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PROCEDURE FOR RISK BASED HYDROLOGIC DESIGN

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ABSTRACT

Risk is inherent in nature and society, while uncertainty depends on investigations. Decision making in water resources is most often a deterministic selection of dimensions which divide the probability distribution of the controlling variable into two regions: performing well and failing to perform. Interpretation of nature is either deterministic, stochastic, or mixed. The various deterministic hydrologic concepts have been used particularly for design of major hydraulic structure. The need exists for bridging the deterministic and stochastic approaches to solving various water resources problems. The large number of sources of risk and uncertainty, and their complexities, explain why they have not yet been fully introduced into the practice of decision making. However, there is a good outlook for significantly improving the decision process in the future by using information on risk.

The decision-making process-which is based on four attributes: benefit, cost, selected technology, and risk and uncertainty-is expected to play a significant role in the future development of water resources. Risk analysis is an alalytical process which complements and aids planning evaluation and design as well as regulatory cost effectiveness analysis. At present, for most of the hydrological designs risk analysis is not being paid attention. Looking at the enormous cost of the construction of the hydraulic structures, and other social, legal, envirinmental and engineering considerations as well as safety aspects, there is an urgent need for adoption of risk based hydrologic design procedures. Risk analysis must fit within a broader evaluation framework, whether it be benefit cost analysis, multiobjective analysis or standard based cost effectiveness analysis. Therefore, risk analysis should be embeded in formal approaches which deal with risk cost trade-offs.

The objective of this report is to present the status of risk based hydrologic design which has been reported in literature. Based on this, a computational procedure has been identified for developing hydrologic design bases for dams and spillways using a risk-based methodology. In order to demonstrate the impact of the risk based analysis on current design practices a few illustrative examples are presented. A brief overview of the hydrologic design criteria adopted by some of the government agencies in India and abroad for the safety evaluation of dams is presented. It is shown that the hydrologic design basis resulting from a risk-based analysis may not always be in conformity with applicable hydrologic design criteria. In addition to that, some of the limitations of the performance of an accurate risk-based analysis are presented.

1.0 INTRODUCTION

Hydraulic structures are designed with reference to some natural events which could be imposed on the structure during its expected service life. This involves hydrologic determination of the magnitude of a design event. In principal, under the statistcal approach the design event can be characterized by an annual exceedance probability or by its reciprocal, the design return period. Any design event which is selected can be expressed either by its magnitude or by its return period, since these two quantities must be directly related by a probability distribution. The engineer's basic task in the design procedure is to determine the "best" design considering the return period of the design event as the basic variable. However, the important issues viz. economic, social, legal and engineering etc. are often overlooked in this design process. On the other hand, risk based design procedure considers all these issues for arriving at suitable return period flood for the design of hydraulic structures.

Though, the return period design method has as its background a large amount of practical engineering experience. However, the fact that the method has been so widely used misleadingly suggests to many engineers the possibility that the return periods presented in design manuals may reflect some approximate non analytical correspondence with optimal economic design. Existing return period design methods fail to systematically account for the many uncertainties in design. The existence of uncertainties in hydrologic and hydraulic design of engineering projects has long been recognized. The hydrological analysis and design involve uncertainties due to various sources. These uncertainties are caused because of errors associated with the input data, output data, inadequate model structure and improper selection of the model parameters. As a result, the calibrated and validated parameters of the models are subjected to uncertainties. Hydrological analysis in general and flood frequency analysis in particular predict the design flood estimates by analysing the annual maximum peak flood records of the limited period. It is preassumed that the sample of annual maximu peak flood is the representative of the population. However, the sample may not be the true representative of the population. Hence, the designer has to take certain amount of risk while designing a particular hydraulic structure. In most of the existing design methods, only the hydrologic uncertainty due to the basic randomness in flood and rainfall frequencies are normally considered (Tung and Mays, 1980).

In view of the many uncertainties the successfulness of engineering designs is accomplished by adopting a safety factor possibly in the form of a high return period. Without a scientific basis for this evaluation, the safety factor has been determined mainly by experience. Consequently, return periods or safety factors for conventional return period design methods are assigned without much of scientific basis and without accounting for the many uncertainties and economic trade-offs.

As pointed out by Yevjevich (1977) there is confusion in using the concepts and terms of risks and uncertainties as applied to the design of hydraulic structres. The concept of hydrologic risk due to the basic stochastic process represented by rainfall events is not new, Yen (1970). However, the concpet of an overall or composite risk is new. The risk involvled in hydraulic structure design is actually a result of the interaction of several types of random variables. The procedure of evaluating risk statements quantitatively for the design of hydraulic structures has been rather elusive even to the most knowleldgeable enginers and theoreticians. Because the variables and parameters for the design of hydraulic structures are random variables, the estimation or assessment of the true values of those random variables is usually based on a limited amount of information and data. This results in various uncertainties, many of which can be integrated to define risk and reliability of hydraulic structure designs. A systematic procedure using analysis and synthesis is needed too for the quantification of risks and uncertainties.

In attempting to introduce risk and uncertainty in water resources decision making, two key questions are posed by practitioners : (i) why is it that many existing water resources decision-making technologies do not currently use methods based on risk and uncertainty and (ii) what will be the benefits from decisions based on risk and uncertainty in comparison with methods that do not use them?

Risk and uncertainty analysis might rightfully be viewed as just another fashionable phrase, giving the impression of a new evaluation philosophy. Acutally, the search for an appropriate risk-analysis paradigm for water resources planning and design disguises some very old and fundamental principles of reliability and uncertainty analysis which have been part of standard engineering practice. This is especially true in water resources engineering because of the natural variability of hydrologic loading factors. However, it seems that much of the early engineering work, which implicitly dealt with the risks of natural hazards and the reliability of engineering structures, has become modified as a body of traditional design standards, criteria, safety factors, and conservative computational procedures. These standards and computational procedures have become substitutes for the continued rational development and testing of newer analytical methods and of refined standards which can aid the fulfilment of the increasing complex requirements for analysis.

Risk analysis in addition, need not be tied to economic efficiency criteria as the sole basis upon which risk-cost trade-offs are to be conducted. Fiering (1976), in discussing the hydrologic basis of reservoir planning and operation, advises caution in becoming too dependent on sophisticated computer algorithms which extend hydrologic data beyond statistical reliability and on optimization models which ultimately become "the focus of the design process rather than its servant". Conversely, empirically based judgements, fixed design critera, and assumptions that served adequately during a period of time when little information and data were available for decisions ought to be reexamined more closely.

The problem of determining probabilities of rare floods by any method (extrapolation of flood frequency curves, use of regional methods, use of joint probabilities, analysis of ancient floods, use of proxy flood data, application of Bayesian theorem, estimation and assignment of probabilities to PMF, etc.) still awaits reliable solutions. A bridge is missing between the deterministic, PMF-based approach and the stochastic (risk-based) approaches. To learn what is happening at the upper tail of flood probability distribution functions, say beyond the 100-year flood, requires a significant research effort in the future.

Flood risks cannot be changed without changing nature. Also, risks cannot be found without collecting and processing a large amount of data using proper concetps or interpretation of the nature of floods and flood-affectng factors. Therefore, one of the major research needs on floods and flood alleviation measures is to understand and quantify flood characteristics. Flood mitigation has two basic activities: (i) identidication of flood characteristics and changing floods by acting on the physical environment and (ii) managing the flood plains. While the second activity has a very long history of human adaptation to flood-prone areas, the first activity is far from exhausted, regardless of the fact that flood control levees were built through the long history of organized societies. The Chinese have tried to leave flood plains to floods for a long time, and to locate communities outside them, until pressure by an increasing population reversed the trend.

The time seems to have come to abandon either purely deterministic or purely stochastic approaches to conceiving and describing the processes of water in nature and within the water resources systems. Instead, the concentraion should be on finding new, integrated, deterministic/stochastic approaches to all phenomena that are governed by physical and geophysical complex laws and regularities, including chance variations.

2.0 THE NEED FOR RISK ANALAYSIS

The designer of any hydrololgic structure is faced with the most important question - what is the risk of failure ? The price of failure of a dam is high due to loss of lives and property; hence the risk of failure of a dam must be minimized. A study of over 1600 dams (Biswas and Chatterjee, 1971) has shown the causes of failure as given in Table 1.

Table 1 : Causes of failure of dams

S. No.	Causes of failure of dams	In percent (%)
1	Foundation problems	40%
2	Inadequate spillway	23%
3	Poor construction	12%
4	Uneven settlement	10%
5	High pore pressure	5%
6	Acts of war	3%
7	Embankment slips	2%
8	Defective materials	2%
9	Incorrect operation	2%
10	Earthquakes	1%

In a more recent study of over 300 dam disasters the authors observed that roughly 35% of the failures were due to inadequate spillway design . The study of dam failures (AWWA, 1966) shows that inadequate spillway design is usually caused by inadequate design flood analaysis and this is the direct concern of the hydrologist. Design floods are estimated either from frequency techniques or as the probable maximum flood.

Based on design criteria, if any hydraulic structure is under designed then there exists risk of its failure, on the other hand, if it is over designed then it leads to higher costs along with larger area of submergence and other concequences. The economic efficiency of meeting a particular design criteria is not known. That is design criteria may be inefficient for a specific situation and may lead to more expensive projects than required. The design criteria may not be correctly set or represent new knowledge or the latest technology and data. Also design criteriamay have been set without rational procedures or with inadequate data and these may not always fit the case, and can limit design flexibility. Further, design criteria do not require a clear definition of the problem, in contrast to formal risk analysis. In the analysisbased on design criteria; there is no scope for updating of new data, hence the use of design criteria discourages development of new information bases. In addition, design criteria based hydrologic analysis does not consider risk, costs and benefits of the alternative solutions and thus various alternatives about acceptable risk and willingness to pay issues can not be examined.

Risk analysis privides a legitimate ethical base for a framework of consent. With risk analysis, the assessment of loss, hazard etc. may be made and this provides much more definitive basis for making decisions about the collective results of actions rather than the individual. Risk based analysis provides the decision maker a broad range of options. It can be a useful mechanism for generation of options, for incerasing insight and for offering greater alternatives. Also, using risk analysis design criteria may be redefined and therefore a better or more soild data base may be identified.

3.0 REVIEW OF LITERATURE

In this section, the various terms associated with the risk bsed hydrologic design have been defined and some of the studies carried out by the various researchers have been briefly reviewed.

3.1 Definitions

For the hydrologic component, risk is the calculable probability of failure e.g. occurrence of a certain flood, occurrence of a drought, etc.. Some of the terms defined by Yevjevich (1985) which are used in risk analysis are mentioned here under.

Risk is conceived here as being equivalent to the probability of exceedence or nonexceedence of critical large or critical small values, respectively, of a particular random variable of a water resouces system. Risk exists objectively in nature, in society, or in technology-based systems, regardless of whether an investigator has properly conceived the controlling random variables or has collected a sufficient amount of data on these random variables. Therefore, risk exists objectively and is not investigator dependent.

Uncertainty is conceived as a lack of knowledge on risk properties, including all errors in models, data, and the length of time spent taking samples. Therefore, uncertainty-in this definition- is investigator dependent. It is measured either by the variance of an estimate from data or by the confidence limits or intervals around that estimate.

Reliability in this text is conceived as the complementary value of the estimated risk to unity. The colloquial term of risk represents the combination of the ture risk and all uncertainties in the form of various types of errors. In general, risk and reliability are estimated by quantitative values of frequency, considered as the best available estimation of probabilities. If one does not know anything about a random variable, subjective assumptions of risk and uncertainty for that variable should be considered only as speculations, or as subjective probabilities, which may sometimes be more misleading. than beneficial : therefore, they should be avoided.

Hashimoto et al. (1982a, 1982b) expand of Fiering's original terminology introduced to account for risk and uncertainty inherent in water resources system performance evaluation. It is clear that the five terms listed below simply represent a set of descriptors which characterize and extend the key components of more traditional engineering reliability analysis; i.e., they focus on the sensitivity of parameters and decision variables to considerations of uncertainty, including some aspects of strategic uncertainty. These terms are:

Robustness - describes the overall economic performance of a water resource project under uncertainty of future demand forecasts, complementing traditional benefit-cost analysis. This extends Fiering's definition from one of sensitivity of system design parameters to variability in future events to one of the sensitivity of total costs to variability in forecasts.

Reliability - a measure of how often a system is likely to fail.

Resiliency - how quickly a system recovers from failure (floods, droughts).

Vulnerability - how severe the consequences of failure may be.

Brittleness - the capacity of "optimal" solutions to accommodate unforeseen circumstances related to an uncertain future.

3.2 Risk and Uncertainties

Basically there have been works to consider the hydrologic uncertainties (inherent, parameter, and model) and also a few attempts to look at the hydraulic uncertainties. Most of the work reported in the literature to consider the hydrologic uncertainties has been through a Bayesian framework. Some works have considered the natural and parameter uncertainties or the natural and model uncertainties and some works attempt to consider all three uncertainties. Most of these works have dealt with improving flood frequency estimates by incorporating parameter or model uncertainties, or both, into the hydrologic analysis.

