SIMULATION MODEL FOR THE
UPPER WABASH SURFACE
WATER SYSTEM

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SIMULATION MODEL FOR THE UPPER WABASH SURFACE WATER SYSTEM

COMPLETION REPORT ON OWRR-A-012 AND 016-IND

PREFACE AND ACKNOWLEDGMENTS

This report presents a part of the work on the application of systems analysis methods to surface water management in Indiana. This effort was supported by the Department of the Interior under projects OWRR-A-012 and 016-IND during the period of 7/1/69 through 6/30/72.

A water resources simulation study of the type with which this report is concerned requires resources that go well beyond those associated with the above projects, at least if one seeks routine application of the model in a water management situation. Only a beginning was made in building a complete simulation model and more was not intended. Even so the material presented could not have been obtained if it were not for the very willing cooperation of many water resource specialists of state and federal agencies. We would like to acknowledge especially the considerable help extended by Bill Andrews, Bob Jackson, Dr. Pasad, Jim Russel, John Simpson and Victor Wenning of the Indiana Department of Natural Resources, Indianapolis, Indiana; Fred Druml, Bill Leagan and Colonel Rhett of the Louisville District Office, Corps of Engineers, Louisville, Kentucky; and Malcolm Hale and Dick Hoggatt of the USGS District Office, Indianapolis, Indiana.

The project was administered by the Purdue Water Resources Research Center, Dr. Dan Wiersma, Director. Drs. Toebes and Chang were principle investigators and A. Lobert, Dave Lyman and J. Walsh served as Graduate Research Assistants.

ABSTRACT

The construction of a digital regional reservoir-river simulation model is described. Simulation as a systems analysis tool is discussed in an introductory way. A comparative discussion of river routing models is given. The best selection of local routing coefficients is discussed when based only on daily flow data. The problems posed by not using precipitation data are stated. The model simulates the authorized surface water management plans
and existing reservoir operating policies for the 8,000 mi² Wabash River Basin upstream of Williamsport, Indiana. It is used in testing systems operating policies for the Upper Wabash Basin.
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SIMULATION MODEL FOR THE UPPER WABASH SURFACE WATER SYSTEM

COMPLETION REPORT #1 ON OWRR-012 AND 016-IND

I. INTRODUCTION

A. ORIGIN AND OBJECTIVES OF PROJECT

This report describes a computer simulation model and background material for the Wabash River and Reservoir System upstream of Williamsport, Indiana. Among the reasons for undertaking this work (in 1969) were:

(a) a concern with the application of systems methodology,
(b) a desire to promote the use of systems techniques to water problems in Indiana,
(c) the belief that models could aid in identifying specific water management problems and data collection needs in Indiana,
(d) the notion that the work would contribute information for the then pending dialogue on regional water resource development questions.

Accordingly attention was directed towards:

(a) adapting existing methodology rather than devising new generalities,
(b) concentrating on a well-defined regional situation with the potential to extend used methods throughout the State,
(c) presenting results in a user's fashion,
(d) aspects of the flood control, quality control and recreation benefit questions; for the purpose of simulation model development other systems objectives are of lesser importance.

Since exposition of concepts is one of the goals, this report is limited to introductory material and an account of the simulation modeling effort. There are four companion reports. These deal with: i. modeling of biochemical oxygen demand in the Wabash River; ii. the question of how to best operate the surface water system; iii. the engineering-economic evaluation of model results; and iv. the structure of the computer programs that make up the quantity model.

Attention was given to (iv) by editing the developed computer programs. Because of duplicating costs the listings and card decks have not been made part of this completion report. They are available on request from the Purdue Water Resource Research Center.
B. SIMULATION MODELS AND SCOPE OF PROJECT

The modeling work was limited primarily to simulation of the physical system. Except for the recreation aspect, demographic, employment and other regional interactive modeling was not considered.

Within these limits the scope of the project can be outlined using Figure 1. Basically a physical water resource simulation model consists of a large collection of digital computer programs. At its center (block 22 of Figure 1) one finds a carefully integrated set of computer programs in the form of "coded relationships" that mimic the behavior of the physical components which make up the surface water system within the selected boundary of the system. These components are reservoirs, outlet works, river reaches, gaging stations, levees, treatment plants, and other such sub-systems. In the present work the size or scale of these components, called the design or "decision variables" (these form a part of the systems input variables), are not at issue. Only the presence or absence of reservoirs not yet built make for systems variants**. Consequently, major attention is directed to the role of reservoir operating policies (block 23, Figure 1) and their approximate optimization (block 30, Figure 1).

The simulation model is of the time sequenced, recursive type. This implies that the condition of the system is computed at regular time intervals, $t_i$, and that, furthermore, feedback features are present which make for interactions between calculations at adjacent time intervals $i$.

The system condition is given in terms of reservoir levels, river stages, river flows, and other so-called state variables. The interval $\Delta t$ is selected equal to 1.0 day in view of the interest in transient system behavior, namely flood propagation, as well as the availability of flow records on a daily basis.

---

* i.e. the boundary of the Wabash River Watershed upstream of Covington Gaging Station, Indiana, near Williamsport.

** In other words: the reservoirs as designed by the Corps Of Engineers are accepted as presenting optimal site developments from the systems point of view. Although a contradiction in terms, this reflects more or less the practical situation, where public acceptance or opposition determine development in a yes-no fashion.

*** True optimization is not feasible using systems simulation; explicit optimization by programming techniques may at best be applied to some facets or to starkly simplified versions of the model, however.
FIGURE 1 - A COMPLETE SIMULATION MODEL
The (model) system output (block 24, Figure 1), often called the *endogenous variables*, has a variety of physical dimensions and is generally so voluminous as to be in need of *statistical analysis* (block 25, Figure 1) prior to further *evaluation* of system performance (block 31, Figure 1).

The model system "runs" under the influence of a main program that provides sequences of daily *input* data (block 21, Figure 1) usually called the *exogenous variables*.

These sequences, e.g. daily reservoir inflows, river side-inflow, etc., are frequently provided by *Generators of flow sequences* that duplicate the statistics of original field data (block 20-11+10). Alternatively one can (as was done in the present work) use the historic data collection (block 10, Figure 1) or also individualized events of different statistical expectation (block 11+22).

The physical system can be thought of as embedded in an economic system. This, in turn, is embedded in a political and an ecological system. The physical model system input and output require translation into economic, sociological or ecological terms since the objectives they are to serve are expressed in those terms. Rapid translation requires separate *evaluation programs* (block 31, 32, Figure 1); these are more difficult to specify objectively than the physical system algorithms. The comparison of their output (block 34) is used in the building of the simulation model while ultimately their judgment is the province of *decision makers* (block 35, Figure 1) who are, supposedly, entirely divorced from the modeling effort.

In summary, this report and the BOD-DO simulation report are primarily concerned with Figure 1-block 22 and involve aspects of blocks 10, 11, 21 and 23. The companion report on the Upper Wabash systems operation relates primarily to Figure 1-block 23 and involves aspects of blocks 11, 30 and 25. Finally, the companion report on engineering-economic evaluation relates primarily to aspects of Figure 1-block 31.

The portion of Figure 1 enclosed in the dashed border will be called the *physical systems model*; the total of Figure 1 represents a "complete" simulation model. The work reported herein covers a complete simulation model only very partially. A "complete" modeling effort requires 10 to 50 times more resources than were available. Only a beginning was made. Not more was intended.

---

* i.e. generated internally in the model
** i.e. given externally and independent of the model
It should be stated clearly that Figure 1 basically refers to simulation of engineering options in surface water management. Regional economic parameters are specified separate from the model. No interactive coupling or feedback between a water sector and regional demographic or employment sectors are incorporated. The model would, however, be a suitable component of a multi-sector or regional planning model. In fact, this prospect was one of the motivations for undertaking the project.

II. BACKGROUND MATERIAL

Although of recent data, the writings that deal with the formulation, use and results of water resource systems models is exceptionally diverse and voluminous. Annotated bibliographies (Kriss and Loucks, 1969; Dracup et al., 1979; Chow and Meredith, 1969; Gygi and Loucks, 1969; Enviro Control Inc., 1971) cover publications appearing at a rate of some 300 per year. Therefore, the following is limited to: (a) brief characterizations of systems modeling aspects; (b) the mention of corresponding key references; and (c) a listing of work relevant to the construction and use of a Wabash Basin system as well as component simulation model.

A. SIMULATION AS SYSTEMS ANALYSIS TOOL

Systems analysis of a specified technological arrangement has two main goals. These are: (a) the simultaneous consideration of all its components in terms of the stated goal or objective to be pursued by means of that system; (b) the best possible way to implement the objective. Stated another way, the quest is for: (a) a systems view; and (b) optimality.

These goals are, in part, mutually exclusive. Strict optimization, requires approaches evolved from operations research. Their use demands complete neglect of many aspects of the real-life system. This is because otherwise the model complexity would preclude successful application of explicit optimization methodology. Needless to say that the resulting solutions may not solve the real problem. In fact, few optimality analysis results have thus far been applied in water resource management.

On the other hand, the retention of a great amount of detail*, as is possible with simulation studies, makes strict optimization infeasible. Rather,

---

*Simulation models can incorporate all kinds non-linear mathematical relationships, stochastic variables, and complex interdependences, because the equations need not be solved. In simulation they need only to be incremented.
results of a simulation model's runs may be likened to data obtained in a laboratory tests for which conditions are changed from test to test. For water systems the number of possible conditions* is generally impractically large. Generation and inspection of all corresponding outputs is not feasible. Hence the optimal systems or optimal operating policy cannot be obtained with certainty.

Simulation can be called both the "most powerful systems analysis tool" as well as an "admission of defeat" or "the end of the rope". The two negative statements reflect simulation's many difficulties. These are (a) the need for different disciplines to cooperate; (b) the very large amount of computer programming; (c) the large input data needs; (d) the large volume of output data; (e) difficulty in accounting for the influence of the "systems environment" that was not modeled; and (f) the (statistical) interpretation problems of the output.

Nevertheless, when it comes to day to day decisions, simulation is favored as a sound approach to identifying incremental improvements. Explicit optimization models may handle portions of the simulation model or they may be used as a screening device for finding the conditions that may be profitably simulated.

Finally, it is noted that simulation efforts can pay dividends to water agencies in terms of agency data management from acquisition through retrieval and assessments of data value.

B. WATER SYSTEM SIMULATION

A reservoir-river system is only one component or sub-system of the total physical water system. In terms of modeling, this is clarified by Figure 2. It states that, in addition to the Surface Storage and Conveyance System Models and the attendant Surface Water Quality Models, (blocks 4 & 5), there exist Atmospheric, Watershed, Sediment Production, Subsurface Storage and attendant Groundwater Quality Models (blocks 1, 2, 3, 6, 7). If now a Surface Storage and Conveyance Model (or "RR-model" for short) is chosen as our system of immediate concern, the other components of the larger water system remain nevertheless present in the form of (exogenous) inputs to and (endogenous) outputs from our system. Such inputs and outputs must be accounted for.

* this includes changes in design parameters or decision variables as well as hydrologic or exogenous input.
FIGURE 2 - A TOTAL WATER SYSTEM SIMULATION MODEL (after Ackerman, 1969)

The physical conditions and the selected objectives led for the Wabash RR Model to a simple accounting, namely elimination of all but one type of input and one type of output. Sediment pickup and deposit, evaporation losses, interbasin and interstate water transfers, return of used flows, and surface-groundwater flows were ignored as input and output variables. The respective reasons are: the lack of sufficient sediment data; the evaporation from the region's lakes just about equals the precipitation on lake surfaces*; the as yet single, and minor example of interbasin transfer in the system; the low consumptive use; and the present lack of administrative arrangements for aquifer systems management. Thus the only preserved ties are those between block 2 & 4 and block 4 & 8 in Figure 2, representing the Wabash RR model's hydrologic input and output, respectively.

Where the output is both the product of the model and the object of later evaluation studies it remains here to account for the choice of model input

* Perry and Corbett (1956)
and hence the possible choice of one or more watershed models. Such models are the domain of scientific hydrology which has evolved very rapidly during the last decade, particularly in terms of analysis. Consequently, there is seemingly available such a variety of models that only an abstract classification is presented in Figure 3. The basic division is between deterministic and probabilistic models. Their respective uses are: (a) forecasting, i.e. transforming a given event or a model input sequence into (always the same) output event \( D(t) \); and (b) prediction, i.e. generating from a specified set of preceding events, a probabilistic series of alternative output events \( R(t) \), having some desired statistical properties. In general models represent a mixture of deterministic** and probabilistic*** elements; they are called stochastic models (blocks 2,3, Fig. 3). Stochastic models yield predicted values of a variable \( V(t) \) or of an interrelated**** variable set \( V^i(t) \):

\[
P(V^i(t)) = D(t) + R(t), \quad i=1,2,3 \text{ say.} \tag{1}
\]

where \( D(t) \) is the expected value \( E(V^i(t)) \) and where the relative importance of the deviation \( R(t) \) varies from zero to 100% when progressing from purely deterministic to purely probabilistic models. The term "model" implies, of course, that the relative importance of \( R(t) \), reflective of the lesser or greater stochastic character of the model, depends on the data available and the kind of problem to be solved. Since knowing or accounting for all influences is infeasible, natural phenomena are always stochastic.

As in other disciplines one employs in hydrology additional subdivisions of the total watershed model or of its components. When there is no regard for the details of actual systems behavior and the (assumed algebraic) model is based only on analytical input-output comparisons to fix model parameters, one deals with a lumped parameter "black box" model (blocks 6,7, Fig. 3). On the other hand, when physical laws and other concepts about systems structure are introduced one deals with a synthesized or "conceptual" model. These may be either of a "lumped parameter" or a "distributed parameter" type; the latter

\[**\text{most models can be regarded either as special cases of or as different presentations of a general model, the ARIMA model (for autoregressive-integrating-moving average model); for a discussion specifically directed towards hydrology, see Dooge (1972).} \]

\[**\text{the term "deterministic" as defined above, does not imply that the model construction did not require statistical considerations; in fact, statistics is essential in model identification and parameter selection.} \]

\[***\text{the probabilistic output from a watershed model is generally digested deterministically by simulation models.} \]

\[****\text{see Chow and Kareliotis (1970).} \]
FIGURE 3 - ABSTRACT OR MATHEMATICAL MODELS
refers to models that are themselves broken down in interconnected sub-models each having its own parameters. Further classifications, important from the mathematical model building point of view, are those of "linear" versus "non-linear" (blocks 8 thr. 15, Fig. 3) and "time invariance" versus "time dependence" (of model parameters) (blocks 16 thr. 23). Models of watersheds or components thereof can be classified only partially by Figure 3 as mixed forms are common. Vigorous work continues in model identification, involving much model output recycling within Figure 3. Survey type information can be found in review papers by Dawdy (1969), Kisiel (1969), Eagleson (1969), Amorocho (1969) and Chow (1969). The interest in the sequel, however, is the possible use of these models' outputs to provide exogenous input to the Wabash Reservoir-River System Model.

C. INPUT FOR WABASH RESERVOIR-RIVER SYSTEM MODEL

An initial purpose of the Wabash RR Simulation Model is to study the adaptation, using a reservoir system, of stochastic hydrologic inputs to outputs of more desirable time, space and quality distributions. The selection of input is a critical part of the study. Figure 4 illustrates various possible choices.

One could start with output from atmospheric models \( P_m(t) \), Fig. 4. Alternatively one can use historic raingage data \( P_n(t) \). These atmospheric model or the historic precipitation data would then constitute the inputs to one or more watershed models, either directly \( P_a(t), 2+a \), or as individualized events \( D(t), 3+b \), or as generated* sequences \( D(t) + R(t), 4+c \).

For the input to the RR model one can start with output from watershed models, \( Q_m(t) \). Alternatively one can use historic riverflow data, \( Q_n(t) \). The watershed model output or the historic flow data would then constitute input to the RR model, either directly \( Q_a(t), 6+a \), or as individualized events \( D(t), 7+b \), or as generated sequences \( D(t) + R(t), 8+c \).

The choice of input to the RR model depends on the purpose one has in mind. When the interest is in building an RR model (i.e. in the model identification phase or recycling in Figure 3) one will use historic records, \( Q_n(t) \), for flow, heat, pollutants, etc. in order to obtain estimates for the systems parameters. When the interest is in validating the model one may use

*Schaake et al. (1972) recently presented methodology to generate precipitation series at multiple sites including ungaged ones and using daily through yearly time steps.
FIGURE 4 - POSSIBLE INPUTS TO THE RESERVOIR AND RIVER SYSTEM MODEL OF BLOCK 9

The model input code is: a = historic data, b = individualized events, c = stochastic data.

Some correspondences between Figures, shown in vertical columns by block numbers, are:

Figure 1: 10 11 20 22 23 31,34
Figure 2:  2→4 1→2→4  4,5 9 –
Figure 3: – 21 22 11,17 – –
Figure 4:  2, 6, 10 3, 7, 11 4, 8 9 13 12
portions of the historic record, \( Q(t) = Q_h(t) \), or also some simple functional input, \((D(t), \gamma \rightarrow b)\), having some order of magnitude relationship to \( Q_h(t) \).

When the interest progresses to evaluating the system model output \( O_m(t) \), i.e. in providing input to evaluation models \((a, b, c \rightarrow 12)\), one will first choose \( Q(t) = Q_h(t) \) as model input to obtain \( O_m(t) = F_m[Q_h(t)] \) in order to verify existing design forecasts.* When one desires a first estimate of systems performance one will choose some statistically derived set \( D(t) = F_m[Q_h(t)] \) to obtain \( O_m(t) = F_d[D(t)] \) for evaluation model input \((10+a+12)\). When more accurate evaluation is desired one will choose generated input, \( V(t) = D(t) + R(t) \), to obtain, using the RR system model, \( O_m(t) = F_v[D(t) + R(t)] \) sequences for evaluation model input, either directly \((10+c+12)\), or after further statistical analysis \((11+b+12)\).

This report deals mainly with the building and with a very preliminary evaluation of an elementary Wabash RR system model. Consequently historic flow data \( Q_h(t) \) and some \( F_m[Q_h(t)] \) in the form of gamma functions were selected as model input. Since work on the evaluation models is still under way it would make little sense to already construct and use stochastically generated input.

In later choosing between the flow-generator input \((8+c)\) or the (precipitation-watershed-flow)-generator input \((4+c \rightarrow 5 \rightarrow 6 \rightarrow a)\) one will have to consider the trade-off between a more "representative" model input and the extra (watershed) modeling effort. In view of the lack of reported studies that compare the output statistics of the two generation methods, it may be hard to make a choice. Consequently, when the need arises for evaluation with long series of \( O(t) \), the generator \( 8 \rightarrow c \) may be selected first as it entails less effort.** This does not imply that more complete models would not be worth their cost. In fact, large basin models that start with atmospheric phenomena and carry through via event simulation to evaluation, i.e.:

\[
1a \rightarrow 2 \rightarrow 3 \rightarrow 5b \rightarrow 6 \rightarrow 9a \rightarrow 10 \rightarrow 12a
\]
do exist (e.g. Thomas, 1971). They have proven to be useful.

