and interrelated among themselves. The general magnitude of rainfall falling on a basin is dependent on the location and orientation of the basin and rainfall intensities vary with the size of the drainage basin. Cover, channel slope, and channel dimensions may be affected by the amount of rainfall. Thus, there is some correlation between climatic and topographic factors. Topographic factors tend to be highly interrelated. Length of stream channel, slope, altitude, and drainage densities are usually highly correlated with each other and with drainage area.

The drainage basin characteristics that were used as independent variables in the regression analysis are listed in Appendix B and were computed as follows:

1. Drainage area (A) in square miles. The total contributing area upstream of the gaging station site as published in the latest U.S.G.S. water resources data reports.

2. Mean basin elevation (E) in feet above mean sea level, determined by the U.S.G.S. by laying a grid over a topographical map of practical scale and averaging the elevations recorded at each intersection. The grid spacing is selected to give at least 25 intersections within the basin boundary. Altitude is a factor that is not in itself a direct cause of flood peaks; however, it serves as a good index of other factors that vary with altitude but for which data are not available, or which are not as amenable to quantitative analysis. Factors such as depth and type of soil, solar radiation, evaporation, temperature, and vegetative characteristics vary with altitude.

3. Gage datum (G), in feet above mean sea level. Obviously highly correlated with basin elevation this factor was considered because it is easier to compute than mean basin elevation.

4. Slope (S), in feet per mile, determined from elevations at points 10 and 85 percent along the main stream channel, from the gage site to the basin divide. This index was found by Benson (1962) to be more effective than other related variables, such as land slope and tributary slope, in representing the overall effect of slope in a basin.

5. Length (L), in miles. Defined as the length of the main stream channel from the gaging station to the basin divide. Stream length and drainage basin when considered together provide an estimate of the shape of the watershed (Benson, 1962). For this reason other measures of shape derived from channel length and drainage area such as average width, and Horton's form factor, were not considered as independent variables.

6. Forest cover (F), as percent of total drainage area, determined from the most recent U.S.G.S. quadrangle maps available. Forest cover was measured using a digitizing planimeter. Defined in this manner, forest cover is more an indicator of land type rather than land use. Land shown as cutover on the U.S.G.S. maps was considered as forest. Non-forest areas tended to be either swamps, meadows, rocky areas, or in eastern Oregon, grassland.

7. Latitude (LAT), in degrees and decimal parts.

8. Longitude (LONG), in degrees and decimal parts.

9. Mean annual precipitation (P), in inches. Determined from isohyetal maps prepared by the Oregon State Water Resources Board (1958 A, B; 1959; 1961 A, B; 1962; 1963; 1967; 1971) for various drainage basins in Oregon, and from a map prepared by the U.S. Weather Bureau (1964). Precipitation data from 1930-1957 and correlation with physiographic parameters were used to prepare these maps. Mean annual precipitation is a general measure of the amount of water supplied to the surface of the ground and is the simplest and most comprehensive index of precipitation.

10. Precipitation intensity (I), defined as the maximum 24-hour rainfall having a recurrence interval of two years. It could be expected that some index of rainfall having a short duration and an element of probability would be correlated with flood peaks. Benson (1962) found that either mean annual precipitation or precipitation intensity could be related to peak discharge but that precipitation intensity gave a better correlation. Harris (1979) found that the two-year 24-hour rainfall was an important variable in explaining the peak discharges from streams in western Oregon. This value was determined from a map prepared by the U.S. National Oceanic and Atmospheric Administration (1973). This atlas includes isohyetal maps of rainfall intensities for durations of six and 24 hours, and for return periods ranging from two to 100 years. Benson (1962) found that rainfall intensities of a given duration for different recurrence intervals were highly correlated with each other and that the values for a single recurrence interval could be used in all regression analysis with no loss of accuracy. The two-year 24-hour isohyetal map is the most accurate of the maps prepared by the U.S. National Oceanic and Atmospheric Administration because it was constructed from the largest data base, and required the least amount of correlation with other factors. For these reasons the two-year 24-hour rainfall intensity was the only precipitation intensity factor considered in this study.

11. Temperature index (T), in degrees farenheit. The mean minimum January temperature, determined from a map prepared by Sternes (1960). Benson (1962,b) found that a winter temperature index could be used as an index of the total accumulated water content of snow, for which actual data were not available. Benson found that this was a significant variable in New England, a region dominated by snowmelt augmented floods. Lystrom (1970) found mean minimum January temperature to be a significant factor in Oregon.

A soils index was initially considered for inclusion as an independent variable but was later discarded because of lack of a suitable index. The only soils index available is the Soil Conservation Service hydrologic soil grouping (A, B, C, and D), and storage values (S). The deficiencies of this index, particularly for use in forested terrain have been discussed under the SCS flood prediction technique. The most recent flood frequency analysis in Oregon by Harris (1979) did not find this to be a significant variable. Because of the questionable physical basis of this index it was not used in the regression analysis for this study.

IV. RESULTS

Results of Distribution Selection Procedure

The calculated values of chi-square for the long record stations in each region are shown in Table II. The chi-square test requires that the value of chi-square computed from the data be less than the tabulated value of chi-square at k-m-1 degrees of freedom. With seven class intervals used for all stations, there are four degrees of freedom for the two parameter distributions and three degrees of freedom for the three parameter distributions. At the 95 percent confidence level, the tabulated value of chi-square is 7.815 for three degrees of freedom and 9.488 for four degrees of freedom.

From Table II it can be seen that no one distribution stands out as being markedly better or worse than the others. In the Willamette region the type I extremal distribution fit seven of the nine stations used in this analysis. The three-parameter log-normal distribution fit six stations. The two-parameter log-normal, log Pearson type III, and log Pearson type III with regional skew fit five stations.

In the Coast region the three parameter log-normal distributions and log Pearson type III distribution with regional skew fit five of the six stations, and the two parameter log-normal, type I extremal, and log Pearson type III distributions fit four of the six stations.

In the Cascade region the log Pearson type III and three parameter log-normal distributions fit all five stations analyzed. The log Pearson type III with regional skew fit four of five stations. The two parameter log-normal distribution fit three stations and the type I extremal distribution fit two stations in this region.

In the Rogue-Umpqua region the three parameter log-normal distribution fit all five stations analyzed. The log Pearson type III, log Pearson type III with regional skew, and two parameter log-normal distributions fit four stations. The type I extremal distribution fit only one of the five stations analyzed.

TABLE II. Chi-Square Values for Long Record Stations

LN2 - 2 Parameter log-normal LN3 - 3 Parameter log normal
T1E - Type 1 external LP3 - Log Pearson type III
LP3/Reg - Log Pearson type III with regionalized skew

Station	LN2	T1E	LN3	LP3	LP3/reg
		COAST REGION			
14378900	2.72*	6.64*	5.52*	5.52*	2.72*
14326600	2.72*	1.60*	1.60*	2.16*	1.04*
14303700	10.00	10.56	3.84*	16.72	7.76*
14299500	3.08*	5.23*	4.75*	3.08*	2.00*
14307610	2.67*	1.33*	2.00*	2.67*	2.67*
14327400	20.92	10.56	18.40	19.52	20.92
		WILLAMETTE REGION			
14204100	3.85*	11.60	3.85*	5.41*	5.41*
14148700	3.80*	8.00*	3.80*	5.90*	3.80*
14190600	15.04	7.20*	1.04*	5.52*	18.40
14169700	4.67*	8.67*	8.00	8.00	4.00*
14184900	18.40	2.72*	4.40*	13.36	18.40
14144870	5.68*	4.21*	2.74*	1.26*	2.74*
14178800	8.08*	5.17*	4.58*	6.92*	8.08*
14209900	12.66	16.66	12.00	23.33	20.66
14192800	19.52	7.76*	8.32	10.56	15.32
		CASCADE REGION			
14333500	10.73	18.27	6.42*	6.15*	6.15*
14134000	2.12*	16.11	3.46*	3.46*	2.12*
14339500	15.50	12.36	4.41*	6.00*	14.54
14158250	1.50*	3.25*	5.00*	1.50*	1.50*
14209100	4.21*	2.00*	2.00*	2.74*	4.21*

(continued)

Station	LN2	T1E	LN3	LP3	LP3/reg
ROGUE-UMPQUA REGION					
14314500	1.04*	12.08	1.04*	3.73*	2.38*
14312100	13.36	10.56	3.84*	8.32	13.36
14361300	5.52*	12.24	5.52*	5.52*	2.16*
14372500	4.55*	4.07*	2.62*	4.07*	4.07*
14375500	4.15*	10.62	4.15*	4.15*	6.85*
BLUE-WALLOWA REGION					
14034370	1.70*	6.60*	1.70*	1.70*	1.70*
14010000	3.04*	11.79	1.88*	2.46*	2.46*
13330500	2.34*	2.07*	1.02*	2.34*	2.34*
13320400	8.35*	3.78*	5.30*	10.78	8.35*
13325000	3.30*	14.18	6.15*	6.15*	2.78*

* Significant at 95 percent confidence level.