A few basic works have been reported to explicitly account for the hydraulic uncertainties of design. Yen and Ang (1971) adopted a scheme to account for subjective uncertainties and consequently proposed a design method based on risk analysis. Tang and Yan (1972) proposed a model whereby other sources of uncertainties associated with the hydraulic design of storm sewers can be systematically analyzed, combined and incorporated in the evaluation of the overall risk of alternative hydraulic designs. Tang, Mays and Yen (1975) have invorporated this risk procedure into a dynamic programming approach for the optimal risk-based design of storm sewer systems. An improved model by Tang, Mays and Wenzel (1976) for the optimal design of sewer systems considers expected damages as a function of colume of flooding and incorporates various uncertainties. A risk-basedprocedure incorporating the inherent hydrologic and hydraulic uncertainties was essentially incorporated as a constraint model within a dynamic programming model for selecting the design of pipe crown elevations, slopes and diameters. Young, Chilldlrey, and Trent (1974) developed an optimal design model for highway drainage culverts which considered economic risks. This model defined economic risks as expected flood losses which were converted to yearly flood risks by using appropriate probabilities for each flood. This procedure, however, did not consider the existence of the various uncertainties mentioned previously and relatle these to an overall or composite risk as was done in Mays (1979) and Tang et al. (1978). Watt and Wilson (1978) developed a procedure to allow for risk and uncertainty in an optimal strategy for design of a system of hydraulic structures on a regional economic basis. This procedure did not actually define uncertainties and risk but did define an optimal return period.

Wood (1975b) developed a model for analyzing flood levee reliability. This model considered hydrologic uncertainties for overtopping and structural uncertainties for structural failure of the levee. The hydrologic uncertainty included the inhert and parameter uncertainties through a Bayesian framework. Bogardi, Duckstein, and Castano (1977) studied the effect of stochastic model choice on hydraulic design using as an example the design of a flood levee.

Yen (1978) presented determination of safety factor on the basis of risk and reliability for hydrologic and hydraulic engineering design. Yen's study presented six different safety factors and applied a characteristic safety factor to storm sewer design. First-order analysis of uncertainties of the rational formula and Manning's equation is used to define hydrologic and hydraulic uncertainties respectively. Many other works are reported in the literature which deal with uncertainties in the design and operation of water resources systems; however these do not attempt to define risk and reliability by integrating the various uncertainties (Tung and Mays 1980).

The first item in considering risk and reliability of hydraulic structure design is to delineate uncertainty and other related terms because of the various opinions and connotations among engineers and even among theoreticians. The uncertainty of a water resource system is an indeterministic characteristic and is beyond our rigid controls. In the design of hydraulic structres, decisions must be made under all kinds of uncertainty. The authors divided uncertainties into four basic classifications:

(i) Hydrologic;
(ii) Hydraulic;
(iii) Structural; and

(iv) Economic.

The classification of hydrologic uncertainties for design problems may be divided into three types: (i) Natural; (ii) parameter; and (iii) model uncertainties. Streamflow processes or rainfall events are frequently considered or assumed to be stochastic processes because of the natural, or inherent, randomnes apparent from observations. Becasue of the lack of perfect hydrological information about these processes or event, e.g. infinitely long historical records, there exists informational uncertaianties about the process which are the parameter and model uncertainty. There is presently much debate among hydrologic researchers concerning the hydrologic uncertainties (parameter or model) and how to consider them. The writers recognize the importance of these uncertainties; however these have not been included in the risk models in the present study.

The classification of hydraulic uncertainties for uncertainties for hydraulic structure design may be divided into several types : (i) Model; (ii) construction and material; and (iii) operational conditions of the flow. The model uncertainty results from the use of a certain hydraulic model to describe flow conditions which is essentially an uncertainty in design discharges. For example, flow through or over hydraulic structres are unsteady and nonuniform which can only be described in one-dimensional form by the St. Venant equations. However, equations such as Manning's are quite commonly used in practice which cannot adequately describe unsteady and nonuniform flow. The construction and material uncertainty results partly from the structure size, manufacturers tolerances or construction tolerances varying widely, misalinement of a hydraulic structure, material variability causing variations in the size and distribution of the surface roughness, etc.

Structural uncertainty refers to the failure structural weaknesses. Structural failures can be caused by many things such as: (i) Water saturation and loss of soil stability: (ii) erosion or hydraulic soil failures; (iii) wave action; (iv) hydraulic overloading; and (v) structural collapse. Economic uncertainties can be the result of uncertainties due to construction costs, damage costs, operation and maintenance costs, inflation, and inconvenience losses. The major concern of the study carried out by Tung and Mays (1980) was to show how the hydrologic (inherent) uncertainty and the hydraulic uncertainties can be incorporated into risk and reliability models to define a composite or overall risk as a function of safety factor.

3.2.1 Sources of risk and uncertainty

To answer the question why the techniques of treating risk and uncertainty in water resources have not yet been extensively and fully introduced, it is sufficient to show how many sources of risk and uncertainty can be identified. The multitude of sources of risk and uncertainty and the significant complexion in both the collection of information on them and the estimation of their characteristics have likely been the reasons that risk/uncertainty technology has not been introduced into practice except in only simple cases (hydrologic extrems, for example).

Fig. 1 is an attempt to enumerate some of the sources for the four types of risk and uncertainty : physical-environmental (nature), technological (human structres and measures), socioeconomic (inter-relationships within society), and human performance (as it affects the sources of risk). The complexity of sources of risks, often with dependence and/or interactions, in concept of the concatenation of random and nonrandom events, makes an overall assessment of risk and uncertainty a very difficult task. Therefore, it is not surprising that the professionals have restricted themselvs to simple problems that are much easier to treat, like the physical safety of dams, flood risk, drought risk, etc.

When a professional is faced with complex natural and human-induced phenomena, usually two extreme options are available, with transitional alternatives : (i) a global (synthetic) approach and (ii) a detailed (analytical) approach. In the global approach, the investigator looks at large entities, such as systems, projects, or structures, then collects data on their performance or failures and finds frequencies of failure or under performance or failures and finds frequencies of failure or underperformance, say as the ratio of the number of "failures" to the total number of such structures, projects, or systems. One such case is the study of the frequency of breaches of large dams, which divided the number of failed dams in a decade or so with the average number of large dams in existence during that period of years. The probability of a large dam failing in the year following the study came to be about 10^{-4} (one in 10,000 large dams), accroding to this global method.

The detailed or anaytical method requires extensive investiagtions as well as time and financial means. For a water resources structure, project, or system, all types

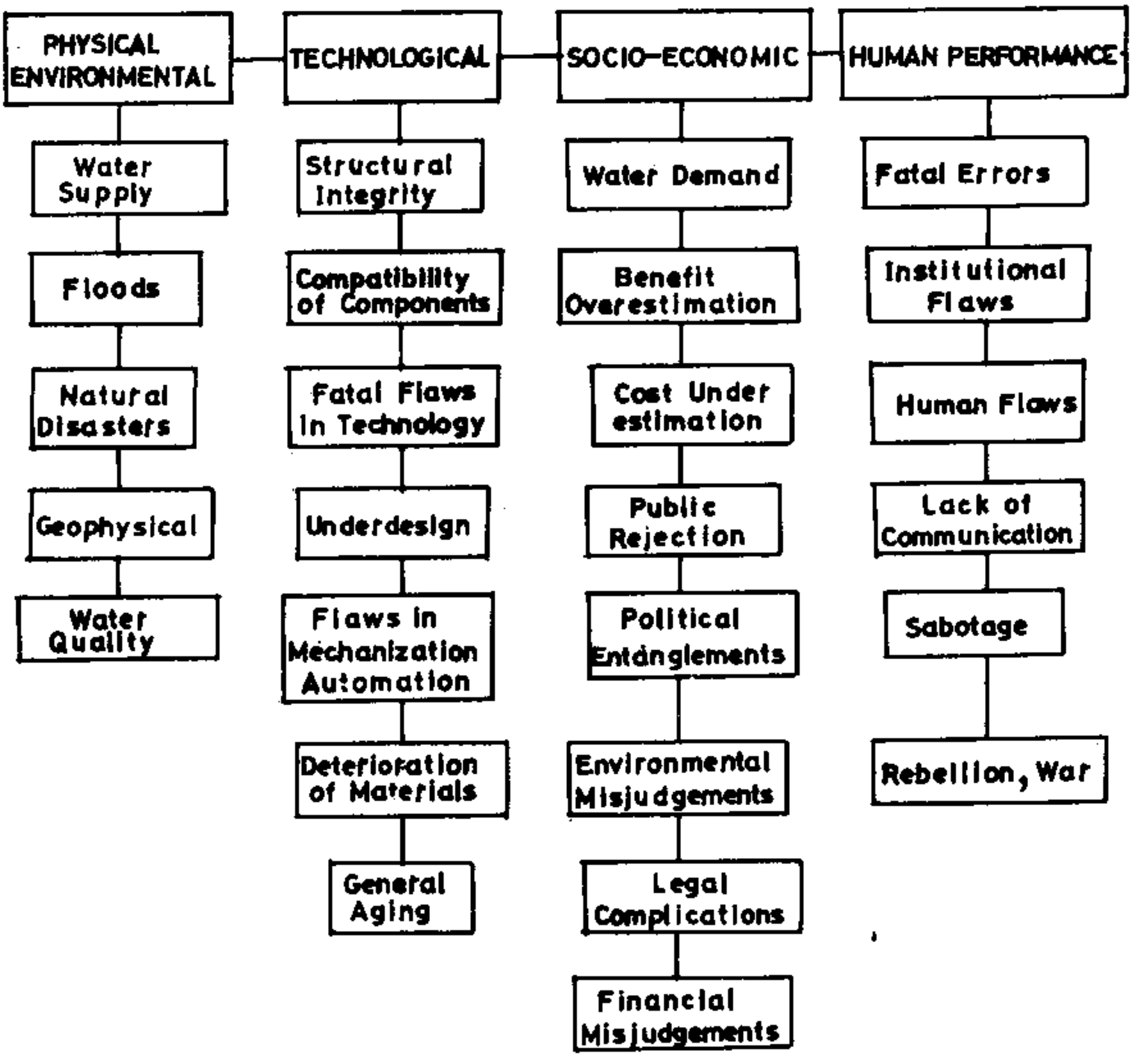

FIG. 1 TYPES AND SOURCES OF RISK AND UNCERTAINTY IN WATER PROJECTS (Yevjevich,1985)

and sources of risk and uncertainty must first be identified as being feasible under given conditions. Then, one should determine whether the sources of risks have independent or dependent random variables, and how dependences can be quantified when they do occur. Next, composite risks are developed. This method needs an update of estimation from time to time as general conditions change with time (aging, deterioration, change of operational purpose or mode, etc.), The lack of a posteriori surveys, such as assessing the evolution of the benefit-cost ratios or similar risk-prone indices, in order to check the validity of the a priori evaluations or decisions, is a drawback of the analytical risk-assessment approach.

What the decision-making methods based on risk and uncertainty can do that the presently available deterministic methods of decision making cannot do? The annual risk involved in any decision must be considered as an attribute of a water resources structure, project, or system. It is similar to counting the annual benefit or annual cost, on an average or for year, as the attributes of these facilities. The probability of failure of a structure or of the performance of a project next year should be as important as the expected benefit or cost, whatever is the definition of the failure. If rare events that exceed a design magnitude mean a damage or loss, there is an average annual loss from that design decision. Also, there is an annual cost for any selected design magnitude. Both are losses to the total benefit realized by the decision. It is implicity assumed, and often proven, that there is an optimal decision, in the sense of a minimum total of the annual cost and the annual loss. Therefore, it can be hypothesized that, based on experience and logical derivation, total losses will be greater when they are not risk-optimized. Besides, the general decision makers, as well as the knowledgeable public, consider it important to have at least a reasonable estimation of risk and uncertainty when a decision is made.

Chow (1979) mentions that risk may be either parametric or non parametric. For a parametric risk, it usually called the exceedance probability $P(X > x)$, where X is a variable and x is a variate, or a particular value of a variable. The parametric risk is usually based on the formulation of a theoretical probability distribution which is essentially a model that is subject to model uncertainty, which means that there is a probable discrepancy between the model and the prototype of the real phenomenon. The commonly accepted procedure to determine the exceedance probability of a hydrologic event for example, is to fit a sequence of its historical data to an assumed probability distribution .

Consider 1000 years of a hypothetical hydrological record whose annual extremes for a ten year period of the record can be selected . The 1000 events must plot on a straight line on a normal probability paper, whereas the sample may plot in various ways but not necessarily, and most probably not, on the straight line. If there is one sample, which represents the only available "realization" of the essemble of time series, the procedure is to use this sample as the basis to establish the best estimate of the population by fitting it to a model. This fitted model is then interpolated or extrapolated to predict the frequencies of future events. Furthermore, control curves or confidence limits can be computed to show the reliability of the predicted frequencies , and nonparametric calculations can be made to give the risk of occurrence of the predicted frequencies to occur in future years.

While the parametric risk is computed on the basis of a probabily distribution of the variable which contains statistical parameters as mean, standard deviation and shewness coefficient; the non parametric risk is computed without such a parametric type distribution. The probability that the m th largest event occurred in N past observations wil be equalled or exceeded in times in n future trials can be non parametrically expressed.