Water quantity and quality management are intertwined. Pollution is a relative term. It is perhaps best characterized as a "stress tensor" as shown in Table 1:

---

*existing designs are based on historic records and not on extensive input cross-correlation or stochastic data generators.

**reference is made to complete models; simplifications or approximation have already been used or else are implied in the text that follows.
Table 1 - Components of Pollution

Despite these many facets, but two components are generally used in simulation studies, namely dissolved oxygen, DO, and temperature. In addition one may find the simplest of accounts for conservative waste, namely dilution. The many intermediate conditions between readily degradable and wholly conservative waste, the chemistry of the great variety of waste composites and their interactive bio-dynamics have so far not been modeled in large systems simulations. At the same time, the refinements of DO simulation with its dependence on local empirical rate constants can be said to outstrip the availability of data required for model validation and verification. Fairly dense networks of continuous samplers are needed, but seldom found.

The variability of degradable waste input is generally ignored (statistics on this topic are hard to get) as is the effect of temporal flow variation (steady state assumption and a 10-day low flow period are nearly universal). Data on thermal and conservative waste loading at time and space intervals that are sufficiently small for model validation, appear yet less available. The Wabash River is no exception in this regard.

D. River-Reservoir System Simulation Models

A fair number of successful river-reservoir simulation studies have been reported. Most include several engineering-economic evaluation models that are part of the program. The Columbia River simulation and hydropower evaluation (Lewis and Schoemaker, 1962) is an example of an extension of classical engineering methods under influence of computer availability. This was
successively followed by work that incorporated: (a) stochastic data generation (blocks 4, 8, Fig. 4); (b) probabilistic and more refined evaluation models, (block 12); (c) elements of mathematical optimization techniques; (d) multi-objective considerations; and (e) increasingly sophisticated watershed model components and forecasting (blocks 30, 33, 34, 35, Fig. 1).

The pioneering effort remains the text by Maass et al. (1962); it was refined and applied to the Lehigh River System by Hufschmidt and Piering (1966). Examples of large scale quantity simulations by Federal agencies are the generalized HEC-3 program (Hydrologic Engineering Center, 1969) which is a highly flexible program collection for the analysis of multi-purpose, multiple reservoir systems using a search technique of successive satisfaction of priority needs. State resource management agencies have also employed large scale simulation. Notable examples are the work on the California Water Plan, the simulation of New York's Oswego Basin (Liu et al., 1972), the Texas Water Plan programs (Evanson and Mosely, 1970) which include explicit optimization algorithms (e.g. SYMILD), and efforts at a lesser scale in Illinois (Ackerman, 1969), Ohio and Wisconsin. The tendency is towards a merging of RR models and hydrologic watershed models (see e.g. Crawford and Linsley, 1964; Caborn and Moore, 1970; Chow, 1969; Dujardin, 1972; O'Donnell et al., 1972; Beard et al., 1972). A specific example is the Tibbee River Basin (Miss) simulation program; it involved the application of precipitation patterns to a set of some 250 unit hydrographs in the basin to construct basin wide river stages as they responded to river modification in selected reaches (Thomas, 1971). Packer et al., (1969) exemplified one of a growing number of mixed analog-digital basin simulation studies.

Early efforts to include quality simulation are the studies on the Merrimac River Basin (Bower, 1962) and work by Montgomery and Lynn (1964). Typical of later generalizations are reports by Pisano (1968), Davis (1968), Feigner et al., (1972), and Orlob (1972). Combined quantity and quality simulation plus mathematical optimization is reported by Sigvaldason et al. (1972). A state of the art review is given by Pentland et al. (1972).

Representative of planning studies in which the total water sector is a component are the 1966 study by Hamilton et al. for the Susquehanna basin and the recent NAR (1972) studies covering the New England States.
E. COMPONENTS OF RIVER-RESERVOIR SIMULATION MODELS

Distinct components or processes in a system that is being simulated require their own component or sub-simulation programs. A fair sized literature exists for the common components of river-reservoir simulation models.

a. Reservoir Simulation

The most accurate and simplest simulations are possible for the simulation of the storage function of reservoirs. In general, reservoir flow dynamics can simply be concentrated in the discharge equations for outlet works, such as valved discharge conduits, syphons, gated and non-gated overflow spillways. Given input and release data the simulation is quite elementary. Considerably more difficult to model are the effects of sediment inflow. The amount and location of lost storage, which is hard to forecast, does effect a reservoir's contribution to systems objectives.

Good progress has been made in simulating temperature stratification in reservoirs [Slotta et al., 1969; Macofsky and Harleman, 1971; Ryan and Harleman, 1971]. Versatile programs for simulation of water quality parameters in reservoirs as function of operation, temperature, and inflow constituents do exist (Orlob, 1972).

b. River Simulation

With reference to Figure 10 there are two primary types of river flow simulations. The first is 2→4→8 or 9→16 or 17→24, i.e. the deterministic, black box, flow forecasting model, known as hydrologic routing. The second is 25→2→5→10 or 11→18 or 19→24, i.e. the deterministic, conceptual, flow forecasting model, known as hydraulic routing. The concepts that go into the hydraulic routing are in turn models of the type 3→7→15→22→25, as indicated in Fig. 10 by 25→2. In all flow as well as in quality routings the total river needs to be discretized in a succession of reaches. The total simulation model is therefore of the distributed parameter type. The basic model for a single reach may itself be of a lumped or distributed parameter type.

In regard to conservative waste the mean convective velocity and turbulent mixing can be simulated readily provided a minimum of empirical data is available. The model is of the type 3→6→12→20 or 21→25 (Fig. 10). Its parameters are also needed in the simulation of non-conservative waste transport (25→3→5→11→19→24). The transport and assimilative capacities for bio-degradable
waste of rivers have long been regarded as a natural resource factor. Sophis-
ticated simulation models, both deterministic and stochastic, are presently
available. Descriptive state-of-the-art sources are Frankel et al. (1965),
Orlob (1972) and Pentland et al. (1972). Among readily available programs
and documentation for both conservative and degradable waste are EPA's DOSAG
and the TWDB's QUAL-1 (Texas Water Development Board, 1970; Marsh, 1971).

F. WABASH BASIN STUDIES

A fortunate circumstance in building a quantity simulation model for the
Upper Wabash River Basin is the considerable database that is available.

In the basin, outlined in Figure 5, the U.S. Geological Survey maintains
as many as 36 stream gaging stations. These have an average record length
of over 30 years. There are also more than 20 climatological stations main-
tained by the U.S. Weather Bureau.

Perrey and Corbett (1956) published a detailed account of the hydrology
of Indiana lakes which includes lake evaporation, lake temperature profiles
and lake level fluctuations.

Based on their gaging station records, the U.S. Geological Survey has made
low flow frequency (Hoggatt, 1962) and flood frequency analyses (Green and
Hoggatt, 1960; Speer and Gamble, 1965).

In early 1968, the Indiana Department of Natural Resources initiated the
State Water Plan. The primary purposes were to appraise the existing uses,
the available quantity and the present quality of the surface and ground water
resources. The ultimate objective was to develop long range plans. Three
reports have so far been published. They deal with agricultural irrigation
(Indiana DNR, 1969), institutional arrangements and programs (Indiana DNR,
1970a) and population projection (Indiana DNR, 1970b).

During 1968, the Indiana Department of Natural Resources hired Harza Engi-
neering Company to study flood plains along the Wabash River and Wildcat Creek
near Lafayette. The purpose was to determine the floodway limits for the con-
dition that the Wabash would be conveying a flood corresponding to 6 inches
of runoff from the entire basin.

The Institute for Water Resources of COE published a study (Boxley, 1969)
that developed a regression model to evaluate flood protection benefits based
on land prices in the Wabash River Basin. The purpose was to provide an al-
ternative to the laborious flood damage studies that are usually needed.
Since 1957, the Indiana State Board of Health (ISBH) has maintained a biweekly quality data sampling program in the Wabash Basin. There are thirteen stations upstream of Williamsport. At these stations samples have been collected for dissolved oxygen (DO), five day biological oxygen demand (BOD), suspended solids (SS), temperature (F°), pH, alkalinity, nitrates, phosphates, total coliforms and algae counts (ISBH, 1957-1969). These thirteen quality stations are shown in Table 2.

<table>
<thead>
<tr>
<th>STATION NO.</th>
<th>STATION NAME</th>
<th>DRAINAGE AREA (MI²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ELL-7</td>
<td>Eel River nr. Logansport</td>
<td>791</td>
</tr>
<tr>
<td>MS-27</td>
<td>Mississinewa River nr. Marion</td>
<td>(27 miles above confluence with Wabash)</td>
</tr>
<tr>
<td>MS-28</td>
<td>Mississinewa River at Marion</td>
<td>(28 miles above confluence with Wabash)</td>
</tr>
<tr>
<td>S-2</td>
<td>Salamonie River nr. Lagro</td>
<td>553</td>
</tr>
<tr>
<td>TR-6</td>
<td>Tippecanoe River nr. Delphi</td>
<td>1,857</td>
</tr>
<tr>
<td>TR-48</td>
<td>Tippecanoe River nr. Ora</td>
<td>839</td>
</tr>
<tr>
<td>WB-271</td>
<td>Wabash River nr. Covington</td>
<td>8,208</td>
</tr>
<tr>
<td>WB-312</td>
<td>Wabash River at Lafayette</td>
<td>7,274</td>
</tr>
<tr>
<td>WB-354</td>
<td>Wabash River at Logansport</td>
<td>3,751</td>
</tr>
<tr>
<td>WB-370</td>
<td>Wabash River nr. Peru</td>
<td>2,655</td>
</tr>
<tr>
<td>WB-409</td>
<td>Wabash River nr. Huntington</td>
<td>710</td>
</tr>
<tr>
<td>WC-3</td>
<td>Wildcat Creek nr. Lafayette</td>
<td>791</td>
</tr>
<tr>
<td>WC-44</td>
<td>Wildcat Creek nr. Kokomo</td>
<td>245</td>
</tr>
</tbody>
</table>

**TABLE 2** ISBH QUALITY GAGING STATIONS IN THE UPPER WABASH BASIN

Statistical analyses of DO, F° and flows Q from the ISBH records in Table 2 were made by Pollack (1965) and by Nedved (1966). Pollack (1965) obtained a linear regression \( Y = a + bx \) (see his Table 24), where \( Y \) is yearly average of DO, F° or Q, and \( X \) is the year (varying between 1957 and 1962). Nedved (1966) followed this up with multiple regressions \( Y = a + bx + cx^2 + dx^3 + \log_e(x) \), where \( Y \) and \( X \) denote the same variables.

A stream quality simulation model for the Upper Wabash River has been built by Weeter (1971). Weeter's simulation program is capable of calculating DO and BOD at selected stations for a slug of water flowing through the main stream. The program includes some analyses of water quality con-
trol alternatives, either by flow augmentation or by waste reduction.

Probably the most important references are the 14 volume Wabash Comprehensive Study Report completed in 1971. This study, participated in by all federal Water Agencies, is one of the original sixteen Type II water resource basin studies that are to cover the United States. They implement recommendations in the 1961 report of the Senate Select Committee on National Water Resources. The basic objective of these studies is to promote the development, conservation and utilization of water and related land resources. A focal point is the assurance of coordination of work by the Federal, State, Local government and private business interests. The Corps of Engineers, Louisville District, had the responsibility of coordinating the study. The studies included the identification of preliminary development plans; conservation needs; identification and evaluation of reservoir sites; local protection projects; economic base and evaluation information; quality assessments; recreation, irrigation, and even hydropower potentials.

While there is clearly a wealth of relevant information available in the Wabash Comprehensive Study Reports, it is not disaggregated in a form that can be used without further study in building a simulation model.
III. DESCRIPTION OF THE SURFACE WATER SYSTEM

A. GENERAL DESCRIPTION OF THE BASIN

The upper Wabash Basin, as defined for this report and shown in Figures 5 and 6 is the Wabash Basin above the Covington gaging station. The length of the Wabash between Covington and its origin near Ft. Recovery, Ohio, is 206 miles. The corresponding drainage area is 8208 square miles, or about 25% of the total Wabash Basin. The topograph is gently rolling. Major tributaries are the Tippecanoe, Eel, Salamonie and Mississinewa Rivers and the Wildcat Creek.

There are (potentially) five major, federal-controlled reservoirs in this area; three have been completed (Huntington, Salamonie, Mississinewa) and two are authorized (Big Pine and Lafayette). All of these reservoirs have earth dams. The major reservoir functions are: flood control, recreation and low flow regulation. However, water supply has recently been considered as another function. The controversial proposed sale of 46 million gallons of water daily to the city of Kokomo from Mississinewa Lake, is an example.

Major urban centers in the Upper Wabash Basin are Lafayette, Logansport, Peru, Wabash, Huntington, Kokomo, Frankfort, Marion, Warsaw and Attica. The total population of this basin is 714,000 (1960). Large scale, highly mechanized grain farming is an important sector of the basin's total economy. The upper and eastern portions are generally devoted to dairy and general farming while the southern section is a grain and livestock area.

B. HYDROLOGY AND GEOLOGY

The Basin's climate is humid-continental. The growing season is about 150 days. July, the warmest month, has an average temperature of 74°F. January, the coldest month, has an average temperature about 26°F.

Precipitation, including snowfall, averages about 37 inches annually. High and low precipitation periods are from April to June and from December to February, respectively. Runoff is about 12 inches; the remainder evaporates. Reservoir surface evaporation is also about 26 inches per year.

Floods usually occur from late winter to early spring when ground conditions cause low infiltration and therefore high runoff. The major cause of
floods in this basin is excessive rainfall. Snowmelt, groundwater release and occasionally an ice jam, merely aggravate flood conditions. The maximum flood of record occurred during March and April of 1913.

The Upper Wabash River Basin was covered by the Wisconsin Glacier. The glaciated deposits and comparatively flat topography of the basin lead to relatively high infiltration of the precipitation.

C. STREAMFLOW CHARACTERISTICS

There are 36 USGS gaging stations with an average of 30 years record located along the main stream and tributaries of the Upper Wabash Basin. Detailed information, including drainage area, number of years of the record, average discharge, recorded maximum discharge, recorded minimum dis-

*found in USGS yearly publication: "Surface Water Records"
charge and recorded maximum gage height for each station that was used, is shown in Table 3. The location of each station is shown in Fig. 8.

Water surface profiles in the basin have been developed by Indiana Department of Natural Resources and the USGS. The profiles have been established by using high and low water marks, gage records and field observations. They include: (1) stream bed elevation profiles (Thalweg); (2) low flow surface profiles; (3) low bank line profiles; (4) high water surface profiles for the floods of March 1913, January 1930, May 1933, February 1936, January 1937, March 1937, May 1943, February 1959. These data are useful in projecting future flood damages and also in estimating energy losses (roughness coefficients) in the Wabash River.

D. FLOOD CHARACTERISTICS

From the damage point of view, the most important flood characteristics are stage, duration and time of occurrence. Table 4 lists gage heights and corresponding discharges for the major damaging floods in the Upper Wabash.

The floods of March 1913, May 1943 and January-February 1950 were most severe. The 1913 flood produced the highest stages. It did so in almost all channel reaches of the basin. Every existing levee in the total Wabash River basin was breached. The maximum stages during the 1943 flood approached the March 1913 stages. The duration was appreciably less than in 1913, however. Constrictions of levees and bridges after 1913 were the cause of higher stages. Because of timing the May 1943 flood caused the greatest agricultural damages. The 1950 flood was also extremely damaging throughout the Wabash River basin. Crest stages were generally slightly lower than the 1943 flood. However, its duration was two months.

Table 4 also lists the official flood stages (used by Corps of Engineers) for each gaging station. These are references for comparing flood magnitudes. Flood frequency analyses for most gaging stations in Indiana have been made by Green and Hoggatt (1960) and by Speer and Gamble (1965). Green and Hoggatt (1960) also developed a regional synthetic frequency curve which permits one to estimate the flood frequency for ungaged areas.

E. LOW FLOW CHARACTERISTICS

Low-flow data find many uses. In studies of industrial waste and municipal sewage disposal into streams, low-flow frequency data define the chance
<table>
<thead>
<tr>
<th>Station No.</th>
<th>Station Name</th>
<th>Drainage Area</th>
<th>Stream Name</th>
<th>Period of Records</th>
<th>Ave. Q (CFS)</th>
<th>Q MAX (CFS)</th>
<th>Q MIN (CFS)</th>
<th>S MAX (FT.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>Covington, Ind.</td>
<td>8208.</td>
<td>Wabash River</td>
<td>1939</td>
<td>6936.</td>
<td>147000.</td>
<td>487.</td>
<td>32.44</td>
</tr>
<tr>
<td>7</td>
<td>Lafayette, Ind.</td>
<td>7247.</td>
<td>Wabash River</td>
<td>1927</td>
<td>6203.</td>
<td>131000.</td>
<td>265.</td>
<td>28.47</td>
</tr>
<tr>
<td>8</td>
<td>Delphi, Ind.</td>
<td>4032.</td>
<td>Wabash River</td>
<td>1940</td>
<td>3315.</td>
<td>85300.</td>
<td>158.</td>
<td>25.60</td>
</tr>
<tr>
<td>9</td>
<td>Logansport, Ind.</td>
<td>3779.</td>
<td>Wabash River</td>
<td>1923</td>
<td>3205.</td>
<td>89800.</td>
<td>97.</td>
<td>21.32</td>
</tr>
<tr>
<td>10</td>
<td>Peru, Ind.</td>
<td>2686.</td>
<td>Wabash River</td>
<td>1943</td>
<td>2294.</td>
<td>68000.</td>
<td>62.</td>
<td>24.46</td>
</tr>
<tr>
<td>11</td>
<td>Wabash, Ind.</td>
<td>1768.</td>
<td>Wabash River</td>
<td>1930</td>
<td>1443.</td>
<td>49600.</td>
<td>17.</td>
<td>24.44</td>
</tr>
<tr>
<td>12</td>
<td>Huntington, Ind.</td>
<td>721.</td>
<td>Wabash River</td>
<td>1951</td>
<td>563.</td>
<td>14900.</td>
<td>2.3</td>
<td>23.20</td>
</tr>
<tr>
<td>28</td>
<td>Bluffton, Ind.</td>
<td>532.</td>
<td>Wabash River</td>
<td>1930</td>
<td>388.</td>
<td>11800.</td>
<td>3.9</td>
<td>16.07</td>
</tr>
<tr>
<td>29</td>
<td>Nr. Warren, Ind.</td>
<td>425.</td>
<td>Salamonie River</td>
<td>1957</td>
<td>356.</td>
<td>13200.</td>
<td>5.</td>
<td>17.05</td>
</tr>
<tr>
<td>30</td>
<td>Dora, Ind.</td>
<td>557.</td>
<td>Salamonie River</td>
<td>1930</td>
<td>495.</td>
<td>16500.</td>
<td>0.34</td>
<td>14.75</td>
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<tr>
<td>31</td>
<td>Portland, Ind.</td>
<td>85.5</td>
<td>Salamonie River</td>
<td>1960</td>
<td>64.5</td>
<td>3460.</td>
<td>0.2</td>
<td>16.96</td>
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<tr>
<td>32</td>
<td>Marion, Ind.</td>
<td>677.</td>
<td>Mississinewa River</td>
<td>1930</td>
<td>625.</td>
<td>25000.</td>
<td>1.1</td>
<td>17.40</td>
</tr>
<tr>
<td>33</td>
<td>Peoria, Ind.</td>
<td>808.</td>
<td>Mississinewa River</td>
<td>1953</td>
<td>671.</td>
<td>28000.</td>
<td>5.</td>
<td>19.26</td>
</tr>
<tr>
<td>34</td>
<td>Eaton, Ind.</td>
<td>310.</td>
<td>Mississinewa River</td>
<td>1952</td>
<td>271.</td>
<td>19400.</td>
<td>2.0</td>
<td>18.53</td>
</tr>
<tr>
<td>35</td>
<td>Ridgeville, Ind.</td>
<td>133.</td>
<td>Mississinewa River</td>
<td>1946</td>
<td>125.</td>
<td>13900.</td>
<td>0.1</td>
<td>16.25</td>
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<tr>
<td>36</td>
<td>Nr. Huntington, Ind.</td>
<td>263.</td>
<td>Little River</td>
<td>1944</td>
<td>223.</td>
<td>5990.</td>
<td>1.0</td>
<td>18.43</td>
</tr>
<tr>
<td>37</td>
<td>Logansport, Ind.</td>
<td>789.</td>
<td>Eel River</td>
<td>1943</td>
<td>704.</td>
<td>14200.</td>
<td>65.</td>
<td>12.20</td>
</tr>
<tr>
<td>38</td>
<td>Delphi, Ind.</td>
<td>1857.</td>
<td>Tippecanoe River</td>
<td>1939</td>
<td>1573.</td>
<td>22600.</td>
<td>1.0</td>
<td>15.10</td>
</tr>
<tr>
<td>39</td>
<td>Lafayette, Ind.</td>
<td>791.</td>
<td>Wildcat Creek</td>
<td>1954</td>
<td>690.</td>
<td>25000.</td>
<td>46.</td>
<td>21.52</td>
</tr>
<tr>
<td>40</td>
<td>Nr. Williamsport, Ind.</td>
<td>329.</td>
<td>Big Pine Creek</td>
<td>1955</td>
<td>241.</td>
<td>12600.</td>
<td>6.5</td>
<td>16.00</td>
</tr>
<tr>
<td>41</td>
<td>Owasco, Ind.</td>
<td>390.</td>
<td>Wildcat Creek</td>
<td>1944</td>
<td>351.</td>
<td>10200.</td>
<td>10.</td>
<td>13.30</td>
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<tr>
<td>42</td>
<td>Monticello, Ind.</td>
<td>1710.</td>
<td>Tippecanoe River</td>
<td>1931</td>
<td>1451.</td>
<td>16800.</td>
<td>103.</td>
<td>...</td>
</tr>
</tbody>
</table>