In the Blue-Wallawa region the log Pearson type III with regional skew, two parameter log-normal, and three parameter log-normal distributions bit all five stations analyzed. The log Pearson type III fit four stations, and the type I extremal distribution fit three stations. No long-term station analysis was done in the Klamath region because there were not five appropriate long-term stations in this region.

The log Pearson type III distribution with regional skew was chosen as the distribution for use in all regions even though it did not fit as many stations in the Willamette and Rogue-Umpqua regions in the long-record station analysis. The main reason for this choice is the inherent flexibility of the log Pearson type III distribution. The ability to adjust the frequency curve by means of the skew coefficient is more important for short-record stations than for long-record stations because of the larger variability associated with short records. From the chi-square test alone it appears that the two parameter log-normal, three parameter log-normal, and log Pearson type III distributions would be equally suitable. With no information other than the chi-square test to base a selection on, there is not enough difference between distributions to justify abandoning the widely accepted log Pearson type III distribution in favor of any of the other distributions examined.

Q_t Values for Gaged Sites

The log Pearson type III distribution was used to calculate the magnitude of the 10, 25, 50, and 100 year return interval flood events for the gaged streams in each region. When fitting the log Pearson type III distribution, the skew coefficient calculated from the data, and a regional skew coefficient taken from a map of regional skew coefficients prepared by the U.S. Water Resources Council, were used in calculating the magnitudes of the individual flood events. If a station record included the flood of December, 1964, the values of the

50- and 100-year flood events were compared with the magnitude and probable recurrence interval of this event as a check on the log Pearson type III estimates. In some cases the frequency curve was adjusted to better correspond to the known return interval of this event. Many stations from western Oregon were included in this study that were also included in the study by Harris et al. (1979). In cases where no additional data had been collected for these stations it was possible to use the log Pearson type III estimates made by Harris. The gaging stations and estimates of the 10, 25, 50, and 100 year flood events are given in Appendix C.

Results of Regression Analysis

Table III shows the results of the regression analysis of flood magnitude against basin characteristics. Also shown in Table III is the coefficient of determination R^2, the average percent error for each regression equation, and the standard error of the estimate of the regression equation in $\log_{10}$ units. All of the independent variables included in the regression equations are significant at the 95 percent level of confidence.

In the Coast region two regression equations are shown for each return interval. Area, mean basin elevation, and latitude of the gaging station site were all significant at the 95 percent confidence level. Area of the watershed and mean basin elevation together account for approximately 80 percent of the variation in flood peaks and the addition of latitude to the prediction equation accounts for an additional 12 percent. Despite the statistical significance of latitude in the model, the prediction equations using area and mean basin elevation are preferred. The equations using area and mean basin elevation alone are much simpler; the regression constant and coefficients of the independent variables in the equations using latitude are awkward. Because of the small sample size it is possible that the significance of latitude indicates sampling variation rather

TABLE III. Prediction Equations for Peak Flows in Oregon

Drainage Basin Characteristics: A = Drainage basin area (mi^2); E = Mean basin elevation (feet); P = Mean annual precipitation (inches); LAT - Latitude of stream gage (decimal degrees); T = Mean min. Jan. temperature (°F)

Equation	R^2	Average error (percent)	Standard error (log_{10} units)
WILLAMETTE REGION			
$Q_{10} = 124\ A^{.79}$	.86	23.3	.129
$Q_{25} = 156\ A^{.80}$	.87	23.9	.127
$Q_{50} = 183\ A^{.80}$	.87	23.9	.127
$Q_{100} = 212\ A^{.80}$	.86	24.1	.129
COAST REGION			
$*Q_{10} = 5.87\ A^{1.04}E^{.49}$	.83	25.7	.140
$Q_{10} = (8.2 x 10^{16}) A^{1.13}E^{.36}LAT^{-9.6}$	.93	14.8	.089
$*Q_{25} = 6.31\ A^{1.01}E^{.51}$	.79	27.3	.155
$Q_{25} = (1.26 x 10^{18}) A^{1.11}E^{.37}LAT^{-10.3}$	.91	18.2	.104
$*Q_{50} = 7.77\ A^{1.01}E^{.50}$	.79	26.1	.155
$Q_{50} = (9.47 x 10^{17}) A^{1.11}E^{.36}LAT^{-10.1}$	.91	18.0	.106
$*Q_{100} = 8.40\ A^{1.00}E^{.50}$	.78	26.0	.161
$Q_{100} = (1.21 x 10^{18}) A^{1.11}E^{.36}LAT^{-10.2}$	.90	19.1	.113
CASCADE REGION			
$Q_{10} = .010\ A^{.44}\ P^{2.15}$	.80	20.4	.143
$Q_{25} = .032\ A^{.44}\ P^{1.97}$	.86	16.1	.113
$Q_{50} = .063\ A^{.45}\ P^{1.87}$	.81	22.0	.132
$Q_{100} = .111\ A^{.46}\ P^{1.78}$	.71	26.9	.178

(continued)

Equation	R^2	Average error (percent)	Standard error ($\log_{10}$ units)
		ROGUE-UMPQUA REGION	
$^*Q_{10} = 125\ A^{.75}$	.39	62.7	.265
$Q_{10} = (7.21 \times 10^{-7}) A^{.77}\ T^{5.50}$	.39	36.0	.189
$^*Q_{25} = 163\ A^{.77}$	.46	52.8	.240
$Q_{25} = (6.76 \times 10^{-6}) A^{.88}\ T^{4.92}$	.74	32.9	.172
$^*Q_{50} = 191\ A^{.80}$	.50	48.6	.228
$Q_{50} = (2.25 \times 10^{-5}) A^{.90}\ T^{4.62}$	.75	30.6	.166
$^*Q_{100} = 221\ A^{.82}$	.53	46.9	.224
$Q_{100} = (5.49 \times 10^{-5}) A^{.92}\ T^{4.41}$	.75	30.0	.168
		BLUE-WALLOWA REGION	
$Q_{10} = 46.7\ A^{.46}$	.34	44.6	.254
$Q_{25} = 67.6\ A^{.47}$	.36	48.2	.265
$Q_{50} = 85.2\ A^{.48}$	.35	52.0	.265
$Q_{100} = 105\ A^{.50}$	.34	55.6	.280
		KLAMATH REGION	
$Q_{10} = 30.8\ A^{.70}$	.42	62.5	.332
$Q_{25} = 41.9\ A^{.79}$	.56	51.7	.282
$Q_{50} = 54.5\ A^{.77}$	.59	47.2	.257
$Q_{100} = 69.6\ A^{.75}$	.61	64.1	.241

*Recommended form of prediction equation.

63

than an actual relationship with peak flows. The regression equations including latitude lead to the conclusion that peak flows decrease in magnitude from south to north in Oregon. This may be true, but more data would be needed to confirm this trend. Finally, the purpose of this research is to develop the best predictive equations, which are not necessarily the same as the best descriptive equations. Without additional data to confirm the trend of decreasing flood peaks from south to north in the state, the inclusion of latitude as a predictive variable in the equations is suspect, and the simpler prediction equation is recommended. The inclusion of mean basin elevation in the prediction equations for the Coast region has very real physical significance. Mean annual precipitation and stream gradient both tend to increase as we progress from low elevations to high elevations in the Coast region. Elevation could also be expected to be related to both the type and depth of soil.

A similar situation occurred in the Rogue-Umpqua region, where the validity of mean minimum January temperature is questionable. Mean minimum January temperature was highly correlated with latitude (simple correlation coefficient, R = .87). In addition, either mean minimum January temperature or latitude could be significantly correlated with peak flows. The mean basin elevations of the stations used in the analysis for the Rogue-Umpqua region range from 750 feet to 3420 feet above sea level. At these elevations it is doubtful whether snowmelt plays a significant role in peak flows. Better areal coverage of the region with gaging stations would be needed to define relationships between latitude and other basin characteristics. Because of the limited areal distribution of gaging stations the equation using area alone is recommended, despite the low R^2 values and high average percent errors.

In the Cascade region, area and mean annual precipitation were significantly correlated with peak flows. Although this model is attractive from both a statistical and physical viewpoint, it should be recognized that only nine stations were used in the development of these equations and that additional data might lead to the adoption of a different model, or the inclusion of additional variables.

In the Willamette region, area of the watershed was the only variable found to be significantly related to peak flows. The high R^2 values indicate the degree to which area is related to peak flows in this region. Mean annual precipitation was barely significant at the 90 percent level, but did not explain enough variation to warrant inclusion in the prediction equations. Additional data with better areal coverage of the region may well indicate the need for inclusion of this variable.

