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SOME THOUGHTS ON ESTIMATING SPILLWAY DESIGN FLOOD

by

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ABSTRACT. There is no universally acceptable technique available to estimate the magnitude and frequency of floods. The "rational method", empirical formulae and regional analysis all have restrictions.

The remaining approaches are deterministic, using design precipitation maxima and probabilistic, the latter using streamflow data to calculate frequencies. The concepts of design maxima and the fitting of statistical distributions to flood frequencies are discussed in detail. It is concluded that a more logical cost/benefit approach is required to decision making in spillway design.

RÉSUMÉ. On ne dispose d'aucune technique qui soit acceptable à tous pour évaluer la grandeur et la fréquence des crues; La methode «rationnelle,» les formules empiriques et l'analyse régionale ont tous un usage borné.

Il nous reste les autres moyens d'aborder le sujet; le premier moyen est déterministe utilisant les maxima de précipitation de projet, le deuxième moyen est probabiliste utilisant les données d'écoulement pour calculer les fréquences des crues; les concepts de maxima de projet et l'ajustement des distributions statistiques aux fréquences des crues sont discutés en detail. On conclut qu'en faisant des décisions en ce qui concerne le projet d'évacuateurs de crue, il faut aborder le sujet plus logiquement en tenant compte des frais par rapport aux avantages;

1. INTRODUCTION

Lord Kelvin once said: "When you can measure what you are speaking about and express it in numbers, you know something about it, but when you cannot express it in numbers, your knowledge is of a meagre and unsatisfactory kind." However, since streamflow data for most rivers are available only for short periods, if they exist at all, Lord Kelvin's statement is especially valid for estimation of design flood for structures built across waterways or near them. Undoubtedly, one of the major problems facing hydrologists today is the lack of a satisfactory technique to obtain a realistic relation between magnitude and frequency of rare floods. Considerable progress has been made in the overall field of flood estimation during the past hundred years or so, but much remains to be done. Biswas (1969a and 1970) Benson (1962) and Chow (1962) have presented excellent discussions of historical developments in this field.

Currently, there is no universally acceptable technique available to estimate the magnitude and frequency of floods. Tables 1 and 2 show some of the suggested approaches for flood estimation for different sizes of catchment areas and for various types of dams.

Empirical formulae, the "rational method", or regional analyses can be used for flood estimation from small drainage areas. However, it is difficult to recommend the use of the rational method for any condition, except perhaps for urban areas within the order of 200 acres or so. Empirical formulae are widely employed, but it is difficult to justify their use, especially as more and more data on streamflow are being collected and are readily available. Regional analysis probably is the best solution for estimating floods from small catchment areas.

There is no sharp distinction between the various methods available for estimating design floods for intermediate and major dams, but broadly they can be divided into deterministic and probabilistic approaches.

TABLE I

Techniques of flood estimation for different sizes of catchment areas

Catchment Areas (square miles)	Methods Suggested
<1	Overland flow hydrograph, the "Rational method"
1–100	The "Rational method"; Unit hydrograph; Flood peaks versus drainage area
100–2,000	Unit Hydrograph; Flood frequencies; Flood Peaks versus drainage area
>2,000	Flood routing; Flood frequencies; Flood peaks versus drainage area

2. The Deterministic Approach

The United States Corps of Engineers, depending on the failure damage potential of a dam, estimates either the probable maximum precipitation (PMP) or the standard project storm (SPS). The PMP is the physical upper limit of precipitation for the area under consideration, and the SPS is defined as the "most severe storm considered reasonably characteristic of specific regions," which when expressed quantitatively means that about 10 per cent of the recorded storms exceed the SPS. (Office of the Chief of Engineers 1952). The PMP or SPS is then used to determine the design flood.

When the concept of the PMP was being developed by the Hydrometeorological Section of the U.S. Weather Bureau, Bernard (1943), one of the originators of the concept, was looking for the "maximum possible or ultimate flood." Later, he admitted that "four years of intensive study have disclosed no method for giving the maximum possible storm rainfall which will apply consistently to all basins." Another eminent hydrologist of that time, Horton (1936), supported the upper limit concept very graphically:

> "flood magnitudes always continue to increase as the recurrence interval increases, but they increase toward a definite limit and not toward infinity . . . A small stream cannot produce a major Mississippi River flood, for much the same reason that an ordinary barnyard fowl cannot lay an egg a yard in diameter; it would transcend nature's capabilities under the circumstances."