**TABLE 3 - STREAM GAGING STATIONS - MAIN DATA**
<table>
<thead>
<tr>
<th>Station</th>
<th>Flood Period</th>
<th>1913</th>
<th>1930</th>
<th>1933</th>
<th>1936</th>
<th>1943</th>
<th>1950</th>
<th>1958</th>
<th>1959</th>
<th>Flood Stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>G.H. (ft)</td>
<td>35.1</td>
<td>27.7</td>
<td>27.6</td>
<td>28.9</td>
<td>32.4</td>
<td>25.9</td>
<td>30.3</td>
<td>28.4</td>
<td>15.0</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>200</td>
<td>91.2</td>
<td>90.2</td>
<td>104</td>
<td>147</td>
<td>69.0</td>
<td>113</td>
<td>98.1</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>G.H. (ft)</td>
<td>32.9</td>
<td>24.1</td>
<td>22.7</td>
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<td>28.5</td>
<td>25.4</td>
<td>26.3</td>
<td>25.3</td>
<td>22.0</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>190</td>
<td>74.6</td>
<td>67.5</td>
<td>93.5</td>
<td>131</td>
<td>90</td>
<td>99</td>
<td>89</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>G.H. (ft)</td>
<td>28.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>25.0</td>
<td>25.5</td>
<td>27.5</td>
<td>13.0</td>
</tr>
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<td>-</td>
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<td>68.5</td>
<td>61.5</td>
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<td>17.8</td>
<td>14.7</td>
<td>17.9</td>
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<td>17.8</td>
<td>19.7</td>
<td>9.0</td>
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<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>140</td>
<td>61.4</td>
<td>42.5</td>
<td>63.7</td>
<td>89.8</td>
<td>70.7</td>
<td>52.5</td>
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<td>G.H. (ft)</td>
<td>28.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>24.5</td>
<td>22.4</td>
<td>22.7</td>
<td>22.6</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>115</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>68</td>
<td>49</td>
<td>53.1</td>
<td>48</td>
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<td>11</td>
<td>G.H. (ft)</td>
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<td>19.9</td>
<td>20.0</td>
<td>22.4</td>
<td>24.2</td>
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<td>12.0</td>
</tr>
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<td></td>
<td>Q_p (10^3 cfs)</td>
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<td>23.0</td>
<td>23.3</td>
<td>35.6</td>
<td>49.6</td>
<td>39.8</td>
<td>36.3</td>
<td>45.3</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>G.H. (ft)</td>
<td>22.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>20.9</td>
<td>-</td>
<td>19.1</td>
<td>23.2</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>68</td>
<td>11.4</td>
<td>14.9</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>G.H. (ft)</td>
<td>21.0</td>
<td>15.2</td>
<td>12.7</td>
<td>12.8</td>
<td>14.7</td>
<td>16.1</td>
<td>14.2</td>
<td>15.0</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>25.0</td>
<td>11.2</td>
<td>6.62</td>
<td>6.78</td>
<td>9.78</td>
<td>11.8</td>
<td>7.89</td>
<td>9.82</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>G.H. (ft)</td>
<td>-</td>
<td>13.2</td>
<td>11.6</td>
<td>12.7</td>
<td>14.8</td>
<td>12.4</td>
<td>11.9</td>
<td>14.1</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>-</td>
<td>12.7</td>
<td>9.51</td>
<td>11.7</td>
<td>16.5</td>
<td>11.1</td>
<td>12.6</td>
<td>15.6</td>
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</tr>
<tr>
<td>32</td>
<td>G.H. (ft)</td>
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<td>15.0</td>
<td>15.6</td>
<td>11.3</td>
<td>15.1</td>
<td>13.2</td>
<td>16.9</td>
<td>12.7</td>
<td>10.0</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>19.2</td>
<td>20.6</td>
<td>20.6</td>
<td>12.5</td>
<td>20.9</td>
<td>15.2</td>
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<td>14.5</td>
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</tr>
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<td>36</td>
<td>G.H. (ft)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>16.9</td>
<td>15.3</td>
<td>18.4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>5.99</td>
<td>3.54</td>
<td>4.71</td>
<td></td>
</tr>
<tr>
<td>37</td>
<td>G.H. (ft)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>13.2</td>
<td>11.8</td>
<td>10.2</td>
<td>11.5</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>17.0</td>
<td>13.1</td>
<td>8.00</td>
<td>12.3</td>
<td></td>
</tr>
<tr>
<td>38</td>
<td>G.H. (ft)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>13.2</td>
<td>16.9</td>
<td>13.3</td>
<td>14.7</td>
<td>15.9</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>12.0</td>
<td>21.4</td>
<td>22.6</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>39</td>
<td>G.H. (ft)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>21.5</td>
<td>19.4</td>
<td>-</td>
<td>7.0</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>25.0</td>
<td>18.4</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>G.H. (ft)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>13.2</td>
<td>16.0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Q_p (10^3 cfs)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>7.58</td>
<td>12.6</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 4 - PEAK DISCHARGES AND GAGE HEIGHTS OF THE MAJOR**
**RECORDED FLOODS - UPPER WABASH RIVER BASIN**
of occurrence of a flow less than that required to dilute wastes to an acceptable level. Low-flow frequency data are also useful in the selection of conservation (for water supply purposes) pool levels in reservoirs. Analysis of low flow frequencies and low flow durations for each gaging station of the State of Indiana has been reported in a joint report by the USGS and State of Indiana (Hoggett, 1962).

For water quality analysis, a standard low flow situation in Indiana is the "7 consecutive day - once in 10 year frequency" low flow ($Q_{L7}$). This value is summarized in Table 5 for key stations in the Upper Wabash basin. The table also gives the values of $Q_{L7}$ divided by drainage area as well as the lowest instantaneous flow rate on record. In the Wabash Comprehensive Study (WCS, 1971), the storage required for water quality and water supply was based on the five driest years of record.

The average time of travel is an important parameter in quality routing. It is shown in Table 6 (culled from WCS, 1971); the associated flows are probably close to the average riverflow. The table shows that a trip (convective transport) of a conservative waste slug from Huntington to Covington would last about 3.5 days; empirical dispersion information has not been reported.

<table>
<thead>
<tr>
<th>Gaging Station</th>
<th>Mileage from Month</th>
<th>$Q_{MIN}$ (CFS)</th>
<th>7 cons. day-1 in 10 year low flow ($Q_{L7}$) (cfs)</th>
<th>$Q_{L7}$ D.A.</th>
<th>Drainage Area ($M^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>271.1</td>
<td>487</td>
<td>630.0</td>
<td>0.08</td>
<td>8,208</td>
</tr>
<tr>
<td>7</td>
<td>311.9</td>
<td>265</td>
<td>535.0</td>
<td>0.07</td>
<td>7,247</td>
</tr>
<tr>
<td>8</td>
<td>330.8</td>
<td>158</td>
<td>204.0</td>
<td>0.05</td>
<td>4,032</td>
</tr>
<tr>
<td>9</td>
<td>353.7</td>
<td>97</td>
<td>204.0</td>
<td>0.05</td>
<td>3,779</td>
</tr>
<tr>
<td>10</td>
<td>370.5</td>
<td>62</td>
<td>87.0</td>
<td>0.032</td>
<td>2,686</td>
</tr>
<tr>
<td>11</td>
<td>387.2</td>
<td>17</td>
<td>26.0</td>
<td>0.015</td>
<td>1,768</td>
</tr>
<tr>
<td>12</td>
<td>409.1</td>
<td>2.3</td>
<td>9.0</td>
<td>0.012</td>
<td>721</td>
</tr>
<tr>
<td>28</td>
<td>433.9</td>
<td>5.9</td>
<td>4.7</td>
<td>0.009</td>
<td>532</td>
</tr>
<tr>
<td>30</td>
<td>394.25</td>
<td>0.34</td>
<td>11.0</td>
<td>0.020</td>
<td>557</td>
</tr>
<tr>
<td>33</td>
<td>374.85</td>
<td>5.0</td>
<td>33.0</td>
<td>0.041</td>
<td>808</td>
</tr>
<tr>
<td>36</td>
<td>405.95</td>
<td>1.0</td>
<td>1.48</td>
<td>0.006</td>
<td>263</td>
</tr>
<tr>
<td>37</td>
<td>329.2</td>
<td>65</td>
<td>95.0</td>
<td>0.120</td>
<td>789</td>
</tr>
<tr>
<td>38</td>
<td>322.2</td>
<td>1.0</td>
<td>188.0</td>
<td>0.106</td>
<td>1,857</td>
</tr>
<tr>
<td>39</td>
<td>317.05</td>
<td>46</td>
<td>658</td>
<td>0.082</td>
<td>791</td>
</tr>
<tr>
<td>40</td>
<td>287.8</td>
<td>6.5</td>
<td>50</td>
<td>0.152</td>
<td>329</td>
</tr>
</tbody>
</table>

Note: beginning of records - 1959

Table 5 - Low Flow Characteristics for Upper Wabash River Basin
<table>
<thead>
<tr>
<th>Stream Name</th>
<th>Begin Reach (Mile)</th>
<th>End Reach (Mile)</th>
<th>Length (Miles)</th>
<th>Time of Travel (Hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wabash R.</td>
<td>Huntington Damsite 411.4</td>
<td>Huntington 409.0</td>
<td>2.4</td>
<td>1</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Huntington 409.0</td>
<td>Wabash 387.0</td>
<td>22.0</td>
<td>12</td>
</tr>
<tr>
<td>Salamonie R.</td>
<td>Salamonie Damsite 397.4</td>
<td>Wabash R. 394.0</td>
<td>3.4</td>
<td>1</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Salamonie R. 394.0</td>
<td>Wabash 387.0</td>
<td>7.0</td>
<td>3</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Wabash 387.0</td>
<td>Peru 371.0</td>
<td>16.0</td>
<td>12</td>
</tr>
<tr>
<td>Mississinewa R.</td>
<td>Mississinewa Damsite 382.1</td>
<td>Wabash R. 375.0</td>
<td>7.1</td>
<td>2</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Mississinewa R. 375.0</td>
<td>Peru 371.0</td>
<td>4.0</td>
<td>2</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Peru 371.0</td>
<td>Logansport 354.0</td>
<td>17.0</td>
<td>12</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Logansport 354.0</td>
<td>Delphi 331.0</td>
<td>23.0</td>
<td>12</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Delphi 331.0</td>
<td>Lafayette 312.0</td>
<td>19.0</td>
<td>12</td>
</tr>
<tr>
<td>Wabash R.</td>
<td>Lafayette 312.0</td>
<td>Covington 271.0</td>
<td>41.0</td>
<td>24</td>
</tr>
</tbody>
</table>

**TABLE 6 - AVERAGE TIME OF TRAVEL MEASUREMENT RESULTS**
F. RESERVOIR DESCRIPTIONS

The Upper Wabash River basin contains many small, natural lakes, most of which are privately owned and used for recreational purposes.

There are five important existing multiple-purpose reservoirs in the Upper Wabash River basin. These are Huntington, Salamonie, Mississinewa, Shafer and Freeman. Two reservoirs, Lafayette and Big Pine, are currently authorized. The Shafer Lake and the Freeman Lake are operated mainly for the purpose of power generation, recreation, and real estate development by private business (Northern Indiana Public Service Co.); their pool fluctuations and, therefore, storage function, are being minimized. As a consequence they were deleted from this study as an operational variable.

Presently, the major objectives of the five Federal Reservoirs are flood control, recreation and low flow regulation. Uses for municipal and industrial water supply (including interbasin transfers) are also under consideration. The location of each reservoir is shown in Fig. 6. Their structural as well as operational characteristic elements are listed in Table 7. It shows that the Mississinewa Reservoir and the Lafayette reservoir provide more than half of the total storage volume in this basin. However, the Big Pine Reservoir and the Salamonie Reservoir are more effective in controlling their individual basin's flood runoff (in inches) because of their smaller drainage areas.

G. WATER QUALITY

There are 78 municipal sewerage systems in the Upper Wabash Basin. Twelve of these are along the Wabash River itself. At the present time, 51 sewerage systems are served by secondary treatment facilities, 5 by only primary treatment facilities, and 22 are without waste treatment facilities. Most of the self-discharging industry is found in the Lafayette region. Six out of eight of the Lafayette region's plants provide secondary treatment.

Thermal pollution from power generation facilities is not presently a major problem in this area. Steam plants in operation on the main stem of the Wabash include a 12.5 MW capacity plant at Celina, Ohio; a 7 MW plant at Bluffton; a 40 MW at Peru; a 52 MW at Logansport; and a 36.8 MW plant at Frankfort. The Federal Power Commission forecasts construction of a 700 MW station at Lafayette before 1980. Cooling water requirements of this station would be about 840 cfs; this exceeds the $Q_{L7}$ (see Table 5).
### RESERVOIRS

<table>
<thead>
<tr>
<th>Status</th>
<th>Huntington</th>
<th>Salamonie</th>
<th>Mississinewa</th>
<th>Lafayette</th>
<th>Big Pine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Completed 1969</td>
<td>Completed 1966</td>
<td>Completed 1967</td>
<td>Authorized</td>
<td>Authorized</td>
<td>Authorized</td>
</tr>
<tr>
<td>411.4</td>
<td>161.1</td>
<td>3.1</td>
<td>7.1</td>
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<tr>
<td>707</td>
<td>553</td>
<td>809</td>
<td>787</td>
<td>329</td>
<td>329</td>
</tr>
<tr>
<td>FC, Rec, LFR</td>
<td>FC, Rec, LFR</td>
<td>FC, Rec, LFR</td>
<td>FC, Rec, LFR</td>
<td>FC, Rec, LFR</td>
<td>FC, Rec, LFR</td>
</tr>
<tr>
<td>earth, concrete</td>
<td>earth, rock</td>
<td>earth</td>
<td>concrete, earth</td>
<td>concrete, earth</td>
<td>concrete, earth</td>
</tr>
<tr>
<td>91</td>
<td>132</td>
<td>140</td>
<td>116</td>
<td>132</td>
<td>132</td>
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<tr>
<td>9500</td>
<td>4760</td>
<td>8000</td>
<td>3350</td>
<td>4620</td>
<td>4620</td>
</tr>
<tr>
<td>3-gated opening</td>
<td>uncontrolled</td>
<td>uncontrolled</td>
<td>controlled</td>
<td>3-40x40 tainter</td>
<td>4-4'x7' gates</td>
</tr>
<tr>
<td>6-6'x6' gates</td>
<td>16' diam. gate</td>
<td>16' diam. gate</td>
<td>3-5'x10' gates</td>
<td>4-4'x7' gates</td>
<td>4-4'x7' gates</td>
</tr>
</tbody>
</table>

| Sluice gate capacity (cfs) | 12,600 | 12,100 | 12,000 | 6,150 | 12,100 |
| Minimum Release (cfs)      | 20      | 20      | 20      | 20      | 20      |
| Zero Storage Zo            | 718     | 684     | 665     | 545     | 510     |
| Min. Pool Sm               | 737     | 730     | 712     | 580     | 543     |
| Seasonal Pool Zr           | 749     | 755     | 737     | 609     | 570     |
| Spillway Crest Zs          | 765     | 793     | 779     | 645     | 596     |
| Flood Pool Zf              | 798     | 793     | 779     | 645     | 636     |
| Min. Pool                  | 1,100   | 13,100  | 23,300  | 19,231  | 8,990   |
| Seasonal Pool              | 12,500  | 58,250  | 75,200  | 95,380  | 38,500  |
| Flood Pool                 | 168,100 | 254,700 | 368,400 | 332,560 | 210,500 |
| Seasonal Storage(Zm-Zr)    | 8,400   | 45,150  | 51,900  | 76,150  | 29,510  |
| Flood Storage(Zm-Zf)       | 164,000 | 241,600 | 345,100 | 313,320 | 201,510 |
| Min. Pool                  | 0.10    | 0.44    | 0.54    | 0.46    | 0.51    |
| Seasonal Pool              | 0.33    | 1.97    | 1.74    | 2.26    | 2.18    |
| Flood Pool                 | 0.33    | 8.61    | 8.52    | 7.90    | 11.95   |
| Seasonal Storage(Zm-Zr)    | 0.23    | 1.53    | 1.20    | 1.80    | 1.67    |
| Flood Storage(Zm-Zf)       | 4.34    | 8.17    | 7.98    | 7.44    | 11.44   |
| Min. Pool                  | 500     | 976     | 1,280   | 1,320   | 687     |
| Seasonal Pool              | 900     | 2,860   | 3,180   | 4,290   | 1,390   |
| Flood Pool                 | 7,900   | 9,340   | 12,830  | 9,470   | 4,710   |

**TABLE 7 - MAJOR FEDERAL RESERVOIRS IN UPPER WABASH RIVER BASIN**
Corps of Engineer's reservoir operating policies require for all five federally controlled reservoirs a release of at least 20 cfs. This mainly benefits localities directly downstream of the reservoir.