In both areas of eastern Oregon, basin area was the only variable found to be significantly related to peak flows. The low R^2 and high average percent error values reflect both the complex nature of the flood hydrology of this area, and the need for much more data. It is possible that the Blue-Wallowa region should be divided into separate regions but more information would be needed to confirm this.

Confidence Intervals for Predicted Values

Equations for determining the confidence interval for a predicted flood value are given in Table IV. These equations reflect the increase in variance with larger magnitude floods. There are certain situations where the designer may wish to increase or decrease the prediction equation flood because of physical characteristics of a watershed. In this situation the designer may find the confidence intervals for the particular region helpful in determining by what amount the predicted flood should be altered. For instance, if the designer is working in an area of very shallow soils in comparison to other watersheds in that area, he may wish to adjust the predicted flood value upward. On the other hand, if the designer is working in a watershed with a swampy area above the culvert site, he may wish to

TABLE IV. Equations for Confidence Interval of Prediction Values

$Q_{t,U,L}$ = Upper and lower confidence limits for predicted value of Q_t

WILLAMETTE REGION

$$Q_{10} = 124\ A^{.79}$$
$$Q_{10,U,L} = Q_{10} \pm t_{(1-\alpha/2,13)} S.E. \left[1 + \frac{1}{15} + \frac{(Q_{10}-187.7)^2}{240,209}\right]^{1/2}$$
$$Q_{25} = 156\ A^{.80}$$
$$Q_{25,U,L} = Q_{25} \pm t_{(1-\alpha/2,13)} S.E. \left[1 + \frac{1}{15} + \frac{(Q_{25}-238.3)^2}{395,163}\right]^{1/2}$$
$$Q_{50} = 183\ A^{.80}$$
$$Q_{50,U,L} = Q_{50} \pm t_{(1-\alpha/2,13)} S.E. \left[1 + \frac{1}{15} + \frac{(Q_{50}-279.5)^2}{542,444}\right]^{1/2}$$
$$Q_{100} = 212\ A^{.80}$$
$$Q_{100,U,L} = Q_{100} \pm t_{(1-\alpha/2,13)} S.E. \left[1 + \frac{1}{15} + \frac{(Q_{100}-323.6)^2}{723,410}\right]^{1/2}$$

COAST REGION

$$Q_{10} = 5.87\ A^{1.04} E^{.49}$$
$$Q_{10,U,L} = Q_{10} \pm t_{(1-\alpha/2,18)} S.E. \left[1 + \frac{1}{21} + \frac{(Q_{10}-189.2)^2}{252,221}\right]^{1/2}$$
$$Q_{25} = 6.31\ A^{1.01} E^{.51}$$
$$Q_{25,U,L} = Q_{25} \pm t_{(1-\alpha/2,18)} S.E. \left[1 + \frac{1}{21} + \frac{(Q_{25}-230.3)^2}{346,217}\right]^{1/2}$$
$$Q_{50} = 7.77\ A^{1.01} E^{.50}$$
$$Q_{50,U,L} = Q_{50} \pm t_{(1-\alpha/2,18)} S.E. \left[1 + \frac{1}{21} + \frac{(Q_{50}-259.1)^2}{452,190}\right]^{1/2}$$
$$Q_{100} = 8.40\ A^{1.00} E^{.50}$$
$$Q_{100,U,L} = Q_{100} \pm t_{(1-\alpha/2,18)} S.E. \left[1 + \frac{1}{21} + \frac{(Q_{100}-290.2)^2}{564,796}\right]^{1/2}$$

(continued)

CASCADE REGION

$$Q_{10} = 0.010\ A^{.44}\ P^{2.15}$$

$$Q_{10,U,L} = Q_{10} \pm t_{(1-\alpha/2,6)} \text{S.E.} \left[1 + \frac{1}{9} + \frac{(Q_{10}-130.8)^2}{92,081} \right]^{1/2}$$

$$Q_{25} = 0.032\ A^{.44}\ P^{1.97}$$

$$Q_{25,U,L} = Q_{25} \pm t_{(1-\alpha/2,6)} \text{S.E.} \left[1 + \frac{1}{9} + \frac{(Q_{25}-187.3)^2}{170,980} \right]^{1/2}$$

$$Q_{50} = 0.063\ A^{.45}\ P^{1.87}$$

$$Q_{50,U,L} = Q_{50} \pm t_{(1-\alpha/2,6)} \text{S.E.} \left[1 + \frac{1}{9} + \frac{(Q_{50}-239.4)^2}{271,193} \right]^{1/2}$$

$$Q_{100} = 0.111\ A^{.46}\ P^{1.78}$$

$$Q_{100,U,L} = Q_{100} \pm t_{(1-\alpha/2,6)} \text{S.E.} \left[1 + \frac{1}{9} + \frac{(Q_{100}-300.6)^2}{422,626} \right]^{1/2}$$

ROGUE-UMPQUA REGION

$$Q_{10} = 125\ A^{.75}$$

$$Q_{10,U,L} = Q_{10} \pm t_{(1-\alpha/2,15)} \text{S.E.} \left[1 + \frac{1}{17} + \frac{(A_{10}-282.1)^2}{269,015} \right]^{1/2}$$

$$Q_{25} = 163.2\ A^{.77}$$

$$Q_{25,U,L} = Q_{25} \pm t_{(1-\alpha/2,15)} \text{S.E.} \left[1 + \frac{1}{17} + \frac{(Q_{25}-377.5)^2}{510,373} \right]^{1/2}$$

$$Q_{50} = 191\ A^{.80}$$

$$Q_{50,U,L} = Q_{50} \pm t_{(1-\alpha/2.15)} \text{S.E.} \left[1 + \frac{1}{17} + \frac{(Q_{50}-457.2)^2}{798,162} \right]^{1/2}$$

$$Q_{100} = 221.2\ A^{.82}$$

$$Q_{100,U,L} = Q_{100} \pm t_{(1-\alpha/2,15)} \text{S.E.} \left[1 + \frac{1}{17} + \frac{(Q_{100}-545.2)^2}{1,206,296} \right]^{1/2}$$

(continued)

BLUE-WALLOWA REGION

$$Q_{10} = 46.7\ A^{.46}$$

$$Q_{10,U,L} = Q_{10} \pm t_{(1-\alpha/2,17)} S.E. \left[1 + \frac{1}{19} + \frac{(Q_{10}-71.6)^2}{12,063} \right]^{1/2}$$

$$Q_{25} = 67.6\ A^{.47}$$

$$Q_{25,U,L} = Q_{25} \pm t_{(1-\alpha/2,17)} S.E. \left[1 + \frac{1}{19} + \frac{(Q_{25}-105.5)^2}{27,802} \right]^{1/2}$$

$$Q_{50} = 85.2\ A^{.48}$$

$$Q_{50,U,L} = Q_{50} \pm t_{(1-\alpha/2,17)} S.E. \left[1 + \frac{1}{19} + \frac{(Q_{50}-134.7)^2}{47,481} \right]^{1/2}$$

$$Q_{100} = 105\ A^{.50}$$

$$Q_{100,U,L} = Q_{100} \pm t_{(1-\alpha/2,17)} S.E. \left[1 + \frac{1}{19} + \frac{(Q_{100}-167.6)^2}{77,047} \right]^{1/2}$$

KLAMATH REGION

$$Q_{10} = 30.8\ A^{.70}$$

$$Q_{10,U,L} = Q_{10} \pm t_{(1-\alpha/2,7)} S.E. \left[1 + \frac{1}{9} + \frac{(Q_{10}-78.9)^2}{19,320} \right]^{1/2}$$

$$Q_{25} = 41.9\ A^{.79}$$

$$Q_{25,U,L} = Q_{25} \pm t_{(1-\alpha/2,7)} S.E. \left[1 + \frac{1}{9} + \frac{(Q_{25}-123.4)^2}{59,202} \right]^{1/2}$$

$$Q_{50} = 54.5\ A^{.77}$$

$$Q_{50,U,L} = Q_{50} \pm t_{(1-\alpha/2,7)} S.E. \left[1 + \frac{1}{9} + \frac{(Q_{50}-156.2)^2}{90,849} \right]^{1/2}$$

$$Q_{100} = 69.6\ A^{.75}$$

$$Q_{100,U,L} = Q_{100} \pm t_{(1-\alpha/2,7)} S.E. \left[1 + \frac{1}{9} + \frac{(Q_{25}-193.6)^2}{133,221} \right]^{1/2}$$

lower the predicted flood value. The confidence intervals can be used as a guide to how much the predicted flood value should be raised or lowered.

A basic precept of applied regression analysis is that the prediction equations are highly suspect if used beyond the range of data from which they were derived. The range of variables used in the final prediction equations is shown in Table V.

