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**HYDROLOGIC MODELING**

by

Ven Te Chow\*

(The Seventh John R. Freeman Memorial Lecture, presented before the Boston Society of Civil Engineers, 17 February 1972.)

**Introduction**

During the early years in hydraulic engineering practice, say prior to sixty years ago, a spillway designed to pass flood 50 to 100 percent larger than the largest which had occurred in a period of record as long as 25 years, was considered adequate. This design criterion is essentially a rule of thumb which involves an arbitrary factor of safety. The concept of factor of safety may be applicable to other engineering designs, but it was soon found out that this practice in hydraulic design is after all entirely inadequate. As one example of absurdities to this criterion, the Republican River in Nebraska was noted to experience a flood in 1935 which was over ten times as large as had ever occurred on that river during 40 prior years of record.

Practicing hydrologists and hydraulic engineers then began to search for better methods of hydrologic design. In 1887, for example, Professor Arthur Talbot of the University of Illinois derived the well-known Talbot formula to determine waterway areas.<sup>1</sup> In 1889, the city engineer of Rochester, New York, Mr. Emil Kuichling, devised the popular rational formula for the computation of peak discharges from urban watersheds.<sup>2</sup> In 1921, Allen Hazen, a consulting engineer, suggested the logarithmic probability analysis of flood data.<sup>3</sup> In 1930, a committee of the Boston Society of Civil Engineers<sup>4</sup> reported that flood hydrographs afford the best basis for the study of drainage areas and concluded that "the base of the flood hydrograph appears to be approximately constant for different floods" and "peak flows tend to vary directly with the total volume of flood runoff." Two years later, another consulting engineer Leroy Sherman proposed the theory of unit hydrographs<sup>5</sup>, which is practically based on the conclusions of the BSCE committee.

In recent years, a host of mathematically sophisticated methods of hydrologic analysis have been developed. All these methods and those proposed in the earlier years for hydrologic design are essentially techniques of hydrologic

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modeling. Most natural hydrologic phenomena are so complex that they are beyond human comprehension, or that exact laws governing such phenomena have not yet been fully discovered. Before such laws can ever be found, complicated hydrologic phenomena (the prototypes) can only be approximated by modeling.

### Type of Hydrologic Models

It has been said that modeling is the process of approximating the prototype for the purpose of evaluating the performance of the prototype. Since the prototypes under consideration are more complex than models can ever be, certain simplifying modeling assumptions must be made in order to provide the model with a presentable or workable form. According to such assumptions, hydrologic models may be classified into various types for the sake of understanding.

Hydrologic models can be divided into two basic categories: models that possess certain physical properties of their prototypes; and models that have only an abstract form. The former category, or the physical models, can be divided into scale models (iconic models), analog models, and simulation models.

A scale model that looks like the prototype is the simplest type. It is exemplified by ordinary hydraulic models of rivers and structures that are investigated in many hydraulic laboratories, and whose scales are based on geometric and force considerations.<sup>6</sup>

An analog model replaces prototype properties with quantities that bear the same relations to each other as do those of the prototype, but they are easier to measure or visualize. For example, the Hele-Shaw model shows the movement of a viscous liquid between two closely spaced parallel plates is analogous to seepage flow in a two-dimensional cross-section of an aquifer.<sup>7</sup> Many electronic analog models for surface and groundwater flows are built on the principle of analogy between the flow of water and the flow of electrical current.<sup>8</sup>

A simulation model retains the essence of the prototype without actually attaining reality itself. It reproduces the behavior of a hydrologic phenomenon in every important detail but does not reproduce the phenomenon itself. In a broad sense, it is commonly used to include the scale and analog models, but the definition adopted here refers specifically to the simulation on digital computers. In hydrology, the Stanford Watershed Model<sup>9</sup> may be therefore described as a simulation model. This model simulates the land phase of the hydrologic cycle in a watershed on a digital computer.

Abstract models, or the second basic category, are generally referred to as theoretical, or mathematical, models since they attempt to represent the prototype theoretically in a mathematical form. These models neither resemble nor imitate prototypes physically but replace the relevant features of the system

by a set of mathematical relationships. According to certainty, or uncertainty, of such relationships, on *a priori* basis, the models can be further divided into deterministic and indeterministic types. A differentiation between deterministic and indeterministic models can be assisted by relating them to the concepts of certainty and uncertainty. Certainty implies that no matter how many times a hydrologic phenomenon is processed under a given set of invariant conditions the same outcome is assumed to result always. On the other hand, uncertainty implies that every time a phenomenon is produced it may be different. Theoretically, certainties may be forecasted while the risk aspect of uncertainties can be predicted with an element of probability. In this sense, therefore, deterministic models make forecastings, while indeterministic models make predications.

Abstract models are the product of modern age, since these quantitative models must depend on adequate mathematical tools which have now become available for practical applications. Such models to be useful must inevitably be complex, yet at the same time be workable. These requirements could not be compromised without the availability of high-speed computers for the solution of the models. Since numerous abstract models have been developed lately and they constitute a significant advance in modern hydrology, this paper will emphasize discussions on these models. Major types of these models as well as the physical models are classified diagrammatically in Fig. 1.

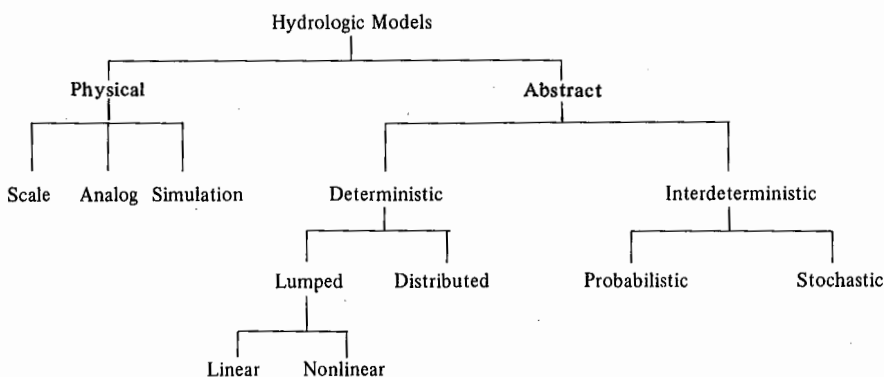


Fig. 1. Classification of Hydrologic Models

In abstract modeling, hydrologic phenomena are treated as systems. By this so-called systems concept, the hydrologic system is considered to consist of an input, an output, and some working medium known as throughput such as the water passing through the system. For example, a watershed can be analyzed as a system. For this system, the input is the rainfall and groundwater inflow; the

output is the evapotranspiration, infiltration, and runoff; and the throughput is the water moving over the watershed. By systems concept, a hydrologic phenomenon can be readily interpreted by modern system-analysis techniques, and then modeled mathematically and solved on computers.

Mathematically, the input and output relationship of a hydrologic system may be represented by

$$Q = \phi I \quad (1)$$

in which  $Q$  is the output,  $I$  is the input, and  $\phi$  is a transfer function which represents the operation performed by the system on the input to transform it into output. For example, the unit hydrograph is a transfer function of the watershed system. It should be noted that input  $I$  and output  $Q$  are time functions and can be also expressed by  $I(t)$  and  $Q(t)$ , respectively, with  $t$  denoting time. The objective of modeling is essentially to derive a mathematical formulation for the transfer function of the system.

### Deterministic Modeling

In deterministic modeling, a hydrologic system is often treated either as lumped or as distributed, although this treatment is equally applicable to indeterministic modeling. A lumped-system model is a gross representation of the hydrologic system as determined from the input and output data pertaining to the system, thus the system is regarded as a single point in space without dimensions. In contrast to this is the distributed-system model which considers the hydrologic processes that are taking place within various distributed points or areas within the internal space of the system. If the internal space is divided into a number of small unit spaces and each unit space is modeled as a lumped system, then the distributed-system model becomes simply a conglomeration of lumped-system models. For demonstrative purposes, each of the two system models may be exemplified as follows:

#### A. A Lumped-System Model

This example describes a general deterministic model.<sup>10</sup> By applying the basic concept of system continuity to a hydrologic system, the following continuity equation may be written:

$$I - Q = S \quad (2)$$



in which  $S$  is the storage of water in the system and thus  $\dot{S}$  is the first derivative of  $S$  with respect to time, or  $\dot{S} = dS/dt$ . The system storage is affected by the throughput and reflects the characteristics of the system. For given physiographic features of a hydrologic system, the storage is likely a function of the input and output and their changes. Therefore, the system storage may be expressed as a mathematical function of the input  $I$  and output  $Q$  and their derivatives with respect to time, or

$$S = f(I, \dot{I}, \ddot{I}, \dots, I^{(m)}, Q, \dot{Q}, \ddot{Q}, \dots, Q^{(n)}) = f \tag{3}$$

in which  $\dot{I}, \ddot{I}, \dots, I^{(m)}$  are the first, second,  $\dots$ ,  $m$ -th derivatives of  $I$ ; and  $\dot{Q}, \ddot{Q}, \dots, Q^{(n)}$  are the first, second,  $\dots$ ,  $n$ -th derivatives of  $Q$ . Because  $I$ , and  $Q$  are time functions,  $S$  is also a function of time.

If the input  $I$  continues steadily, the output,  $Q$ , will increase and finally approach a steady state at which  $I = I_*$ ,  $Q \rightarrow Q_*$  and all derivatives of  $I$  and  $Q$  with respect to time will approach zero. The subscript  $*$  indicates the steady state. At the steady state, the storage may be assumed essentially proportional directly to  $I$  and  $Q$ ; thus, Eq. 3 becomes

$$S_* = f(I_*, Q_*) = aI_* + bQ_* \tag{4}$$

in which  $a$  and  $b$  are coefficients.

Expanding Eq. 4 in Taylor's series about the steady state<sup>11</sup> yields

$$S = \sum_{m=0}^m a_m(I, Q) I + \sum_{n=0}^n b_n(I, Q) Q \tag{5}$$

where  $a_m(I, Q) = \partial f / \partial I$  and  $b_n(I, Q) = \partial f / \partial Q$ . Because these coefficients  $a_m$  and  $b_n$  are functions of  $I$  or  $Q$ , or both, the above differential equation is nonlinear. If the coefficients are constant or independent of  $I$  and  $Q$ , then the equation becomes linear and its solution will be greatly simplified, leading to a linear model. For practical purposes, the coefficients may be assumed as functions of certain characteristic values of  $I$  and  $Q$ , such as the average input  $\bar{I}$  and average

output  $\bar{Q}$ , of the peak input  $I_p$  and peak output  $Q_p$ . Thus, for a particular hydrologic event, the coefficients are constants and the equation is linear and can be readily solved for the case of that event. However, the characteristic values of  $I$  and  $Q$  obviously vary from event to event. After the equation is solved linearly for a particular event, the system storage can be then considered as nonlinear by expressing the coefficients as functions of the characteristic values of  $I$  and  $Q$ .

Substituting Eq. 5 for  $S$  in Eq. 2 and solving for  $Q$ , a system equation containing a transfer function in the form of Eq. 1 may be written as

$$Q = \left( \frac{a_m D^{m+1} + a_{m-1} D^m + \dots + a_0 D - 1}{b_n D^{n+1} + b_{n-1} D^n + \dots + b_0 D + 1} \right) I \quad (6)$$

in which  $D^m = d^m/dt^m$ ;  $D^n = d^n/dt^n$ ; etc. It can be seen that the transfer function is in the form of a fraction in which the numerator and the denominator are polynomials that may be denoted by  $M(D)$  and  $N(D)$ , respectively. Thus, Eq. 6 is written as

$$Q = \frac{M(D)}{N(D)} I \quad (7)$$

Eq. 6 or 7 is the general lumped-system hydrologic model. It can be shown that many mathematical models that have been proposed are special cases of this general model.