Such a comparison, however, is not valid. It is granted that a fowl cannot lay an egg bigger than herself, but is her limiting size known? Horton puts a limiting value to the hen by stating that she is an "ordinary barnyard fowl" whereas there is no way to establish a limit to the "meteorological hen", ordinary or extraordinary, that is responsible for precipitation. The fallacy of the upper limit concept was officially recognized by the U.S. Weather Bureau (Co-operative Studies Section 1960) in the 1950's:

> "At one time the concept of probable maximum precipitation (PMP) was expressed in terms of words 'maximum possible'. However, in considering the limitations of data and understanding implicit in an estimate of 'maximum possible' precipitation, it seemed that there was sufficient uncertainty to substitute for the expression 'maximum possible' the more realistic one 'probable maximum.' This was done with no intention or implication of making the values any different. 'Probable maximum' simply seems to be more descriptive and more realistic."

It should be realised that with our present state of knowledge, it is impossible to determine the upper limit of any natural event. For example, is it possible to have 100 inches of rainfall in an hour? Feller (1957) answers a similar question, concerning a natural event, i.e., the possibility of a man living 1000 years. The same argument can very well be used to answer the earlier question on rainfall.

> "It is impossible to measure the life span of an atom or a person without some error, but for theoretical purposes it is expedient to imagine that these quantities are exact numbers. The question then arises as to which numbers can actually represent the life span of a person. Is there a maximal age beyond which life is impossible, or is any age conceivable? We hesitate to admit that man can grow 1000 years old, and yet current actuarial practice admits no bounds to the possible duration of life. According to formulas on which mortality tables are based, the proportion of men surviving 1000 years is of the order of magnitude of one in $10^{10^{36}}$ – a number with 10^{27} billions of zero. This statement does not make sense from a biological or sociological point of view, but considered exclusively from a statistical standpoint, it certainly does not contradict any experience. There are fewer than 10^{10} people born in a century. To test the contention statistically, more than $10^{10^{35}}$ centuries would be required, which is considerably more than $10^{10^{34}}$ lifetimes of the earth. Obviously, such extreme small probabilities are compatible with our notion of impossibility. Their use may appear utterly absurd but it does no harm and is convenient in simplifying many formulas. Moreover, if we were seriously to discard the possibility of living 1000 years, we should have to accept the existence of maximum age, and the assumption that it should be possible to live x years and impossible to live x years and two seconds is as unappealing as the idea of unlimited life".

Once the fallacy of the upper limit concept is recognized and abrogated, it becomes axiomatic that any design value of precipitation, and hence the flood, will have some probability of occurrence, however small, and will entail some degree of risk, however slight. It is admitted that by using a high value of PMP, the probability, and consequently the risk, can be reduced to a minimum, but cannot be eliminated entirely.

There has been growing concern among hydrologists about the ambiguity of the relationship between the hydrometeorological estimation of the spillway design flood and the risk criteria — notably by Alexander (1957, 1959, 1963, 1965a), Benson (1964), Biswas (1966a, 1966b, 1969b), Gruner (1963, 1967), Hershfield, (1962a, 1962b, 1965), and Yevdjevich (1963, 1968). The traditional approach of estimating PMP is a combination of theoretical and empirical methods. The procedure is to devise a physical model whose parameters are then maximized to produce an estimate of PMP (Knox 1960, Wiesner 1964, Bruce 1961). One of the major steps involved in the maximization technique is the estimation of dew points, the method used being "somewhat subjective" (Hounam 1960). But, as the U.S. Weather Bureau (1961) has pointed out, the dew point is a random variable and does not appear to have a definite upper limit. The procedure is further handicapped by the fact that precipitation increases by about 7 per cent due to a change of one degree in dew point (Alexander 1965a and b), and, hence, the dew point would have to be evaluated within a degree or so to be of any practical value.

It should also be realized that the probable maximum flood (PMF) obtained by using the concept of PMP is after all an estimate, and, like any other estimate, is subjected to inaccuracies and limitations of data and imperfection of methodologies. Recently, Ackermann (1964) has questioned the degree of acceptance accorded to such estimates. Different studies, using the same approach, and for the same area, are known to have given very different results. There is, as discussed earlier, a possibility that the estimate of PMF may be exceeded. But, human nature being what it is, all the factors are estimated from a very conservative viewpoint, and, as such, the final result is often a very high flood. This leads to another problem, namely the economy of the project.

