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A Continuous Simulation Model for Regional Stormwater Management Planning Analysis

by

A. Charles Rowney

A thesis presented to the School
of Graduate Studies and Research
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ABSTRACT

A mathematical simulation model suitable for continuous simulation of stormwater quality and quantity in a regional river basin has been developed. This model responds to an identified need for an improved capability for planning analysis of urban stormwater pollution. Developed specifically for a regional planning level analysis, the model contains a number of features which distinguish it from existing alternatives. The model, which is structured as a modular system of functional routines oriented towards a sequential analysis of time series water quantity and water quality data, is flexible and comprehensive. The scope and level of sophistication of the model is appropriate for typical stormwater quality manangement investigations as presently practiced, and the model can be readily expanded to meet changing needs.

To provide an economical and more flexible alternative instream routing capability, a routing scheme based on convolution of a response function is included in the model. A method for deriving the required response function has been developed which is different from existing approaches; testing against established approaches shows that this method produces results consistent with two-dimensional models, but requires a relatively smaller amount of computer time.

An alternative catchment runoff volume algorithm has also been developed. This approach uses an Antecedant Precipitation Index to update soil moisture conditions and thereby eliminates the complexity of typical loss functions based on soil moisture accounting. Testing of the model against observed flow data and against other models shows it to be superior to comparable alternatives, and more economical.

Test applications of the model have been carried out against an hypothetical catchment; this test catchment was consistent with conditions encountered in Ontario. The results of this testing indicate that the model is appropriate for its intended purpose. A principal conclusion of this testing is that single valued criteria for water quality management are inappropriate; an alternative is suggested in the form of an allowable design space, which is more comprehensive than single valued approaches.

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External support from several sources is also acknowledged. Dr. Lam at the Canada Center for Inland Waters in particular is thanked for his review of the instream routing algorithm developed in this research. Opportunities were provided to test the model on behalf of the City of Calgary and the City of Ottawa in practical applications, and the insight provided by these opportunities is appreciated. John MacKensie and Philip Cheung, who discovered and reported 'bugs' in the model code during these practical applications, are thanked both by myself and on behalf of the future users who will not suffer the same problems.

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1.0 Introduction and Problem Definition

1.1 General

A facet of water resources engineering which has come to be of increasing importance in recent years is the management of water pollution. It has been well documented that stormwater can be a significant carrier of pollutants in some river basins, and as a result a major effort has been made to develop analytical techniques and control technology so that impacts of pollutants borne by stormwater can be assessed and if necessary mitigated (Amades et al, 1980; Beard et al,1979; Bedient et al, 1980; Delleur et al,1979; Meadows et al, 1978; Oliver et al, 1981; and others.) This effort has been accompanied by a proliferation in available means of analysing stormwater pollution, and a growth of experience in application of these methods. A major purpose in this development has been the creation of mathematical models capable of simulating physical processes related to the problem of stormwater pollution.

These models now provide the capability to assess many common stormwater quality management problems; however, as discussed in later sections of this chapter, the present state of the art of simulation models still leaves much to be desired. This situation has arisen partly because the need for planning of stormwater quality management has been generally recognized only relatively recently, and engineering practice in this area is still developing. As a result, available models applied in this area tend to be, for a number of reasons, unsuitable for present engineering practice; in some cases this is because they are applied in contexts for which they were not intended.

This research was undertaken to create a simulation model suitable for regional stormwater quality management planning, as an improvement on and alternative to existing models. This report describes experience in the application of existing models, identifies weaknesses in those models, and proposes a simulation approach which represents a significant improvement over existing methods. Tests of the proposed model and its constituent algorithms are described, and an application of the model in the form of a test of present Ontario regulatory policy is presented.

1.2 Problem Definition

This section identifies basic characteristics of a simulation analysis for stormwater quality management planning, to provide an appropriate perspective to view existing models and approaches to analysis.

Specifically the following issues are discussed:

a. current approaches to specification of stormwater pollution control criteria are briefly discussed;

b. a definition of the general function of a planning model is adopted; and

c. a concept of the physical system which is typically at issue in a regional stormwater pollution management planning study is developed.

1.2.1 Receiving Water Protection

Legislation

The principal Canadian federal legislation which is pertinent to the problem of water pollution control in the context of urban drainage has been cited by the Urban Drainage Policy Committee of the Canada-Ontario Agreement (COA) on Great Lakes Water Quality as the Environmental Protection Act (COA,1980).

Other environmental legislation in Canada is predominantly in the form of provincial regulations. In Ontario, this includes: the Ontario Water Resources Act, the Environmental Assessment Act, the Conservation Authorities Act, the Planning Act, the Municipal Act, the Local Improvement Act, the Drainage Act, the Lake and Rivers Improvement Act, the Beds of Navigable Waters Act, and the Fisheries Act. These Acts are not specific to the problem of stormwater pollution management, but do provide the provincial ministries with the means to protect receiving waters from discharges of deleterious substances, including pollutants borne by stormwater runoff.

More specific regulations in Canada are still in a developmental stage. In Ontario, for example, the Ministry of the Environment (MOE) has indicated (MOE, 1983) that at present the primary documents which are pertinent to the application of this authority to stormwater quality management are (i) the Provincial Water Quality Objectives (PWQO) (Ontario Ministry of the Environment, 1978) and (ii) the Proposed Model Policies for Urban Drainage Management (COA, 1980). The PWQO sets concentration limits on pollutants and defines Ontario policy as regards surface water protection. The Proposed Model Policies, prepared in response to the needs of the Urban Drainage

Subcommittee of the Canada-Ontario Agreement on Great Lakes Water Quality, present a proposed policy for management of stormwater quantity and quality, and does not have the status of an officially adopted policy document at present.

The matter of primary interest here is that, in Canada, the legal recognition of, and requirement for, a specification of instream water quality objectives for stormwater has only occurred relatively recently, and appears to be still in a state of development. This is paralleled in the United States where, as indicated by Swank (1980), the primary legislation pertinent to stormwater quality management appeared in the early 1970's, and where the first legislation requiring an analysis of instream impacts of pollutants was introduced in 1972 (and modified in 1977).

As is discussed in Section 1.3, this has resulted in some of the available models predating present pertinent legislation and, concomitantly, present needs of analysis.

Criteria

Prior to the early 1970's pollution control criteria were usually expressed in terms of allowable effluent conditions. One of the reasons that this approach was favoured in the past is that it facilitated the regulatory process. More recently however, regulations and engineering practice have become more oriented towards instream conditions. This shift has occurred for a number of reasons, including the following:

- As pointed out by Minton et al (1978), criteria based on receiving water conditions are a more direct measure of protection than those based on effluent loadings. From an environmental point of view, the primary concern is the level of pollution which occurs in the receiving stream, rather than in the effluent stream itself. Receiving water conditions are not explicitly dealt with if criteria are stated in terms of effluent conditions.

- There are economic reasons to apply controls based on instream conditions. For example it can be considered that where actual pollutant levels are lower than objectives, an assimilative capacity for pollutants exists. If this difference is viewed as an allowable degradation, pollutants can be discharged at a rate which causes the objective concentration to be achieved instream. This can mean costs of pollutant control are reduced.

- It has been pointed out that, in principle, pollution control requirements should be a function of the intended use of the receiving water. An assessment of pollutant discharges which recognises this must be made on the basis of instream conditions.

As a result, criteria for instream quality control are now generally expressed in terms of instream concentrations or intensities. Couching pollution management standards in terms of instream conditions can therefore be considered at this time to be an accepted and necessary practice in general planning for water quality management. As described below, this trend has limited the current usefulness of models oriented towards catchment loadings only.

Although most legislation does not yet espouse the concept, an additional change in specification of pollution control criteria appears to be emerging. This takes the form of a recognition of the inadequacy of water quality management criteria expressed as single valued objectives. Traditional and in most cases present practice in water pollution control (Southerland, 1981) has been to assess instream pollutant impacts in terms of a single, low-frequency design condition. This practice facilitates legislation and simplifies analysis, since a single design condition can be defined. Unfortunately, there are a number of difficulties inherent in this approach.

o As noted by Minton et al (1978), water usage varies not only geographically but temporally. Seasonal water changes in water use (for example in the form of summer bathing) should in principle be reflected by seasonal specifications for instream conditions.

o Medina points out (1983) that analysis of cost effectiveness where damages are dependent on degree of pollution requires a knowledge of frequency of occurrence of more than one design condition.

o Seasonal variations in instream conditions (such as temperature), as noted by Doudorouf et al (1970), may also require a seasonally based analysis.

It is therefore preferable from both theoretical and practical perspectives to account for the temporal variation of instream conditions. The U.S. Environmental Protection Agency (EPA) appears to have accepted the need for information based on more than a single design condition; a recent (1979) EPA report recommended that the numbers or frequency of exceedences of a standard should be specified for stormwater quality problems. At this time, however, Canadian legislation does not appear either to require or preclude this type of assessment. The Ontario PWQO for example, do specify an analysis based on instream conditions; however they generally state the requirement for control in terms of a single objective concentration.

The need for an analysis which is responsive to instream temporal variations in concentration has implications in terms of the simulation model which can be used. Primarily, a continuous simulation is required due to the inherent limitations of a design event simulation of water quality. As cited by Medina (1982) there are several practical reasons why a design event approach to analysis is inadequate:

- A reliable probability or frequency of occurrence cannot be assigned to the design condition (Linsley and Crawford, 1974).

- The variability of runoff pollutant loads means that the most critical instream condition cannot necessarily be associated with a low flow period (Heaney, et al., 1977).

- There is no accepted way of defining appropriate antecedent conditions for the design event (Heaney, et al., 1977).

It follows that, in models applied to the problem of stormwater pollution control, those models which are not and cannot be used for continuous simulation have inherent disadvantages when applied to regional stormwater quality analysis.

Summary

From the foregoing discussion, it appears that present engineering practice requires that a simulation model for stormwater quality management planning should:

1. be capable of defining the instream impact of the pollutant load; and
2. be based on a continuous simulation approach.

1.2.2 The Role of a Planning Model for Stormwater Quality Management

Duttweiler et al (1976), Ellis et al (1981), Grimsrud et al (1976), Loucks et al (1981), Strzepek et al (1981), and others have contributed to definition of the role of planning in water resources. Definition of the planning process is somewhat subjective, so there is no clear basis for selection of any particular perspective from which the problem of water quality management may be viewed. The common nature of the problem, however, tends to result in certain common concepts. A view which is accepted here as representative and reasonable was presented by Jamieson (1978). This definition divides the analytical process into three phases; planning, design, and operation:

a) The planning phase takes place in the initial stages of development, and is characterised by a general lack of detailed data. This phase should screen a large number of possible alternative plans or solutions and should result in a reduced number of alternatives for further analysis. The planning phase may also provide information on monitoring networks, or on optimal receiving water design objectives.

b.) The design stage operates on the data and feasible alternatives in detail, and results in precise sizes, costs, and other data necessary to permit implementation of a single plan.

c.) The operational stage is designed to provide information used in regulating the system as implemented. Operational models may have a role in interpreting data measuring system performance, or in evaluating the impact of an observed or contemplated change in inputs or system components.

Planning models may therefore be considered to provide a means of resolving and comparing the effects of alternative management plans.

The planning model may be further classified (Loucks et al, 1981; Stzrepek et al, 1981) as either descriptive or optimizing. Descriptive models in some way represent the impact of a possible design condition in a manner which permits the analyst to better understand the implications of that condition. Optimizing models actively select or derive an optimum plan in some systematic manner. Although there is a considerable history of use of optimizing techniques in water quality management, the approaches which have thus far been developed do not lend themselves to a detailed analysis of river systems where highly variable pollutant loads, such as those generated by stormwater discharges, are at issue. In such cases, system design is generally approached by systematic use of descriptive models which resolve the 'best' design solution after a number of trial simulations. Figure 1.1, taken from a recent Ministry of Environment report (Rowney et al, 1984) is illustrative of this process. As shown, the design of a regional control pond system involves an indefinite series of simulations and incorporates both well

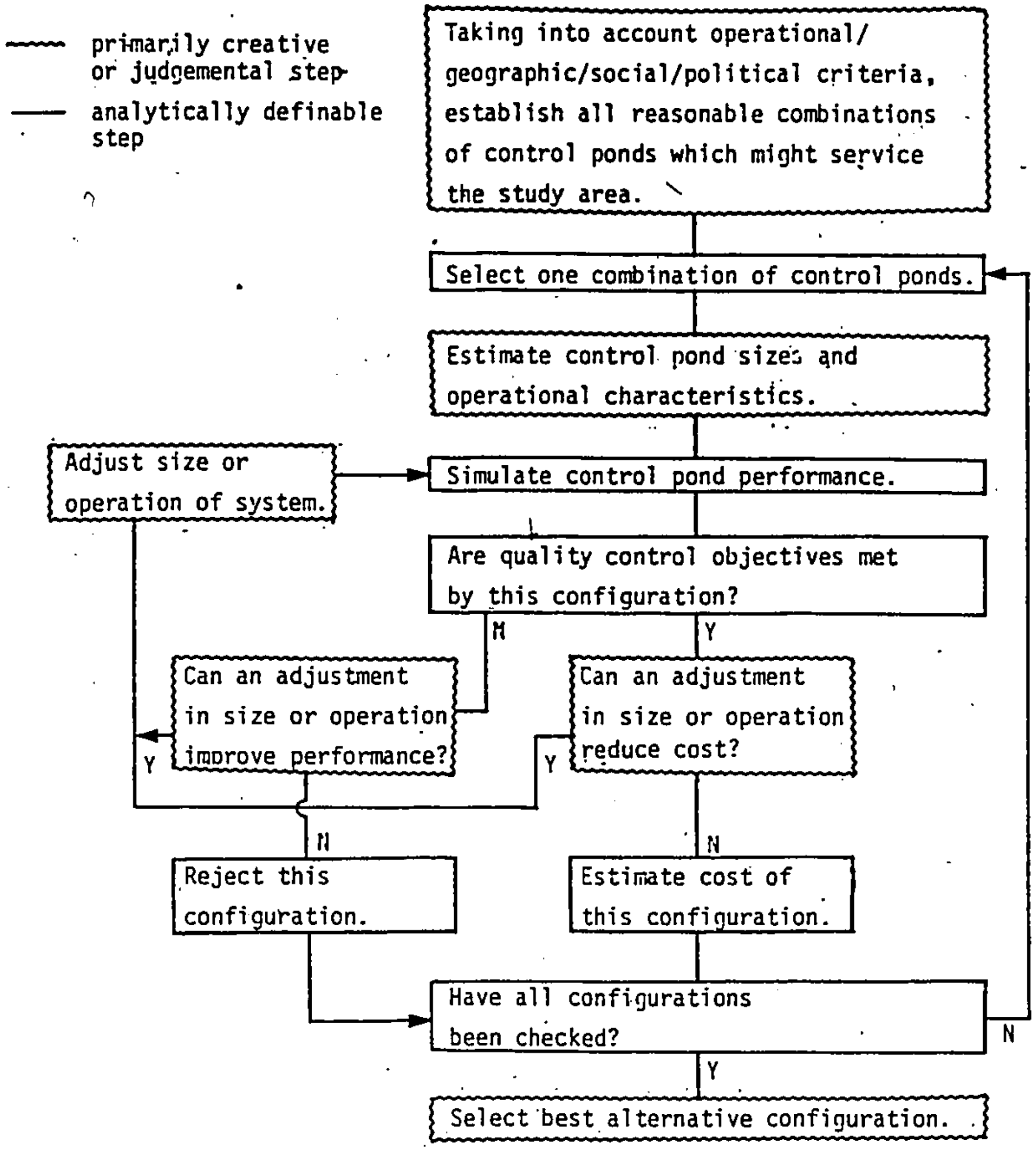


FIGURE 1.1 A VIEW OF THE PROCESS OF CONTROL POND SYSTEM DESIGN.

defined (analytically tractable) steps and poorly defined (creative or undefined) steps. It is the difficulty in providing an adequate description of the various decision variables which has limited the development of optimizing models for use in stormwater quality management to date.

Whatever type of analysis or planning model is applied, it should be capable of quantifying instream impact of pollutant loads. This reflects present legislative approaches to water quality management which, as discussed above, appear to be generally oriented towards specifying environmentally acceptable conditions in terms of instream concentrations. It also reflects the recognition that costs and benefits are related to extent of damage or degree of control (Loucks et al, 1981); establishing these requires a quantitative instream analysis.

Implicit in the above definition is the need for the planning model to achieve this analysis economically and to do so in an environment where little detailed information is available for model calibration and application. As demonstrated below, some existing models which are otherwise suitable fail to meet this criterion.

It is also noted that the water quality planning model will generally be used in areas where water quantity analysis is concurrently being done. Further, there will be additional analyses at later stages of design, and it should be possible to reconcile later results with those of the planning study. To achieve these latter objectives it is required that the runoff portion of the quality planning model is consistent with present engineering practice in water quantity analysis.

Summary

In addition to the need to assess instream conditions in the long term, the regional stormwater management planning model should be:

3. economical to apply;

4. oriented towards the limited level of data typically available in the context of a planning study;

5. consistent with other and subsequent quality or quantity simulation models which may be used in the same geographic region; and

6. able to incorporate, where appropriate, results of other simulation models used in the same area.

1.2.3 The Physical System

Achieving the analysis described above requires a model package which can represent individual components of the physical system and their interrelationships. No exact definition of the physical system for which this must occur can be made, since no two physical systems or management problems are alike. However, examination of past analyses allows a definition of an appropriate general conceptual system for which a general planning model can be created.

Development of major urban centers in river basins in North America has typically resulted in a system of the type shown in Figure 1.2, taken from the master drainage plan for the Town of Ajax in Ontario. As indicated, development begins at the lower end of the basin, and proceeds upstream. A few specific examples of this progression may be found in a number of Ontario cities, including: Ottawa on the Rideau River (RRSMS, 1981), Ottawa on Sawmill Creek (City of Ottawa, 1983). Ajax on Carruthers Creek (Town of Ajax, 1980), and Oakville on Golf Course Creek (Town of Oakville, 1980). Examination of these and other systems discloses that a common need for stormwater quality management planning is the ability to assess urban, rural, and undeveloped areas connected by a receiving stream. Also, it is noted that a commonly cited stormwater management control device for developing areas is a stormwater detention pond for water quantity control.

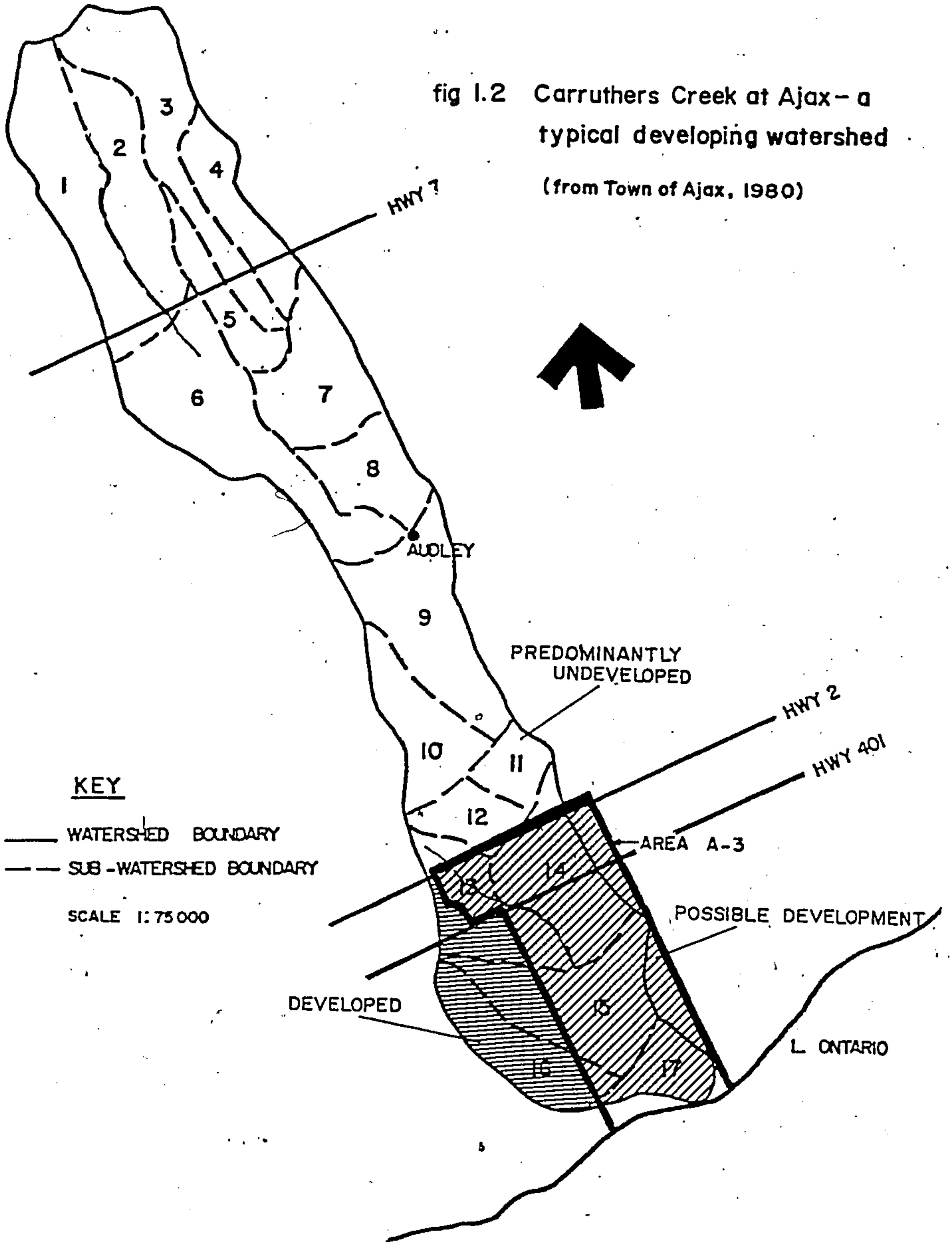


fig 1.2 Carruthers Creek at Ajax – a typical developing watershed

(from Town of Ajax, 1980)

It is therefore implicit that a general model for planning analysis of river basins at a regional level should include the means to assess a number of sub-basins linked by a receiving river network. Further, the analysis and mitigation of impacts implies control, so capability to assess impact of control devices such as stormwater detention pond should be incorporated into such a model.

Summary

It is clear that in addition to the other characteristics described above, a planning model should have the scope to simulate:

7. catchment rainfall/runoff response, for both rural or undeveloped and urban areas;

8. control devices which may affect the catchment runoff response; and

9. instream transport of runoff and pollutants from catchments with or without control devices.

It is recognised above that no two systems or planning situations are alike, and it is implicit in this recognition that the model should also be:

10. structured to make it possible to readily incorporate changes to fundamental interpretive or process algorithms.

1.3 Review of Existing Simulation Approaches

This section reviews currently available approaches to stormwater simulation, and presents some examples of application of these approaches. The objective of this review is to identify the applicability of existing approaches to the problem of regional stormwater quality management planning described above.

1.3.1 The Current State of the Art of Simulation Models.

Several authors have reviewed the available analytical models. Donahue et al (1981) made a systematic comparison of hydrologic models for detention basin analysis and found few to be suitable for planning, as a result of inappropriate data requirements. None of the models cited were capable of quality analysis. Delleur et al (1979), in a recent critical review, discussed available simulation techniques, and described a number of currently available models. Little basis for comparison was found regarding accuracy or validation, but some of the models could be compared in terms of overall capability. Grimsrud et al (1976), reviewed available technology for water quality simulation models for planners, on behalf of the United States Environmental

Protection Agency. Most recently, Whitehead (1984) reviewed the international application of mathematical models of water quality and pollutant transport. In general, examination of the models described in the above reviews shows that there are few simulation models available which might be considered for river basin stormwater quality management planning.

A general statement of one of the major difficulties with existing models can be taken from Frere, (Haith et al, 1981) who classifies existing models as either 'planning' or 'research' types. He defines the latter as being characterised by a requirement for data beyond that typically available at a planning stage, and by excessive descriptive detail. He notes that the former, suitable for application in the context of a planning study, represent a very small fraction of the available total. As such, the number of models which can be used in regional quality management studies is currently relatively limited.

A second major difficulty with available models is that typically the scope of simulation is inappropriate. Models which have the capability to simulate the physical components associated with a regional stormwater quality management planning analysis are few in number, and those that exist tend to be more complex than can be justified for a planning analysis.

At present, the state of the art of stormwater quality simulation models can be represented by four specific models, namely STORM, SWMM, HSPF, and the Dorsch model (referenced below). These are briefly described and evaluated below.

One of the most widely applied models for planning analysis of pollution control facilities exists in the form of a continuous simulation package distributed by the Hydrologic Engineering Center under the acronym STORM (Storage Treatment Overflow and Runoff Model). The model was originally developed in 1974 as a part of the development of a Master Plan for San Francisco combined sewer abatement. Created as a planning tool, the model was viewed as a more appropriate alternative to existing approaches (Roesner et al, 1974). The model represents the best available technology for stormwater quality management simulation available up until 1974. In fact, the model is still one of the best available water quantity/ quality planning models, and the literature demonstrates that the model is still commonly used for that purpose.

Roesner used a build-up/ washoff method to simulate pollutant generation on the watershed. This approach is also applied, without substantial difference, in the more recently developed models described below. The model originally used a volumetric coefficient to calculate direct runoff quantity, but recent versions have incorporated a variation of the SCS method (described below) as an alternative. This variant attempts to represent changing antecedent conditions by increasing the available soil storage capacity due to evaporation and percolation, and reducing it due to infiltration. The model is oriented towards providing long-term statistical information on event totals for pollutant masses and water volume. (Hydrologic Engineering Center, 1976).

While the approach created by Roesner et al provides a useful, economical, and versatile model for examination of pollutant loadings from individual catchments, it is not able to simulate receiving water, nor storage removal processes. The method is therefore not capable of simulating systems or networks. This was consistent with and appropriate for both the specific analysis and the general time period for which the model was originally developed; the model predates present emphasis on receiving water conditions. Since the model code is not structured in a way which facilitates fundamental changes in any of its basic algorithms, this limitation is essentially unavoidable. As discussed below, the model must as a result be used in conjunction with other models when transport of pollutants is of interest. The scope of the model does not meet the needs of many analyses presently encountered.

Table 1.1 shows Roesner's approach (as well as the other three models evaluated here) compared to the criteria developed in section 1.2. As indicated, the main failings of Roesner's model, in the context of the present work, are (i) limited physical scope, (ii) difficulty to adapt to alternative simulation approaches, (iii) inability to incorporate results of other models, and (iv) lack of suitable runoff rate algorithms.

Another package, representing the end result of a major ongoing model development effort in the United States, is SWMM (the Stormwater Management Model). It is noted that SWMM and HSPF described below are the major simulation models resulting from a decade of effort involving an expenditure of over 40 million dollars by the EPA (Swank, 1980) and represent the major single effort in stormwater quality simulation in North America to date.

The SWMM model represented the state of the art at the time it was developed (Huber et al, 1982). The model has been supported by the EPA since its original release in 1971, and has been under ongoing development since then. SWMM is presently in its third version (Huber et al, 1982). The model uses a method similar to that of Roesner et al for pollutant generation,

TABLE 1.1 A COMPARISON OF SOME AVAILABLE MODELS

MODEL	STORM	SWMM	HSPF	DORSCH
AUTHOR	Roesner et al	Huber et al	Johanson et al	Geiger et al
CAN CALCULATE INSTREAM IMPACT	No	Yes	Yes	No
CAPABLE OF CONTINUOUS SIMULATION	Yes	Yes	Yes	Yes
DATA REQUIREMENTS	Appropriate	May be high	Excessive	Moderate
ABLE TO INCORPORATE EXTERNAL SERIES	No	No	Yes	No
CONSISTENT WITH QUANTITY MANAGEMENT MODELS	No	Yes	Yes	Yes
SIMULATES RURAL AREAS	Yes	No	Yes	No
SIMULATES URBAN AREAS	Yes	Yes	Yes	Yes
SIMULATES CONTROL POND REMOVAL PROCESSES	Yes	Yes	Yes	No
SIMULATES INSTREAM TRANSPORT	No	Yes	Yes	No
CAN BE READILY MODIFIED	No	No	No	No
OVERALL SCOPE	Limited	Somewhat Limited	Appropriate	Limited
OVERALL LEVEL OF SOPHISTICATION	Appropriate	Excessive	Excessive	Excessive
RELATIVE COST OF USE	Low	High	Excessive	High

although the algorithms incorporated for that purpose are slightly more sophisticated, since they allow for non-linear pollutant growth rates. They can be considered to represent an improvement over earlier versions, since as indicated by Sartor and Boyd (1972) and by Whipple(1977) linear rate of build-up is not consistent with physical observations. The model has good scope for simulating receiving water transport of pollutants. Users may simulate the transport system as a series of completely mixed conduits or may utilise a dynamic one-dimensional link-node receiving water model. Reservoirs may be simulated as either plug flow or completely mixed.

SWMM provides comprehensive descriptive capabilities, but is unwieldy for regional planning purposes. It must be used in three major blocks (representing runoff, receiving water response, or storage and treatment) in regional studies. Each block is large, which makes installation difficult on some computer systems. It also tends to make application expensive, and repeated application awkward and expensive. The code structure makes it impossible, for practical purposes, to modify the fundamental model algorithms. The model is therefore best applied in local or detailed studies. As noted in the case studies presented below, the sophistication and cost of application has led some users to apply only part of the model, and abandon other sections in favor of more appropriate, less costly models.

As shown in Table 1.1, the model is unsuited for use in regional planning studies because of (i) cost, (ii) difficulty to modify fundamental algorithms, and (iii) its orientation towards primarily urban applications.

Another major package is one prepared by Johanson et al (1981) on behalf of the EPA, distributed under the acronym HSPF (Hydrologic Simulation Program Fortran). Their package represents the combined capabilities of several precursors, notably the Stanford Watershed model, and includes algorithms from ARM (the Agricultural Runoff Model), and NPS (the Non-Point Source model). The model is at the state of the art level, yet employs approaches similar to STORM and SWMM for pollutant generation. In addition, a variety of extensions to the basic build-up/wash-off method are permitted. Detailed simulation of pesticides is possible, for example, using techniques from ARM. Further, the pollutants may, in pervious areas, be linked to eroded soil. The transport of pollutants is simulated by storage routing through a well mixed river or reservoir element.

This package is designed for application on more modest computer systems than SWMM, but is still expensive to run, and is associated with a massive data management and initial programming requirement. While the model is suitable for application in major management or operational contexts, it is considered here to be expensive compared to the limited resources usually available in a planning study. Further, the model requires data which is unlikely to be available at a planning stage (i.e., up to 25 hydrologic parameters alone.)

In terms of the criteria shown in table 1.1, the model is unsuitable for regional planning analysis primarily due to (i) cost, (ii) sophistication and data requirements, and (iii) scope of some of the algorithms.

A fourth package which has been developed is a proprietary continuous simulation model of the Dorsch Consult Ltd. (Geiger et al, 1980). The model is capable of simulating an urban system in great detail, and employs a complex hydraulic analysis routine for analysis of hydraulic network problems. The model computes quality using a build-up/wash-off concept, and a pollutograph approach is used in routing pollutant loads from the land surface.

The model is not, however, suitable for regional planning studies. This is partly due to the requirement for a large data base for proper calibration of the quality routines, but is primarily due to the lack of a quality routing/transport system. Thus, although the model is suitable for a limited analysis of an urban system, it is not able to simulate a regional system since there is not an adequate treatment of receiving waters.

As shown in table 1.1, the main reasons why the model is unsuitable for regional planning analysis are (i) its limited physical scope, and (ii) inability to modify its fundamental algorithms.

Summary

The above review of available models indicates some of the shortcomings and strengths of existing simulation approaches. In general, it can be concluded that existing models are generally unsuitable for use in a regional stormwater management planning analysis; the specific reasons for this vary from model to model, but the main difficulty lies in their scope and orientation. The fundamental source of this problem would appear to arise from the fact that every one of the models discussed above was initially developed prior to the present general emphasis on regional planning of stormwater quality management based on instream conditions. An examination of how these models perform in practice, presented below, lends support to this conclusion.

1.3.2 Experience with Application of Existing Simulation Models

The literature contains a large number of documented applications of descriptive simulation techniques for water resources problems. To highlight the strengths and weaknesses of currently available simulation models, a number of applications have been selected as representative and are discussed below.

A major development proposed in the vicinity of Manotick, Ontario, recently prompted a stormwater management study to "determine quantity and quality aspects of runoff from the South Urban Community", partly in view of an MOE policy that development on the Rideau (river) must be based on the principle of "no deterioration" (Gore and Storrie, 1979). This study was completed using both the model packages of Roesner et al and Huber et al (described above) for quality analysis. A detailed analysis of receiving water impacts was not carried out despite the fact that the motivation for the study was protection of the receiving water. This was accomplished since MOE, in "the absence of any regional watershed analysis"

(Gore and Storrie, 1979) established effluent criteria for the development. This study represents the traditional approach to water quality management, which is to assess on the basis of effluent conditions.