For example, let  $M(D) = -a_0 D + 1$  and  $N(D) = b_0 D + 1$ . Then simplifying Eq. 7 and using Eq. 2, it can be shown that

$$S = a_0 I + b_0 Q \quad (8)$$

which is the well-known Muskingum equation or the linear model used in the Muskingum flood routing method.<sup>1,2</sup>

Now, let  $N(D) = 1$  and  $a_{j-1} = (-1)^{j-1} C^j/j!$  with  $j = 1, 2, \dots, m+1$ , where  $C$  is a constant. Then, using Taylor's expansion of  $I(t-C)$  about  $t$ , it can be shown that

$$M(D) I(t) = I(t-C) \quad (9)$$

Thus, Eq. 7 becomes

$$Q(t) = I(t-C) \quad (10)$$

which is the mathematical expression for the well-known linear channel model.<sup>13</sup>

Now, let  $M(D) = 1$  and  $N(D)$  has a root of  $1/K$  or  $N(D) = 1 - KD$ . Then, Eqs. 7 and 2 yield

$$S = KQ \quad (11)$$

which is the mathematical expression for the well-known linear reservoir model.<sup>13</sup> In a similar manner, it can be shown further that if  $M(D) = 1$  and  $N(D)$  has  $n$  real roots of  $1/K_1, 1/K_2, \dots, 1/K_n$ ; Eq. 7 will result in the Nash model representing a series of linear reservoirs.<sup>14</sup>

For the application of Eq. 6 or 7 to practical problems it is necessary to know the values of  $m$  and  $n$ ; that is, the highest orders, respectively, of the derivatives of  $I$  and  $Q$ . In general, the values depend on the problem under consideration. In order to determine the most suitable values of  $m$  and  $n$  for flood studies, various values  $m$  and  $n$  have been assumed and hydrologic data were then fitted to Eq. 5. By comparing the storage values computed by different fitted forms of Eq. 5 for various values of  $m$  and  $n$ , with the corresponding actual storage values computed directly from the hydrologic data, it was found that the most suitable values of  $m$  and  $n$  are, respectively, 1 and 2. Derivatives of  $I$  and  $Q$  of higher order were found to be insignificant and they can be dropped without causing appreciable errors in fitting the model. For practical application, therefore, the following simplified form of Eqs. 5 and 6 may be used, assuming constant coefficients:

$$S = a_0 I + a_1 \dot{I} + b_0 Q + b_1 \dot{Q} + b_2 \ddot{Q} \quad (12)$$

and

$$Q = \left( \frac{1 - a_0 D - a_1 D^2}{1 + b_0 D + b_1 D^2 + b_2 D^3} \right) I \quad (13)$$

Using the above lumped-system model to fit the storage, effective rainfall and direct runoff for the storm of April 4-9, 1941 on Wills Creek watershed, near Cumberland, Maryland, the results are shown in Fig. 2.

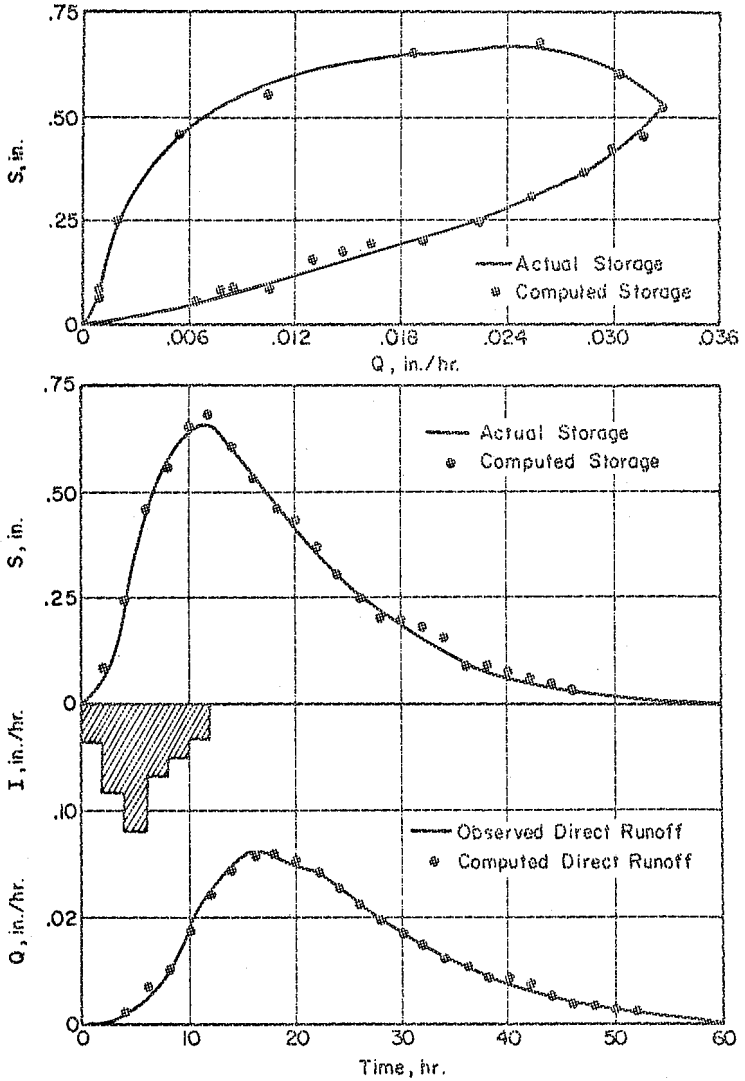


Fig. 2 Storage, effective rainfall and direct runoff graphs for storm of April 4-5, 1941 on Wills Creek watershed

## B. A Distributed-System Model

As the distributed-system model considers the internal space of the system, its mathematical formulation must contain space dimensions or coordinates. The simplest distributed-system model to describe the flow of surface water over a watershed is the application of the kinematic-wave theory<sup>15</sup> resulting in a one-space-dimensional model<sup>16</sup>. In fact, such a model has been applied to Mekong Delta flood studies.<sup>17</sup> The example to be given here is a two-space-dimensional model for the surface flow over a watershed due to rainfall.<sup>18</sup> The physical configuration of the watershed model is impervious and rectangular. It consists of two equal-sized rectangular plane surfaces for accommodating overland flows, which intersect to form a trough in the middle of the watershed for serving the main channel flow. The overland flows are assumed to travel in a direction normal to the line of intersection of the two side planes toward the channel flow. The watershed flow thus consists of two flow components, namely the overland flows and the channel flow. Since the watershed is assumed impervious, the watershed flow involves only the surface flow. The influence of the raindrop impact is assumed as an overpressure head added to the hydrostatic pressure head. The surface roughness is represented by the Darcy-Weisbach resistance factor. The distributed-system model of the watershed flow is a mathematical and hydrodynamic formulation considering the curvilinear nature of the streamlines of two dimensions on the watershed surface; while the third dimension, or the depth of flow, is taken as the dependent variable. The velocity and pressure are averaged over a vertical depth of flow and across the flow cross-sectional area. For this model, the unsteady spatially varied equations of continuity and momentum are normalized in order to reduce the equations to dimensionless forms and to save computation effort. The watershed length and the critical values of depth and velocity for the equilibrium discharge at the outlet are used as the terms of normalization. Thus, in a vector form, the normalized hydrodynamic equations to represent the watershed model are as follows:

$$\frac{\partial U}{\partial t} + \frac{\partial F}{\partial x} + \frac{\partial G}{\partial z} = H \quad (14)$$

$$\text{where } U = \{y; q; p\}$$

$$F = \{q; q^2/y + P_1y^2; qp/y\}$$

$$G = \{p; qp/y; p^2y + P_1y^2\}$$

$$H = \{I; P_2y - P_4(f + f^*)q(q^2 + p^2)^{0.5}y^{-2} + VI;$$

$$P_3y - P_4(f + f^*)p(q^2 + p^2)^{0.5}y^{-2} + P_5VI\}$$

in which  $y$  = normalized depth of flow;  $q$  = normalized discharge in the main channel direction;  $p$  = normalized discharge in the lateral direction;  $I$  = normalized rainfall intensity;  $V$  = normalized vertical raindrop velocity;  $f$  = Darcy-Weisbach resistance coefficient;  $f^*$  = apparent resistance coefficient due to raindrop impact;  $P_1 = (1 - \sin^2\theta - \sin^2\Psi)^{0.5}/2 \cos\theta$ ;  $P_2 = I \tan\theta/y_c$ ;  $P_3 = P_2P_5$ ;  $P_4 = L/8y_c$ ; and  $P_5 = \sin\Psi/\sin\theta$ . In addition,  $\theta$  = watershed slope angle in the main channel direction;  $\Psi$  = watershed slope angle in the lateral direction;  $L$  = watershed length in the main channel direction; and  $y_c$  = critical depth for the equilibrium discharge at watershed outlet.

To examine the nature of Eq. 14, the following two matrices are defined:

$$A = \frac{dF}{dU} = \begin{vmatrix} 0 & 1 & 0 \\ a_2^2 - 2a_1a_2 & 2a_1 & 0 \\ -a_1\beta_1 & \beta_1 & a_1 \end{vmatrix}$$

$$B = \frac{dG}{dU} = \begin{vmatrix} 0 & 0 & 1 \\ -a_1\beta_1 & \beta_1 & a_1 \\ \beta_2^2 - 2\beta_1\beta_2 & 0 & 2\beta_1 \end{vmatrix}$$

where  $a_1 = g/y$ ,  $a_2 = q/y + (2P_1y)^{0.5}$  and  $a_3 = q/y - (2P_1y)^{0.5}$ , all being the eigenvalues of A; and  $\beta_1 = p/y$ ,  $\beta_2 = p/y + (2P_1y)^{0.5}$  and  $\beta_3 = p/y - (2P_1y)^{1/2}$ , all being the eigenvalues of B. All these eigenvalues are real and distinct, but they are neither positive definite, nor negative definite. If all three eigenvalues of either A or B were positive, or negative, definite, Eq. 14 would be of elliptic type. If they were definite but had different signs, Eq. 14 would be of hyperbolic type.<sup>19</sup> In the proposed watershed model, the eigenvalues of A and B do not fit these cases; thus Eq. 14 is neither elliptic nor hyperbolic although it is quadratic.

Eq. 14 can be solved on computer by a four-step scheme of numerical integration combining the concepts of Lax-Wendroff,<sup>20</sup> Burstein<sup>21</sup> and Lapidus.<sup>22</sup> This scheme is to express Eq. 14 in a second-order accurate difference form, which includes an artificial viscosity term that sharpens discontinuities but has no significant effect on the continuity of streamlines and the accuracy of the results.

In applying the proposed scheme to the solution of the watershed flow model, the following major assumptions are made: The falling raindrops are sufficiently dense so that their mass and momentum flux can be considered continuous around any grid point of the computer solution. The effect of raindrop impact on the surface flow is taken as an addition to the Darcy-Weisbach resistance factor. The watershed flow is not affected by surface tension and has a momentum coefficient equal to one. Its initial depth and velocity profiles of the flow are given, and there is no flow across any boundary except the outlet. Fig. 3 shows the profiles of depth and velocity vector on half of a square watershed, a typical computer output of the distributed-system model.