The initial PMF approach was to seek an "ultimate flood" which obviously has no probability associated with it, except a limiting exceedence probability of zero. The flood obtained by such analyses is sometimes so high that if it is incorporated in a spillway design, the structure may be rendered uneconomic. So the tendency is to adjust the value downwards as far as "experience and judgement" will allow. Thus, the sum total of the analyses often reduces the procedure to obtaining a vague high design flood value without having any idea about its frequency of occurrence. An actual example will make the problem more appreciated.

TABLE II

Design flood according to types of dams

Category	Impoundment danger potential		Failure damage potential[a]		Spillway design flood
	Storage (acre-ft)	Height (ft)	Loss of Life	Damage	
Major	$>50{,}000$	>60	Considerable	Excessive or as matter of policy	Probable maximum, most severe flood considered reasonable possibly on the basin
Intermediate	1,000 to 50,000	40 to 100	Possible but small	Within financial capability of owner	Standard project; based on most severe storm or meteorological conditions considered reasonably characteristic of the specific region
Minor	$<1{,}000$	<50	None	Of same magnitude as cost of dam	Frequency basis; 50–100 year recurrence interval

[a]Based on consideration of height of dam above tailwater, storage volume, and length of damage reach, present and future potential population and economic development of the flood plain.

In 1935, during the design of the Rincon de Bonete Hydroelectric Scheme on the Rio Negro, in Uruguay, streamflow records were available for a period of 27 years, and the highest recorded flow was 135,000 cusecs. The design flood estimated for the project was 325,000 cusecs, 62 percent of which was earmarked for the spillway and the rest was to be temporarily stored in the reservoir that had a surface area of approximately 282,000 acres. In April, 1959, 14 years after the completion of the project, the Rio Negro drainage area experienced intense rainfall on three occasions within a period of 28 days. The precipitation was caused by the meeting of a cold Antarctic air mass with warm moisture-laden air from the sub-tropical zone of the Atlantic seaboard (Gruner 1963). The temperature difference between the two air masses was estimated to be 23 °F (12.8 °C), and the resultant sudden cooling produced intense precipitation. Within a period of 10 days, nearly 14.2 million acre-feet of water was estimated to have precipitated over an area of 9.4 million acres. As the ground was nearly saturated by the previous heavy rainfalls, 80 per cent of the precipitation ran off, and the subsequent peak flow was about 605,000 cusecs. Nearly 340,000 cusecs were discharged over and around the dam and the rest was temporarily stored in the reservoir which rose 15 ft above the designed maximum level. Fortunately, the dam did not fail. However, had it done so, some 11 million acre-feet of water would have been suddenly released with catastrophic consequences. Theoretically, the design flood had an estimated recurrence interval of 10^3 years, and on the same basis the 1959 flood had a 500,000 year frequency. The Meteorologists estimate that similar conditions could give rise to a temperature differential of as much as 35°F (19.3°C), which if centered over the Rio Negro catchment, would produce the astronomical figure of 21 million acre-feet of precipitation. Obviously, if the spillway is designed for this condition, the scheme would become uneconomic. So the problem is, does the designer provide for a major flood which may make the scheme uneconomic (and may never occur) or does he accept a calculated risk by taking a lower value?

Hershfield (1962a, 1962b, 1965), in an attempt to attach a yardstick to the PMP concept, analysed 24-hour rainfalls from about 2,600 stations in the United States with a total of 95,000 station-years of data. Defining the PMP as the "largest rainfall (precipitation) that a station is ever likely to experience for a particular duration," (Hershfield 1962a) he used two parameters, mean ($\bar{x}$) and standard deviation (σ), to obtain an empirical estimate of PMP. The study indicated that all the largest observed rainfall values were within a frequency factor or reduced variate (K) of 15 for "short" records and 12 for "long" ones. (The relationship is $x_t = \bar{x} + K\sigma$).

However, it may be possible to associate a probability value with the concept of storm transposition without resorting to extrapolation. The probability of occurrence (P_T) of a storm, having a rainfall depth more than d inches for a specified duration t, and occurring over a particular catchment A_c, is a function of transposition probability P_t (in space) and probability of occurrence P_r (in time) which may be considered independent of each other (Alexander 1963, 1965b). The transposition probability P_t, which may be defined as the probability of a design storm being centred within a particular catchment area A_c within a homogeneous region A_h, is given by:

$$P_t = \frac{A_c}{A_h} .$$

The probability of occurrence (P_r) of a storm above rank (r) in any year can be obtained from the "effective" period of records N_e years:

$$P_r = \frac{r}{N_e} .$$

Hence,

$$P_T = \frac{A_c}{A_h} \times \frac{r}{N_e} .$$

For example, if $P_r = 1/50$ and $P_t = 1/20$, then $P_T = 1/50 \times 1/20 = 1/1000$, which in effect gives a return period of 1000 years from a record of only 50 years, without having recourse to extrapolation.