One conclusion of this study was that a regional study would be "most desirable to assist in the formulation of a more realistic set of guidelines and criteria..." (Gore and Storrie, 1979), and such a study was recommended. This study demonstrates the requirement for a descriptive planning model capable of simulating instream behavior. It also highlights the fact that existing available packages do not entirely suit even a limited planning study individually. The model by Roesner et al lacked a basic storage treatment algorithm, and that by Huber et al was too costly for the required continuous simulations. To achieve the necessary quality and quantity analysis for the planning study therefore required use of more than one model.

The limitations of the type of study described above were recognized. Partly in response, the Province of Ontario in conjunction with the federal government and local municipal governments followed the above analysis with a more comprehensive study of a region surrounding the original study area. (Rideau River Stormwater Management Study 1981; ibid 1981; Rowney et al, 1982).

This study was undertaken pursuant to a number of policies and objectives intended to resolve questions regarding causes and mitigation or prevention of water quality degradation of the Rideau River. A central issue of the study was to assess the way in which stormwater quality management measures would meet instream water quality criteria.

A large component of the study was therefore an assessment of control measures at a planning level for a large urbanizing rural/ undeveloped region. After a screening of alternative models, it was found necessary for reasons of scope and economy to apply the model by Roesner et al in combination with an advective/dispersive one-dimensional river model to simulate both pollutant runoff and transport. A third model had to be created to provide a convenient means of simulating pollutant removal by detention ponds. The net effect of these three models was to provide information and analytical techniques at a level suitable for a regional planning study. However, the models had to be operated independently and interfaced in sequence to achieve the necessary results. This was a necessary but inappropriate approach to simulation analysis. The combination of existing models is adequate but does not represent an effective tool for planning analysis.

This study, completed in 1982, further demonstrates the present general lack of suitable comprehensive models for river basin water quality planning analysis, and the need for such a model to include algorithms for source, control and transport of pollutants in some suitable single framework.

Also faced with the need to assess a physical system beyond the scope of available models, Walesh (1976) described an approach to establish a comprehensive water resource plan using a package of hydrologic-hydraulic-water quality models. He pointed out the lack of any single suitable model available at the time, and described a package of models applicable to such a problem. It was necessary to combine two sub-models (an hydraulic submodel and a water quality submodel) from a precursor to the model by Johansen et al (described above) with a U.S. Army Corps of Engineers backwater program and to add a new submodel for economic analysis to the package.

Amades et al (1980) describe a study conducted under the 208 Planning Area (a legislative designation in the U.S.). In this study, the impact of detention basins on basin pollutant export was studied using a combination of models. STORM provided runoff quantity estimates while a submodel (HLOAD) developed for the study area generated pollutant estimates. HLOAD is essentially a rating curve method which calculates a pollutograph from given hydrograph and calculated load data. To determine the impact of detention ponds, a new pond simulation sub-model (STOREME) was written. No analysis of instream transport was conducted. The need to extend the capabilities of STORM for use in this study demonstrates the incomplete nature of the STORM model algorithms, compared to current needs of analysis.

Jettmar et al (1980) used a combination of STORM and a modified version of SWMM -RECEIVE to assess dynamic water quality conditions. Cost was cited as the reason for not using SWMM for the complete study. To reduce costs, STORM was used to simulate stormwater runoff quantity and quality, and SWMM was used to simulate pollutant transport. The study demonstrates the limited scope of STORM, and the excessive cost and complexity of SWMM. In a planning level analysis the complete capabilities of SWMM were not used despite the extra effort involved in combining two models where one would in principal have sufficed; STORM is appropriate for this level of analysis, but incomplete.

Medina et al (1981) have examined instream impacts of pollutant inputs to a river by combining runoff time series calculated by STORM with a simplified analytical solution for instream transport. The analysis used a lumped approach, and assumed steady-state one-dimensional uniform conditions instream. This approach would be useful in situations where a specific input point is of interest and where the time scales and morphology of the system make the above assumptions appropriate. However, in a regional plan where multiple inputs and variable hydraulic conditions exist, the method would be of limited direct use. The study demonstrates again the need for a model with a scope of simulation appropriate to a planning analysis.

Summary

From the above case studies, the major difficulties with existing models lie in the scope and orientation of the model structure. This is compounded by a tendency for cost to increase with scope. The most suitable characteristics of existing models appear to be in the pollutant source formulations currently available. The least suitable characteristics appear to be in water quantity routines and in river transport routines; both tend to be costly, and runoff quantity routines are in addition either over-sophisticated or not consistent with water quantity analyses.

1.4 Conclusions

1. Although existing models make a regional analysis of

stormwater quality management planning alternatives possible, the state of the art of water pollution simulation is still at a level where improved techniques in simulation are necessary.

2. Specific weaknesses of existing simulation approaches have been identified. In general, existing simulation packages presently available tend to represent only a part of the physical system, are comprehensive and relatively massive management models, or are not oriented towards regional stormwater quality management planning analysis.

3. To some extent, this situation reflects changing policies and practice in water resources planning analysis. Approaches which were satisfactory in the past become insufficient due to changes in objectives and required information.

4. Characteristics of an appropriate model for regional stormwater quality management analysis have been defined.

5. The major improvement required in simulation models for a regional stormwater management planning analysis is the development of an approach which is consistent with the overall needs of such an analysis. These needs have been identified.

6. In principle, all algorithms for simulation of

physical processes could be improved. However, it is suggested without further proof that the extensive and successful use of the build-up and washoff algorithms (of the type used by Roesner et al, Huber et al, and Johansen et al) currently prevalent in the literature indicates that although it could be improved, this aspect of model development has progressed to a point which is suitable for practical application to water quality planning management.

7. Two algorithms for physical processes which are most in need of improvement for use in regional water quality management planning models are the calculation of direct runoff, and the routing of pollutants instream.

2.0 Research Objectives

The foregoing general review has highlighted the major weaknesses of existing models for water quality management planning, and has concluded that existing approaches are inadequate for a regional planning level analysis of stormwater pollution. A number of areas in the current practice of water quality simulation and analysis were identified as having a potential for improvement.

Of the possible areas for research and improvement, three in particular were selected for emphasis in this research. These were the general structure of the planning model, the means by which runoff volumes are determined, and the means by which pollutants are routed instream. These were chosen as priorities for development because they appear to be the least satisfactory of the presently applied approaches.

The following specific objectives were established for the present research:

> Objective 1: To create an improved continuous quality/quantity simulation model for planning level analyses of pollution by stormwater runoff in regions developing from rural to urban land use.

Objective 1 is the primary research objective and responds to the general need for an improved capability for a simulation model, identified in Chapter 1. It is implicit in objective 1 that the model should, to the extent possible, meet the criteria which have been established in Chapter 1.

> Objective 2: To devise and test an algorithm for routing pollutants instream, for use in the above model.

Objective 2 was defined in response to one of the specific weaknesses perceived to exist in most present models, namely the cost associated with traditional approaches to river pollutant routing. It was considered that an alternative approach might be developed which would be less costly than, but superior to or at least consistent with, existing approaches.

> Objective 3: To devise and test an algorithm for calculating direct runoff, for use in the above model.

Objective 3 was defined as a result of two main factors. First, a need was identified in Chapter 1 for the model to include a runoff algorithm consistent with those currently accepted for use in stormwater quantity analysis. Second, most existing approaches which meet this requirement, such as the Horton/ kinematic wave formulation adopted by Huber et al, have been found to be expensive or inappropriate for rural land use applications. It was therefore decided to investigate the possibility of creating a runoff algorithm which is consistent with existing water quantity simulation approaches, but which is suitable for continuous simulation in a planning model. This inherently required an innovative approach to runoff simulation.

> Objective 4: To use the above model to examine the present MOE policy of specifying water quality objectives as single valued concentration limits.

Objective 4 was defined for two reasons. First, it provides a test case for model application. Second, it was noted during the review of current legislation and criteria that (i) the predominant approach to water quality criteria is to specify single valued concentration standards, and that (ii) such a practice is not preferred for a number of practical reasons. This test case would provide further insight into the validity of a single-valued approach to legislation, and contribute to substantiating or negating this approach.

Scope:

To provide reasonable limits on the scope of research, the following general limitations were imposed.

Pollutant generation

Pollutants born by direct stormwater runoff were defined as the only quality constituents to be considered in the present version of the model. This reflects the orientation of the present research. However, to provide for the potential need to simulate other sources, it was proposed that the model be structured in a way which makes fundamental changes in assumptions regarding pollutant sources possible.

It was proposed that existing techniques be used to provide the model with algorithms for pollutant generation. As indicated in the literature review, the existing build-up or rating curve methods which are used in most models have been successfully applied in a number of instances. Despite their admitted limitations, these methods are already computationally economical and regarded as effective for use in planning level models. It is recognised that this choice provides a potential limitation to model use in that erosion equations such as the Universal Soil Loss Equation (mentioned above) are not included in this version of the model.

Pollutant Constituents and Removal Mechanisms

It was decided to incorporate two groups of pollutants for simulation by the model, namely sediments and indicator bacteria. This accords with the nature of the major pollutants in urban runoff (Whipple et al, 1981). In accordance with current practice, removal mechanisms for these pollutants were assumed to be represented by discrete settling and by first order decay, respectively(Zison et al, 1978). This approach was regarded as providing a fairly comprehensive capability since in common usage such pollutants as fecal coliform bacteria, total coliforms, heavy metals associated with sediments, settleable solids, and nutrients have been simulated in this way. As with the pollutant source assumption, to provide for the event that substances with other removal mechanisms might potentially be of interest, it was proposed to make replacement of the fundamental transport and control pond algorithms in the model possible.

Control device

A single control device, the stormwater detention pond, was proposed for simulation in the proposed model. Control ponds in particular were selected for inclusion in view of the current interest in North America in control ponds as water quality control devices. Further, an adequate theoretical basis exists to allow the derivation of algorithms for completely mixed, plug flow, or intermediate pond mixing conditions. Again, it was decided to structure the model to allow incorporation of other types of control devices in the future should this become necessary.

Summary

In essence, the limitations in scope set above provide the model with the capability to simulate practical physical problems but keep the initial model development within reasonable bounds. A meaningful resolution of the stated research objectives is possible within these limitations in scope.

3.0 Literature Review

Chapter 1 has already reviewed various approaches to simulation models, and has developed the needs of an overall planning model. This chapter reviews specific algorithms which may be appropriate in a model developed in the context of the scope and objectives of this research.

3.1 Analytical Methods for Pollutant Source Simulation

As reviewed by Walesh (1980) and others (Delleur et al, 1980), there are several basic approaches to assessing pollutant loadings or impacts. These can be grouped in a number of ways according to the precise application of each method, but can be placed for convenience into six categories: rating curve methods, unit load methods, concentration/flow methods, pollutograph methods, and other empirical methods.

The rating curve method (Bedient et al, 1980), can be used to calculate annual yields of sediment or pollutant. The method relies on an integration of a mass transport rate / frequency curve (which is produced by combining flow / frequency and mass transport rate / flow curves):

$$\text{Mass} = \int Mr(q).p(q)\ dq \qquad (3.1)$$

where Mr(q) is the mass rate of pollutant outflow associated

with flow rate 'q',
and p(q) is the frequency of
occurrence of 'q'

The method provides useful estimates of net load at an instream point provided data is available, but is not readily applied in predictive or source identification studies unless an adequate empirical basis exists for defining the fundamental curves.

The unit load method (Haith et al, 1981), uses 'known' pollutant contribution rates for various land uses (units - mass/area/time) to calculate total contributions from areas of varying composition:

$$My(j) = sum(mi*ai,n) \qquad (3.2)$$

where My(j) is the seasonal total mass outflow from area 'j',
mi is the seasonal contribution per unit area of land use type 'i',
ai is the area of land use type 'i' in area 'j',
and sum(b(i),n) refers to the summation of the function 'b' evaluated over all 'n'.

The method provides an indication of source magnitudes, but not of event distribution or instream impact unless the method is used in conjunction with other models. Identification of appropriate contribution rates may be difficult.

Concentration-flow methods (Walesh, 1980), use the product of a representative concentration and flow rate to calculate a catchment load for any event or series of events. Determination of stream or control device response using such a method by itself is not possible:

$$Mr(j,t) = q(t)*Cm(j) \qquad (3.3)$$

where Mr is the mass rate of pollutant 'j' at time 't',
q(t) is the flow rate at time 't',
and Cm is the mass concentration of pollutant 'j'

Pollutograph methods (Young et al, 1979), assume a typical temporal distribution for pollutant runoff. Pollutant response in a watershed can be determined by convolution in a manner analogous to the unit hydrograph method commonly used in hydrology:

$$Mr(t) = \int R(t-tau)*X(tau).dtau \qquad (3.4)$$

where R is the response curve,
X is the forcing function (precipitation),

and Mr is the mass rate of
pollutant outflow.

Required parameters for the method (such as peak concentration and time of the pollutograph) are either assumed or determined as simple functions of the design event.

Regression methods (see for example Jewell et al, 1982), have been used in an attempt to correlate pollutant characteristics such as total load or peak intensity to various functions of basin and rainfall parameters:

$$Mr(t) = f(\text{area, land use, season} \ldots) \qquad (3.5)$$

Some of the above methods could be considered as examples of this approach. Regression methods in general hold promise for application in planning studies, for example when sufficient data exists to produce regional parameters, but are presently limited by a lack of data. (Walesh, 1980)

Other methods exist, such as the EPA screening method (an empirically based variation of the the unit load approach) or the Universal Soil Loss Equation (see for example Shahane, 1982). These methods generally fall under one of the above categories, and may or may not be appropriate in any given study.

The above methods have been used in 'desk-top' approaches by themselves, because of requirement for only limited computational power (Walesh, 1980), and forms of some have been used in simulation models. The common characteristic of 'desk-top' methods is that they are by themselves poorly suited to comprehensive planning analysis at a regional level, since control devices are not simulated and since transport in the receiving water is not accounted for. The methods may, however, be useful in demonstrating the general magnitude of impacts associated with urbanisation.

The limited capabilities and scope of the above basic analytical methods have been evident, and have led to a variety of extensions of the methods to more complex and comprehensive models. By virtue of the numerical effort and complexity involved, these models are typically presented in the form of numerical computer models.

An element common to most quality simulation models, and to the above models in particular, is the implicit assumption that pollutants may be considered to build up at a certain rate during dry periods, and to be transported from the catchment toward the outlet during wet periods. Approaches used to calculate build-up either assume that pollutant build-up is a function of area and time for each land use, or to assume that pollutant build-up is a function of some characteristic catchment parameter (such as gutter length or number of catch basins) and time. Specific alternative relations which have been proposed can be grouped as follows:

a.) a power linear build-up in which

$$Pt = Pt' + k1*t**k2 \;;\; Pt < \text{or} = Plim \tag{3.6}$$

where Pt is pollutant available for washoff,
Pt' is Pt at beginning of time step,
Plim is a maximum value of Pt,
and k1 and k2 are calibration parameters.

b.) an exponential formulation

$$Pt = Plim(1-exp(k1*t')) \tag{3.7}$$

c.) a Michaelis-Menton formulation

$$Pt = Plim*t'/(k1+t') \tag{3.8}$$

d.) a growth-limited relation of the type:

$$Pt = Pt' + k1*(1-Pt')/Plim \tag{3.9}$$

e.) a growth-limited relation of the type:

$$Pt = Plim*(1 - (k1*z + 1))/(k2*z**2 + k3*Z**3 +1) \tag{3.10}$$

where z is the number of time steps since the last event,
'**' denotes exponentiation,

and k3 is a calibration parameter.

Equations 3.6 through 3.10 all describe build-up as a monotonic increasing process, and assume an upper limit for the build-up of pollutants. Roesner et al used equation 3.6 with k2 set to one. This has been a useful approximation, but non-linear relations such as the equations 3.7, 3.8, 3.9 or 3.10 may be preferable. Huber et al used equations 3.7 through 3.9, Johansen et al used equation 3.9, and the Dorsch model uses equation 3.10.

The Dorsch model uses a pollutograph approach of the type included here as equation 3.4 to convert available pollutant load to a pollutant washoff rate during an event. The models by Roesner et al, Huber et al, and Johansen et al, however, use an alternative approach which is essentially common to all three. These models compute washoff rate during an event using a relation of the type:

Wt = Pt*(1-exp(-k1*R)) (3.11)

where Wt is the washoff rate,
and R is the runoff rate.

As well as the build-up/washoff approach to pollutant source simulation, a rating curve approach in the form:

Wt = k1*Q**k2 (3.12)

has been used (Huber et al, 1982) to directly calculate pollutant rate from runoff rate.

It can be concluded from the above examination of specific models and from reviews of the problem of pollutant simulation (Huber et al,1982) that despite major funding and development over the last decade, the problem of pollutant build-up is still essentially at a level of lumped empirical relations. Parameter estimation for these relations is essentially based on reported literature values, or on calibration to observations if data exists. There does not appear to be any clear basis for preference among the above relations.

It is noted however, that relations of the types described above are at a level of sophistication appropriate for planning level analysis, and that models of the type 3.5 through 3.8 have been successfully applied in water quality analysis. They have been applied in Ontario before (for example in the case studies listed in chapter 1), and hence data exists which can be used in the absence of calibration data. Simulation of Fecal Coliform bacteria, for example, has been done in Ontario using a constant concentration (Rowney et al, 1982). It is therefore further concluded that the basic algorithms for pollutant generation and washoff are, in their present form, appropriate for use in quality planning management models.

3.2 Pollutant Routing

This part of the review deals with methods of solving the fundamental equations governing the instream transport of pollutants, and with the literature as it pertains to the runoff quantity algorithm adopted in this model.

The General Transport Equation

Analysis of receiving water impacts of pollutants generally involves characterization of pollutant concentration both in space and in time. Such an analysis may be based purely on observed or measured values or, more typically in a planning context, may involve the use of simulation techniques to provide insight into factors not explicitly measured.

In the latter case, it is necessary to formulate a cause and effect description of the behavior of pollutants in the receiving body. This can be achieved by means of the general conservation equation (Rinaldi et al, 1981):

$$\partial p/\partial t + \mathrm{div}\,(va*p) + \mathrm{div}\,(vm*p) - D' = I \qquad (3.13)$$

where $\partial p/\partial t$ is the time variation of the intensive quantity 'p', div (va*p) is the advective flux of 'p',

div (vm*p) is the mobility flux of 'p',

I represents interactions such as sources or losses, and

D' is the dispersive flux, composed of turbulent dispersion and molecular diffusion.

Typically, the dispersive flux is represented as a net transport rate proportional to the concentration gradient (Fickian dispersion):

$$D' = \mathrm{div}\ D\ \mathrm{grad}\ p \qquad (3.14)$$

where D is a coefficient matrix

The quantity 'p' usually does not, or is considered to not, display significant self mobility. Dissolved materials behave in accordance with this assumption, while floating or settleable materials do not. For the purposes of this work, dissolved materials are of interest. In such a situation, the conservation equation becomes:

$$\partial p/\partial t + \mathrm{div}\ (va*p) - \mathrm{div}\ D\ \mathrm{grad}\ p = I \qquad (3.15)$$

or, in one dimension,

$$\partial p*A/\partial t + \partial(A*va*p)/\partial l - \partial\ (A*D\ \partial p/\partial l)/\partial l = A*S \quad (3.16)$$

where A is the sectional area,
l is the length term,
and S is a net source/sink rate.

Equations 3.13 and 3.16 both state that the time rate of change of concentration at any point is equal to the sum of the contributions of advective and dispersive fluxes, sources, and withdrawals of the substance. For the first order decay envisaged here, the term AS includes a source and a reaction rate:

$$S=-Kp+S' \tag{3.17}$$

Equation 3.13 results by definition directly from an examination of mass balance within a control volume. The Fickian term in equation 3.16, however, is essentially empirical (Kay, 1957). Use of a Fickian term to describe molecular and turbulent dispersion was examined in a classic paper by Taylor (1921), whose work showed the analogy to be appropriate after some initial period of time. Since then, a number of authors have examined the problem of determining the coefficient D. Notably, Taylor (1953), Aris (1956), and Elder (1959) derived relationships for the coefficient D. More recent research has to some extent superseded these early results. Fischer (1966) showed transverse velocity distributions to be significant is determining D in an open channel. Others have discussed the disparity between predicted and observed results and have proposed various remedial schemes. Entrapment in dead zones was discussed by Thackston et al (1970) and by Sabol (1978). McQuivey (1976) was successful in using a purely convective model during the initial period of mixing. Aris's (1956) moment transformation analysis has been extended by Tsoi (1978) and Stefan (1981).

Despite this activity, uncertainty remains in estimation of dispersion in natural rivers (Tsoi, 1978). The significant factor for purposes of this research, however, is that use of a Fickian analogy to describe the combined effects of turbulent and molecular dispersion is generally accepted in the literature. In fact, except for methods combining stochastic and deterministic components (such as the 'random walk' or 'Monte Carlo' techniques described below) it is the only technique used. Equation 3.16 may therefore be considered to be the fundamental relation for instream transport in a one-dimensional system; the terms on the left hand side are fixed, and the terms on the right hand side will depend on assumptions regarding the nature of the substance 'p'.

As discussed below, analytical solutions to equation 3.16 have been derived for particular cases, and numerical solutions have been proposed for more general problems or for analytically difficult problems.

Analytical Solutions of the Transport Equation

Numerous analytic solutions have been proposed for special cases of equation 3.16. These various solutions arrive in general from assumptions regarding the nature of the reactive term (S), the spatial or temporal variability of the coefficients (U, A, or K), or the relative magnitudes of the various terms.

Assuming constant coefficients (uniform flow and channel section) equation 3.16 becomes:

$$\partial p/\partial t + va*\partial p/\partial l - D\,\partial(\partial p/\partial l)/\partial l = S \qquad (3.18a)$$

Further assuming steady state conditions and a point source, for a constituent undergoing first order decay, equation 3.18a can be re-written as:

$$D*\partial(\partial p/\partial l)/\partial l - va*\partial p/\partial l - kp = 0 \qquad (3.18b)$$

For a constant input concentration, the solution to equation 3.18b has been given (Hydroscience, 1971) as:

$$C(x)=Co*\exp((v - \mathrm{sqrt}(v**2+4*K*D))/2*D)x) \qquad (3.19)$$

with Co the initial concentration
C(x) the concentration at point x
v the velocity

K the reaction rate

and D the dispersion coefficient

In conditions where dispersion predominates, equation 3.18b becomes:

$$D*\partial(\partial p/\partial l)/\partial l - Kp = 0 \qquad (3.20)$$

$$\text{and } C(x) = Co*\exp(- \mathrm{sqrt}(K/D)*x) \qquad (3.21)$$

Where advective flux predominates:

$$v*\partial p/\partial l - K*p = 0 \qquad (3.22)$$

$$\text{and } C(x) = Co*\exp(K*x/v) \qquad (3.23)$$

These simplistic cases are not appropriate in stormwater management problems where temporal variations are significant. Further, the single first order decay used in equation 3.19 may be inadequate to describe the quantity 'p' of interest. Dissolved oxygen, for example, has several possible source or sink terms associated with it.

Although not directly pertinent to the present work, it is noted that the classic Streeter-Phelps (1925) equations represent the earliest (Rinaldi et al,1979) solutions of the time-varying biochemical analytic solutions. Since then, a number of solutions for various special cases of equation 3.18b have appeared. Some have modified the equations above to account for biological factors not incorporated by Streeter and Phelps. Others have attempted to incorporate dispersion (Rinaldi et al, 1979). In general, analytic solutions exist for time-varying constituents with time-varying flows, and for special cases of uniform non-steady flow. A recent review by Medina et al (1981) contains a number of such solutions, while the classic compilation by Carslaw and Jaeger (1959) provides analytic solutions to many special cases. In fact, given uniform flow, area, and dispersion coefficients, the solution of any arbitrary input function may often be derived from the integration of an impulse response function, as described by Fischer (1979) and others (Medina et al, 1981;Thomann, 1972).

The application of the analytic solutions either as-is or by superposition provides, in theory, solutions for some practical situations. Use of superposition essentially uses the analytic solution as a response function, with the output series obtained as a convolution of the input function. This has been done by Fischer, and more recently by Medina et al(1981). The difficulty with these methods is that they are inherently subject to the same restrictions as the analytic solutions from which they are derived. The method described by Medina, for example, used the uniform steady flow solution for a short duration constant input as a response function. The model was applied in a river using one hour time steps according to the input and flow conditions during each step. The results of this application appear to be good. It is noted, however, that with variable velocities across the stream section and particularly in short reaches where response can be non-Gaussian, the use of this model would be questionable.

The application of analytic solutions, particularly where multiple sources and non-uniform conditions exist, can be difficult (Meyers, 1971). For this reason, numerical approximations of equation 3.18b are often applied as alternatives to the analytic solutions.

3.3 Numerical Solutions of the Transport Equation.

Although a limited number of authors have attempted to solve the advection/dispersion problem in other ways, (Jensen et al,1980; Ahlstrom et al, 1976), the classical approach to numerical solution of this problem has been to adopt an Eulerian perspective and solve equation 3.18b by finite difference or finite element methods (Ahlstrom et al, 1976). As noted by Koutatis et al (1980), the recent popularity of these methods is generally due to the advent of digital computers.

One approach to a numerical solution of equation 3.18b is to consider the river system to be represented by a series of simpler systems. The cells in series approach is an example of this. This method essentially considers the system to be represented by a series of well mixed cells. Under these conditions, an alternative to equation 3.18b may be applied:

$$d(V*C)/dt = S(t) - Q*C \tag{3.24}$$

where Q is the net outflow,
S(t) is the net source/sink,
C is the reactor concentration,
and V is the cell volume.

Equation 3.24 is an ordinary first order differential equation, and is simpler to solve than equation 3.18b since no spatial derivatives exist. The equation is solved simultaneously or singly for all the cells in the system. In fact, the method may be considered a finite element approach. Stefan and Banks (Stefan et al, 1981) have applied the method. They showed the method to produce dispersive effects similar to those incorporated in more complete advective/ dispersive descriptions as a consequence of the numerical propagation of constituents from one cell to the next. The magnitude of this effect, and the results of the method, are highly sensitive to the number of cells used to represent the river. Recently, the method was used to provide a simple transport model for use in conjunction with the model by Roesner et al (CDM/Resource Analysis, 1981). The model (RWQM) used a Crank-Nicholson formulation to solve the system of equations.

An alternative to this approach was used by Chen (1970) who employed a very similar equation but included a simple dispersive term, written here as:

$$dV*C/dt = S(t) - Q*C - A*D\ dC/dx \qquad (3.25)$$

where A is the sectional
area of the cell

Solution of equation 3.25 for a number of elements, particularly if a simultaneous solution is used, provides an approximation to the second order term in equation 3.18b since fluxes from both bounding elements are considered.

It should be noted that, although a 'top-down' solution (forward difference scheme) is possible for both of the above equations, such a solution will necessarily result in a propagation of input loads through the entire system with each time step. This propagation, depending on the number of cells used, may dampen quickly, but may be significant. A simultaneous solution of the cells for the whole system is one way to reduce this effect, but will require more computer space and time.

Numerical methods have been used to solve equation 3.7 both for steady state (Water Resources Engineers, 1972) or dynamic input conditions (Huber et al, 1982). This is done by either finite difference or finite element approximations of equation 3.18b. Numerous examples of this exist (Shen, 1978); finite difference schemes (Grubert, 1976; Abbott et al, 1975; Thatcher et al, 1975), finite element schemes (Varoglu et al, 1978; Narasimhan, 1978; Gambolati, 1979) and combined difference/element schemes (Niemeyer, 1978; Koutitas et al, 1980) have been applied. Textbooks specifically reviewing the subject (Meyers, 1971) of numerical analysis of equations of the type of equation 3.18b (second order linear parabolic partial differential) are available.

The details of these numerical methods will not be discussed here. There are certain general conditions inherent in their application. Chief among these is the characteristic requirement for large computer core and time. Further, since these methods all solve a continuous function in finite steps, some degree of approximation is necessary, and results in errors in the calculated result. These errors can generally be kept within desired limits by using smaller temporal and spatial discretizations, but the result is an increase in cost.

As described by Bella (1970), the numerical errors of finite difference approximations can be quantified and interpreted as a pseudo-dispersion coefficient. These errors result from propagation of a concentration change through each element length (Δx) in each time step (_t). This basically results in a propagation speed of:

$$c = \Delta x/\Delta t \tag{3.26}$$

although other rates, including the instantaneous propagation noted above, can occur. The impact of this effect can be minimised by careful choice of time or distance steps or can sometimes be incorporated into the physical dispersion term so that the total dispersion is appropriate. It can be shown (Bella et al, 1970) that the magnitude of the numerical dispersion term in a convective difference scheme is:

$$Dp = u/2*((1-2*y)*\Delta x - u\Delta*T) \tag{3.27a}$$

where Dp is a numerical dispersion coefficient analogous to D in equation 3.18b

T is the computation time step,

u is the river velocity,
and y is a weighting factor dependant on the difference scheme

For a backward difference scheme, y=0 and:

$$Dp = u/2*(\Delta x - u\Delta*T) \tag{3.27b}$$

while for a centered difference scheme y=1, and:

$$Dp = u**2\Delta*T/2 \tag{3.27c}$$

In equation 3.27b, a careful choice of Δx and ΔT can eliminate Dp, while in equation 3.27c, this cannot be done. In either case, ΔT must decrease with an increase in u to avoid an increase in Dp. This analysis is illustrative of the basic problem with numerical methods.

The finite element schemes which have been proposed offer somewhat more flexibility in describing boundary conditions than the finite difference schemes, but unfortunately, as described by Koutitas et al (1980), this is usually offset by an increase in computer storage costs and run times. A review by Varoglu et al (1978) cites papers by Lam, Ehrlig, van Genuchten, Mercer, Faust and Smit which show that "numerical methods give acceptable numerical results for diffusion dominated flow problems but when convection is a strong component of dispersion the existing numerical solutions have an intrinsically unstable character which exhibits oscillation, overshoot, undershoot, artificial diffusion, negative concentrations ...". (Varoglu et al, 1978).

Since many streams and rivers are of the type where dispersion is of minimal or secondary importance (Hydroscience, 1977), this is an unfortunate situation. It can be concluded that although these methods are useful and flexible, inherent difficulties exist in their application.

4.0 Model Development

Introduction

As discussed in Chapter 1, a number of criticisms may be made of existing models. One of the most basic of these is that there has been a tendency apparent in the literature to create models for single purpose applications and to structure them in a way which makes application to other conditions difficult. Since no two problems are identical, this represents a significant liability.

The models by Roesner et al, and by Huber et al, for example, are created in relatively large blocks of code, and data storage is essentially all in core. This has advantages in terms of speed of execution and code size, but makes it difficult to make fundamental changes in any of the basic model algorithms. It also makes complete comprehension of the model function at a user level somewhat difficult. This can tend to result in errors in model function, or at least inappropriate applications of the original model. Note, for example, the common use of the extended transport algorithm of SWMM in simulation of surcharged systems despite the fact that it does not properly handle momentum changes at pipe junctions.

Another structural problem with existing models for application in planning studies has been an incompatibility of sophistication. As indicated in the literature review (Chapters 1 and 3) some existing models which in principle have the scope to solve typical planning problems use methods which are expensive or are more sophisticated than the data available at a planning stage would warrant. This has resulted in some cases in the use of several different models in one study despite the availability of comprehensive models, to reduce cost or complexity. One of the goals which was maintained while the code structure for the present model was under development was to incorporate capabilities which are relatively comprehensive, useful, and consistent in approach and level of sophistication.

The concept in model structure which was applied in this research is different from that which is available in existing alternatives, and was designed to circumvent some of the problems in application which have been associated with existing models.

4.1 Design Assumptions

The model was created in accordance with the objectives of this research, and with the following governing assumptions and concepts:

1.) It is assumed that the quality planning phase is associated with a quantity planning analysis. As indicated in Chapter 1, it is appropriate that the water quality analysis be conducted in conjunction with a water quantity analysis. For example, regulatory agencies in Ontario (COA, 1980) already require or promote determination of certain basic hydrologic information such as pre and post developed flow rates and flood lines as part of the planning analysis. Appropriate methodologies for this have been specified at municipal and provincial levels of government.

The user of the planning model is therefore assumed to have available basic hydraulic and hydrologic information used in water quantity analysis. The quality planning model should utilise this information where appropriate for consistency and simplicity, and should contain runoff volume and rate algorithms which can be reconciled with those used in the water quantity analysis.

2.) Little calibration data will be available in typical studies. The quantity buildup and transport routines should appropriate for such a situation; for example use of existing algorithms (such as those of STORM or SWMM) where appropriate would be of benefit since the user could, in the absence of direct data, make use of reported model parameters.

3.) A statistical interpretation of the impacts of urbanization and control in the form of concentration/ duration and number/ duration exceedance curves will be required. This is in keeping with current practice (Rowney et al, 1982; Southerland, 1981) in stormwater quality planning analysis as discussed in the literature review. A continuous simulation approach was used to generate the synthetic time series information which could be used to create the required exceedance curves. An associated interpretive component was included in the model to achieve this.

4.) The planning model will primarily be used in situations where instream transport can be adequately simulated using time-varying, non-uniform, one-dimensional advection and dispersion with a linear uncoupled first order decay or discrete settling reaction term.