### Indeterministic Models

Indeterministic behavior of hydrologic phenomena may be described in many ways. One tangible approach is to hypothesize the risk in uncertainty as definable by an element of probability. In fact, this does not imply that all uncertainties can be measured in terms of probability. On the basis of this understanding, a simple modeling concept can be taken by assuming that hydrologic events are purely random variables. In this way, hydrologic data have been analyzed by many mathematical models of probability distribution. Among the commonly used such probability models are the lognormal distribution,<sup>23</sup> Gumbel's extreme-value distribution,<sup>24</sup> and the log-Pearson Type III distribution recently recommended by the Water Resources Council.<sup>25</sup>

However, the concept of prediction implied in the indeterministic model is more than one of pure randomness since the occurrence of hydrologic event may be affected by its antecedent event or events. In fact, it has been discovered that

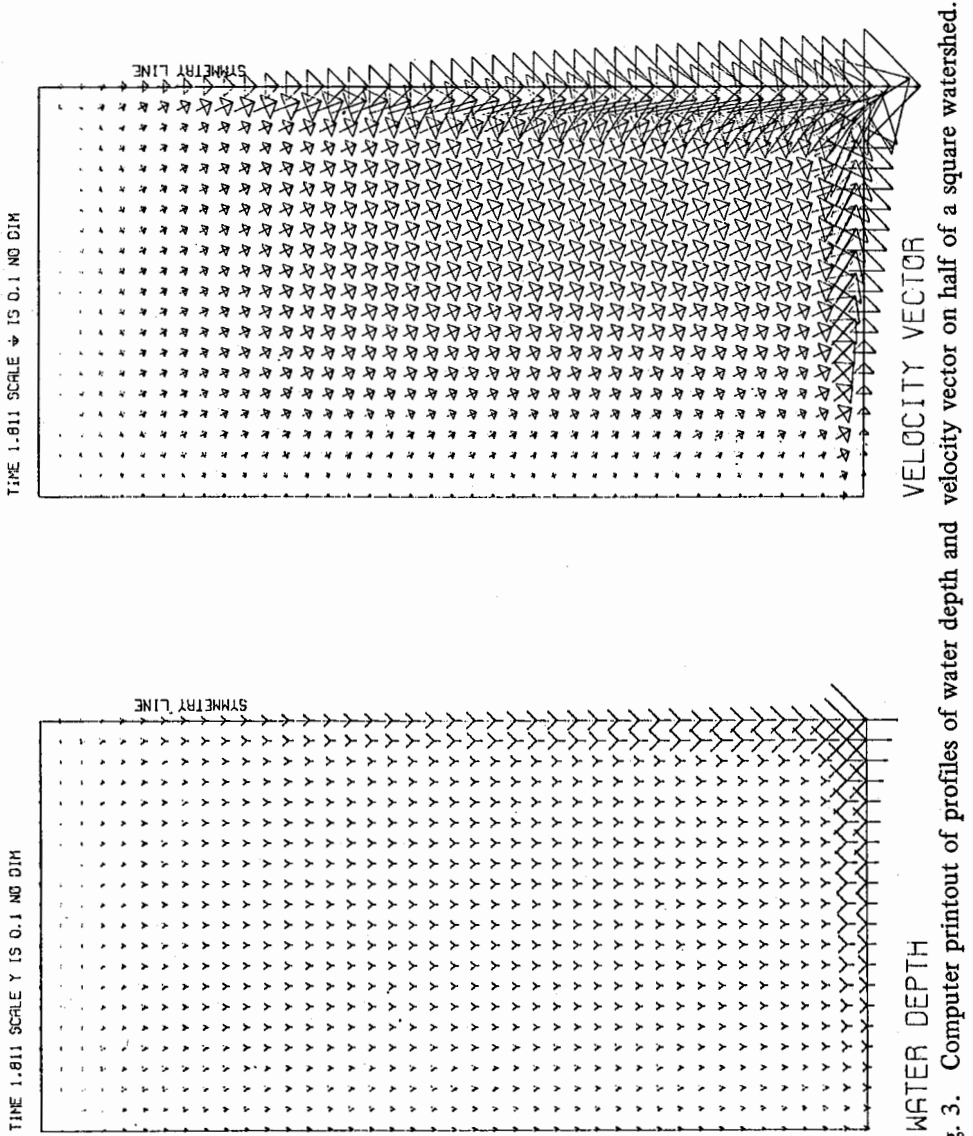


Fig. 3. Computer printout of profiles of water depth and velocity vector on half of a square watershed.



the variability of groups of recorded streamflows in their natural order of occurrence is actually larger than if the same flows occurred in random sequence.<sup>26</sup> This means that hydrologic events do not usually occur in random sequences. By assuming hydrologic events as pure random variables is simply to ignore the effect of sequence since the variables may occur in different sequences but not in a random fashion. In order to cope with this situation, the hydrologic process may be treated as stochastic processes. For example, the record of a hydrologic phenomenon may be analyzed as a time series and thus, available mathematical models of time series can be used as stochastic models to represent the hydrologic process involved. The theoretical meaning of stochastic process may be further elaborated.<sup>27</sup>

A probability distribution is the distribution of a random variable whose specific value cannot be predicted exactly except in terms of chances. Mathematically speaking, a random variable  $X(\omega)$ ,  $\omega \in \Omega$  describable by a sample point  $\omega$  is a function defined on the sample space  $\Omega$  of an experiment such that for every real number of the variate  $x$  there exists a probability  $P[\omega; X(\omega) \leq x]$ . Once the probability function is formulated for a given problem, it is independent of when or where it occurs except under either a given or an average condition. However, a random variable may have a different probability distribution for each point on the time scale, or for each point on the space coordinates. Because of their dependence on time or space and because there can be, at least in theory, an infinite number of them, these families of random variables constitute the so-called stochastic process. Thus, a stochastic process is a family of random variables  $[X(t, s); t \in T, s \in S]$  that depend on a parameter or parameters which belong to an indexing set  $T$  or  $S$ , or indexing sets  $T$  and  $S$ , of time  $t$  or space  $s$ , or both.

In a way, it can be seen that the deterministic process and the "purely" probabilistic process are only two special cases of the stochastic process. When the probability or certainty of the random variable is one, the stochastic process simply reduces to deterministic. When the probability is independent of any parameter index, time or space, and the family of random variables belongs to the same population, the stochastic process becomes purely probabilistic, in which no deterministic components exist. On a scale of probability from 0 to 1, the purely probabilistic and the deterministic cases occupy respectively the two extremities, while the stochastic process may occur anywhere between them. Take, for instance, the simple first-order Markov chain model which is a stochastic model. This model consists of two terms; namely, the trend term and the noise term. For the special cases, the noise term may be zero, thus producing a deterministic model; or the trend term may be zero, then resulting in a probabilistic model.

Today, stochastic modeling is at the highest level of hydrologic modeling, although it has not been well developed in view of many practical difficulties yet to be overcome. By stochastic modeling, all components of a hydrologic system can be theoretically described by stochastic processes.<sup>28-32</sup> In the system, the input, the output and the transformation of input to output in the form of throughput passing through the system may be therefore represented mathematically by time series since these component processes, in general, change with time and are functions of time. The transformation of input to output is characterized by the physical features and hydrologic behavior of the system. All the processes are assumed to be governed by mathematically simulated stochastic laws. They may be denoted by  $[u_t; t \in T]$  where  $u_t$  is a stochastic variable at time  $t$  which is a parameter running over an index set  $T$  or over the time range under consideration. Thus, the input stochastic process is denoted by  $[X_t; t \in T]$  where  $X_t$  is the input stochastic variable; the output stochastic process by  $[Y_t; t \in T]$  where  $Y_t$  is the output stochastic variable; and the throughput stochastic process, representing the transformation of input to output, by  $[Z_t; t \in T]$  where  $Z_t$  is the throughput stochastic variable. These stochastic processes can be simply denoted by  $[X_t]$ ,  $[Y_t]$ , and  $[Z_t]$  respectively. They may not be considered as independent but as a stochastic vector  $[X_t, Y_t, Z_t; t \in T]$  or  $[X_t, Y_t, Z_t]$ .

The time parameter  $t$  in the stochastic processes may be either continuous or discrete. For practical and analytical purposes and for a possible solution of the mathematically simulated model by digital computers, the stochastic processes may be taken as discrete time functions. The index set  $T$  represents a length of time long enough to describe the hydrologic phenomenon under consideration. Units of the time parameter  $t$  can be chosen in convenient time intervals so that for the integral values of  $t = 1, 2, \dots, T$ , the stochastic variables define the respective processes in satisfactory detail. It should be noted that the time interval to be chosen for the discrete time parameter will affect the simulated stochastic laws of the processes. In general, smaller time intervals will make the stochastic laws more complicated as the magnitude and extend of dependence among the stochastic variables based on the historical hydrologic data of a process will be greater and in more detail.

The input-and-output relationship of a stochastic hydrologic system may be represented mathematically by a system equation as Eq. 1:

$$[Y_t] = \phi \{ [X_t], [Z_t] \} \quad (15)$$

where  $\phi \{[X_t], [Z_t]\}$  is the transfer function that represents the operation performed by the system on the input and the throughput in order to transfer them into output.

In most cases, the input, output and throughput of a hydrologic system are amounts of water, although in certain cases they can be taken as energy or other forms of medium. By the basic principle of system continuity, the output is equal to the input minus the throughput which is the amount of flow in the system. Thus, a single transfer function may be written similar to Eq. 2:

$$\phi \{[X_t], [Z_t]\} = [X_t] - [Z_t] \quad (16)$$

Hence, from Eqs. 15 and 16, the hydrologic system equation becomes

$$[Y_t] = [X_t] - [Z_t] \quad (17)$$

For the t-th time interval, or the time interval from t to t+1, Eq. 17 may be written as

$$Y_t = X_t - Z_t \quad (18)$$

where  $X_t$  is the input in the t-th time interval,  $Y_t$  is the output in the t-th time interval, and  $Z_t$  is the change of throughput in the t-th time interval.

In order to demonstrate the use of the above equation, the watershed is taken as a hydrologic system which has the precipitation as its input, the streamflow or runoff as its output, and the change in basin storage, due to storage and depletion as well as to evapotranspiration plus other losses, as its throughput. For the watershed, Eq. 18 becomes

$$Y_t = X_t - E_t - S_{t+1} + S_t \quad (19)$$

where  $X_t$  is the total amount of precipitation input to the watershed during the t-th time interval;  $Y_t$  is the total amount of runoff output from the watershed during the t-th time interval;  $E_t$  is the total amount of evapotranspiration plus other losses during the t-th time interval;  $S_t$  is the basin storage at time t; and  $S_{t+1}$  is the basin storage at time t+1.

To illustrate the application of the above mathematical formulation of stochastic hydrologic systems, a stochastic annual storm-flood model for watershed systems may be discussed. The annual storm-flood is the runoff of a flood produced by an annual storm. The annual storm is a storm which

produced the maximum peak discharge of flood flow in a water year. Therefore, for N water years of storm and flood records, there are N annual storms and N annual storm-floods.

The annual storm is treated as the input stochastic process to the watershed system which transforms the annual storm into the annual storm-flood. The stochastic process of the annual storm is denoted by  $[X_t; t \in T]$  where the time increment for t in general may be conveniently taken as one hour and T is the duration of the storm considered in the analysis. It is evident that the hourly rainfall process in an annual storm is nonstationary as the probability of transition between the hourly rainfalls of a storm changes with time since the storm began and hence depends on the time of transition. The nonstationary discrete-time process can be described by a first-order nonhomogeneous Markov chain to represent the stochastic process of the annual storm under consideration. This Markov chain may be written as

$$X_t = \lambda_t X_{t-1} + \epsilon_t \quad (20)$$

where  $X_t$  is the stochastic variable of hourly rainfall in the annual storm,  $\lambda_t$  is the Markov or regression coefficient, and  $\epsilon_t$  is the random component of  $X_t$ . The subscript t implies that the process is nonstationary as the process and its parameters change with time in the process.

