RISK AND RELIABILITY IN WATER RESOURCES MANAGEMENT: THEORY AND PRACTICE

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INTRODUCTION

This is scheduled to be the closing talk of the Third Kovacs Colloquium on "Risk, Reliability, Uncertainty and Robustness in Water Resources Systems". In its final form, it will attempt to capture and summarize the lessons and conclusions to be derived from the lectures and discussions at the Colloquium. The emphasis in the final paper will be on the state of the art in the theory of risk and reliability in water resources management, and on whether and how it finds its way into practice.

Overall, there seems to be a considerable gap between theoretical models and results, and the way in which real water resources systems are planned, designed, operated and managed. At the conclusion of the Colloquium, we may be able to determine whether this is indeed the case, and if so, how the gap might be narrowed.

The following notes serve as a draft for some of the points I view as central to the issue of risk and reliability in water resources management.

THE PHASES OF RISK MANAGEMENT

The phases of risk management are (Plate, 1996):

- List the potential modes of failure of the system.
- Calculate or estimate the likelihood of each failure mode.
- Describe the physical consequences of each failure mode.
- Assess the damages and losses due to each failure mode, and the value, on a selected scale, assigned to each damage or loss.
- Generate the options for mitigating each of the failure modes, wholly or partially.
- Compute the costs of each option.
- Synthesize all of the above into a framework for management of the water resources system, and select the optimal policy for planning, design and operation.

We shall now consider each of these phases. To help in making the discussion more concrete, we refer to a project in which a reservoir is to be constructed on a river, for water supply. The reservoir is designed to regulate the flows, and is operated to provide water to a distribution system. The water is delivered through a treatment plant to the distribution system.

Potential failure modes of the system

Examples:

- The sum of the inflow into the reservoir plus the amount stored in it, is low, resulting in **[S<D] = [Supply < Demand].**
- Demands are higher than forecasted, resulting in **[D>S].**
- Pollution of the water in the reservoir prevents its use, even with the existing treatment plant.
- The treatment plant fails, and the water cannot be used until the plant is back in operation.
- An element of the distribution system fails, eliminating or reducing service to some customers.
- The dam fails, causing damage or loss of life downstream.

Each failure mode will require a different model for reliability analysis (Shamir and Howard, 1981). The models range from aggregate to detailed, as follows.

- Lumped supply - lumped demand: the entire system is depicted as having a single source and a single demand, each a random variable, with a known joint probability distribution of supply and demand. The joint PDF is integrated over the region where **[S<D],** to yield **P[S < D].** When the PDF has a simple form, the computations can be carried out analytically (Shamir and Howard, 1981), by the so-called "statistical-analytical approach". A modification of this approach, which considers also the uncertainty of the probability distributions, especially their tails, has been developed by Ben-Haim (1996).

  The PDF of the demand should be made conditional upon the PDF of the supply, in one or two ways. First, the two variables may both depend on the same external condition, such as hot dry weather. In this case, there is a negative correlation between demand and supply: demands are high when supply is low. The other dependence has to do with demand management. It is wrong to assume that demands are not affected by the level of supply. Public appeals to reduce consumption at times of droughts, followed by compulsory measures to reduce consumption when the situation becomes worse, are very relevant to the analysis. In this case the correlation between demand and supply is positive.

  One might argue that consideration of demand management should be relegated to the phase in which management options are considered, but in fact it is necessary to consider this aspect already at the time the probability **P[S<D]** is computed.

  The analytical approach is still feasible when the number of sources and demands is more than one, say two or three. However, once this number increases the computations become very cumbersome, since they require integration of joint probability functions of several random variables over complicated domains.

- Articulated supply and/or demand: the system is described as a set of several elements (up to a few tens), in series-parallel connection (Shamir and Howard, 1981; Yang et al., 1996-a). Fault tree methods are useful in this case. The computations are based on the probability of failure of each element, and allow explicit evaluation of the residual capacity of the system when each failure occurs.

