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Abstract

Extreme value type I (Gumbel) and type II distributions are fitted to annual flood peaks from 152 recording stations throughout New Zealand. If Q_T is defined as a flood with probability of exceedence in any one year of 1/T (T is return period in years), and $\bar{Q}$ is the mean annual flood, then the plot of the flood frequency $Q_T/\bar{Q}$ vs T on Gumbel probability paper for a particular catchment is shown to depend on the location of that catchment.

By grouping catchments with similar flood frequency attributes, eight regions are suggested and master curves developed for $Q_T/\bar{Q}$ vs T and extrapolated to T = 200 years. These flood frequency regions tend to correspond with climatic regions. Further, after grouping all the eastern and all the western catchments into two larger flood frequency regions, there was enough data in each region to develop master curves that are extrapolated to T = 1000 years. These curves resemble curves drawn for catchments in eastern and western parts of the British Isles.

Multiple regression equations are obtained for estimating $\bar{Q}$ as a function of catchment area and one or two catchment rainfall parameters. Single equations applied in each island have prediction errors which show geographical patterns. Thus nine mean annual flood regions are defined which tend to correspond to lithologic and climatic regions, and a regional equation is suggested for each. These equations and the regional curves are the foundation of a suggested design flood estimation method, which is illustrated with examples.

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Notation

a	Constant of regression equation
A	Catchment area (km^2)
a_1, a_2	Constants of regression
b_1, b_2	Exponents of regression equation
c	Constant in a standard error equation
C	Coefficient (Chap. 1)
C_F	Coefficient of variation of estimate of $Q_T/\bar{Q}$ from a curve for given T
C_P	Coefficient of variation of prediction of $\bar{Q}$
C_R	Coefficient of variation of regression estimate of $\bar{Q}$
C_V	Coefficient of variation of annual maxima flood series
C_{vj}	Coefficient of variation of annual maxima at jth station
CS	Sample coefficient of skew
df	Distribution function
EV	Extreme value
EV1	Extreme value type 1 distribution
EV2	Extreme value type 2 distribution
EV3	Extreme value type 3 distribution
E( )	Expected value
F( )	Distribution function
f( )	Probability density function
GEV	General extreme value distribution
i	Rank of a flood peak
J	The additional period of record, in years, outside the continuous record
K	Frequency factor
k	GEV shape parameter (Chap. 3)
k	Number of stations in a region (Chap. 4)
L	Projected lifetime of a structure (Chap. 1)
LP3	Log-Pearson type 3
M	Total length of station years spanned by the data in a group
m	Constant in a standard error equation (Chap. 3)
m	Number of independent variables in regression equation
n_j	Length of record at jth station (years)
N	Length of record (years)
N_G	Average length of record in a region (years)
N_U	Period of recording necessary to estimate $\bar{Q}_{obs}$ with the same accuracy as $\bar{Q}_{est}$ (years)
P( )	Probability
pdf	Probability density function
P_t	Rainfall rate (mm/hr) for design storm of duration t equal to time of concentration for catchment, and return period T
Q	Flood peak variate
Q_i	An individual annual flood peak
Q_T	The flood peak estimate for a return period T
$\bar{Q}$	The mean annual flood
$\bar{Q}_{est}$	$\bar{Q}$ estimated from regression equation (m^3/s)
Q_{max}	Maximúm annual flood peak
Q_{med}	Median of the annual flood peaks
$\bar{Q}_{obs}$	$\bar{Q}$ estimated from flood record (m^3/s)
r	Risk of one or more floods exceeding Q_T in L years
R	Rainfall factor (Chap. 1), Multiple correlation coefficient (Chaps. 3, 4)
se	Standard error
S	Catchment shape factor (Chap. 1) Sample standard deviation (Chaps. 3,4)
S_R	Standard error of $\log_{10}\bar{Q}_{est}$
t	Student "t" statistics
T	Return period (years)
u	GEV location parameter
var( )	Population variance
x	Variate
X_T	Quantile estimate of variate x
$\bar{x}$	Sample mean of variate x
y	Reduced variate for the EV1 distribution
y_N	Reduced variate for the Normal distribution
α	GEV scale parameter
μ	Population mean
ϱ	Interstation correlation between annual maxima
σ^2	Population Variance

Preface

The material presented in this report is a summary of flood information that was available up to the mid 1970's. We think that the regional frequency curves presented in Chapter 3 will prove reasonably robust and will receive relatively minor changes in future analyses. We anticipate improvements to the equations presented in Chapter 4 for estimating mean annual flood will result from the following recent developments:

— The virtual completion of the NZ Land Resource Inventory Worksheets, and the filing of this information in a computer accessible form. Parameters from this inventory may explain some of the variation in flood percentiles between catchments.