The transition probability of the hourly rainfalls in the annual storm process is

$$g_t(x_t | x_{t-1}) = P[X_t = x_t | X_{t-1} = x_{t-1}] \quad (21)$$

where the subscript t indicates the nonstationarity and  $x_t$  is the variate of the stochastic variable  $X_t$ . Thus,

$$\begin{aligned} P(X_t = x_t) &= \sum_{x_{t-1}} P[X_t = x_t | X_{t-1} = x_{t-1}] p(X_{t-1} = x_{t-1}) \\ &= \sum_{x_{t-1}} g_t(x_t | x_{t-1}) p(X_{t-1} = x_{t-1}) \end{aligned} \quad (22)$$

where  $p(X_{t-1} = x_{t-1})$  is the probability for  $X_{t-1} = x_{t-1}$ . Eq. 22 implies that the probability of the outcome  $x_t$  depends on the probability of its antecedent  $x_{t-1}$  and this dependence is represented by the transition probability of Eq. 21.

The annual storm-flood depends not only on the antecedent annual storm-rainfall but also on the corresponding physical condition of the watershed. It can therefore be assumed that the hourly flood flow  $Y_t$  in the  $t$ -th time interval depends upon the hourly rainfall  $X_{t-1}$  of the annual storm at the  $(t-1)$ -th time interval as well as upon the basin storage  $S_t$  at time  $t$ . Corresponding to the time increment for hourly rainfalls, the increment of  $t$  for flow is also taken as one hour. The process  $[Y_t; t \in T]$  can then be suitably described by

$$Y_t = \phi_t(X_{t-1}, S_t) + \epsilon_t \quad (23)$$

where  $\phi_t(X_{t-1}, S_t)$  is some function of  $X_{t-1}$  and  $S_t$  to represent the deterministic component of  $Y_t$ , and  $\epsilon_t$  is a random variable uncorrelated with  $X_{t-1}$  and  $S_t$  but to provide the random component of  $Y_t$ . The subscript  $t$  implies that the process and its components are all nonstationary. The length of  $T$  is the duration of the flood to be considered in the analysis.

When sufficient data are available, the function  $\phi_t(X_{t-1}, S_t)$  can be derived by multiple regression. For the case under consideration, a multiple linear regression is found suitable. Thus, Eq. 23 may be written as

$$Y_t = a_t X_{t-1} + b_t S_t + c_t + \epsilon_t \quad (24)$$

where  $a_t$  and  $b_t$  are nonstationary regression coefficients and  $c_t$  is the nonstationary intercept of the linear regression line-of-fit. The random component  $\epsilon_t$  may be assumed as normally and independently distributed or as distributed according to a probability law suitable to the given data.

In the analysis of annual storms and annual storm-floods, the evapotranspiration and other losses in the hydrologic process may be ignored because they are insignificant in the relatively short durations of the storms and floods under consideration. Thus, Eq. 19 reduces to

$$S_{t+1} = X_t - Y_t + S_t \quad (25)$$

The phenomenon of transforming hourly rainfall to hourly flood flow in the watershed system, as influenced by the basin storage, may be described by a one-step bivariate nonhomogeneous Markov process which is represented by a family of two-dimensional stochastic vectors  $[X_t, S_{t+1}]$  as

$$\begin{aligned}
 P_{ij}(t) &= f_t(x_t, s_{t+1} \mid x_{t-1}, s_t) \\
 &= P[X_t = x_t, S_{t+1} = s_{t+1} \mid X_{t-1} = x_{t-1}, S_t = s_t] \quad (26)
 \end{aligned}$$

where  $P_{ij}(t)$  is the transition probability of the bivariate process from state  $i$  at  $t-1$  to state  $j$  at  $t$ .

Since Eq. 23 assumes that the flood flow  $Y_t$  is a function of  $X_{t-1}$  and  $S_t$ , the transition probability of the hourly flood flows for the annual storm-flood process is

$$h_t(y_t \mid x_{t-1}, s_t) = P[Y_t = y_t \mid X_{t-1} = x_{t-1}, S_t = s_t] \quad (27)$$

where  $x_{t-1}$ ,  $y_t$  and  $s_t$  are respectively the variates of the variables  $X_{t-1}$ ,  $Y_t$  and  $S_t$ . Thus, the probability of the flood flow is

$$P(Y_t = y_t) = \sum_{x_{t-1}, s_t} h_t(y_t \mid x_{t-1}, s_t) P(X_{t-1} = x_{t-1}, S_t = s_t) \quad (28)$$

From Eqs. 21 and 27, it can be shown<sup>32</sup> that

$$P_{ij}(t) = g_t(x_t \mid x_{t-1}) h_t(y_t \mid x_{t-1}, s_t) \quad (29)$$

Let the two-dimensional state vector  $[X_t, S_{t+1}]$  of the bivariate Markov process assume discrete values  $\delta [X_t, S_{t+1}]_k$  which can be represented by a point in a two-dimensional plane. The coordinates of this point are  $X_t$  and  $S_{t+1}$ . If  $X_t$  can assume discrete states  $m = 1, 2, \dots, M$ , where each state represents a convenient range of hourly rainfall amounts, and  $S_{t+1}$  can assume discrete states  $n = 1, 2, \dots, N$ , where each state represents a convenient range of hourly basin storages, then the state vector  $[X_t, S_{t+1}]$  can assume  $MN$  discrete states, i.e.,  $K = MN$ .

Following Eq. 26 the transition probability of the bivariate process from state  $i$  at  $t-1$  to state  $j$  at  $t$  is

$$f_t(x_t, s_{t+1} \mid x_{t-1}, s_t) = p_{ij}(t), i, j = 1, 2, \dots, K \quad (30)$$

For each  $t = 1, 2, \dots, T$ , there will be a stochastic matrix of size  $K$ . Hence, there will be  $T$  stochastic matrices, each of size  $K$ , for the proposed bivariate process.

Now, assume the initial probability for the bivariate process,  $p_j(0)$  for  $j = 1, 2, \dots, K$ . Since the initial rainfall is always zero, only the probability of the

initial basin storage should be assumed. The absolute probability of the bivariate Markov process can be derived inductively<sup>32</sup> to be

$$p_j(t) = \sum_{i=1}^K p_i(t-1) p_{ij}(t) \text{ for } j = 1, 2, \dots, K \quad (31)$$

where  $p_j(t) = p(X_t = x_t, S_{t+1} = s_{t+1})$  and the state  $j$  refers to the state of the state vector  $[X_t, S_{t+1}]$ .

Summing up the transition probability  $p_{ij}(t)$  of Eq. 29 over all the values of  $x_t$ ,

$$\begin{aligned} \sum_{x_t} p_{ij}(t) &= \sum_{x_t} g_t(x_t | x_{t-1}) h_t(y_t | x_{t-1}, s_t) \\ &= h_t(y_t | x_{t-1}, s_t) \end{aligned} \quad (32)$$

Substituting this equation in Eq. 28, the probability distribution of the flood flow  $Y_t$  is

$$p(Y_t = y_t) = \sum_{x_t, x_{t-1}, s_t} p_i(t-1) p_{ij}(t) \quad (33)$$

where  $p_i(t-1) = p(X_{t-1} = x_{t-1}, S_t = s_t)$  is, as shown by Eq. 31, the absolute probability of the bivariate process being in state  $i$  at time  $t-1$ , and  $p_{ij}(t)$  is, as shown by Eq. 30, the transition probability of the bivariate process from state  $i$  at  $t-1$  to state  $j$  at  $t$ .

The basin storage depends on the input rainfall and the storage at the beginning of the rainfall interval. From the joint probability, the marginal probability of the basin storage  $S_{t+1}$  can be calculated by

$$p(S_{t+1} = s_{t+1}) = \sum_{x_t} p(X_t = x_t, S_{t+1} = s_{t+1}) = \sum_{x_t} p_j(t) \quad (34)$$

where the summation is over all values of  $x_t$  for  $m = 1, 2, \dots, M$ .

The stochastic annual storm-flood model is now represented by Eq. 25 in which the components  $X_t$ ,  $Y_t$  and  $S_t$  are expressed by Eqs. 20, 24 and 25 itself, respectively, and their probabilities by Eqs. 22, 31 and 32, respectively. As an exercise, this model has been applied to the French Broad River Basin in North Carolina. Twenty-seven annual-storm records (1935-62) and hydrographs of the corresponding floods were used to determine the model.<sup>32</sup> Fig. 4 shows the

computed hydrograph by this model, or Eq. 24, as compared with the observed hydrograph for the annual storm-flood of the hydrologic year 1937-38. An exact agreement between the two hydrographs is not expected because the computed hydrograph is generated stochastically and the scattered portion of the observed data may be caused by the random component.

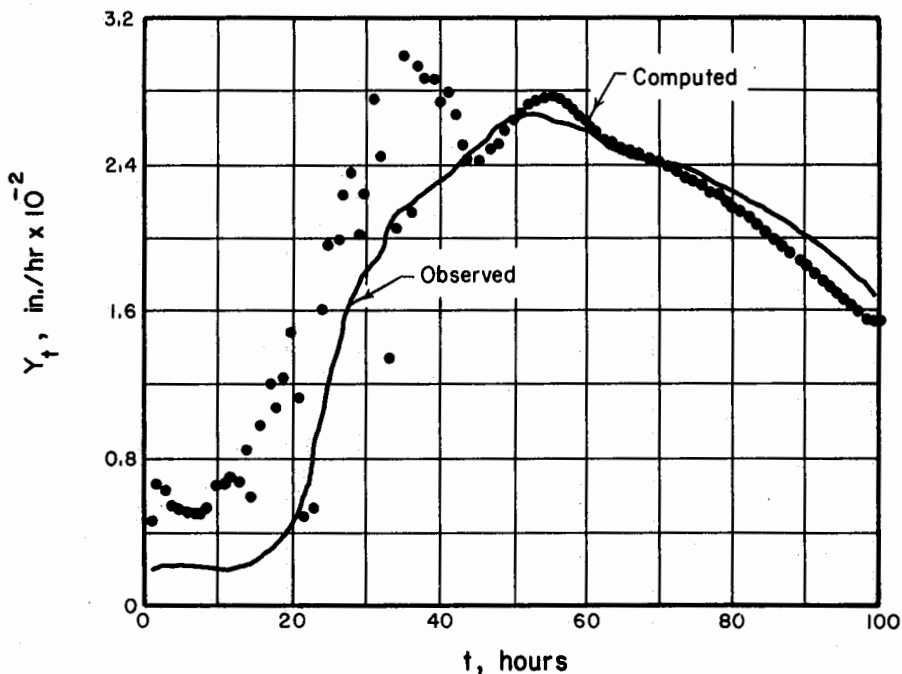


Fig. 4 Computed and Observed Hydrographs for Hydrologic Year 1937-38

The above example is a relatively complex stochastic model as it is the result of a recent research, and it should be further improved for practical use. Such complex models are generally handicapped by the limited amount of existing hydrologic data and the storage capacity of available computers. For the models to be practicable, more data and much larger computer memory would be required. However, for practical purposes, much simpler stochastic models are now available.