- Reduced network model of the distribution system: a skeletonized network is used to describe the system, and analytical methods are used to compute topological measures on the network, such as connectivity and reachability (Wagner et al., 1988-a), which are considered to be measures of the system's reliability. The hydraulic performance of the system is not computed accurately for each failure mode (because it is too costly to do), but may be estimated in some fashion.

  A different approach has been proposed by Ostfeld and Shamir (1996), applicable also to multi-quality supply systems. It is based on optimization of the system, under constraints which specify the performance of the system in full operation as well as at times of failure. To accomplish this, two or more "backup sub-systems" are defined, such that when a component of the system fails, one of the backup sub-systems is still capable of supplying the consumers. The performance of each backup sub-system is specified in constraints, which specify the quantities to be supplied, the pressures, and the water quality at the consumer nodes. This level of service need not be the same as for the full system, with no failure, although it can be the same, if so desired. The cost of system operation under normal conditions, without a failure, plus the cost (possibly including penalties for reduced service) at times of failure weighted by the expected time of their operation (which is a small fraction of the total time) are incorporated into the objective function.

- Full network model: A fully detailed model of the network is used in a simulation mode (Wagner et al., 1988-b; Yang et al., 1996-b). In each run, one or more components are removed (failed), and the residual capability of the remaining system is computed with a network solver. The results of simulations are analyzed, to yield the values of the selected reliability criteria for the system.

  Wagner et al. (1988-b) incorporated into the model a function approximating the quantity extracted from the system by a consumer, as a function of the pressure at the connecting node. This amounts to a reduced service at the time a failure occurs. Yang et al. (1996-b) simulate the performance of the system over time, each element subject to failures according to its MTBF, and to repair according to its MTTR.

Likelihood of failures

Examples:

- Using hydrological records, possibly extended through simulation with a longer meteorological record, estimate the probability distribution of cumulative inflows for the relevant time horizons, and of the resulting shortage (see below the section on forecasting).

- Using pump maintenance and pipe break records, estimate the probability of failure of system components, its duration and time to repair, then compute for each failure mode the resulting shortage in supply.
- Using information on past demands, and a model for the effect of various demand management strategies, estimate the probabilities of different demand levels, dependent of the supply level, and estimate the joint probabilities of supply and demand.
- Using flow records, possibly extended by paleo-hydrological data, estimate extreme (high or low) values of flow and river stage.

The above procedures are relevant for cases in which failures are caused by deviations around an expected value. A different approach is used when one is to design for an event of a prescribed return period or frequency of occurrence. The procedure followed in this case is:

- Calculate the magnitude of the event of the given return period.

When the return period does not exceed the length of the historical record, the magnitude of the design event can be computed with some confidence. An example is the five-year flow required for minor drainage facilities. The more difficult cases are when the return period is very long, and the estimate of the computed event is highly uncertain. This is the case with the PMF, or even the 100-year flood. Klemes (1996) expresses severe criticism of the procedures for estimation of the magnitude of very rare hydrological events, principally the design flood. He does so with considerable sarcasm, in reference to the behaviour of old-time bureaucracies.

Design events should actually be selected according to the same procedure as any other analysis of risk. But because the probabilities of such events are highly uncertain (Klemes, 1996), and the consequences very severe, the preferred procedure is to set the return period, arbitrarily, and put all the emphasis on computing the magnitude of the corresponding event. Logical questions may be posed, such as:

- What are the actual expected consequences and damages which would result from the event whose return period has been prescribed.
- What is the sensitivity of the damage to variations in the return period.
- What is the sensitivity of the cost of the system to variations in the return period.

Figure 1 shows schematically the cost of a system versus its level of reliability, which is an expression of the magnitude of the rare event it is designed to sustain. The shape of the curve is typical. At low values of reliability there is a steep part, where sizeable increments of the system are required in order to achieve the basic level of reliability. Then there is a flatter portion of the curve, where a modest increase in cost achieves a large increment of reliability. As higher levels of reliability, the curve bends upward, indicating that the same increment of reliability costs more and more. When perfect reliability is approached (obviously, no perfectly reliable is possible, although this might be stated as a goal) the cost of the system soars. It is reasonable to expect that the optimal level of reliability lies in the area of where the curve bends sharply upwards. If the damages due to failure are not, or cannot be, computed, the cost versus-reliability curve may be a good device for aiding decision making. The marginal cost of added reliability can be used to guide decision makers to selecting the solution.