TABLE V. Range of Variables in Final Prediction Equations

Region	Area (mi^2)	Precipitation (inches)	Elevation (feet)
Willamette	0.37 - 5.19		
Coast	0.29 - 2.58		260 - 2820
Cascade	0.21 - 8.00	50 - 88	
Rogue-Umpqua	0.75 - 6.42		
Klamath	0.97 - 10.60		
Blue-Wallowa	0.26 - 6.93		

In addition to these constraints, these prediction equations were derived from predominately forested watersheds and are not meant for use in agricultural areas, or areas with less than approximately 40 percent forest cover.

V. CONCLUSIONS AND RECOMMENDATIONS

From the long-record station analysis we can conclude that the log Pearson type III distribution with regional skew is appropriate for use in predicting peak flows from small watersheds in Oregon. However, the long-record station analysis did not indicate that the log Pearson type III distribution was superior to either the two parameter log-normal distribution or the three parameter log-normal distribution. The use of the log Pearson type III distribution was based on the results of the long-record station analysis, and in part on the inherent flexibility of this distribution when analyzing skewed distributions. Based on this analysis there is not enough evidence to justify the adoption of any of the other distribution considered in this study.

Use of any frequency distribution to predict the 50- and 100-year flood events based on 10 to 20 years of record is suspect. The reliability of these flood estimates is enhanced by the knowledge of the probable return interval of the December 1964 flood event. The predicted values of the 10- and 25-year flood events based on 10 to 20 years of record should be fairly good. It is axiomatic that the results of any analysis will not be more reliable than the data which is analyzed. This fact must be recognized when considering the values of the predicted flood peaks.

The regression equations developed in this study can be used to predict the magnitude of specific flood events for small ungaged watersheds. Drainage basin area is the most important variable in explaining the variation of flood peaks from small drainage basins in Oregon. In addition to area, mean annual precipitation, and mean basin elevation are highly correlated with flood peaks in certain areas of the state.

Based on the results of the regression analysis, the most confidence can be placed in the prediction equations developed for the Willamette, Coast, and Cascade regions. Less confidence can be placed in the equations developed for the Rogue-Umpqua, Blue-Wallowa, and

Klamath regions. When considering use of any of these equations it must be recognized that they were developed from the largest available, but still small, data base. Considering the lack of information on small-area flood peaks in Oregon, these prediction equations provide a better method for design of culvert crossings, based on real world data; rather than rules of thumb, or outdated empirical methods.

The equations developed in this study are intended to be used with other information in helping to design optimally sized culvert crossings. The main advantages of these equations is that they are specifically designed for use on small forested watersheds; the size watersheds that typically require culvert installations during forest road construction. An additional advantage of these equations compared to flood prediction equations now in use is that they allow computation of confidence limits for any particular flood estimate.

Need for Further Research and Monitoring

This study graphically indicates the need for more information on small-area flood peaks. Longer records would enable us to estimate long return interval floods more accurately and better areal coverage of the state with gaging stations would provide more information on regional factors influencing flood peaks. The need for such coverage is greatest in the Cascades, Rogue-Umpqua region, and eastern Oregon. To adequately predict the 50- and 100-year flood peaks will require continuous flood records of at least the scope of the present stream gaging program in Oregon for another 25 to 50 years. It seems likely that if more watersheds were gaged over the Blue-Wallowa region that better prediction equations would result if this region were subdivided, and/or combined with other regions.

One can only conclude that with additional flood data, the prediction equations developed in this study can be continually improved. This would result from a better definition of hydrologically homogeneous flood regions, as well as from a more reliable prediction equation to fit all the hydrologic variables which may affect the flood producing characteristics within a given flood region. At the present time, the equations developed in this study provide design engineers with the current state-of-the-art in flood prediction from small forested watersheds.

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A P P E N D I C E S

APPENDIX A. Maximum Discharge at Gaging Stations
Used in Flood-Frequency Analysis

Station Number	Station Name	Years of Record	Maximum Discharge (ft^3/sec)	Date
	WILLAMETTE REGION			
14312100	Lady Ck. nr. Rhododendron	13	710	12-21-64
14144870	M. Fk. Willamette R. Trib. nr. Oakridge	18	82	12-22-64
14148700	Fern Ck. nr. Lowell	22	52	2-10-61
14161200	Lookout Ck. Trib. #3 nr. Blue	11	52	12-20-57
	River	15	75	12-11-56
14161600	Lookout Ck. Trib. nr. Blue River	15	75	12-11-56
14169700	Bear Ck. nr. Cheshire	21	455	1-21-72
14178600	Short Ck. at Breitenbush Hot Springs	12	206	12-21-64
14178800	Wind Ck. nr. Detroit	24	231	12-21-64
14181700	N. Santiam R. Trib. nr. Gates	17	132	1-30-65
14184900	Sheek Ck. nr. Cascadia	25	116	12-22-65
*14190600	Soap Ck. Trib. nr. Suver	25	86	3-2-72
14192800	S. Yamhill R. Trib. nr. Willamina	25	420	12-21-55
14197300	Panther Ck. nr. Carlton	16	612	12-21-64
14203800	Beaver Ck. nr. Glenwood	17	472	2-2-63
14204100	Bateman Ck. nr. Glenwood	26	145	12-21-55
14209750	Whiskey Ck. nr. Estacada	12	249	12-22-64
*14209900	Dubois Ck. at Estacada	21	508	12-22-64

(continued)

Station Number	Station Name	Years of Record	Maximum Discharge (ft^3/sec)	Date
	COAST REGION			
11530850	M. Fk. Smith R. Trib. nr. Obrian, CA.	12	102	12-22-64
11533000	Lopez Ck. nr. Smith River, CA.	12	570	3-3-72
14299500	Asbury Ck. nr. Cannon Beach	26	314	2-10-61
14301400	Patterson Ck. at Bay City	17	300	1-28-65
14303650	Squaw Ck. nr. Neskowin	13	305	12-4-75
14303700	Alder Brook nr. Rose Lodge	25	218	1-21-72
14306700	Needle Branch Ck. nr. Salado	15	64	1-11-72
14306800	Flynn Ck. nr. Salado	15	139	1-21-72
14306810	Deer Ck. nr. Salado	15	201	1-28-65
14306830	Lyndon Ck. nr. Waldport	11	162	1-28-65
14306850	S. Fk. Weiss Ck. nr. Waldport	14	54	1-28-65
14306880	Mill Ck. nr. Yachats	10	295	1-28-65
14307550	Deadwood Ck. Trib. at Alpha	12	119	12-22-64
14307610	Siuslaw R. Trib. nr. Rainrock	21	62	1-21-72
14307640	Sam Ck. nr. Minerva	13	800	1-21-72
14326600	Gettys Ck. nr. Myrtle Point	25	245	2-10-61
14327100	Geiger Ck. nr. Bandon	16	206	2-10-61
14327240	Milbury Ck. nr. Port Orford	12	286	1-4-66
14327400	Dry Run Ck. nr. Port Orford	24	213	1-18-71
14378550	Hunter Ck. nr. Gold Beach	13	870	1-30-65
14378800	Harris Ck. nr. Brookings	14	269	2-20-68
*14378900	Ransom Ck. nr. Brookings	25	378	3-2-72

(continued)

Station Number	Station Name	Years of Record	Maximum Discharge (ft^3/sec)	Date
	CASCADE REGION			
14134000	Salmon R. nr. Government Camp	52	1300	12-23-64
14145690	Swamp Ck. nr. McRedie Springs	11	120	12-22-64
14147400	Tumble Ck. nr. Westfir	12	98	12-22-64
14158250	Hackelman Ck. nr. Upper Soda	16	102	12-11-56
14158950	Twisty Ck. nr. Balknap Springs	13	202	12-22-64
14208850	E. Fk. Shellrock Ck. nr. Government Camp	10	105	1-21-72
14209100	Kink Ck. nr. Government Camp	20	94	12-21-64
14333490	Elkhorn Ck. nr. Prospect	11	111	12-22-64
*14333500	Red Blanket Ck. near Prospect	52	3190	12-22-64
14335080	**Fireline Ck. nr. Butte Falls**	12	261	3-2-72
*14339500	S. Fk. Little Butte Ck. at Big Elk	22	145	5-25-42
	KLAMATH REGION			
10390400	Bridge Ck. nr. Thompson Reservoir	12	218	12-22-64
11348080	Big Sage Reservoir Trib. nr. Alturas, CA.	11	175	3-26-71
11348560	Turner Ck. Trib. near Canby, CA.	11	42	10-12-63
11489350	Horsethief Ck. nr. Macdoel, CA.	11	635	12-22-64
11491800	Mosquito Ck. Trib. nr. Shevlin	15	42	12-22-64
11494800	Brownsworth Ck. nr. Bly	12	66	12-22-64
11497800	Currier Ck. nr. Paisley	14	168	3-10-67
11501300	Crystal Ck. nr. Chiloquin	15	65	12-22-64
11509400	Klamath R. Trib. nr. Keno	16	18	12-23-64