### Practical Considerations

As mentioned earlier, deterministic models make forecasts, while indeterministic models make predictions. A deterministic model forecasts by transforming one sequence of hydrologic events (the input to a hydrologic system) into another sequence (the output from the system). An indeterministic model predicts by generating sequences of hydrologic events. The generating process may be done by using a mathematical random-number generator to produce random values on a computer and then feed the latter into the random element of the model, such as  $\epsilon_t$  in Eq. 20 or 24, thus producing various output sequences.<sup>33</sup> The generating process will take into account the order of events in the sequences. Theoretically speaking, if the indeterministic model is correctly built, the statistical characteristics of the historical data which determine the model must be preserved in the generated sequences. This principle provides a criterion to test the validity of an indeterministic model.

From the above discussion, it can be seen that the two types of abstract model are complementary rather than competitive in the practical application. It is perfectly possible to take the generated output from an indeterministic model, a sequence of events that could occur, and transform them using a deterministic model. The deterministic model then produces a forecast of what would happen, given what could happen. By this procedure, Chow and Ramaseshan<sup>34</sup> generated runoff data from historical rainfall data for the French Broad River Basin in North Carolina. In this case, given a predicted rainfall sequence the runoff resulting from the rainfall could be forecast.

The two types of model can also be used independently. For example, an indeterministic model built on historical streamflow could predict future streamflows. Such predicted streamflows can be used to design a water resource system over the period of planning projection. Fig. 5 shows one historical sequence and 98 stochastically generated sequences of average monthly flows which are used for the design study of the Trans-Texas Water System.<sup>35</sup> Selected sequences from such water supply and demand sequences were then used to help find the optimal implementation plan, or the plan to be developed at various stages of the planning horizon having the least total expected cost. For such a study, Fig. 6 shows the present value cost response surface where the least expected cost of 4.34 billion dollars occurs when the change in canal capacity is  $\Delta C = -1000$  cfs, i.e., so much less than a given design capacity, at every interval of time  $\Delta T = 10$  years.

On the other hand, for short period data, the actual input to a deterministic model may already be available. For example, deterministic flood-routing models are widely used for flood forecasting in river control works. Such a model is an operational one and would, for this case, forecast the

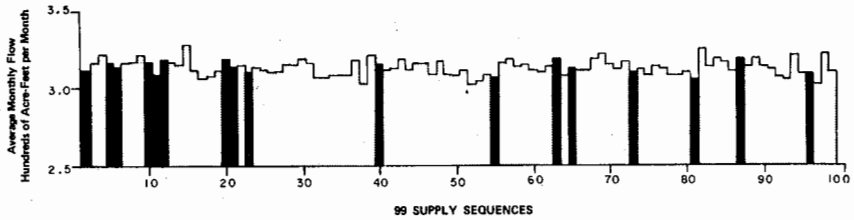


Fig. 5. 99 Average Monthly Flow Sequences Used in the Trans-Texas Water System Study<sup>3,5</sup>.

downstream flood that would occur from the observed upstream flood. Used independently as so described, the two types of model have separate purposes: indeterministic models are tools for planning, while deterministic models are tools for operation.

The procedure of hydrologic modeling may be either analytic or synthetic, although the distinction cannot be too exact. The analytic procedure is to assume certain physical principles that can be theoretically described. Given these principles, a means of analysis is developed, and the product is an analytic model. For example, Eq. 3 for the development of a general deterministic model is a principle which is described by a mathematical expression and soluble by mathematical analysis. The procedure is therefore analytic. As for the synthetic procedure, it is to combine the elements, or subsystems, of a system into an entity which will represent the system; and the product is a synthetic model. For example, The Stanford Watershed Model<sup>9</sup> is built by this procedure, and the entity to be represented is the land phase of the hydrological cycle of a watershed. If the way of combining the subsystems is not well defined or generally agreed upon, the synthetic model may vary between model builders. The subsystems are building blocks, and they may be individual analytic models. Thus, an analytic model usually describes a narrow and restricted system in hydrology so that the problem is made manageable, whereas a synthetic model can be complicated and cover as broad a scope of hydrologic phenomena as one wishes.

Whatever form the model takes it inevitably contains a number of unknowns, i.e., parameters, that serve to characterize a given hydrologic phenomenon. To be justifiable in a model's structure and to make it practically useful, it is desirable that the parameters be physically meaningful and capable of estimation. This point is of vital importance if the model is used to assess the effects of natural or man-made changes in the hydrologic system, or if a model developed from a known hydrologic region is to be transposed to an unknown

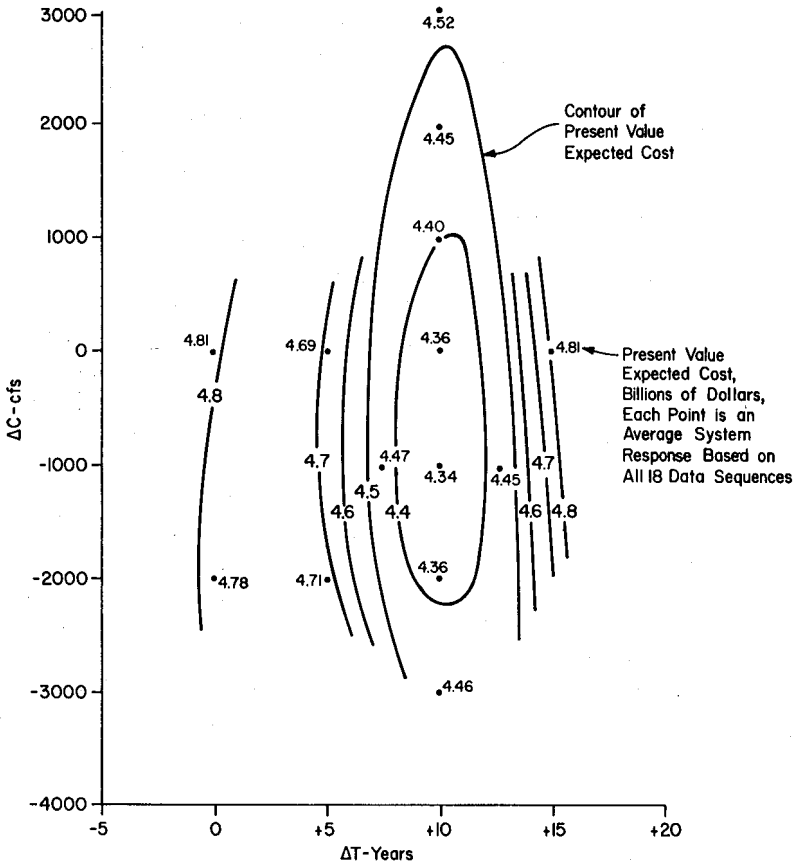


Fig. 6. Present value cost response surface<sup>35</sup>

hydrologic region. At present, hydrologic modeling technique has not advanced sufficiently to allow reliable estimates of the absolute values of the parameters to be determined. Furthermore, the parameters are not independent of one another; it is their relative magnitudes that count. This explains why certain models can satisfactorily reconstruct events for the hydrologic system to which they were fitted but are useless when applied to situations where a knowledge of the absolute values of certain parameters is required. When too many parameters are involved, usually in synthetic models, the interdependence of parameters is frequently aggravated. In a synthetic model, a wide variety of data can be used in the determination of the parameters. The model structure is not predetermined and it can be built to use whatever data are available. This flexibility is both a virtue and an evil, because each model can be tailored to fit a given hydrologic phenomenon but this has led to a vast proliferation of models whose relative merits are uncertain.

During the process of modeling or selecting of a model, one is invariably faced with a decision concerning simplicity versus completeness of the model. A simpler model is easier to understand and apply, and probably cheaper to use. This preference has resulted in an oversimplified and unrealistic model, such as the rational formula. On the other hand, a good model should involve minimum assumptions and approximations. This choice has led to certain components being more sophisticated than others and thus has biased the overall model structure towards special objectives. Both the analytic and synthetic modelings face the same problem, but the decisions are not easy to make as the effects of assumptions and approximations are not so obvious. To be effective, the model builder should be at least capable to distinguish the relative significance of the assumptions and approximations and between the components of the model. A desire for completeness in a model may result in one which is unwieldy to manage and may contain various components that are of incomparable significance. It has been suggested that if a simple model will do, no more complexity is necessary. This rule fails in practice unless "do" is better defined.

### Conclusions

This paper has introduced briefly the status of hydrologic modeling today. Hydrologic models are classified according to various assumptions. Emphasis is given to abstract models and their examples are illustrated. In practice, hydrologic modeling is partly science and partly art. It is an art since ingenuity and judgement enter into the modeling process and its assessment. As a science, it requires an advanced knowledge in physical principles and mathematical methods, but a main problem is to develop models with their practical application in mind. Many practicing engineers hesitate to accept new

techniques, because they find difficulties in understanding and applying them. It is therefore apparent that the practical value of hydrologic modeling in the future will depend on a better communication between practicing engineers and theoretical hydrologists.

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# DISPERSION ESTIMATES OFF NOBSKA POINT USING RECORDING CURRENT METERS

By M. D. Palmer<sup>1</sup> and J. B. Izatt<sup>2</sup>

## Abstract

Dilution estimates were made from the data collected by a recording current meter operated for two weeks off Nobska Point. The estimates were compared to estimates made by other methods. The 5 and 6 hour dilution rates for the ebb and flood tides range from 4690/1 to 8100/1 with the lower value occurring during the ebb tide. Five hour dispersion coefficients of 8.6 and 8.2 x 10<sup>5</sup> cm<sup>2</sup>/sec were determined from the current meter data. Generally, the hourly dispersion predictions from the current meter data were larger than estimates by other means and considered more representative. On the other hand, the steady state dispersion predictions from the current meter data require some other validation.

## Background

The Town of Falmouth, Massachusetts, has proposed building a domestic sewerage system with disposal of the effluent to take place in Vineyard Sound off Nobska Point. Bumpus,<sup>1</sup> as the result of drogue and drift bottle studies, has proposed that the site of the outfall should be located in approximately 30 meters of water 0.504 km south of Nobska Point. Bumpus<sup>1</sup> and Charnews and Jaffee<sup>2</sup> have applied several techniques to determine the probable extent of the sewage plume and what dilution might be expected if the outfall was located as mentioned above. The intent of this paper is to provide similar estimates from data collected concurrently by a current meter. The data were supplied by EG&G, Woods Hole.

Charnews and Jaffee<sup>2</sup> undertook dye studies to determine dispersion patterns for a tracer released at the location of the outfall. From these results, surface dispersion due to wind driven and tidal currents may be studied. Bumpus<sup>1</sup> bases his steady state mixing estimates on the assumption that one-tenth of the sound is new water in a tidal cycle. This is considered a conservative estimate by Bumpus. The volume of water in the sound changing every tidal cycle was determined from the low water volume and mean tidal rise. Similar computations were made for a net flow assuming 10 percent water exchange on a tidal cycle. In addition, immediate mixing estimates were made by multiplying the sound cross-section above a diffuser by an estimated mean velocity. An eddy diffusion coefficient was also determined employing an estimated velocity. The dispersion estimates determined from current meter data

<sup>1</sup>Environmental Systems Engineer, Water Quality Surveys Branch, Ontario Water Resources Commission.

<sup>2</sup>Management Services Officer, Systems & E.D.P. Branch, Ontario Water Resources Commission.



presented subsequently do not require velocity estimates. The time history sequence of the measured currents are maintained in the analysis. However, current meter methods are based upon currents measured at a single fixed point and it is necessary to extrapolate the measurements to areas for dispersion estimates. This requires a reasonably homogeneous velocity field over the area of interest. Therefore current meter measurements should be supplemented by concurrent dye and drogue measurements.