II. SYSTEM OBJECTIVES AND MANAGEMENT

Presently the foremost objective of the reservoir system (in conjunction with the further channel improvement, levees, land treatment and land management) is to reduce flood damage in the Upper Wabash basin as well as along the downstream reaches of the Wabash River. In fact, the major amount of flood damage mitigation benefit is realized in the reaches below Covington Station.

Major floods in the Wabash basin generally occur either in the winter or in the spring. This makes for complementary use of the reservoir system for recreational purposes in the summer and early fall seasons. At present the region has only limited outdoor recreational opportunities. Its population is projected to increase rapidly in the coming decades. Therefore, the need of recreational space, as well as for municipal and industrial water supply and water quality control, may shift the relative weights of priorities away from flood control and towards conservation uses. The study of such developments is an important reason for building a simulation model as considered in this report.

I. OPERATING POLICIES

Detailed COE operating plans exist for each of the 5 (potential) Federal reservoirs. In principle, the upper stream three reservoirs (Huntington, Salamonie, Mississinewa) were to be operated as a single unit so as to obtain maximum flood control benefits consistent with flood storage capacity available. Their primary objective was to reduce flood stages at the cities of Wabash, Peru, and Logansport, Indiana, and about 60,000 acres of agricultural land and related developments. A secondary objective was to reduce the floods of the Wabash River below Logansport (operation in conjunction with the operation of the downstream reservoirs). Consequently, the Logansport stage (station 10) was made the major control variable in the three reservoir operation policy. (see Table 8 and Figure 7)
The major objectives of the Lafayette Reservoir operation is to control the flood stage in the vicinity of Lafayette, as well as at the downstream damage centers of Terre Haute, Mt. Carmel and Evansville. The Big Pine Reservoir operation plan is designed to protect the area near Williamsport as well as the downstream damage centers along the Wabash River and Ohio River. Therefore, the Lafayette-Big Pine policy contains the Terre Haute, Mt. Carmel, Evansville, as well as Lafayette stages as control variables.

The recreation period for Huntington reservoir (located the farthest north) is scheduled to begin on April 15 and to end at September 30 each year. Recreation seasons at the other reservoirs are to begin on April 1 and to end September 15.

In regard to low flow regulation, Corps of Engineer's policy is to release at least 20 cfs for each reservoir in this system. A copy of the most recent Corps of Engineers operating policy for the upper three reservoirs of the basin is shown in Table 8 with corresponding Rule Curves in Figure 7.

The new operating policies devised in this study aim at operating all the five Federal Reservoirs as one unit. The detailed description of the needed policy will be presented in a subsequent report.
<table>
<thead>
<tr>
<th>Schedule</th>
<th>Logansport Stage (Feet)</th>
<th>Pool Elevation (Feet)</th>
<th>Season of Year</th>
<th>Regulation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Below 9 or 2 days before 9^1/</td>
<td>737^3/</td>
<td>(SR) 1 December - 14 April</td>
<td>Release inflow necessary to maintain pool, provided release indicated in Maximum Release Table is not exceeded.</td>
</tr>
<tr>
<td></td>
<td>Below 9 or 1 day before 9^2/</td>
<td>730^3/</td>
<td>(SR) 1 December - 14 March</td>
<td>Release at rate indicated in Maximum Release Table.</td>
</tr>
<tr>
<td></td>
<td>Below 9^3/</td>
<td>712^3/</td>
<td>(HR) 1 December - 14 March</td>
<td>Maintain pool dictated by Rule Curve while meeting flood control and minimum flow requirements. See rule curves on second page of this table.</td>
</tr>
<tr>
<td>B</td>
<td>Same as Schedule A^4/</td>
<td>737-795^2/</td>
<td>(SR) 1 December - 14 April</td>
<td>Release inflow up to rate of 3,000 cfs.</td>
</tr>
<tr>
<td></td>
<td>Same as Schedule A^4/</td>
<td>730-793^2/</td>
<td>(SR) 1 December - 14 March</td>
<td>Release inflow up to rate of 1,000 cfs.</td>
</tr>
<tr>
<td></td>
<td>Same as Schedule A^4/</td>
<td>712-779^2/</td>
<td>(HR) 1 December - 14 March</td>
<td>Release at constant rate of 100 cfs.</td>
</tr>
<tr>
<td>C</td>
<td>9 and above R or 2 days before 9 R</td>
<td>737-798^5/</td>
<td>All year</td>
<td>Regulate sluice gates to induce surcharge to a maximum pool elevation 800. At elevation 800, with sluice gates fully open, raise spillway gates as necessary to prevent pool from rising above elevation 800. When the pool peaks, maintain all gate settings attained until reservoir recedes to elevation 798. At elevation 798 close spillway gates as necessary to pass inflows only (spillway + conduit) and hold 798 pool. When spillway gates are fully closed, regulate sluice gates to continue passing inflows only until outflows are consistent with release rates indicated in Maximum Release Table; then regulate in accordance with Schedule B.</td>
</tr>
<tr>
<td></td>
<td>1 day before 9 R</td>
<td>730-793^5/</td>
<td>All year</td>
<td>Release inflows up to conduit capacity. Inflows in excess of conduit spillway will be uncontrolled through the spillway. Maintain rate settings attained until pool peaks and recedes to elevation 792. At this time, adjust gates to resume passing inflows only until outflows are consistent with allowable release rates indicated in Maximum Release Table; then regulate in accordance with Schedule B.</td>
</tr>
<tr>
<td></td>
<td>9 and above R</td>
<td>712-779^5/</td>
<td>All year</td>
<td>Release inflows up to conduit capacity. Inflows in excess of conduit capacity will be uncontrolled through the spillway. Maintain gate settings attained until pool peaks and recedes to elevation 779. At this time adjust gates to resume passing inflows only until outflows are consistent with allowable release rates indicated in Maximum Release Table; then regulate in accordance with Schedule B.</td>
</tr>
<tr>
<td>D</td>
<td>Not considered</td>
<td>Above 798^5/</td>
<td>All year</td>
<td>Monitoring only.</td>
</tr>
<tr>
<td></td>
<td>Not considered</td>
<td>Above 779^5/</td>
<td>All year</td>
<td>Monitoring only.</td>
</tr>
</tbody>
</table>

1/ Damage state in reach of river referenced to Logansport gage.
2/ Logansport (Station #9) flood stage is 17 feet. When there is a high percentage of storage utilization, and reservoir outflows will not add to crest stages at any station along Wabash or Ohio Rivers, releases may be made when stage is below 17 ft. so long as river continues to fall and release rates indicated in Maximum Release Table are not exceeded.
3/ Minimum pool
4/ Flood pool
5/ Spillway crest.
6/ No storage will be released which will add to crest stages at any station along Wabash or Ohio Rivers.
<table>
<thead>
<tr>
<th>STAGE</th>
<th>LITTLE RIVER NEAR HUNTINGTON</th>
<th>HUNTINGTON RESERVOIR</th>
<th>SALAMONIE RESERVOIR</th>
<th>MISSISSINNEWA RESERVOIR</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft</td>
<td>Release (cfs)</td>
<td>Release (cfs)</td>
<td>Release (cfs)</td>
<td>Release (cfs)</td>
</tr>
<tr>
<td>Below 5</td>
<td>5,500</td>
<td>7,000 - HR</td>
<td>7,000</td>
<td></td>
</tr>
<tr>
<td>5-8</td>
<td>5,000</td>
<td>6,500 - HR</td>
<td>6,800</td>
<td></td>
</tr>
<tr>
<td>8-10</td>
<td>4,500</td>
<td>6,000 - HR</td>
<td>6,600</td>
<td></td>
</tr>
<tr>
<td>10-12</td>
<td>4,000</td>
<td>5,500 - HR</td>
<td>6,400</td>
<td></td>
</tr>
<tr>
<td>12-13</td>
<td>3,500</td>
<td>4,000 - HR</td>
<td>6,200</td>
<td></td>
</tr>
<tr>
<td>Above 13</td>
<td>3,000</td>
<td>3,500 - HR</td>
<td>6,000</td>
<td></td>
</tr>
</tbody>
</table>

1/ These release must be modified as necessary to not produce combined total flows in excess of 8,000 cfs at Wabash, except as prescribed in Schedule C.

2/ Huntington = Station #36

**FIGURE 7 - COE OPERATING POLICY AND RULE CURVES FOR THE THREE UPSTREAM RESERVOIR IN THE UPPER WABASH BASIN**
IV. RIVER COMPONENT SIMULATION

A. SYSTEMS GRAPH AND SYSTEMS STRUCTURE

a. Systems Graph

Any systems analysis starts with drawing a systems graph. This is a graphic that displays the "essentials" of the system. The systems features and its boundary reflect the questions being asked. As stated in Section III-H, the selected focal points were the reservoir system configuration and operation for only three purposes, namely flood control, low flow regulation, and reservoir related recreation. At this stage water supply, irrigation and conjunctive operation with ground water bodies are not considered. Consequently, the selected systems graph of Figure 8 is limited to a tree-like network of river reaches with reservoirs on several branches and, further, the location of some of the population centers that figure in the recreation question. The geometry of such graphs* is critical for the feasibility of explicit optimization studies. In building a simulation model, however, the river-reservoir configuration is of limited concern.

b. Systems Structure

In addition to the reservoirs \( R_j \), their contributory watersheds \( W_{ij} \), and the river net represented by lines, the Figure 8 shows gaging station locations (crossed circles) for which historical flow records are available. These locations could be called the "information input nodes" of the system. In the actual model these nodes were effectively displaced (using appropriate estimated corrections for the flow series) to the confluences of the Wabash River with its tributaries. Between these confluences there are six Wabash River reaches, I, II, ..., VI.

The selected systems purposes, namely flood control, low flow regulation and recreation are largely complementary. When reservoir levels are low the available flood storage is maximum, but recreation demand is minimum because of the climate and vice versa. To a first approximation one may also neglect low flow augmentation and split the then dual purpose system into two alternating (in time) single purpose systems whose initial conditions provide the tie that binds them together.

For the recreation model the system consists mainly of the reservoir system with an emphasis on inflow statistics and the effects of reservoir

*in terms of reservoirs being in series, or in parallel, or just because of their number.
FIGURE 8 - UPPER WABASH SYSTEMS GRAPH; WITH GAGING STATION AND MAIN STREAM REACHES
FIGURE 9 - CHARACTERISTIC DATA FOR UPPER WABASH RESERVOIR-RIVER SYSTEM
level fluctuations on recreational development. From the flood control point of view the model consists mainly of a succession of main river reaches for which a part of the tributary inflow at the beginning of each reach can be manipulated by reservoirs. The other tributary contributions and the side inflows downstream of reservoirs and along the main river, cannot be controlled. Individual reach models are also basic to quality modeling.

The system and its model structure have thus been kept relatively simple. The building experience gained, however, will be most helpful in refining and extending it.

**B. MODELS FOR RIVER REACH ROUTING**

The propagation of floods through a river systems is a rather difficult computational problem. This is particularly true if one searches for optimal operation of the basin's system of reservoirs. An adequate discussion of routing models, particularly of linear models, is needed to clarify the potential that systems model efforts may hold for improved management.

a. **Overview of Available Models**

With reference to Figure 10 one can divide the seeming* variety models that describe the motion of water in a river reach in deterministic and probabilistic models. Their respective uses are: (a) forecasting, i.e. transforming a given event of a model input sequence into (always the same) output event $D(t)$; and (b) prediction, i.e. generating from a specified set of preceding events, a probabilistic series of alternative output events $R(t)$, having some desired statistical properties. Many models are a mixture of deterministic and probabilistic elements; they are called stochastic models (cf. blocks 2, 3, Fig. 8). Stochastic models yield predicted values of a variable $V_i(t)$ or of an interrelated variable set** $V_i(t)$:

$$P(V_i(t)) = D(t) + R(t), \quad i=1, 2, 3 \text{ say}$$

where $D(t) = \text{the expected value } E[V_i(t)]$ and where the relative importance of the deviation $R(t)$ varies from zero to 100% when progressing from purely deterministic to purely probabilistic models.

The next classification is that of black box versus conceptual models (4,6 vs 5,7; Figure 10). When there is no regard for the details of actual

*most models are special cases of or different presentations of the general ARIMA (i.e., autoregressive-integrated-moving average) model, discussed in detail by Box and Jenkins (1970); for further discussion see Appendix A.

**See Chow and Kareliotis (1970)
FIGURE 10 - CLASSIFICATION OF ROUTING MODELS
system behavior and the (assumed algebraic) model is based only on analytical input-output comparisons to fix model parameters, one deals with a lumped parameter "black box" model (or also "the systems identification approach"). On the other hand, when physical laws and other concepts about systems structure are introduced one deals with a synthesized or "conceptual" model (or also the "classical approach"). Conceptual models may be either of a "lumped parameter" or "distributed parameter" type; the latter refers to models that are themselves broken down in interconnected sub-models each having its own parameters.

Further classification, important from the mathematical model building point of view, are those of "linear" versus "nonlinear" (5 thr. 15, Fig. 10) and "time invariant" versus "time dependent" (of model parameters) (16 thr. 23, Fig. 16). Examples of river reach models are inserted in Fig. 8. They are shown in the linear and non-linear classification, since time dependence of parameters is a rare case.

The phenomenon of "steady" and "unsteady" flow in channels is non-linear in detail. Shear flow theory (15, Fig. 10) is a (collection of) conceptual, non-linear models. Its results are a.o. used as a deterministic model to forecast energy loss rates in channel flows. Early examples of a black box approach to the energy loss question for steady flow are the Chezy and the Manning Equations.

Unsteady flow in channels is also a non-linear phenomenon as a whole. The basic, non-linear, partial differential equation that describes unsteady channel flow in terms of the conservation of mass and momentum are the Saint-Venant equations. These are well-known and have been discussed for years because of their great difficulty when it comes to solving them for practical application. Only in the recent past computer simulation has made actual use possible. Such applications are called "hydraulic routing" (11, Fig. 8).

Although hydraulic routing (i.e. the use of the "complete non-linear channel flow" model is possible, it is, as a rule, not feasible in practice because of the lack of detailed channel geometry data (at say every 100 R, where R = hydraulic radius of the river). Thus there have been developed various black

* differential is the limiting case of a distributed parameter model
box models for, and approximations to the non-linear solution. Most are linear models. Many are black box models. Examples of such black box or parameter models include the Muskingum (McCarty, 1938), the Larg and Route, (Clark, 1945) and the successive routing (Kalinin & Milyukov, 1957) models. Other linear models have a conceptual basis by being linearizations of the complete non-linear model. Probably* the most responsible is the Linear Channel Routing or LCR model (Harley, 1967). A considerably coarser linearization is the Diffusion Analogy model (Hayami, 1951).

b. Mathematical Comparison of Models

In order to compare the linear models, two features need to be discussed. These are (a) the number of parameters needed to reasonably simulate** the complete non-linear model solution, and (b) the linearity of the models which makes it possible to use superposition in their application and, conversely, to disaggregate an input into a succession of elementary inputs called pulse inputs. Mathematically a pulse or impulse (of unit size) is defined by the Kronecker Delta function, \( \delta(t) \); physically it is a (rectangular wave) input of very small duration. For truly linear systems the response (or elementary systems output) to a unit pulse input \( \delta(t) \) can be calculated and is called the unit impulse response or kernel function, \( h(t) \). Knowing for a given model the \( h(t) \) permits calculating the systems output \( O(t) \) when the input \( I(t) \) is disaggregated in a series of successive impulses \( a_i \delta(t) \). This process, called convolution, is illustrated in Figure 11. This illustration is only an approximation, called "discrete convolution"; algebraically it is:

\[
0(m\Delta t + \tau_0) = \sum_{m-n=0}^{M} \sum_{n=0}^{N} h(m\Delta t - n\Delta t) I(n\Delta t)
\]

In Figure 7, \( M = 7 \), \( N = 3 \), representing the durations of \( h(t) \) and \( I(t) \), respectively.

Originally the theory, adapted to engineering applications by von Kármán and Biot (1940), involved the process shown in Fig. 11 when \( \Delta t \to 0 \) and \( M \to \infty \), \( N \to \infty \). The advantage of working with these limiting cases was not just a matter of greater precision but primarily the possibility to use functional

---

*only incidental comparisons with the non-linear parent model have so far been reported. The model presumes validity of the Chezy Equation.
**according to one or another comparison method and a criterium for goodness of fit.
FIGURE 11 - THE INPUT - OUTPUT RELATION FOR A LINEAR SYSTEM HAVING H(t) AS UNIT IMPULSE RESPONSE

(Note: the a-scale is compressed to 50%; in the h-scale the a₁ would be twice as large.)
transform theory* to facilitate or, in fact, make feasible the convolution computation of Eq. 2.

From the Figure 11 one may infer that rather discontinuous \( h(t) \)'s still yield smooth outputs \( O(t) \) when \( \Delta t \to \text{small} \). When, given an input, the delay in initial response, \( \tau_o \), and the "length" of \( h(t) \) (in Fig. 11 this is \( 7\Delta t \)) are equal the systems outputs \( O(t) \) look alike even though the \( h(t) \) shapes may be rather different. For this reason a basic comparison between various models is best made in terms of their unit impulse functions \( h(t) \) rather than the \( O(t) \) resulting from applying an \( h(t) \) to some input \( I(t) \). The comparison is then, furthermore, independent of one or another model input selection.