Chow (1979) further states hydrologic extreme values can be treated as independend random variables. As such the exceedance probability so ccmputed represents a pure risk. If the hydrologic events are correlated and contain deterministic components, a stochastic model may be formulated and the probability so computed may be referred to as non pure risk or stochastic risk. Once the hydrologic model is established, furthermore , it may be used to generate hydrologic data sequences by the Monte Carlo methods. Since the historical data is only one 'realization', the generated data may constitute many sequences or 'realizations' as desired. All the generated data are supposed to posses similar, if not exact, statistical characteristics of the historical data. Tahe mathematical relationships are justified if the theoretical assumptions are fulfilled. One major assumption is that the sample is an acceptable representation of the population and thus can be served as a basis to fit the probability distribution model. The 10 year samples taken from the 1000 year population can be any trace of the realiations but rarely the trace of population.

The author mentions that flood risk mapping is the concernstone of the new, comprehensive approach to the reduction of flood damage. The ultimate use of such maps requires resolution of four interdpendent aspects of the problem:

(i) determination of acceptable risk,

(ii) calculation of corresponding flood levels,

(iii) preparation and distribution of maps and

(iv) implementation of landuse restrictions.

Inspite of incrreased funds being devoted to structural flood control measures and flood disaster assistance, there has been continual upward trend in flood damages in various parts of the world due to primarily continuous encroachment on flood plains. Concern for these investments, in terms of their cost effectiveness and politcal and envionmental acceptability has led to adotpion of broader, more comprehensive strategies for flood damage reduction. A cornerstone of both the National Flood Insurance Program in the United States and the National Flood Reduction Program in Canada is the production of flood risk maps which identify clearly those urban and potentially urban areas which are prone to flooding.

Young et al. (1974) presented the procedure for optimal design of highway drainage culverts with objective to incorporate economic risks into culvert design and relate the economic design to conventional design practice. Currently, culvert design is based on hydrologic and hydraulic considerations. Althouth, economic risks are indirectly considered, they have not been directly used in the design of culverts. Economic risks are expected losses and can be divided into three general categories: (i) direct damage to roadway and culvert, (ii) traffic related losses, and (iii) losses due to flood damage in the upstream flood plain. These losses are converted to yearly flood risks by applying the appropriate probablity for each flood.

A balanced design is expected to consider both construction costs and economic risks. To a close approximation, actual data on construction costs vary linearly with culvert area; i.e. small areas yield high risks, large areas yield low risks. The sum of the annual construction cost and risk yields a total cost which has a minimum. It is assumed that alinement, balancing of cut and fill volumes, and traffic vilumes are resolved.

The major disadvantage of the economic design procedure is stated to be the extensive data and considerable computations that are required to obtain the minimum economic design. In order to overcome this disadvantage, a general relationship between the conventional design and the economic design is desired. For determination of general relationships between conventional and economic culvert designs, a sample of conventionally designed culverts is taken. All the sites considered in the study are located on rulal interstate highway in order to make the sample homogeneous. The analysis procedure adopted by the authors for deriving the economic design is automated by use of a digital computer. The analysis calculates the economic design with its expected flood related loss or risk.

Hutchison and Watt (1979) state that flooding is, to a large extent, a natural phenomenon. The levels of rivers and lakes rise and fall with variations in the input rainfall and or snowmelt. As a result, 'flood plain lands which are above the water surface during times of low flow are periodically claimed by the lake or river for the purpose of storage and conveyance during high flow. Since the time immemorial, mankind has in many instances taken advantage of this periodic inundation and flushing of the lake and river systems by reaping benefits in agricultural, water transportation, logging, water supply etc. In recent times, increased urbanization and industrilization as well as increased demand for water supplies, waste removal, land based transportation and water based recreation have served to intensify development of flood plain lands and has led to continuously increased flood damage. Occupation of lands which are suceptible to periodic flooding results in a risk of damage and loss of life. Recognition of this risk, often after severe flood, has led to pressure for so-called flood control structures - dams, reservoirs, diversion channels, dykes etc.

In terms of effective reduction of flood damage, there is a broad spectrum of uses as mentioned below for flood risk maps, (i) such maps are a means of informing the people of living in the areas where there is a risk of flooding. (ii) such maps would encourage an integrated approach to the development of river valleys as far as floods are concerned. (iii) these maps would help in identifying potential flood disaster areas before a flood occurred and hence expedite planning comprehensive solutions to flood problems. (iv) these maps could form the basis for implementing landuse restrictions on the part of government. The restrictions could be imposed on developments in flood risk area (Hutchison and Watt, 1979).

The use of flood risk maps in the above mentioned ways requires resolution of four independent aspects of the problem

(i) The determination of the socially and technically acceptable level of flood risk i.e. the enceedance probabilties or return periods.

(ii) The calculation of the flood levels corresponding to the specified risk level(s).

(iii) The preparation of distribution of flood risk maps which show clearly this information, and

(iv) The practical implementation of landuse restrictions.

The determination of socially and technically acceptable level of flood risk is extremely complex. The following observations by Ian et al. (1979) are intended to set this aspect of the problem in perspective. As yet, no satisfactory guidelines or techniques have been developed and substantial research is necessary to develop such guidelines. It should be recognised that the acceptable level of flood risk depends on the use to be made of the maps. For example, if the maps are going to be used primarily for information and planning purposes, then a number of flood lines corresponding to several specified return periods (e.g., T = 25, 100, 500 years) should be identified on the map. On the other hand, if the maps are going to be emboided in official plans, zoning by laws and other restrictive devices on landuse, then practical considerations will dictate the delineation of preferably one but not more than two lines and much more determination is required to determine the acceptable risk.

A primary input to the decision making process would be location specific or regionalized economic analyses. These would show which levels of protection are economically efficient given various levels of future development and different weights for intangibles, externalities etc. Except for isolated studies this has not been done, instead the level of protection is often selected arbitarily.

Prakash (1985) mentions that risk-based analysis has been used by water resources engineers and planners for the screening and evaluation of structural and nonstructural flood control options over the past two decades or more. On the other hand, planning and design of water supply and hydroelelctric dams have mostly been governed by the various regulations. Recently, a feeling has been developing that some of these regulations result in overly conservative designs. An example is the Nuclear Regulatory Commission Regulatory Guide (USNRC, 1977) for the hydrologic design of uranium mill tailings impoundments. Before 1983, the hydrolocgic design basis for these impoundments was the PMF series (i.e., probable maximum flood [PMF] + standard project flood [SPF] + 100 year flood + normal operating level of the reservoir). Now, the design basis is the 6-hour probable maximum precipitation (PMP) superimposed on normal reservoir operating conditions (USNRC, 1977, 1983). Application of the NRC Regulatory Guides for safety-relateld structures to other non-safety-related or non-nuclear projects may also result in overly conservative designs. Risk-based analysis offers an alternative design methodology which deserves attention.

3.3 The problems of acceptable risk

Stakhiv (1985) states that both lines of risk analysis, one for determining the "safe" level of an accumulation of low-level chronic exposure events and the other for an acceptable level for preventing catastrophic failure, seem to have converged on a "rule of thumb" acceptable level of risk of a "one in a million" chance. That seems to be the social standard by which the outcomes of risk analysis are likely to be judged, regardless of the demonstrated marginal benefits for lower standards. Given this abstract, probabilistically stated measure of acceptable risk criterion must somehow be transformed by engineers into operational design standards for a system and its components. This is by no means an easy task. The probabilities of some postulated extreme natural hazard event, such as earthquqakes, volcanic eruptions, or rare floods, cannot, be credibly extrapolated, even if an acceptable social risk standard is provided as the basis for a design standard. In the situtation of a deterministically computed rare flood event, such as the probable maximum flood (PMF) used for dam design, the exceedance frequency of the PMF may well be less than the "one in a million" occurrence.

The premise is that formal risk analysis is encouraged, regardless of the natural tendency of the public decisions to converge on a relatively narrow range of acceptable risk and the difficulties in actually conducting risk analysis for many water resources problems. In many instances, risk analysis is currently required but not practiced because the analytical tools are not well developed. In addition, what ever application has been attempted, it has taken many inconsistent and incomplete forms, due to the very pragmatic constraints of professional conservatism, bureaucratic and institutional interia, public risk aversion, and political caution. Although risk and uncertainty analysis is clearly stated and required, for example, in the principles and guide-lines for water resouces planning, numerous conventional design practices and traditions within the respectgive engineering professions impede the application of risk and uncertainty analysis as an additional decisison-aiding tool. The inherent resistance to risk analysis is magnified when dealing with low-probability high-consequence events, such as dam failure or extensive, large-scale water supply shortages.

3.4 Risk and Reliability Analysis for Extreme Events

Stakhiv (1985) mentions that dam failure falls in the theoretically difficult and perhaps intractable realm of involuntarily imposed high-consequence low-probability technological risks. The problem of analyzing both high-frequency failures (local flood control) and low-freqnency failures (dam safety) by using a common evaluation paradigm seems to reflect the engineering analog of the social scientists' dilemma of the intransitivity in choice encountered in the aggregation of individual perferences to collective social choices. It is doubtful whether decision theory can provide too much more assistance for resolving this dichotomy. For that reason, the engineering profession's resort to conservative standards as the basis for design may well be justified. Nevertheless, there are aspecvs of each problem which may benefit from a reevaluation of traditional practices.

For example, in the case of the hydrologic design criterion for dam safety. Newton (1983) suggested that the PMF standard for spillway design and reservoir capacity be examined as a joint probability problem rather than as a "worst-case" deterministic computation. In actuality, Newton suggested an intermediate approach to risk analysis for dam safety. Rather than dealing with the numerous risks and uncertainties inherent in such a complex problem, he was suggesting that analysts concentrate on refining the estimate of the PMF, which is a primary hydrologic design criterion. Ballestero et al. (1984) reaffirm and extend Newton's approach by proposing the use of "causal analysis" for rare floods, characterized by more closely examining the physical processes which produce the observed floods. This method is favoured over the simple and traditional methods of curve fitting, which do not use, at least directly, any considerations of the causative physical processes of extreme events (e.g., extratropical, convective storms or rain on melting snow).

The author states that a risk-based analysis needs to consider the consequences and costs of reservoir operation (including damages from high lake levels and discharge, and also damage to the dam and from interruption of services) and the relative likelihood of such events. In general, four metrics are used to describe the consequences for each alternative considered :

(i) likely loss of life.

(ii) economic damages from lake levels, releases, and damage to the dam;

(iii) the cost of actions associated each modification of the dam, reservoir, and any flood warning system; and

(iv) the cost of discontinued or interruptions in service due to damage to or the failure of the dam because of an extraordinary hydrologic event.

However, the National Research Council's Committee on Safety Criteria for Dams (NRC), USA also recognized that in order to conduct risk-cost analyses, estimates of frequencies of extreme flood events would be required along with explicit probability distribution functions for other loading and resistance factors as well as economic benefits and costs. The NRC committee sought "to strike reasonable balances between what is theoretically desirable and what is practical based on current technologies". Thus, the NRC committee amended their recommendations for risk-based analysis with several important caveats.

First, in considering the range or probable hazards from dam failure, the committee chose to categorize dams, based on quaiitative criteria, into low-medium and high-hazard dams. The hazard classifications are based on some combination of measures of the (a) population at risk, (b) likely loss of life, (c) economic losses, and (d) potential dam failure as a proportion of PMF. In reconsidering proposed hydrologic criteria to be used as the basis for a set of evaluation-decision rules in lieu of a formal risk-cost analysis, the NRC committee found that it was reasonable to separate new and existing high-hazard dams.

The NRC committee approached the issue of the present applicability of risk analysis to dam safety rehabilitation in the following manner :

The risk analysis approach has provided a significant trend toward improved assessments and toward selecting more rational, sitespecific spillway evaluation standards within the laslt few years. Though risk-cost analyses may appear to represent the most desirable approach to the goal of dam safety (i.e. in quantifying hazard, failure probability, and acceptable damage) at this time, this method has certain important problem areas or limitations that the user needs to consider (NRC, 1985).

Among the limitations listed were those delineated by an earlier ICODS (Interagency Committee on Dam Safety, 1983) critique of risk analyses along with other deficiencies listed by the NRC committee. The following points were raised:

(i) Risk cost analyses requires estimates of the exceedance probability of extreme hydrologic events. These probabilities are highly variable and are likely to affect the choice of alternatives.

(ii) Many instangible factors cannot be measured in economic terms (loss of life, social dislocation, environmental effects).

(iii) Relevance of analyzing a one-time low-probability catastrophic event by annualizing damages (expected-value approach) is questionable.

(iv) Reliability of hydrologic-hydraulic models has not been sufficiently determined, placing into question many of the critical decision variables needed for economic and loss-of-life analyses. These include rate of breach formation, flood stage, travel time, flow velocity, and debris load.

(v) Forecasting future development below a dam is highly uncertian.

(vi) Reliance on downstream warning and evacuation plans for estimating the threatened population and likely loss of life is questionable.

(vii) Depth and duration of overtopping of dams without failure is largely unknown, as are the effects of encroaching on the freebroad and the probability of spillway failure itself.