The two techniques for determining dispersion and dilution are complementary. Dye and drogue measurements are Lagrangian techniques which provide area characteristics. The predominant variable is space. These measurements are normally limited to daylight hours during reasonably calm sea conditions. Current meter records are continuous with respect to time at one location. The predominant variable is time. The two techniques are obviously not independent. Resultant dispersion and dilution characteristics are a mixture of both, although one may predominate in a particular case. Unfortunately, it is not possible to determine which variable is the more important in this case as concurrent Lagrangian measurements are not available. This means that the current meter estimates require some validation. Comparison with the estimates of Bumpus<sup>1</sup> provide some validation. However, the assumptions involved in Bumpus's computations are qualitative.

The results of the current meter study are presented in the following. The techniques are summarized. Further details on the techniques can be obtained from the appropriate references.

## Results

The current meter was operated at the proposed site of the outfall from November 26, 1969, to December 10, 1969, at a depth of 10m. A longer period of record would be desirable. The data set resulting from the operation consisted of readings of current speed and direction reduced to 10-minute averages. The data time sequence was smoothed using a moving average technique over  $\pm 20$  minutes to make the data suitable for analysis. A measure of the smoothing can be obtained by comparing some data statistics. The resultant and arithmetic mean velocities for the complete record were 3.8 cm/sec at  $154^{\circ}\text{N}$  and 59.3 cm/sec for the smoothed data and 1.9 cm/sec at  $154^{\circ}\text{N}$  and 59.3 cm/sec for the raw data set. The smoothed data were reduced to obtain a two-dimensional frequency table (Table 1). The tidal effect is clearly evident with some 40 percent of the readings occurring in the north-east direction, and 45 percent occurring in the south-west direction.

To determine a resultant velocity magnitude, it is necessary to find a mean velocity during an ebb tide, then compare it to the mean of the flood tide.

TABLE 1 Frequency Analysis for Current Meter Data

## DIRECTION (DEGREES)

SPEED (cm/sec)	DIRECTION (DEGREES)										ROW SUMS
	337.50- 22.49	22.50 67.49	67.50- 112.49	112.50- 157.49	157.50- 202.49	202.50- 247.49	247.50- 292.49	292.50- 337.49	337.49- 382.49	382.50- 427.49	
0.00 - 12.99	0.000	0.049	0.345	0.839	0.691	0.000	0.049	0.049	0.049	0.049	2.023
13.00 - 25.99	0.099	1.085	2.713	2.368	3.355	3.453	0.296	0.099	0.099	0.099	12.468
26.00 - 38.99	0.000	2.713	2.269	0.543	0.247	5.131	0.148	0.000	0.000	0.000	11.051
39.00 - 51.99	0.000	5.969	0.888	0.000	0.000	6.364	0.049	0.000	0.000	0.000	13.271
52.00 - 64.99	0.000	7.943	0.049	0.000	0.000	7.745	0.000	0.000	0.000	0.000	15.738
65.00 - 77.99	0.000	7.795	0.000	0.000	0.247	9.423	0.000	0.000	0.000	0.000	17.464
78.00 - 90.99	0.000	8.337	0.000	0.000	0.000	5.871	0.000	0.000	0.000	0.000	14.208
91.00 - 103.99	0.000	3.207	0.000	0.000	0.000	4.489	0.000	0.000	0.000	0.000	7.696
104.00 - 116.99	0.000	2.368	0.000	0.000	0.000	1.332	0.000	0.000	0.000	0.000	3.700
117.00 - 129.99	0.000	0.296	0.000	0.000	0.000	1.085	0.000	0.000	0.000	0.000	1.381
COLUMN SUMS	0.099	39.763	6.265	3.749	4.539	44.894	0.543	0.148	0.099	0.099	132.000

Towards this end, twelve tidal cycles were selected from the record. Eight of the tidal cycles were selected so that they were sequential in time, while the other four were selected in sequential pairs. The results are summarized in Table 2.

From Table 2 it can be seen that the direction of the current off Nobska Point remains relatively constant varying between  $56^{\circ}$ - $65^{\circ}$  on the flood tide and between  $231^{\circ}$ - $237^{\circ}$  on the ebb tide. However, the resultant velocity magnitude varies widely up to 50 percent on the flood tide and 40 percent on ebb. In the first group of eight half tidal cycles, if taken in pairs starting with the first two, it would appear that there is a large net flow alternately on the ebb and flood cycles. However, starting with the second half cycle, and taking pairs, there is little net flow. This phenomenon also appears in the other pairs of tidal cycles. These records indicate that misleading results can occur by assessing the net flow on the basis of measurements over one tidal cycle. They may also indicate the need for several current meters operated at different locations. Considering the twelve tidal cycles presented in Table 2, the resultant speeds for all twelve on the flood and ebb tides are 54.10 cm/sec and 54.29 cm/sec respectively. Thus, for the twelve cycles chosen here, representing 45 percent of the data, the net transport velocity is 0.16 cm/sec to the south-west (ebb tide direction). The overall arithmetic mean velocity magnitude is 54.20 cm/sec.

These results may be compared with those found by Bumpus<sup>1</sup>. The arithmetic mean velocity magnitude is 1.06 knots (54.20 cm/sec) compared to 1.15-2 knots found by Bumpus.<sup>1</sup> The net speed found here is 0.004 knots (0.16 cm/sec) whereas Bumpus found a value of 0.1 knots in his net flow analysis. The effect of this difference is to lower the net transport from 200 million  $m^3$  to 10 million  $m^3$  and to decrease the dilution from 2850/1 to 134/1. Thus the dilution is 20 times lower than found by Bumpus. The meaning of this small dilution or net transport integrated over a period of 14 days is not clear. The mean velocities for each ebb and flood cycle are so large that material will be transported long distances during a single cycle into regions which may have different dispersive qualities.

The summary of the twelve tidal cycles indicates that the directions of the current at the proposed outfall site remain constant at  $233^{\circ}$  and  $60^{\circ}$  for the ebb and flood tides respectively. During the 14 days when the current meter operated, it is likely that meteorological conditions varied. It would thus appear that meteorological effects have little effect on subsurface currents at the depth at which the current meter was located. It is difficult to estimate the current characteristics at other depths and locations from one current meter site without the aid of other current meter installations. However, initially dispersion should be in a direction slightly more offshore than found by Charnews and Jaffee<sup>2</sup> in their surface dye experiments. The difference in direction is not surprising since subsurface and surface currents are generally different.

TABLE 2 Mean Resultant Velocities for Selected Half Tidal Cycles

START DATE	END DATE	DIRECTION	RESULTANT SPEED cm/sec.
Nov. 27 11:56	Nov. 27 18:00	61.6°	55.0
Nov. 27 18:00	Nov. 28 00:06	233.2°	68.6
Nov. 28 00:44	Nov. 28 06:04	56.1°	64.2
Nov. 28 06:44	Nov. 28 12:44	233.1°	45.4
Nov. 28 12:25	Nov. 28 18:31	62.5°	48.0
Nov. 28 18:31	Nov. 29 00:25	232.5°	66.0
Nov. 29 1:25	Nov. 29 7:25	59.8°	64.6
Nov. 29 7:31	Nov. 29 13:31	233.6°	45.0
Dec. 1 2:25	Dec. 1 8:31	59.3°	64.2
Dec. 1 8:31	Dec. 1 14:25	237.6°	45.6
Dec. 1 14:50	Dec. 1 20:50	64.8°	34.8
Dec. 1 20:50	Dec. 1 02:50	234.5°	64.7
Dec. 1 20:25	Dec. 2 3:13	233.5°	58.5
Dec. 2 3:25	Dec. 2 9:25	58.2°	41.0
Dec. 2 9:31	Dec. 2 16:00	233.5°	50.5
Dec. 2 16:31	Dec. 2 22:25	59.4°	45.4
Dec. 2 22:31	Dec. 3 3:13	233.0°	36.3
Dec. 3 3:25	Dec. 3 10:44	60.4°	70.0
Dec. 3 10:50	Dec. 3 15:50	235.3°	49.1
Dec. 3 17:00	Dec. 3 22:31	60.1°	45.4
Dec. 5 18:06	Dec. 6 00:13	61.3°	51.7
Dec. 6 0:13	Dec. 6 6:06	231.2°	46.0
Dec. 6 6:13	Dec. 6 12:13	60.2°	65.1
Dec. 6 12:38	Dec. 6 18:00	233.2°	75.7
	Resultant	233°	0.16 cm/sec

The current meter data may also be used as input to stochastic models for the prediction of dilution rates. The currents were classified into states by speed and direction. For example, 10 speed states (0-13 cm/sec; 13-26 cm/sec, etc.) and 8 direction states ( $337.5^{\circ}$ - $22.5^{\circ}$ ;  $22.5^{\circ}$ - $67.5^{\circ}$  etc.) were defined. Thus, it is possible to categorize each current reading by its speed and direction. By comparing each current state with the one following, it is possible to find the probability of a current transferring from one state to another in one time period. Using the complete record of readings, a matrix called the first order Markov Chain transition probability matrix, is developed. For this study two transition matrices were developed, one for ten-minute interval data and one for hourly interval data. A more complete outline of the method is contained in publications by Palmer and Izatt.<sup>6,5</sup>

The frequency table shown in Table 1 gives the probability of being in any one of the 80 possible states based on the period of record. The probability of going from one state to another in one hour is given by the probability transition matrix. Thus, the probability of starting in state A and proceeding in 5 hours to state B through 4 intermediate states may be found. The distance travelled may be calculated by assuming that the speed for each state remains constant for one hour, then multiplying the speed and time together. The distance travelled in five hours may be weighted by the probability of that sequence of states occurring. The weighted distance travelled can be resolved into components in the 4 major directions. If all the possible sequences for all initial and final states are considered, then a weighted mean distance and standard deviation of distance in each direction can be found for the time period. The number of possible sequences increases by the power of the number of hours considered and computer costs limit the practical sequence to 5 states or 5 hours for hourly transition matrices. Diffusion coefficients can be found from the standard deviations using the following equation (Hinze<sup>4</sup>).

$$\langle y^2 \rangle = 2 \epsilon t \quad (1)$$

in which  $\langle y^2 \rangle$  is the variance of particle separation in  $\text{cm}^2$ ;  $\epsilon$  is the diffusion coefficient in  $\text{cm}^2/\text{sec}$ ; and  $t$  is the time in seconds. The mean velocity for five hours may be found by dividing the weighted distance by the time. In the case of strong tidal action, as occurs off Nobska Point, the two directions perpendicular to the tidal current do not have enough readings in the current meter record to give meaningful results. A summary of results for 5 hours is given in Table 3. The mean velocities shown in Table 3 are a bit low since the Markov Chain analysis ignores some of the smaller probability sequences in order to economize on computer time. The diffusion coefficients for both ebb and flood tide direction are about the same.