Common linear models and the corresponding input response functions except for model #10** are shown in Figure 12. A comparative discussion of their features is started most conveniently by considering model #10. The outflow \( q(t,t) \), of a wide, rectangular (model) reach following the "application" of a rectangular hydrograph of very short duration \( \Delta t \) (when \( \Delta t \to 0 \)), depends on the Chezy roughness \( C \), the bottom slope \( S_o \), the channel length \( \ell \), the base flow \( q_o \), and the depth of flow \( y_o \). Taking the continuity and the momentum equations, combining them with the Chezy Equation, then linearizing the result by perturbation around \( q_o \), and finally writing the result in non-dimensional form, Harley and Dooge formulated the following model for two-dimensional open channel flow:

\[
A \frac{\partial^2 q}{\partial x^2} - B \frac{\partial q}{\partial x \partial t} - C \frac{\partial q}{\partial x} = \frac{\partial^2 q}{\partial t^2} - D \frac{\partial q}{\partial t}
\]  

(4)

*e.g. Fourier transform or Laplace transform; for computer calculation, one uses the discrete Fourier transform and the discrete Laplace transform, called Z-transform. In transform notation Eq. 2 reads:

\[
O(z) = H(z)I(z); \quad H(z\Delta t) = h(z\Delta t)/\Delta t; \quad z = \text{complex}
\]  

(3)

**for model #10 Harley (1967) gives

\[
q(x,t) = q_1 + q_2 = \delta(t-x/c_1)e^{-px} + h(x/c_2-x/c_2)(e^{sx rt}) I[2\pi R]
\]

where:

\[
c_1 = q_o/y_o + \sqrt{gy_o} \quad ; \quad c_2 = q_o/y_o - \sqrt{gy_o} \quad ; \quad p = \frac{s_o}{y_o} \left( \frac{2-2f}{p^2+2f} \right) \quad ; \quad r = \frac{s_o q_o}{2y_o} \left( \frac{2+f^2}{p^2} \right)
\]

\[
s = s_o/2y_o \quad ; \quad h = \frac{s_o q_o}{2y_o} \left[ (1-f^2/4)(1-f^2/4)/p^2 \right]^{1/2} \quad ; \quad I[1] = 1st \text{ order modified Bessel function}
\]
<table>
<thead>
<tr>
<th>MODEL</th>
<th>REFERENCE</th>
<th>PARAMETERS</th>
<th>SYSTEM EQUATION</th>
<th>UNIT IMPULSE RESPONSE EQUATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. LINEAR RESERVOIR</td>
<td>CLARK (1945)</td>
<td>I - PARAMETER</td>
<td>$O(t) + K \frac{d}{dt} O(t) = I(t)$</td>
<td>$h(t) = \frac{1}{2} e^{t/K}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>K = storage parameter</td>
<td></td>
<td>$K_1 \times K_2 \times K_3$</td>
</tr>
<tr>
<td></td>
<td>Empirical</td>
<td></td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td>2. LINEAR CHANNEL</td>
<td>DOOGHER (1957)</td>
<td>I - PARAMETER</td>
<td>$O(t) = I(t - t)$</td>
<td>$h(t) = \delta(t)$</td>
</tr>
<tr>
<td></td>
<td>Linearisation</td>
<td>t = delay</td>
<td></td>
<td>$t/t$</td>
</tr>
<tr>
<td>3. LAG AND ROUTE</td>
<td>CLARK (1945)</td>
<td>I - PARAMETER</td>
<td>$O(t+t) + K \frac{d}{dt} O(t+t) = I(t)$</td>
<td>$h(t) = \frac{1}{2} e^{(t-t)/K}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>K = storage parameter</td>
<td></td>
<td>$n_1 = 1; n_0 = 2; n_2 = 10$</td>
</tr>
<tr>
<td></td>
<td>Empirical</td>
<td>t = delay</td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td>4. SUCCESSIVE ROUTINGS</td>
<td>KHAN (1967)</td>
<td>I - PARAMETER</td>
<td>$O(t+t) + K \frac{d}{dt} O(t+t) = I(t)$</td>
<td>$h(t) = \frac{1}{2} e^{(t-t)/K}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>K = storage parameter</td>
<td></td>
<td>$n_1 = 1; n_0 = 2; n_2 = 10$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>n = number of routes</td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td>5. MUSKINGUM</td>
<td>MCCARTHY (1969)</td>
<td>K = storage p.</td>
<td>$O(t) = K(1-X) \frac{d}{dt} O(t) = I - K_2 \frac{d}{dt}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>X = weighting factor</td>
<td></td>
<td>$L_2 = L_1; K_2 = K_1; q_1 = q_1$</td>
</tr>
<tr>
<td></td>
<td>Empirical</td>
<td></td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td>6. DIFFUSION ANALOGY</td>
<td>HAWKINS (1967)</td>
<td>K = diffusion coeff.</td>
<td>$O(t) = K \frac{d^2}{dx^2} O(t)$</td>
<td>$h(t) = \frac{1}{2} e^{(t-t)/K}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>c = advective velocity</td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td>7. GAMMA DISTRIBUTION</td>
<td>DOOGHER (1959)</td>
<td>I - PARAMETER</td>
<td>$O(t+t) + K \frac{d}{dt} O(t+t) = I(t)$</td>
<td>$h(t) = \frac{1}{2} e^{(t-t)/K}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>K = storage parameter</td>
<td></td>
<td>$n_1 = 1; n_0 = 2; n_3 = 10$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>n = number of routes</td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td>8. MULTIPLE MUSKINGUM</td>
<td>LAURENSON (1967)</td>
<td>I - PARAMETER</td>
<td>$O(t+t) + K \frac{d}{dt} O(t+t) = I(t)$</td>
<td>$h(t) = \frac{1}{2} e^{(t-t)/K}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>K = storage parameter</td>
<td></td>
<td>$n_1 = 1; n_0 = 2; n_3 = 10$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>X = weighting factor</td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>n = number of routes</td>
<td></td>
<td>$t/t_0$</td>
</tr>
<tr>
<td>9. LAGGED DIFFUSION</td>
<td>HAWKINS (1967)</td>
<td>I - PARAMETER</td>
<td>$O(t+t) + K \frac{d}{dt} O(t+t) = I(t)$</td>
<td>$h(t) = \frac{1}{2} e^{(t-t)/K}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>K = storage parameter</td>
<td></td>
<td>$n_1 = 1; n_0 = 2; n_3 = 10$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>c = advective velocity</td>
<td></td>
<td>$t/t_0$</td>
</tr>
</tbody>
</table>

**FIGURE 12 - OVERVIEW OF SOME LINEAR ROUTING MODELS**
where A, B, C and D are functions of 3 dimensionless parameters which, in turn, are functions of the five dimensional parameters enumerated above.

The dimensionless parameters are: $F_0$ = base flow Froude number = $q_0/gy_0^{3/2}$; $L$ = dimensionless channel length = $g^2F_1^2/c^2l$; and $t_o$ = period of response delay = e.g. time needed for elementary wavelet to traverse reach = $t/(gy_0)^{1/2}$.

From the sketches for model #10 in Figure 12 one readily sees the variety of (unit impulse) response shapes that can arise, depending on the values of $C$, $S_o$, $l$, $q_o$ and $y_o$. A merit of the LCR model is that it is basically a 5-parameter model with each of the parameters having a direct meaning in open channel flow theory. Using dimensional analysis this model is condensed to a three-parameter model with each of the parameters ($F_0$, $L$, and $t_o$) still being familiar ones in hydraulics. For these reasons one may expect the LCR model to forecast results over a wider range, while using more rational parameter dependences than other models. Consequently, the LCR may be adopted as a standard for comparison of other models.

In view of the large number of parameters that govern the flood wave propagation and deformation phenomenon it is clear that the less the number of parameters a model has available for fitting it to empirical data, the more difficulty one may have in selecting model parameters and in having that model predict events adequately. Figure 12 shows three groups of models having 1 (#1,2), 2 (#3,4,5,6) and 3 (#7,8,9,10) parameters. (As said, model #10 is really a 5-parameter model). The simplest linearization is represented by model #2. Neglecting more terms in the already linearized equation (3) can be done in several ways. For example:

$$A \frac{\partial^2 q}{\partial x^2} - B \frac{\partial^3 q}{\partial x \partial t} = \frac{\partial^2 q}{\partial t^2}$$

or, secondly:

$$-C \frac{\partial q}{\partial x} = D \frac{\partial q}{\partial t}$$

The solution to Eq. (5) is:

$$q(x,t) = q(o,t-x/c_1) = q(o,t-t_1)$$

*one should realize that LCR or, for that matter, any other algebraic expression is a model and not reality itself; whether one or another model is "better", depends partially on the goodness of its prediction and partially on the context wherein the model is used.*
where \( c_1 = q_0/y_0 + (g y_0)^{1/2} \).

The solution to Eq. (6) is

\[
q(x,t) = q_0 t-x/(1.5q_0/y_0) = q_0 (t-t_2)
\]

(8)

Invoking the principle of superposition, one can in both cases write the model in terms of what its solution implies, namely:

\[
0 = I(t-t)
\]

(9)

This is called writing the model, or the one-parameter system, in an operator notation or systems notation or black box model notation. In general a conceptual systems function such as Eq. 2 is not conveniently written in black box notation. On the other hand, a pure black box systems equation, based on fitting parameters using input-output comparisons, cannot possibly be written in a differential form which reflects something of the internal structure of the system.

The model #1 is a black box model. A purely physical analog would be a reservoir with outlet device such that, at all times, the outflow \( O(t) \) is proportional to the storage \( S(t) \)*. Model #1 is rather defective in that it cannot model translation of a wave with a base flow. Model #2 is defective in that it cannot model the flood wave attenuation which occurs in all but very short reaches.

Since the models are linear they may be combined into a two-parameter model #3 which then provides translation (at speed \( l/\tau \)) as well as attenuation. However, this model is still not very satisfactory because one would (on physical grounds) expect the passage of a bell shaped wave at the end of longer reaches, rather than continue to have a vertical wave front. A better 2-parameter model is model #4. It represents the repeated application of model #1. Clearly variation of \( n \) permits better adaptation to differences in reach lengths.

An older 2-parameter model is the Muskingum routing model. Its peaked \( h(t) \) and the lack of an independent delay time choice make it the poorest of the 2-parameter models. In fact, it is not very different from model #1,

*If the outflow is an overflow spillway, \( O \propto H^{3/2} \), so that one should have \( S \propto H^{3/2} \); if the outflow is submerged \( O \propto H^{1/2} \) and \( S \propto H^{1/2} \); if the outflow is a proportional weir, \( O \propto H \) and \( S \propto H \). No large reservoirs follow these trends over their full operational range. For small watersheds one has approximately the linear relation \( O \propto S \); for large watersheds \( O \propto S^{0.75} \).
except for the negative, end of the reach response immediately following imposition (at \( t = 0 \)) of an input impulse at the beginning of the reach. This response is physically not plausible. By contrast, the 2-parameter model \#6 gives more adequate results. But then model \#6 is a lineariza-
tion, whereas model \#5 is only a black box model.

The two parameter models, except \#3, suffer the defect of having no parameter that provides for translation of flow as expressed in an explicit choice of delay time \( \tau \). And model \#3 cannot produce a wave-shaped response. The situation can be remedied by preceding models \#4 and \#6 with model \#2. This yields the 3-parameter models \#7 and \#9. Another 3-parameter model, obtained from the n-fold application of the Muskingum model yields \( h(t) \) having some oscillatory portion (cf. the negative \( h(0) \) - values of model \#5). While of lesser quality than models \#7 & \#9, it is a material improvement over model \#5.

This brings us back to model \#10. Although a 3-parameter model analy-
tically, 5 physical parameters went in a given \( h(t) \) selection as discussed above. The role of channel length and Proude number are brought out explicit-
ly.

This concludes a comparative discussion of routing models; it is provided to motivate the selection made in building the simulation model. Some fur-
ther discussion of comparative model features is given in Appendix A.

c. Comparison between Models in Terms of Their Application

From an applied point of view, hydraulic routing is needed whenever close control of \( q(x,t) \), i.e. the discharge at many locations and many in-
stances of time, are needed. An excellent example of applied hydraulic routing is given by Garrison et al. (1969). For present modeling purposes there is no need to insure close spatial channel control. Apart from this, recent, localized river geometry data are not available. This is one set of reasons why linear routing models are of interest. Their development can be based on records of gaging stations located relatively sparsely through-
out the basin.* A second reason why linear routing models were given much attention is that they permit the simple superposition of many reservoir outflows in a river system no matter how complex. It holds promise of being able to optimize the reservoir system operation using control theoretic concepts.

*we note that, should no gaging records be available at all, then one would again resort to (approximate) hydraulic routing methods.
The Muskingum model has frequently found its way in agency practice. This model represents the poorest selection from among the ones discussed above. That it has, nevertheless, found application is due to:
(a) early introduction and adoption of the Muskingum model;
(b) the consequent availability of ("comparable") parameter values (K and X);
(c) the fact that convolution often smoothes out differences between the response O(t) even when the unit response functions, h(t), are rather different;
(d) the probability that most prototype situations are time-dependent, whereas models are assumed to have time-invariant parameters;
(e) the time dependent contamination of reach input data with unknown side inflows along that reach.
(f) deviation from the Chezy flow relation in actual rivers;
(g) computer duplication of desk calculations for parameter estimation.

Stated differently: available field data, the difficulty to improve on the Chezy or a similar assumption, and adherence to routine parameter estimation methods, make that adoption of different models may not pay.

Because it does represent standard agency practice the Muskingum method was adopted as one of two initial models for which parameters (descriptive of reach flow conveyance) were estimated. The other model that was selected, is called the Second Order Linear model. It belongs to the same empirical category as the models #1, 4 and 5 in Figure 12. A comparison of these two models with the use of other models shown in Figure 12, and in particular with the LCR model, will be forthcoming shortly.

C. ESTIMATION OF FLOOD ROUTING PARAMETERS

a. Flow Record Adjustments

A central problem of any black box or systems identification approach is the estimation of the model parameters from simultaneous analysis of systems input and output. For a river reach the input and output are flow records at adjacent gaging stations. These stations are generally so far apart that their cumulative flows are not equal because of side inflows between stations. None of the models discussed above can account for side inflow to the channel. A simple method was adopted to take side inflows

*see Appendix A
into account. Their estimated sum \( \sum Q_{u_j}(t) \), representing a "correction flow", was added to the upstream station's record. This correction flow was estimated from

\[
\sum Q_{u_j}(t) = \frac{Q_r(t)}{A_r(t)} \sum A_{u_j}
\]

wherein \( Q_r(t) \) = discharge at day \( t \) at a reference* gaging station [cfs]; \( A_r \) = the reference station's watershed area [mi\(^2\)]; and \( A_{u_j} \) = the area of the \( j \)-th ungaged watershed contributing to the side inflow between the inflow station I and the outflow station 0.

In addition to this adjustment of flow records, a second adjustment of the same type was made to more conveniently build the simulation model. From section IV-3 and with reference to Figure 6 it is seen that the simulation model consists of a succession of reaches (I through VI) that are connected by input nodes. These nodes are located at the confluences of the major tributaries of the Wabash River. Many of these tributaries have or are slated to have reservoirs that will regulate their input into the input nodes, i.e. into the Wabash River. A quantitative estimate of the propagation and attendant modification characteristics (of floods flowing through each of these Wabash reaches) is what we seek to embody in model parameters. In the present case these are \( K \) and \( X \), the two parameters of the Muskingum Model (model 5, Fig. 12). However, for good reasons gaging stations are not located at river junctions. In order to obtain "proxy flow records" at river junction, representing input and output to a reach (which is the basic river component of the simulation model), again the Equation 10 was applied.

The model is explained best by illustrating its use to say the uppermost reach of the basin (i.e. reach I). Figure 13 presents this reach I after renumbering (cf. Figure 7; e.g. sta. 1 \( \equiv \) sta. 36 \( \equiv \) Huntington, etc.) The historical input and output flow records, \( Q_I(t) \) and \( Q_0(t) \) are obtained from:

\[
\{Q_I(t)\}_{hist} = Q_1(t) + Q_2(t) + \frac{Q_1(t)}{A_1} \left( A_{u1} + A_{u2} + A_{u1} \right)
\]

\[
\{Q_0(t)\}_{hist} = Q_4(t) - Q_3(t) - \frac{Q_1(t)}{A_1} \left( A_{u3} + A_{u4} \right)
\]

*In general \( Q_r(t) = Q_I(t) = \) inflow to the reach. Where records have become affected by man-made controls, a different reference station was used for those years that these controls were present. This was the case e.g. for records after 1967, the year that Salamonie Reservoir began operating.
\[ A_{u1} = \text{ungaged watershed area between station 1 and junction 1} \]

\[ A_{u2} = \text{ungaged watershed area between station 2 and junction 1} \]

\[ A_j = \text{gaged watershed area} \]

\[ A_{u_R} = \text{ungaged watershed between junctions 1 and 0} \]

\[ QI(t) = \text{proxy inflow to reach } I \]

**FIGURE 13 - CONSTRUCTION OF PROXY FLOW RECORDS**

The daily (proxy) discharges \( QI(t) \) and \( QO(t) \) are, of course, not equal (their differences being the effect from which we try to estimate reach parameters). However, on a yearly basis the reach inflow, \( QI_y \), will not equal the reach outflow \( QO_y \), either. The yearly discrepancy \( QO_y - QI_y = \Delta Q_y \) is a random variable. One might correct each water year's inflow in some fashion. One method would be to replace Eq. 10 by some other model such as:

\[
\sum_{j} Q_{u_j}(t) = \left( \frac{Q_r(t)}{A_r(t)} \right)^m \sum_{j} A_{u_j}^n \tag{10a}
\]
where \( m \) and \( n \) would differ from year to year such that

\[
\Delta Q_y = QO_y - QI_y = 0 \tag{10b}
\]

Even though the tables in Appendix C show that, on occasion, \( \Delta Q_y \) can be quite large,* refinement of routing model parameters, using say Eq. 10a, is considered to yield insufficient benefits in terms of the present project's objectives. We propose to return to this point in Section IV-C-4.

For the total historical record periods used and for the basin as a whole**, the difference between the routed historical (proxy) flows and the observed historical (actual) flows at gaging station #6 (the downstream border of the system) is only 0.40%. This small difference gives hope that at least no major systematic error is made by having adopted the area-proportioning model of Eq. 10.

b. Estimation of River Reach Routing Parameters

The determination of parameter values for a selected model*** involves first of all the adoption of a method for matching the shapes of the hydrographs \( \{QO(t)\}_{est} \) as computed by that model (i.e. Eq. 14), to the shapes of the historical hydrographs \( \{QO(t)\}_{hist} \). Limiting ourselves to a single flood (or to some selected record portion), the parameter values sought are those that satisfy the matching method's criterion function.

Following determination of the best model parameter sets (herein called \( K, X \)) for single events (or record portions), there remains the problem of how to estimate the overall optimal parameter set which makes for the best resemblance between total input and total output hydrographs. The place of both estimation problems is illustrated in Figure 14.

The Fig. 14 also points out that what is "best" depends on the application one has in mind. When it comes to simulating river reach effects on flood waves, two natural matching or shape parameters suggest themselves. These are: (a) the flood wave's travel time through the reach, \( t_o \); and (b) some parameter descriptive of the flood wave's deformation when moving through the reach. The corresponding matching method is the "moments" method.

---

*The tables show comparisons of historic (observed) values, \( \{QO(t)\}_{hist} \), with the computed \( \{QO(t)\}_{est} \) = routed \( \{QI(t)\}_{hist} \). However, because the routing model preserves inflow volume (see Eq. 15a), the volume of \( \{QO(t)\}_{est} \) equals the volume of \( \{QI(t)\}_{hist} \).