(continued)

Station Number	Station Name	Years of Record	Maximum Discharge (ft^3/sec)	Date
	ROGUE-UMPQUE REGION			
11517840	Dona Ck. nr. Klamath River, CA.	13	83	12-22-64
11522210	Benjamin Ck. nr. Happy Camp, CA.	13	146	12-22-64
14307685	Mult. Ck. nr. Tiller	13	540	12-22-64
14308950	Beaver Ck. nr. Drew	13	130	1-14-74
14310900	W. Fk. Frozen Ck. nr. Myrtle Creek	14	300	12-26-55
*14312100	Parrot Ck. at Roseburg	26	290	12-21-55
14312300	Monks Ck. nr. Roseburg	17	260	2-10-61
*14314500	Clearwater R. above Trap Ck.	49	1620	12-23-64
14316600	Dog Ck. nr. Idleyld Park	13	810	12-22-64
14317700	White Ck. nr. Peel	13	950	12-22-64
14318600	N. Umpqua R. Trib. near Glide	12	188	12-26-55
14320600	Cabin Ck. Trib. nr. Oakland	19	246	11-23-61
14322700	Bear Ck. nr. Drain	15	674	2-10-61
14339200	Constance Ck. nr. Sams Valley	10	950	12-2-62
14361300	Jones Ck. nr. Grants Pass	26	1350	12-22-56
14362050	Kinney Ck. nr. Mckee Bridge	14	384	1-15-74
14369800	Butcherknife Ck. nr. Wonder	16	432	1-3-66
14370200	Round Prairie Ck. nr. Wilderville	16	373	1-3-66
*14372500	E. Fk. Illinois R. nr. Takilma	39	15200	12-22-64
*14375500	W. Fk. Illinois R. nr. Obrian	22	16100	12-23-64
14377800	Snailback Ck. nr. Selma	18	329	12-21-64

(continued)

Station Number	Station Name	Years of Record	Maximum Discharge (ft^3/sec)	Date
	BLUE-WALLOWA REGION			
13290150	N. Pine Ck. nr. Homestead	14	226	3-30-65
13318100	McIntyre Ck. nr. Starkey	14	34	5-7-79
*13320400	Little Ck. at High Valley nr. Union	23	207	2-24-57
13322300	Dry Ck. nr. Bingham Springs	14	60	1-25-75
*13325000	E. Fk. Wallowa R. nr. Joseph	65	450	7-25-37
13329700	Trout Ck. Trib. nr. Chico	13	21	4-18-76
*13330500	Bear Ck. nr. Wallowa	54	1730	6-15-74
13333050	Bufford Ck. near Flora	13	36	4-28-78
13333100	Doe Ck. nr. Imnaha	15	158	3-12-72
*14010000	S. Fk. Walla Walla R. nr. Milton	48	2530	6-29-65
14019400	Elbow Ck. nr. Bingham Springs	13	105	6-25-75
14034370	Willow Ck. Trib. nr. Heppner	20	26	1-30-65
10438600	Vance Ck. nr. Canyon City	14	39	12-21-64
14038750	Beech Ck. nr. Fox	12	28	1-3-66
14040900	Bruin Ck. nr. Dale	11	57	4-9-76
14041900	Line Ck. nr. Lehman Springs	15	90	1-30-65
14043800	Bridge Ck. nr. Prairie City	16	98	5-15-75
14043850	Cottonwood Ck. nr. Galena	15	98	4-1-78
14043900	Granite Ck. nr. Dale	10	66	4-7-79
14044100	Paul Ck. nr. Long Ck.	11	56	1-23-70
14047350	Rock Ck. nr. Hardman	14	117	1-30-65
14077800	Wolf Ck. Trib. nr. Paulina	14	300	12-22-64
14081800	Ahalt Ck. nr. Mitchell	23	122	12-21-64

*Not used in regression analysis.

APPENDIX B. Drainage Basin Characteristics Used in Multiple Regression Analysis

Station Number	Drainage Area (mi^2)	Mean Basin Elevation (ft)	Gage Elevation (ft)	Slope (ft/mi)	Main Channel Length (mi)	Mean Annual Precipitation (in)	Precipitation Intensity (in)	Forest Cover (percent)	Temperature Index (°F)	Latitude	Longitude
WILLAMETTE REGION											
14131200	3.82	4010	2550	554	5.00	77	3.70	81.0	28	45.32	121.83
14144870	0.50	2150	1587	710	1.30	55	3.00	100.0	29	43.67	122.43
14148700	0.44	1540	930	817	1.20	50	3.50	100.0	31	43.86	122.68
14161200	0.39	2460	1440	1490	1.10	118	4.00	100.0	26	44.22	122.24
14161600	0.37	2380	1400	1490	1.00	90	4.00	100.0	26	44.21	122.26
14169700	5.19	857	500	68	5.30	46	3.50	54.0	32	44.16	123.35
14178600	2.00	3320	2280	707	2.34	80	3.40	100.0	20	44.95	121.98
14178800	1.03	3010	1680	1420	1.50	77	3.50	100.0	21	44.76	122.12
14181700	0.40	1440	1030	529	1.40	69	4.00	100.0	26	44.76	122.39
14184900	1.35	1780	800	714	1.90	68	3.40	100.0	28	44.39	122.51
14192800	1.81	625	280	284	2.40	70	2.50	54.0	29	45.04	123.47
14197300	3.19	1400	560	518	2.60	91	3.80	89.0	28	45.31	123.35
14203800	4.31	940	540	121	3.20	66	3.60	92.3	28	45.67	123.29
14204100	1.27	1140	410	467	2.40	65	3.50	86.6	26	45.63	123.26
14209750	1.06	2570	2030	476	2.34	66	3.00	100.0	26	45.21	122.16

(continued)

85

Station Number	Drainage Area (mi^2)	Mean Basin Elevation (ft)	Gage Elevation (ft)	Slope (ft/mi)	Main Channel Length (mi)	Mean Annual Precipitation (in)	Precipitation Intensity (in)	Forest Cover (percent)	Temperature Index (°F)	Latitude	Longitude
					COAST REGION						
11530850	0.29	2600	1680	1740	1.10	80	6.20	100.0	30	41.92	123.77
11533000	0.92	1500	40	375	4.20	80	4.80	98.0	39	41.96	124.20
14299500	1.97	1010	30	1030	2.97	110	4.00	98.5	36	45.82	123.96
14301400	1.87	530	80	360	2.10	97	3.50	85.6	34	45.53	123.89
14303650	2.11	820	270	281	2.09	100	5.50	39.0	33	45.12	123.90
14303700	1.09	950	240	397	3.00	96	4.80	37.6	33	45.02	123.85
14306700	0.27	650	440	580	1.10	100	5.20	80.0	34	44.51	123.86
14306800	0.78	947	685	174	1.30	98	5.00	100.0	34	44.54	123.85
14306810	1.17	1030	600	458	1.60	95	5.00	95.0	35	44.54	123.88
14306830	0.90	510	250	170	1.50	100	5.50	100.0	35	44.45	123.98
14306850	0.33	840	400	600	0.60	90	3.90	84.8	35	44.39	124.03
14306880	1.65	960	80	346	3.71	88	4.50	100.0	37	44.22	124.11
14307550	0.75	1000	500	308	1.60	100	5.00	100.0	31	44.17	123.70
14307610	0.42	500	40	479	1.30	88	4.50	95.2	33	44.07	123.88
14307640	2.58	980	460	374	2.27	90	4.30	96.3	32	44.15	123.95
14326600	1.45	600	150	194	1.90	59	4.40	66.2	36	43.01	124.21
14327100	1.36	260	120	100	2.80	55	4.50	97.8	40	43.10	124.38
14327240	0.80	1600	720	1390	1.10	112	5.90	100.0	28	47.71	124.25
14327400	0.86	690	40	542	1.40	80	5.50	95.3	39	42.69	124.43
14378550	0.98	2820	2320	327	1.50	112	4.00	83.7	38	42.41	124.25
14378800	1.05	600	423	80	2.27	78	5.00	96.3	40	42.10	124.31

(continued)

Station Number	Drainage Area (mi^2)	Mean Basin Elevation (ft)	Gage Elevation (ft)	Slope (ft/mi)	Main Channel Length (mi)	Mean Annual Precipitation (in)	Precipitation Intensity (in)	Forest Cover (percent)	Temperature Index (°F)	Latitude	Longitude
					CASCADE REGION						
14134000	8.00	4800	3445	590	5.20	88	4.30	93.1	23	45.27	121.72
14145690	1.51	4130	2640	1260	2.00	62	3.40	91.3	22	43.66	122.21
14147400	1.52	2850	1820	670	2.50	50	3.50	76.6	29	43.88	122.38
14158250	0.21	4630	3964	1590	0.90	85	3.50	48.0	23	44.40	122.13
14158950	1.18	2910	2560	302	2.22	72	4.00	100.0	24	44.23	122.04
14208850	2.80	4230	3480	388	2.00	60	4.00	85.2	23	45.14	121.90
14209100	3.75	3400	2200	343	3.80	60	3.50	90.1	23	45.07	121.96
14333490	1.50	4620	3080	775	3.10	56	3.00	100.0	26	42.79	122.40
14335080	4.77	4310	3260	357	3.43	58	3.10	100.0	26	42.57	122.40