Physical consequences of failure modes, and the associated damages and losses

The consequences are first measured in physical terms, such as the amount of water not supplied, the number of hours customers go without water, the area inundated, the number of people evacuated, the number of lives lost. The next step, which is considerably more difficult, is conversion of these consequences into "damage values".

Valuation of risk

The risk is expressed as a product of the damage times its probability of occurrence. Assessing the damage is one of the most difficult phases of risk management. In this respect, it is instructive to differentiate between different types of failures. Let us return to the example of the design of a water supply reservoir.

Damages due to shortages in supply of water to production systems, be they agriculture or industry, can be estimated as the loss of production value. While there may be intangible components, such as loss of market position and loss of reputation, one can still turn to the market place and to rulings of the courts (with all their failings in reflecting true values!) to determine the value of shortage, or at least some estimate thereof.

When the shortage is to the domestic sector, the damage is less obvious. It is common to assume that domestic consumption is rigid and must be met with very high reliability. In reality, the reliability required may be too high, since there is more elasticity in urban water consumption than often assumed.

Options for mitigating each failure mode, and their cost

Examples:

- Sizing the reservoir for higher supply reliability.
- Adding capacity to the treatment plant.
- Adding redundancy in the distribution system.
- Designing the dam to withstand a larger flow.
- Using demand management options, to reduce consumption when there is a shortfall in supplies.