The author states that despite the many recognized unknowns and uncertainties of applying risk analysis to the dam safety rehabilitation problem, the NRC committee contends that it is those very reasons that make risk analysis an attractive technique as long as the following conditions and factors are kept in the proper perspective:

(i) Risk-based analyses, as presently performed, generally are not intended to raplace appropriately conservative design standards. Rather, risk-based anayses provide additional information to decision markers to help them decide how limited funds can best be allocated to reduce risks.

(ii) Risk-based analyses are not intended to providee a sole basis for taking decisions. They only provide a portion of the information needed.

(iii) By performing sensitivity studies, many of the problems with performing risk-based analysis can be minimized and the results bounded.

(iv) The process of performing a risk-based analysis often uncovers factors or sensitivity relationship that might otherwise not be identified.

(v) Those factors that cannot be measured in economic terms, such as loss of human life, can be accounted for in separate risk-based analyses and given the appripriate weight.

The NRC committee primarily addressed the pragmatic manner in which certain common evaluation principles can be applied to address low-probability high-consequence events. They recognized that the large degree of uncertainty surrounding both the natural hazard and the potential consequences could not be reliably addressed through a probabilistic risk analysis. In contrast, what the US General Accounting Office (GAO) report on water resource project uncertainty analysis pointed out was that the strategic planning uncertainties, such as in forecasting population, water-use demands and project benefits and costs, may be more important than design reliability analysis for more frequently occurring natural hazards that water resoufces agencies typically plan for (US GAO, 1978). These uncertainties may influence and skew the selection of the most feasible alternative and its optimal scale, upon which the engineering design would ultimately be based.

The author mentions that engineering professon's proclivity for preferring a prior design standards, or "targets" is perhaps unconsciously reflected in the term reliability analysis. Reliability analysis can be thought of as the risk analysis component of project scale and design optimization (rather than optimal plan selection) based on desired target outputs. Risk analysis for water resources project planning, as delineated in the principles and guidelines, serves the broader purpose of assisting in the evaluation of the most appropriate alternative technology to solve a given problem. Risk-cost analysis ought to be but part of a broader evaluation and decision making framework for publicly owned water projects. Risk analysis can be included within a broader benefit-cost or multiobjective analytic framework which reflects more than just economic efficiency pricniples, but directly reflects risk as a component of maximizing social welfare.

Reliability analysis for project design, emphasizing safety and reliability of outputs, comprises but several among many planning ojectives in the selection of a socially optimal alternative. The goal of planning is to choose that project and combination of outputs which is most economically efficient and socially desirable, including health and safety effects. This also means that avoiding certain unacceptable risks becomes part of the evaluation and trade-off analysis. One of the functions of planning is to assess the level of unacceptable risks. This is accomplished through many direct and indirect methods. Standards are merely static reflections of the engineering profession's risk aversion preference.

Nevertheless, most standards reflect the tested accumulation of sound conventional wisdom, which should not be immediately discarded in the rush for applying theoretically elegant evaluation models. Fiering (1976) discusses the brittleness of optimal solutions, i.e., the failure of optimal (efficient) solutions to tolerate ambiguity, changes in technology, deviation from design assumptions, and the use of highly complex mathematical algorithms based on nonrepresentative statistical distributions to fit empirical data, Fiering essentially advocates the examination of alterntive solutions that may be less efficient but which are more robust in accommodating the inevitable uncertainty associated with planned outputs (i.e., strategic uncertainty). He asserts that, indeed, "conventional wisdom (i.e., engineering judgment) might be selecting non-optimal but significantly more robust results than our finely-turned but brittle mathematical models."

Fiering continued his examination of the physical, theoretical and empirical bases of optimal reservoir system design (Fiering, 1982). He uncovered that not too surprising result (to engineers) that decisions based on many judgmental and empirically based engineering design practices related to the reliability, resiliency, and economic behaviour of alternative water resources solutions were not that much different than the "theoretically" determined optimum. The point of risk analysis, in a project planning and design setting, however, is not necesarily to confirm the validity of current engineering design practices or use of standards, although this may be the ultimate outcome of the analysis, but the use the information about uncertainty and risks in order to create a firmer basis upon which to formulate and choose among alternative solutions.

The integration of strategic risk analysis with traditional reliability analysis incorporated within a benefit-cost or multiobjective evaluation framework may be too much to ask for in water resources engineering, particularly for catastrophic events. The use of risk analysis techniques to at least reexamine and more rationally set design standards, safety factors, or design criteria to fit varying circumstances may be the more appropriate application of risk-analysis concepts, especially in relation to the extreme events that pose an intractable decision problem. Ultimately, the best that can be expected from risk analysis may be the use of methods which emphasize evaluation and trade-offs among alternatives and which highlight the significant trade-offs among alterntives and which highlight the significant aspects of the influence of uncertainty on the selection of alternatives, on project scale, and on computing benefits and costs (Stakhiv, 1985).

3.5 Evaluation frameworks for risk analysis

The NRC committee study on dam safety criteria (NRC, 1985) recognized that risk analysis is but an evaluation aid, fitting within a broader evaluation framework for determining socially optimal choices. Much of the previous discussion, covered the assertion that potentially catastrophic natural events or technologies may be inappropriately analyzed by currently accepted evaluation paradigms. By trying to extend conventional risk-benefit-cost analysis to low-probability high-consequence events.

The principles and guidelines may indeed provide a sound benefit and cost measurement and accounting system, but the net-benefits decision rule used in most water resources project justification may be entirely inappropriate for low-probability high-consequence events. Regardless of how sophisticated a risk reliability analysis is conducted, the broader evaluation framework within which the analysis fits may negate or skew the results. This ill-fitting hierarchy of evaluation tools is among the problems that were uncovered by Kunreuther and Linnerooth (1983), who criticized engineers for conducting risk analysis in a relatively unorganized and adhoc sequence.

Lord (1982), in discussing multiobjective models for water resources evaluation, provides an insight into some of the reasons why a truly comprehensive and uniformly applied risk analysis method or evaluation framework may never be possible. All water resource development and management decisions are made in a context which includes three major elements: (i) the decisions are multilevel and involve a combintion of decision authorities; (ii) the decisions are multi-interest and include various value sets and preferences; and (iii) the decisions are multiattribute, involving conflicting objectives and trade-offs.

Vesely (1984) supports the findings of Lord's intuition about the intractability of some complex problems through an examination of the status of probabilistic risk assessment (PRA) as promoted and practiced through the nuclear reactor safety studies program. He concludes that "PRA remains an art and is by no means a symmetrical codified science." Vesely (1984) then goes on to list several observations on the practice of PRA:

(i) The probabilities derived from a PRA are dominated by the uncertainties associated with model error and subjective judgments and assumptions.

(ii) Most PRA's are based on an incomplete modeling of all relevant risks, especially unforeseen, external events.

(iii) The dominant risk contributors in a typical PRA are often those for which the information is most uncertain and judgments most subjective.

(iv) PRA's are largely analyst-dependent and are not generally replicable through independent analysis.

In the case of dam safety risk analysis, for example, the multi-level, multi-interest, multi-attribute quandary of public decision making is further magnified by the overlapping responsibilities of many agencies with different evaluation philosophies.

Moser and Stakhiv (1986) discussed several alternative risk-evaluation philosophies and decision frameworks as another element in their understanding of the diffusion and diversity of risk-analysis approaches. The methods seem to fall under three familiar generic evaluation perspectives:

(i) a regulatory type decision process based on fixed standards and criteria (e.g., cost-effectiveness analysis, least cost design for a given standard, comparative/relative risk assessments).

(ii) normative decision making with explicit or implicit decision rules (e.g., benefit-cost analysis; maximization of net benefits.

(iii) eclectic, relativistic decision making based on multiple objectives, criteria, or decision attributes (e.g., multiobjective optimization) where the relative importance of various objectives is explicitly derived through examining preferences of decision makers.

These generic approaches each have risk and uncertainty analytic counterparts and engineering evaluation paradigms. For instance, traditional engineering reliability analysis is most often associated with cost-effectiveness analysis, i.e., designing a reliable and safe project for the least cost, given a set of standards or desired outputs. However, if risk analysis is to be promoted more vigorously, especially for low-probability high-consequence events, multiobjective analysis is probably most appropriate for this class of problems. It allows the considration of a greater number of important decision variables and seeks to incorporate subjective values based on sound engineering judgment in an explicit manner.

Multiobjective evaluation assumes a more eclectic attitude toward decision making, without a predetermined decision rule such as "least-cost" or "maximize net benefits". Therefore, it may be the most preferable, if not the only appropriate evaluation approach for low-probability high-consequence events. In the example of dam safety, benefits, costs, loss of life, and other engineering reliability factors are objectives which would receive different implied "weights" depending on the relative importance to decision makers. Three relatively distinct forms of multiobjective analysis have emerged, reflecting to a large degree the tradition of normative evaluation philosophy, but without prescribed decision rules. Multiobjective optimization, multi-attribute utility, and multicriteria or multidimensional analytical methods are the three approaches which represent different forms of evaluation. In these methods, the reltive importance of decision variables, attributes, or criteria are not fixed a priori, but are analytically determined or otherwise revealed through pairwise comparisons or sensitivity analysis of the outcomes.

Many of the more recently proposed engineering reliability evaluation frameworks can easily be adapted to the multiobjective analytic viewpoint (Hashimoto et al. 1982a, 1982b; Duckstein and Plate 1984; Asbeck and Haimes 1984). This does not obviate the need for some form of benefit-cost analysis, however, rather, benefits and costs are simply two, among many, important decision variables or objectives, rather than being the primary determinants of the viability of a safety measure (Stakhiv, 1985).

Risk and uncertainty are involved in any of the three hierarchical decision-making phases, namely : in selecting goals, objectives, and purposes of a water resources system or projects. There is always a risk of not meeting a goal, objective, or purpose in a plan or design, regardless of whether the decision maker, the planner, or the designer identifies it or not. Decision making on these hierarchical levels uses two approaches : the deterministic approach, which neglects any risk aspect, and the combined deterministic-stochastic approach, which uses the best assessment of underlying risks and uncertainties.

In an effort to accomplish goals, objectives, and purposes, various activities are needed, such as establishing the general policies and strategies of water resources development, construction, maintenance, and protection, and by planning, design, construction, maintenance, and operation of projects or systems of projects. In each of these activities risk and uncertainty exist, regardless of whether they have been neglected or have been introduced at each level of various decision-making procedures.

For some time there has been a distinction in hydrology between physical or deterministic hydrology, and stochastic or statistical hydrology. In reality, most water supply and water demand processes, as well as other processes that affect decision making in water resources, may be conceived of or interpreted as deterministic-stochastic processes and phenomena. It can be easily shown that approaches to the description of hydraulic phenomena have been changing from deterministic to stochastic when the scales of space, or time, or the variable magnitude are changed.

The PMP and PMF concepts were developed in the 1930s basically for design purposes, either in flood control works or in spillway sizing for dams and reservoirs. The lack of data on floods (relatively short samples) and uncertainties associated with the fitted and extrapolated flood frequency curves or functions further contributed to the development of these concepts. Originally, the concept of maximum possible precipitation (MPP) was advanced. Even today this concept is sometimes presented. There are claims that one can determine the PMP or MPP that has "virtually no risk of being exceeded" (Myers 1967). Evidently, the major players with the concept found that PMP is a more rational concept because it permits the value to be exceeded, however rarely, instead of supporting MPP, which should not be exceeded. The originators always had an escape if MPP were exceeded, namely that "uncertainties in data" would explain the error in the computed MPP. However, they agreed on PMP as a value that can be exceeded in nautre even if uncertainty in data and computations were not present. No originator attached his name to PMP or PMF, likely because of the lack of precision in the concepts.

Recently, several attempts have been made to investigate the feasibility of assigning probability to PMF. This implicitly acknowledges two facts: (1) that PMF can be exceeded with a given small probability and (2) even if one could accept that PMP can not be exceeded, the river basin response to PMP with varying factors would still make PMF a random variable subject to exceedence. These general attempts often concluded, without proofs, that probabilities of exceedence of PMF may be in the range

$$P(Q_{max} > PMF) = 10^{-3} - 10^{-5}$$

or one in 1,000 years to one in 100,000 years, on the average; in some cases it may even be 10^{-6} (a range of 2-3 order of magnitude). Some researchers have been advocating a value for (1) 10^{-4} or the 10,000-years return period flood peak.

The introduction of risk analysis in using a reduced design flood such as

$$Q_{max}, \text{design} = a \cdot Q(PMF)$$

with a <1, requires the assessment of the coefficient "a" and the probability of this design flood. Without finding "a" and Q(PMF) and/or the probability distribution function $F(Q_{max})$, it would be impossible to assign probabilities to floods in the range of economically optimized design floods.

The estimation of the range on flood frequency curves between $p = 10^{-2}$ to 10^{-7} from hundred-year to ten-million-year return period floods, by a reliable determination of the upper tail of flood probability distribution functions, is a very uncertain undertaking. However, societies in many parts of the world often require a reduction of the risk, while at the same time they do not find the PMF-based solutions economically affordable. Therefore, the 1930s produced PMF concept is pushing many projects into uneconomical ranges of decision making (Yevjevich, 1985).