— A complete revision of the rainfall intensity information as given by Tomlinson in "The Frequency of High Intensity Rainfalls in New Zealand, Part I" *Water and Soil Technical Publication No. 19* and by Coulter and Hessell in "The Frequency of High Intensity Rainfalls in New Zealand, Part II" *NZ Meteorological Service Miscellaneous Publication 162.*

— Indications that mean annual flood can be estimated from field measurements of channel morphology as discussed by Mosley in "Prediction of Hydrologic Variables from Channel Morphology, South Island Rivers" *Journal of Hydrology (NZ) 18 (2):* 109-20.

— A large increase in the number of stations which in the 1980's would have recorded sufficient data to use in this type of study.

Arising out of the latter is a reassessment of flood frequencies in the Otago/Southland area where a number of notably large floods occurred in the 1978-80 period. The reassessment was completed just in time to be published as an appendix to this report.

Acknowledgements

This report is one outcome of a continuing programme of data collection and analysis carried out for the National Water and Soil Conservation Organisation. Without the data collected (usually in appallingly bad weather) and processed by many staff of catchment authorities and the MWD, and the foresight of those responsible for setting up hydrometric networks, the study reported here would not have been possible. Grateful thanks are extended to MWD and catchment authority staff who made data available to us. We are indebted to many technical staff who have assisted in various phases of the project, and we thank colleagues who have contributed encouragement, constructive criticism and helpful discussion.

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We have used climatological data collected and published by the NZ Meteorological Service and topographic maps produced by the Department of Lands and Survey.

1 Introduction

1.1 Introduction

Many works and structures associated with natural waterways are subject to flooding. These range from small farm dams and culverts on minor roads, through flood protection works and major bridges, to major dams. In designing these works, engineers have to estimate the magnitude of the flood which is to be withstood during the projected life of the structure. An appropriate estimate of this "design flood" is fundamental to ensuring that economic engineering designs with adequate standards of safety are acheived.

Flood estimation for design purposes can be carried out using either the deterministic concept of a "maximum probable flood" for a particular catchment, or with the statistical concept of a "flood magnitude with a probability of exceedence". The former (Dalrymple 1964) is used where exceedence of the design level could lead to catastrophic failure. The object is to estimate the flood that is unlikely to be exceeded. The method first estimates a maximum rainfall for the catchment and then the corresponding flood peak assuming the catchment to be in a condition which would lead to maximum runoff. Since neither the maximum rainfall nor the maximum runoff for known rainfall can be estimated with certainty, the word probable is used when no specific probability is given. In contrast, statistical methods attach specific probabilities to flood magnitudes.

Recent decades have seen the widespread use of benefit-cost analysis methods for assessing the relative merits of different projects competing for capital resources. For projects where flood magnitude is a design parameter it is necessary to attach specific probabilities to this magnitude. Then the expected cost of flood damage can be balanced against the cost of providing enhanced protection.

1.2 Return Period, Risk and Design Life

In New Zealand the need to ensure the safety of major hydro-electric developments stimulated an early interest in flood estimation methods. Benham (1950) introduced a statistical flood estimation method that has been used ever since. In this method the design flood Q_T is defined as the flood which is exceeded on average once in T years; T is termed the return period, and Q_T is termed the T-year flood which has a probability of exceedence in any one year of 1/T. If the projected life of the structure is L years and assuming independence of annual maxima, the risk r of at least one T-year flood occuring in L years is;

$$r = 1 - (1 - 1/T)^L$$

This expression is evaluated for a range of L and T values in the Table 1.1 (it is presented graphically by Ministry of Works and Development (1979)).

Table 1.1 Risk of exceedence for specified L and T.