The non-steady, non-uniform conditions are likely to be necessary in a regional study, since long reaches and time scales are implied. Dispersion is generally of secondary importance in this context (Hydroscience, 1977), but to permit an expanded range of application, the river routing schemes which have been developed do incorporate a mechanism for including the effects of dispersion in river pollutant routing. The restrictions to the reactive terms are consistent with the scope of research, and with the pollutant constituents described in Chapter 3.

5.) In accordance with the objectives defined in Chapter 2, the model was structured to make use of alternative components as simple as possible. This was done by creating the individual functional units in the model as separate modules, inter-connected only by the quantity/ quality time series inputs/outputs which are read or generated.

6.) Where appropriate, existing models or algorithms were used. This is a logical perspective, adopted to promote the use of suitable existing experience and to minimize useless duplication as much as possible (Walesh, 1976).

7.) The model was designed to use relatively little core space and computer time. As discussed above, one of the reasons existing models have been unsatisfactory is high cost and large computer requirements. In light of the role of the model as a planning tool, and particularly in view of increasing proliferation of microcomputers of limited capacity in engineering, this is an important aspect in model design.

4.2 Model Structure

In view of the foregoing, the model structure shown in figure 4.1 was adopted. Essentially, the model consists of a number of functionally and structurally independent modules linked by disk files which store simulation and input data.

The command module controls the course of the computer run by invoking the other modules in the sequence and manner chosen by the user. The control module is the only part of the model the user interacts with directly. Functions such as file management, data reading and writing, plotting, and simulation are all invoked by the control module after the user input instructions have been interpreted.

There are three main modules representing the simulation functions of the model: the runoff module, the control pond module, and the river transport module. The runoff module can read rainfall data, and create runoff data files on disk or tape. This output includes a time history of runoff quantity, quality,

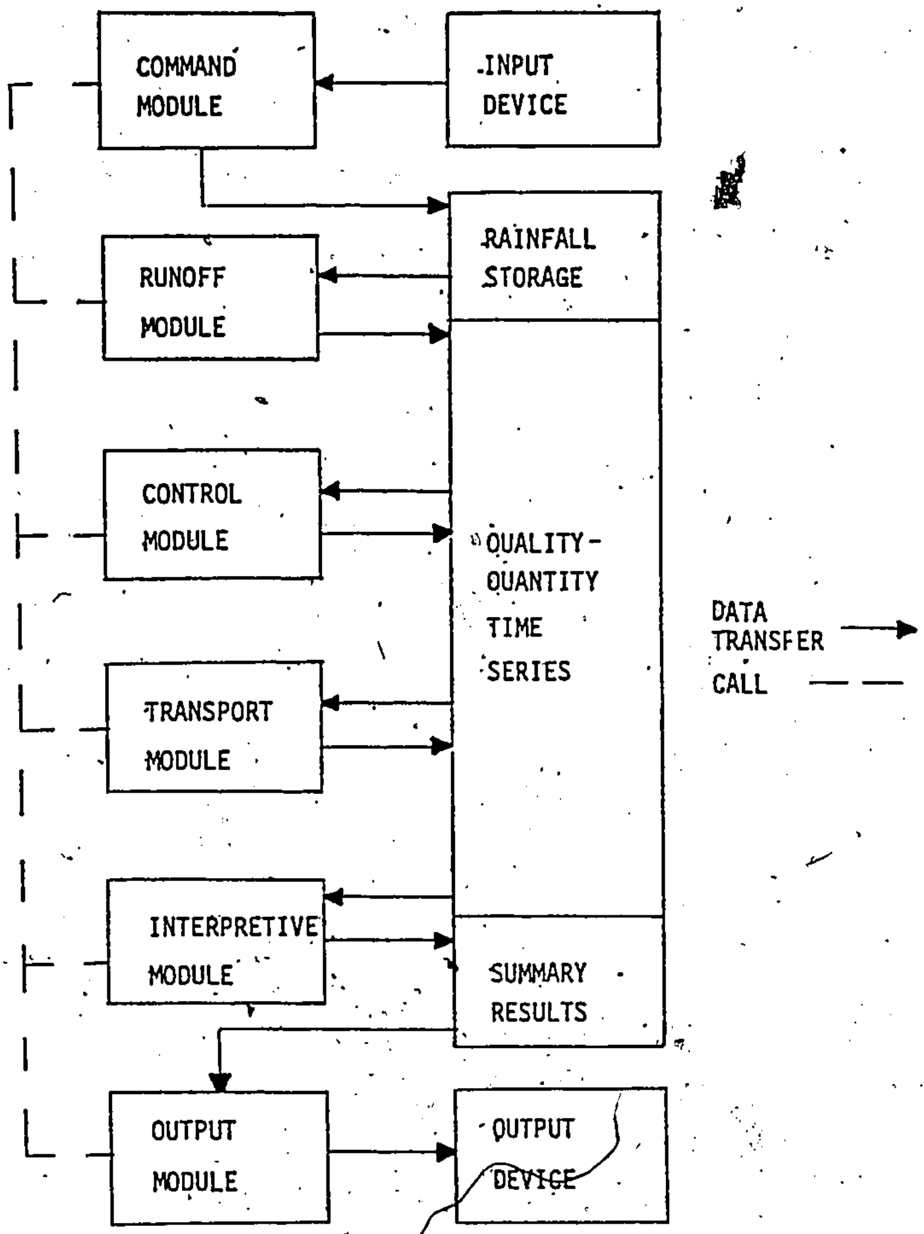


fig. 4·1 Conceptual Formulation of Program Components and timing information from the catchment. The control pond simulation module routes the stored quality/quantity series and records the result on a designated disk file. Similarly, the transport module routes the quality constituents from point to point instream.

The interpretive modules generate statistical information about the synthetic time series which represent catchment outflows, pond outflows, or instream conditions. At this time, two types of basic statistics are calculated. The model calculates information comparing two files, and calculates summary statistics (over user-specified intervals) of individual files.

The output module can be called at any time during or on completion of the run to print or plot detailed or summary results of the simulation.

The system described above differs from existing continuous simulation quality models such as those by Roesner et al, Huber et al, Johansen et al, or Geiger et al in several basic ways. First, the fundamental components necessary to simulate a large dendritic river basin are present in a simpler or more comprehensive way than has previously been the case. Second, the structure created allows the user to simulate the system in a top-down manner rather than simultaneously, which reduces the computer core space requirement of the model. Finally, the highly modular structure of the model allows the user to readily replace or supplement the routines provided with alternatives which may be more appropriate in their particular application.

A further advantage to the modular scheme is that the calculated time series reside permanently on disc. This allows the user to refer repeatedly to disc files without re-calculation, which can have cost benefits in model use. Typically, for example, a study of an urbanising basin encounters some sub-catchments which are expected to change in the future, and some which are expected to remain static. By retaining the simulated series for the static areas on disc, the user need only simulate them once; the information can subsequently be used in an overall analysis of both the existing and future land uses. Another example is in the analysis of control devices. Catchment runoff need not be simulated for every run where a different control pond is simulated. Rather, the catchment outflow can be simulated once, and the stored information is then available for routing through any number of different storage devices. In fact, in principle a single computer run can use the same time series for routing through several different control ponds for comparative purposes.

Appendix D contains a detailed listing of the data input requirements for the model. Table 4.1 is a listing of the available model commands. As shown, there are presently 15 commands which the user can invoke. These commands can conveniently be grouped into three categories: simulation commands, control commands, and utility commands.

Simulation commands

These are the actual functional commands of the model, and represent the entire repertory of simulation capability. The commands allow the user to generate a synthetic time series of stormwater runoff from any basin, and to route that or any combination of similar series through a river reach. A detention control device can be simulated, and if necessary a flow split can be coded. Although most applications of the model are likely to be in dendritic (branching) systems, the flow split is incorporated to allow the user to simulate special system configurations. For example, a number of control ponds in parallel can be coded using the flow split routine, as can losses to irrigation channels or other lateral branchings.

Control commands

These commands allow the user to control the simulation run. The commands are required to initialise the run and to terminate it. By using these commands, the user designates rainfall input devices, selects which (if any) pollutants are to be simulated, and defines default parameters for pollutant accumulation, washoff, or rating curve calculations.

Utility commands

The various output and interpretive modules are invoked by these commands. The disk files referred to by the model are in a binary format to enhance speed and reduce disk space. To view any of the files, or to plot them, therefore requires interpretive modules to convert the disk file data to a legible format. The user can plot or print segments of the file as required, or can dump the entire disk file as a printed or plotted series. Also, the user can calculate calibration or frequency curve statistics using these commands, and can input observed flow series for use by the model if gauging data is available.

4.3 Summary

In terms of the criteria for planning models defined in Chapter 1, the proposed model :

- o is capable of simulating instream impacts of stormwater pollution,
- o uses a continuous simulation approach,

TABLE 4-1 AVAILABLE MODEL COMMANDS

NAME	FUNCTION
GENERATE	Causes the simulation of catchment runoff and, optionally, of pollutant load generation by runoff.
POND	Routes flows and, optionally, pollutants through a detention storage pond.
REACH	Routes flows and, optionally, pollutants through a river reach using finite difference approach.
CONVOLUTE	Routes flows and, optionally, pollutants through a river reach using convolution approach.
ADD SERIES	Sums two flow and pollutant series.
SPLIT SERIES	Divides a flow and pollutant series into two parts.
START	Begins run, sets span of run, defines rain and flow input file numbers, and sets pollutant simulation on or off.
FINISH	Terminates run.
POLLUTANT RATES	Establishes pollutant source calculation method, and sets parameters for pollutant source calculation
PLOT SPAN	Plots flow or pollutants for a specified short period.
DUMP PLOT	Plots all flow or pollutant records for one or two series.
PRINT SPAN	Prints flow or pollutant for a specified short period.
DUMP PRINT	Prints all pollutant and flow records for one series.
CALIBRATE	Calculates some basic statistics comparing two pollutant or flow series.
STORE	Converts flow gauge records to internal model format.
EXCEEDANCE CURVES	Calculates number and duration of flow or concentration exceedances.

- is able to readily accept measured or simulated data from other sources, and similarly able to provide data series to external uses,
- is capable of simulating a system of rainfall/runoff, control, and transport components,
- will readily accept alternative algorithms for any and all of its fundamental simulation or interpretive elements,

which are all requirements defined for the model. The three remaining items not mentioned above, namely economy, data requirements, and consistency with other models are discussed in later sections of this report. It is noted here, however, that the model appears to meet these criteria as well. The proposed model structure is therefore fundamentally more suitable for use in a planning environment than alternatives, and meets the general requirements of objective 1.

5.0 Instream Pollutant Routing

5.1 Introduction.

As discussed in the literature review, there is a lengthy history of use of the basic advective/ dispersive transport equation for pollutant routing. One, two and three dimensional models have been proposed for rivers, and a wide variety of alternative mathematical approximations to the transport equations exist. Some of these are exact solutions to special cases, while others are numerical approximations which sacrifice exactitude for flexibility or economy and which allow solution of otherwise intractable problems.

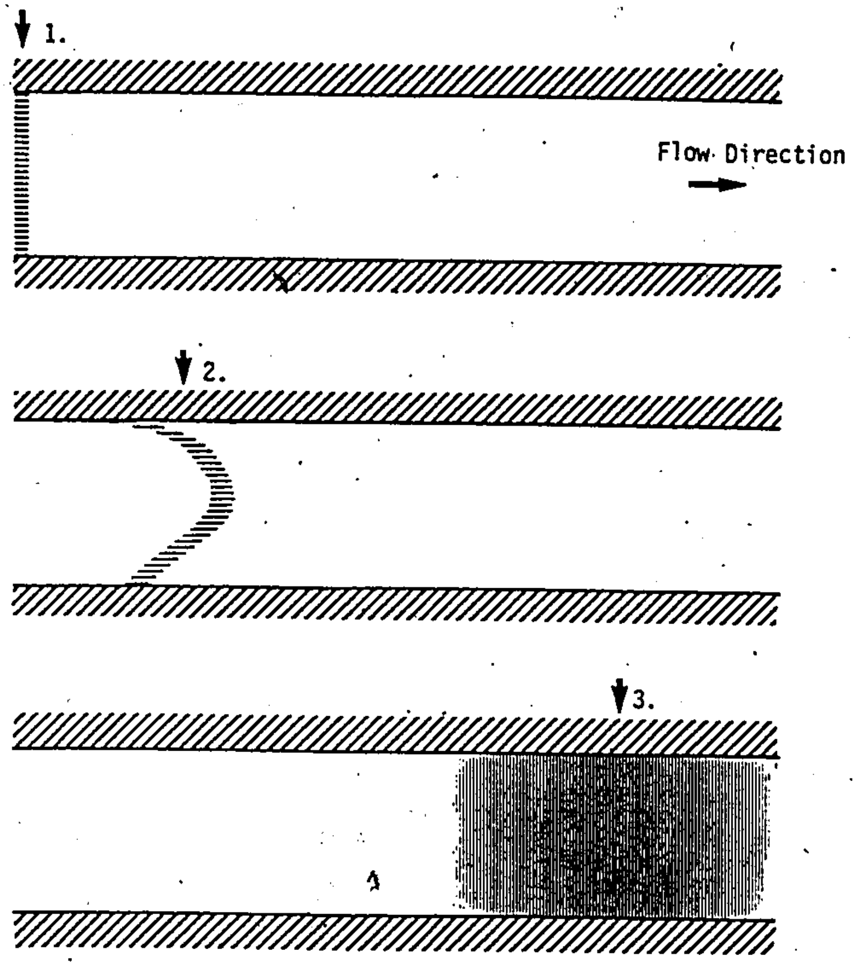
A number of these schemes provide satisfactory solutions for the advective/dispersive transport equation when properly applied in appropriate conditions. To provide the option of using a traditional river routing scheme in the proposed model, one of these has been adopted as one means for river routing. This is described in Appendix C. However, it was recognized that there are drawbacks which typically accompany numerical approximations of the fundamental transport equation. These are described in Chapter 3 and include costs or complexity resulting from computational requirements of the numerical scheme. To provide for an alternative means of river routing in accordance with objective 2, an alternative routing scheme was developed for the model. This scheme is described in this chapter.

It is generally conceded that the process of dispersion of a slug of pollutants in a river occurs in a characteristic manner which can be described as having two stages, as shown in figure 5.1. First, there is a period when local advective patterns tend to distort the cloud of pollutants in a way which is dependant primarily on the velocity distribution across the stream section. This tends to stretch the cloud along the stream, and results in concentration variations across the stream. Some time after this period of initial mixing, variations in concentration tend to even out, and the cloud disperses in a manner described by the Fickian relation, which can be written relative to the centroid of the cloud as:

$$\partial C/\partial t = \partial(Ex*\partial C/\partial x)/\partial x \qquad (5.1)$$

where Ex is a longitudinal dispersion coefficient,
x is distance along the stream,
and C is pollutant concentration.

Typically, as noted in the literature review, river dispersion problems are solved by assuming that the stream behaves according to equation 5.1 throughout the process of mixing. This results in a one-dimensional



1. A source is injected.
2. Initial mixing reflects local advective pattern and is non-uniform across the channel.
3. Lateral variations have evened out and longitudinal mixing progresses.

FIGURE 5.1 STAGES IN MIXING IN A RIVER

river transport equation of the following form:

$$\partial C/\partial t = Vx*\partial C/\partial x + \partial(Ex*\partial C/\partial x)/\partial x + S \qquad (5.2)$$

where Vx is a longitudinal velocity,
and S represents losses or additions of pollutant due to reaction or sources/sinks.

Equation 5.2 provides a means of solving most one dimensional river transport problems. However, there are certain limitations inherent in its application. These can be grouped into numerical and physical categories. The first set of problems relates to the need to solve equation 5.2 numerically for the general case. This is usually done by dividing the river reach into small elements and solving finite approximations of equation 5.2 over successive short time intervals (Patankar, 1980). With sufficiently small time and distance elements the result can be made sufficiently accurate. However, there is a cost penalty associated with this approach, since more time and distance steps imply more numerical calculations and hence greater computational costs. It is an inherent paradox that as a general rule, the less important the dispersive term in equation 5.2 is, the more numerical computations (typically resulting from a finer time step) are required to achieve appropriate results.

The second set of problems are the result of physical processes which do not exactly match the assumed representative equation. In the initial period of mixing, for example, equation 5.2 is not necessarily appropriate. This initial period can be significant, as indicated by Fischer et al (1979) who give as an example transverse mixing on the MacKenzie River, which was not complete for at least 300 miles downstream from a source. Another example can be found in the St. Clair river in Ontario. Recent instream observations of mixing indicate that transverse mixing is only accomplished after a travel distance well beyond the 100 km distance to the receiving lake (Hamdi et al, 1983). Finally, recent theoretical investigations (Dewey et al, 1982) seem to indicate that the initial period of mixing persists for much longer than was previously anticipated. This effect means that, equation 5.2 may be inadequate for significant distances and times after the pollutant has been injected.

This problem is familiar in the form of typical dye cloud results when instream dispersion is measured. Commonly, the observer will obtain results which indicate a long 'tail' of dye trailing behind the main cloud. (Rideau River, 1981; ibid 1981). There have been attempts to account for this observation within the framework of the existing one-dimensional relation (Thackston et al, 1970 and others). It is also possible to abandon the one-dimensional approach when the initial period of mixing is significant and replace it with a multi dimensional model.

Another aspect of the same problem may be considered to exist in situations where sources are introduced at the side of a river, and ultimately spread across the river. The velocity in a channel at the side is typically less than the average velocity, so in such a case the most appropriate value of longitudinal velocity will vary during the period of mixing even if flows are steady. Further, if a number of sources are located along a channel, there can in principle be several velocities representative of a single instream section by virtue of the different progress of mixing of each source. A one dimensional model of the classical type solves equation 5.2 assuming unique values of velocity at each point and does not respond to this fact. It is noted that the non-uniform initial mixing condition potentially occurs whenever a sewer or side channel enters a significantly larger main stream, and this is the type of source commonly at issue in regional stormwater planning.

The above and other problems can in principle be addressed by using multi-dimensional river models if they are deemed important enough to warrant inclusion in an analysis. The two dimensional form of equation 5.2 can be written as:

$$\partial C/\partial t = - Vx^*\partial C/\partial x - Vy^*\partial C/\partial y + \partial(Ex^*\partial C/\partial x)/\partial x + \partial(Ey\partial C/\partial y)/\partial y + S \quad (5.3)$$

where Vy is a transverse velocity,
and Ey is a transverse
dispersion coefficient,

However there is an inherent cost penalty in application of such models. Adding a second dimension to the equation not only provides more detail than is desirable in the type of planning analysis considered here, but increases the cost of numerical solution. For both reasons a two dimensional transport analysis of the traditional form was considered inappropriate in this model.

The emphasis in developing an instream transport algorithm for this research was to investigate the possibility of deriving an instream routing method which is more economical to use than traditional approaches, which is able to respond to the initial mixinq period to some degree without excessive increases in sophistication, yet which can be reconciled with existing methods.

5.2 Proposed Routing Method

General Approach

For the river transport algorithm, it was decided to adopt a convolution routing scheme which employs a response function derived from a two dimensional analysis of the river. For the purposes of this research such a scheme, as is demonstrated below, is capable of providing results consistent with more complex two dimensional models, but at a cost which is less than the simpler one-dimensional models. Although appropriate in principle, convolution approaches have not been commonly applied in stormwater quality simulation models; they have been primarily been associated with a systems approach to river or reactor response, and when used have tended to be by application of the impulse response function which is a solution to equation 5.1 (Medina et al, 1981). The approach to obtaining a response function which is proposed in this research is new.

Assumptions

Pollutant inputs which are dealt with in advective/dispersive transport models generally presuppose that quantities of pollutant mass introduced to a river are continuously divisible. Dissolved materials meet this supposition well. Since it is readily divisible, this mass tends to dissociate and spread after input to the system due to the essentially random turbulent fluctuations in velocity encountered in the river.

It is approximately equivalent to view the pollutant mass introduced to a river as consisting of a large or semi-infinite number of composite particles. In this case, exact definition of the dispersion and transport of a pollutant mass would require that the position each single particle of which the mass is constituted be known. A definition of particle position can be expressed as the following:

$$P(t) = \int V(t)\,dt + Po \qquad (5.4)$$

where P is a vector position of the particle,

Po is the starting position,

and V is a velocity vector.

Since the velocity at any instant in a non-steady non-uniform flow is a function of position as well as time, solution of 5.4 is not straight forward. Furthermore, the velocity vector can be considered to include a random component representing turbulent fluctuations, so for predictive purposes the final position of the particle must be expressed as a probability. Finally, the semi-infinite number of particles of which the mass is composed makes enumeration of individual particles by equation 5.4 in practical problems impossible. Even for an individual particle, further expansion of equation 5.4 by inclusion of the turbulent velocity component quickly results in an expression which is difficult to solve.

It is for this reason that the above (Lagrangian) approach to describing dispersion is usually abandoned when practical problems exist. Instead, the dispersing mass is treated as a continuously divisible substance behaving according to the net effect of local concentration gradients, velocity patterns, and Chick's Law (equation 5.3). As discussed in Chapter 3, the resulting equation is amenable to numerical methods, is theoretically justified, and experimentally shown to be reasonable.

In this analysis, however, an approximation of equation 5.4 is used to generate a response function for a two - dimensional system of the type described by equation 5.3. It is emphasised that the approach used here does not contradict or replace equation 5.3. In fact, this analysis provides an alternative solution to a particular case of equation 5.3, the traditional two dimensional advective/ dispersive transport equation, using elements of the interpretation implied by equation 5.4

Primary assumptions made in this derivation are:

1. Mixing and transport in the river can be

approximated as a two dimensional process.

2. In the direction of the river channel, the transverse variation in longitudinal velocity is the predominant factor in determining the longitudinal spread of pollutants. This assumption is reasonable, and has been the basis of fundamental research by Taylor, Fisher and others.

3. Across the stream, dispersion is the primary factor causing a lateral drift of pollutants. This assumption essentially lumps the mixing caused by turbulence or secondary currents into a dispersive term and eliminates advective movement. In the absence of bends or disturbances in the channel, this is also a reasonable approximation (Fischer, 1979).

4. The lateral spread of pollutants may be described using the Fickian dispersion relation (equation 5.1 expressed in the y-direction, across the stream). As discussed in the literature review, the FIckian assumption is prevalent in the literature.

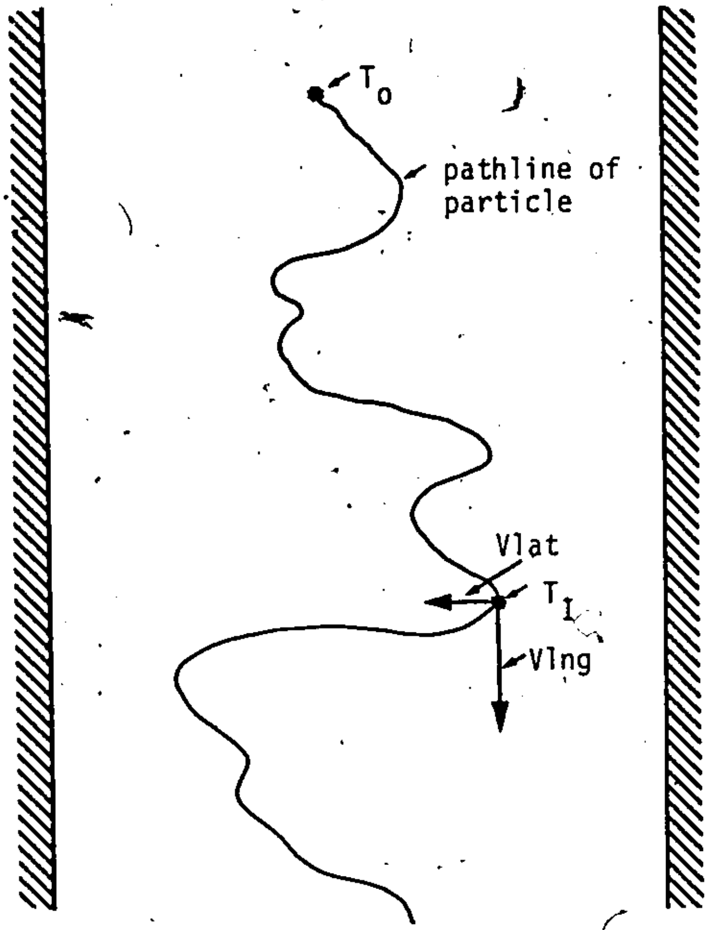
5. The river can be divided along stream lines into sections of characteristic velocity and concentration. Between these sections, transverse dispersion provides lateral travel of pollutants,

while advection is by definition insignificant. Along these sections, advection is the primary factor determining particle position. This is an approximation which has been used with success by Sayre and others in developing the 'stream tube' model of river dispersion.

Points 1 through 4 correspond to a mixing condition of the type shown in equation 5.3 with the second and third terms on the right hand side omitted. This represents a slightly simplified form of two dimensional transport which has been used in a number of previous models (Hamdi et al, 1979 and others). The result is more sophisticated than typical one dimensional models. To this point, the development of the response function is no more or less intuitive than any of the traditional river transport models, since it is based on the same fundamental equations.

Summary of Solution Technique

Under the conditions described above a conceptual model of instream transport can be developed. As shown in figure 5.2, a particle group introduced into the stream at some position will move downstream at a velocity determined by its lateral position at any instant. At the same time, lateral motion in the stream will occur as dictated by molecular diffusion and turbulent dispersion, in this case approximated by a Fickian



T_o- Starting position of particle.
T_1- Position of particle at time t1.
Vlat- Lateral component of velocity at time t1.
Vlng- Longitudinal component of velocity at time t1.

FIGURE 5.2 CONCEPT OF PARTICLE BEHAVIOUR INSTREAM term.

The position of the particle after some time (t1-t0) would be defined by equation 5.4 according to the time history of particle velocity. Under the conditions described above, the lateral and longitudinal movements of the particle can be treated as independent and integrated separately. Determination of the final particle position therefore can be achieved by a separate treatment of the lateral and longitudinal velocities, provided that the time history of each is known. Alternatively, if the time history of particle position in the lateral direction is known, determination of the final longitudinal position can be achieved by recovering and integrating the time history of longitudinal velocity from this information. This is the general approach used in this method.

In essence, a finite approximation to equation 5.4 is applied.

1. The river is divided laterally into sections of uniform velocity along the river, and the time span of simulation is divided into an integer number of equal time intervals.

2. All possible lateral position sequences over the time span are enumerated, and the longitudinal position of a particle following each possible

sequence is calculated.

3. The probability of each sequence, or alternatively the numbers of particles following each sequence, can be calculated and hence the numbers of particles at each of the longitudinal positions determined in step 2 can be determined.

4. The result is a known longitudinal distribution of the input slug at the end of the simulated time span.

Detailed Development of Solution Technique

In general, the problem to be solved is to find the longitudinal distribution of an initial particle mass after some elapsed time period. The stream is conceptualised as having a longitudinal velocity which varies according to some lateral distribution and which can be approximated as a number of sections of constant velocity. A dispersive movement across the stream is assumed to be described by a Fickian dispersion term. The solution is required for a reach of uniform characteristics along the stream, and steady in time.

As indicated above, the distance travelled downstream by a particle group released into one of the stream sections after some long period of time would be the sum of the incremental distances travelled by the particle during its stay in each section. By knowing how long the particle remains in each section, and the velocity in each section, the magnitude of these incremental distances could be determined. In principle, therefore, if the lateral path taken by the particle could be described, its downstream position could be found.

An interpretation of this approach can be made as follows. A particle group is introduced into one stream section. After a single short time step Δt, the particle will move some distance Δx forward (along the river) and some distance Δy sideways.

This situation can be written:

$$P(t+\Delta t) = P(t) + \Delta y + \Delta x \qquad (5.5)$$

Dividing, by the time step _t,

$$(P(t+\Delta t)-P(t))/\Delta t = (\Delta y + \Delta x)/\Delta t \qquad (5.6)$$

Now as Δt becomes infinitely small, equation 5.5 approaches:

$$dP(t)/dt = d_{\Delta}y/dt + d_{\Delta}x/dt \qquad (5.7)$$

or,

$$dP(t)/dt = Vx + Vy \qquad (5.8)$$

which is an alternative to equation 5.4, expressed at a point. In other words, the conceptual model described above can be viewed as a finite approximation of equation 5.4.

Using equation 5.5, determination of the path of a particle over a long time period becomes the problem of finding:

$$P(t+n\Delta t) = P(t) + \mathrm{sum}(\Delta y(i),n) + \mathrm{sum}(\Delta x(i),n) \quad (5.9)$$

where the notation

$\mathrm{sum}(b(i,j,k\ldots),n,o,p\ldots)$

represents the sum of $b(i,j,k\ldots)$ evaluated over the range of n,o,p and so on.

The third term on the right hand side can be evaluated once the behavior of the second term on the right hand side is known. Solution of the transport problem in this research was achieved by application of equation 5.9 under particular conditions of assumed behavior of the two right hand terms.

Numerical Method.

Let the time span over which routing occurs be divided into 'n' intervals of equal length Δt, and to step forward from time zero to time '$n\Delta t$' in time steps of size Δt.

Let the river be divided into 'm' intervals across its width. For convenience, let the intervals be of equal width, Δy, and designate the intervals as:

$$Y = (y1,\ y2,\ y3,\ \ldots\ ym)\ . \quad (5.10)$$

where Y is the set of intervals yk across the stream

Define a possible path sequence for a particle released at yi as being any combination length 'n' of members of Y:

$$Yseq = \{yi, yj1, yj2, \ldots yjn\} . \tag{5.11}$$

where yi is the section at which
the particle group begins,
and yjn is the section location of
the particle group at step
'jn'

Equation 5.11 implies an additional simplifying assumption in the proposed method, and is a consequence of the finite approximation of time and distance described above. The equation states that the possible lateral paths which can be taken by a particle over the time span of 'm' intervals can be approximated as a sequence of an integer number of positions across the stream. By definition, as Δy and Δt approach zero the sequence of equation 5.11 can approach any possible physically real particle path.

Since the number of time and distance steps in this solution are fixed and finite, all possible Yseq can be determined. To solve the routing problem it therefore remains, for any sequence Yseq described by 5.11, to determine the longitudinal position traveled after 'm' time steps, and the number of particles which follow the sequence Yseq.

Estimation of Numbers of Particles Following a Particular Sequence.

In view of assumptions 1 through 4, it is possible to describe the lateral movement of particles by means of an equation in the form of equation 5.1. This reflects the fact that, although an alternative frame of reference is being applied to achieve a solution, the method still assumes applicability of the traditional dispersive behavior of a particle group introduced to a river.

Taking Ey to be constant, the solution for equation 5.1 can be derived by elementary calculus and is given in any text on instream transport. This solution is written here for an unconfined source in a water body with a background concentration field:

```
 C(y,t) = Mo/sqrt(4*t*Ey*Pi)*
                         exp(-x*x/4/Ey/t)+Cb(y)   (5.12)
```

where Mo is the injected mass,
Ey is the lateral dispersion coefficient,
t is elapsed time,
Cb is background concentration,
and C is total concentration

For Cb(y) equal to zero, equation 5.12 can be seen to take the form of a Gaussian distribution with:

mean position = 0 (5.13)

and

variance = sqrt(2*Ey*t) (5.14)

At any time, equation 5.12 can be used to solve for concentration as a function of location directly. To determine the fraction of mass M in any interval distance 'y' equation 5.12 must be integrated over the interval 'y'.

If a particle group is located at a point in one stream section at time t1, then the number of particles in any of the possible stream sections at time t2 can be determined by setting:

$$t = t2 - t1 \quad (5.15)$$

in equation 5.12, and integrating the result between y1 and y2 where y1 is the distance from the point of input to one lateral boundary of a stream section, and y2 is the distance from the point of input to the other lateral boundary of that section.

$$M(y0,y1,y2) = \int C(f,t)\, df \quad (5.16)$$

where M(y0,y1,y2) is the mass at time t between section boundaries y1 and y2, for a mass introduced at y0,

and f is a constant of
integration.