4.0 PROCEDURE FOR RISK BASED DESIGN

While designing any hydraulic structure some of the questions which may come in the mind of any designer are:

(i) Why should we do risk analysis?
(ii) What should be the return period for which the structure should be designed?
(iii) What is the risk involved when we design a structure having a design life of n years for a T year return period flood?
(iv) How much risk is permissible?
(v) How are estimates of risk incorporated into decision making?
(vi) How do decision makers treat uncertainties associated with different risks and hazards?
(vii) What are criteria for comparing and evaluating different risk management policies?
(viii) How can risk analysis be reconciled with the use of long standing engineering practices, criteria, and standards?

The technique of probable maximum flood, despite its name, is a totally deterministic concept and as such has no risk associated with it. Because there is no proof of the existence of extreme boundaries in the meteorological factors which cause floods. (Yevjevich, 1968) states that the concepts of maximum probable precipitation, maximum probable flood and other similarly named imaginary events may be considered as arbitrary. They are concepts of expediency. Frequency analysis, on the other hand, accepts events of any magnitude as being possible; although as the magnitude increases so the probability of occurrence decreases (Kite, 1977).

Mc Caig and Erickson (1959) state that in the past it has been common practice to design major dams for floods having theoretical return periods of upto 10,000 years. The ASCE Hydraulics Division Committee on Hydrometeorology (ASCE, 1973) has suggested that the probable maximum flood is perhaps equivalent to a design period of 10,000 years. This elementary procedure takes no account of the increase of risk with increasing project life or of the economically optimum design.

4.1 Economic Design

A second procedure sometimes used in the design of hydraulics structure relates the design spillway capacity not only to the magnitude and frequency of possible floods but also to the monetary value of dam, the unit cost of the spillways and the value placed upon the lives and property of the people downstream of the dam. McCaig and Erickson (1959) have provided a description of this method of design using in their example log normal distributions of fall and spring floods.

If the average annual losses for a particular structure can be expressed as:

$$C_1 = \sum \Delta L \, P \tag{1a}$$

Where, ΔL is the incremental average loss for a particular design flood, x, in dollars and P is the exceedence probability of that design flood; and if the average annual cost of spillway is given by:

$$C_2 = \Delta x \, Q \tag{1b}$$

Where, Δx is the incremental cost, in daollars per cumec, of providing spillway capacity for flow Q cumec; the optimum structure design will occur when:

$$C = C_1 + C_2 \tag{1c}$$

is at a minimum. That is for a particular structure and a set of flood flows with different structure capacities a graph of C versus capacity or design flood can be obtained.

McCaing and Erickson (1959) assumed a 2-parameter lognormal probability distribution for flood events so that:

$$p = \frac{1}{\sqrt{2\pi}\,\sigma_y} \int_y^{\infty} e^{-\frac{(y-\mu_y)^2}{2\sigma_y^2}} \, dy \tag{1d}$$

where y is the logarithm of the flood event, x, and μ_y and σ_y are respectively the population mean and standad deviation estimated from the logarithms of the recorded flood events. The optimum design capacity Qd, can be obtained by substituting equations (1a), (1b) and (1d) in equation (1c), differentiating and equating to zero.

The ASCE Hydraulics Division Committee on Hydrometeorology (1973) has described a similar procedure to Mc Caig and Erickson but desired for the re-evaluation of the spillway capacity of existing dams. A series of alternate project designs are identified by their spillway design floods, e.g. the 500 years design project, the 1000 years design proejct etc. This series would include the existing project. For each of the possible projects the costs associated with an array of floods with return periods varying from very low to very high are determined.

Kite (1977) states that damages caused by the various floods to each of the alternate proejct designs should include upstream damages (in the event of overtopping and subsequent failure of the dam) to recreation, piers, boats, buildings, loss of power, loss of water supply; to the structure itself including dam fill eroded, repair time, power house losses, switchyard losses etc., and damage downstream of the dam including deaths, injuries, property damage, compensation for loss of water supply, power supply, telephone, road accesses and lost employment. The ASCE (1973) example death was valued at $ 150,000, permanent disabling injury at $200,000 and a non-disabling injury at $10,000. It has been further suggested that the property damages should be determined by carrying out a stage damage analysis using measured flood profiles.

For each project the average annual risk can be calculated by arithmatic strip integration of the area beneath the return period damage curve. The cost of each alternate project design is known and can be converted to an average annual cost. This cost is known as the `operating rate ` (McCaig and Erickson, 1959), may include items for interest, taxes, depreciation , etc., and normally ranges between 8 and 10 percent of the total capital cost . Curves of the type shown in Fig. 2 can then be drawn and the optimum project design determined. (Kite, 1977).

The series of alternate projects might consist of one dam design with flood of successively longer return period being accommodated by a longer spillway, by downstream flood protection work, by paving the dam top and downstream dam surface to reduce erosion from overtopping, by construction of an upstream reservoir to reduce inflows or by other similar means.

4.2 Risk Design

Neither of the two techniques described above include the concept of total risk. For any hydraulic sturucture there is a total risk of failure which can be broken down

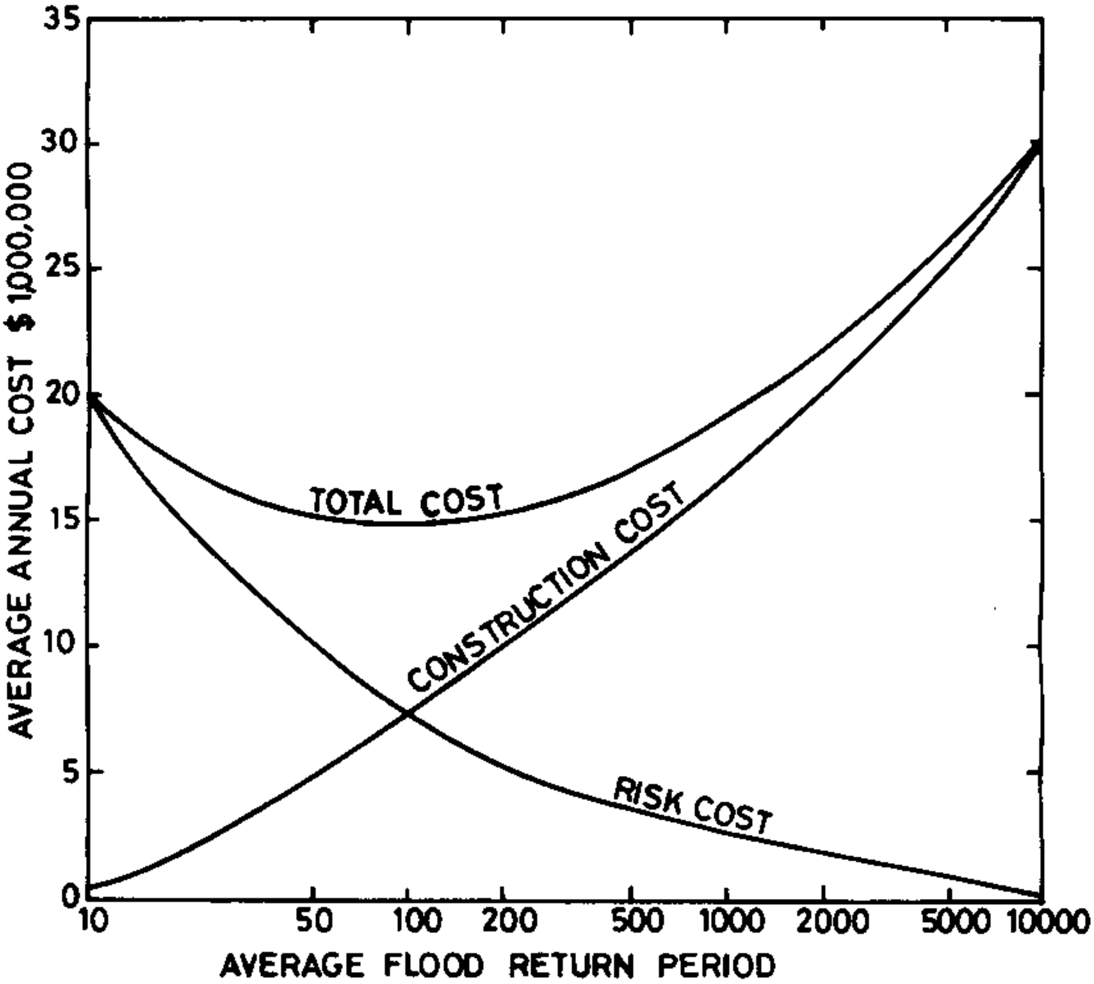

FIG. 2 AVERAGE ANNUAL COSTS FOR DIFFERENT DESIGNS (EXAMPLE ONLY) (KITE, 1977)

into the risk of failure of each project component i.e. hydrologic, hydraulic and structural. The risk within any component can then be broken down into true risk and uncertainty. Yen and Ang (1971) have used the terms objective risk and subjective risk.

The calculation of risk is based on the assumption that the underlying event distribution is known. As an example, if it is known that flood magnitudes in a particular river valley location follow the lognormal distribution and that the time-distribution of the floods follow a Poisson distribution then the risk that the flood of a certain magnitude will iccur in the next five years can be computed exactly.

Uncertainty occurs because the basic data available contain random measurement and computation errors, systematic errors, non-humogenity in the time, loss of information in changing from a continuous record to a discrete data set and so on. These imperect data are then used to estimate the parameters of the assumed population distribution. Uncertainty generally increases as the variance of the sample data increases and decreases as the sample length increases.

Thomas (1971) has evaluated the errors in streamflow estimates made from a continuous stage record while Moss (1969) has related the standard error of discharge estimates to the number of streamflow measurements made per year and the associated costs of maintaining the station.

The effect of uncertainty on the parameters of the population distribution can be included in an analysis by computing the standard error of estimate of the particular distribution at the required probability level. Confidence limits around the expected event magnitude can then be calculatled.

To summarize this concept, hydrologic risk is made up of basic risk and uncertainty both of which can be evaluated. What cannot be evaluated is the error caused by selectling the worng distribution to fit the sample data. It is true that the goodness of fit of a distribution, once chosen, can be measured using the Chi-Square or Kolmogoroy-Smirnov or similar tests and thus the best-fitting distribution can be selected. Generally, however, the sample data will occupy the central portion of a frequency distribution while the event magnitudes which it is required to compute will be in the extremes so that the best-fitting distribution may not necessarily be the best to use.

The computation of stantard errors of estimates for various common distributions has been described earlier. The remainder of this chapter will cover the calculation of basic risk, the assumption being made that the underlying distribution is known.

The return period for which a structure should be designed is calculated based on the risk acceptable. Risk is nothing but the probability of occurrence of a flood at least once during successive years of design life. Risk acceptable depends upon economic and policy considerations.

If for a time invariant hydrologic system the probability of occurrence of an event, x, greater than the design event, x_o, during a period of n years is P, then the probability of non-occurrence, Q is 1-P.

If this design event has a return period of T years and a corresponding annual probability of exceedance of p then:

$$p = \frac{1}{T} \tag{2}$$

The probability of non-occurrence in any one year is:

$$q = 1 - \frac{1}{T} \tag{3}$$

The probability of non-occurrence in n years is:

$$Q = \left(1 - \frac{1}{T}\right)^n \tag{4}$$

Hence, the probability that x will occur atleast once in the n years i.e. the risk of failure, R is:

$$R = P = 1 - \left(1 - \frac{1}{T}\right)^n \tag{5}$$

i.e.

$$R = 1 - \left(1 - \frac{1}{T}\right)^n \tag{6}$$

where,

R is the risk

T is the return period for which the structure should be designed, and

n is the design life of the structure.

For example, if a hydraulic structure is having a design life of 100 years, the risk involved if it is designed for 50 years return period flood may be computed as below.

Here, n = 100 years, T = 50 years and the risk (R)involved may be computed by substituting the values of n and T in equation (5) as given below:

$$R = 1 - \left(1 - \frac{1}{50}\right)^{100}$$

= 0.867, i.e. 86.7%.

Based on the risk acceptable the return period for which the structure should be designed can be ascertained.

Table 2 gives return period associated with various degrees of risk and expected design life using equation (5). Fig. 3 provides solution of equation (5).

Table 2 : Return Periods Associated with Various Degrees of Risk and Expected Design Life

Risk %	Expected Design Life							
	2	5	10	15	20	25	50	100
75	2.00	4.02	6.69	11.0	14.9	18.0	35.6	72.7
50	3.43	7.74	11.9	22.1	29.4	36.6	72.6	144.8
40	4.44	10.3	20.1	29.9	39.7	49.5	98.4	196.3
30	6.12	14.5	28.5	42.6	56.5	70.6	140.7	281.0
25	7.46	17.9	35.3	52.6	70.0	87.4	174.3	348.0
20	9.47	22.9	45.3	67.7	90.1	112.5	224.6	449.0
15	12.8	31.3	62.0	90.8	123.6	154.3	308.0	616.0
10	19.5	48.1	95.4	142.9	190.3	238.0	475.0	950.0
5	39.5	98.0	195.5	292.9	390.0	488.0	976.0	1949.0
2	99.5	248.0	496.0	743.0	990.0	1238.0	2475.0	4950.0
1	198.4	498.0	996.0	1492.0	1992.0	2488.0	4975.0	9953.0

For example, for what return period must a highway engineer use in his design of a critical underpass drain if he is willing to accept: (i) only 10% risk that flooding will occur in the next five years?