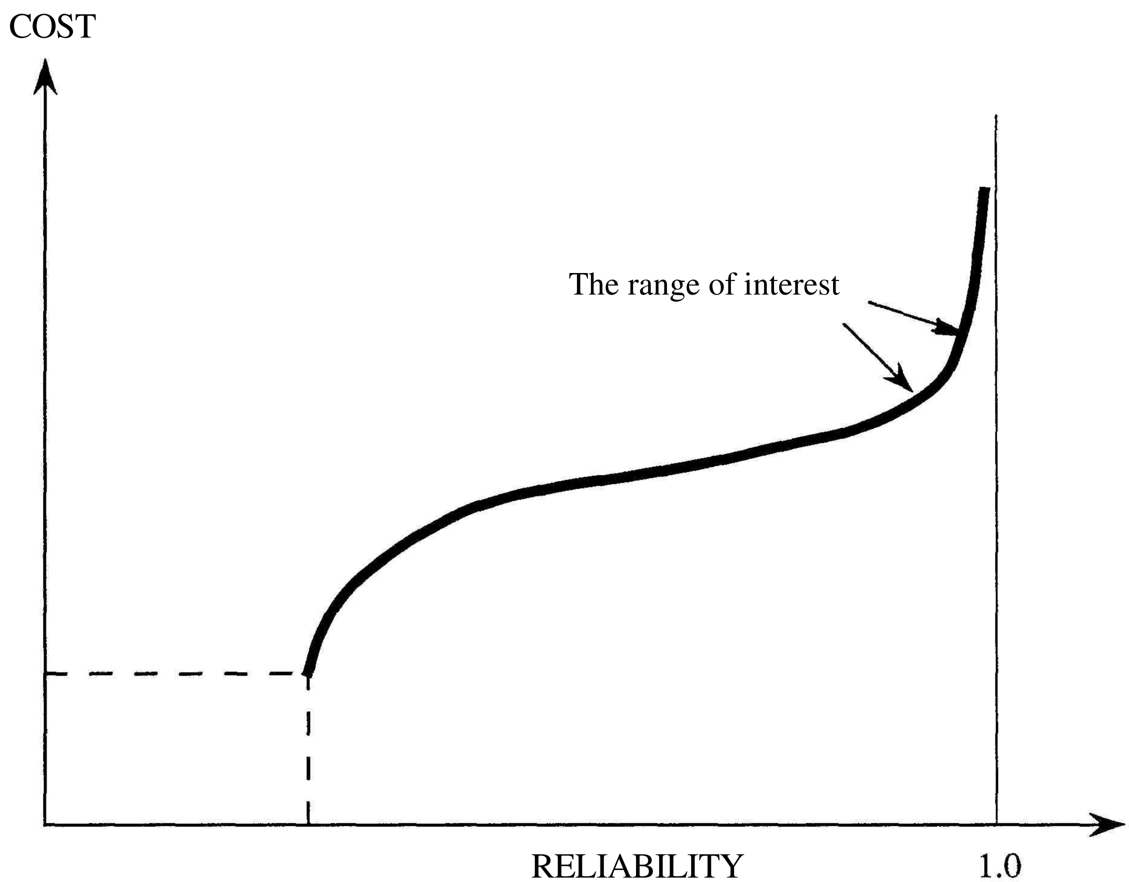

FIGURE 1: SELECTION OF THE DESIGN RELIABILITY

ACCORDING TO THE COST OF RELIABILITY

Synthesize all of the above into a framework for management of the water resources system, and select the optimal policy

To fix ideas, consider the following case. A reservoir of size **D** is to be constructed on a river, for water supply. The annual flow in the river, q, which will be the inflow to the reservoir, has been measured over a period of years. A probability density function $\mathbf{f_Q(q)}$ has been fitted to the data. The central part of $\mathbf{f_Q(q)}$ may be known with an acceptable degree of confidence. It is based on measured data, possibly extended through hydrologic simulation, with a longer series of meteorological data. The tails of the distribution have a much greater degree of uncertainty. The tail to the left may be known with greater confidence, but the one to the extreme right is always highly uncertain.

The PDF of the inflow and the benefit/loss function are shown in Figure 2.

The net benefit from water supply is denoted **B[q|D].** It is a function of the inflow, **q**, and depends on the size of the reservoir, **D**. The unconditional benefit function, **B[D]** lies above the curve **B[q|D],** tangent to it at the point $\mathbf{q = q_D}$, where the benefit is $\mathbf{B[D] — B[q_D|D]}$**.**

In examining the functions $\mathbf{f_Q(q)}$ and **B[q|D]** we can observe several ranges, as follows:

$\mathbf{0 < q < q_1}$

This is the left tail of $\mathbf{f_Q(q)}$, the region of low flows, droughts and shortages. Reliability of the PDF in this range is low, but may not be too bad, since the record usually does contain some low values, and, at least, there is a low limit (zero) to the possible values. For water supply, this is the most critical range of the possible outcomes.

The actual value of **B[q|D]** in this range (which moves rapidly from benefits to losses, as q approaches zero) is difficult to forecast, because it depends on the recourse taken by consumers at the time a shortage occurs. The actual damage to consumers may be a lot lower than frequently assumed. For example, urban water demands are usually treated as fixed and inflexible, signaling a very high loss due to shortage. In reality, however, urban consumers adjust to shortages, when they occur as a result of natural causes (not so much when they are due to human error or mismanagement!), without much real damage.

The cost which accrues to the water supply utility due to shortage may be much greater than the actual real losses to individual consumers and to society as a whole. They are the result of claims made through the judicial system and pressures exerted through the political system.

Hence, the value of **B[q|D]** which actually materializes may be substantially different from that assumed a-priori, through an economic analysis.

The above is relevant for the planning and design phases. In real-time operation, the situation is somewhat different. Operators tend to assign a much more dramatic drop in **B[q|D]** when shortages occur than would be justified by a strict economic evaluation. This is because the operators' "personal damage function" is highly non-symmetric: it drops sharply when shortages occur, but does not rise when an operator does a good (i.e., efficient) job. Operators are fired for shortages; they are rarely awarded for having saved some money or water. This is why operators tend to be conservative in their actions.