L	T = 10	T = 50	T = 100	T = 1000
10	0.651	0.183	0.096	0.010
50	0.995	0.636	0.395	0.049
100	1.000	0.867	0.634	0.095
200	1.000	0.982	0.866	0.181

Thus, for example, the probability of the 100 year flood being exceeded at least once in a ten year period is 0.096, in 50 years 0.395, and in 100 years 0.634. This reasoning applies to one river, and the probability of exceedence during a specified time interval at any one of a number of rivers is much greater. If say 10 independent river basins and a 10 year period are considered, the probability of at least one 100 year flood being exceeded in any one of the 10 basins is 0.634 and the probability of at least one 1000 year event is 0.095. Thus large floods are not remote events for consideration by a few specialists, but real possibilities of concern to the whole community. Risk is a difficult concept to convey to the wider community which is easily lulled into a false attitude of complete safety. The risk of flooding to a community is perhaps best conveyed by comparison with the risk of other hazards that are tacitly accepted. Examples include the risks of earthquake damage, traffic accidents and nuclear power plant accidents. Pilgrim and Cordery (1974) and Burns (1977) provide useful discussion and further references on this subject.

In practice, r and L are often unstated and a fixed value for T is used for a particular class of works. Thus in New Zealand hydro-electric earth and rockfill dams, and dams subject to the risk of progressive failure on overtopping, have been designed to pass the 1000 year flood, while concrete dams not subject to the risk of progressive failure on overtopping are designed for the 500 year flood. Similarly, state highway bridges are designed to pass the 100 year flood, while culverts for state highways are generally designed to pass the 10 year flood without heading-up and the 100 year flood with heading-up to a maximum level of 0.5m below the road surface. For some smaller structures no clear standards exist and in some cases there is a lack of rationality. Situations exist where authorities with statutory responsibility for one part of a catchment adopt higher design standards than a second authority in the same catchment for the same class of work (Heiler 1975).

Selection of a design recurrence interval is a subject deserving careful attention. For the credibility of designers, its interpretation as a socially tolerable risk is information that should be carefully explained.

1.3 Methods for Estimating Design Floods

The engineer must estimate a design flood in a range of design situations. This design flood is a hypothetical flood which results from a design rainfall over a catchment, usually assumed to be in an average state of wetness. It is a quantity with a probabilistic meaning defined in the previous section and must be distinguished from forecasts of actual floods which result from real rainstorms over a catchment and whose magnitudes will depend on the prior states of wetness of the catchment. This distinction is important because quite separate techniques are applicable for each situation (Pilgrim and Cordery 1974).

Where an adequate record exists, for example 20 years of annual maximum flood information, a suitable frequency distribution is fitted to the series of flood maxima, and extrapolated. An account of probability methods is provided in Chapter 3.

Where an adequate record does not exist, a number of methods are available.

(i) Empirical methods which relate a flood characteristic to measureable catchment and climatic parameters.
(ii) The unit hydrograph method.
(iii) Simulation methods, where a specific conceptual representation of the routing of precipitation input through the catchment system is used.
(iv) Regional frequency methods, in which magnitude-frequency curves are developed for a region. This enables estimation of any flood Q_T given the mean

annual flood $\bar{Q}$ (the mean of the annual peaks). $\bar{Q}$ may be estimated from measured catchment and climatic characteristics. This is the method used in this study. It is also known as the index flood method. Another version of the method relates Q_T directly to catchment and climatic characteristics.

It is pertinent to review the development of all these techniques in the New Zealand context.

1.3.1 Empirical Methods

Early flood estimation methods involved fitting an envelope curve to observed extremes for a region to give an empirical estimate of a maximum flood, usually with catchment area as a parameter (Schnackenberg, 1949). Envelope curve methods have been largely replaced by empirical methods involving probability; of these the best known are the Rational Method and Technical Memorandum No. 61 (TM61) (NWASCO 1975).

Although the Rational Method is in wide use, it is not an accurate deterministic description of the way in which a catchment modifies rainfall to yield the peak runoff (French *et al.* 1974). An alternative and useful interpretation of the Rational Method is statistical. Here the method links runoff rates of given frequencies with rainfall rates of the same frequencies as follows:

$$Q_T = C.P_t.\ A/3.6$$

where

Q_T = peak discharge rate of return period T (m^3/s)

P_t = the design rainfall rate (mm/hr) for a storm of return period T and duration t equal to the time of concentration for the catchment

A = the catchment area (km^2)

C = an empirical coefficient which provides the link between peak runoff and peak rainfall. It embodies the net effect of catchment losses, storage effects, etc.