Here, the risk involved (R) is = 0.10, n = 2 years,

The return period to be adopted may be computed by substituting these values in the following equation:

$$R = 1 - \left(1 - \frac{1}{T}\right)^n$$

or

$$0.10 = 1 - \left(1 - \frac{1}{T}\right)^5$$

or T = 48.1 years.

This means that there are 10% chances that a 48.1 years flood will occur once

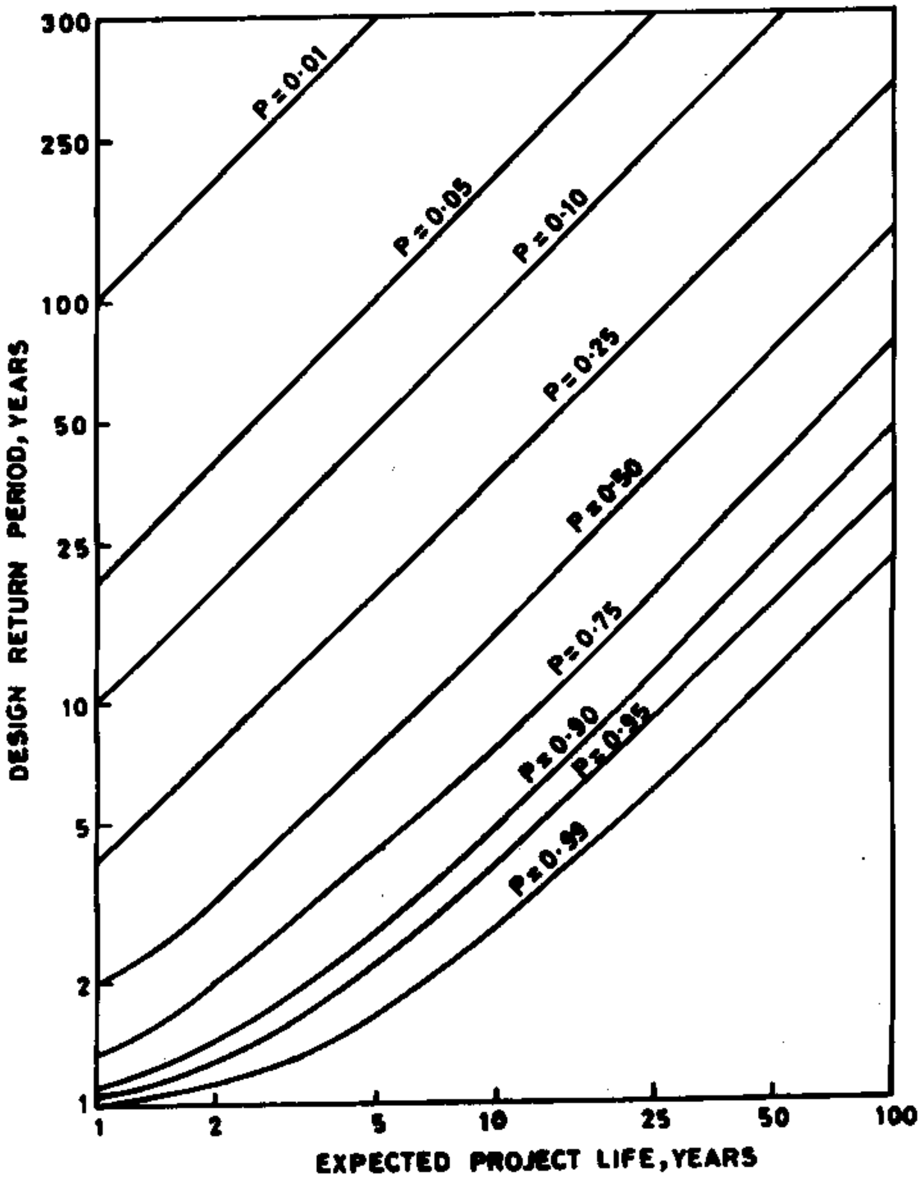

Fig.3 Theoretical probability of failure for given project life and design return period (Kite, 1977)

or more in next five years.

These values may also be read from the Table 2.

If one is interested in knowing exactly once, twice, thrice or any other number of times the occurrences of a flood then concepts of Binomial distribution or Poisson distribution as discussed below should be used.

4.2.1 Binomial distribution

The Binomial distribution is a discrete distribution and is based on Binomial theorem which states that probability of exactly x successes in n trials is:

$$P(x) = \binom{n}{x} p^x q^{n-x} \tag{7}$$

where,

$$\binom{n}{x} = \frac{n!}{x!\,(n-x)!}$$

q = 1-p

p = probability of exceedance/success,
q = probability of nonexceedance/failure,
x = number of exceedance/successes,
n = total number of events.

The assumptions for Binomial distribution are same as for Bernoulli trials. Tossing a coin or drawing a card from a pack are examples of Bernoulli trials which operates under the following three conditions:

(i) Any trial can have either success or failure, true or false, rain or no rain.
(ii) Successive trials are independent
(iii) Probabilities are stable

Binomial distribution is valid under above three conditions.

If a dam is having project life of 50 years then what is the probability that flood with return period of 100 years will occur (i) once, (ii) twice during the life of the dam.

Here, n = 50 years, T = 100 years

the probability of exceedance p is:

$$p = \frac{1}{T} = \frac{1}{100} = 0.01$$

$$q = 1 - p = 1 - 0.01 = 0.99$$

$$x = 1$$

$$n\text{-}x = 50 - 1 = 49$$

$$P(1) = \binom{50}{1} . (0.01)^{1} . (0.99)^{49}$$

$$= \frac{50!}{1!\,(49)!} (0.01)^{1} (0.99)^{49}$$

= 0.306

= 30.6%

This means that there are 30.6% chances that 100 years return period flood will occur once during the project life.

4.2.2 Poisson distribution

The terms of a Binomial expansion are little inconvenient to compute in any large number. If n is large (> 30) and p is small (< 0.1) then Binomial distribution tends to Poisson distribution.

$$P(x) = \frac{\lambda^{x} . e^{-\lambda}}{x!} \tag{8}$$

where, $\lambda = np$

The conditions for this approximation are:

(i) The number of events is discrete;
(ii) Two events can not coincide;
(iii) The mean number of events in unit time is constant; and
(iv) Events are independent.

The above example can be solved with Poisson distribution also. Its solution using the Poisson distribution is given below.

For this case, $\lambda = np = 50/100 = 0.5$

$$P(1) = \frac{\left(50 \cdot \frac{1}{100}\right)^1 e^{-(0.5)}}{1!} = 0.303 = 30.3\%$$

The results obtained by Binomial distribution and Poisson distribution are almost same.

Kite (1977) states that if the series of recorded or measured events are not an annual series but a partial duration series with an average of K observations per year then the probability that the T-year event will be equalled or exceeded in n consecutive years is :

$$P = 1 - [1 - 1/T\,K]^{nk} \qquad (9)$$

Figure similar to Fig. 3 (for which K=1) can be drawn for all values of K.

If failure is associated not with exceedence of the design event but with failure to reach the design event, e.g. a drought design, then the return period must be redefined.

In the event that the design return period is made equal to the expected project life there is a 63.4% chance of failure of the rroject. This can be shown in Equation (5) by putting T = n:

$$P = 1 - (1 - 1/n)^n \qquad (10)$$

In the limit as $n \to \infty$

$$(1-1/n)^n \to 1/e = 0.368 \qquad (11)$$

and so, for large n, P tends to 63%.

Similarly, supposing that a project has been designed against a hydrologic event of return period T years then the risk of failure after completion of n′ years of the expected project life of n years can be calculated.

Writing Equation (4) as

$$Q = [(1-1/T)^T]^{T/n} \qquad (12)$$

and using the same asymptotic approximation as in Equation (11), Gill (1972) has shown that for a given value of P or Q there is a linear relationship between T and n, as :

$$Q = (1/e)^{T/n} \qquad (13)$$

$$n = T \ln (1/Q) \qquad (14)$$

A frequently used approximation resulting from Equation (5) is :

$$T \approx n/p \qquad (15)$$

Gumbel (1955) termed this the "design quotient".

The probabilities referred to above are all probabilities of occurence of an event of a certain magnitude. Also of interest is the average probability of occurrence of all events above that certain magnitude. For example, in a series of n annual events the number, m, of events which equal or exceed the T-year event is (n+1)/T. The annual probability of occurrence of the maximum event is 1/(n+1), of the second largest event is 2/(n+1), of the third largest event is 3/(n+1), etc. so that, the average probability p of the n′ events which exceed the T-year event is given by :

$$\bar{p} = \left(\frac{1}{n+1} + \frac{2}{n+1} + \frac{3}{n+3} + \ldots + \frac{n'}{n+1}\right) / n' \qquad (16)$$

Benson (1967) has shown that this expression reduces to :

$$\bar{p} = (n+T)/2T\,(n+1) \qquad (17)$$

which, as n approaches infinity, becomes

$$\bar{p} \approx 1/2T \qquad (18)$$

Thus, in general, the average probability of occurrence of all events above the T-year event is approximated by the probability of the 2T-year event. For example, the average probability of occurrence of all event greater than the 100-year event is approximately 0.005, whicn corresponds to the 200-year event.

The expressions developed so far have all been distribution-free, that is, no assumptions have been made regarding the underlying event distribution. If it is required to estimate the event magnitude corresponding to the design return period computed from, for example, Equation (5) then a probability distribution must be assumed.

To show the wide variation possible in the results of this assumption of a distribution, Fig. 4 is adapted from Gumbel (1955). This figure shows the relationship between design quotient and the reduced variable, z.

$$Z = (x - \mu)/\sigma \qquad (19)$$

Those distributions shown in Fig. 4 which have fixed coefficient of skew are the normal with $\gamma 1 = 0$ and the double exponential (or type I extermal) with $\gamma 1 = 1.3$. The coefficients of skew have been arbitrarily chosen for the other distributions shown on the figure.

If the assumptions are made that events are independent and the mean number of events in unit time is constant then the Binomial and Poisson distributions can be used to evaluate risk. For a Poisson distribution of event occurrences and an extermal type I distribution of event magnitudes, Shane (1966) has defined the design event, x, as :

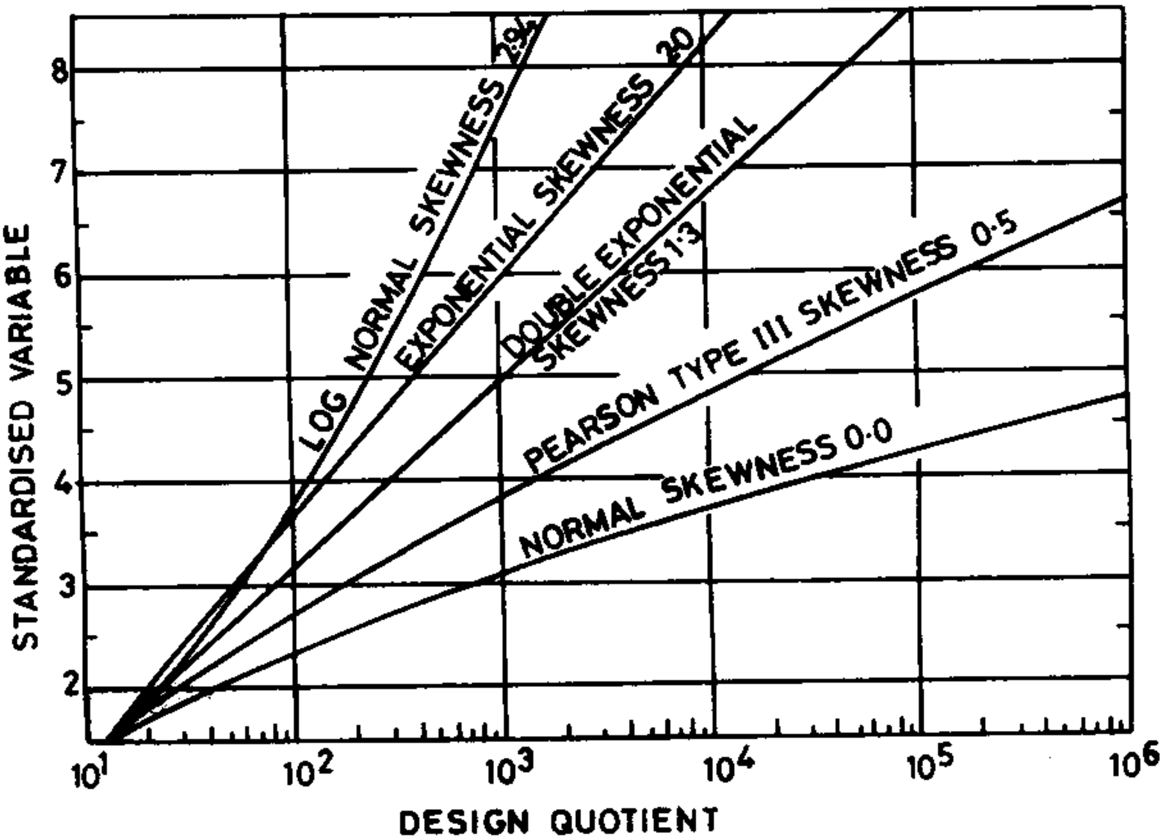

FIG. 4 STANDARDIZED VARIABLE Vs DESIGN QUOTIENT (KITE,1977)

$$x = v + \gamma \ln (\lambda F) \tag{20}$$

where v is a base flow, γ is a parameter of the extermal distribution, λ is the expected rate of occurrence of events, λ=np, in the Poisson process and F is the risk favtor. The maximum likelihood estimates of γ and λ are given (1966) as :

$$\hat{\gamma} = \bar{X} - \nu = x - v \tag{21}$$

and

$$\lambda = n_e/n \tag{22}$$

where n_e is the number of events recorded and n is the period of record.