TABLE 3 Summary of Dispersion Characteristics for 5 Hours

Direction (Degrees)	Weighted Standard Deviation $\langle y^2 \rangle^{1/2}$ (cm)	Weighted Diffusion Coefficient $\epsilon$ (cm <sup>2</sup> /sec.)	Mean Velocity $\langle U \rangle$ (cm/sec.)
60°	1.76 x 10 <sup>5</sup>	8.60 x 10 <sup>5</sup>	51.6
150°	.11 x 10 <sup>5</sup>	.03 x 10 <sup>5</sup>	3.8
240°	1.72 x 10 <sup>5</sup>	8.22 x 10 <sup>5</sup>	55.0
330°	.001 x 10 <sup>5</sup>	.00 x 10 <sup>5</sup>	1.3

A measure of the dilution taking place can be found using the following equation (Foxworthy et al<sup>3</sup>)

$$\langle C_{\max}(x) \rangle = \frac{Q}{(2\pi)^{1/2} \langle y^2 \rangle^{1/2} \langle \bar{U} \rangle} \quad (2)$$

in which  $\langle C_{\max}(x) \rangle$  is the maximum concentration at a distance  $x$  in mg/cm<sup>2</sup>;  $Q$  is the mass discharge in mg/sec.;  $\langle y^2 \rangle$  is the weighted mean spread in cm<sup>2</sup>; and  $\langle \bar{U} \rangle$  is the weighted mean velocity in cm/sec. This is essentially a two-dimensional equation as current meter readings are two-dimensional. From the consultant's report, the source concentration of PO<sub>4</sub> as P is 30 mg/l and the effluent is being discharged at the rate of 0.156 m<sup>3</sup>/sec.

In order to make the source two-dimensional it is necessary to assume that the outfall consists of a circular pipe discharging at a speed of 1 m/sec. The diameter of the pipe is then 0.492 meters. Taking a 1 cm wide slice at the maximum diameter gives a source of 150 mg/sec. Substituting in Equation (2) yields  $\langle C_{\max}(x) \rangle = 4.7 \times 10^{-6}$  mg/cm<sup>3</sup> on the flood cycle and  $\langle C_{\max}(x) \rangle = 4.5 \times 10^{-6}$  mg/cm<sup>3</sup> on the ebb cycle for a 1 cm thick layer. The percentage dilution is then 0.016 percent and 0.015 percent on the flood and ebb tides respectively.

An alternative way of finding dilution characteristics involves defining an initial state vector and multiplying it by the transition probability matrix (Palmer and Izatt<sup>5</sup>). This defines a final state probability vector after one time period. Weighted mean distances travelled can be found by multiplying the probability of a state by the speed and the time interval and summing those in the same direction. This produces a mean distance travelled in one time period for each of the eight directions. The final state vector can be used as an initial state vector for the next time step. The procedure may be carried on for as many

steps as desired. Figure 1 illustrates the contours predicted for 1, 3 and 6 hours for an initial state with 100 percent probability of a current going north-east at 33.5 cm/sec. and for an initial state with 100 percent probability of a current going south-east at 33.5 cm/sec.

The areas represented by the contours do not represent the shape of an effluent plume released at the source. On the average, an effluent released at the source will form a plume which will be contained somewhere within the contours after 1, 3 and 6 hours. The plume would not necessarily occupy the total area within the contour. If a constant depth for an effluent plume is assumed then a pessimistic estimate of its volume may be obtained by measuring the area of the contours in Figure 1 and multiplying by the constant depth. The dilution rate may then be found if the amount of effluent released in 1, 3 and 6 hours is known.

A depth of 10 meters has been assumed and the areas, volumes, and dilutions for 1-6 hours for a flood tide and an ebb tide have been calculated and summarized in Tables 4 and 5. The dilution is 4690/1 parts seawater to effluent for an ebb tide after 6 hours, and 8100/1 for a flood tide. There is a 42 percent difference in initial dilution in the two directions after 6 hours. This method is restricted to times less than a half tidal cycle because the probabilistic nature of the solution forces flow in both directions even after the currents have reversed direction. An approximation for the steady state dilution may be found by averaging the dilution for the two directions. This yields a steady state dilution of 6345/1 parts of seawater to sewage.

### Discussion

The current meter data indicates that the average tidal current varies up to 50 percent in magnitude but remains constant in direction. Analysis of mean tidal resultant currents shows that several tidal cycles must be considered when a net current is sought. Definition of a net current based on one tidal cycle may be in error if the current is to be used as a mean value to represent exchange processes for many tidal cycles.

The dispersion estimates determined from the current data are summarized in Table 6. It is observed that the current meter estimates are much larger than those predicted by Bumpus<sup>1</sup> confirming his qualifications that the estimates are conservative. For hourly predictions the current meter method is probably more representative as the temporal variations are large. The mean resultant current directions for the ebb and flood tides are probably more representative of the initial dilution for a sub-surface outfall than are the results obtained by Charnews and Jaffee<sup>2</sup> in their surface dye tracing experiments. The current meter data indicate that initially dilution will take place in a direction more off

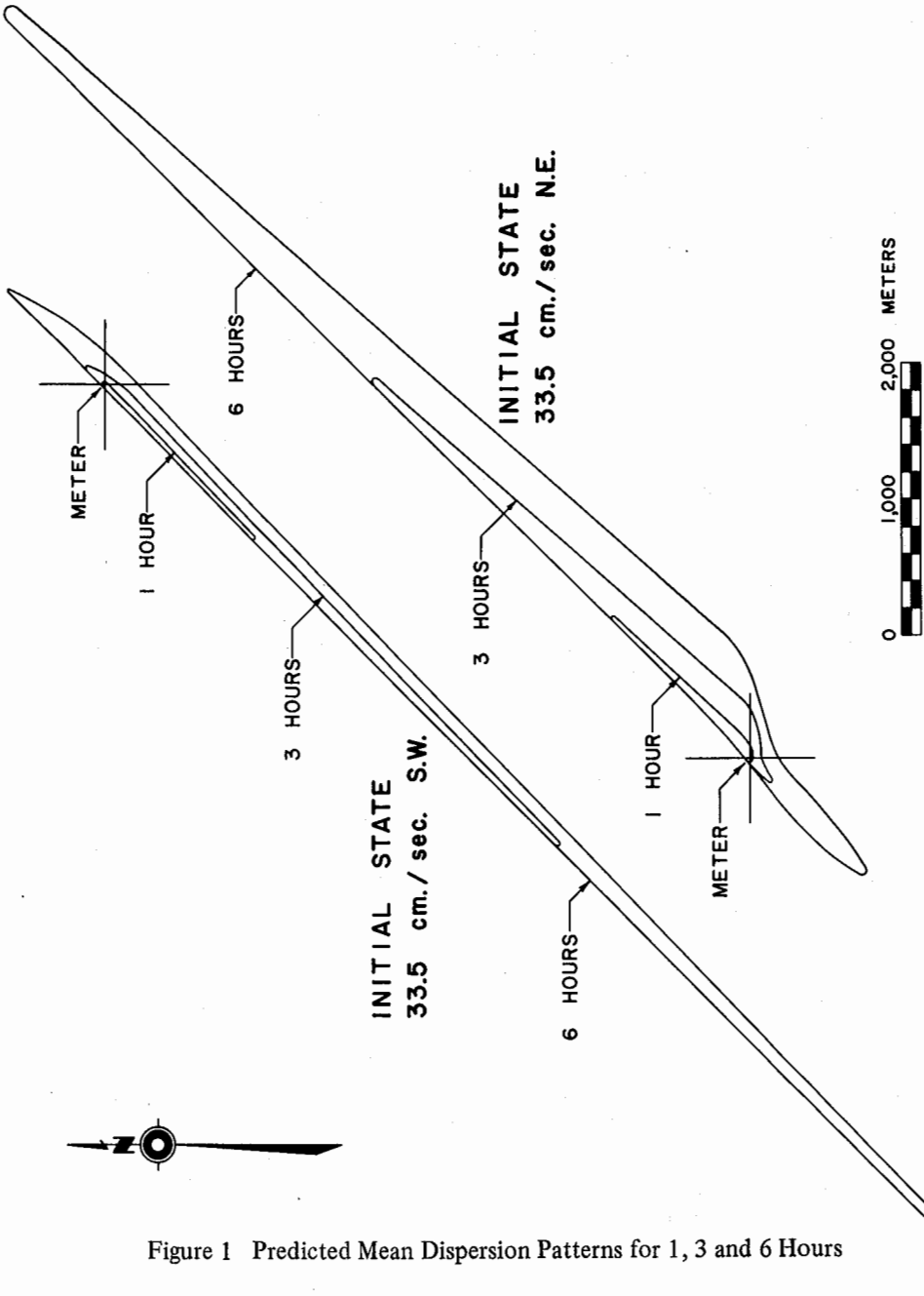


Figure 1 Predicted Mean Dispersion Patterns for 1, 3 and 6 Hours



TABLE 4 6 Hour Dilution Characteristics for Flood Tide

HOURS	AREA (m <sup>2</sup> )	VOLUME (m <sup>3</sup> )	SOURCE (m <sup>3</sup> )	DILUTION (Parts Seawater/Effluent)	
1	.86 x 10 <sup>5</sup>	8.6 x 10 <sup>5</sup>	584	1472/1	(.07%)
2	3.56 x 10 <sup>5</sup>	35.6 x 10 <sup>5</sup>	1168	3050/1	(.03%)
3	9.25 x 10 <sup>5</sup>	92.5 x 10 <sup>5</sup>	1752	5270/1	(.02%)
4	13.98 x 10 <sup>5</sup>	139.8 x 10 <sup>5</sup>	2336	5980/1	(.02%)
5	21.58 x 10 <sup>5</sup>	215.8 x 10 <sup>5</sup>	2920	7390/1	(.01%)
6	28.40 x 10 <sup>5</sup>	284.0 x 10 <sup>5</sup>	3504	8100/1	(.01%)

TABLE 5 6 Hour Dilution Characteristics for Ebb Tide

HOURS	AREA (m <sup>2</sup> )	VOLUME (m <sup>3</sup> )	SOURCE (m <sup>3</sup> )	DILUTION (Parts Seawater/Effluent)	
1	.2 x 10 <sup>5</sup>	2 x 10 <sup>5</sup>	584	343/1	(.3%)
2	1.3 x 10 <sup>5</sup>	13 x 10 <sup>5</sup>	1168	1111/1	(.09%)
3	3.7 x 10 <sup>5</sup>	37 x 10 <sup>5</sup>	1752	2105/1	(.05%)
4	6.7 x 10 <sup>5</sup>	67 x 10 <sup>5</sup>	2336	2870/1	(.03%)
5	11.7 x 10 <sup>5</sup>	117 x 10 <sup>5</sup>	2920	4000/1	(.02%)
6	16.4 x 10 <sup>5</sup>	164 x 10 <sup>5</sup>	3504	4690/1	(.02%)

shore than found by Charnews and Jaffee.<sup>2</sup> The steady state current meter dilution predictions from single fixed point measurements require extrapolation to large areas. Consequently these predictions require validation by other methods. It is the authors' opinion that the steady state prediction from the meter data is too large.

The application of the dispersion estimates to an actual effluent would require some consideration on both positive and negative buoyancy effects. None of the estimates considers buoyancy directly. The dye studies of Charnews and Jaffee<sup>2</sup> do provide a qualitative indication of the possible dispersion pattern of a positively buoyant effluent. Normally surface effluent releases dilute more slowly than subsurface releases. Buoyancy effects should be incorporated into the estimates if the range of expected specific gravities for the effluent and receiving water were known.

TABLE 6 Comparative Dilution Rate Estimates

PERIOD	BUMPUS			CURRENT METER		
	(DILUTION ESTIMATES) (Parts Seawater/Effluent)		DISPERSION COEFFICIENT ( $\text{cm}^2/\text{sec.}$ )	(DILUTION ESTIMATES) (Parts Seawater/Effluent)		5 HOUR DISPERSION COEFFICIENT ( $\text{cm}^2/\text{sec.}$ )
	Tidal Exchange	Net Flow		5 hr. Mean	6 hr. Mean	
Ebb Tide	---	---	$0.1 \times 10^5$	6660/1	4690/1	$8.6 \times 10^5$
Flood Tide	---	---	$0.1 \times 10^5$	6250/1	8100/1	$8.2 \times 10^5$
Steady State	1700/1	2850/1	---	6455/1	6345/1	---

## References

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**Frank Alwyn Marston**  
1885 – 1972



Frank A. Marston was an engineer's engineer. Always a gentleman, friendly, conscientious and having a sense of pleasant dignity. His passing on February 22, 1972 marks the end of a full life and a busy career.