This is illustrated in figure 5.3. The fraction of the injected mass, Mf, between y1 and y2 is defined for convenience as:

Mf(y0,y1,y2) = M(y0,y1,y2)/Mo (5.17)

The integration in equation 5.17 cannot be performed directly (Kennedy et al, 1976) so a numerical approximation must be made. This can either be achieved by using tables of the normal distribution, or by using relations of the type derived by Abramowitz and Stegun (1965). Their numerical expressions for the normal curve and its integrated form for T > 0.0 are written here as:

FT(T)=1.0/(B+B1*T*T+B2*T**4+B3*T**6) (5.18)

where B=2.490895
B1=1.466003
B2=-0.024393
B3=0.178257
and FT is the value of the
normal curve at T.

and as:

PT(T)=1.0-FT(T)*(A1*Q+A2*Q*Q+A3*Q**3) (5.19)

where Q=1/(1+A*T)

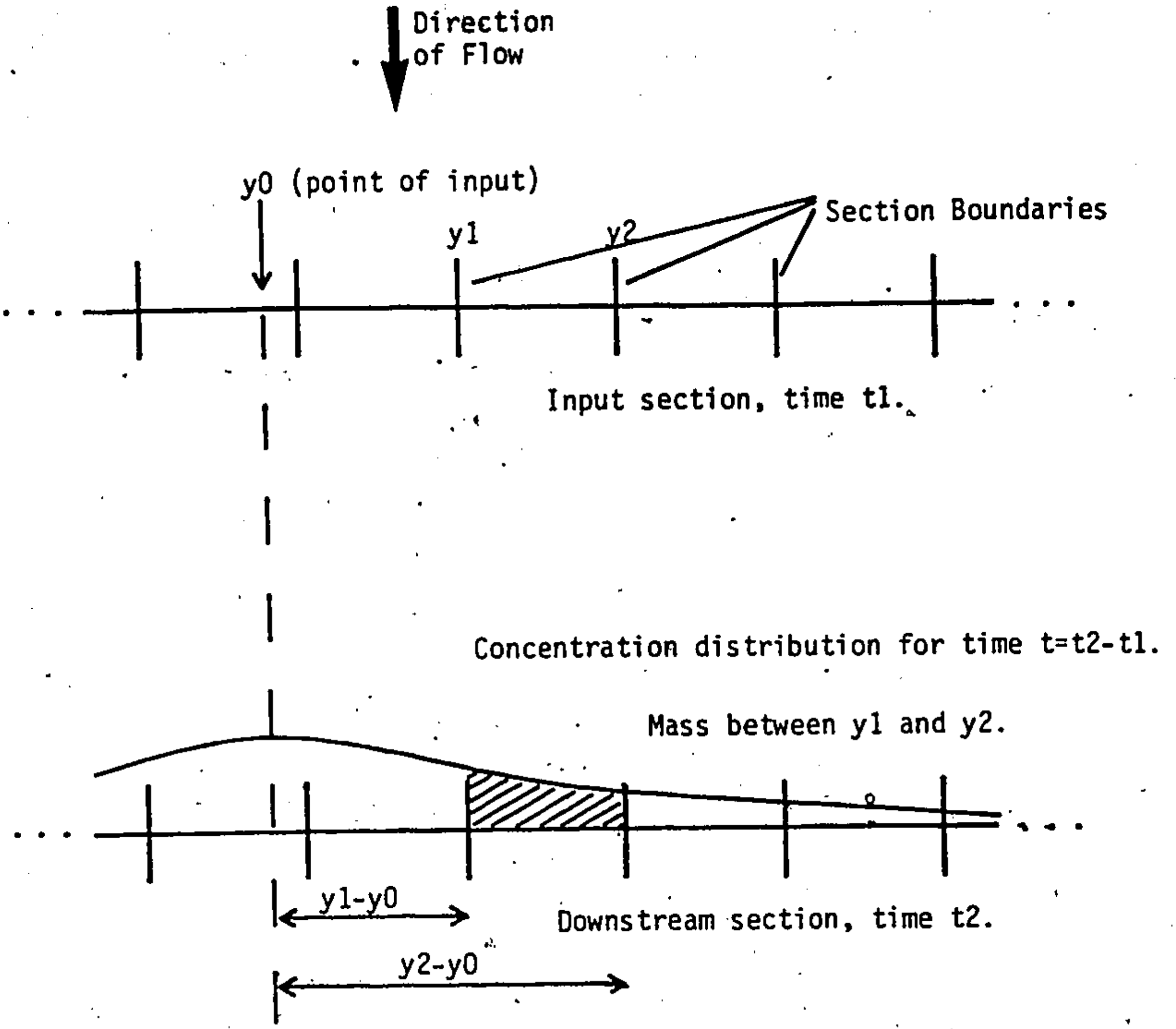


FIGURE 5.3 DEFINITION SKETCH FOR EQUATION 5.16
(DETERMINATION OF MASS IN A SECTION
AT TIME t2-t1 AFTER INJECTION)

with $A=0.33267$
$A1=0.43618$
$A2=-0.12017$
$A3=0.93730$
and PT is the integral of the normal curve from minus infinity to T.

Using the principle that the normal curve is symmetrical about $T=0$, equations 5.18 and 5.19 were used in this work for the entire range of T (+ or - infinity) as follows:

$$F(T) = FT(\mathrm{abs}(T)) \tag{5.20}$$

where F is the value of the normal curve at T.

$$P(T) = \left\{ \begin{array}{l|l} PT(T) & T > \text{or} = 0 \\ 1-PT(\mathrm{abs}(T)) & T < 0 \end{array} \right\} \tag{5.21}$$

where P is the integral of the normal curve from minus infinity to T.

To determine the pollutant mass between 'yi' and 'yi+1' at time 't' when a mass 'm' is released at point 'yj' at time 'to', the following relations are used:

$$\sigma = \mathrm{sqrt}(2*Ey*(t-t0)) \tag{5.22}$$

$$xyi = (yi-yj)/\sigma \tag{5.23a}$$

$$xyi+1 = (yi+1-yj)/\sigma \tag{5.23b}$$

$$M(yi,yi+1) = Mo*(P(xyi+1)-P(xyi)) \qquad (5.24)$$

With 'n' stream sections defined, equation 5.22 is applied once and equations 5.23 through 5.24 'n' times to obtain the mass reaching each section.

After one time step, the above calculations have converted one point source of mass to 'n' distributed sources of mass. Over the next time step, from time t2 to time t3, it is necessary to determine how these 'n' distributed sources are redistributed. Since equation 5.12 is linear in M, this is equivalent to the problem of establishing the behavior of the mass distributed in any single section. The mass will evidently become redistributed across the 'n' available sections again.

A direct solution to this problem is again not possible, and the numerical approach applied in equation 5.24 is less exact than in the first case. This is because in the second time step the initial conditions at the beginning of the step are not point sources. Equation 5.12 must therefore be replaced by:

$$C(y,t3) = \int C(f,t2)/sqrt(4*Pi*Ey*(t3-t2))* \qquad (5.25)$$
$$exp(-(y-f)**2/4/Ey/(t3-t2))df$$

where f is a variable of integration.

for which there is no direct solution. An approximate solution to equation 5.25 is therefore required.

In principle, term by term integration of a fitted polynomial or other approximations could be used to achieve a close numerical solution to 5.25. In this work, a solution to equation 5.25 obtained by representing the distributed mass existing at the end of time t3 as a series of masses located at the mid-point of each section was tested. As shown below, it appears that the approximation involved is sufficient for the purposes of this research. With this approximation, equations 5.22 through 5.24 can be applied for each of the 'n' elements with 'yj' taken as the midpoint of each element.

In the second and each subsequent time step, there are 'n' different particle groups travelling to 'n' different locations, so the mass in each section becomes the sum of 'n' different sources.

A sequence of lateral section positions is redefined as:

$$Y = \{y1, y2, y3 \ldots ym\} \qquad (5.26)$$

where Y is a sequence of 'm' positions y1, y2, and so on, during intervals 1,2,3...m.

in which position is approximated as the midpoint of the occupied section. Then the fraction of particles travelling along a particular pathline is:

$$\mathrm{Frac}(Y) = \mathrm{prod}(Mf(y1,y2-\Delta y,y2+\Delta y),Mf(y2,y2-\Delta y,y3+\Delta y)\ldots Mf(ym-1,ym-\Delta y,ym+\Delta y) \qquad (5.27)$$

where the notation prod(a,b,c...z)
refers to the product of the
series a,b,c ...z.

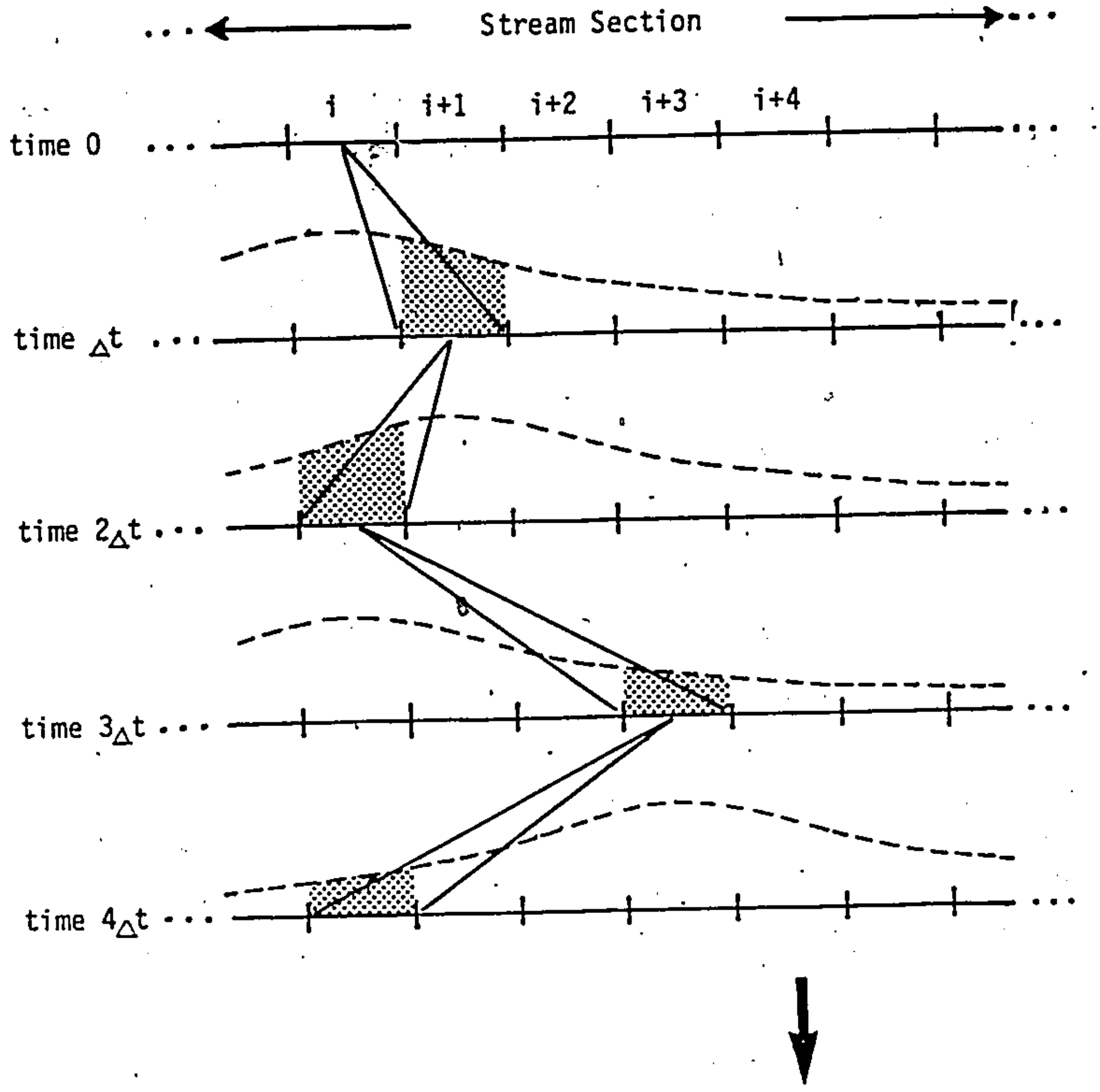
This situation is depicted in figure 5.4. As shown there, each possible path accounts for a fraction of the particles, and the fraction for each can be determined by application of equation 5.27.

Determination of the numbers of particles travelling any section sequence has therefore been reduced to the problem of solving equation 5.27.

It is noted at this point that the computational requirements in solving the above progression quickly become significant, since each step results in 'n' different destinations for 'n' different sources. After 'm' time steps:

$$Npaths = n**m \qquad (5.28)$$

where Npaths is the number of
possible particle paths 'Y'
n is the number of stream
sections,
and m is the number of time steps.



------ Mass fraction distribution for a step of size Δt.

▨ Mass fraction in section.

Arbitrary sequence shown is

$Y = (y_i, y_{i+1}, y_i, y_{i+3}, y_i)$

FIGURE 5.4 DEFINITION SKETCH FOR EQUATION 5.27. (APPROXIMATION OF LATERAL SEQUENCE OF POSITION)

For even a moderate number of river sections and time steps, this number can be large.

Equation 5.27 is general, however the use of equation 5.24 in solving equation 5.27 is only correct in the stated conditions of an infinitely wide channel or a channel where the dispersing cloud does not impinge on the channel boundaries. For the case of a narrow channel or a channel where the cloud impinges on the side because it is released near shore, further images of the original source must be added, and equation 5.24 is replaced by an alternative.

Sayre and Chang used the method of images to solve for transverse mixing of a constant side channel line source in a river channel, and this has been adopted and further tested by Hamdi et al(1979). This relation is written here for a channel with uniform cross-sectional velocity as:

C(y,t) = Mo/sqrt(2*Ey*Pi)*

sum(Et(xo,i),n) (5.29)

where Et(xo,i)=exp(-vx*(xo-2*i*b)**2/4/Ey/x)+Cb(y)

vx is the channel velocity,

b is the channel width,

n ranges from plus to minus infinity,

and Mo is found from the mass rate of pollutant input divided by the channel depth and river flow velocity.

For this research, it was necessary to derive a relation similar to 5.29 but which can be used for a source at the midpoint of one of an integer number of equal stream sections. Figure 5.5 depicts the general approach to this problem.

The section widths in this analysis are restricted to equal widths. Then the first image is located at a point:

$$do = dn + \Delta y/2 \qquad (5.30)$$

and the target section spans the positions 'dm' through 'dm+1'. Taken with the river bank as a datum, a section boundary 'di' is located at:

$$di = (i-1)*\Delta y \qquad (5.31)$$

so the displacements of the section boundaries (dm and dm+1) from the source (do) are:

$$\begin{aligned} d1 &= dm-do \qquad (5.32a) \\ &= (m-n-1/2)*\Delta y \end{aligned}$$

and:

$$\begin{aligned} d2 &= dm+1-do \qquad (5.32b) \\ &= (m-n+1/2)*\Delta y \end{aligned}$$

Let 'yi' be defined as the displacement of an image

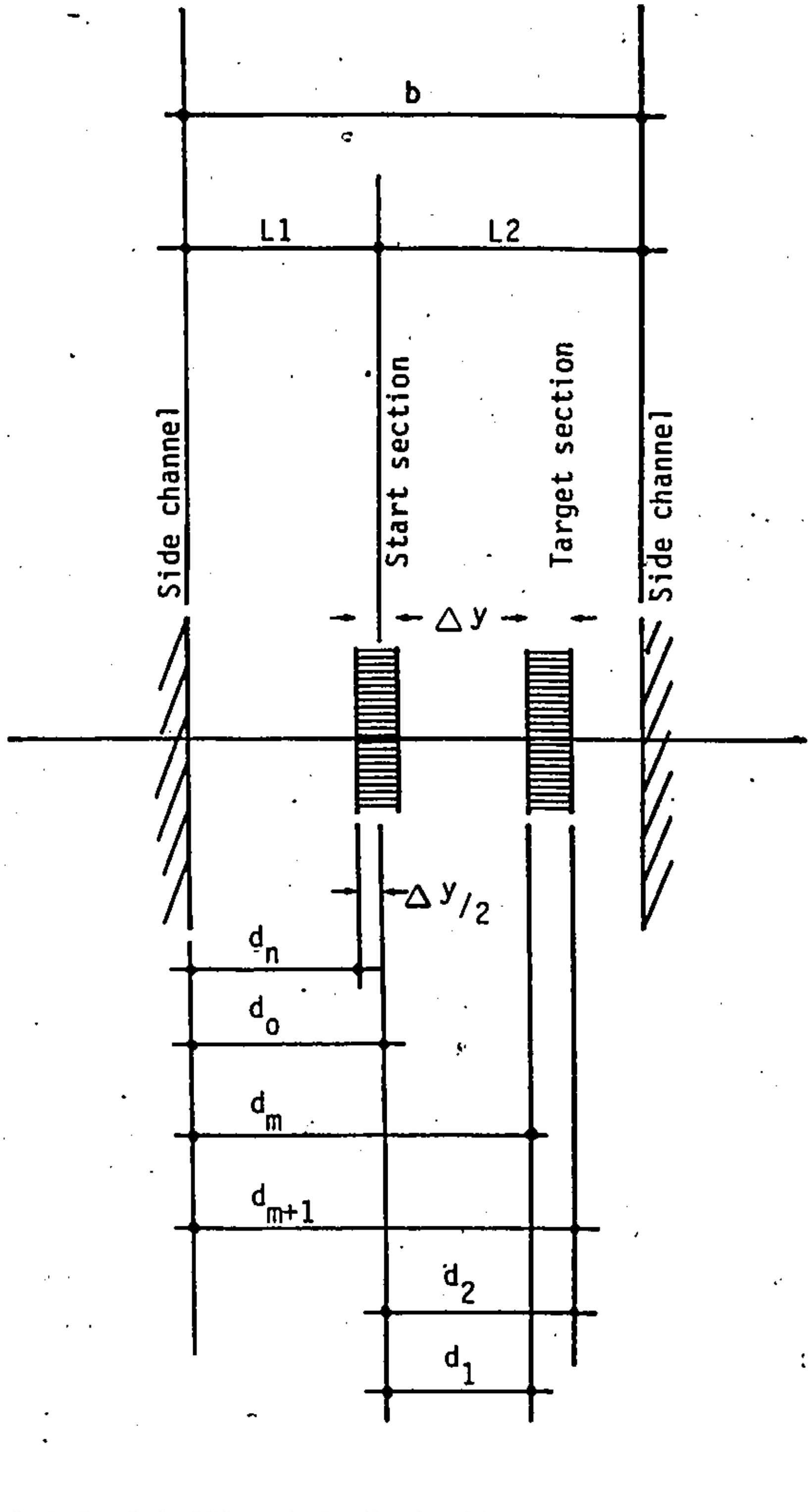


FIGURE 5.5 DEFINITION SKETCH FOR SOLUTION OF MASS DISTRIBUTION FOR ARBITRARY POINT OF INPUT IN A CONFINED CHANNEL.

from start point 'do', and 'yo' as the displacement of a point of observation from the start point. The distance of the start point from the two banks are L1 and L2, where:

$$b = L1 + L2 \qquad (5.33)$$

Then there can be defined an infinite number of images contributing to concentrations observed at any point dm instream. These are located at:

$$\begin{aligned} yi=\{-2*L1,(2*L1+2*L2),-(4*L1+2*L2) \\ (4*L1+4*L2),-(6*L1+4*L2)...\} \end{aligned} \qquad (5.34a)$$

and at:

$$\begin{aligned} yi=\{+2*L2,-(2*L2+2*L1),(4*L2+2*L1), \\ -(4*L2+4*L1),(6*L2+4*L1)...\} \end{aligned} \qquad (5.34b)$$

distances from the point of input.

Collecting terms and using 5.31, the total series of sources and images can be written:

$$\begin{aligned} yi=\{...-(4*b+2*L1),-4*b,-(2*b+2*L1),-2*b,-2*L1,0, \\ 2*L2,2*b,(2*b+2*L2),4*b,(4*b+2*L2)...\} \end{aligned} \qquad (5.35)$$

For L1 zero, a line source at the bank occurs, and equation 5.18 can be recovered. Letting:

$$K1 = Mo/2/sqrt(4*Pi*Ey*t) \qquad (5.36)$$

and

$$K2 = -1/(4*Ey*t) \qquad (5.37)$$

equation 5.12 can be restated for the source and its images as:

$$C(y,t) = K1*sum(exp(k2*(yo-yij)),n) \tag{5.38}$$

where yij is the jth term
in 5.24,
and n ranges from plus to
minus infinity

which can be expanded to:

$$\begin{aligned} C(yo,t) = K1*(&exp(K2*yo**2 \\ &+ exp(K2*(yo+2*L1)**2) \\ &+ exp(K2*(yo-2*L2)**2)) \\ &+ K1*sum(exp(K2*(yo+2*j*b)**2) \\ &\qquad +exp(K2*(yo+2*j*b+2*L1)**2),n) \\ &+ K1*sum(exp(K2*(yo-2j*b)2) \\ &\qquad +exp(K2*(yo-2j*b-2*L2)2),n) \end{aligned} \tag{5.39}$$

where n ranges from 1 to
plus infinity

which is the general solution to equation 5.1 for a line source in a channel of finite width. The mass between two points 'dm' and 'dm+1' from a source at 'do' can be derived, as before, by integrating equation 5.38.:

$$M(d1,d2) = \int C(f,t)\ df \tag{5.16}$$

Equation 5.38 can be integrated in parts since the terms are all additive, and can therefore be solved using equation 5.16 and 5.17. This is most easily accomplished by letting:

$$\sigma = \mathrm{sqrt}(2*Ey*(t-to)) \tag{5.22}$$

$$xyi = (yi-yj)/\sigma \tag{5.23a}$$

$$xyi+1 = (yi+1-yj)/\sigma \tag{5.23b}$$

as before. Then equation 5.16 can be altered to include the image terms as a summation as well as the original source term:

$$Mf(do,yi,yi+1) = Mf(do,0,yi+1)-Mf(do,0,yi) \tag{5.40a}$$

$$\begin{aligned} Mf(do,0,yk) = {} & P(yk/\sigma) \\ & + P((yk+2*L1)/\sigma) + P((yk-2*L2)/\sigma) \\ & + \mathrm{sum}((P((yk+2*j*b)/\sigma) \\ & \qquad +P((yk+2*j*b+2*L1)/\sigma),n) \\ & + \mathrm{sum}((P((yk-2*j*b)/\sigma) \\ & \qquad +P((yk-2*j*b-2*L1)/\sigma),n) \end{aligned} \tag{5.40b}$$

Therefore, a relation has been derived which allows estimation of the mass in each stream section after some period of time in a bounded stream. A means to handle the problem of estimating the lateral component of the pathline from time step to time step has been defined, and the solution of equation 5.27 is now possible.

Longitudinal Particle Travel

The problem of determining the longitudinal travel represented by a particular particle group path can be stated as the problem of defining:

$$Zx(t1) = \int vx(tau).dtau \qquad (5.41)$$

where vx is the velocity along the channel,
and tau is a variable of integration.

since as stated above the lateral and longitudinal components of velocity can be treated independently in this development.

If applied to a river reach where longitudinal velocity is not a function of longitudinal position, equation 5.41 can be simplified for the stated conditions of discrete time intervals and constant section velocity to:

$$Zx(t1) = sum(\Delta x(i),n) \qquad (5.42)$$

where $\Delta x(i)$ is the distance travelled during time step i,
and n is the number of time steps of size Δt in the interval 0 to t1.

and since:

$$\Delta x(i) = vx(i)*\Delta t \tag{5.43}$$

where vx(i) is the average velocity during time step i.

where:

$$vx(i) = vbar(xi,xi+1) \tag{5.44}$$

where vbar(xi,xi+1) is the average of the longitudinal velocity as the particle moves from position i to i+1 in the sequence X defined in equation 5.24.

then:

$$Zx(t1) = \Delta t*sum(vbar(xi,xi+1),n) \tag{5.45}$$

Therefore, determination of the longitudinal particle position is essentially determining the average particle velocity during each time step as it travels the lateral pathline series defined by equation 5.26.

Calculation of 'vbar' is complicated by the indeterminate nature of the path from xi to xi+1. The simplest empirical interpretation of the lateral motion of particles expressed by equation 5.1 and its solution 5.12 might be to infer an average lateral distribution of velocities in the particle cloud, which results in the observed lateral displacements of those particles.

In this case, if it is assumed that lateral velocity is essentially constant over a short time period, it can be stated:

$$vbar(xi,xi+1) = sum(v(j),n)/(xi+1-xi) \qquad (5.46)$$

where v(j) is a longitudinal
velocity in section 'j'
and n ranges from xi to xi+1

which is the arithmetic average of velocities of the stream sections between and including the terminal and end points of a jump during a time step Δt.

In fact, dispersion occurs as a random process engendered by random turbulent fluctuations. This means that particles travel from section xi to section xi+1 by an indeterminate number of different routes. In this interpretation of equation 5.12, the number of possible routes and hence velocities encountered from xi to xi+1 is infinite. Calculation of Vbar using equation 5.46 therefore constitutes an approximation which was introduced and tested in this work. The validity of this assumption depends on how closely a particle path is likely to approximate a path which is linear in time between two points.

If the time step Δt is short, the assumption of an average Vbar calculable as in equation 5.46 is intuitively reasonable. This can be argued since, for any given probability value, the average distance traveled by a particle decreases with time span. By taking Δt sufficiently small, variations in longitudinal velocity within a span of lateral distance can in principle be made to decrease until the probability of a particle having occupied a position of significantly different velocity (lateral position) is as small as desired. Stated another way, this is essentially the same argument used in justifying finite difference methods. As the time or distance steps decrease, the finite approximation improves.

Support for use of equation 5.46 over larger steps of time can be derived by considering the form of equation 5.12. This equation can be viewed as an expression of the probable position of a particle, since the relation represents the final distribution of a large number of essentially independent individual particles (Fischer et al, 1979). If it is accepted that equation 5.10 applies for any arbitrary short time, a necessary consequence is that the highest probability of any intermediate location between points 'xi' and 'xi+1' lies along the most direct route between them.

To demonstrate this, let the position of a particle be described as:

```
    p(x,t|xo,to) =                                    (5.47)
          k1*exp(k2*(x-xo)**2/(t-to))/sqrt(t-to)
```

where p(x,t) is the probability
density function for the
particle position x
at time t given that
the particle is at xo at time 0

which is in keeping with equation 5.10. Then if a particle is at position 'x' at time 't', the probability that it started at 'xo' at time 'to' and also was at position 'x1' at an intermediate time 't1' can be stated:

$$P = p(x1,t1|xo,to)*p(x,t|x1,t1) \tag{5.48}$$

where P is the probability that the
particle moves from 'xo' at
time 'to' to 'x' at time 't'
via 'x1' at time 't1'.

since as previously noted it is assumed that subsequent particle motions are independent.

Letting the intermediate position 'x1' be a fraction 'a' of the interval '(x-xo)', occurring at time 't1' which is a fraction 'b' of the interval '(t-to)' then:

$$\begin{aligned} P &= k**3/(\mathrm{sqrt}(b*t)*\mathrm{sqrt}((1-b)*t))* \\ &\quad (\exp(k2*(a*x)**2/(b*t))* \\ &\quad \exp(k2*((1-a)*x)**2/((1-b)*t)) \\ &= k**3/t/\mathrm{sqrt}(b-b**2)* \end{aligned} \tag{5.49}$$

(exp(k**2*x**2/t*(a**2/b))*
exp(k**2*x**2/t*((1-a)**2/(1-b))

taking for convenience:

C1 = k**3/t (5.50a)

and

C2 = exp(k**2*x**2/t) (5.50b)

then

P = C1/sqrt(b-b**2)* (5.50c)
C2**(a**2/b)
C2**((1-a)**2/(1-b))
= C1/sqrt(b-b**2)*
C2**((a**2/b)+((1-a)**2/(1-b)))

Now in this problem C2 is always a fraction less than 1, so P is maximised for any time interval 'b' when the exponent of C2 is minimised. Defining

C2EXP = a**2/b + (1-a)**2/(1-b) (5.51a)

then

C2EXP = (a**2*(1-b) + (1-a)**2*b)/(b - b**2) (5.51b)

and

dC2EXP/da = (1/(b-b**2)*(2*a*(1-b) - 2*b*(1-a))(5.52)

The range of 'b' (the time fraction t1/t) is 0 to 1 so:

1/(b-b**2) = 0 (5.53)

and as a result P is greatest if:

2*a*(1-b) = 2*b*(1-a) (5.54)

To solve this, let 'a1' and 'b1' be an arbitrary pair of points satisfying 5.54. Then choose 'k' such that

a1 = k*b1

then 5.52 must satisfy:

k*b1*(1-b1) = b1*(1-b1*k)

or, k*b1 - k*b1**2 = b1 -k*b1**2

which is only true if k=1 so that:

a1 = b1

and therefore 'a' must equal 'b'. Also, note that the second derivative of 'C2EXP' with respect to 'a' is:

C2EXP''= (1/(b-b**2))*(2) (5.55)

which is greater than zero for all 'b' and guarantees that 5.53 defines a minimum.

The net result is that for any particle which has travelled from point 'xo' to 'x' in the interval 'to' through 't', the most likely position of that particle at any time 't1' which occurred between 'to' and 't' is a linear fraction of elapsed time from the beginning point.

For example, the most likely position at time 't1=t/2' is 'x/2', or in general, :

$$\max(P(x1,t1|xo,to,x,t)) = (x-xo)*(t1-to)/(t-to) \quad (5.56)$$

Thus, the most likely route between the points is the line which accords with the averaging assumption made in equation 5.46. Other routes become increasingly less likely as they deviate further from this assumption. For these reasons, equation 5.46 was tested for use as a means of calculating the longitudinal travel represented by any path line X.

In summary, a numerical approach to solving a particular case of two dimensional advection/dispersion conceptual problem represented by equation 5.10 has been derived. Taken together, under the specific instream conditions assumed here, equations 5.25, 5.45, and 5.46 provide a means of estimating the longitudinal distribution of an input slug of pollutants, as follows:

(1) 'Npath' different pathlines are established, as all the possible sequences from up to 'n' points of input to 'n' terminal points over 'm' time steps.

(2) For each pathline, a mass fraction and downstream distance is established using 5.25, 5.45 and 5.46

(3) By assembling the results of all pathline calculations from step (1) and (2), a mass distribution over downstream distance is obtained. This represents the response function of the stream.

This represents a method of routing pollutants instream which solves the mixing equation in a way which is different from existing approaches both in concept and in detail. The response function which results is, as dictated by the original equations (5.1 and variations) linear in concentration. Therefore, once the response function has been derived, it is possible to apply existing systems techniques to route one or more time series through a river reach. Specifically, an input series can be convoluted over the response function to achieve the output series.

The validity of the method depends on:

1. The validity of the fundamental equation (5.1 and assumptions).

2. The validity of equations 5.25, 5.45, and 5.46.

Point 1. above pertains equally to traditional methods, and reflects the fundamental theory of dispersion rather than the routing method used in this work. This aspect was therefore accepted a priori and not examined further.

Point 2, however, was tested to establish whether results comparable to existing methods are obtained using the proposed method and if so how the method compares to traditional approaches in terms of computational effort.

5.3 Testing the Proposed Routing Method

5.3.1 Comparison to the Hamdi et al (1979) Model

To determine how well the approximation of distributed masses in each stream section performs in terms of routing lateral pollutant distributions from one time step to the next, the routing scheme developed above was compared to the results of an existing two dimensional model. The model used for this testing is the MOE model developed by Hamdi et al (1979) for use in establishing mixing zone requirements for industrial waste discharges into rivers.

This model was selected for two main reasons. First, the model has been tested by the authors against chloride data for a shore based discharge, collected in the St. Clair river. This provided a set of parameters for the test case which are representative of a specific physical location, and incidentally provides a verification of the benchmark model.

A further reason for selecting the model by Hamdi et al for testing is that it employs an exact solution of equation 5.3 for the case of vertical line source injected into a uniform steady stream flow with zero lateral advection and zero longitudinal dispersion. This is analogous to equation 5.29, but appears in the form:

```
C/Co = Qo/h/sqrt(u*Pi*Ey*x)                          (5.57)
          *sum(exp(-u*(y-2*j*b)**2/4/x/Ey),n)
```

where Qo is source flow rate,
Co is source concentration,
h is river depth,
b is river width,
u is velocity along the river,
and C is the concentration at a point 'x' downstream and 'y' from shore.

Notation in 5.57 has been changed somewhat from that of the authors to conform to the conventions used herein.

Equation 5.57 represents a situation which can be well represented by the method developed above, so there is a common basis for comparison between the two models. Also, the exact solution provided by the benchmark model for the test case does not incorporate numerical errors which are inevitably present in models using finite approximations. As a result, differences in solutions between the two in the test case can be assumed to predominantly result from approximations in the tested model rather than in the benchmark model.

The model by Hamdi et al was modified somewhat for use in this testing. Principally, this involved inclusion of routines to provide an integrated result for mass between grid points at a section. Also, the capability to undertake a routing in multiple steps as described above (equations 5.27 and 5.40) was added to permit an immediate comparison of the two methods.