$q_1 < q < q_2$

This is the central body of the PDF. Deviations from the expected inflows and from the expected performance are relatively small. The expected value criterion is a good measure for evaluating the design as well as the operation.

$q_2 < q$

The PDF $\mathbf{f_Q(q)}$ is not as reliable in this range, but can still be reasonably based on historical data. The benefit function, **B[q|D],** remains close to its design value, possibly with some increase (or decrease) from the design value **B[q(D) | D].** As **q** increases further above the limit $\mathbf{q_2}$, the reliability of the PDF decreases, and **B[q | D]** begins to drop. The strength of the expected value criterion decreases.

$q_2 << q$

As **q** grows further beyond $\mathbf{q_2}$, reliability of the statistical calculations drops, as does the reliability of the benefit function **B[q|D].** This range is frequently ignored in the computation of **EV{B[q|D]}.**

$q \longrightarrow q_{max}$

$\mathbf{q_{max}}$ is the extreme value for which the system must be designed; it is the "load" which the system has to withstand without failing in a catastrophic mode. The damage due to the occurrence of $\mathbf{q_{max}}$ is not included in the computation of the benefit (cost) of the project. The implicit value assigned to $\mathbf{B[q_{max}|D]}$ is essentially (negative) infinite, and it does not figure in the economic valuation of the decision.

On the other hand, the cost of the project or facility is highly dependent on the value selected for $\mathbf{q_{max}}$. For our example, this is the dam design for the PMF (or some fraction thereof). The use of the expected value of benefit/cost is not appropriate, since it would include the product of a very large (uncertain) consequence by a very low (difficult to estimate) probability. The product in this case has little significance.

Nevertheless, there is great value in attempting to assign values to the consequence of the extreme event and to its probability. Critics warn that attempting to assign probabilities to such rare events is folly, in the statistical and practical sense. They must, however, suggest another way for making decisions.