3. Frequency Approach

The probabilistic approach to the flood estimation was taken up quite enthusiastically during the 1920's and 1930's, but later it gradually lost ground. This was on two counts: lack of long records of streamflow and the failure by hydrologists to recognise the extent of the sampling error. Consequently, hydrologists were unable to predict the floods of higher recurrence intervals. The situation, at present, is somewhat better; more years of records are available and the sampling error is better appreciated. However, the problem still remains complex. It is because of the large confidence intervals of the estimated flood variable for the standard levels of significance (5% or 10% level) at the higher recorded values, especially when errors due to both flow measurement and sampling are considered. One approach to design could be the use of the upper limit of the confidence interval at the 5% or 10% level of significance, but even then it will not be a very satisfactory technique.

The major defect of the probabilistic approach is that long periods of streamflow data are not available to test a distribution one way or the other. Extensive studies (Markovic 1965, Panchang and Aggarwal 1966, Biswas and Sangal 1970), have indicated that log-normal, double-exponential a ndgamma distributions can all be applied to flood frequency analyses. The late Emil J. Gumbel (1966) categorically advocated the use of extreme value distribution for flood frequency analyses: "It seems that the rivers know the theory (extreme value). It remains to convince the engineers ... of the validity of this analysis." However, it is not possible to ascertain that one function is more suitable than another in fitting an observed individual station sample. There may be several theoretical justifications for choosing a particular distribution which could describe the flood phenomena, but invariably they make a number of assumptions which are not satisfied in real life. The two major problems are that most of the distributions used for analyses are asymptotic in nature (which implies that the number of observations has to be very large to define the distribution uniquely), and that the independent variables considered are generally somewhat interdependent.

If the estimates of higher floods are obtained solely by the method of extrapolation of frequency curves, the result is to be viewed with extreme caution, but nevertheless it has been suggested that such a procedure is an analytical tool available to the designer (Joint Committee on Floods 1953). On the other hand, it should be realised that if it is possible to choose the correct distribution function, and if there is no change in the flood population outside the observed example, the extrapolation to a linear fit should also be linear. According to Ezekiel and Fox (1953):

> "When there is a good logical basis for the selection of a particular equation, the equation and the corresponding curve can provide a definite logical measurement of the nature of the relationship. When no such logical basis can be developed, a curve fitted by a definite equation yields only an empirical statement of the relationship and may fail to show the true relation. In such cases a curve fitted freehand by graphic methods, and conforming to logical limitations on its shape, may be even more valuable as a description of the facts of the relationship than a definite equation and corresponding curve selected empirically, but fitting less well.
>
> In any event, estimates of the probable value of the dependent variable cannot be made with any degree of accuracy for values of the independent variable beyond the limits of the cases observed; and can be made most accurately only within the range where a considerable number of observations is available. It may be possible to extrapolate the curve if its equation is based on a logical analysis of the relation as well as on the cases observed; but in that case the logical analysis, and the statistical examination, must bear the responsibility for the validity of the procedure".

The sampling error is another important factor which has to be considered in any flood frequency analysis. Since the numerical values of parameters of a chosen distribution are estimated from a limited sample, there is a strong possibility that substantial errors could result due to non-representativeness of the period of record. Fortunately, hydrologists are now aware of such possible errors. Benson (1952), in a classic study, demonstrated the variance of sample estimates as a function of sample size by assuming a theoretical 1000 years of record of annual flood peaks which when plotted on extreme-value paper defined exactly a straight line graph. Later Nash and Amorocho (1966) showed that the standard errors of sample computations of flood magnitudes for very high return periods converge toward a fixed proportion of the estimates. However, the error considered was for sampling variance only. Any error of the sample not conforming to the assumed distributions, normal and double exponential, was not treated.

4. Dam Failures

As far as the author is aware, no concerted attempts have been made to prepare a comprehensive report on dam disasters and their causes. Often it is impossible to obtain details of any disaster because, not surprisingly, the authorities concerned do not like to advertise failures. In the majority of such cases, whatever information that may be available is usually vague. However, it is possible to list some 400 important failures to date based on reliable information. (Biswas and Chatterjee 1971). The figure includes repeated failures of certain dams as well as instances of a chain of failures. Estimated damages from some major dam disasters are shown in Table III.