The dispersion parameters for the test case were based on the St. Clair river dispersion analysis. Parameters were:

E_y = 0.41 sq. m. per sec.

b = 110 m

h = 10 m

t = 0.6 hours

Δy = 10m

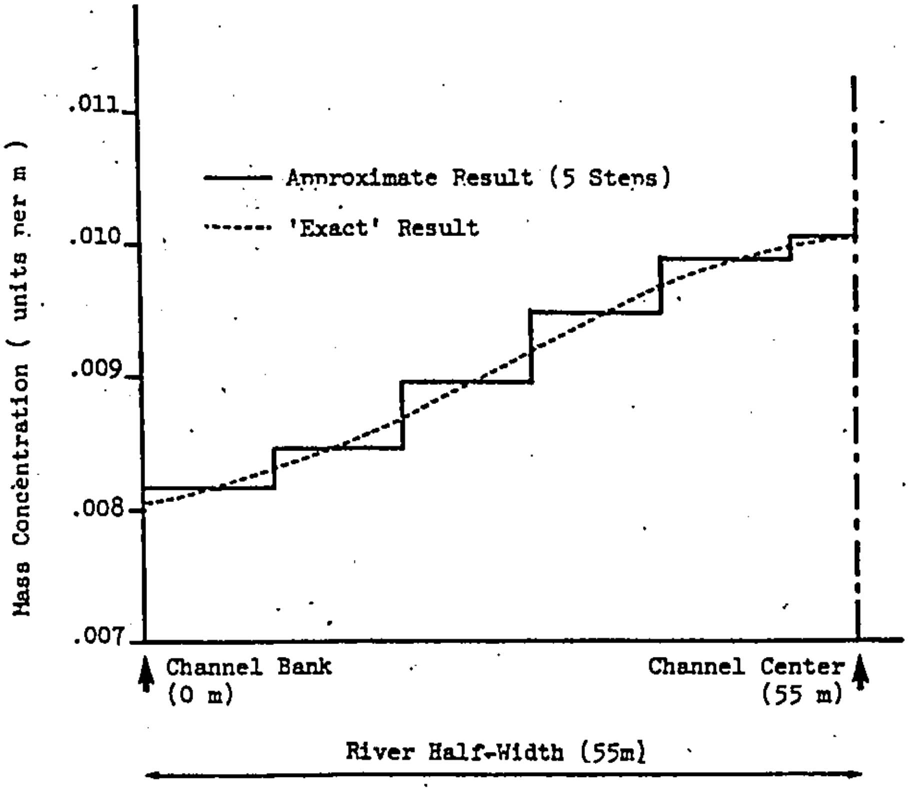
Images calculated = 5

The solution was obtained for 1, 2, 3, 4 and 5 steps taken in the x (downstream) direction. Table 5.1 compares results of the single ('exact') step and multiple step analyses, and figure 5.6 plots the

FIGURE 5.6 Comparison of Lateral Distributions Predicted
by the M.O.E. Model and the Proposed Approximation

Tabulated Result - Comparison of Computed Mass in Section (1 unit input total)

Case \ Section		0 - 10m	10 - 20m	20 - 30m	30 - 40m	40 - 50m	50 - 60m	TOTAL
'Exact' Solution		0.08271	0.08367	0.08805	0.09573	0.10170	0.09557	0.99930
Approximation	2 Steps	0.08135	0.08435	0.08947	0.09502	0.09924	0.10090	0.99976
	3 Steps	0.08142	0.08443	0.08949	0.09501	0.09922	0.10080	0.99994
	4 Steps	0.08157	0.08454	0.08952	0.09494	0.09909	0.10060	0.99992
	5 Steps	0.08174	0.08465	0.08954	0.09487	0.09894	0.10050	0.99998
Largest Error ('Exact' compared to Approximate)		0.00136	0.00098	0.00149	0.00086	0.00276	0.00493	



Plotted Result - Comparison of 'Exact' and Approximate Mass Distributions concentration curve of the MOE model against the step function concentration curve (mass in the section divided by the section width) obtained by the method derived in this work.

As shown, there is little difference between the results. The largest differences in mass between the analytic solution and the approximate step-by-step method are less than 0.3%. Total errors in mass at a section are always less than 0.1%. The concentration curves also indicate essentially the same result between the two methods. It is concluded therefore that, provided the lateral distance step is not large, the approximations used to derive equation 5.27 as solved by equation 5.40 (a point representation of mass distributed across the section) appear to be appropriate. Further, it appears from this example that the approximate step by step approach produces appropriate results even if the number of steps taken (in this case 5) is small.

5.3.2 Testing of Time Discretization

Further testing was carried out to provide an indication of the number of time steps required for the proposed method to result in a stable solution. The calculated longitudinal distribution is, as can be surmised from the form of equations 5.29 and 5.44, initially highly dependent on the number of calculation steps. The results obtained are better as the number of steps increases. To undertake an examination of this effect, results of the Hamdi et al model were not useful, since that model does not provide a solution for a non-uniform cross-sectional velocity distribution. A hypothetical channel shape was therefore established for testing, and appropriate parameters used to estimate velocities in-stream.

Parameters used were:

channel slope = 0.001 m/m
Manning 'n' = 0.035

The results of this calculation are shown in figure 5.7. As shown, a typical parabaloid velocity distribution resulted.

Figure 5.8 shows for a single source at the stream centerline the effect of the number of calculation steps on the instream mass curve. As shown, the solution quickly converges to a stable mass curve.

Figure 5.9 superimposes the cumulative mass curves for the five and six step calculations, and also contains the incremental mass curves, both for the cases of the centerline source and a line source across the width of the channel. As shown, by the fifth and sixth calculation step, the longitudinal concentration distribution has reached a stable form. A less

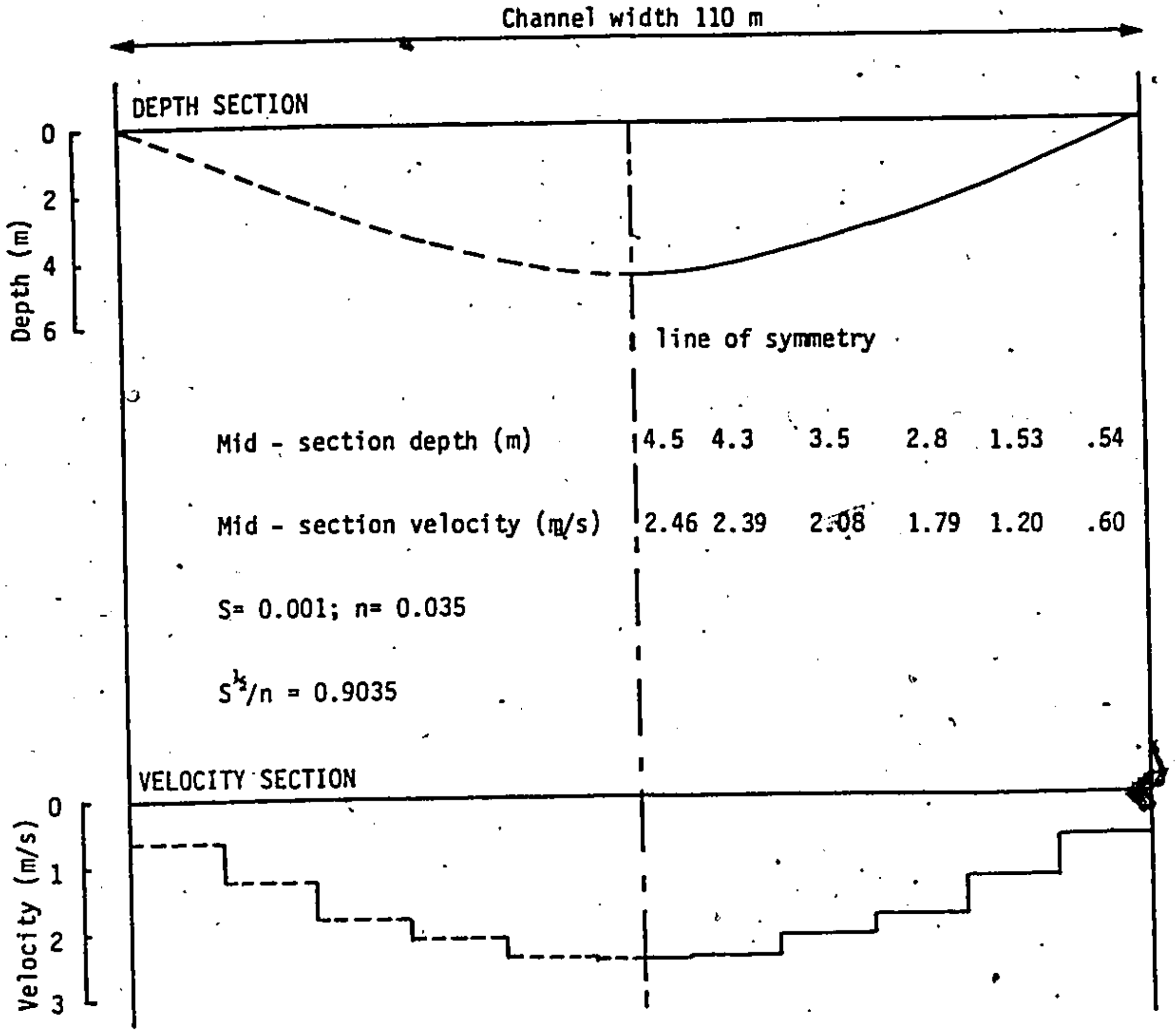


FIGURE 5.7 VELOCITY DISTRIBUTION IN TEST CASE CHANNEL

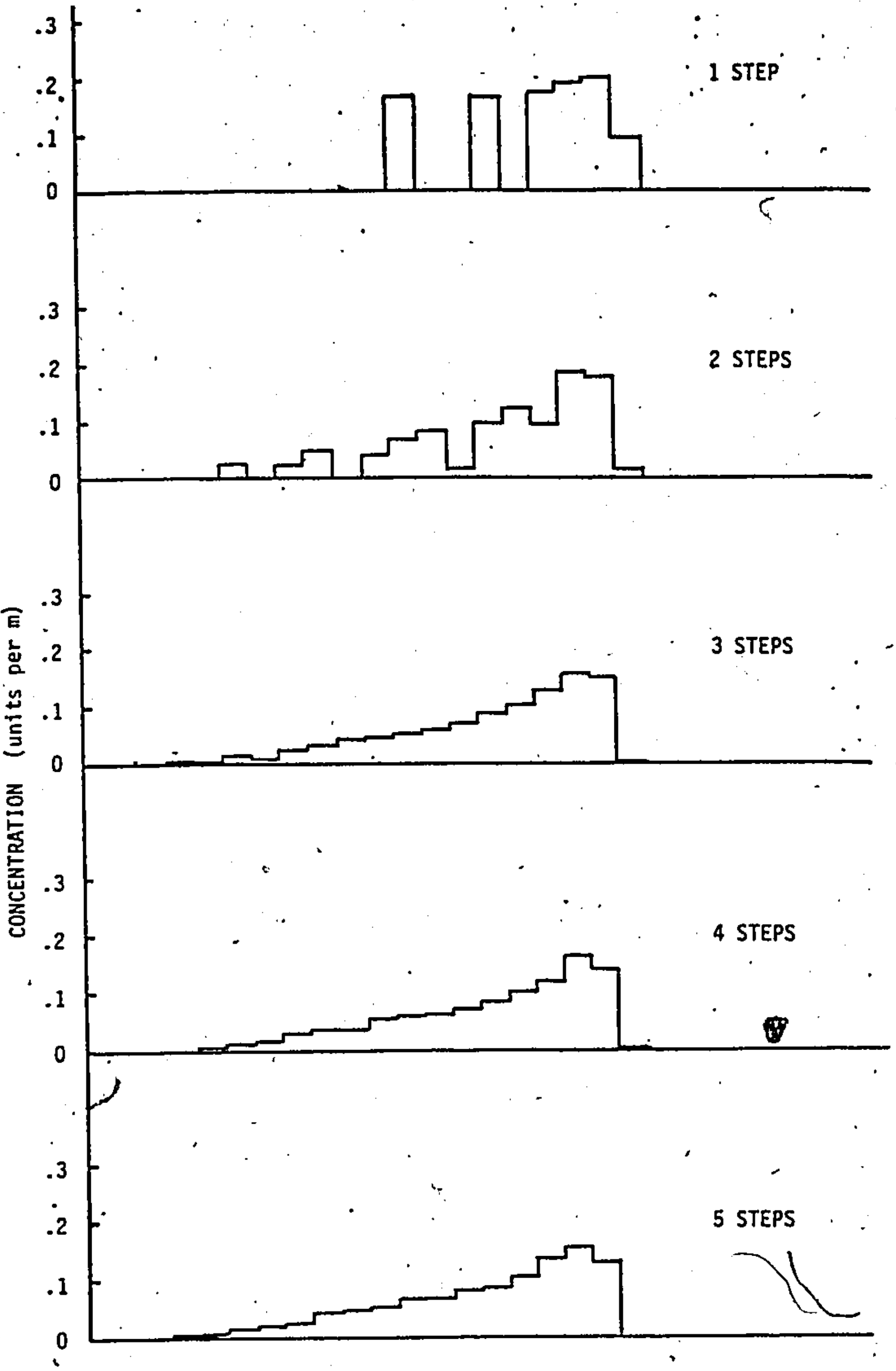


FIGURE 5.8 INFLUENCE OF TIME DISCRETISATION ON RESPONSE CURVE FOR TEST CHANNEL

FIGURE 5.9 COMPARISON OF CUMULATIVE MASS CURVES WHEN 5 and 6 TIME STEPS ARE USED IN SIMULATION

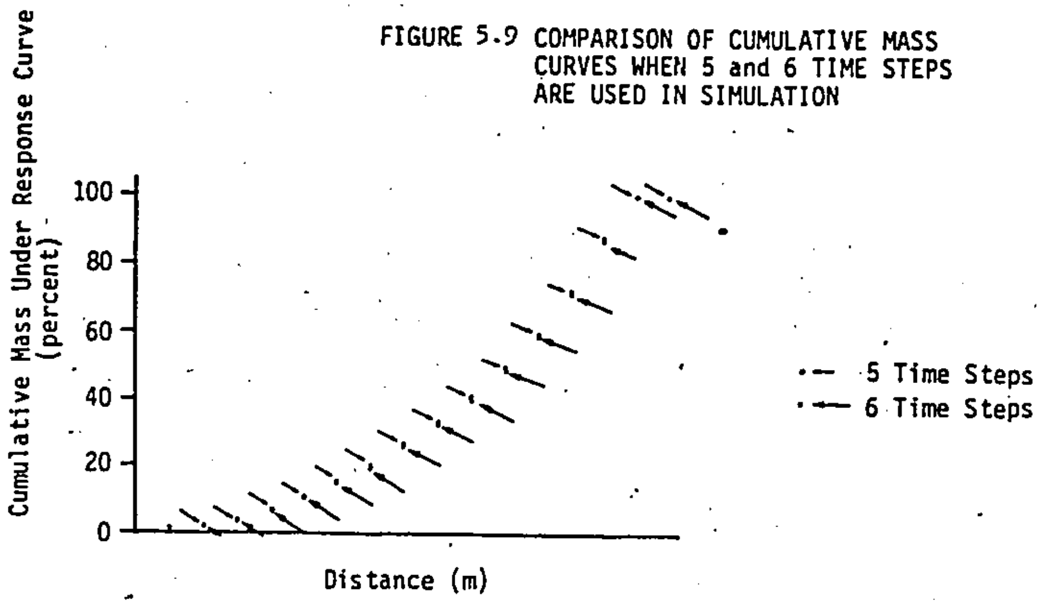
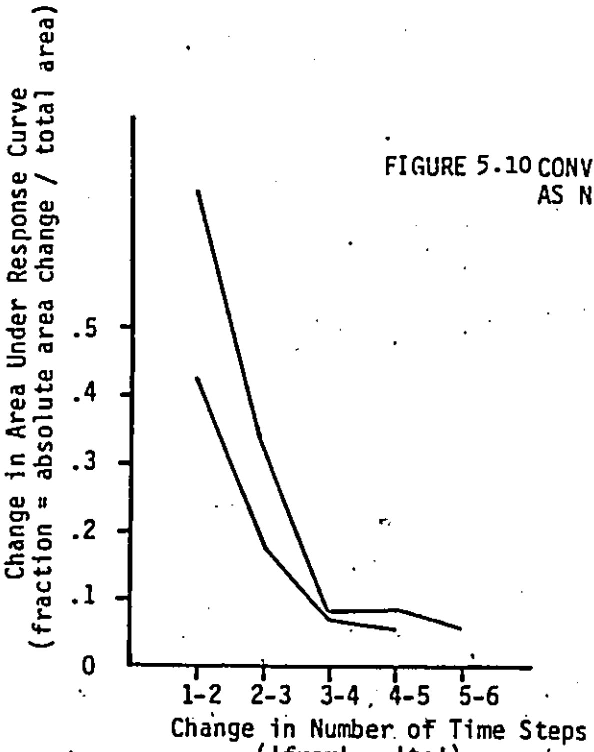


FIGURE 5.10 CONVERGENCE OF RESPONSE FUNCTION AS NUMBER OF TIME STEPS INCREASES



subjective indication of this observation was obtained by calculating total difference between the incremental mass curves in successive steps.

This was defined as:

$$E(k,k+1) = \mathrm{sum}(\mathrm{abs}(Mk(i)-Mk+1(i),n) \qquad (5.58)$$

where Mj(i) is the mass in distance increment 'i' in a calculation made using 'j' steps,

and E(k,k+1) is a measure of the difference between simulated results using 'k' calculation steps and using 'k+1' calculation steps.

Figure 5.10 plots the result for the centerline and line source discharges. As shown, this calculation indicates a pattern that the change in the curves has approached a minimum.

The number of steps could be carried out further, but as the change in both cases has reached a small magnitude (about 0.05 out of a mass of 1.00), and because the plotted curves are close, the calculations were terminated. This calculation does show a weakness in the method developed here, which lies in the number of calculations which are necessary. The last of the above steps took approximately 28 minutes central processing time on an Amdhal 470, which represents a current commercial value of about $1500. This is because the number of overall iterations to obtain the six step result was eleven to the power six (equation 5.28), and each iteration requires a number of nested loops to complete equation 5.40. A further iteration would increase the time and equivalent cost of the run to about five hours and sixteen thousand dollars, which constituted an unacceptable burden on the computer system.

More important in the present work, it is noted that to stop the runs at the previous (fifth) step would have produced essentially the same result at a much smaller cost. The method therefore appears to be suitable for practical use from a cost perspective, even though it is evident that obtaining high accuracy with small time and distance steps can only be accomplished with a massive cost burden.

5.3.3 Comparison to the Simons and Lam Model

To establish how the proposed method compares to more traditional two dimensional forms, the method was tested against a model developed by Simons et al (1982) at the National Water Research Institute. Their model was originally developed for use in a lake environment, and is capable of solving equation 5.3 for an arbitrary non-uniform non-steady two dimensional system. Time differencing in the model is explicit, but spatial differencing is implicit; at the discretion of the user, either an upstream or a central difference scheme can be applied. The model uses a constant square grid pattern, described by the authors as a single Richardson lattice with a staggered distribution of variables.

To provide a common basis for comparison, the Simons and Lam model was set up for the same problem as was described in section 5.3.2. It is interesting to note that to achieve a stable solution, the grid space had to be limited to 5 meters in the lateral and longitudinal directions, compared to the 10 meter lateral stream section and 123 m longitudinal step used in the proposed method. Further, the time step in the Simons and Lam model had to be limited to 2 seconds for the model to converge to a solution, compared to the 432 second time step used in the proposed method.

Figure 5.11 compares the results of the two models. It is clear that in this test case, the results of the two approaches are essentially the same, and it is concluded that the proposed method provides a valid alternative to traditional routing approaches.

It is also noted that the original objective of economy appears to be met by the proposed approach. Compared to a simulation time of 2.5 minutes for the proposed

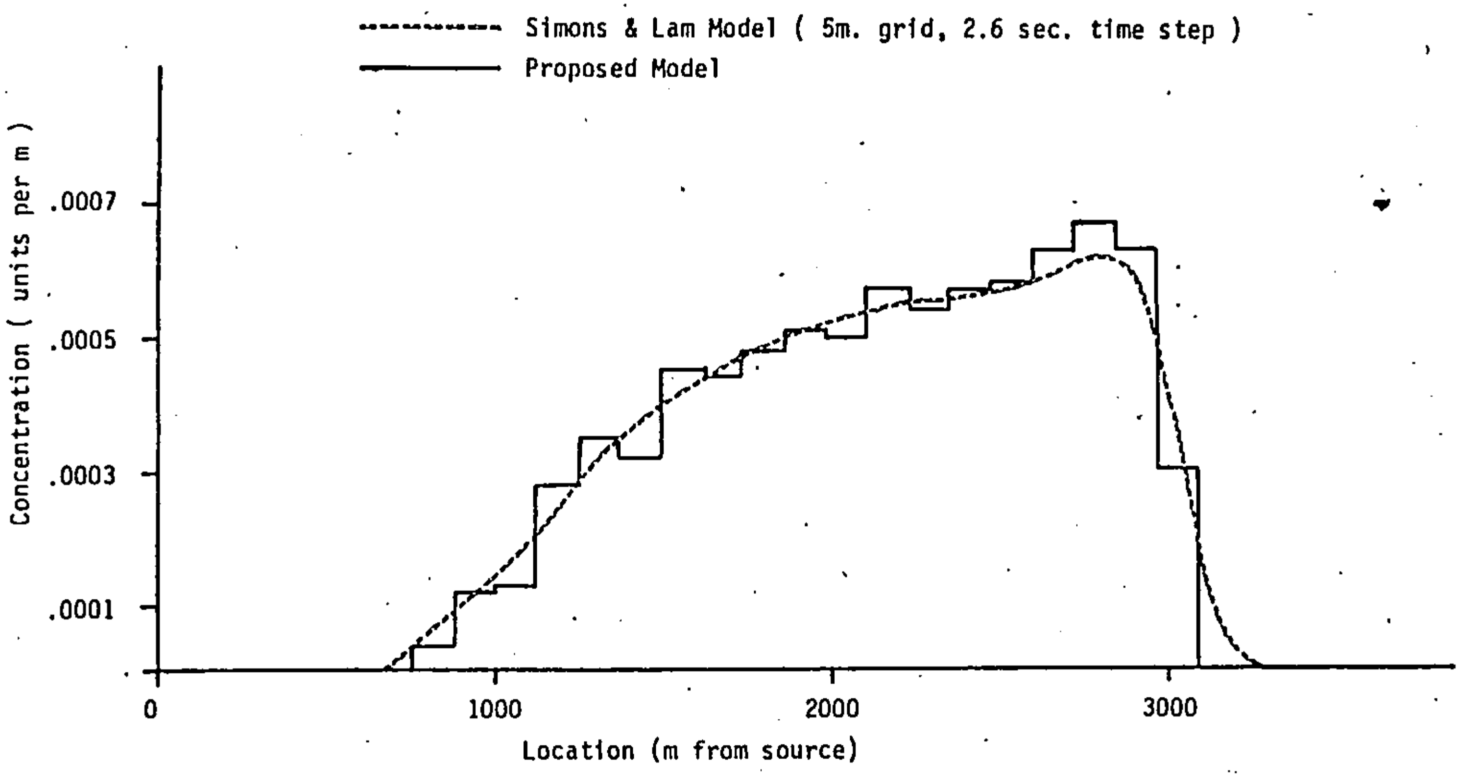


FIGURE 5.11 LONGITUDINAL CONCENTRATION DISTRIBUTION - COMPARISON OF RESULTS

method, the Simons and Lam model required 60 minutes, or 24 times as much computer time and hence cost to produce essentially the same result.

5.3.4 Comparison to One-Dimensional Approaches

Finally, to provide some indication of how the method would compare to traditional one-dimensional approaches in the test case reach, the method described by Fischer et al (1979) was used to estimate the longitudinal concentration/distance curve for a slug injected as a line source in the hypothetical test channel. Fischer's method uses a numerical approximation of the channel geometery to solve the relation:

$$Ex = -1/A^* \int u'd \int 1/Ey/d \cdot \int u'd.dy.dy.dy \qquad (5.59)$$

As pointed out by Fisher, the method provides only an approximation of Ex, however it does provide some basis for comparing results. The calculated result for the test channel puts:

$$Ex(approx.) = 25.45 \text{ sq. m. per sec.}$$

which, based on the selected time and distance step shows a calculated variance of the cloud to be:

$$\begin{aligned} \sigma &= sqrt(2*Ex*t) \qquad (5.22) \\ &= sqrt(2*25.45*3600*0.6) \\ &= 331.6 \text{ m} \end{aligned}$$

over the 0.6 hour test period. The simulated result has its center of mass approximately coincident with the calculated curve, but has a variance of about 565 m, which is significantly higher. This would result in a relatively broad, flat shape inconsistent with the results of the other two models. Equation 5.59 is an approximation only. To create a more realistic comparison of traditional and proposed approaches, an alternative method was used.

The response function generated by the Simons and Lam model was used as a test case. A standard deviation for the response function was estimated by the method of moments to be 576 m and, using equation 5.22, an Ex of 0.41 sq.m. per sec. was calculated. This allowed generation of a one-dimensional response curve with approximately the same spread as predicted by the Simons and Lam model, and therefore a more meaningful basis for comparison.

As shown in figure 5.12, however, there is still a significant difference in results between the traditional one-dimensional method and the modified method. The traditional one-dimensional relationship always must result in a gaussian response curve as a result of its mathematical form. The proposed method tends to a shape which appears to better reflect the influence of the river velocity distribution.

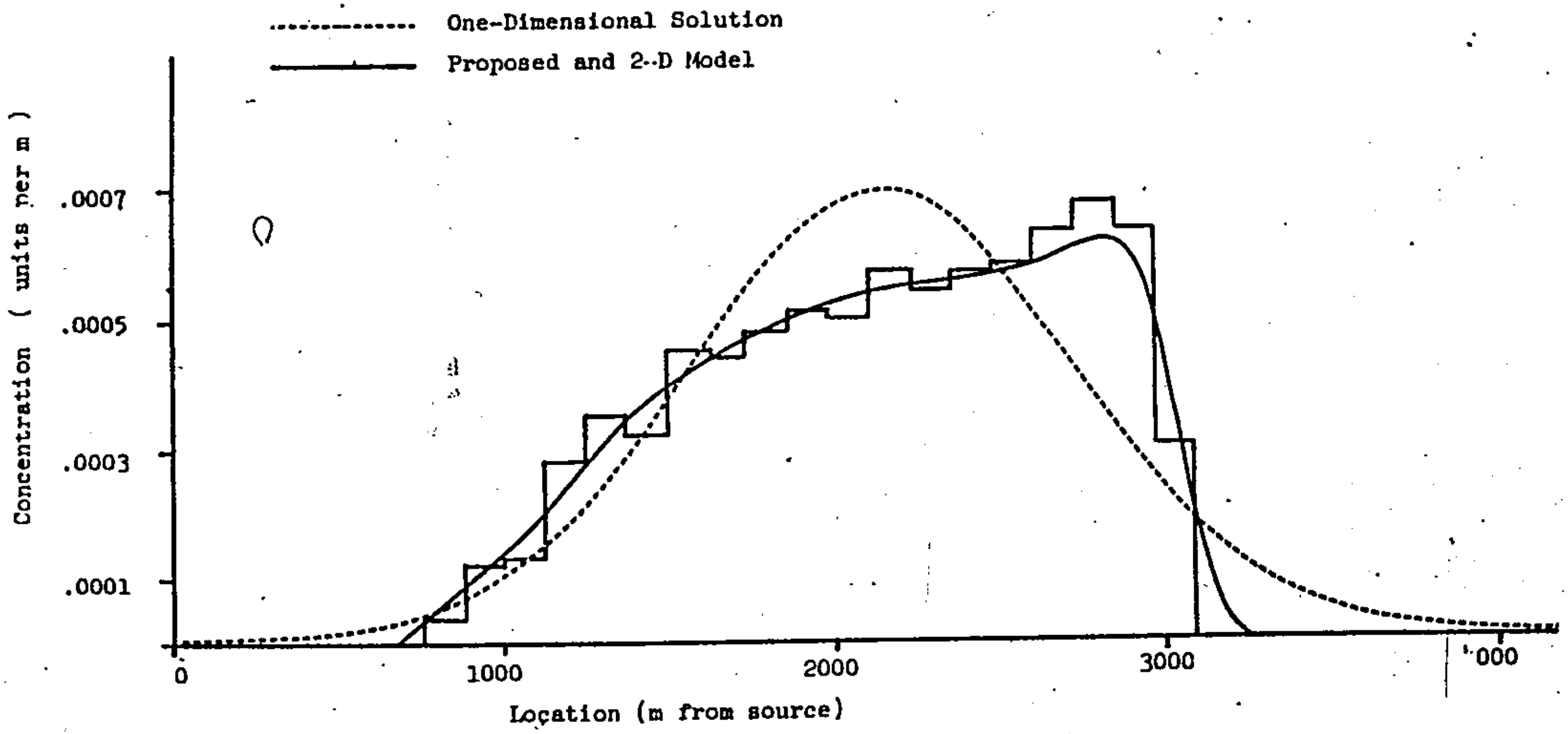


FIGURE 5.12 Comparison of 2-D and Proposed Method With One-Dimensional Solution

5.4 Application of the Proposed Method

To use the proposed model of dispersion in a river routing scheme, it is possible to apply the method directly to a series of input loads, and hence generate an output from the reach. As indicated above, however, a convolution method was selected for use in the proposed planning model, to achieve economy of operation.

The above method for obtaining a response curve is combined with an appropriate convolution algorithm most easily by converting the result into a mass/time form rather than the derived mass/distance result. To do this, the model is run over successive different time spans, and the mass/distance curves are used to interpolate concentration/time results for a selected location.

The response function takes the form of a discrete series:

$$CR(q) = \{C1(q), C2(q), C3(q) \ldots Cn(q)\} \qquad (5.60)$$

where Cj is the concentration at the end of the reach at time step 'j' resulting from a unit load of mass introduced at the top of the reach at the beginning of time step 1.

and q is the average flow rate in

the channel over a time
interval $n*\Delta t$.

Note that the response CR includes an appropriate reduction in concentration for losses to decay.

With the concentration/time curve produced, it remains to perform the convolution. The present version of the model achieves this using the following form:

$$C(t) = sum((Ci(q(i))*M(t-i*\Delta t),n) \tag{5.61}$$

where Ci is an element of CR(q)
and n ranges from 1 to the length of CR.

To determine a flow value for use in equation 5.61, the model keeps a running account of average flow over time intervals $n*\Delta t$:

$$q(k) = sum(Q(k-i),j)/n \tag{5.62}$$

where the range of j is
from 1 to n,
n is the length of CR.
and Q(m) is the flow rate during
interval 'm'

In this form, the response function for each input slug is according to the average flow existing during the response period. Some approximation is inherent here, since unless river flows are constant, at any time the concentration at the bottom end of a reach will be the sum of responses representative of different average instream flows. Further, the response function itself spans a period of time with one average flow approximating the rate during that period. These are facts inherent to some degree in any linear systems model of this type, but do not preclude obtaining useful model results. (Thomann, 1972; Medina 1981; ibid 1982).

5.5 Summary

A method for routing pollutant constituents instream has been proposed. This method is based on fundamental assumptions corresponding to traditional methods, but uses a conceptually different routing scheme. The method has been verified against traditional solutions, with good agreement. It is therefore considered that the method can be used where ever the physical system would justify the assumptions of a corresponding traditional approach.

The proposed method could in principle be used for routing an input load series of any length. In this application, however, the method was used to produce a response function suitable for use in a convolution routing scheme. Selection of a convolution routing scheme in the proposed model has several advantages.

1. The convolution routing takes significantly less

computer time, and hence incurs less cost, than either traditional numerical methods or the proposed method when applied to a lengthy input series.

2. The convolution routing scheme allows flexibility in the model application. By choosing an appropriate response function, the model can recover the results of some traditional one-dimensional models. Alternatively, by employing the response function derived in this research, the model can approach the results of more sophisticated two dimensional models.

It is emphasised also that the incorporation of convolution scheme as a basic routing method in the proposed model does not preclude use of other methods. Either the traditional scheme also developed for the model, or an alternative scheme proposed by the user can be incorporated should the user deem this necessary. As indicated in Chapter 4, the proposed model has been designed so as to make it possible to supplant any of its basic algorithms.

6.0 Runoff Quantity Estimation

6.1 Introduction

As indicated in previous chapters, the runoff quantity portion of the proposed model should be economical and effective in the context of a regional planning study. There are a number of alternatives which have been proposed for stormwater quantity analysis, particularly in urban settings, but there are relatively few which are suitable for use in large rural areas. The popular combination of a Horton loss relation and a kinematic wave catchment routing, for example, is best applied in relatively small urban catchments and is therefore not entirely suitable for the present purpose.

In addition to obtaining a runoff rate relationship which is theoretically sound, the model should be based on an algorithm which is consistent with present practice in stormwater quantity management. This accords with the requirement that it should be possible to reconcile results of the model with other analyses oriented towards water quantity. Ideally, if the choice of runoff quantity analysis is suitable, the proposed model could be used for water quantity analysis in place of alternative separate models. This could achieve an additional economy of effort by eliminating the presently applied redundant process of applying separate analyses to quantity and quality oriented studies.

North American practice in runoff volume estimation is well enough developed that a general consensus exists as to which basic approaches are appropriate for water quantity estimation, but details of application vary according to legislative jurisdiction and local preference. It was therefore decided to emphasise Ontario as a benchmark for present practice in water quantity analysis. This was done because Ontario is of immediate practical importance to the University of Ottawa; it is contended that Ontario practice is at least as well developed as that of any other area of North America; and, Ontario practice is generally consistent with other North American areas.

The overall approach adopted for water volume estimation was to adapt the SCS method to continuous simulation for water volume estimation, and to use a linear convolution scheme for catchment routing. The rationale for and development of these schemes are presented below.

6.2 Overview of Catchment Runoff Estimation

Runoff Volume Estimation

A review of available methods for catchment runoff schemes indicated that the SCS method of volume estimation is the only simulation approach which is

(Wisner et al 1983):

a. generally recognised as acceptable for water quantity analysis;

b. economical and in need of only a limited amount of input data; and

c. suitable for urban and rural areas when properly applied.

This method was therefore adopted as a basis for runoff volume estimation in pervious areas, with the recognition that the method would have to be adapted for use in a continuous simulation model. This adaptation represented the main effort in creating a runoff quantity algorithm for use in the proposed model.

To allow for possible small impervious areas, a volumetric coefficient approach was incorporated in the basin quantity simulation algorithms, as shown in figure 6.1. This aspect is further discussed in the appendices.