**i.e. by routing a continuous 13 year record \( \{QI(t)\}_{hist} \) through all reaches down to station 6

***the work reported in this section involved the Muskingum model; this model has the parameters \( K \) and \( X \) or, alternatively, as used in Eq. 14 the parameters: \( C_0, C_1, C_2 \) with \( C_2 = 1-C_0 - C_1 \).
outlined in Section IV-C-c. An alternative, non-parametric matching by the common least square fit method is reported in Section IV-C-d.

![Routing Model Parameter Selection Diagram]

**Figure 14 - Routing Model Parameter Selection for a River Reach**

As shown in Figure 14 one starts with the selection of a routing model. The Muskingum routing model selected herein is then applied to historical (proxy) inflow record \( \{QI(t)\}_{hist} \) to obtain estimated outflows \( \{QO(t)\}_{est} \). To this end the Muskingum Equation:

\[
\{QO(t)\}_{est} + K(1-X) \left( \frac{d\{QI(t)\}_{est}}{dt} \right) = \{QI(t)\}_{hist} - KX \frac{d\{QI(t)\}_{hist}}{dt} \quad (13)
\]

is first converted into a finite difference form with \( \Delta t = 1 \) day (see Appendix B). It then reads:

\[
\{Q_o\}_{est} = \omega \{Q_{o-t-1}\}_{comp} + C_1 \{I_{o-t-1}\}_{hist} + C_2 \{I_t\}_{hist} \quad (14)
\]
where: \[ C_0 = \frac{K\text{-}XX\text{-}0.5}{K\text{-}XX\text{+}0.5} \]; \[ C_1 = \frac{KX\text{+}0.5}{K\text{-}XX\text{+}0.5} \]; \[ C_2 = 1 - C_0 - C_1 \] (15a,b,c)

The Eq. 14 states that today's (estimated) outflow from the reach is proportional to yesterday's previously computed outflow; to yesterday's historic inflow; and to today's inflow.

The relation \[ C_2 = 1 - C_0 - C_1 \] insures that on say a yearly basis \( QI_y \) \( \text{hist} = QO_y \) \( \text{est} \), which is an assumption of any of the routing models considered. As discussed in Section IV-C-a the yearly discharge volume \( QO_y \) \( \text{est} \) does not equal \( QO_y \) \( \text{hist} \). The determination of the best parameter values \( C_0 \), \( C_1 \), \( C_2 \) (or, alternatively, \( k \) and \( \lambda \)) was based on the (proxy) volume \( QI_y \) \( \text{est} \). For further discussion see Section IV-C-e.

c. Routing Parameter Estimation by the Moments Method

Bell shaped curves (e.g. the probability density of independent events) are often characterized by shape parameters called "moments" \( m_i \). Theoretically one needs an infinite number of parameters to fully describe a curve. However, the higher the moment*, the less its statistical significance will be, generally speaking. For flood wave plots (plots of \( Q \) vs \( t \)), such moments, or rather the differences between moments at the beginning and end of a reach, provide a meaningful way to describe the reach characteristics. The first moments \( M_1 \) and \( M_0 \) locate, in time, the centroids of the flood wave. These provide the flood wave delay period or also the travel time through the reach which is represented by the parameter \( K \) in the Muskingum model:

\[ M_1 - M_0 = t_I - t_0 = t_0 = K \] (16)

The difference between the second moments or variances \( S_I^2 \) and \( S_0^2 \), provides some measure for a flood wave's broadening or also its decrease in peak flow (for zero side inflow). The relationship with the Muskingum model parameters (see Appendix B) is:

\[ S_0^2 - S_I^2 = K^2(1-K) \] (17)

For the moments method, satisfying the matching criterion function (block 5, Fig. 14) amounts to solving \( K \) and \( \lambda \) from Eqs. 16 and 17. In Figure 14 this

* \( m_0 \) = zeroeth moment = area under bell shaped curve; \( m_1 \) = first moment = first order statistic = mean = location of centroid = \( \bar{Y} \); \( m_2 \) = second moment = second order statistic = variance with respect to the mean = \( (Y-\bar{Y})^2 \); \( m_3 \) = third moment = third order statistic = skewness = \( (Y-\bar{Y})^3 \); etc.
amounts to (2+5) & (4+5) followed by 5+3.

The moments method was tried by selecting twelve distinct flood waves. For each storm and for all six reaches of the upper Wabash the first two moments of \( \{Q_I(t)\}_{\text{hist}} \) and \( \{Q_0(t)\}_{\text{hist}} \) were computed. From the resulting values for \( M_1, M_0, S_1^2 \) and \( S_0^2 \), a total of 72 sets \( *K^* \) were computed. Unfortunately, the results appeared unsatisfactory; roughly half of the \( *K^* \) were negative and roughly half of the \( *K^* \) were larger than unity. Such results seem physically unrealistic. Of course one can argue that we merely adopted a model; if its parameters need to have anomalous values, so be it, as long as the transformation of \( Q_I(t) \) to \( Q_0(t) \) is duplicated. However, there are two other, perhaps more important reasons for the anomalies than an ill-selected model. The first is that precipitation excesses were not uniform (due to variations in rainfall, soil infiltration, etc.) as was assumed in accounting for side inflow (i.e. in forming the proxy records). Secondly the data time interval \( \Delta t = 1 \) day, necessitated by flow record availability*, may be too large. As far as the model itself is concerned, it is recognized that \( *K^* \) and \( *X^* \) may be functions of stage. It is also possible that the gage rating curve corrections necessitated by river geometry change, do not fully reflect actual flow variation for a given stage at a given time.

Another problem with the moments method is the need to select simple (as opposed to compound) storms. This makes for the problem how to separate base flow \( q_0(t) \) from the hydrograph. Perhaps yet more important is that only a portion of the total records \( \{Q_I(t)\}_{\text{hist}} \) gets used.

d. Estimation of River Reach Parameters-Least Square Fit

For reasons noted above and because it was felt that the total record should be utilized, the moments method was replaced by common least-square fitting to obtain the matching criterion function:

\[
\text{MINIMIZE } e^2 = \left\{ (Q_0(t))_{\text{hist}} - (Q_0(t))_{\text{est}} \right\}^2
\]

In Figure 14 this amounts to (3+5) & (4+5) followed by 5+3. Implementing Eq. 18 represents a fair sized problem in quadratic programming. Therefore, Eq. 18 was implemented by graphic plotting and inspection. To this end \( K_i \) and \( X_i \), \( i = 1, 2, \ldots \), were varied systematically within the constraints:

*while storm hydrographs records may be available with finer than daily flow resolution, it was decided not to invest (at the initial stage of overall model building) efforts in refinement of model components.
These constraints amount to requiring that the impulse response function \( h(t) \) (see Figure 11) is positive for all values of \( t \) (this is analogous to requiring \( K > 0 , X \leq 1 \)).

In computing \( \{Q_0(t)\}_\text{est} \) used to implement the Eq. 18, total gaging station records per water year were chosen as \( \{Q_0(t)\}_\text{hist} \) rather than only selected flood hydrographs. By computer a matrix of the factor \( R^2_y = \frac{(1-Er^2)_y}{\Sigma (Q_0(t))_\text{hist}^2} \) was plotted in a \( C_0 , C_1 \) coordinate system. The (linear) constraints of Eq. 19 were also plotted in these diagrams. The maximum values of \( R^2 \) within the constraints identified the optimal values \( C_0^* \) and \( C_1^* \) (and hence \( C_2^* = 1 - C_1^* - C_2^* \)). These yield by Eq. 15, the optimal Muskingum Model parameter set \( K , X \) for the reach in question.

Note that implementing Eq. 18 using the total record implies a definition for block 9 of Figure 14 (selection of a method for matching multiple wants).

e. Discussion of Parameter Estimation Results

The six river reaches, their lengths, and the arithmetic averages (over the total record period of the computed coefficients \( C_0 , C_1 \) and \( C_2 \) are shown in the Table 9. These are the values used in the first version of the Upper

<table>
<thead>
<tr>
<th>SUBREACH NO.</th>
<th>UPSTREAM END MILEAGE</th>
<th>DOWNSTREAM END MILEAGE</th>
<th>LENGTH MILES</th>
<th>MUSKINGUM COEFFICIENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>406</td>
<td>394</td>
<td>12</td>
<td>0.13 0.17 0.70</td>
</tr>
<tr>
<td>II</td>
<td>394</td>
<td>375</td>
<td>19</td>
<td>0.05 0.22 0.73</td>
</tr>
<tr>
<td>III</td>
<td>375</td>
<td>354</td>
<td>21</td>
<td>0.03 0.12 0.85</td>
</tr>
<tr>
<td>IV</td>
<td>354</td>
<td>322</td>
<td>32</td>
<td>0.00 0.28 0.72</td>
</tr>
<tr>
<td>V</td>
<td>322</td>
<td>317</td>
<td>5</td>
<td>0.09 0.39 0.52</td>
</tr>
<tr>
<td>VI</td>
<td>317</td>
<td>288</td>
<td>29</td>
<td>0.36 0.33 0.31</td>
</tr>
</tbody>
</table>

**TABLE 9 - AVERAGED RIVER REACH ROUTING PARAMETERS OBTAINED BY A LEAST SQUARE MATCHING CRITERION FUNCTION**

Note: For coefficients of individual water years, see Appendix C.

\(^* R^2 \) resembles, but is not identical to the usual correlation coefficient.
Wabash Simulation model reported herein. It is noted that, due to differences between recording periods of gaging stations, the periods for which the coefficients are developed differ slightly for each reach.

In order to examine the best fit parameters for possible temporal trends, the tables in Appendix C show inflow and outflow [in inch/mi²], the computed $R^2$, and the $C_0$, $C_1$ and $C_2$ by reach and by water year. These Tables show that, in the average, the optimization factor $R^2 \approx 0.98$. Lower values of $R^2$ do associate, by definition, with discrepancies between actual and assumed ungaged inflows for the year and the reach in question.

In general the coefficient $C_2 >> C_0$ or $C_1$. This indicates (see Eq. 14) that a reach's outflow on day $t$ is determined primarily by the inflow on day $t$ and to a much lesser extent by the outflow and the inflow on day $t-1$. Going downstream the values of $C_0$ and $C_1$ increase. One would expect this on account of the increase in storage going downstream.

It may be noted that $C_2$ tends to increase after 1966. This may reflect the influence of man-made storage as Salamonie and Mississinewa reservoirs were put in operation in 1967 and 1968, respectively.
V. RESERVOIR SYSTEMS COMPONENT

A. GENERAL

The Huntington Reservoir, which is the most upstream storage component of the basin, is located on the main stream. The other four federal reservoirs are all located on separate tributaries. The five reservoirs and the river reaches form a parallel network as shown in Figure 8.

According to COE policies reservoir outflows are selected based on river stages and quality conditions (present as well as the forecasted ones) at downstream "control stations". Thus the reservoir operating policies contain difficult to handle feedback features. Furthermore, the reservoir inflow-outflow relationships are generally nonlinear. They cannot be linearized in contradistinction to what is being done for the river reach models discussed in Chapter IV.

The selected time step of the simulation is one day. For the purposes of economic evaluation a monthly model and corresponding policies may be needed. These have not yet been formulated. As exogenous model input, two types of daily runoff were used, namely: (a) gaging station flow records; and (b) synthetic storm hydrographs. These synthetic storm hydrographs had the form of gamma functions. Their computation is outlined in Section VI-A-b.

The shape parameters of the conceptual hydrographs were determined on the basis of selected floods found in the daily flow records. The synthetic storm hydrograph volumes were incremented (in steps of 1" runoff/\text{mi}^2$, say). These volumes will be assigned* a recurrence interval based on a statistical analysis of the recorded flow data.

The Corps of Engineer's reservoir design selections, both structural and operational, were accepted as basic exogenous data in building the simulation model. However, the computer programs were so constructed that one can readily make test runs that examine the effects of design changes (such as e.g. changes in the recreational pool elevations or seasons).

The reservoir release decisions in the initial simulations were, as stated, based on Corps of Engineer's operating policies. In this way we sought duplication of previous analyses using the new simulation model. The

*method used and results are set forth in companion report.
development of a new systems operating policy is presented in a subsequent report. That new policy is one of several that may be tried in seeking whether it is possible to improve surface water management in the Upper Wabash by operating all reservoirs more as a unit.

B. RESERVOIR CHARACTERISTICS

In the reservoir component simulation three aspects require attention. These are the reservoir input flows, the reservoir shape and outlet works, and the reservoir operating policy.

Only one reservoir, Big Pine, has a gaging station sited exactly at the dam site. For other reservoirs one has to estimate the inflow from nearby gaging stations records. These nearby stations may be either on the same stream or on a nearby stream. In principle a reference station's drainage area should about equal the drainage area for the reservoir. At the same time the two drainage areas should be located as close as possible to each other. Finally the reference record should be sufficiently long. For example, station No. 12 (see Figure 8) is the best station to use for estimating the Huntington Reservoir inflow. However, its record is relatively short (starting in 1951). The next best station (station 28) was therefore used for the period prior to 1951.

The daily inflow for a reservoir was computed from

\[ I(t) = \frac{Q_g(t)}{A_g} A_{res} \]  \hspace{1cm} (20)

where \( I(t) \) = reservoir inflow at day \( t \) [cfs]; \( Q_g(t) \) = flow rate of gaging station at day \( t \) [cfs]; \( A_g \) = drainage area controlled by gaging station \([\text{mi}^2]\); \( A_{res} \) = drainage area controlled by reservoir \([\text{mi}^2]\).

Table 10 gives the code numbers of the gaging station that were used for inflow determinations, the drainage areas controlled by the reservoirs, and the periods over which the gaging station records were employed. Additional information, such as the average flow rate, maximum flow rate and minimum flow rate, can be found in Tables 3, 4 and 5.

The reservoir shape and outlet characteristics are given in the form of rating curves. Referring to Table 11, these are the: (1) surface elevation - storage capacity curve; (2) surface elevation - surface area curve; (3) surface elevation - spillway discharge capacity curve; and (4) surface elevation - sluiceway discharge capacity curve.
<table>
<thead>
<tr>
<th>RESERVOIR</th>
<th>STATION NO. (SEE FIGURE 3)</th>
<th>DRAINAGE AREA SQ. MI.</th>
<th>PERIOD CHOSEN BEGIN</th>
<th>END</th>
</tr>
</thead>
<tbody>
<tr>
<td>Huntington</td>
<td></td>
<td></td>
<td>Jan. 1969</td>
<td>date</td>
</tr>
<tr>
<td>upstream</td>
<td>28</td>
<td>717</td>
<td>1930</td>
<td>1950</td>
</tr>
<tr>
<td>downstream</td>
<td>12</td>
<td>532</td>
<td>1951</td>
<td>1968</td>
</tr>
<tr>
<td>Salamonie</td>
<td></td>
<td>553</td>
<td>Apr. 1967</td>
<td>date</td>
</tr>
<tr>
<td>downstream</td>
<td>30</td>
<td>557</td>
<td>1930</td>
<td>1966</td>
</tr>
<tr>
<td>Mississineva</td>
<td></td>
<td>807</td>
<td>Apr. 1968</td>
<td>date</td>
</tr>
<tr>
<td>upstream</td>
<td>32</td>
<td>677</td>
<td>1930</td>
<td>1952</td>
</tr>
<tr>
<td>downstream</td>
<td>33</td>
<td>808</td>
<td>1953</td>
<td>1967</td>
</tr>
<tr>
<td>Lafayette</td>
<td></td>
<td>787</td>
<td></td>
<td></td>
</tr>
<tr>
<td>upstream</td>
<td>41</td>
<td>390</td>
<td>1944</td>
<td>1953</td>
</tr>
<tr>
<td>downstream</td>
<td>39</td>
<td>791</td>
<td>1954</td>
<td>date</td>
</tr>
<tr>
<td>Big Pine</td>
<td></td>
<td>329</td>
<td></td>
<td></td>
</tr>
<tr>
<td>upstream</td>
<td>40</td>
<td>329</td>
<td>1955</td>
<td>date</td>
</tr>
<tr>
<td>adjacent</td>
<td>41</td>
<td>390</td>
<td>1944</td>
<td>1954</td>
</tr>
</tbody>
</table>

**TABLE 10 - INFLOW GATING STATIONS FOR 5 FEDERAL RESERVOIRS IN THE UPPER WABASH BASIN**

The functional relationships in Table 11 are least square fits to information originally presented in tabular and graphical form. The equations were derived to facilitate use of rating curve information in the computer programs that make up the Wabash simulation system. The fitting equation was a power function \( Y = a(Z-Z_0)^b \). As shown in Table 11, the storage and surface area relations for two of the reservoirs required two equations each. Other structural characteristics such as the dimensions of dam, spillway and sluice gate, the crest elevation of spillway, are given in Table 7.
<table>
<thead>
<tr>
<th>HUNTINGTON RESERVOIR</th>
<th>SALAMONIE RESERVOIR</th>
<th>MISSISSINewA RESERVOIR</th>
<th>LAFAYETTE RESERVOIR</th>
<th>BIG PINE RESERVOIR</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>STORAGE CAPACITY - ELEVATION RELATION</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S: Storage capacity in acre-feet</td>
<td>Z: Elevation in feet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S = 0.113(Z - 718)^{3.23}$</td>
<td>$S = 0.0208(Z - 684)^{3.48}$</td>
<td>$S = 0.0233(Z - 665)^{3.51}$</td>
<td>$S = 0.291(Z - 540)^{3.0}$</td>
<td>$S = 3.88(Z - 510)^{2.25}$</td>
</tr>
<tr>
<td>$(Z &gt; 768')$</td>
<td>$(Z &gt; 720')$</td>
<td>$(Z &gt; 717')$</td>
<td>$(Z &gt; 580')$</td>
<td>$(Z &gt; 543')$</td>
</tr>
<tr>
<td>$S = 6.31(Z - 718)^{2.20}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$(Z &lt; 768')$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>A: Surface area in acres</th>
<th>Z: Elevation in feet</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$A = 0.365(Z - 718)^{2.23}$</td>
<td>$A = 0.0725(Z - 684)^{2.48}$</td>
<td>$A = 0.0818(Z - 665)^{2.51}$</td>
<td>$A = 0.873(Z - 540)^{2.0}$</td>
<td>$A = 8.74(Z - 510)^{1.25}$</td>
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<td>$(Z &gt; 720')$</td>
<td>$(Z &gt; 717')$</td>
<td>$(Z &gt; 580')$</td>
<td>$(Z &gt; 543')$</td>
</tr>
<tr>
<td>$A = 13.9(Z - 718)^{1.20}$</td>
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<tr>
<td>$(Z &lt; 768')$</td>
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</table>

| **SPILLWAY CAPACITY - ELEVATION RELATION** | | | | |
| **Qₕ**: Spillway capacity in CFS | Z: Elevation in feet | | | |
| $Qₕ = 400(Z - 765)^{1.60}$ | $Qₕ = 330(Z - 793)^{2.06}$ | $Qₕ = 740(Z - 779)^{1.97}$ | $Qₕ = 1460(Z - 645)^{2.0}$ | $Qₕ = 297(Z - 596)^{1.60}$ |
| $(Z > 765')$ | $(Z > 793')$ | $(Z > 779')$ | $(Z > 645')$ | $(Z > 596')$ |

| **SLUICEGATE CAPACITY - ELEVATION RELATION** | | | | |
| **Q₉**: Sluicegate capacity in CFS | Z: Elevation in feet | | | |
| $Q₉ = 1220(Z - 720)^{0.5}$ | $Q₉ = 650(Z - 684)^{0.616}$ | $Q₉ = 638(Z - 665)^{0.62}$ | $Q₉ = 423(Z - 544)^{0.582}$ | $Q₉ = 1076(Z - 525)^{0.473}$ |
| $(Z > 720')$ | $(Z > 720')$ | $(Z > 700')$ | $(Z > 544')$ | $(Z > 543')$ |

**TABLE 11 - RESERVOIR SHAPE AND OUTLET CHARACTERISTICS**
The operation policies of the reservoirs include such parameters as: (a) minimum pool elevation; (b) seasonal pool elevation; (c) flood pool elevation and (d) the beginning and the end of recreational season. This information is also shown in Table 7.