Benson (1960) investigated the variation which occurs when small samples are used to estimate a frequency distribution for which the parameters are knwon exactly. Starting with a known frequency curve Benson (5) constructed short random data sets, drew best-fitting curves and estimated events at different return periods from those curves. From a basic set of 1000 points, 100 records of ten points, 40 records of 25 points, 20 rocords of 50 points and 10 records of 100 points each were drawn. It was found that records of up to 25 points could not define satisfactorily even short-term events. Long-term records (40 to 50 points) were found to define event magnitudes up to the length of the records with reasonable accuracy.

Yen and Ang (1971) have described a procedure for designing hydraulic structures on the basis of a risk analysis. Using as an example the design of an urban sewer system, an overall project risk was chosen on the basis of possible property damage. The hydraulic and hydrologic risks are combined as α_h and are related to the structural risk, α_s, and overall risk, α

$$(1 - \alpha) = (1 - \alpha_s)(1 - \alpha_h) \tag{23}$$

Yen and Ang then defined the combined hydraulic and hydrologic risks as :

$$\alpha_h = P(x > Q_c)\, P(N > \upsilon) \tag{24}$$

where $P(x>Q_c)$ is the probability of an event X exceeding a design event, Q_c, (the hydrologic risk) and $P(N>\upsilon)$ is the probability that N, a random variable, will exceed υ, a safety factor (hydraulic risk), where

$$\upsilon = Q_b/Q_c \tag{25}$$

and Q_b is the discharge actually used in design. N was assumed to be distributed lognormally with unit mean and a variance, σ_N^2 equal to the total of the variances of the uncertainties such as inaccuracy of measurement, systematic errors in computation, etc.

$$\sigma_N^2 = \sigma_1^2 + \sigma_2^2 + ... \qquad (26)$$

If α and α_s are known, then α_h can be determined from Equation (23) and for various values of υ, the safety factor corresponding values of $P(X>Q_c)$, the hydrologic risk, can be found. The equvalent design return period can be found from Equation (5) and, assuming a probability distribution to fit the observed data, the corresponding event magnitude, Q_c, is found. Yen and Ang (1971) used a type I extermal distribution although any other suitable distribution could equally well have been used. By plotting values of Q_c versus α (or $Q_b = \upsilon Q_c$ vs α) the optimum discharge can be found. Thus, by defining rigorously the hydrologic risk, the common hydraulic practice of using a safety factor to include the effects of hydraulic risk is provided with a scientific basis.

In the event that no stremflow records are available at the design site, Davis et al. (1973) have described a method of evaluating uncertainty by considering the distribution of rainfall events. If the number of rainfall events per season, N, is Poisson distributed with mean λ, i.e.

$$P_N(x|\ \lambda) = \frac{\lambda^x e^{-\lambda}}{x!} \qquad (27)$$

and if the amount of rainfall, R, per event is exponentially distributed

$$P_R\ (k\ /\ u) = ue^{-uk} \qquad (28)$$

where l/u is the mean rainfall per event, then the return period of k units of rain in a season, T, is

$$T_R(k\ /\ \lambda,u) = [1 - \exp\ (-\ \lambda e^{-uk})] \qquad (29)$$

By using a linear rainfall-runoff relationship

$$Q = C\ (R-A) \qquad (30)$$

where C is a coefficient depending upon the rainfall characteristics of a given watershed and A is a measure of initial abstraction, also depending on the watershed, then an expression for the probability density distribution of the flood return period, T_Q, can be given as

$$T_Q\,(y/\lambda,u) = [1 - \exp\{- \lambda + \lambda P_Q\,(y/u)\}]^{-1} \tag{31}$$

where P_Q (y/u) is the distribution function of runoff per event.

Uncertainty is included in the analysis (Davis, 1973) by considering the parameters λ, u and c as variables. Davis et al. assumed that λ & u could be described by a 2-parameter gamma distribution while a beta distribution was used for c.

The results of this approach provide design flows relying only on rainfall data for watersheds with ungauged streams by taking into account the uncertainty of the site parameters. It was found by Davis et al. (1973) that a closed form solution was not possible and so data generation was used to derive the distribution of the flood return period.

4.3 Risk-Based Design Methodology

Prakash (1985) states that a risk-based analysis of a water resources development project involves the following steps :

(i) Determine a frequency distribution (continuous or discrete) for the magnifude of different floods, q. It could be as in Kite (1977).

normal dis ribution $$- f(q) = \frac{1}{\sigma\sqrt{2\pi}} \exp[-(q-\mu)^2/2\sigma^2] \tag{32}$$

lognormal distribution $$- f(q) = \frac{1}{q\sigma_g\sqrt{2\pi}} \exp[-\{\ln q-\mu_g\}^2/2\sigma_g^2] \tag{33}$$

Pearson Type III $$- f(q) = \frac{1}{\alpha\,\Gamma(\beta)} \left\{\frac{q-\gamma}{\alpha}\right\}^{\beta-1} \exp[-(\frac{q-\gamma}{\alpha})] \tag{34}$$

log *Pearson Type III* - $f(q) = \frac{1}{q\alpha\,\Gamma(\beta)} \left\{\frac{\ln\ q-\gamma}{\alpha}\right\}^{\beta-1} \exp\left[-\ \frac{\ln}{a}\right.$ (35)

exponential - $f(q) = r \exp[-r\,(q + q^*)]$ (36)

or any other appropriate frequency distribution.

(ii) Estimate the damage likely to occur in the ranges of floods q_o to q_c and from q_c to PMF where q_o = flood levgl with minimum damage and q_c = critical flow above which the dam fails.

Express the damage as a continuous or discrete function of the magnitude of the flood, say,

$$D(q) = M\,\{1 - \exp[-\,s\,(q - q_o)]\} \quad (37)$$

This is an exponential relation, other relationships may be equally viable.

(iii) Estimate the average annual expected damage

$$E(D) = \int_{q_o}^{q_c} D(q)\ f(q)\ dq + \int_{q_c}^{PMF} (M + L)\ f(q)\ dq \quad (38)$$

in which

M = maximum possible limiting damage occurring after dam break
L = loss of services and cost of rebuilding the dam after failure

(iv) Estimate the annualized costs of construction for different feasible alternatives.

(v) Estimate the sum of expected annual damages and annualized costs for all feasible alternatives and rank them or estimate the marginal incremental reduction in expected annual damages and the corresponding marginal incremental incremental addition in annualized costs and rank the options by the ratios of the two.

(vi) Make a decision regarding the level of risk or damages that can be tolerated against the corresponding cost and select the appropriate design.

The unanswered philosophical and real questions implicit in this approach are:

(i) What, if any, is the annualized cost of human lives likely to be lost at different flood levels ?

(ii) How much are we willing to invest today for losses that may occur 50 or a hundred years hence, or perhaps never ?

(iii) The additional expenses to provide for additional safety are definite real costs whereas the extreme events against which we propose to protect are unlikely to occur. How much real expense should be incurred against unlikely events ?

(iv) Is the annualized value a real of anticipated future damages, particularly those resulting from failure, which is a one-time catastrophic occurrence from which the owner may never recover ?

(v) How can we assign probabilities to extreme hydrologic events like the PMF ? Current state-of-the-art hydrology does not permit precise estimates of these probabilities or the underlying probability distributions.

(vi) How should one account for intangible factors like social and environmental impacts ?

4.4. Impacts of Risk-Based Methodology on Designs of Hydraulic Structures

To illustrate the impact of the risk-based methodology on the designs of hydraulic structures, particularly dams, Prakash (1985) considered a dam at a location where the PMF peak is 120,000 cfs and the various design options include the dam or spillway to be designed to fall at 20,000, 30,000, 40,000, 50,000, 100,000, or 120,000 cfs. The resulting estimates of expected annual damages are given in Table 3.

Table 3 : Estimates of Expcted Damages (Prakash, 1985)

Dam/ Spillway Failure Flood (cfs)	Return period (years)	Percent change of Occurrence in a 50-year Life of Dam	Expected Annual Damage (dollars)	Annualized Cost of Construction (dollars)	Sum of Annual Cost and Damage (dollars)	Incremental Benefit Incremental Cost
120,000 (PMF)	10^6	0.005	131,400	410,000	541,400	-
						0.001
100,000	10,000	0.499	131,500	400,000	531,500	-
						0.084
50,000	2,000	2.47	144,160	250,000	394,160	-
						0.390
40,000	1,000	4.88	163,680	200,000	363,680	-
						1.01
30,000	500	9.52	214,080	150,000	364,080	-
						2.72
20,000	100	39.5	350,120	100,000	450,120	-

The above computations assume that the cost of services lost due to failure plus the cost of rebuilding the dam is two times the maximum possible damage; the function and probability density function of floods are described by Eqs. (36) and (37) respectively, and,

$$D(q) = 0 \qquad q \leq 10{,}000 \text{ cfs} \tag{39}$$

$$D(q) = M\{1 - \exp[-s(q - 10{,}000)]\} \qquad 10{,}000 \leq q \leq q_c \tag{40}$$

$$D(q) = M + L \qquad q_c \leq q \leq 120{,}000 \text{ cfs} \tag{41}$$

$r = 9.2 \times 10^{-5}$; $q^* = 30,000$ cfs; and $s = 10^{-4}$

The values shown in Table 3 indicate that, depending on the risk one is willing to accept, other design bases below the PMF may also be feasible. Both the decision criteria (i.e. minimal) annual cost and an incremental benefit-cost ratio of greater than unity) suggest that the dam or spillway should be designed for a peak flood of 40,000 cfs, whereas current design practice may require a design flood of 60,000 to 120,000 cfs.

The author states that in the safety evaluation of the Lake Wohlford Dam in southern California, USA the regulatory requirement was to upgrade the dam or spillway to safely pass the PMF. The dam has a battery of six 6-ft x 3.125-ft siphon spillways and a 120-ft-long side-channel spillway. The discharging capacity of the side-channel spillway gets severely impeded at high flows due to submergence. A number of options were evaluated:

(i) Lower the crest of the side-channel spillway by 1 ft.
This would pass the PMF safely (without overtopping the dam), but would increase the peak outflow by about 61 percent. Peak outflows during less severe but more frequent floods than the PMF would also increase, resulting in larger expected annual damages on the downstream.

(ii) Provide an auxiliary spillway
This would also pass the PMF safely, but would increase the peak outflow by about 37 percent. As in the previous case, this option would also increase the expected annual damages on the downstream side due to less severe but more frequent floods than the PMF.

(iii) Provide a 2-ft-high parapet wall to raise the dam height
This would also pass the PMF safely and increase the peak outflow by about 7 percent. The resulting incremental damages during less extreme but more frequent floods would be smaller than those predicted for the previous two cases. The potential for damages on the upstream side being insignficant, this option was judged to be the best.

If the 7-percent increase in downstream damages was unacceptable, then it may have been worthwhile to examine the expected damages or risks associated with the option of not upgrading the dam. In that case, risk-based analysis would suggest that the dam should be designed for a flood less than the PMF (i.e., the no-action option would appear to be perferable).

5.0 CURRENT DESIGN CRITERIA/STANDARDS AND PRACTICES

Some of the design criteria/standards and practices available in literature as well as the current design criteria/standards and practices adopted in India and by some of the foreign organisations are mentioned below.

Biswas (1971) has given some of the return periods commonly used for different types of structures as given in Table 4.

Table 4 : Some of the return periods commonly used for different types of structures (Biswas, 1971)

S. No.	Type of dams	Years
	A. Major dams with probable loss of life	
1	Earth Dam	1000
2	Masonry or concrete dam	500
	B. Dams with no likelyhood loss of life	
3	Costly dams with no likelyhood of loss of life	500
4	Moderately costly dams	100
5	Minor dams	20

5.1 Current Indian Design Criteria/Standards and Practices

The designer is generally concerned with the return period for which the structure should be designed. In India, the present practice (BIS code) is to specify the design flood for various structures in terms of frequency or return period. The design criteria for some of the hydraulic structures in discussed below.

5.1.1 Criteria for fixing spillway capacity

The hydrologic design criteria for fixing spillway capacity as prevalent in India are mentioned in IS 11223-1985, "Guidelines for fixing Spillway Capacity". According to these guidelines various inflow design floods that need to be considered for various functions of spillways are:

(i) Inflow design flood for the safety of the dams

It is the flood for which the performance of the dam should be safe against overtopping, structural failure and its energy dissipation arrangements, if provided for a lower flood, should function reasonably well.

(ii) Inflow design flood for efficient operation of energy dissipation works

This flood could be lower than the flood for safety of dam and for this the dissipation arrangements, are expected to work most efficiently.

(iii) Inflow design flood for checking extent of upstream submergence.