Runoff Rate Estimation

A convolution scheme was proposed for catchment routing. This approach is suitable for large areas, is economical, is consistent with the runoff volume estimation approach, and can be applied in both urban

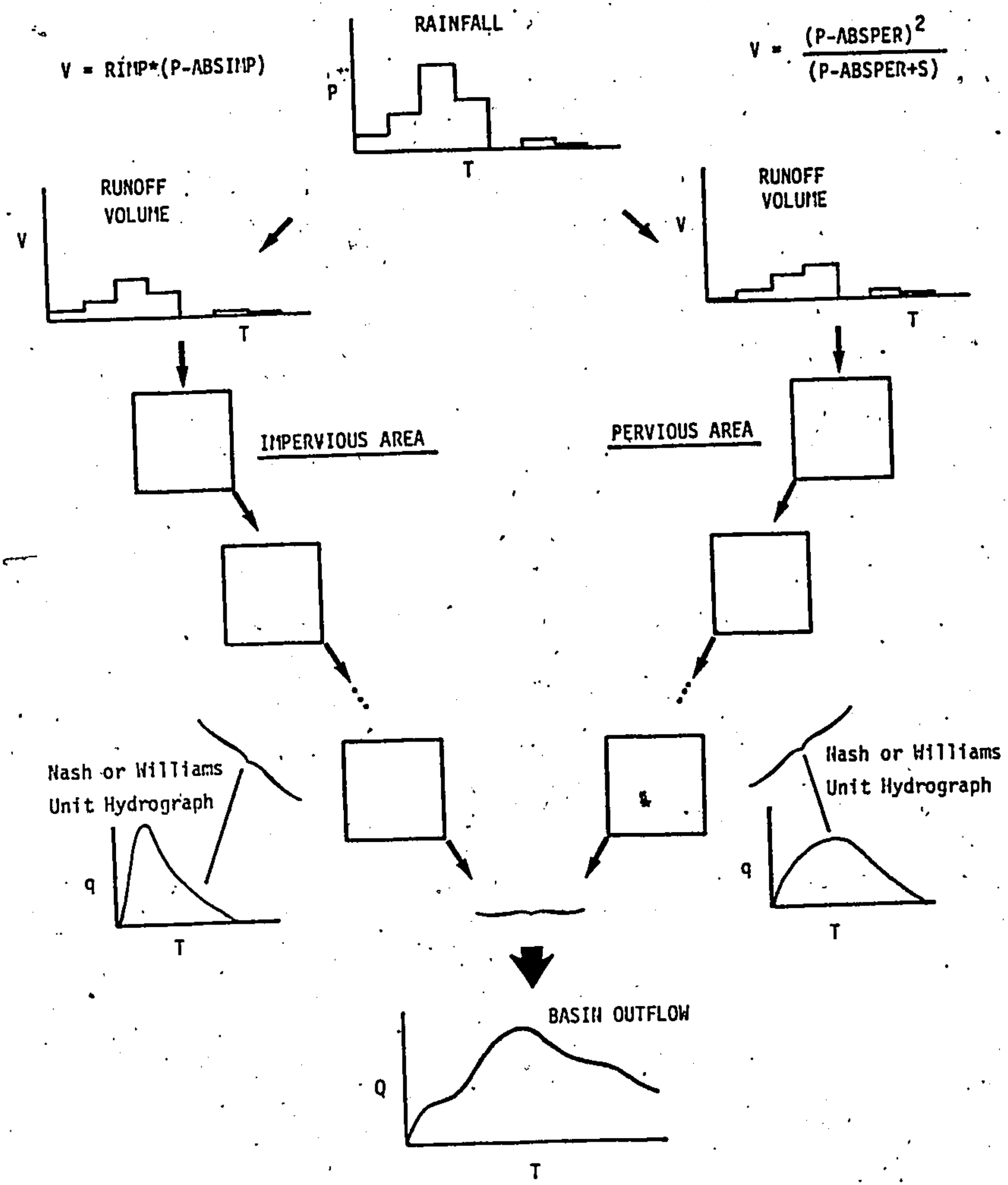


FIGURE 6.1 CONCEPTUAL SKETCH OF BASIN RUNOFF QUANTITY ROUTING

and rural settings. Furthermore, as shown by Wisner et al (1983), an appropriate choice of convolution function can result in rate estimates which are consistent with detailed urban models such as SWMM.

Recent IMPSWM work resulted in a package of separate algorithms for urban or rural basins and includes (i) a sequence of linear reservoirs and (ii) a system of two single linear reservoirs in parallel. The proposed model is consistent with that work, but proposes to consolidate the capabilities of the various IMPSWM algorithms into a single comprehensive algorithm.

Model Overview

Figure 6.1 depicts the conceptual scheme for runoff quantity estimation. As shown, the model splits the catchment into two sections. One section, representative of a small impervious area, calculates runoff volume using a volumetric coefficient approach where precipitation is reduced by an initial abstraction and then further reduced by a coefficient. The second section uses a modified SCS relation to calculate excess runoff. In both cases, the excess precipitation, if any, is routed through a system of single linear reservoirs. The two systems are independent and can be of either the Nash (Nash, 1957) type, or the Williams (Williams et al, 1973) type. As a result, by appropriate choice of reservoir parameters, any of the IMPSWM algorithms can be recovered. This in turn allows the user to incorporate the results of that work and hence to achieve flow rate results which are consistent with those of more detailed stormwater management models such as SWMM.

A more detailed description of the catchment runoff algorithms appears in appendix A. The following section discusses the development and testing of the modified form of the SCS runoff volume relation which is proposed herein.

6.3 Runoff Volume Estimation

Overview

As discussed above, a modification of the SCS relation was tested as a means of generating excess runoff from precipitation. The method used is based on the SCS excess precipitation function but, in contrast to the SCS approach, uses an API (Antecedant Precipitation Index) to update hydrologic loss parameters from one event to the next.

This differs from previous continuous simulation models in that there is no attempt in the method derived here to explicitly simulate a soil water balance. Typically, as in the case of STORM, SWMM, or HSPF, accounting for soil moisture variation has been physically based, at least to the extent that the simulation explicitly simulates the effect of such factors as evaporation or infiltration in order to compute changes in soil moisture. Where an API has been used, it has either been used as a regression parameter applied along with other parameters to estimate loss coefficients directly, or has been used to estimate initial conditions for single event models. The method has not been applied in a continuous simulation model of the type considered here.

In contrast, in the proposed runoff model, the API is calculated by the model continuously, and is used to update loss parameters for the selected runoff equation. This approach could in principle be applied to any loss equation, including for example the Green-Ampt loss equation or Horton's equation used by SWMM, or the SCS equation used by STORM. Such an application could be achieved by providing a regression equation of some type which would allow computation of parameters for the loss equation directly from the known value of the API.

In this work it was decided to adopt the IMPSWM variation of the SCS method as the fundamental loss equation, and test the use of this relation in conjunction with an API for continuous simulation. This method was selected because:

1. It is generally accepted for use in water quantity analysis in Ontario by regulatory agencies. This suggests that a continuous model derived from this loss equation will meet the requirement stated above, namely that it should be acceptable for and consistent with current approaches to flood control analysis.

2. The method is computationally less demanding than some alternatives, such as those used by the Stanford model or SWMM. This is consistent with the need for economy and simplicity in application described above.

3. There is a large body of successful experience in Ontario with the SCS method in various forms used in water quantity planning studies, including the IMPSWM variation from which this model is derived. This suggests that application of such a method is appropriate in Ontario. It also suggests that existing experience with the SCS or related methods can be applied in applications of the new method; results of prior studies, for example, should be reconcilable with results from the proposed model in the same basin.

Model Development

The SCS equation as applied in the SCS method of direct runoff determination takes the following form:

$$Q = (P - .2S)**2/(P + .8S) \tag{6.1}$$

where S is the maximum potential
abstraction,
P is the event total
precipitation, and
Q is the event total runoff.

Although the SCS method was derived based on event total volumes, it is assumed that applying a time history of cumulative total precipitation to equation 6.1 will result in a time history of cumulative runoff volume for an event. Incremental runoff volume can therefore be determined by differencing.

Equation 6.1 was based on two primary assumptions. The first was the empirically supported observation that the ratio of potential to maximum loss was equal to the ratio of actual to potential runoff, which led to:

$$Q = (P - Ia)**2/(P - Ia + S) \tag{6.2}$$

The second major assumption was that the initial abstraction Ia has an average value of :

$$Ia = 0.2S \tag{6.3}$$

Substitution of 6.3 into 6.2 produces equation 6.1. In practice, it was found convenient to apply a linearising function of the form:

$$CN = 1000/(S + 10) \qquad (6.4)$$

(for S in inches)

to produce a parameter CN, the curve number, which represented the soil condition. Adjustment of the curve number was possible, to account for antecedent moisture conditions in a limited way, by introducing the concept of an antecedent moisture condition (AMC) class. There were three AMC classes, I, II and III representing very dry, average, and wet conditions respectively. The curve number was tabulated in a form that made it possible to switch from one AMC condition to another by using appropriate alternative CN values.

The method has been applied in the above form in Canada and the United States for some time, and various attempts have been made to improve its performance. Roesner et al used equation 6.2 in their model and attempted to vary the parameter S according to physical processes. Evaporation and deep percolation were used to increase S, and infiltration to decrease S. In that model, therefore, S is seen essentially as an available soil storage volume. This approach has been used successfully, but requires lengthy calibration of the functions relating evaporation and percolation to the change in S.

More recently, Wisner et al(1983) in the course of a program of model development at the University of Ottawa, consolidated previous investigations of the SCS method and outlined some general weaknesses. These included recognition of a tendancy for the initial abstraction estimate of 0.2S to be too high, and a lack of adequate responsiveness to antecedent moisture conditions. They proposed that a rationally derived initial abstraction be used in place of 0.2S, which essentially means that equation 6.2 be applied. They also investigated use of an antecedent precipitation index (API) as a means of varying CN to reflect antecedent moisture conditions for use in single event simulations. This approach, discussed further below, was found to significantly improve the performance of the model in application to analysis of discrete events.

The API has been defined in a number of ways, but generally takes a recursive form of the type:

$$API(i) = KAPI*API(i-1) + P(i) \qquad (6.5)$$

where KAPI is a constant,
typically ranging between 0.8 and 0.95 per day,
and P(i) is the precipitation in the time period (i-1) to (i)

It can be seen from equation 6.5 that the API is essentially an indicator of the past precipitation history, and that the constant KAPI is a parameter indicating the memory of the system.

A similar definition is reported by Bruce and Clarke (1966) who define an API in the form:

API(i) = K*Pi + K*K*P(i-1) + K**3*P(i-2) ...   (6.6)

This relation produces a result that can be shown to be a constant multiple of the API produced by equation 6.5. In their definition, the API is applied in a manner similar to that attributed to Kohler et al (1951). The API is calculated and used in conjunction with graphical curves which provide runoff volume as a function of precipitation and season. This example illustrates one of the main ways in which the API has been applied, namely as an independent variable in a regression approach to runoff estimation.

A second basic approach to application of API is to use it in conjunction with infiltration equations as an aid to estimating parameters for those equations. This arises because a variety of infiltration equations have been developed (Kostiakov, 1932; Horton, 1940; Philip, 1957; Holtan, 1961) which are based to some degree on physical oncepts but which have empirical parameters. It is possible to relate those parameters to an API. Doing this makes the infiltration equation responsive to initial moisture conditions without need to provide a physical description of variation of the parameters.

Foroud et al (1981) used this approach in conjunction with a loss equation of the type:

$$f(t) = a*\exp(-b*t) + fc \qquad (6.7)$$

where f(t) is the loss rate under
excess rainfall conditions,
fc is a minimum loss rate,
and 'a' and 'b' are parameters.

Equation 6.7 is essentially the Horton (1940) model expressed in a slightly different form for convenience. Foroud et al (1981) successfully related 'a' and 'b' to the API (although it is not clear from their publication precisely how API was defined) as follows:

$$a = 19.1*\exp(-1.5*API) + .43 \qquad (6.8a)$$

$$b = .61*(\exp(.38*API)-\exp(-.33*API)/ \qquad (6.8b)$$
$$(\exp(.38*API)+\exp(-.33*API))$$

They concluded that the Horton equation parameters can be expressed as a function of an antecedent precipitation index.

Intermediate between the two above approaches is a recent work by Harms (1983), who used a model attributed to Anderl:

$$Q = (P - Ia)*Phi - Phi/a*(1-exp(\_a*(P-Ia))) \qquad (6.9)$$

where Phi is an empirical volumetric loss coefficient,

in conjunction with a modified form of the API, defined as:

$$APIe = API* + (100 - API*)*.01.Eb \qquad (6.10a)$$

where Eb is the percent urban cover in the basin,

with

$$API* = API + (100 - API)*P/25 \qquad (6.10b)$$

Harms further defined a 'week number' WN calculated in the same form as equation 6.10, and provided regression relations for the parameters in equation 6.9.:

$$Ia = f1(APIe,Eb,Ew,WN) \qquad (6.11a)$$

$$Phi = f2(P,Eb) \qquad (6.11b)$$

$$a = f3(APIe,WN,P) \qquad (6.11c)$$

where Ew is the percent rural cover in the basin,

with equations 6.11 a, b, and c, all empirical expressions. Using this approach, Harms achieved excellent results in application to observed events.

Of direct interest is recent work by the University of Ottawa IMPSWM program (Wisner et al, 1983); Jobin, 1982) which suggests that for design event analysis, the SCS curve number can be plotted as a function of an API calculated by equation 6.5. Although no direct relation between the two was derived, that analysis indicated that a CN derived from known rainfall and runoff volume (referred to as the CN* to differentiate it from the SCS tabulated values) does show a trend indicating that a functional relation between API and CN is possible. That work also supports the above concept that Ia can be expressed as a function of API, and demonstrates a means of improving the SCS method for use in single event simulation.

It is concluded that the principle of using an API as a regression parameter is well established. The possibility of combining an API with the SCS method for continuous simulation has thus far not been investigated, but in principle, the approach has merit since the form of equation 6.2 suggests that a combination of 6.2 and 6.5 or 6.6 will result in a relation of the form:

$$Q = f4(CN, P, Ia) \tag{6.12a}$$

with, according to standard practice,

$$CN = f5(\text{land use, soil type}) \tag{6.12b}$$

Also, as demonstrated in the IMPSWM results, there may be a functional relation:

$$CN^* = f6(API, CN) \tag{6.12c}$$

as well as

$$Ia^* = f7(API, Ia) \tag{6.12d}$$

Combining the above suggests that an expression of the form:

$$Q = f8(P, \text{land use}, \text{soil type}, API, Ia) \tag{6.12e}$$

can be expected. This accords with previous research in this area as described above.

The problem faced in this research is therefore primarily one of determining whether the SCS method can be rendered suitable for continuous simulation by relating the loss parameter to an API.

In this work, the approach adopted was to drop the parameter CN entirely from further consideration, and to operate directly on S. This was done because CN is a single valued function of S with no inherent meaning and as such was considered to represent an unnecessary extra computational step.

With CN eliminated, it is necessary to relate the basic loss parameter S to the API. Regarding the basic equation 6.2 as a consequence of empirical observation, this becomes a problem of curve fitting. The conditions which are necessary for the function relating these two are:

(1) S must be a single-valued function of API. This reflects the need for a unique value of direct runoff for any given combination of precipitation and soil moisture.

(2) The function must be at least piecewise continuous over the range of API. Since API as defined here is a continuous function over its range, this is required to provide continuous results from the model.

3) $dS/dAPI < 0$ (6.13)

over the range of API. The loss parameter S should increase as the API decreases. This inverse relationship is required to ensure that losses increase with dryness.

In addition to the above basic requirements, the IMPSWM experience with the CN * approach suggests that change in S with API will be least at high API, since the CN curve approaches the maximum value before API reaches its maximum range. This implies the desirability of a function where:

(4) d/dAPI(d/API(S)) > 0 (6.14)

which renders the function concave up and results in dS/dAPI less as API increases.

A plot of S* vs API (Fig. 6.2) suggests that an inverse or exponentially decreasing function of API would be appropriate; both of these would also meet the above conditions. Initially, a function of each type was tested. One function was:

$$S^* = 1/(k1 + k2 * API) \qquad (6.15)$$

which is an inverse linear function asymptotic to the x axis (S*=0) and, if k1<0 or k2<0, to API=k1/k2. The second function took the form:

$$S^* = k1 + k2 * \exp(k3 * API) \qquad (6.16)$$

which, since k3 must be less than zero, is an exponential form asymptotic to S*=k2 as API approaches infinity, and intersecting the Y-axis (API=0) at k1.

Both relationships were fitted to the Seymaz data as shown in figure 6.3. As shown, both curves have a reasonable fit to the preliminary data. However, equation 6.16 appears to be preferable for practical purposes. The best fit for equation 6.17, for example, can lead to infinite and negative values for S* if the vertical asymptote is to the right of the y-axis as in this example. Also, the line approaches zero along the

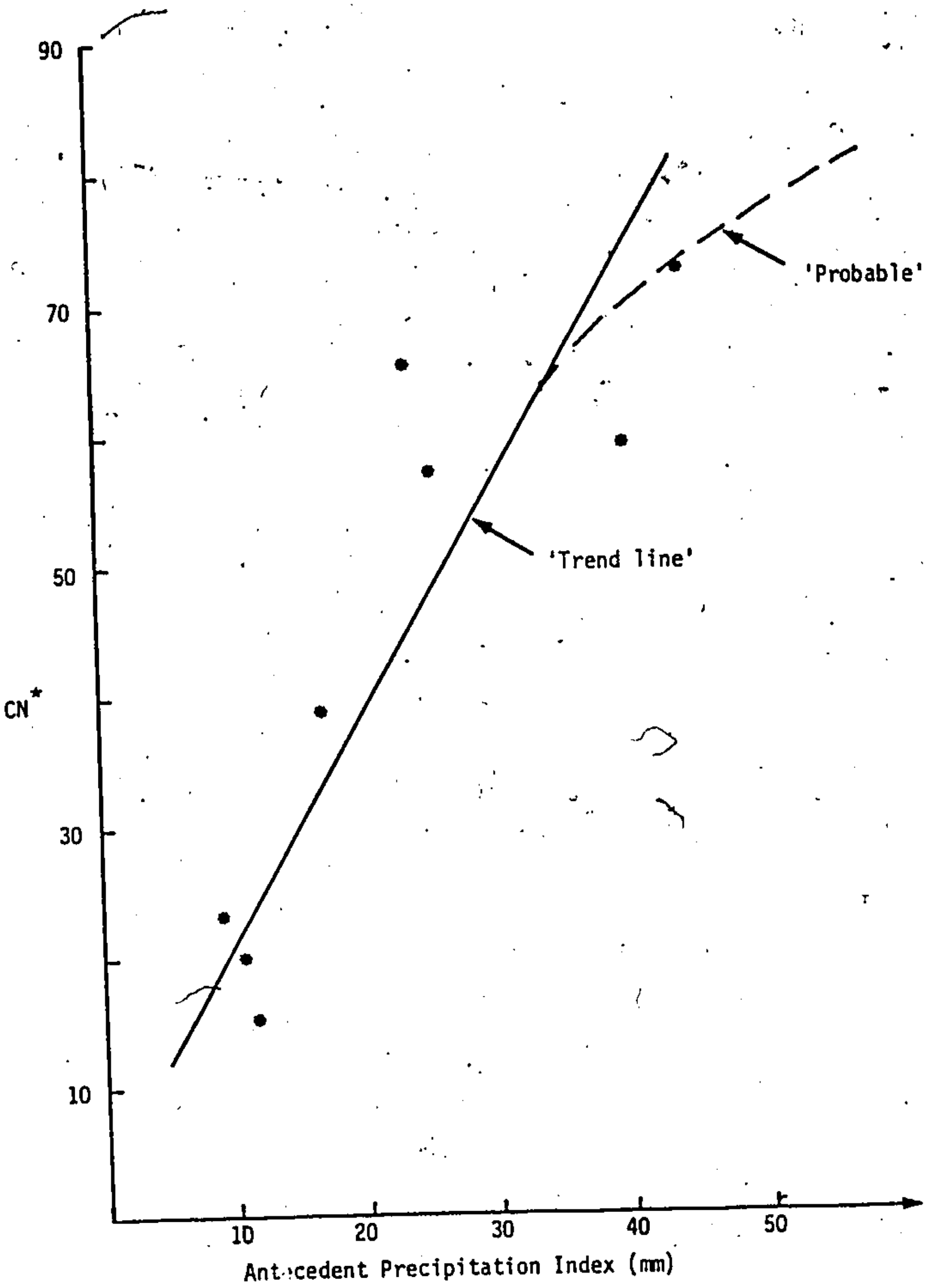


FIGURE 6.2 CN* vs API AT SEYMAZ (After Jobin, 1979)

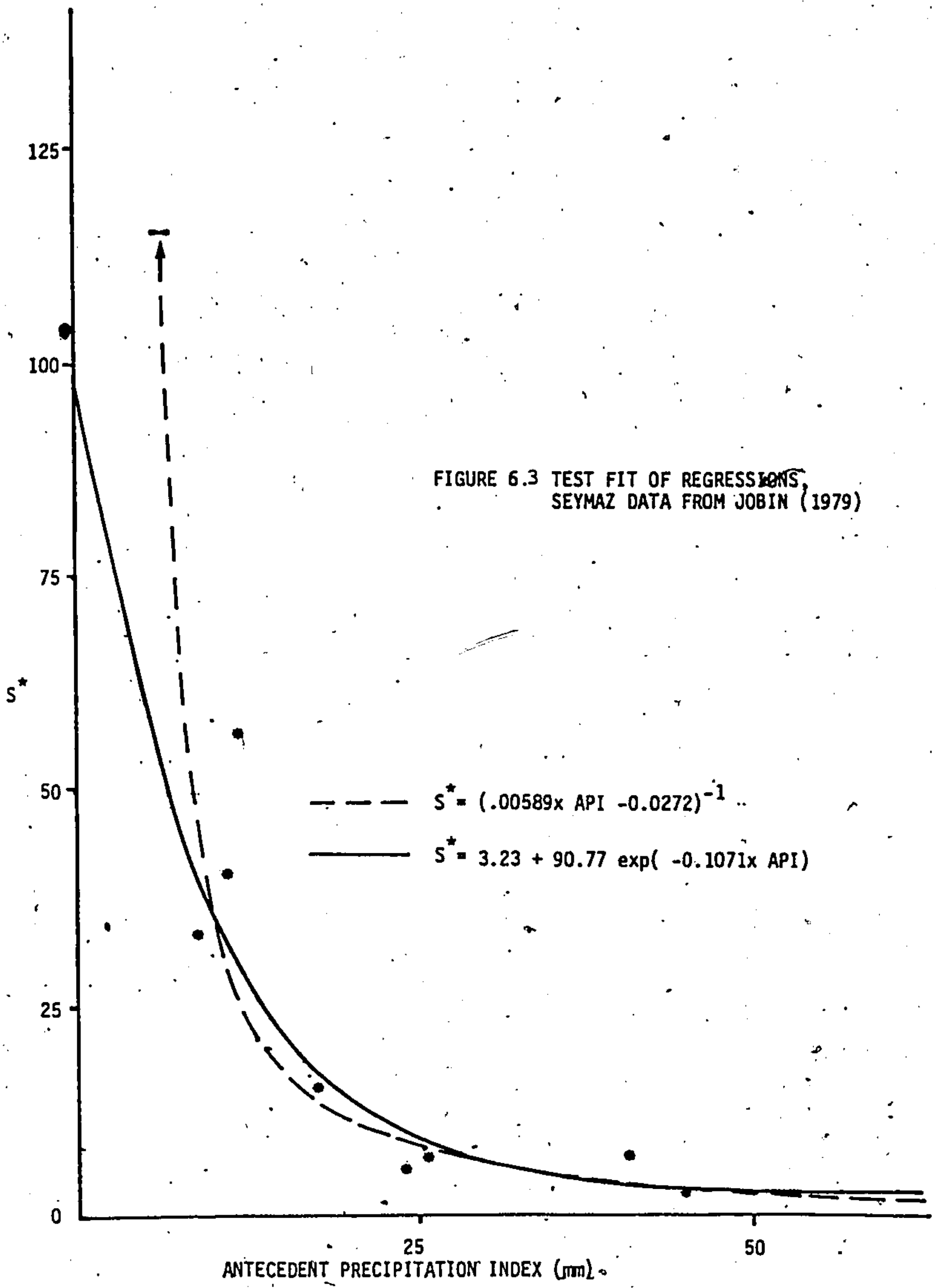


FIGURE 6.3 TEST FIT OF REGRESSIONS, SEYMAZ DATA FROM JOBIN (1979)

x-axis which implies a 100% runoff condition rather than, as would be desirable, some limiting value. There are modifications of 6.15 which could be used, however equation 6.16 has the apparent added advantage of a direct interpretation of basic parameters.

In this relation, k1 is the minimum value of S*, while k2 plus k2 is the maximum. These can potentially be understood readily in terms of the original AMC III and AMC I conditions, or in terms of observations under very dry or very wet conditions if available. In that event, the remaining parameter, k3 becomes a calibration parameter and the model is essentially reduced to a one parameter model.

For convenience, and following above concepts, equation 6.16 was applied in the form:

$$S^* = Smin + (Smax-Smin)*\exp(-Sk*API) \qquad (6.16)$$

In addition to equation 6.16, a coefficient approach was introduced to the model, so that:

$$Qt = Qi + Qp \qquad (6.17a)$$

where Qt is the total direct runoff volume, composed of
Qi, the impervious area contribution,
and Qp, the pervious area contribution

with

$$Qi = (P-Iai)*Cr \qquad (6.17b)$$

where Iai is the impervious area
initial abstraction,
and Cr is the pervious area
runoff coefficient.

in which

$$Iai(t) = \begin{cases} Iai(t-1) - Pi & \Big| \; Pi > 0 \\ Iai(t-1) + Ei & \Big| \; Pi = 0 \end{cases} \qquad (6.17c)$$

where Ei is evaporation

Equation 6.17b and 6.17c have been applied for use in impervious areas by Roesner et al and others, and is analogous to the first term on the right hand side of equation 6.9. The final component in equation 6.17 is the pervious area runoff, restated here as:

$$Qp = (P-Iap)**2/(P-Iap+S*) \qquad (6.17d)$$

where Iap is the pervious area
initial abstraction,
and S* is as described in 6.16.

Equation 6.17 provides a complete basis for estimation of direct runoff, and is the relation adopted for use in the proposed model.

6.4 Summary

The proposed model includes an alternative method of runoff volume estimation. The method, a derivation of the SCS approach in pervious areas together with a volumetric runoff coefficient in impervious areas, is consistent with present practice in runoff volume simulation. However, it is fundamentally different in concept from existing approaches, in that it uses an API concept to provide a continuous update of the SCS loss parameter.

It was therefore considered necessary to test the algorithm against observed data, to validate the model in at least some physical catchments of the type for which the overall model is intended. It was also considered necessary to test the model against an existing model, to establish whether the proposed algorithm represents a positive alternative to other approaches. As indicated in previous chapters, the approach proposed by Roesner et al represents the most successful alternative for catchment quality/ quantity analysis to date, and this was chosen as the primary 'benchmark' model. Testing of the proposed runoff algorithm against real data, and against other models, is described in chapter 7.

7.0 Application of Model

7.1 Introduction

This chapter provides the results of general testing of the model and its runoff quality/ quantity algorithms. (Some additional testing, carried out on routines not described in previous chapters, is described in Appendices A through C.)

Testing was carried out in two main areas:

a.) The model was tested in simulation against real data and against other models in several different types of catchments. This was done (i) to determine model how well the model was able to reproduce observed runoff quality/quantity data; and (ii) to establish model performance in comparison to alternatives. The comparison to other models considered results from STORM and HSPF since these were the two existing models identified in chapter 1 to most closely meet requirements of a regional planning model.

b.) As a more general test application of the model, a hypothetical test watershed was established and assessed. This second assessment (i) provided a comprehensive demonstration of model function and (ii) examined the consequence of using a single valued criterion for regulation of water quality. Although hypothetical, the test case was established to be consistent with conditions in the Rideau river watershed above Ottawa; this gave the example practical relevance.

The catchment areas for which measurements were available were the Chesterton Drive, Alta Vista, Wixon Creek, and Sawmill Creek catchments. Physical catchment parameters for the Wixon creek catchment were provided by the IMPSWM program (Wisner et al, 1983). The other catchment parameters were taken from recent engineering reports (Rideau River Stormwater Management Study, 1981; ibid, 1981; City of Ottawa, 1982). Table 7.1 provides summary data for the catchments; detailed information is available in the cited references.

The catchments provide a wide range of sizes and composition. Essentially, they may be grouped into:

a. small urban catchments (Alta Vista and Chesterton Drive) typical of established residential communities in the Ottawa area;

b. a large pervious rural watershed (Wixon Creek); and

c. a large watershed of mixed composition (Sawmill Creek).

The catchments include a range of conditions generally

TABLE 7.1 CATCHMENTS TESTED IN SIMULATION

NAME	TOTAL AREA (ha)	IMPERVIOUSNESS (%)	PREDOMINANT LAND USE
Alta Vista	178.5	22.4	66% residential 34% open urban park
Chesterton Drive	71.9	28.2	79% residential 21% open urban park
Sawmill Creek	2428	14.0	34% residential 15% commercial/ industrial 51% open land
Wixon	1016	0.0	60% pasture/ scrubland 25% cropland 15% wooded

consistent with those encountered in the drainage area planning studies cited in chapter 1 (Town of Ajax, City of Ottawa, and Town of Oakville). They are also typical of catchments in the Rideau River drainage area below Manotick (Gore and Storrie et al, 1982). As such, they may be considered to be generally representative of catchments in drainage areas for which the proposed model is intended.

7.2 Comparative Calibration to Ottawa ~~Catchment~~ Data (Small Urban Catchments)

Rainfall and runoff data was measured for a limited number of events in the Chesterton and Alta Vista catchments in 1981 as a part of the Rideau River Stormwater Management Study (1981). Results were available from a STORM model simulation of the catchments in that study. These data were used to (i) provide an initial test of the performance of equation 6.17, in the context of an residential urban area, and (ii) provide a comparison of the proposed relation with STORM model results for the same area.

Analysis

Simulation and comparison in these catchments were facilitated since the previous STORM analysis (Gore and Storrie et al, 1981) had established depths of runoff and rainfall for the measured events in the two areas.

Figure 7.1, which is a listing of a computer output generated by a statistical analysis program used to assess the data, contains this information.

The analysis was carried out as follows:

(1) The impervious area runoff depth was estimated from rainfall depth using the coefficient approach selected for use in the proposed model (see Appendix A).

(2) Pervious area runoff detph was estimated using the proposed runoff volume algorithm. To determine the API for use in this process, precipitation data from the nearby Ottawa International Airport gauge was used, and a literature value of KAPI (see Appendix A) equal to 0.9 (Bruce and Clarke, 1966; Wisner et al, 1983) was applied. Figure 7.1 contains the calculated API values used in this analysis.

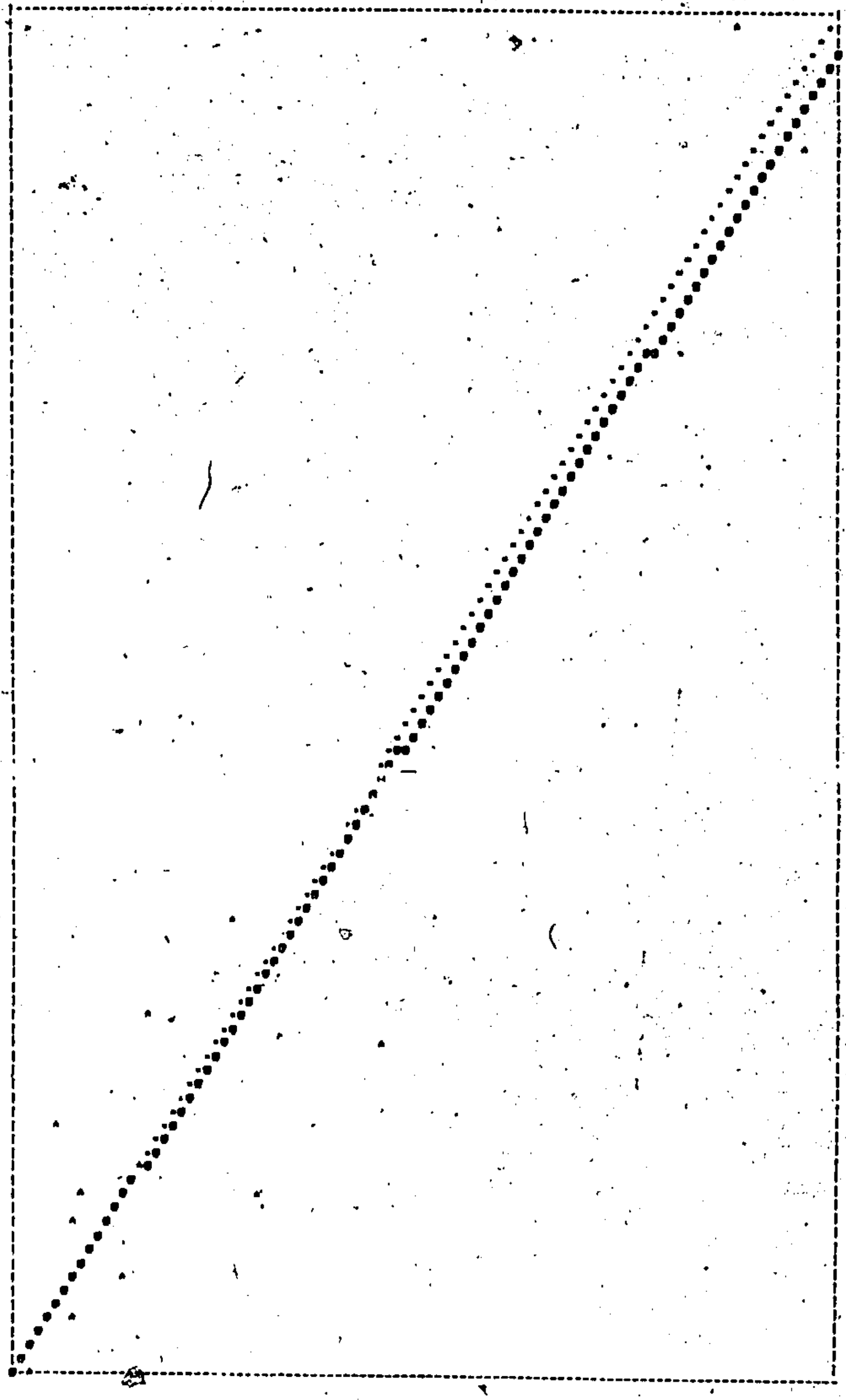
(3) Estimated total runoff volume for the available events was compared to the observed volume, and parameters were improved by successive approximation. The statistical routine which generated figure 7.1 was used to determine when the 'best' fit was obtained, with the criterion for a best estimate of parameters being a minimum root mean square (RMS) error in runoff depth estimation subject to an average error of zero in runoff estimation.