C. RESERVOIR ROUTING

All five reservoirs are equipped with gates for outflow control. The discharge at a fixed reservoir pool level can thus be varied by adjusting the gates. Reservoir discharges and hence gate openings are controlled by the operating policy of the reservoir in question. This mode of operation is referred to herein as "controlled reservoir routing". On the other hand, when the reservoir passes water via an ungated spillway or uncontrolled sluice gates (i.e. gates that are 100% open), the releases are dependent only on structural dimensions. This mode of operation is referred to herein as "uncontrolled reservoir routing".

a. Uncontrolled Reservoir Routing: In the "uncontrolled operation mode" a reservoir's release is being constrained by the structural dimensions of the outlet and the reservoir pool level elevation. Storage volume, surface elevation and "uncontrolled" discharge are linked by three relationships. These are the reservoir continuity equation; the pool elevation-storage capacity curve; and the pool elevation-outlet capacity (spillway + sluicegate) curve.

The daily continuity equation is:

\[ \frac{I_k + I_{k-1}}{2} - \frac{Q_k + Q_{k-1}}{2} = S_k - S_{k-1} \]  

(21)

where \( I_k, I_{k-1} \) = reservoir inflows at days \( k \) and \( k-1 \); \( Q_k, Q_{k-1} \) = reservoir outflows at day \( k \) and \( k-1 \); \( S_k, S_{k-1} \) = reservoir storage volume at day \( k \) and \( k-1 \).

The reservoir pool elevation-storage capacity curve were fitted by power functions of the form:

\[ S_k = a(Z_k - Z_o)^b \]  

(22)

where \( S_k \) = reservoir storage volume at day \( k \); \( a, b, Z_o \) = constants; \( Z_k \) = reservoir surface elevation at day \( k \).

The reservoir pool elevation-outlet capacity curves were also fitted by
power functions of the form:

\[ Q_k = c(Z_k - Z_1)^d \quad Z_k < Z_s \]  \hspace{1cm} (23)  

\[ Q_k = c(Z_k - Z_1)^d + e(Z_k - Z_s)^f \quad Z_k > Z_s \]  \hspace{1cm} (24)  

where \( Q_k \) = sluicegate capacity at pool elevation \( Z_k \) in Eq.(23), or \( Q_k \) = sluicegate + spillway capacities at pool elevation \( Z_k \) in Eq.(24); \( c, d, Z_1 \) = constants in the sluicegate capacity equation; \( e, f \) = constants in the spillway capacity equation; \( Z_s \) = spillway crest elevation.

Solutions for \( S_k, Z_k \) and \( Q_k \), given \( S_{k-1}, Z_{k-1}, Q_{k-1} \) and \( I_k \), can be obtained by solving the Eqs. 22, 23 or 24. Note that Eqs. 22, 23 and 24 are nonlinear. They were solved iteratively. (See Chapter VII).

b. Controlled Reservoir Routing: When the reservoir release is controlled by the operating policy rather than the outlet capacity, the operating policy will replace Eq. 23 and 24 in solving for the values of \( S_k, Z_k \) and \( Q_k \). The operating policy is generally in the form:

\[ Q_k = P[S_{k-1}, SS, QC_{i,t-m}, QD_{j,t-n}, OC_L] \]  \hspace{1cm} (25)  

where \( S_{k-1} \) = reservoir storage level at day \( k-1 \), \( SS \) = season; \( QC_{i,t-m} \) = discharge (or stage) at control station \( i \) with \( m \) days lead or lag, \( i=1,2,\ldots \), and \( m=m(i) \); \( QD_{j,t-n} \) = discharge (or volume) at demand station \( j \) with \( n \) days lead or lag, \( j=1,2,\ldots \), and \( n=n(j) \); \( OC_L \) = operating constraints (e.g. in terms of pool levels).
VI. EXAMPLES OF SIMULATION MODEL INPUT AND OUTPUT

A. SYSTEMS INPUT

a. Historic Inflow

Daily flow records have been used for all studies. The data were obtained from a tape file of the Indiana Department of Natural Resources* (up to 1967) or directly from USGS Water Resource Data for Indiana (1968, 69, 70). The simulation model relies heavily on computer generated graphical output. This readily generated output can function, quite separate from its use in the building of the Wabash RR model, as a useful overview for any interested party of Indiana stream data characteristics. A sample of such output is displayed in Figure 15 for the 1968 water year. For comparison's sake the records for main stream gaging stations 6 (most downstream), 8, 10 and 36 (most upstream) are displayed in \( \text{ft}^3/\text{sec}/(\text{mi})^2 \). It is interesting to note in Figure 16 the differences between record samples of this type for tributaries and the resulting compound hydrographs for the main stream. The Figure 16 reflects the variability of precipitation and runoff over the basin and the consequent deviations from the area-proportioning assumption of Eq. 10. A statistical study of these records together with precipitation records was considered several times but could not be justified at this stage of simulation model building.

In Section IV-C-c the adjustments were outlined, that were made to these original records in order to obtain the more convenient (from the modeling point of view) proxy records. In Section II-C and the corresponding Figure 4, the various types of input to the RR model (block 9, Fig. 4) were discussed. The proxy records \( \{QI(t)\}_{\text{hist}} \) can be regarded as a mixture of historic data \( (Q_h, \text{Fig. 4}) \) and watershed model data \( (Q_m, \text{Fig. 4}) \) in as much Eq. 10 or 10a represent crude models of atmosphere-watershed behavior.

For the purpose of system benefit evaluation it is necessary to ascribe probabilities to simulation model outputs. To this end a series of individualized events (corresponding to block 7, Figure 4) were developed that can be ascribed a probability level. These synthetic inflows or gamma function

---

*This tape contains daily flow data for all Indiana gaging stations. A special program was designed to read this tape and convert it to a tape that can be part of the simulation model programs as run in the Purdue University computer center.
FIGURE 16 - COMPUTER PLOTS FOR NORMALIZED DAILY FLOWS IN 1968 FOR THREE TRIBUTARY AND ONE MAIN STREAM GAGING STATION
inputs are explained in Section VI-A-b; the probability aspects are treated in the companion reports. Probabilistic model input (denoted by block 8, Fig. 4) has not yet been made part of the simulation model input.

b. Synthetic Model Inputs

In order to define the effects of changes in operating policies for the Upper Wabash System of reservoirs, it is desirable to use sharply defined, generalized inflow patterns. The use of (portions of) the historical records \( Q(t) \) is not sufficient. They are too "fuzzy", both in time and space distributions, to clearly discern the sometimes minor effects of policy changes. For better definition one could use say triangular flood hydrographs. One could then select a shape for these triangular hydrographs and increment their areas, i.e. systematically vary the flood volume \( V \) given by:

\[
V = A \cdot R_i^1, \quad i = 1, 2, 3 \ldots
\]  

(26)

where \( A \) = watershed area \( \text{[mi}^2\text{]} \); \( R_i^1 \) = rainfall excess of storm \( i \text{[inch]} \); and \( R_i^1 = 1''', 2''', 3''' \ldots \). The respective probabilities of having such runoff magnitudes may then be determined by an independent study of basin runoffs. This will be part of the evaluation model construction (Fig. 1-34, 35; Fig. 4-12). For the present, quantitative sensitivity of the system to inputs \( R_i^1 \) is the only concern.

The investigation of systems sensitivity to operating policy changes was started out by using triangular hydrographs as synthetic model input. Concern arose that the incremental changes in stages and flows would be insufficiently realistic. It was decided to use somewhat less crude a model and to employ synthetic sub-basin hydrographs whose shape would correspond better to the historic hydrographs of simple (as opposed to compound) floodwaves measured for that sub-basin.

Now the exact flow hydrograph shape, \( Q(t) \), depends on many variables. The primary ones include drainage area \( A \), rainfall excess \( R \), and precipitation pattern \( P(t) \). Using one of the models discussed in Figure 12 (e.g. model #4, with parameters \( K \) and \( N \)) together with the superposition procedure outlined in Figure 11, one could convolute \( P(t) \) to yield \( Q(t) \). Using a matching model (see Figure 14) one could determine the best \( K \) and \( N \) for the sub-basin and the runoff volume in question.
However, as stated before, rainfall data were not involved during the first go-around in building the Upper Wabash simulation model. Consequently, the model \#4 in Figure 10 (i.e. the Kalinin-Milyukov or also Nash model) was fitted without the benefit of precipitation data. This procedure is defensible only on the grounds that tentative results are needed to guide model development; it should be replaced in the next round by hydrologically more responsible procedures. The necessary research for the Upper Wabash basin is nearly in hand (Lee, Blank, Delleur, 1972; Kisisel and Delleur, 1971).

For a number of basin inflow stations, over 20 simple floods of various size were selected. Their base flows were assumed to vary linearly from the first to the last day of the selected hydrograph and subtracted out. Further refinement appears unnecessary for the purpose at hand. The resulting hydrographs were fitted (using a matching criterion; see Figure 14) by model \#4 of Figure 12:

\[
Q(t) = 26.7 \frac{R \ast A}{K \Gamma(N)} \left( \frac{t}{K} \right)^{N-1} e^{-t/K}
\]  

(27)

Herein \( Q(t) \) = discharge [cfs]; 26.7 = conversion factor between [cfs] and \[\text{inch/} \text{mi}^2/\text{day}] ; \( R \) = rainfall excess [inch]; \( A \) = drainage area [\text{mi}^2]; \( K \) = shape parameter [days]; \( N \) = shape parameter [-]; \( \Gamma(N) = \text{Gamma Function} \) (for present fitting purposes, fractional values of \( N \) are acceptable). The shape parameters are constrained by:

\[
N \geq 1, \quad K > 0
\]  

(28)

The expressions for the "time to peak", \( t_p \), and the peak flow, \( Q_p \), for the above model are:

\[
t_{\text{peak}} - t_{\text{begin}} = t_p = (N-1)K
\]  

(29)

\[
Q_p = 26.7 \frac{R \ast A}{K \Gamma(N)} (N-1)^{N-1} e^{1-N}
\]  

(30)

In regard to matching one could use empirical \( t_p \) and \( Q_p \) values to solve for \( N \) and \( K \). As was the case with the estimation of the routing parameters, the least square matching criterion was used. The results for station 36 (Huntington) are shown in Table 12. The table lists the selected storms; the peak dates and flows (record wise and fitted model wise); the rainfall excess \( R \); the parameter estimates \( \hat{N} \) and \( \hat{K} \), the time to peak \( t_p \); and the flood duration
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<th>BEGINNING CFS</th>
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Average **= 6.46  Average **= .56  Average TP = 2.7  Average TB = 11.7

TABLE 12 - SYNTHETIC HYDROGRAPH OR GAMMA FUNCTION PARAMETERS FOR INFLOWS AT GAGING STATION #56
or base length, $t_B$. Again, as before, the values $N$ and $K$, also shown in Table 12, were obtained by simple arithmetic averaging. This means that, for a given station, the time to peak, and hence the base, is always the same, and that $Q(t)$ is strictly proportional to $V = A * R_1$ of Eq. 26. From a hydrologic point of view this model is rather deficient. Its improvement must await introduction of precipitation data. This is planned for the continuation of this work.

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<th>$K$</th>
<th>TIME TO PEAK (DAY)</th>
<th>TIME BASE (DAY)</th>
<th>POWER &quot;b&quot; FOR $Q_p = a^b$</th>
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**TABLE 13 - GAMMA FUNCTION CHARACTERISTICS FOR THE UPPER WABASH BASIN**

Averaged values for $N$ and $K$ (such as the $N$, and $K$ in Table 13) for the various input stations are shown in Table 13. The shapes, fixed in terms of $N$ and $K$, were used in the synthetic flow simulation. Since the same storms were used at these stations, it may be possible to yet isolate dependences on drainage area, $A$. Figure 17 shows such correlations. They were used to estimate averaged values for stations for which calculations such as represented in Table 12, had not been made. Note that Figure 17 shows that

$$t_p = c_1 A^{c_2} ; K = c_3 A^{c_4} ; c_2 = c_4$$

Inserting into Eq. (29) gives the approximation:

$$N-1 = \left(\frac{c_1}{c_3}\right) A^{c_2} c_4 = \frac{c_1}{c_3} = t_p/K$$

The ratio $c_1/c_3$ is shown in Table 13, last column, showing that $c_1/c_3 = 1.15$. 
FIGURE 17A - SYNTHETIC HYDROGRAPH PARAMETER CORRELATIONS
FIGURE 17b - SYNTHETIC HYDROGRAPH PARAMETER CORRELATIONS
B. SYSTEMS OUTPUTS

In the following some initial simulation model results will be presented. The body of selected simulation output is presented in the companion report on operating policy. It will be recalled that, except for adding or subtracting reservoirs as a whole, the consideration of operating policies is, for the time being, a primary use of the simulation model. Therefore, the computer programs have been designed to facilitate changes in operating parameters such as pool levels, recreation periods, minimum release requirements.

Initial results from the Upper Wabash Simulation model were obtained using the Muskingum Routing model (Model #4, Figure 12) as well as another two-parameter model, called the Second Order Linear model, which is described in Appendix A. The differences in simulation output using one or the other model, were minor. Consequently, only the results involving the Muskingum model are presented in the following.

Furthermore, the examples are limited to those involving the Corps of Engineers operating policies as given in Table 8, Figure 7 and as mentioned in section III-I. Results for other policies are found in the companion report on Operating Policies. It will be recalled that the present Corps of Engineers operating policies view the three upstream reservoirs (Huntington, Salamonie, and Mississinewa) as one operating unit. Their daily release decisions are interrelated (see Table 8 and Figure 7). The other two reservoirs (Lafayette and Big Pine) are treated more or less as independent units. Their daily release decisions relate to each other and to the three upstream reservoirs in an indirect and partial way, namely via the stage at the Lafayette gaging station (#7). That stage is involved at some point in all five policies.

Of the four examples of simulation output shown below, three were obtained using $\{QI(t)\}_{hist}$ for the 1950 water year. This year is the most recent of the three major flood years in the basin (cf. Table 4). The fourth set of examples involves synthetic inflows in the form of scaled Gamma Functions (see section VI-A-b).

a. Systems Flows in the Absence of Reservoirs

An option of the simulation model is the complete deletion of one or more reservoirs. Figure 10 is an example of results obtained when all five federal reservoirs (that have been built or are authorized) are being deleted.
FIGURE 18 - NATURAL FLOWS IN THE UPPER WABASH BASIN FOR THE WATER YEAR 1950
The computer drawn model output given in Figure 18 is identical to the actual gaging station records. In other words, only the simulation model's input and output portions were used and all else in the simulation model was bypassed.

The output of Figure 19, also for the 1950 water year, is obtained when the simulation model input consists only of the historical tributary inflows. The simulation programs then computes tributary proxy flows and side inflows. These are added in at the main stream junctions. All main stream gaging station hydrographs of Figure 19 are simulated, routed flows. These flows serve as a reference from which to measure (for the water year 1950) the effects of reservoirs and their operating policies. These flows cannot and are not intended to fully duplicate the gaging station records. As explained previously, the simultaneousness of side inflows, embodied in Eq. 10, and the typical deviations from Eq. 10 condition as revealed in e.g. Figure 16, and the use of average routing coefficients, causes differences between the \( \{Q(t)\}_{\text{hist}} \) and \( \{Q(t)\}_{\text{est}} \). On occasion these differences may be considerable. They do not fundamentally impair the purpose of the simulation program output, however, since the differences between outputs involving the same set of approximations are the final object of study.

b. Systems Flows Using Reservoirs

The output of Figure 20 also pertains to the water year 1950. Again the simulation model input consists only of prescribed, historical tributary inflows. Now, however, the Upper Wabash system contains everyone of its five reservoirs. They are operated according to present COE policies. A comparison with the Figure 19 output demonstrates that flood stage reductions are present at all main stream stations. As stated in section VI-B-a, a comparison of Figure 20 with the output of Figure 18 would not be in order.

The Figures 18, 19 and 20 are in a plotting format designed to overview a full year's simulation. It is too compact for easy quantitative comparison. In addition, a good study of simulation output does require more in the way of systems state records than only main river flows. The simulation programs have been built to provide large scale multiple outputs. The Figures 15 and 16 already contained samples thereof. The Figure 21 is a portion of another of these records showing the superposition of state parameters for the non-reservoir and the three-reservoir system for the water year 1950. It represents the conditions that would occur in the system as presently built (1972) and operated using present COE policies, should the hydrology of the
FIGURE 19 - ROUTED TRIBUTARY AND SIDE INFLOWS IN THE UPPER WABASH BASIN WITHOUT RESERVOIRS - WATER YEAR 1950
FIGURE 21 - COMPARISON OF SYSTEM STATE PARAMETERS FOR THE UPPER WABASH BASIN FOR THE NON-RESERVOIR CASE AND THE THREE-RESERVOIR CASE. WATER YEAR 1950; 1972 COE OPERATING POLICIES
water year 1950 be duplicated.

Note that the Salamonie Reservoir would almost be reaching its prescribed flood pool level on February 18 (i.e. day 141 of the water year) while, at the same time, the other two reservoirs display much unused storage space. This indicates that certain improvements in reservoir operating policies are feasible.

Again a comparison of the hydrographs at the Logansport gaging station demonstrates the significant reductions in peak discharge (or stage) achieved by the upper three reservoirs in the Upper Wabash basin. This reduction has less and less influence when one moves to more downstream river locations.

c. Systems Response to Synthetic Inflows

The computer drawn output of Figures 22 through 26 resulted from using synthetic tributary and side inflows in the form of gamma functions. Their generation was discussed in section VI-A-b. It will be recalled that their purpose was to insure accurate measurement of the incremental effects of policy changes. The Figures 22 thr. 26 demonstrate that synthetic inputs can also be useful in the study of systems behavior as it relates to the magnitude and distribution of hydraulic variables.

The outputs in Figures 22 thr. 26 were again obtained using the present (1972) COE operating policies for the case of a five reservoir system and the water year 1950. The inflow hydrographs (tributary plus side inflows) were scaled to represent 1", 2", 3", 4" and 5" of uniformly distributed runoff from the total Upper Wabash Basin. The use of regional flood volume frequency curves developed by Green and Hoggart (1960), permitted to associate return periods of 1.4, 5.2, 30, and 130 years to the 1", 2", 3" and 4" runoffs, respectively.