(continued)

Station Number	Drainage Area (mi^2)	Mean Basin Elevation (ft)	Gage Elevation (ft)	Slope (ft/mi)	Main Channel Length (mi)	Mean Annual Precipitation (in)	Precipitation Intensity (in)	Forest Cover (percent)	Temperature Index (°F)	Latitude	Longitude
					ROGUE-UMPQUA REGION						
11517840	2.90	3333	1680	520	4.50	35	2.40	100.0	25	41.84	122.92
11522210	1.19	2020	1040	1180	1.60	55	4.50	100.0	28	41.77	123.40
14307685	2.65	2960	1720	571	3.23	55	2.60	98.0	31	43.10	122.75
14308950	1.61	3010	2350	465	1.60	22	3.50	80.0	31	42.81	122.99
14310900	3.16	1550	860	545	3.10	30	2.50	76.6	33	43.09	123.20
14312300	1.25	750	445	236	1.90	30	2.80	30.2	33	43.25	123.40
14316600	3.93	3240	1430	818	2.60	60	3.50	100.0	31	43.30	122.64
14317700	3.92	3230	1600	781	3.62	58	3.50	82.0	33	43.22	122.85
14318600	0.75	1180	650	517	1.50	38	3.00	49.3	34	43.32	123.17
14320600	1.28	810	440	188	2.30	45	3.00	21.9	34	43.44	123.31
14322700	5.13	1280	555	163	3.80	45	3.00	48.2	34	43.63	123.37
14339200	6.42	1570	1320	72	5.10	18	2.50	48.3	30	42.51	122.89
14361300	7.41	1980	1040	334	4.10	29	3.00	94.4	30	42.44	123.29
14362050	2.83	3420	2180	641	2.66	32	3.60	100.0	29	42.09	123.13
14369800	3.07	1760	1220	376	3.10	38	3.00	83.7	31	42.34	123.57
14370200	3.16	1630	972	332	2.70	34	3.00	99.1	31	42.38	123.50
14377800	1.62	3050	1268	1240	2.50	42	4.60	92.0	31	42.29	123.69

(continued)

Station Number	Drainage Area (mi^2)	Mean Basin Elevation (ft)	Gage Elevation (ft)	Slope (ft/mi)	Main Channel Length (mi)	Mean Annual Precipitation (in)	Precipitation Intensity (in)	Forest Cover (percent)	Temperature Index (°F)	Latitude	Longitude
BLUE-WALLOWA REGION											
13290150	2.89	5110	4310	513	1.54	32	1.80	85.0	8	45.09	116.90
13318100	1.80	4480	4190	136	2.47	26	1.80	95.0	16	45.33	118.45
13322300	1.37	4460	3960	400	1.44	40	2.40	78.1	21	45.64	118.12
13329700	0.26	4510	4340	391	0.44	11	1.20	77.3	15	45.60	117.26
13333050	0.47	4440	4350	100	0.40	21	1.20	53.6	18	45.89	117.28
13333100	5.49	4520	3780	197	4.63	13	1.20	67.6	18	45.75	117.02
14019400	0.68	3410	2440	972	1.19	47	2.20	50.6	20	45.71	118.12
14034370	1.11	4310	3600	508	1.54	28	1.00	82.4	22	45.88	119.33
14038600	6.54	5060	4000	439	3.44	30	1.40	97.0	8	44.29	118.98
14038750	1.94	5190	4520	409	2.69	23	1.60	80.6	10	44.57	119.11
14040900	4.63	5220	4270	468	2.30	30	1.40	100.0	11	44.90	118.79
14041900	2.27	4580	4065	225	2.57	30	1.50	96.6	13	45.17	118.71
14043800	6.93	5350	5620	249	3.26	23	1.70	99.0	12	44.54	118.54
14043850	3.89	5130	4420	297	3.24	25	1.60	98.0	8	44.65	118.87
14043900	1.90	4130	3740	161	1.49	19	1.40	71.4	11	44.89	119.01
14044100	3.50	4490	3760	446	2.74	18	1.30	41.0	10	44.72	119.13
14047350	6.25	4100	3620	136	4.06	25	1.40	88.8	22	45.08	119.57
14077800	2.15	5150	4190	260	4.78	20	1.40	75.0	22	44.28	119.82
14081800	2.28	5096	4680	314	1.53	25	1.50	96.5	18	44.43	120.35

(continued)

Station Number	Drainage Area (mi^2)	Mean Basin Elevation (ft)	Gage Elevation (ft)	Slope (ft/mi)	Main Channel Length (mi)	Mean Annual Precipitation (in)	Precipitation Intensity (in)	Forest Cover (percent)	Temperature Index (°F)	Latitude	Longitude
					KLAMATH REGION						
10390400	10.60	6170	5055	288	7.77	20	1.60	98.4	12	43.02	121.20
11348080	2.54	5220	4930	96	2.20	16	1.20	70.0	16	41.58	120.70
11348560	0.97	4950	4880	11	1.30	17	1.20	50.0	16	41.51	121.04
11489350	9.98	5300	4870	56	5.80	16	1.80	90.0	16	41.69	122.05
11491800	2.63	5090	4900	67	2.92	22	1.60	100.0	13	43.09	121.55
11494800	2.20	6610	5480	468	4.32	17	1.60	89.6	17	42.43	120.84
11497800	2.46	6660	6220	179	5.94	24	1.70	95.0	12	42.72	120.88
11501300	5.77	5070	4190	328	4.27	17	1.60	97.0	20	42.56	121.84
11509400	1.02	4500	4240	315	2.04	19	1.50	97.0	20	42.13	121.96

APPENDIX C. Discharges at Selected Flood-Frequencies at Gaging Stations (ft^3/sec)

Station Number	Q_{10}	Q_{25}	Q_{50}	Q_{100}
		WILLAMETTE REGION		
14131200	450	551	627	703
14144870	59	81	100	122
14148700	43	53	62	71
14161200	54	66	75	84
14161600	70	90	107	124
14169700	439	556	649	746
14178600	182	239	288	342
14178800	157	196	226	257
14181700	103	124	141	161
14184900	98	122	141	161
14192800	252	324	382	444
14197300	399	499	579	664
14203800	360	468	561	668
14204100	106	132	152	173
14209750	127	179	223	271

(continued)

Station Number	Q_{10}	Q_{25}	Q_{50}	Q_{100}
		COAST REGION		
11530850	74	91	104	117
11533000	393	546	671	805
14299500	277	306	327	347
14301400	205	259	303	349
14303650	275	300	350	400
14303700	163	204	236	269
14306700	44	51	55	59
14306800	101	120	135	149
14306810	155	182	201	219
14306830	111	141	163	187
14306880	221	265	298	329
14307550	81	94	103	112
14307610	46	58	68	77
14307640	548	660	742	824
14326600	223	262	290	316
14327100	170	206	232	257
14327240	243	272	355	423
14327400	186	234	271	309
14378550	344	408	458	509
14378800	217	250	272	294

(continued)

Station Number	Q_{10}	Q_{25}	Q_{50}	Q_{100}
		CASCADE REGION		
14134000	559	717	846	982
14145690	69	135	209	313
14147400	100	123	209	313
14158250	78	103	123	145
14158950	95	139	180	227
14208850	113	139	159	179
14209100	95	120	140	161
14333490	72	113	152	199
14335080	100	196	324	531
		KLAMATH REGION		
10390400	113	234	285	340
11348080	164	192	214	236
11348560	40	51	61	71
11489350	193	331	472	654
11491800	48	82	115	157
11494800	47	60	72	85
11497800	155	201	235	268
11501300	53	71	86	103
11509400	10	19	31	50

(continued)

Station Number	Q_{10}	Q_{25}	Q_{50}	Q_{100}
		ROGUE-UMPQUA REGION		
11517840	41	63	81	102
11522210	90	124	152	182
14307685	350	470	504	540
14308950	134	192	243	300
14310900	269	304	325	342
14312300	244	305	353	402
14316600	295	430	643	834
14317700	260	428	610	950
14318600	111	144	170	198
14320600	202	320	364	409
14322700	572	673	747	822
14339200	775	965	1111	1260
14361300	731	1014	1257	1526
14362050	200	290	360	450
14369800	431	538	620	702
14370200	427	580	704	837
14377800	261	305	336	366

(continued)