Table III

Estimated damages of some dam disasters

Dam	River	Country	Year Failed	Damage in Million \$ U.S.
Mill River	Mill	U.S.A.	1874	1.0
Lynde Brook	Lynde Brook	U.S.A.	1876	1.0
Johnstown	Little Conemaugh	U.S.A.	1889	100.0
Brokaw 2	Wisconsin	U.S.A.	1938	0.7
Malpasset	Le Reyan	France	1959	68.0
Bab-i-yar	Dneiper	U.S.S.R.	1961	4.0
Baldwin Hills	Owens	U.S.A.	1963	50.0
Mayfield	Cowhitz	U.S.A.	1965	2.5
Wyoming	Sybille Creek	U.S.A.	1969	1.5
Pardo	Seco de Frias	Argentina	1970	20.0

5. Risk Criteria

The existing technique of estimating spillway design flood seems to be a compromise between hydrology, economy, and some rather vague risk criteria. Politically and psychologically, it is probably more prudent to avoid any mention of words like "risk". The PMP approach has a great advantage over the probabilistic one in this respect as it gives a strong implication, at least to the layman, that it is a completely safe technique. It must be realised that the use of risk criteria does not necessarily mean that the actual risk being taken is automatically increased; in fact it could, if desired, be kept the same as in the PMP concept. All it does is to rationalize the whole process, and admit the simple fact that if a design flood Q has a probability P associated with it, then a higher flood of magnitude $Q+dQ$ will correspondingly reduce the associated probability to $P-dP$.

The minimum return periods of design floods, as recommended by Schnackenbert (1949), for various categories of dams are shown in Table IV. If a major dam is designed for a return period of 1,000 years, it may sound relatively safe, if considered individually. However, on a cumulative probability basis, the perspective is not so excellent. Currently, according to the

Register of the International Commission on Large Dams, there are approximately 10,000 large dams in the world. If, for the sake of argument, it is assumed that there are 1,000 independent river basins in the world with respect to flood occurrences, and each of the 10,000 dams are designed for the 1,000-year flood, it would mean that the 1,000-year flood will occur, on an average, once a year at one of the independent river basins. This fact, strangely enough, has not received any attention so far from the hydrologists and water resources planners. Furthermore, the problem is bound to get worse on a collective probability basis in the future as more dams are built then go out of use. It immediately leads one to the Borel (1962) approach which states that risks having probabilities of 1 in 10,000 or 1 in 1,000,000 should be accepted, as many risks of the same order are accepted in everyday life.

TABLE IV

Minimum return periods for dams

Type of Dam	Return Period (years)
Major dams with loss of life	
Earth Dams	1000
Masonry or concrete dams	500
Costly dams with no likelihood of loss of life	500
Moderately costly dams	100
Minor dams	20

Use of human lives as a criterion for quantitative estimation of spillway design flood seems, at least to the writer, impracticable for the reason that the productive capacity of any community is limited, and nowhere in the life of a community are unlimited funds available to provide for the complete safety of its individual members. The traffic engineer, in contrast, has to consider accident rates for economic assessment of highways and, as such, has to put a value on human lives. Assignment of monetary value to human lives is wide open to criticism, but as a matter of interest the following values are quoted from the *Traffic Engineering Handbook* (Baerwald 1965).

TABLE V

Human life values

Age (years)	Cost per person (U.S. $)	
	Male	Female
<15	24,000	17,000
15–54	50,000	31,000
>55	10,000	8,000

It is neither proposed nor advocated that the problem of the population explosion of the world can be solved by allowing dams to fail regularly in populous districts! What is advocated, however, is that research in the field of probability of rare precipitation as well as runoff should be accelerated, and attempts should be made to rationalize the procedures for those whose "failure cannot be tolerated". Benson (1964) suggests that it is

> "no more meaningful than saying that hurricanes or earthquakes or tidal waves cannot be tolerated. The best that can be done is to reduce the probability of failure to an extremely small amount; it cannot be eliminated. The engineer should stop deluding himself and the public into the belief that the use of PMP as presently developed entails no risk and affords complete protection. If all the fields of modern engineering and their relation to safety are considered, it becomes evident that this is the only field in which the illusion of complete safety is maintained".

It is high time that hydrologists and water resources planners seriously question our existing philosophy in this field.

6. Conclusion

The basic philosophy behind the current techniques of estimating spillway design flood has not changed much over the last 50 years and leaves much to de desired. The present technique is a compromise between hydrology, economics and some vague risk criteria. It is suggested that a more logical approach will be to use the concept of marginal cost and benefit to design spillways. Once economic analyses are complete, decision-makers will have to make value judgement with respect to possible damages due to failures.

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