French *et al.* (1974) showed that the coefficient C increases somewhat with the return period T, and gave typical values for central and south-east New South Wales.

Aitken (1975) found this ratio for a catchment to be essentially constant for different return periods. For urban catchments Schaake *et al.* (1967) suggested C could be estimated as a function of the portion of impervious area in the catchment and the slope of the main channel.

The second quantity requiring estimation is the duration of the design rainstorm. The critical duration is closely approximated by the minimum time of rise for a number of hydrographs. These are not available for ungauged catchments and traditional time of concentration formulae are not reliable. Heiler (1974) developed an estimator for this time constant for catchments of peninsular Malaysia. Once a duration is established, storm rainfall can be estimated, and in Heiler's case C was estimated as a function of rainfall intensity; application of the method was restricted to catchments with areas in the range 1 km^2 to 100 km^2. Adaptation of this statistical interpretation of the Rational Method for New Zealand conditions would be a valuable contribution.

Another well-known empirical method in New Zealand is known by its publication number, TM61 (NWASCO 1975) and is an adaptation of various American methods. It is recommended for catchment areas up to 1000 km^2. The method is:

$$Q_T = 0.0139\ CRSA^{3/4}$$

where

C = coefficient dependent on the physiography of the catchment,

R = rainfall factor dependent on the design storm,

S = catchment shape factor,

A = catchment area (km^2).

The factor C is determined as the product of two factors W_{IC} and W_S. W_{IC} is determined from a table which has soil type and surface cover as parameters, and W_S is obtained from a graph having channel length and slope as parameters. R is the ratio of the design rainfall for the catchment to the adjusted standard rainfall at Kelburn, Wellington. As with the Rational Method, the rainfall duration must be determined by an empirical time of concentration formula. The shape factor is a function of the catchment area and length. The development of the method is discussed by Campbell (1959). When first introduced in 1953 this method met an urgent need for a standard procedure for flood estimation for ungauged catchments. Its value was greatly enhanced by the publication of a probability analysis of high intensity rainfalls (Robertson 1963).

1.3.2 Unit hydrograph methods

The unit hydrograph method developed in the 1930's has become a widely used hydrological tool. The unit hydrograph (UH) is the flow record from a saturated catchment when a unit of rainfall falls uniformly for unit time. As part of each storm is required to saturate the soil, the UH represents only the "quickflow".

The quickflow from a rainfall excess of various amounts over a succession of time units is calculated by superposition of the set of unit hydrographs that correspond to the rainfall excess. Thus the catchment is assumed to respond linearly, in that runoff from a particular portion of storm rainfall is unaffected by concurrent runoff from other portions of the storm. These assumptions have been tested in numerous studies and for small and medium sized catchments have been found adequate for most engineering design purposes. With a UH determined from a number of storms and for average ratios of excess to total rainfall, design floods for a catchment can be estimated from design storms of the same probability. An important advantage of this versatile method over those described previously is that the shape of the flood hydrograph is calculated, and not merely the peak rate of flow; this is of importance in routing studies, in drainage design and in other situations where it is necessary to know the length of time the water level is above a particular stage. Possibly the main limitation of the method is the subjectivity in determining the volume and distribution of the rainfall excess. Where the lack of flow records prevent the derivation of the UH, procedures have been developed for synthesising typical UH curves by relating characteristics of the hydrograph shape to catchment characteristics. These procedures include the well-known Snyder method and the US Soil Conservation Service dimensionless hydrograph (Linsley *et al.* 1975). These methods have been used successfully within two regions of similar hydrological characteristics (Hoffmeister 1976). Also the Snyder method gave satisfactory results for ungauged tributaries for the Waikato and Clutha Rivers (Jowett and Thompson 1977), although Coulter (1961) queries the wide applicability of the methods.