Station Number	Q_{10}	Q_{25}	Q_{50}	Q_{100}
		BLUE-WALLOWA REGION		
13290150	185	220	238	250
13318100	26	33	39	44
13322300	56	66	73	78
13329700	19	31	42	55
13333050	29	40	48	57
13333100	111	164	210	261
14019400	113	147	175	203
14034370	21	34	46	60
14038600	49	73	95	120
14038750	41	67	92	122
14040900	102	186	272	382
14041900	59	75	88	101
14043800	83	113	137	163
14043850	82	98	111	124
14043900	66	127	192	278
14044100	70	132	198	284
14047350	196	341	487	667
14077800	172	261	345	446
14081800	88	115	136	156

APPENDIX D. Class Limits for Long Record Stations
Used in Chi-Square Test

WILLAMETTE REGION

Station 14184900. Sheek Ck. near Cascadia

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	17.7	27.2	28.2	22.3	17.7
2	.286	28.1	38.9	41.0	45.4	28.1
3	.429	40.1	49.3	51.6	62.4	40.1
4	.571	55.9	59.0	62.0	73.7	55.9
5	.714	79.7	73.8	73.9	80.3	79.7
6	.857	126.4	94.6	90.3	84.0	126.4
7	1.0	∞	∞	∞	∞	∞
	χ^2	18.40	2.72	4.40	13.36	18.40

Station 14169700 Bear Ck. near Cheshire

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	120.7	103.6	121.8	118.4	120.5
2	.286	160.1	159.2	163.8	163.0	163.6
3	.429	199.0	208.6	203.8	204.6	192.1
4	.571	243.8	261.0	248.3	250.5	235.0
5	.714	302.1	325.3	305.4	309.0	296.4
6	.857	402.2	423.9	397.0	404.5	410.3
7	1.0	∞	∞	∞	∞	∞
	χ^2	4.67	8.67	8.00	8.00	4.00

(continued)

Willamette Region, continued

Station 14190600 Soap Ck. Trib. near Suver

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	13.2	21.0	21.5	15.5	13.2
2	.286	21.0	29.5	31.8	31.8	21.7
3	.429	30.0	37.0	39.9	44.8	28.2
4	.571	41.6	45.0	47.6	54.8	39.2
5	.714	59.4	54.7	55.9	62.0	57.2
6	.857	94.4	69.8	67.0	66.3	97.3
7	1.0	∞	∞	∞	∞	∞
	χ^2	15.04	7.20	1.04	5.52	18.40

Station 14178800 Wind Ck. near Detroit

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	49.4	41.3	49.4	48.9	49.4
2	.286	63.1	60.3	61.8	62.7	63.1
3	.429	76.2	77.3	74.3	75.6	76.2
4	.571	90.8	95.2	88.7	85.1	90.8
5	.714	109.7	117.2	108.1	104.4	109.7
6	.857	140.0	151.0	140.9	142.2	140.0
7	1.0	∞	∞	∞	∞	∞
	χ^2	8.08	5.17	4.58	6.92	8.08

97

Willamette Region, continued

Station 14144870 Middle Fork Willamette R Trib.

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	11.2	6.6	11.1	10.9	11.0
2	.286	15.7	14.5	15.5	15.7	15.5
3	.429	20.3	21.6	20.1	20.6	20.1
4	.571	25.9	29.1	25.7	26.2	25.7
5	.714	33.6	38.2	33.5	34.0	33.5
6	.857	47.2	52.3	47.5	48.0	48.2
7	1.0	∞	∞	∞	∞	∞
	χ^2	5.68	4.21	5.68	1.26	2.74

Station 14192800 S. Yamhill River Trib.

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	37.4	51.3	62.3	48.6	37.7
2	.286	63.1	83.7	91.2	103.4	64.3
3	.429	91.7	112.2	116.4	144.7	94.6
4	.571	129.9	142.6	142.5	174.0	134.0
5	.714	187.6	173.6	179.9	192.1	193.3
6	.857	306.3	237.1	219.3	201.1	311.1
7	1.0	∞	∞	∞	∞	∞
	χ^2	10.52	7.76	8.32	10.56	15.32

Willamette Region, continued

Station 14204100 Bateman Ck. near Glennwood

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	37.6	34.4	37.6	37.1	37.2
2	.286	46.9	46.0	47.3	47.3	46.6
3	.429	55.7	56.3	56.1	56.5	55.3
4	.571	65.3	67.3	65.7	66.2	64.8
5	.714	77.4	80.7	77.6	78.2	77.1
6	.857	96.6	101.3	96.1	97.3	97.8
7	1.0	∞	∞	∞	∞	∞
	χ^2	3.85	11.60	3.85	5.41	5.41

Station 14148700 Fern Ck. near Lowell

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	7.6	7.2	8.6	7.4	7.5
2	.286	11.5	13.4	14.1	12.7	11.4
3	.429	15.9	19.0	19.0	18.0	15.7
4	.571	21.4	24.8	24.2	23.9	21.1
5	.714	29.4	32.0	30.5	31.4	29.2
6	.857	44.6	43.0	40.0	43.0	45.7
7	1.0	∞	∞	∞	∞	∞
	χ^2	3.80	8.00	3.80	5.90	3.80

Willamette Region, continued

Station 14209900 Cubois Ck. at Estacada

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	18.3	-20.4	27.6	19.8	17.6
2	.286	34.1	27.8	47.6	46.1	34.8
3	.429	54.8	70.7	68.7	82.6	57.0
4	.571	85.3	116.1	94.3	117.8	88.8
5	.714	137.4	171.8	130.1	154.9	141.8
6	.857	255.2	257.2	194.0	193.9	260.4
7	1.0	∞	∞	∞	∞	∞
	χ^2	12.66	16.66	12.00	22.33	20.66

CASCADE REGION

Station 14209100 Kink Ck. near Government Camp

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	28.8	29.1	30.6	28.4	29.6
2	.286	37.1	38.7	42.0	39.7	37.5
3	.429	45.1	47.2	50.7	49.1	45.9
4	.571	54.1	56.3	58.9	58.4	55.0
5	.714	65.8	67.4	67.7	68.8	66.7
6	.857	84.9	84.4	79.2	82.4	85.6
7	1.0	∞	∞	∞	∞	∞
	χ^2	4.21	2.0	1.04	3.73	2.38

Cascade Region, continued

Station 14158250 Hackelman Ck. near Upper Soda

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	- 18.5	12.2	18.5	18.2	18.5
2	.286	25.2	23.3	25.0	25.3	25.2
3	.429	32.0	33.2	31.7	32.3	32.0
4	.571	39.9	43.7	39.6	40.3	39.9
5	.714	50.6	56.5	50.4	51.1	50.8
6	.857	69.0	76.2	69.2	69.9	70.1
7	1.0	∞	∞	∞	∞	∞
	χ^2	1.50	3.25	5.00	1.50	1.50

Station 14339500 S. Fk. Little Butte Ck. at Big Elk Ranger Station

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	44.1	54.2	56.1	45.8	43.9
2	.286	59.4	68.5	73.6	71.1	58.2
3	.429	74.6	81.2	87.0	90.8	72.5
4	.571	92.3	94.8	99.6	107.7	89.7
5	.714	115.9	111.3	113.1	122.8	113.8
6	.857	156.0	136.8	130.7	136.8	159.2
7	1.0	∞	∞	∞	∞	∞
	χ^2	15.50	12.36	4.41	6.00	15.54

Cascade Region, continued

Station 14134000 Salmon R. near Government Camp

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	168.0	111.9	169.0	167.7	168.0
2	.286	220.6	198.2	217.3	225.1	220.6
3	.429	272.1	274.8	266.6	262.8	272.1
4	.571	330.5	356.1	324.5	319.1	330.5
5	.714	407.7	455.8	403.8	399.1	408.7
6	.857	535.5	608.7	540.9	545.9	535.5
7	1.0	∞	∞	∞	∞	∞
	χ^2	2.12	16.11	3.46	3.46	2.12

Station 14333500 Red Blanket Ck. near Prospect

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	276.9	140.6	274.5	275.0	275.0
2	.286	388.9	350.2	370.1	379.7	379.7
3	.429	505.1	536.4	477.3	483.6	483.6
4	.571	643.6	733.9	613.6	622.9	622.9
5	.714	835.9	976.0	815.7	818.6	818.6
6	.857	1174.0	1347.6	1200.6	1201.6	1201.6
7	1.0	∞	∞	∞	∞	∞
	χ^2	10.73	18.27	6.42	6.15	6.15

COAST REGION

Station 14378900 Ransom Ck. near Brookings

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	61.6	44.8	61.6	60.5	61.6
2	.286	83.1	79.1	83.7	83.4	83.1
3	.429	104.6	109.6	105.4	105.6	104.6
4	.571	129.5	141.9	130.3	130.9	129.5
5	.714	163.1	181.5	163.5	164.6	163.1
6	.857	219.9	242.3	219.2	222.8	219.9
7	1.0	∞	∞	∞	∞	∞
	χ^2	2.72	6.64	5.52	5.52	2.74