(iv) Inflow design flood for extent of downstream damage in the valley.

The criteria for classification of dams is based on size of the dam and the hydraulic head (MWL - average flood level on downstream). The classification for the dam is greater of the two indicated by the two parameters as given in Table 5.

Table 5 : Criteria for classification of dams based on size and hydraulic head

Classification	Gross storage (in million cubic meters)	Hydraulic head (in meters)
Small	Between 0.5 and 10	Between 7.5 and 12
Intermediate	Between 10 and 60	Between 12 and 30
Large	Greater than 60	Greater than 30

The inflow design flood for safety of the dam would be as given in Table 6.

Table 6 : Design flood for safety of dam

Size as determined above	Inflow design flood for safety of Dam
Small	100 year flood
Intermediate	Standard Project Flood (SPF)
Large	Probable Maximum Flood (PMF)

Floods of larger or smaller magnitudes may be used if the hazard involved is high or low. The relevant parameters to be considered in judging the hazard in addition to the size would be :

(i) distance to and location of the human habitations on the downstream after considering the likely future developments.

(ii) maximum hydraulic capacity of the downstream channel at a level at which catastrophic damage is not expected.

For more important projects dam break studies may be done as an aid to the judgment in deciding whether PMF needs to be used. Where the studies or judgment indicate an imminent danger to present or future human settlements, the PMF should be used. Any departure from the general criteria as above on account of larger or smaller hazard should be clearly brought out and recorded.

Probable Maximum Flood

Probable maximum flood is the flood caused by probable maximum pre;cipitation. Probable maximum flood is generally obtained by using unit hydrograph and rainfall estimates of PMP. The probable maximum strom is defined as the most severe storm considered reasonably possible to occur. The ASCE Hydraulics Division Committee on Hydrometeorology (1973) has suggested that the probable maximum flood is perhaps equivalent to a design return period of 10,000 years.

Standard Project Flood

SPF is the flood caused by standard project storm which is generally obtained from a survey of severe storms in the general viscinity of the drainage basin or severe storms experienced in meteorologically similar areas.

5.1.2 Criteria for design flood estimation for barrages

For barrages, the CWC 1968 criteria are applicable. Diversion dams or weirs and barrages have usually small storage capacities, and the risk of loss of life and property down stream would rarely be enhanced by failure of the structure. Apart from the loss of the structures by its failure, this would bring about disruption of irrigation and communications that are dependent on the barrage. In consideration of these risks involved the CWC criteria redesigned for floods of frequency 50 to 100 years. For barrages, it requires the use of a 100 year return period flood or standard project flood whichever is higher.

5.1.3 Criteria for design flood estimation for weirs (ungated headworks)

In the case of small reservoirs where the release of stored water due to the failure of the dam would not appreciably enhance the flood hazard downstream, the spillway capacity may be designed for a design flood of specified frequency, say 50 to 100 years as recommended by the Central Water Commission.

5.1.4 Criteria for design flood estimation of road and railway bridges

For road bridges, the Indian Road Congress IRC: 5-1970, Section-I General Features of Design applies. According to this, the design discharge for which the waterway of a bridge is to be designed shall be the maximum flood observed for a period of not less than 50 years; shall be discharge from an another recognised method applicable for that area; shall be discharge found by the area velocity method; by unit-hydrograph method; and the maximum discharge fixed by the judgment of the engineers responsible for the design with comparison of above mentioned methods is to be adopted. For railway bridges, a 50-year flood is to be used for smaller bridges carrying railways of lesser importance like minor lines and branch lines. In the case of larger bridges i.e. those carrying main lines and very important rail lines, a 100-year return period flood is to be adopted as per the railway codes (Indian Railway Standards - 1963).

5.1.5 Criteria for design flood estimation for cross drainage structures on irrigation networks

The BIS Code of practice for design of cross drainage works [IS:7784(part-I)1975] recommends that the design (of waterway) in such cases may be based on 10 to 25-year frequency flood with increased afflux. However, the foundations and free-board etc., should be checked to be safe for the increased afflux and velocities due to a 50 year or 100 year return period flood.

For very large cross drainage works, damage to which is likely to affect the canal supplies over a long period the design should be based on maximum probable flood. It is quite probable that a flood of higher magnitude than the design flood may pass through the structure posing great danger to the stability of foundation and the structure. Return period to take care of this unprecedented and unforseen nature of flood intensities in cases of important structures, an adequate margin of safety is envisaged in the estimation of design discharge. For this purpose, the design discharge may be increased by the percentages given in Table 7 for obtaining the foundation and freeboard design.

Table 7 : Criteria for design flood estimation for cross drainage structures on irrigation networksfoundation and free-board design.

Catchment area (in square kilometers)	Increase in design discharge
upto 500	30% to 25% decreasing with increase in area
500 to 5000	25% to 20% decreasing with increase in area
5000 to 25000	20% to 10% decreasing with increase in area
above 25000	upto 10%

As per Central Water Commission criteria, waterways for canal aqueducts should be provided to pass a 50-100 year return period flood, but their foundations and free-boards should be for a flood of not less than 100-year return period.

The Government of Gujarat has adopted a still severer criteria for cross drainage works of Sardar Sarovar Narmada Canal, which are given in Table 8.

Table 8 : Criteria for design flood estimation for cross drainage structures on irrigation networks adopted by Gujarat Goverment

Catchment area (in square miles)	Design flood to be adopted	
	For design	For checking
0 to 10	100 year flood	100 year flood + 30%
10 to 50	- do -	- do -
50 to 200	- do -	P.M.F.
200 and above	- do - (or S.P.F.)	P.M.F.

5.1.6 Design criteria for flood control schemes

The broad criteria recommended and adopted for design of flood control schemes are given in Table 9.

Table 9 : Design criteria for flood control schemes

Predominantly agricultural	**25 year return period flood on small tributaries and 50 year flood on major rivers**
Town protection works	100 year return period flood
Important industrial complexes, assets and lines of communications	100 year return period flood

According to Ganga Flood Control Commission, subject to availability of observed hydrological data, the design HFL may be fixed on the basis of flood frequency analysis. In no case, the design HFL should be lower than the maximum on record. For small rivers carrying discharge upto 3000 cumecs, the design HFL shall correspond to 25 years return period flood. For the river carrying peak flood above 3000 cumecs, the design HFL shall correspond to 50 years return period.

However, if the embankments concerned are to protect big township, industrial area or other places of strategic importance the design HFL shall generally correspond to 100 year return period flood.

The Rashtriya Barh Ayog recommends that benefit-cost criterion should be properly adopted. But since the relevant data for such an analysis may not be available the Ayog recommends (i) for predominantly agricultural areas: 25-year flood frequency (in special cases, where the damage potential justifies, adopted); (ii) for town protection works, important industrial complexes etc: 100-year flood frequency (for large cities like Delhi, the maximum observed flood, or even the maximum probable flood should be considered for adoption).

Each site is individual in its local conditions, and evaluation of causes, and effects. While, therefore, the above mentioned norms, may be taken as the general guidelines, the hydrologist, and, the designer would have the discretion to vary the norms, and the criteria in special cases, where the same are justifiable on account of assessable and acceptable local conditions; these should be recorded, and, have the acceptance of the competent authority.

5.2 Current Design Criteria/Standards and Practices Adopted by Some Foreign Organizations

Prakash (1985) states that there is some variance in the hydrologic design bases used by different federal, state, and local agencies in USA at the present time. The hydrologic design standards used by some of the federal agencies are abstracted in Tables 10, 11 and 12 (National Academy Press 1985).

Table 10 : Hydrologic Design Standards, U.S. Department of Agriculture, Forest Service

Hazard Potential	**Size/Class**	**Spillway Design Flood**
High	A	PMF
	B	PMF
	C	1/2 PMF to PMF
	D	100-yr TO 1/2 PMF
Moderate	A	PMF
	B	1/2 PMF to PMF
	C	100-yr TO 1/2 PMF
Low	A	1/2 PMF to PMF
	B	100-yr to 1/2 PMF
	C	50-yr to 100-yr

A = dams 100 feet or more in height, or those impounding 50,000 acre-ft or more

B = dams 40 to 99 feet in height, or those impounding 1,000 to 49,999 acre-ft

C = dams 25 to 39 feet in height, or those impounding 50 to 999 acre-ft

D = dams less than 25 feet in height, or those impounding less than 50 acre-ft

To be in conformance with these practices and standards and to make existing dams safe or safer, one has to adopt one or the other of the following options:

(i) Raise the dam if the spillway size is to be kept fixed and there is no potential for any major damage upstream of the dam.

(ii) Lower the crest of the spillway if the height of the dam is not to be raised.

(iii) Enlarge the length of the spillway if the height of the dam is not to be raised.

(iv) Provide an auxiliary spillway if a suitable site is available and options (i), (ii) and (iii) are not feasible.

(v) Use some combination of the above four options.

(vi) Divert floods upstream of the dam.

(vii) Modify reservoir operations.

(viii) Breach the dam.

Implementation of option (i) may result in additional expected annual damages due to additional inundation upstream if the dam does not fail. In cases (ii), (iii), (iv) and (v), if the dam does not fail, then there may be additional expected annjal damage on the downstream due to recurrent floods (less than the design-basis flood) during the life of the dam.

Table 11 : Hydrologic Design Standards, U.S. Department of Agriculture, Soil Conservation Service

Class of Dam	Product of Storage and Effective Height (acre-ft x ft)	Energy Spillway Hydrograph	Freeboard Hydrograph
(a)	<30,000	P_{100}	P_{100}+0.12 (PMP-P_{100})
	>30,000	P_{100}+0.06 (PMP-P_{100})	P_{100}+0.26 (PMP-P_{100})
	All*	P_{100}+0.12 (PMP-P_{100})	P_{100}+0.40 (PMP-P_{100})
(b)	All	P_{100}+0.12 (PMP-P_{100})	P_{100}+0.40 (PMP-P_{100})
(c)	All	P_{100}+0.26 (PMP-P_{100})	PMP

(a) = dams licated in rural or agricultural areas where failure may damage agricultural and, country roads, and farm buildings

(b) = dams located in predominantly rural or agricultural areas where failure may damage isolated homes and facilities

(c) = dams located where failure may cause loss of life and serious damage

*Applicable when an upstream dam is located so that its failure could endanger a lower dam

Table 12 : Hydrologic Design Standards, U.S. Army Crops of Engineers

Hazard Potential	Size/Class	Spillway Design Flood
Low	Small	50-yr to 100-yr
	Intermediate	100-yr to 1/2 PMF
	Large	1/2 PMF to PMF
Significant	Small	100-yr to 1/2 PMF
	Intermediate	1/2 PMF to PMF
	Large	PMF
High	Small	1/2 PMF to PMF
	Intermediate	PMF
	Large	PMF

Small = 50 to 1,000 acre-ft in storage and 25 to 40 ft in height.

Intermediate = 1,000 to 50,000 acre-ft in storage and 40 and 100 ft in height

High = 50,000 acre-ft in storage and 100 ft in height

6.0 CONCLUDING REMARKS

On the basis of this study the following concluding remarks can be made:

(i) Risk analysis can be very useful in understanding how variability and uncertainty about key decision variables influence the choice of hazard control measures and their design by considering the economic, social, legal and engineering issues. However, there does not currently exist a uniform body of knowledge that comprises a sound basis for risk analysis, especially as a substitute for engineering standards and design criteria which have evolved to deal with the inherent variability of natural hazards. Despite this flaw, which affects the uniform practice of risk analysis, there exist many theoretically sound risk-assessment techniques and risk-evaluation models which can be adapted to many problems in water resources engineering. A typical procedure for developing hydraulic design bases for dams and spillways using risk based methodilogy presented by Prakash (1985) has been described in Section 4.

(ii) In the current Indian hydrologic design practice, risk analysis is not considered. Risk-evaluation methods should be employed which directly factor in engineering judgment and subjective values. Efforts should be made to study the hydrologic design of the existing dams and other important hydraulic structures, and suitable risk based methodology should be evolved for the risk based design procedure. Risk-assessment methods must be compatible with the broader evaluation and decision rules used in selecting an optimal risk-cost design, since it is but a complementary aid to evaluation.

(iii) Studies may be carried out to examine the applicability of risk based design by comparing the risk based design and the current design practice considering the various economic, safety, engineering and the other related aspects.

(iv) The current practices and design criteria need a fresh look in terms of potential for increased damages during less severe but more frequent floods both upstream and downstream of the dam.

(v) Designs based on risk-based analysis may result in less expensive modifications to existing dams and less expensive capital investment in new dams provided the risk and its acceptability are precisely defined. In many cases, such designs may even result in less expected annual damages during the life time of the dam (Prakash, 1985).

(vi) Further work is needed to develop and acceptable method to incorporate the potential for and cost of loss of lives in risk-based analysis of dams.

(vii) Further work is also needed to account for failures due to mechanisms other than severe floods (e.g. piping, foundation settlement, etc.) in the expcted annual damage concept. The probability of such failures is not directly amenable to statistical analysis. Some researchers have attempted to analyze such events using the Bayesian probabilistic model (FEMA 1984). However, the method is highly subjective and imprecise.

(viii) Considerable research work is needed to assign probablities to extreme hydrologic events, including the PMF, and to quatify intangible factors like social and environmental impacts.

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