He was born in Worcester, Massachusetts on September 21, 1885, the son of Charles F. and Abby M. Marston. He attended grammar and high schools in Worcester from which he entered Worcester Polytechnic Institute, graduating in 1907 with the degree of B.S. in Civil Engineering. In 1960 the Institute awarded him an honorary Doctorate of Engineering.

In 1907 young Marston entered the employ of the Boston engineering firm of Metcalf & Eddy and began an association which was to continue until his retirement in 1966. He was made a partner of the firm in 1920 and was a senior partner at the time of his retirement. His work was mainly in relation to public water works and wastewater disposal systems, for which he received wide recognition and the highest honors of his profession.

He was Consulting Engineer to the New York Board of Water Supply from 1937 through 1966, and Chairman of the Board of Engineers (on wastewater disposal) for the District of Columbia from 1954 through 1964. Major projects with which he was connected include a wastewater treatment plant, large diversion sewers, and investigation and report on a sewerage system for the District of Columbia; sewers and a treatment plant for Louisville, Ky.; and reconstruction of water pumping stations and a new water filtration plant for Wilmington, Del. He also participated in some two hundred other projects of various types and sizes in his nearly 60 years of practice. He gave expert testimony in a number of court cases and also served on boards of arbitration.

In addition to his work in sanitary engineering, Mr. Marston had a deep interest in the field of soil mechanics and foundations. This interest led him in

1925-1926 to attend a series of lectures on soils mechanics at Massachusetts Institute of Technology, and again in 1936 to attend another such series at the Harvard Graduate School. Still following the same interest, he attended in 1948 the 2nd International Conference on Soil Mechanics and Foundation Engineering in Rotterdam. He was a member of the executive committee of the ASCE Soil Mechanics and Foundation Division for seven years, four as chairman.

Mr. Marston was active in a number of professional societies and associations. He joined the Boston Society of Civil Engineers in 1909, served as its president in 1927-1928, and was made an Honorary Member in 1960. Over the years he authored some thirteen papers which were published in the Journal.

He joined the American Society of Civil Engineers as a Junior Member in 1910, became an Associate Member in 1917, a Member in 1920, a Fellow in 1959, and was President in 1959-1960. In addition to his membership in BSCE and ASCE, Mr. Marston was a Life Member in ASTM, AWWA, APWA, NEWWA. He was a member of the American Institute of Consulting Engineers, a Diplomate of the American Academy of Environmental Engineers, a member of NSPE and MSPE, NEWPCA, International Association for Hydraulic Research, International Society of Soil Mechanics & Foundation Engineers, and United States Committee on Large Dams.

Mr. Marston was registered as a Professional Engineer in fourteen states, the District of Columbia, and the Province of Ontario.

A resident of Wollaston, Mass. he was active in community affairs there. He joined the Colonial Savings and Loan Association of Wollaston as a director in 1930, was elected president in 1941, and president emeritus in 1966. He was a man of strong religious convictions, being a 50-year member of the First Baptist Church in Wollaston and serving as Moderator for 30 years. He was also a collector of stamps and a member of both the American Philatelic Society and the Society of Philatelic Americans.

Mr. Marston married Bertha Wheeldon of Worcester, Mass. in 1911. Mrs. Marston passed away in 1952. He is survived by two sons, Alwyn F. of Weymouth, Mass., and Philip L. of Essex, Conn., and two daughters, Mrs. Ruth Alice Dexter of Cranford, N.J. and Mrs. Priscilla Stewart of Gloucester, Mass. He was devoted to his children and they in turn held him in love and high respect.

Frank A. Marston was dedicated to his profession, practiced a high standard of ethics and insisted on quality workmanship. In return he enjoyed a rewarding career, the high esteem of his engineer colleagues, a fine reputation and a host of friends.

Edwin B. Cobb

# PROCEEDINGS OF THE SOCIETY

## Minutes of Meeting

### Boston Society of Civil Engineers

January 19, 1972.—The regular monthly meeting of the Boston Society of Civil Engineers was held jointly with the Massachusetts Section of the American Society of Civil Engineers at Nick's Restaurant, 100 Warrenton Street, Boston, Mass. The members of the two societies assembled at 5:30 P.M. for cocktails. At 6:00 P.M. dinner was served, and the business meeting commenced at 7:30 P.M.

President Robert Barton of the Mass. Section A.S.C.E. called the meeting to order and introduced the guests at the head table. He then turned the meeting over to President Ernest A. Herzog, to conduct any business for the Boston Society of Civil Engineers.

President Herzog announced that the minutes of the previous meeting held on June 2, 1971 had been published in the July 1971 issue of the Journal, and that the reading of those minutes would be waived unless there was objection.

President Herzog announced, with regret, the following had been lost to the Society by death during recent months:

Thomas R. Camp, elected a member October 14, 1930, who died November 19, 1971

Charles C. Cain, elected a member May 18, 1948, who died September 12, 1971

Theodore M. Beach, elected a member September 21, 1909, who died during 1971

Frederick S. Gibbs, elected a member February 18, 1942, who died November 19, 1971

Clyde W. Hubbard, elected a member June 20, 1950, who died November 12, 1971

William G. Horton, elected a member June 21, 1944, who died August 5, 1971

James M. Mullarkey, elected a member February 16, 1966, who died June 6, 1971

Atle Bene, elected a member January 22, 1964, who died August 21, 1971

Ashley Q. Robinson, elected a member December 19, 1917, who died January 17, 1972

Richard E. Cavanaugh, elected a member November 16, 1966, who died February 15, 1971

The members present stood for a moment of respectful silence in memory of the deceased.

President Herzog called upon the Secretary for any business that he might have. The Secretary moved, it was seconded and VOTED "that the Board of Government of the Boston Society of Civil Engineers be authorized to transfer a sum not to exceed \$8,000 from the principal of the Permanent Fund to the Current Fund to meet Current Expenditures".

The Secretary then announced that applications to membership had been received from the following:

Hector Zayes-Bazan, Jamaica Plain, Mass.

George R. Bird, North Tonawanda, New York

Thomas L. Brown, Cambridge, Mass.

Domenic Thomas Bua, Peabody, Mass.

Robert E. Germain, Needham, Mass.

Joseph A. Handziak, Taunton, Mass.

Gino L. Mastrabattista, West Roxbury, Mass.

John F. McFarland, Cambridge, Mass.

Arnold F. Thibault, Coventry, Rhode Island

Christian A. Wethe, Lewes, Delaware

Paul J. Traudeau, Needham Heights, Mass.

James W. Weaver, Reading, Mass.

Paul A. Dipietro, Somerville, Mass.

Thomas C. Anderson, Arlington, Mass.

Richard H. Bender, Framingham, Mass.

Transfer from Grade of Junior

Richard F. Desrosiers, Pembroke, Mass.

Albert B. Pincince, Pasadena, California

President Herzog then returned the conduct of the meeting to President Barton of the American Society of Civil Engineers who introduced Mr. Rogers, District

Director of A.S.C.E. Mr. Rogers discussed the removal of Article 3 from the Code of Ethics of the A.S.C.E. He also discussed the revision of Bulletin 15 which is published by A.S.C.E. In addition he discussed a wage guideline which is published by the A.S.C.E. Members of the Mass. Section discussed committee organization which was underway.

President Barton introduced the speakers for the evening, Mr. Anthony J. DiSarcina and Mr. Lawrence Whipple. The speakers are Associate Members of the American Society of Civil Engineers and co-chaired a Mass. Section Committee "Compensation Task Force". In November 1971 that Committee published a bulletin entitled "Guidelines for Wages and Fringe Benefits". Messrs. DiSarcina and Whipple described the

formation and the activities of the Committee. They also described the conclusions reached leading to many of the statements made in the publication. Following their discussion of their Committee's efforts, the members present engaged in a spirited discussion of the entire matter of wage and benefits for Civil Engineers.

Seventy-one members and guests of both societies attended the dinner, and seventy-nine members were present at the meeting following the dinner.

President Barton adjourned the meeting at 9:50 P.M.

Paul A. Dunkerley  
Secretary





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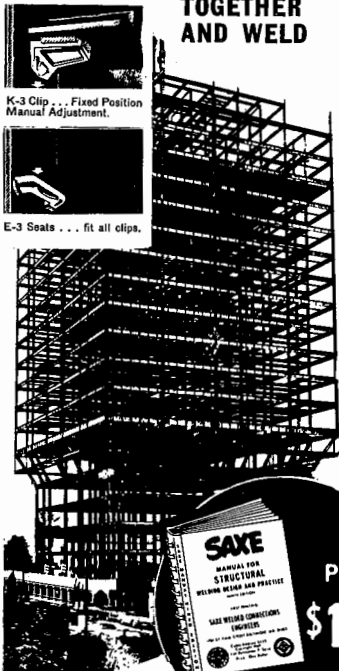
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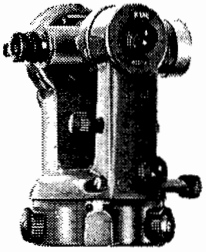
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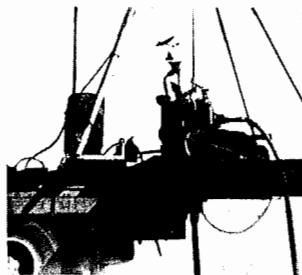
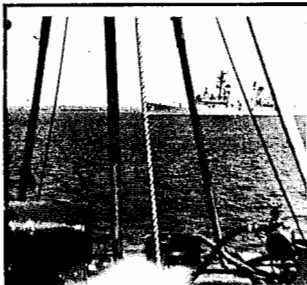
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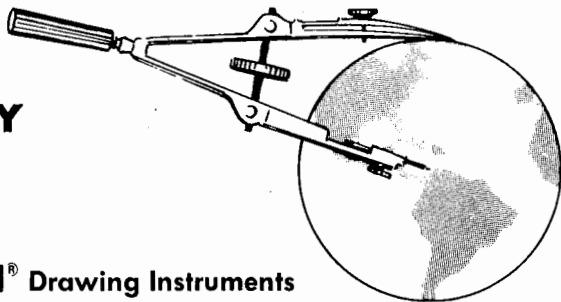
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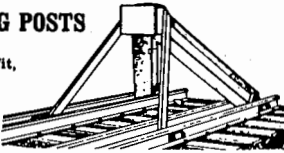
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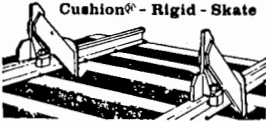
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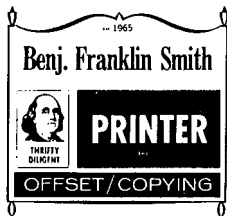
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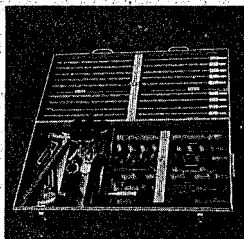
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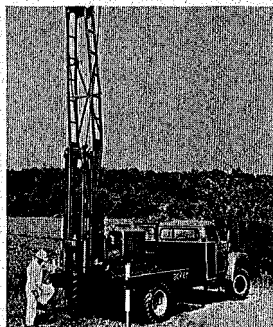
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