Station 14326600 Gettys Ck. near Myrtle Point

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	80.4	83.5	84.1	79.6	79.4
2	.286	101.6	105.5	111.7	108.2	103.3
3	.429	121.6	125.9	133.3	131.7	124.6
4	.571	143.6	147.6	153.6	154.4	147.3
5	.714	171.9	174.2	175.9	179.4	175.1
6	.857	217.0	214.9	205.3	218.8	218.6
7	1.0	∞	∞	∞	∞	∞
	χ^2	2.72	1.60	1.60	2.16	1.04

Coast Region, continued

Station 14303700 Alder Brook near Rose Lodge

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	30.0	38.9	43.7	35.7	29.9
2	.286	46.9	60.3	63.4	69.9	48.5
3	.429	66.1	79.3	80.6	98.6	62.4
4	.571	90.8	99.4	98.5	118.8	85.7
5	.714	128.0	124.1	120.0	132.5	123.6
6	.857	199.9	162.0	151.8	140.2	206.3
7	1.0	∞	∞	∞	∞	∞
	χ^2	10.0	10.56	3.84	16.72	7.76

Station 14299500 Asbury Ck. near Cannon Beach

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	161.5	158.3	161.6	160.2	161.4
2	.286	181.9	178.4	183.0	182.7	183.5
3	.429	199.3	196.3	200.8	200.8	196.3
4	.571	217.0	215.2	218.3	218.7	213.7
5	.714	237.8	238.4	238.4	239.3	235.6
6	.857	267.9	274.1	266.6	268.9	270.1
7	1.0	∞	∞	∞	∞	∞
	χ^2	3.08	5.23	4.75	3.08	2.00

104

Coast Region, continued

Station 14307610 Siuslaw R. Trib. near Rainrock

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	13.0	11.4	13.0	12.7	12.7
2	.286	17.0	17.2	17.4	17.3	17.2
3	.429	21.0	22.3	21.5	21.6	21.4
4	.571	25.6	27.7	26.0	26.2	26.0
5	.714	31.5	34.4	31.8	32.2	32.0
6	.856	41.5	44.6	41.0	41.7	41.8
7	1.0	∞	∞	∞	∞	∞
	χ^2	2.67	1.33	2.00	2.67	2.67

Station 14327400 Dry Run Ck. near Port Orford

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	57.6	55.2	56.7	56.7	57.6
2	.286	74.0	75.4	74.0	74.7	74.0
3	.429	89.8	93.3	90.2	91.2	89.8
4	.571	107.4	112.3	108.0	109.1	107.4
5	.714	130.2	135.6	130.7	131.9	130.2
6	.857	167.3	171.3	166.9	168.7	167.3
7	1.0	∞	∞	∞	∞	∞
	χ^2	20.92	10.56	10.40	19.52	20.92

105

BLUE-WALLOWA REGION

Station 14034370 Willow Ck. trib. near Heppner

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	1.9	.2	1.8	1.9	1.1
2	.286	3.1	3.0	3.1	3.0	2.9
3	.429	4.6	5.6	4.6	4.3	4.0
4	.571	6.7	8.4	6.6	6.3	5.7
5	.714	9.9	11.7	9.9	9.5	8.8
6	.857	16.6	16.9	16.8	17.2	17.1
7	1.0	∞	∞	∞	∞	∞
	χ^2	1.70	6.60	1.70	1.70	1.70

Station 14010000 S. Fork Walla Walla R. near Milton

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	536.0	445.2	530.4	535.2	535.2
2	.286	680.8	647.3	633.5	666.5	666.5
3	.429	818.4	826.7	746.4	793.8	793.8
4	.571	970.6	1017.1	887.3	941.1	941.1
5	.714	1166.7	1250.6	1092.3	1142.7	1142.7
6	.857	1482.0	1608.8	1473.6	1507.2	1507.2
7	1.0	∞	∞	∞	∞	∞
	χ^2	3.04	11.79	1.88	2.46	2.46

106

Blue-Wallowa Region, continued

Station 13330500 Bear Ck. near Wallowa

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	643.0	633.1	638.2	643.0	643.0
2	.286	761.2	755.5	754.3	761.2	761.2
3	.429	866.7	864.2	859.4	866.7	866.7
4	.571	977.5	979.6	970.9	977.5	977.5
5	.714	1113.0	1121.1	1108.9	1113.0	1113.0
6	.857	1317.6	1338.1	1319.8	1317.6	1317.6
7	1.0	∞	∞	∞	∞	∞
	χ^2	2.34	2.07	1.02	2.34	2.34

Station 13320400 Little Ck. at High Valley near Union

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	51.7	54.4	56.5	52.5	50.8
2	.286	69.8	76.0	83.1	66.3	70.1
3	.429	88.0	95.2	103.6	80.4	88.8
4	.571	109.2	115.6	122.5	98.7	110.1
5	.714	137.4	140.5	143.0	127.1	138.7
6	.857	185.5	178.8	169.6	188.2	187.9
7	1.0	∞	∞	∞	∞	∞
	χ^2	8.35	3.78	5.30	10.78	8.35

Blue Wallowa Region, continued

Station 13325000 East Fork Wallowa R. near Joseph

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	65.0	50.3	64.9	64.0	64.4
2	.286	81.4	74.5	79.2	79.8	80.9
3	.429	96.8	95.9	93.6	94.1	96.1
4	.571	113.7	118.7	110.3	110.5	113.0
5	.714	135.3	146.6	132.9	132.6	134.9
6	.857	169.4	189.4	171.2	172.2	171.7
7	1.0	∞	∞	∞	∞	∞
	χ^2	3.30	14.18	6.15	6.15	2.78

ROGUE-UMPQUA REGION

Station 14312100 Parrot Ck. at Roseburg

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	65.9	66.8	79.3	68.5	64.5
2	.286	90.3	103.7	110.7	109.3	91.4
3	.429	115.1	126.6	135.1	141.8	117.4
4	.571	144.2	150.9	158.1	169.9	147.2
5	.714	183.8	180.7	183.2	195.4	186.8
6	.857	251.8	226.5	216.3	219.8	254.5
7	1.0	∞	∞	∞	∞	∞
	χ^2	13.36	10.56	3.84	8.32	13.36

Rogue-Umpqua Region, continued

Station 14375500 West Fork Illinois River

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	2426.3	1764.0	2427.6	2426.3	2380.0
2	.286	3201.7	2979.9	3206.3	3201.7	3329.2
3	.429	3963.4	4059.6	3979.1	3963.4	4184.5
4	.571	4829.9	5204.9	4834.9	4829.9	5087.6
5	.714	5078.9	6609.8	5980.1	5994.3	6183.9
6	.857	7889.6	8764.7	7879.0	9004.2	7830.2
7	1.0	∞	∞	∞	∞	∞
	χ^2	4.15	10.62	4.15	4.15	6.85

Station 14314500 Clearwater R. above Trap Ck.

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	230.2	202.31	231.8	233.4	229.9
2	.286	268.7	251.1	264.6	261.9	271.8
3	.429	302.7	294.5	295.7	290.1	296.8
4	.571	338.0	340.5	330.0	322.8	331.4
5	.714	380.8	396.9	374.4	367.6	376.2
6	.857	444.6	483.4	444.8	448.4	449.5
7	1.0	∞	∞	∞	∞	∞
	χ^2	1.04	12.08	1.04	3.73	2.38

Rogue-Umpqua Region, continued

Station 14361300 Jones Ck. near Grants Pass

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	166.4	93.1	165.3	164.7	162.8
2	.286	226.8	201.6	219.8	223.2	236.9
3	.429	287.8	298.0	277.8	282.0	305.8
4	.571	358.9	400.2	348.5	352.0	380.3
5	.714	455.5	525.6	448.9	450.3	472.9
6	.857	620.8	718.0	630.1	633.3	615.6
7	1.0	∞	∞	∞	∞	∞
	χ^2	5.52	12.24	5.52	5.52	2.16

Station 14372500 East Fork Illinois River

Class Interval	P	LN2	T1E	LN3	LP3	LP3/reg.
0	0	0	0	0	0	0
1	.143	1693.8	1343.3	1752.5	1654.7	1655.3
2	.286	2357.1	2364.1	2503.9	2447.2	2469.5
3	.429	3039.7	3270.7	3218.7	3207.7	3242.9
4	.571	3841.4	4232.3	4010.1	4053.4	4093.2
5	.714	4961.6	5411.6	5031.3	5137.9	5164.9
6	.857	6904.5	7221.1	6674.5	6882.4	6842.6
7	1.0	∞	∞	∞	∞	∞
	χ^2	4.55	4.07	2.62	4.07	4.07