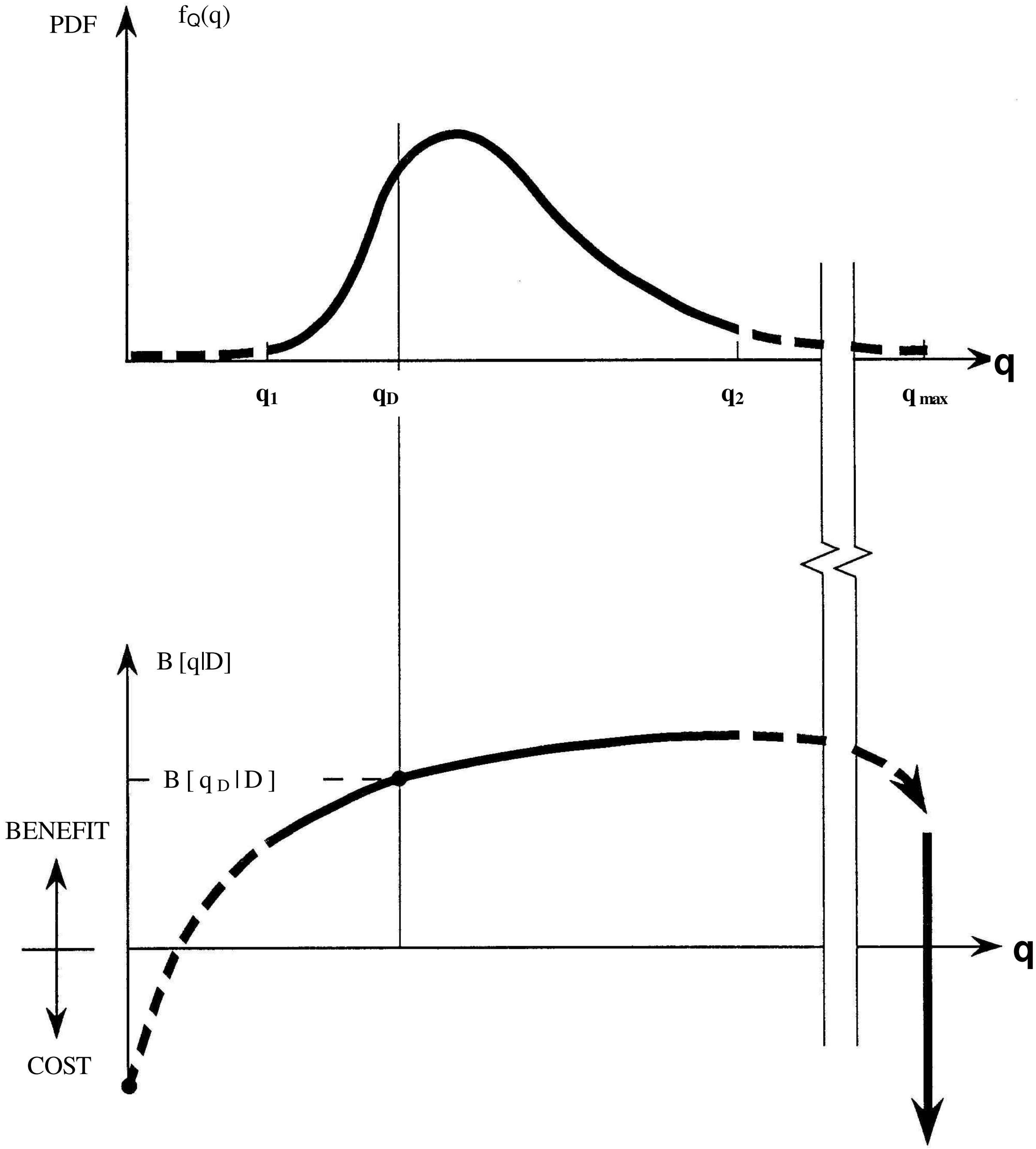

FIGURE 2: BENEFIT FUNCTION FOR A STOCHASTIC INFLOW

One way to estimate the magnitude of the extreme event, without necessarily assigning it a probability, is to use a physically based approach. This is what is done with the PMF. A PMP is estimated, on the basis of the most extreme conditions which can be reached in the atmosphere, supported by transportation of selected observed extreme storms to the watershed in question. When snowmelt is significant, a temperature profile is selected on the basis of historical data, to melt the snowpack. The runoff from this PMP and snowmelt is then computed, with the initial conditions on the watershed assumed to be those which will produce the largest flows.

Once the PMF has been computed, it is instructive to see where it falls relative to the extrapolated PDF of extreme flows. A degree of confidence can be achieved if the two approaches are convergent. Continuous hydrological simulation is a better way to approach the estimation of extreme flows than a single event calculation.

The role of forecasting

Forecasting is used to modify the a-priori probability distribution of future time series: hydrological data and of demands. Figure 3 indicates its two phases, identified as "forecast" and "possible futures", in real-time forecasting.

The first phase extends from time NOW over some period, during which future flows can be viewed in a sense as a projection from the current state, using data from the immediate past: river flows upstream, precipitation in the recent past, snow pack, temperatures, demands. Data used in this phase can extend from some time in the past all the way to time NOW. However, often there is a gap between measurement of the data and its availability for the forecasting model.

Hydrological forecasting is carried out with a watershed simulation model. The model tracks the state of the watershed, and keeps a running record of such variables as soil moisture, groundwater levels, snow pack, and reservoir levels. The model is run every time new data become available. It also uses records and forecasts of the relevant meteorological variables, such as temperatures and precipitation, to simulate the response of the watershed and compute the inflows into the reservoirs. As shown in Figure 3, there is a confidence band of the forecasted inflow, whose width increases with time.

The period over which this forecast is useful depends on the size and response of the watershed, and the quality of the meteorological forecast. Typically, the larger the watershed, the longer the dependable "forecast period". It can range from hours (something in the order of 72 hours) for a small to medium size watershed, to several days or even a few weeks on a large river basin.