Figure 7.1 Analysis of
Ottawa Catchment
Data

```
OTTAWA CATCHMENT DATA SUMMARY OUTPUT:
COMPARISON OF CALCULATED AND OBSERVED FLOW VOLUMES

CATCHMENT     1:-  ALTA VISTA DRIVE

   RUNOFF PARAMETERS:

      IMPERVIOUS ABST. (MM):         1.500
      IMPERVIOUS VOL. COEFF.:        0.900
      PERVIOUS ABST. (MM):           3.000

      SMIN (MM):                     4.000
      SMAX (MM):                   500.000
      SK (MM) :                      0.110

   OBSERVED AND CALCULATED RUNOFF:

---- OBSERVED ----        --- ESTIMATED ---          API         S*       TOTAL
  RAIN       RUNOFF       -   RUNOFF (MM)   -        (MM)        (MM)     (MM)
  (MM)       (MM)         IMPERV.     PERV.


0.1300E+02  0.2900E+01  0.1035E+02  0.2607E+01  0.2740E+02  0.2835E+02  0.4342E+01
0.2020E+02  0.1470E+02  0.1683E+02  0.1301E+02  0.5250E+02  0.5540E+01  0.1387E+02
0.7800E+01  0.1300E+01  0.5670E+01  0.2511E+01  0.6530E+02  0.4377E+01  0.3218E+01
0.5300E+01  0.1600E+01  0.3420E+01  0.3204E+00  0.3530E+02  0.1421E+02  0.1015E+01
0.8000E+01  0.1700E+01  0.5850E+01  0.1362E+01  0.3610E+02  0.1335E+02  0.2368E+01
0.1180E+02  0.4400E+01  0.9270E+01  0.4242E+01  0.4100E+02  0.9455E+01  0.5368E+01
0.3400E+01  0.8000E+00  0.1710E+01  0.2264E-01  0.4750E+02  0.6669E+01  0.4006E+00
0.7100E+01  0.1500E+01  0.5040E+01  0.1358E+01  0.4320E+02  0.8283E+01  0.2182E+01

CATCHMENT     2:-  CHESTERTON DRIVE

   RUNOFF PARAMETERS:

      IMPERVIOUS ABST. (MM):         1.500
      IMPERVIOUS VOL. COEFF.:        0.900
      PERVIOUS ABST. (MM):           3.000

      SMIN (MM):                     4.000
      SMAX (MM):                   500.000
      SK (MM) :                      0.110

   OBSERVED AND CALCULATED RUNOFF:

---- OBSERVED ----        --- ESTIMATED ---          API         S*       TOTAL
  RAIN       RUNOFF       -   RUNOFF (MM)   -        (MM)        (MM)     (MM)
  (MM)       (MM)         IMPERV.     PERV.


0.2220E+02  0.1350E+02  0.1863E+02  0.8755E+01  0.2970E+02  0.2291E+02  0.1528E+02
0.7600E+01  0.7100E+01  0.5490E+01  0.1493E+01  0.4080E+02  0.9577E+01  0.4125E+01
0.5700E+01  0.2800E+01  0.3780E+01  0.5580E+00  0.3960E+02  0.1036E+02  0.2688E+01
0.5500E+01  0.4900E+01  0.3600E+01  0.3251E+00  0.3330E+02  0.2673E+02  0.2490E+01
0.3900E+01  0.2400E+01  0.2160E+01  0.1013E+00  0.4620E+02  0.7113E+01  0.1462E+01


    DATA ANALYSIS:

      BEST FIT LINE (CALCULATED VS OBSERVED FLOW VOLUME):
        SLOPE= 0.967E+00 AND INTERCEPT= 0.893E-01

      SUMMARY STATISTICS:
        NUMBER OF FLOWS:                 13

        AVERAGE OBSERVED FLOW :        0.458E+01
        AVERAGE SIMULATED FLOW:        0.452E+01

        AVERAGE ERROR IN SIMULATION:-0.601E-02
        RMS ERROR IN SIMULATION:       0.145E+01

        AVE RATIO (SIM/OBS):           0.107E+01
        RMS RATIO (1-SIM/OBS):         0.341E+00

        MAX OBSERVED FLOW:             0.147E+02
        MIN OBSERVED FLOW:             0.800E+00

        MAX SIMULATED FLOW:            0.153E+02
        MIN SIMULATED FLOW:            0.401E+00
```

(7.1 cont'd)

```
SIMULATED  VS  OBSERVED FLOW (MM)

- RANGE OF PLOT IS  0.401 TO 15.280 MM. (BOTH SCALES)
- . DENOTES EXACT AGREEMENT
- B DENOTES LINE OF BEST FIT
- * ARE DATA POINTS
```



The final simulation demonstrates a clear agreement between observed and simulated results. A number of statistics are supplied in figure 7.1 which show that the total estimated runoff volume, average error, and best fit line all compare well with the ideal observed figures. The best fit line, for example, has a slope of 0.97, which is close to the ideal of 1.00; the best fit intercept of 0.09 mm, compared to an ideal 0.00 mm, is also quite reasonable compared to the average runoff of 4.6 mm. The plotted results of figure 7.1 also indicate a reasonable agreement between the two.

Comparison to STORM

Although a detailed analysis of the STORM calibration in this catchment is not available, Gore and Storrie et al (1981) report an average error in simulation in these catchments of -0.11 mm (8.3%) for the 10 best of the available 13 events. By comparison, the proposed algorithm achieves an average error of -0.06 mm (7.0%). This is essentially the same result. This is reasonable, since the catchments considered in this analysis are relatively impervious. As shown in figure 7.1, the calculated impervious area contribution contributes the major portion of the calculated total runoff volume, with pervious area contribution in the range of one half to one tenth of the total. The impervious component, which is simulated the same way in both the proposed model and STORM, therefore dominates the simulation; results of the two models in this type of catchment should be similar, and the result obtained here is appropriate.

From the comparative testing in these catchments, it is concluded that:

- The proposed algorithm is at least as effective as the alternative model, STORM, for simulation of small, relatively impervious, urban catchments.

- The proposed algorithm is appropriate for use in small predominantly urban catchments.

7.3 Detailed Calibration to Sawmill Creek Flow Volume Measurements (Large Catchment, Mixed Compostion)

The next test of the method was undertaken with continuous data measured on the Sawmill Creek catchment during 1981 as a part of the Rideau River Stormwater Management Study. This was undertaken in two stages. First, a preliminary analysis was conducted using the parameters from the urban catchment analysis presented in section 7.2. Second, quantity simulation parameters were established by calibration and verification to the observed Sawmill Creek flow data.

Preliminary Sawmill Creek Analysis

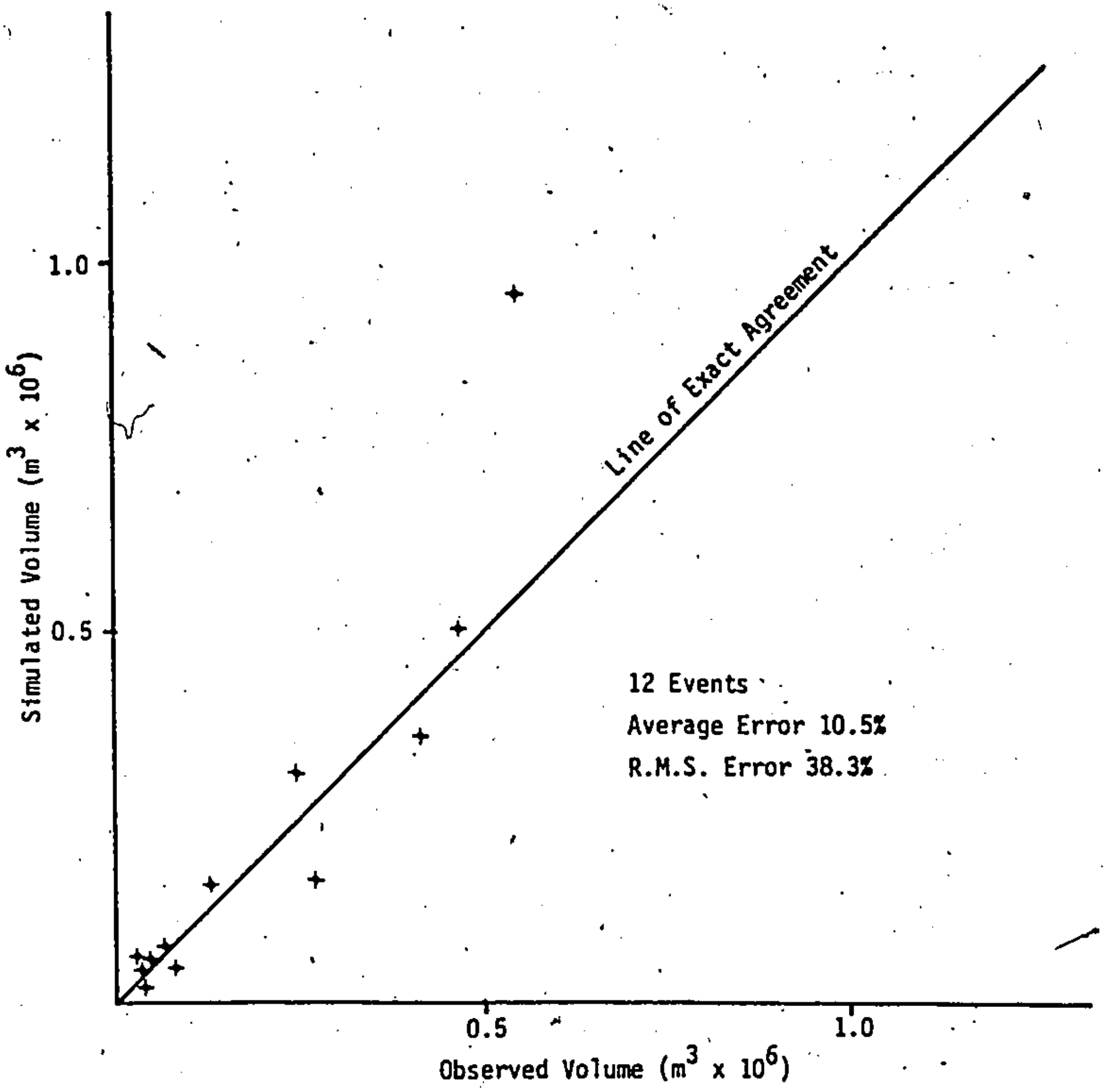
As a first trial, parameters from the Alta Vista catchment were used to estimate runoff volumes in Sawmill Creek. This was considered reasonable because of the physcial proximity of the catchments (Rideau River Stormwater Management Study, 1981). A constant average base flow of 0.142 cubic meters per second (5 cfs) was used in this initial analysis based on observed dry weather values.

The result of this preliminary estimate is as depicted in figure 7.2. The average error and the root mean square error were calculated by the proposed model (CALIBRATE command, described in Appendix D) to be +10.5% and +38.3% respectively.

Detailed Sawmill Creek Analysis

The preliminary runs in the urban and Sawmill Creek catchments were followed by a systematic optimization of the model for Sawmill creek catchment. To do this, the 1981 rainfall/runoff record period was divided approximately in half. Out of the 17 available events, the first 9 provided a calibration set, and the last 8 provided a verification set. Rainfall was taken from point estimates at an Atmospheric Environment Service Gauge (Heatherington Station).

FIGURE 7.2 PRELIMINARY SAWMILL CREEK ANALYSIS



12 Events
Average Error 10.5%
R.M.S. Error 38.3%

The process by which calibration was achieved was:

(1) Periods immediately before and after direct runoff periods were selected, and base flow parameters in the model were adjusted to provide a good representation of base flow.

(2) Direct runoff parameters were adjusted systematically to provide an overall optimum adjustment to wet (runoff) period records.

The criterion for optimization was similar to that used in the urban catchment analysis: RMS error in ratio of simulated to observed runoff volume was minimized subject to maintaining an average error of zero.

The final model parameters obtained, were as shown in table 7.2:

Table 7.2 Sawmill Creek Calibration

Parameter(*)	Significance	Final value
RIMP	impervious area runoff	1.00
ABSIMP	" " "	4.06 mm
SMIN	pervious area runoff	25.4 mm
SMAX	" " "	50800.0 mm
SK	" " "	0.1378 mm -1
ABSPER	" " "	4.32 mm
SLOSK1	base flow	0.00134
SLOSK2	" " "	0.58 mm/s/mm
BASMIN	" " "	0.0 cu m/s

(Appendix A defines the parameter notation used in this table)

The calibration parameters in Table 7.2 were then used for simulation of the verification events. The statistics shown in table 7.3, generated by the model, were the result of that simulation.

Table 7.3 Results of Sawmill Creek Calibration

Case	Average Error (%)	RMS Error (%)
first 9 events	-0.3	18.1
last 8 events	1.7	30.9
all events	0.6	24.9

Figures 7.3a and 7.3b provide plotted results comparing the calculated and observed runoff volumes and peaks. Appendix E contains a complete listing of the model run which produced the results in table 7.3 and figures 7.3a and 7.3b.

It is noted that results of the simulation are generally good, but that there is an evident outlier in the plot in figure 7.3a. This event, from rainfall on 81/8/4 to 81/8/7 was recorded on several gauges in the immediate vicinity (within a 20 km radius) of Sawmill creek. Event totals recorded on these gauges are given in table 7.4 (Rideau River Stormwater Management Study, 1981):

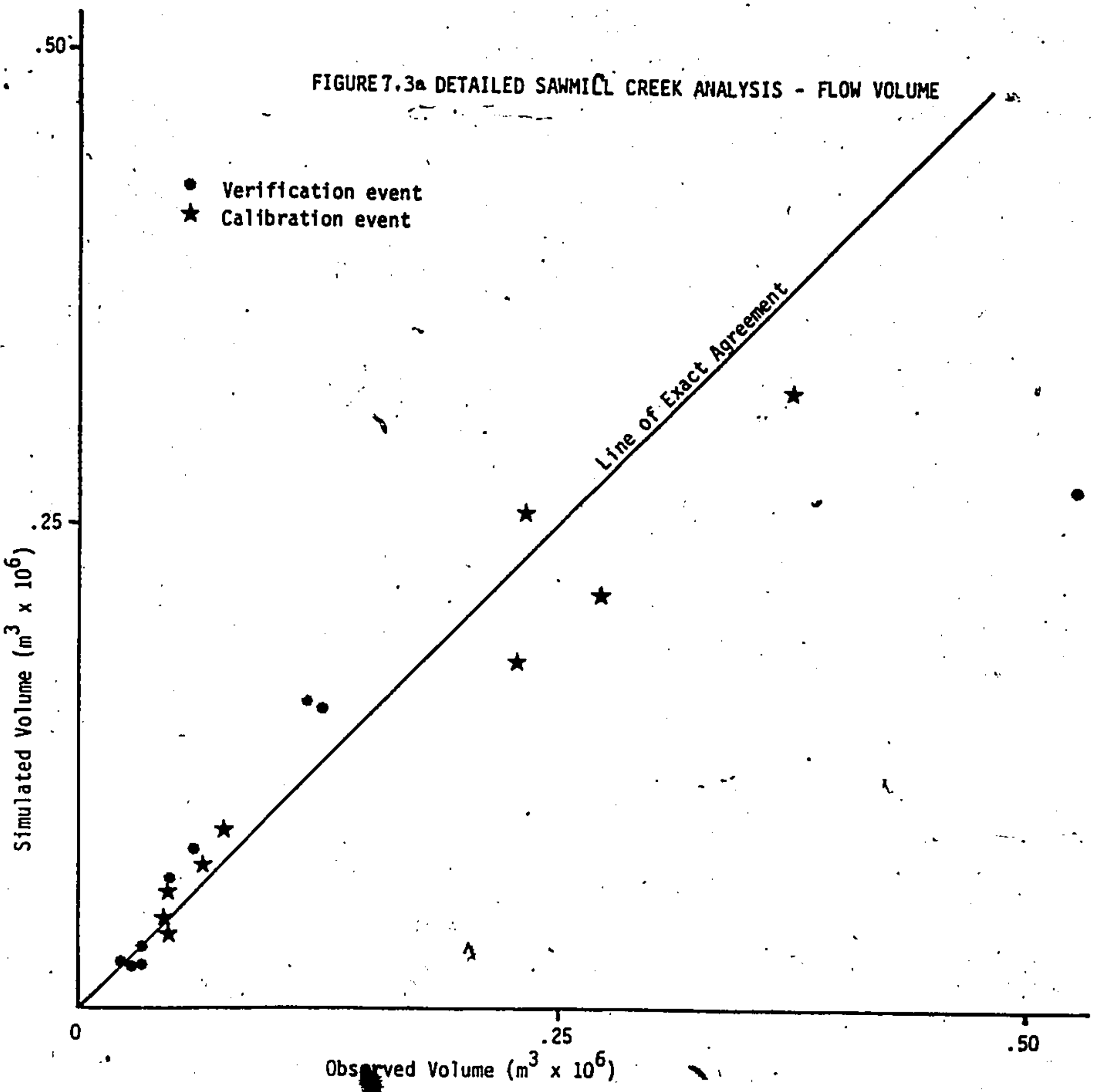


FIGURE 7.3a DETAILED SAWMILL CREEK ANALYSIS - FLOW VOLUME

FIGURE 7.3b DETAILED SAWMILL CREEK ANALYSIS - FLOW RATE

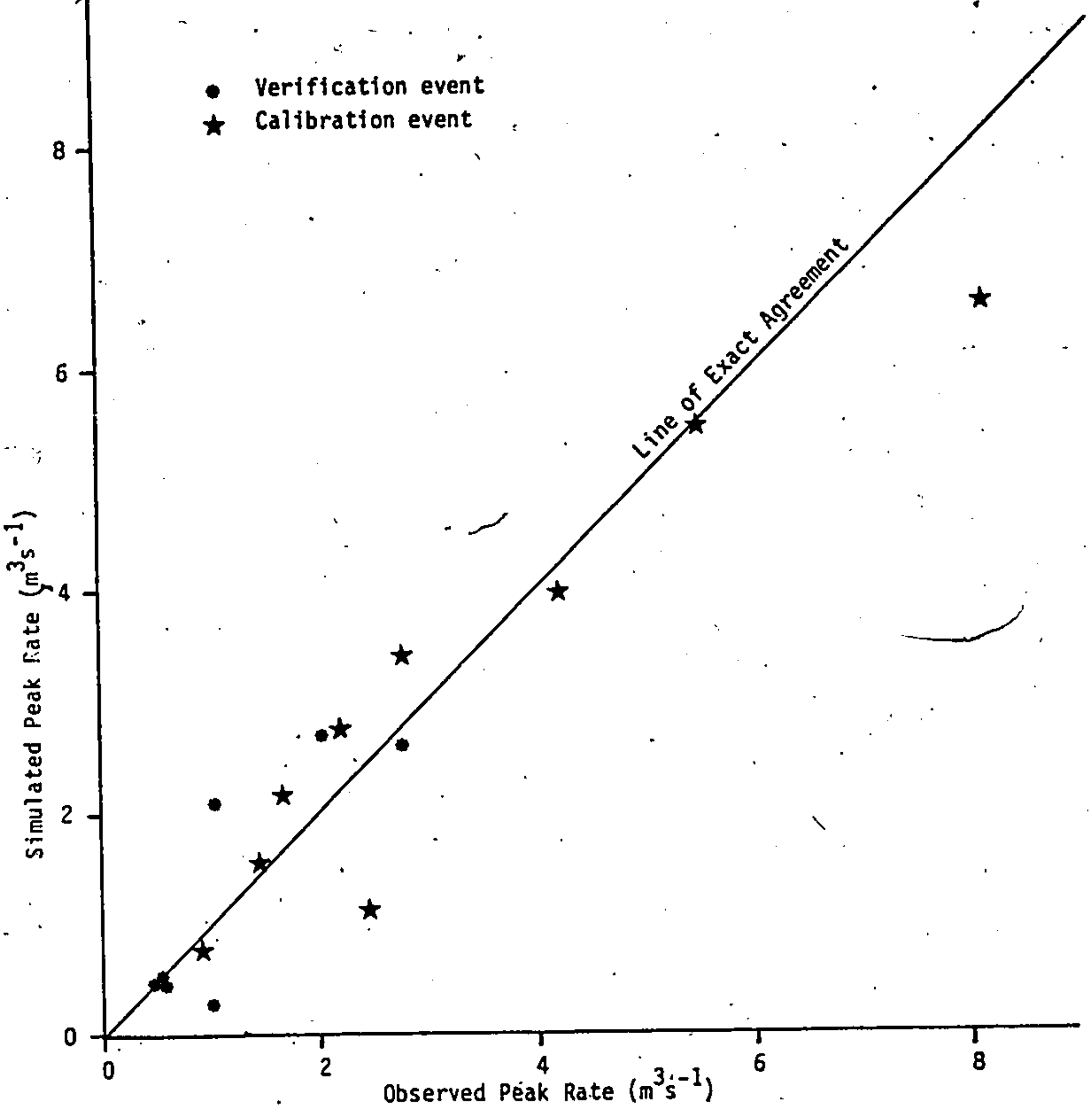


Table 7.4 Rainfall Event Totals, Event of
81/8/4 to 81/8/7

Gauge	Observed Volume (mm)
Charkay	3.0
Alesther Rd.	13.4
Windsor	55.2
Norway Cres.	63.6
Heatherington	65.1 *
Ottawa Airport	116.7

* used in simulation

Variation in results for this event is significant. Note that the Ottawa Airport gauge, located approximately across the diameter of the watershed from the Heatherington gauge indicated a much higher rainfall, and that the simulated result is low. It is therefore indicative, although not proven, that the outlier results from an inappropriate (low) input value for rainfall. Statistics calculated without the outlier event resulted in:

Table 7.5 Results of Sawmill Creek Calibration Omitting the Event of 81/8/4 to 81/8/7

Case	Average Error (%)	RMS Error (%)
first 9 events	-0.3	18.1
last 8 events	8.7	7.6

It is concluded from the above calibration results that:

o The model is appropriate for use in large sized (up to approximately 2500 ha) catchments of mixed urban and rural composition.

7.4 Detailed Comparative Calibration to Wixon Creek Flow Volume Measurements (Large Catchment, Undeveloped)

A third test of the runoff algorithm used in the model was made on data obtained for the Wixon Creek catchment. As described above, Wixon Creek was chosen both because recorded flow data were available (Water Survey of Canada Gauge 02LA004), and because it represented a very pervious and relatively large catchment useful as a third test case for the model. The model was tested (i) by calibration to recorded flow records and (ii) by comparison to the STORM model calibration in that watershed.

Analysis

The criterion for calibration in this catchment was the same as in the Sawmill Creek analysis: RMS error between observed and simulated flow volumes was minimised subject to maintaining an average error of zero, with error defined as (1-(volume simulated)/(volume observed)).

Appendix E contains a plot of observed and simulated flows for Wixon Creek, generated by the proposed model (DUMP PLOT command, see Appendix D). Figure 7.4 contains the same information in summary form. It is evident in both cases that the model performs well in this catchment.

The RMS error calculated by the proposed model was just over 10%; figure 7.5 provides a more detailed depiction of the error in simulation. As indicated, most events are simulated with errors in the 0 to 30% range, and almost all have less than a 50% error. This is a

FIGURE 7.4 WIXON CREEK CALIBRATION - FLOW VOLUME

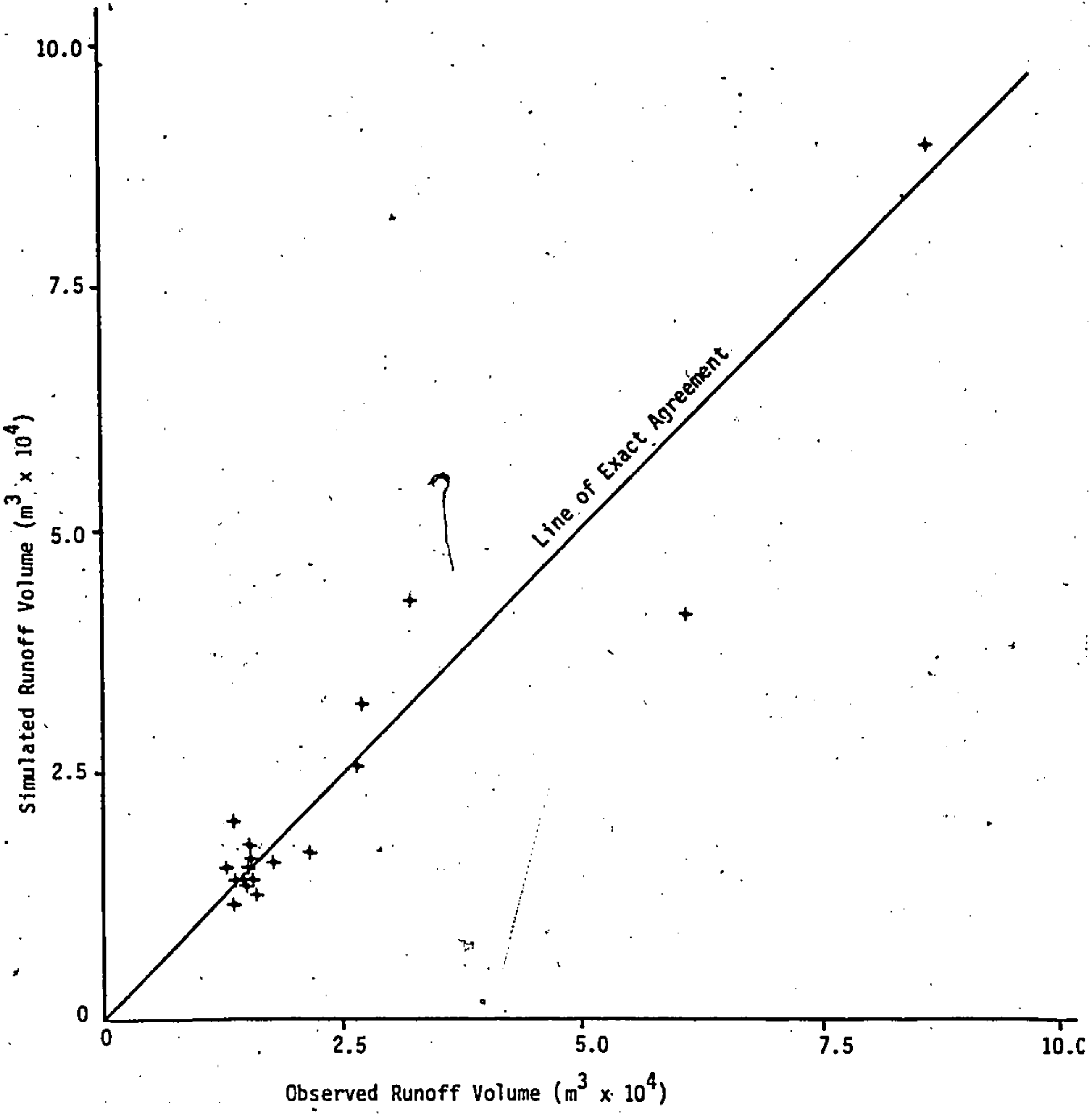
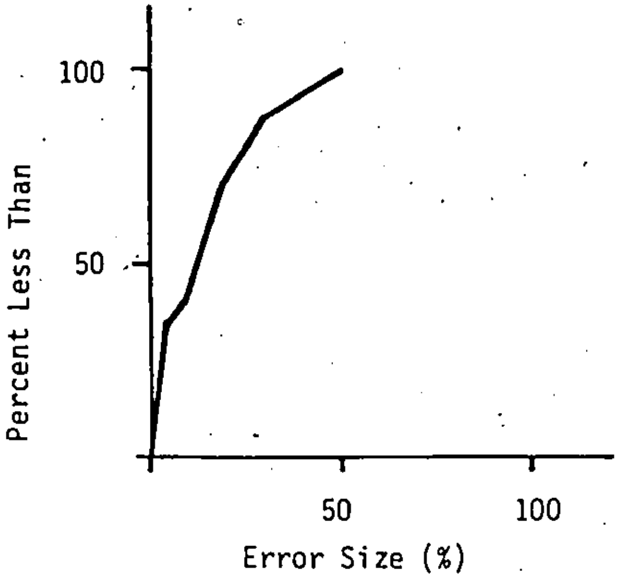


FIGURE 7.5 OBSERVED ERRORS IN WIXON CREEK VOLUME SIMULATION

Tabulated Result

Percent Error	Number L.T.	Percent L.T.
± 5	6	35
± 10	7	41
± 20	12	71
± 30	15	88
± 40	16	94
± 50	17	100

Plotted Result



reasonable result.

Comparison to STORM

The STORM model was calibrated to the Wixon creek data concurrently with the above described analysis using the proposed model (Halliday, 1984). The criterion for calibration was the same as in the Sawmill Creek analysis (minimum RMS error subject to zero average error). However, because STORM does not generate the required statistics itself, the STORM calibration required creation of a secondary program which analyzed the STORM output; it was found that incorporation of such a routine in the original STORM model would be more difficult and time consuming than use of a second program.

The best result achieved in the STORM calibration resulted in an RMS error of 36% and took several days of concentrated effort. The degree of effort was partly due to the number of calibration parameters and partly due to the two-stage model setup (Halliday, 1984). With a best fit RMS error of 36%, STORM provided a best result demonstrably inferior that of the proposed model.

It is concluded from the above calibration and comparison that:

- The proposed model is likely to be superior to

STORM in simulation of runoff quantity in rural areas.

- The proposed model is appropriate for use in simulation of runoff quantity in large rural catchments.

It is also noted that (i) the entire calibration effort using the proposed model was less than one day, compared to several days for STORM, and that (ii) the computer cost per run of the proposed model as determined by system intrinsics was approximately half of the cost per run for STORM.

The first point resulted primarily from the structure of the proposed model, which as mentioned above includes diagnostic routines not available in STORM; statistics for calibration had to be calculated with an anciliary program in the STORM runs, but were generated internally in the proposed model. This reflects programming technique more than any inherent suitability of the runoff algorithm, but does at least illustrate the utility of structural concepts embodied in the proposed model.

The second point is an inherent result of using a model which is based on algorithms which are suitable for a planning analysis. The intended economy of operation of the model has clearly been achieved since it has been shown to be at least as economical as STORM. As discussed in chapter 1, STORM is both appropriate for a planning level analysis and significantly less costly than existing alternatives; since it is more economical than STORM, the proposed model must ipso facto be economical enough for planning and less costly than alternatives itself.

7.5 Detailed Calibration to Sawmill Creek Water Quality Measurements (Large Catchment, Mixed Compostion)

As well as the quantity data described above for Sawmill Creek, quality monitoring results were available from field work carried out for a period of several weeks in the summer of 1981 to quantify fecal coliform pollution of that stream (Rideau River Stormwater Management Study, 1981). Further testing was therefore carried out on Sawmill Creek to establish how the model performs in a quality/ quantity simulation. This testing was against the measured fecal coliform bacteria data, and also against HSPF results in the same catchment.