In all Figures unrouted Gamma Functions have been shown for the Covington station (station #6) using triangular signs. The decreases in flood peak flows made possible by five reservoirs and their present COE operating policies are readily seen and determined. It should be noted that the vertical scales of Figures 22 thr. 26 have been adjusted to contain the peak flow in the plots; the actual flow reductions for rarer floods are greater than those for less rare floods as indicated by lesser rainfall excess values.
SYNTHETIC TRIBUTARY INFLOWS CORRESPONDING TO 1.0 INCH OF RUNOFF, WATER YEAR 1972, ONE-OPERATING POLICIES, RETURN PERIOD 1.4 YEARS

FIGURE 22 - HYDROGRAPHS ALONG UPPER WABASH RIVER RESULTING FROM SIMULTANEOUS
Figure 23 - Hydrographs along Upper Wabash River resulting from simultaneous synthetic tributary inflows corresponding to 2.0 inch of runoff. Water Year 1950; five-reservoir case; 1972 COE operating policies, return period 6.2 years.
FIGURE 24 - HYDROGRAPHS ALONG UPPER WABASH RIVER RESULTING FROM SIMULTANEOUS SYNTHETIC TRIBUTARY INFLOWS CORRESPONDING TO 3.0 INCH OF RUNOFF. WATER YEAR 1950; FIVE-RESERVOIR CASE; 1972 COE OPERATING POLICIES, RETURN PERIOD 30 YEARS
FIGURE 25 - HYDROGRAPHS ALONG UPPER WABASH RIVER RESULTING FROM SIMULTANEOUS SYNTHETIC TRIBUTARY INFLOWS CORRESPONDING TO 4.0 INCHES OF RUNOFF, WATER YEAR 1990: FIVE-RESERVOIR CASE; 1972 COE OPERATING POLICIES, RETURN PERIOD 130 YEARS.
FIGURE 26: HYDROGRAPHS ALONG UPPER WABASH RIVER RESULTING FROM SIMULTANEOUS SYNTHETIC TRIBUTARY INFLOWS CORRESPONDING TO 5.0 INCHES OF RUNOFF, WATER YEAR 1972. FIVE-RESERVOIR CASE: RETURN PERIOD UNCERTAIN.
VII. COMPUTER PROGRAM FEATURES

A. GENERAL

The programs discussed in this report are processed by the Purdue Computer Center. The center has a CDC 6500 Computer System with a 96K memory capacity of 60-bit words. There are furthermore three IBM 7094 computers. Much of their capacity is used to drive Purdue's remote on-line computing system. This system counts around 60 remote teletype terminals by which the center may be accessed using PROCSY (Purdue Remote On-line Console System). Also available is a 563 CALCOMP plotter. Using computer library plotting subroutines one may obtain output on a line printer or/and as CALCOMP digital incremental plots. The latter option has been used extensively in the Wabash Simulation work. The three-dimensional CALCOMP formats were developed especially for this study. The programming language used for the simulation model is FORTRAN IV.

One can distinguish three phases in the computer communication needed for the simulation model. These are: (i) the compilation, checking, tabulation and transformation of the basic data; (ii) data analyses and simulation computations; and (iii) presentation of output, in particular the plotting.

For the first phase there are four input forms that were used: (i) card decks; (ii) magnetic tapes; (iii) permanent files (PFFILES); and (iv) teletype. The majority of programs in this study used a combination of card decks and PFFILES. A 800 bpi magnetic tape, which contains Indiana daily runoff data up to water year 1967, was obtained from the Indiana Department of Natural Resources. Data portions needed for the model are transferred onto PFFILES for operation of the simulation model. Daily runoff data after 1967 were punched in cards and then transferred into PFFILES.

B. PROGRAM DESCRIPTIONS:

Only some key programs that constitute the Upper Wabash simulation model are presented briefly in the following. More extensive documentation is maintained at the Purdue Water Resources Research Center. Auxiliary programs of developmental or intermediate use will not be mentioned at all. The presentation is limited to the mention of gross features, and of main program and
important sub-routines names. In addition a compact presentation of program interrelationships has been developed.

a. Moments Computation

The objectives of the Moments Computation Programs include: (i) compute and plot the direct runoff hydrographs for the inflow and outflow flood hydrograph in a given reach; (ii) compute the first 4 moments of the inflow and outflow flood hydrographs in a given river reach; (iii) compute the first and second moments of the kernel function of this reach.

The compacted flow diagram is shown in Figure 27.

---

**MAIN PROGRAM**

1. INPUT station number, year, reach number of the flow data
2. INPUT drainage area, beginning days, end days and year for each storm
3. CALL QTRANF to obtain daily flow
4. Compute the direct runoff (base flow is assumed to be a straight line)
5. PUNCH storm hydrographs
6. CALL MOMNT to obtain hydrograph volume and first four moments
7. Compute the first two moments for the kernel function
8. Use library subroutines—quick plots QIKPLTL and QIKPLTP to plot (4)
9. Repeat 1 to 8 for 3 reaches

**QTRANF**

1. INPUT the year and flow data Q(I,J)
2. Determine number of days in the year
3. Transform Q(I,J) into one-dimensional form X(k)
4. OUTPUT 2 and 3

**MOMNT**

1. INPUT flow hydrograph and drainage area
2. Determine the volume, in inch, for the direct runoff
3. Determine the first four moments and variance
4. OUTPUT 2 and 3

---

**FIGURE 27 - MOMENTS COMPUTATION PROGRAMS**

b. Determination of Routing Coefficients

The objectives of the Routing Coefficient programs were to determine the best fit (by least square matching) sets of the routing coefficients
for Muskingum and Second Order Linear models. The compacted flow diagram is presented in Figure 28.

**MAIN PROGRAM**

1. READ Channel System Characteristics
2. READ flow data QD(I,J)
3. CALL QTRANSF to transform QD(I,J) into one dimensional X(k)
4. Repeat 2 and 3 for all inflow stations
5. Compute side inflow by area proportion.
6. Route all upstream reaches by given predetermined coefficients
7. Assign a mxn matrix of values for 2 routing constants
8. CALL MUSK or CALL ROUT for each matrix unit
9. Compute correlation coefficient "R" for each unit of the matrix
10. Two routing constants were determined from the matrix unit with highest "R"
11. CALL WBPLOT to plot the observed and computed results
12. PRINT out the matrix

**QTRANSF**

1. INPUT the year and flow data Q(I,J)
2. Determine number of days in the year
3. Transform Q(I,J) into one dimensional form X(k)

**MUSK**

1. INPUT coefficients C₀, C₁, C₂
2. and flow Iₜ, Iₜ₋₁, Qₜ₋₁
3. Compute Qₜ by
   
   \[ Qₜ = C₀ Qₜ₋₁ + C₁ Iₜ + C₂ Iₜ₋₁ \]
4. OUTPUT Qₜ

**ROUT**

1. INPUT coefficients α, β, γ, and flow Iₜ, Qₜ₋₁, Qₜ₋₂
2. Compute Qₜ by
   
   \[ Qₜ = α Qₜ₋₁ + β Qₜ₋₂ + γ Iₜ \]
3. OUTPUT Qₜ

**WBPLOT**

1. INPUT observed discharge and computed discharge series
2. CALL library subroutes PLOT, SCALE, AXIS, NUMBER, SYMBOL, and LINE
3. PLOT out two discharge series

**FIGURE 28 - DETERMINATION OF ROUTING COEFFICIENTS**

c. System Test Programs

The objectives of the System Test programs are: (i) provide integrated simulation frame for the river reach components and reservoir components of the simulation model; (ii) test the routed system outflows against the ob-
served outflows; (iii) simulate Upper Wabash system for the no reservoir option; (iv) plot observed and simulated flows at the end of the Upper Wabash River System.

The compacted flow diagram is shown in Figure 29.

---

**MAIN PROGRAM**

1. READ Control data
2. CALL YEAR to get flow data
3. CALL FREQ to compute frequency distribution for observed data
4. Compute inflow and lateral inflow for the reach
5. CALL ROUT or CALL MUSK
6. Repeat 4 and 5 for reaches I through VI
7. CALL FREQ to compute frequency distribution for simulated flow
8. PUNCH and PRINT both the observed and simulated flow series

**YEAR**

1. INPUT the station number, year, tape number (I)
2. READ data from (I)
3. Determine number of days in the year
4. OUTPUT 2 and 3

**FREQ**

1. INPUT flow series and base level
2. Sorting peak value above the base level
3. Compute the density distribution for peaks
4. OUTPUT 3

**MUSK**

1. INPUT coefficients $C_0$, $C_1$, $C_2$ and flow $I_t$, $I_{t-1}$, $Q_{t-1}$
2. Compute $Q_t$ by
   $$Q_t = C_0 Q_{t-1} + C_1 I_t + C_2 I_{t-1}$$
3. OUTPUT $Q_t$

**ROUT**

1. INPUT coefficients $\alpha$, $\beta$, $\gamma$, and flow $I_t$, $Q_{t-1}$, $Q_{t-2}$
2. Compute $Q_t$ by
   $$Q_t = \alpha Q_{t-1} + \beta Q_{t-2} + \gamma I_t$$
3. OUTPUT $Q_t$

---

**FIGURE 29 - SYSTEM TEST PROGRAMS**

d. Gamma Function Computation

The objectives of the Gamma Function Computation programs include: (i) compute the characteristic elements of chosen storm hydrographs for a given gaging
station; (ii) determine best fit coefficients, K, N of the gamma function for a given flood; (iii) correlate flood hydrograph characteristics (volume, time to peak, etc.) with K, N.

The flow diagram is shown in Figure 30.

e. Upper Wabash Simulation Model—COE Five Reservoir Policy Programs

The objectives of the COE five-reservoir policy programs included: (i) simulation of the system under the 1972 COE operating policies for all five federal reservoirs; (ii) simulation of the effects on the downstream reaches when one or more of the five reservoirs is deleted; (iii) simulation of the effects on the downstream reaches resulting from changes in operational parameters (pool levels, recreation periods, etc); (iv) plotting of the simulation results.

The flow diagram is shown in Figure 30. This Figure contains the most comprehensive and compact listing of the total preliminary Upper Wabash Basin Simulation Model.
*****PROGRAM MAIN*****

1. READS year, last day-to-be-plotted, number-of-days-to-be-plotted
2. CALLS STRMPIT

*****YEAR*****

1. READS daily discharge in binary off tape
2. Reformats data for YEAR and station for calling program

*****ROUTE*****

1. Calculates reach inflow
2. Calculates reach outflow
3. Repeats 1 and 2 for each reach

*****STRMPIT*****

1. CALLS YEAR for each station
2. Sets parameter constants for 'route'
3. CALLS ROUTE for natural flow
4. CALLS WABCORP
5. CALLS ROUTE for station 7 flow
6. CALLS RELESLF
7. Calculate new routed flow for Station 7
8. Repeat 6 and 7 until new routed flow is equal to last routed flow for Station 7
9. CALL RELESBP
10. CALLS ROUTE for reservoir flow
11. Sets arrays for 'PERSPEC'
12. Repeats 3 through 11 for each day
13. Sets 'PERSPEC' parameters
14. CALLS PERSPEC

*****WABCORP*****

1. Checks each reservoir for flood level
2. Determines maximum release
3. Calculates release for each reservoir
4. CALLS NVAL

*****NVAL*****

1. Calculates storage
2. Determines which reservoir elevation was requested and calculates the elevation

*****RELESLF*****

1. INPUT daily inflow, control station's discharges and reservoir storage at the beginning of the day
2. Compute reservoir release discharge (corp's policy)
3. Compute reservoir storage at the end of the day
4. OUTPUT 2 and 3

*****RELESBP*****

1. INPUT daily inflow, control station's discharges and reservoir storage at the beginning of the day
2. Compute reservoir release discharge (corp's policy)
3. Compute reservoir storage at the end of the day
4. OUTPUT 2 and 3

******PERSPEC******

Plots "three-dimensional" plot of flow for stations 36 to 6 on the Wabash for specified number of days

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FIGURE 31 - UPPER WABASH SIMULATION MODEL-COE FIVE RESERVOIR POLICY PROGRAMS
VIII. SUMMARY AND CONCLUDING COMMENTS

A. GENERAL

1. A beginning was made with the construction of a surface water simulation model for the State of Indiana. The initial portion of such a model covers the upper part of the Wabash, the river that drains better than three-fourth of the state. This report concerns primarily the model building phase. Companion reports deal with the operating policy and the output evaluation questions.

2. It has been possible to construct a model for the 8,000 mi$^2$ Upper Wabash Basin that can serve to test reservoir operating policy changes for their effect on flood peak reduction, recreation pool levels, low flow augmentation, and the like.

3. From the hydrologic and hydraulic points of view the model may yet be further refined. How much additional effort in that direction is warranted in terms of obtaining a first-cut simulation model is a question to be answered after the present programs have seen initial use in engineering-economic evaluation of the model. This phase is at the moment far from being complete.

4. A case can be made for model refinement and continued updating quite separate from its use in reservoir benefit evaluation. The merits of doing so include:
   
   (a) surface water simulation models provide a framework to guide improvements in the organization and interfacing of many water resources data;
   
   (b) the input-output programs needed in simulation provide means for easy access, efficient graphical output and overview of water data;
   
   (c) the actual use of data in a simulation model context can provide guidance in data collection decisions.

B. MODEL FEATURES, BUILDING DIFFICULTIES, AND LIMITATIONS

5. The model simulates the surface water parameters (daily reservoir levels, daily releases, and daily river flows) throughout an 8,000 mi$^2$ basin containing five surface reservoirs and six main river reaches. These parameters are generated from (a) historic tributary flows and (b) synthetic hydrographs. The key element in this generation is the reservoir operating policy.
6. The model is specific to the Upper Wabash Basin. No special effort was made to provide a generalized program. However, the specific model building difficulties and resulting limitations are probably typical of other simulation efforts. No systematic discussion of these specific difficulties has been found in the literature.

7. The difficulties in building a first approximation to a complete (i.e. from input data collection to benefit evaluation) simulation model include:

(a) gaging station records that cover different periods. Although methods exist to complement records, they will require much other data (particularly precipitation data) and/or involve efforts whose magnitude cannot be justified prior to a first complete model design.

(b) ungaged tributary and overland side inflows will almost always be encountered. They must be accounted for. For this accounting problem the same comments of point 7a hold.

(c) even simple operating policies tend to give rise to complicated logic and every change in operating policy tends to require much programming effort. One reason is the use in practice of control stations throughout the basin which makes for feedback features in the policies.

(d) Another is incorporation of forecasts of flow conditions in operating policies. In the present model the absence of atmospheric and watershed phases limit a fully realistic simulation of the surface water system.

8. Several limitations are at present built into the model. They can be largely removed, but at a price in terms of new data collection and analysis. As said before, a judgment on the trade-off between an improved quantity model and the merit of the improvements at the benefit-evaluation stage is yet to be made. The present limitations include (see also point 7):

(a) Use of historical records which will never be duplicated in the future;

(b) Use of synthetic records based on the assumption of uniform, simultaneously commencing runoff over the basin;

(c) river reach routing coefficients determined as averaged values from a study of flows of different magnitudes (total records were used; not only floods). The coefficients can be expected to depend on stage. They will also depend on river bed geometry which is subject to change both in the past as well as in the future.
C. PRELIMINARY RESULTS

9. Computer generated graphs have been presented that clearly portray the potential of the model to yield quantitative information on the reservoir levels and river flow rates in the Upper Wabash Basin. These results can be generated for any combination of existing and planned reservoirs and for several reservoir operating policies.

10. The operating policies used to generate the samples of model output presented in this report are those employed (or designed for authorized reservoirs) by the Corps of Engineers. One of the results (see Figure 21) indicates that some improvement in those policies may be possible.

11. Little difference could be observed in the hydrographs at the outlet of the basin (i.e. at the Covington gage, station #6) when using routing coefficients obtained for the Muskingum model or those obtained for the Second Order Linear model.
IX. REFERENCES


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X. APPENDICES

The Appendices A, B and C are found in the companion report entitled: "Operating Policy for the Upper Wabash Surface Water System".
LIST OF NOTATIONS

\( A_{uj} \) = area of ungaged watershed \( j \) [mi\(^2\)]
\( C \) = Chezy coefficient \([(ft/sec)^{2.5}]\)
\( C_{1,2,3} \) = coefficients in finite difference form of the Muskingum Equation [ - ]
\( D(t) \) = expected value of \( V_i(t) \)
\( F \) = Froude Number = \( q_o / (g y_o)^{1/2} \) [ - ]
\( g \) = gravitational acceleration [ft/sec\(^2\)]
\( H \) = head on weir [ft]
\( h(t) \) = unit impulse response or kernel function
\( I \) = systems input
\( I_k \) = inflow for day \( k \) [ft\(^3\)/day]
\( I(t) \) = daily inflow
\( K \) = coefficient in Muskingum Equation [day]
\( L \) = dimensionless channel length
\( M \) = first moment [day]
\( n \) = number of stages; index
\( O \) = systems output; outflow [ft\(^3\)/sec]
\( O_k \) = outflow for day \( k \) [ft\(^3\)/day]
\( O_m(t) \) = daily simulation model output flows [cfs]
\( P(\ldots) \) = predicted value of \( \ldots \)
\( q \) = flow per unit width [ft\(^2\)/sec]
\( q_o \) = base flow per unit width [ft\(^2\)/sec]
\( Q_h(t) \) = historic daily flows [cfs]
\( \{Q_l(t)\}_{\text{hist}} \) = historical proxy inflow [cfs]
\( Q_{LY} \) = yearly inflow into a reach [ft\(^3\)/yr]
\( Q_{L7} \) = 7 consecutive days - once in 10 years low flow [cfs]
\( (Q_0(t))_{est} \) = routed historical proxy flow [cfs]
\( (Q_0(t))_{hist} \) = historical proxy outflow [cfs]
\( Q_y \) = yearly outflow from a reach \([\text{ft}^3/\text{yr}]\)
\( Q_r \) = reference flow \([\text{ft}^3/\text{sec}]\)
\( Q_{u,j} \) = ungaged flow at node \( j \) \([\text{ft}^3/\text{sec}]\)
\( R_i \) = rainfall excess [inch]
\( R(t) \) = probabilistic variable
\( S \) = storage \([\text{ft}^3]\)
\( S \) = Second moment \([\text{day}^2]\)
\( S_o \) = slope of channel bottom \([\text{ft/ft}]\)
\( t \) = time [day]
\( t_p, TP \) = time to peak [day]
\( V_i(t) \) = physical variable \( i \)
\( x \) = coordinate along channel bottom
\( X \) = coefficient in Muskingum Equation \([-]\)
\( Y_o \) = depth of base flow [ft]
\( Z \) = (reservoir pool) elevation [ft]
\( \Gamma(N) \) = gamma function
\( L \) = length of channel reach [ft]
\( \tau \) = time delay [day]