A guide to the order of loss rates that should be used is given by Pilgrim (1966), who summarised published loss rate information in New Zealand. As most loss rates were low (50% of loss rates were less than 2.5 mm/hr and 80% were less than 5.1 mm/hr), it was concluded that the inaccuracies in transferring these values from one region to another should cause only very small errors in design floods. Nevertheless, more work is needed on loss rate estimation in New Zealand as loss rates are important in situations where flow forecasts are required. For tributaries in the Motueka catchment Beable (1976) found loss rates to be related to antecedent wetness, storm intensity and the portion of catchment in exotic forestry.

1.3.3 Simulation methods

Under this heading is grouped a variety of methods for representing catchment response to precipitation. Catchments are simulated with "models" that are simplified representations of complex real-world systems. Models can be (a) physical, (b) analogue, or (c) mathematical. Mathematical models represent the behaviour of a catchment by a set of equations and logical statements expressing relationships between hydrological variables and model parameters, with an input of precipitation and other climatic measurements and an output of stream discharge. Such models can be classified as: "lumped" or "distributed" depending upon whether variations in processes over the catchment are considered; "time variant" or "time-invariant" depending upon whether variations in time of the model are considered; "stochastic" or "deterministic" depending on whether probabilistic notions are included; and "conceptual" or "empirical" depending on the structuring of the model.

Extensive reviews of these models are given by Clarke (1973) and Chapman and Dunin (1975). One use of such models is the extension of a record of streamflows given a record of precipitation. At present the model parameters are usually estimated by fitting a predicted output hydrograph to an observed output hydrograph over a period of concurrent rainfall and flow records. Future developments are aimed at enabling the estimation of model parameters from observed physical characteristics of the catchment without the need for a period of observed flow record for model calibration.

1.3.4 Regional flood frequency methods

Regional flood frequency methods have been applied widely, for example in North America (Thomas and Benson 1970), in the British Isles (NERC 1975), and in Malaysia (Heiler and Chew 1974). The index flood approach used in this study averages the chance sampling variation in flood frequency in a region, while preserving the variation due to differences in catchment characteristics. Development of the method involves:

(i) collecting annual maxima for a number of flow stations in the area thought to be homogenous;

(ii) drawing frequency-magnitude curves for each station ($Q_T/\bar{Q}$ vs T);

(iii) drawing a frequency-magnitude curve giving a general $Q_T/\bar{Q}$ vs T relationship for use in the region;

(iv) obtaining a regression relationship to estimate the mean annual (or index) flood $\bar{Q}$ from measurable catchment and climatic parameters.

The regression relationship can then be applied at ungauged locations to estimate $\bar{Q}$. Knowing $\bar{Q}$, the regional frequency curves **(iii)** can be applied to determine Q_T for a specified return period T. This method is one way of extending a data base from a number of sites to cover a region. Design flood estimates have to be made for many more sites than can ever be gauged. This is justification for developing the method in New Zealand, where it should be noted that no quantitative information is available on the accuracy of currently used empirical methods. It has not previously been applied on a New Zealand-wide basis. It has the advantage that it is based directly on flood records whereas empirical and unit hydrograph methods rely on the transformation of rainfall into runoff.

With the quantity of new data available by the middle of the 1970's it was considered feasible to develop the regional flood frequency method. Regional inferences about flood frequencies were made to provide a design flood estimation method for catchments having little or no recorded data. The frequency distribution which is most generally applicable to the annual maxima series has been determined. The first step was to assemble and check records of annual maximum flows as outlined in Chapter 2.

Chapter 3 examines the fit of different frequency distibutions to these data and determines which is most generally applicable. The cumulative frequency distribution for each station, the ratio $Q_T/\bar{Q}$ versus T, is plotted. Where individual curves from a region are similar they are averaged to form a regional curve which is postulated to apply for catchments in the region. Derivation of these curves and delineation of different regions is described. Supplementary results provide for the estimation of standard errors of estimate of $Q_T/\bar{Q}$ and Q_T.

Chapter 4 describes the derivation of a procedure for estimating $\bar{Q}$ for cases where inadequate "at site" information is available, and uses a relationship between $\bar{Q}$ and measurable catchment and climatic characteristics. Different relationships are found to hold for different regions of the country and in general these regions differ somewhat from the flood frequency regions of Chapter 3. Examples of application of the method are given in Chapter 5, and Chapter 6 is a summary.