The reliability of this forecast deteriorates as time progresses, and we move into the second phase. Here, all "possible futures" are equally likely. Each starts from the final state of me first phase forecast (which is itself a random variable, as seen in Figure 3) and represents the inflows

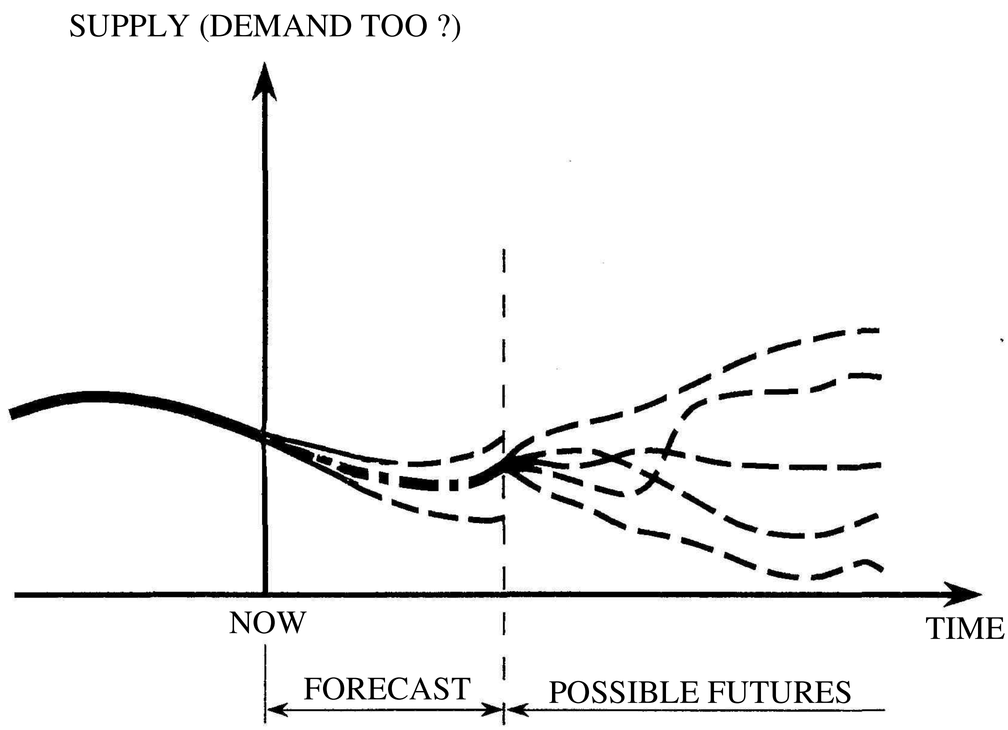

FIGURE 3: THE ROLE OF FORECASTS

which would have been experienced if one particular year of historical meteorological data were to occur from that date on.

These trajectories can be used to estimate the probability distribution of cumulative inflows to any selected point in future time, thus providing the data needed for analyzing the performance of the water supply system. Given this forecast, the operator and decision maker have an improved basis for evaluating alternative management policies.

Thus real-time forecasting has an important effect on operational reliability of the water resource system. But the capacity for real-time forecasting to improve system reliability should also be figured as an element in the planning and design phases. To demonstrate, let us look at the design of dam safety for the PMF.

The PMF is calculated as an accumulation of all possible worst-case conditions: the largest storm possible is dropped on a saturated watershed, at the same time a large snowmelt occurs. This results in the largest flow deemed possible, for return period of hundreds to thousands of years (depending on whether lives are in danger). The PMF is also used to dictate the freeboard for the relevant season of year.

If the operator has access to reliable measured data and a good weather forecast, backed by a calibrated and tested watershed model, he may be able delay vacating space in the reservoir in anticipation of a storm. Soil moisture may not be as high, the snow pack not as deep, the forecasted storm not as severe, the forecasted temperature not as high -- as used in the PMF calculations. Thus a good forecast provides extra reliability, or, alternatively, an opportunity for less wasteful operation with the same level of safety.

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