Analysis

The calibration was carried out as follows:

(1) A least squares regression of the observed water flow rate and F.C. concentration data was carried

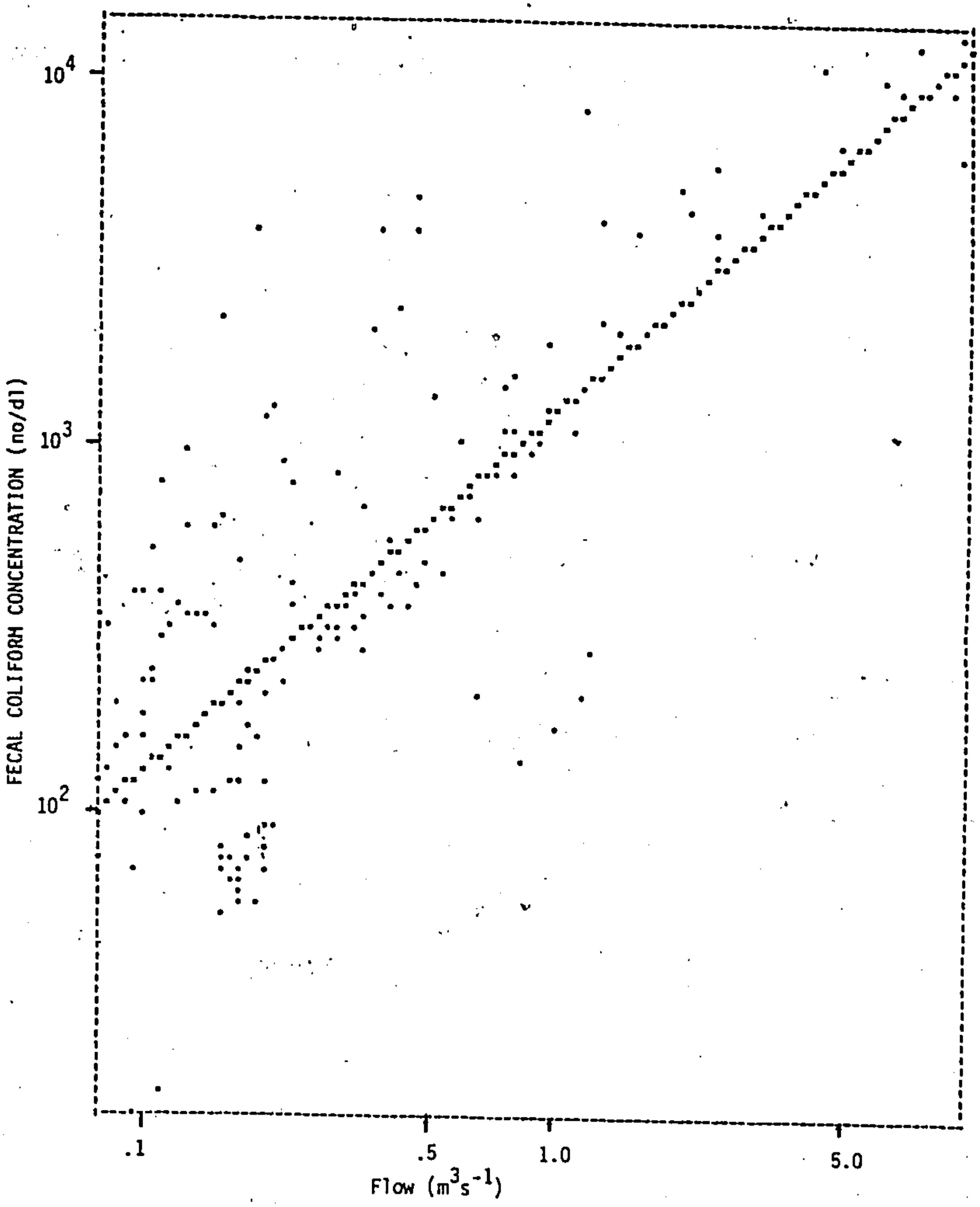


FIGURE ~~7.6~~ PRELIMINARY ANALYSIS OF FECAL COLIFORM AND FLOW DATA FOR SAWMILL CREEK, 1981 out to provide an initial evaluation of bacteria source characteristics. This regression showed that bacteria discharge rates could be correlated with flow rates for flows greater than base flows, and a rating curve approach was selected for bacteria simulation. The results of this regression are shown in figure 7.6.

(2) The model was run using the calibrated flow simulation parameters determined as described in section 7.3. Parameters for bacteria simulation were adjusted by successive approximation to achieve a best overall fit.

In this analysis, measures of fit were objective but application of these measures was not. Due to the highly variable nature of F.C. bacteria instream, limited number of events (5) in the sampling period, and presence of extraneous (not stormwater runoff) sources of F.C. pollution, a meaningful objective criterion for a best fit in this case is difficult to define. The model was therefore adjusted to minimise the average error and the RMS error concurrently, with equal weight given to each. This accords with typical practice in this type of analysis (see for example Gore and Storrie et al, 1981; ibid 1982).

The result of this effort is presented in appendix E in the form of plots of observed and simulated fecal coliform bacteria generated by the proposed model.

These results are also summarised in table 7.6

7.6 Sawmill Creek Fecal Coliform
Bacteria Calibration

Mass Error (%)		Peak Rate Error(%)	
Average	RMS	Average	RMS
-13.2	-19.7	-3.6	-9.1

Since variation of measured fecal coliform bacteria in this type of problem is typically over orders of magnitude (Rowney et al 1982), the above results are considered to be reasonable.

Comparison to HSPF

The calibration by the proposed model may be compared to an HSPF calibration carried out on the same data on the same watershed (City of Ottawa, 1982). The calibration by HSPF was carried out by the City in the course of a major analytical effort undertaken on the Sawmill Creek watershed during the Rideau River Study.

The event definition used in the HSPF study was somewhat different, from that used with the proposed model; the HSPF study defined fewer (four) events in the simulation period. However, as shown in table 7.7, the two calibrations can be compared on the basis of the sizes of the errors simulated. The proposed model performed as well or better than the reported HSPF calibration in the three situations compared. This is not a conclusive test of capability, but does indicate that the results of the two models are essentially similar when applied in this type of problem.

Table 7.7 Comparison of Calibration to Fecal Coliform Bacteria Using HSPF and the Proposed Model

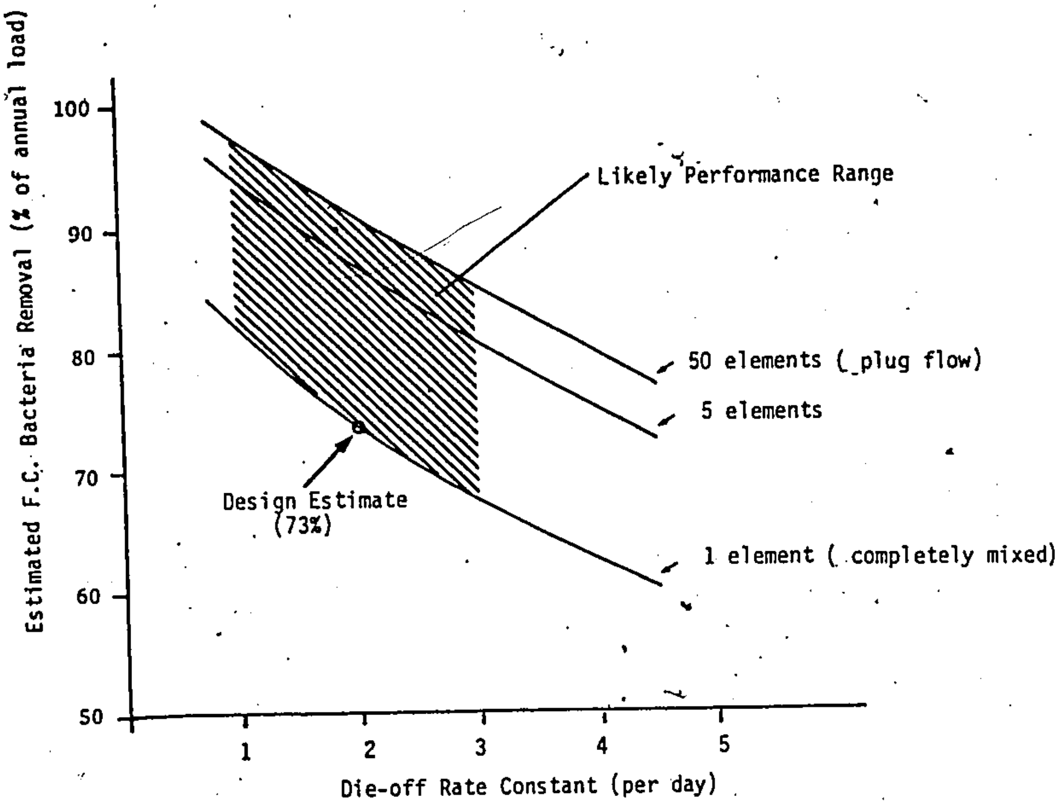
Error (%)	HSPF (5 events)	Proposed Model (6 events)
maximum +ve error	463%	232%
maximum -ve error	575%	459%
average error	163%	115%

Application to Pond Analysis

A further comparison of HSPF and the proposed model was possible on Sawmill creek since the HSPF study referenced above included an assessment of several alternative detention control pond configurations. One of these control alternatives was a single major on-line facility contemplated for the lower end of Sawmill creek.

It was decided to test the proposed model by simulating the control pond configuration defined by the city of

FIGURE 7.7 CONTROL POND ANALYSIS IN SAWMILL CREEK



Ottawa, using calibrated catchment quality/ quantity parameters developed as described above. A graphical representation of the degree of control which is simulated for the pond is shown in figure 7.7.

The solution field in that figure represents the impact of different assumptions regarding pollutant dieoff and pond mixing characteristics. The findings of the Rideau Study suggest a bacteria dieoff rate corresponding to a T90 of 1 to 3 days (Rideau River Study Stormwater Management Study, 1981) and a range slightly wider than this was simulated. The other range of the solution field results from simulations between the two mixing extremes of plug flow and complete mixing.

The design value shown in the figure represents the best design estimate; this assumes an expected dieoff rate T90 of 48 hours (from the Rideau Study results) and assumes the (conservative) completely mixed case. As indicated, the design condition shows an annual removal of bacteria of about 73%.

HSPF, which can represent only one pond type (completely mixed) was used by the City to investigate only one condition, which was equivalent to the above design estimate; the HSPF analysis obtained a result of 76%.

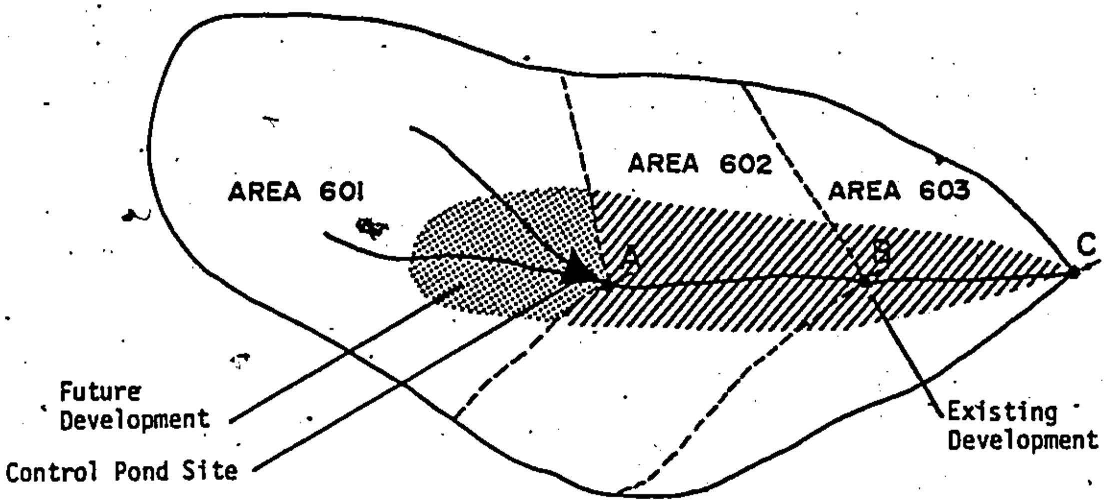
This demonstrates a close agreement between the proposed model and HSPF for pond analysis. It also demonstrates the increased range of information available with the proposed model as a result of the economy; scope, and simplicity of application inherent in its design. Calculation of the effect on pond removal of alternative mxing conditions and dieoff rates was done by the proposed model in one run by making a series of simulations of several different pond conditions; total computer cost for the runs was an order of magnitude less than that for the HSPF analysis despite the more comprehensive result with the proposed model.

o It is concluded that in this catchment, the proposed model produced catchment runoff and control pond simulation results equivalent to those obtained with HSPF.

7.6 Testville Analysis

The Testville study area was composed as shown in figure 7.8. Testville was designed to be compatible with the general catchment described in chapter 1, and also to be representative of the physical reality of the Rideau system tested above. As indicated, the test area had a total land surface of 6000 acres of which half was developed. The developed area was taken to be 35% residential, and 65% open land. The development scenario to be analysed was an investigation of the

FIGURE 7.8 TESTVILLE AREA PLAN



Summary of Basin Characteristics

Existing Conditions:

Area 601 - 3000 hectares undeveloped
Area 602 - 1500 hectares, 35% residential 65% open
Area 603 - 1500 hectares, 35% residential 65% open

Future Development:

Area 601 - 35% residential 65% open
Area 602 - as-is
Area 603 - as-is

Reach A-B:

Length - 15,000 m
Width - 70 m
Slope - 0.005 m/m

Reach B-C:

Length - 7,500 m
Width - 70 m
Slope - 0.005 m/m

Possible Control Site:

On-line pond, on main channel at point 'A'.

consequences of development in the undeveloped upper area.

This test case is of practical relevance. The growth pattern described above is typical of Ontario basins as discussed in chapter 1. The data used in this analysis are generally those developed in the Sawmill creek analysis described above. Water quantity parameters were taken from the Sawmill Creek quantity calibration described in section 7.1. Fecal coliform pollution was simulated by assuming 20000 no/dl concentrations in runoff from developed areas, 10000 no/dl in runoff from undeveloped areas, and 40 no/dl in base flow. These figures were based on results observed in monitoring and analysis undertaken during the Rideau Study (1981). Overall composition and dimensions of the land area are consistent with the land surface between the South Urban Growth area near Ottawa, and Mooney's Bay (Gore and Storrie, 1979; Gore and Storrie et al, 1981).

Since Sawmill creek is tributary to the Rideau river near Mooney's Bay, and since the bacteria concentration data are taken from Ottawa measurements, this Testville example is closely and specifically representative of conditions in the Ottawa area.

The analysis of Testville was undertaken in three stages. First, the degree of degradation in the stream under present conditions was estimated. Second, the impact of further urban development was assessed by simulating the effect of the anticipated development. Third, several different control pond designs were tested to determine how they would each control degradation by bacteria pollution in the receiving stream. Since the Rideau River study (1981) has already provided information on the magnitude of control ponds required to protect the river, there are parallel results against which the conclusions of the current Testville analysis may be compared.

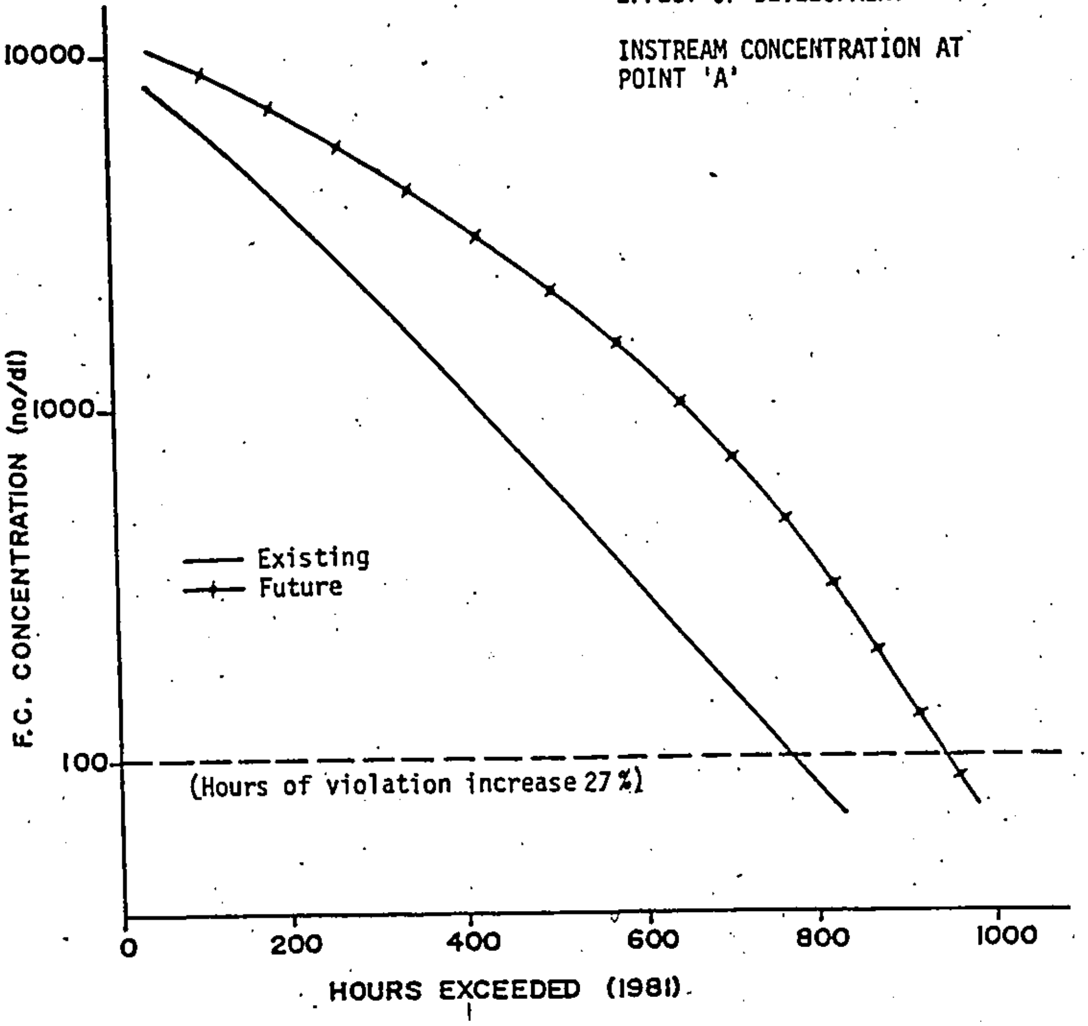
7.6.1 Existing Pollutant Loads.

The model was coded to represent the Testville area in the present condition, with no development in the uppermost basin. This scenario was run, and the model produced exceedance/frequency and number/frequency of the type shown in figure 7.9 and 7.10. As shown, the river is simulated to experience violations of the Ontario standard of 100 no/dl approximately 20 times per year for an average duration of 45 hours per occurence under existing conditions at the bottom of the Testville area. This is a resonable result, since the simulated instream conditions are similar to those occuring in the Rideau River at present (Gore and Storrie et al, 1981); this does not imply a calibration or validation since the catchment is hypothetical, but it does substantiate that results of the model are

FIGURE 7.9

EFFECT OF DEVELOPMENT -

INSTREAM CONCENTRATION AT POINT 'A'



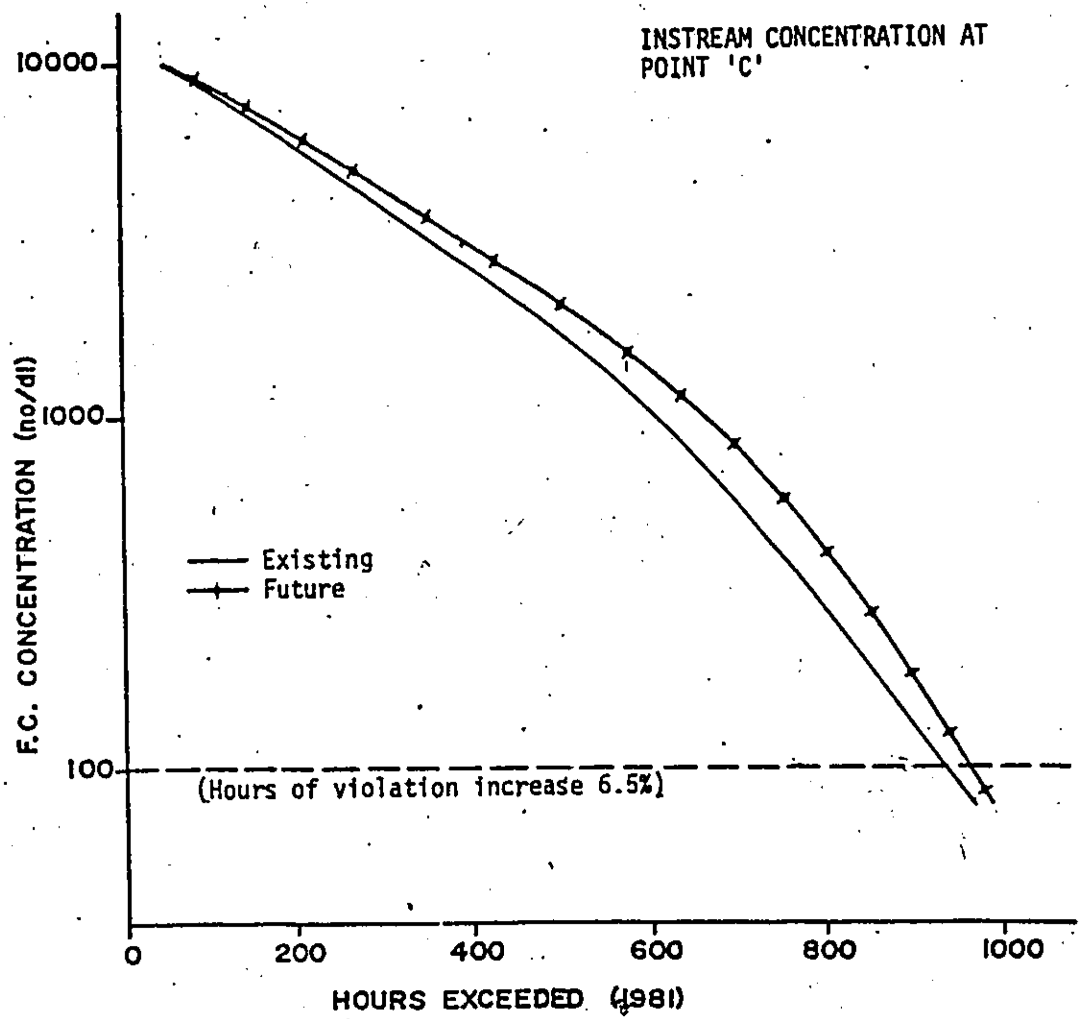
CHANGE IN NUMBERS OF EXCEEDANCES

LEVEL (no/dl)	100	1000	10000
Existing Number	16	14	0
Future Number	20	23	22

FIGURE 7.10

EFFECT OF DEVELOPMENT -

INSTREAM CONCENTRATION AT
POINT 'C'



CHANGE IN NUMBERS OF EXCEEDANCES

LEVEL (no/dl)	100	1000	10000
Existing Number	20	21	9
Future Number	20	21	9

consistent with the type of physical system which is being simulated.

7.6.2 Future Pollutant Loads.

The model code for the undeveloped case was converted to represent the developed case by changing the percent impervious area in the upper catchment from zero to 35%. As shown in figures 7.9 and 7.10, the impact near the development area is significant. However, in the lower reach of the river this impact is reduced to negligible proportions. This observation is also consistent with results of the Rideau study (1981), and substantiates that the kind and degree of simulated impact is appropriate. It was therefore concluded that the Testville scenario provides a reasonable basis for model testing.

7.6.3 Control Pond Configuration 'A'

The control pond selected for this example was assumed to be a single, in-line device 2 m deep and equivalent to a 3mm volume over area tributary to it. It was known that this was a relatively small pond, from results of the Rideau Study (1981).

The pond was assumed to have a 300 mm concrete pipe as an outlet, with an unrestricted spillway to allow overflows for storage volumes exceeding the pond maximum depth of 2m. The flow/stage curve for the outlet pipe was derived from U.S. Bureau of Roads nomographs (Design of Small Dams, 1976). The pond was coded as operating in a continuous mode (continuous flow through) and was assumed to behave as a completely mixed reactor.

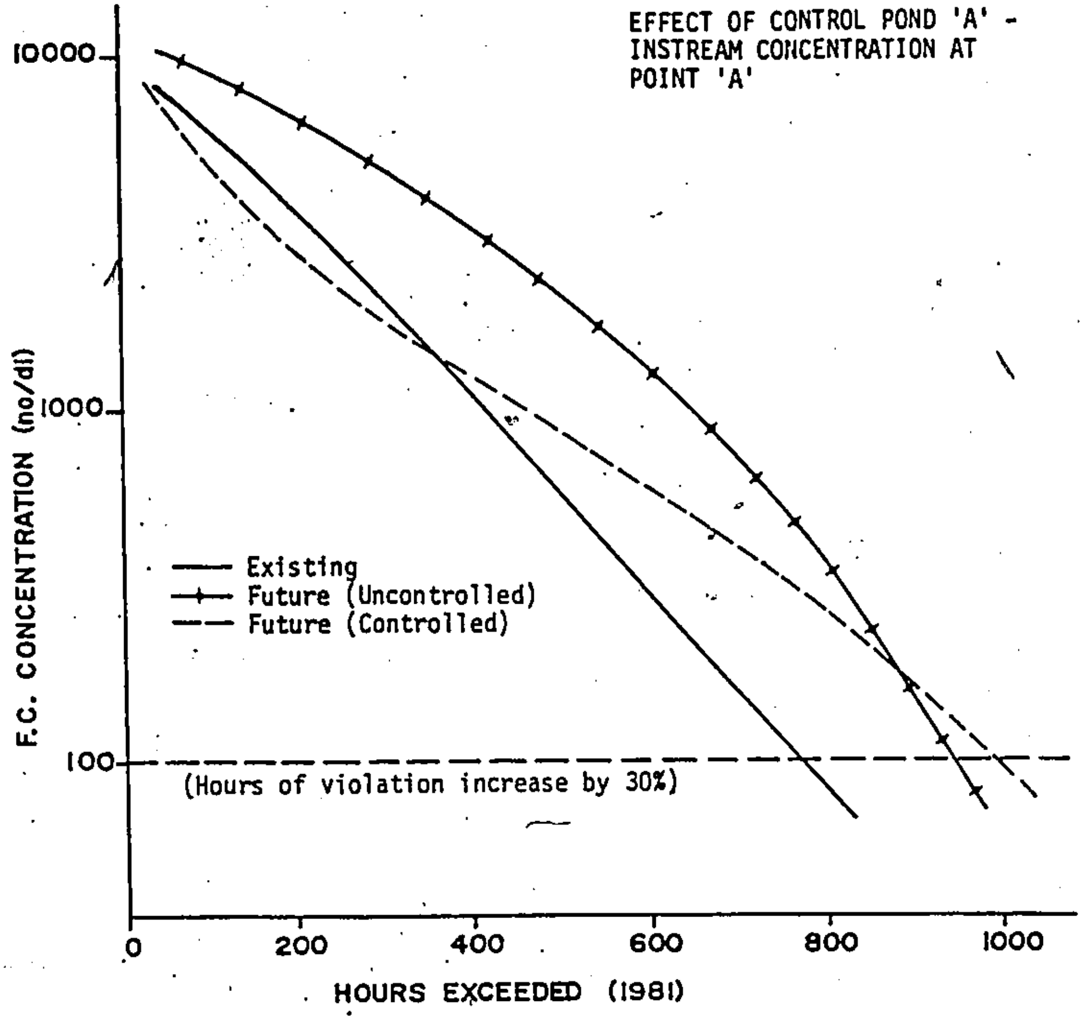
The impact of the pond is shown in figure 7.11. The results of this simulation show that the pond has some impact on large events (high concentration short duration and low numbers of exceedances) but little effect on more frequent events (low concentration long duration and frequent exceedances).

Examination of the model output shows that the apparent increase in durations of exceedances at the low level results from mixing characteristics in the pond. The size and configuration developed is not large enough to provide an adequate period for dieoff, so a large degree of control is not accomplished. However, the pond volume is large enough to act as a reservoir of pollutant mass for a significant period after an event. The pond therefore acts as a buffer which reduces peak concentrations, but increases lower level concentrations, particularily after an event.

Figure 7.12 demonstrates this. That figure is a plot of concentration against time for one large event, taken from each of the three simulated cases (existing, future, and future controlled). As shown, the event

FIGURE 7.11

EFFECT OF CONTROL POND 'A' - INSTREAM CONCENTRATION AT POINT 'A'



CHANGE IN NUMBERS OF EXCEEDANCES

LEVEL (no/dl)	100	1000	10000
Existing Number	16	14	0
Future Number	20	23	22
Number with Pond	20	16	1

POND CHARACTERISTICS

Volume — Equivalent to 3 mm over basin.
Depth — 2m to spillway.
Outlet — 300mm concrete pipe; unrestricted spillway.
Inlet — Unrestricted.
Operation — Continuous, completely mixed.

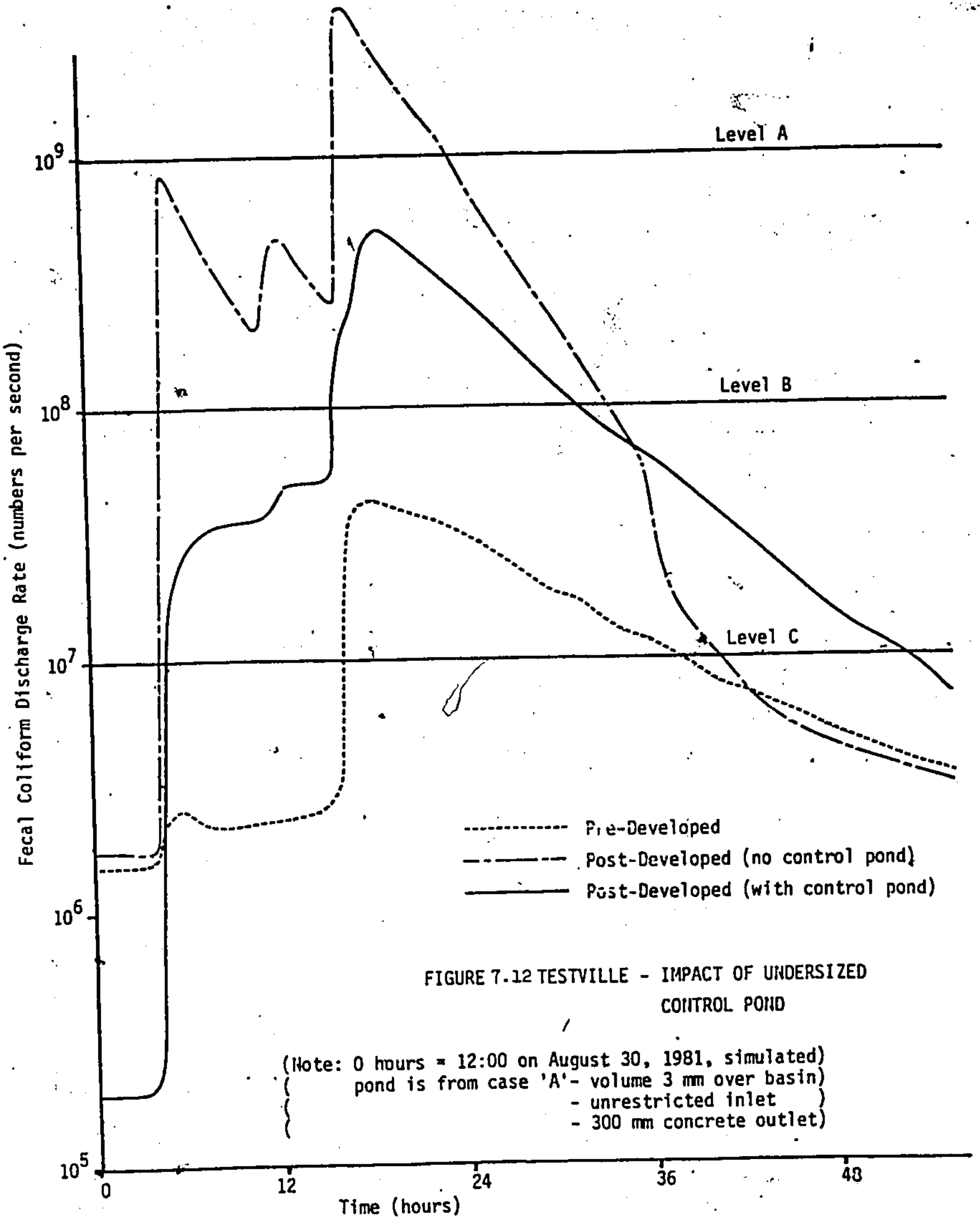


FIGURE 7.12 TESTVILLE - IMPACT OF UNDERSIZED CONTROL POND

(Note: 0 hours = 12:00 on August 30, 1981, simulated)
(pond is from case 'A'- volume 3 mm over basin)
(- unrestricted inlet)
(- 300 mm concrete outlet)

exhibits a significant increase in overall mass, duration, and peak rate of pollutant concentration after urbanization, which is appropriate. With the control pond in place, peak concentrations are lowered; this is due to the relatively low concentration existing in the residual volume in the pond at the start of the event. Inflows during the early part of the event are satisfactorily retained, and the concentration is lowered. With continued inflow, however, the available pond storage capacity is used up; high concentration outflows then occur, and persist for some time after the event.

With a larger pond, this effect would not be felt, since more of the stormwater mass would be retained for a longer time; dieoff of bacteria would reduce the ultimate outflow concentration. WIth the pond as it is, however, the result on outflow concentration is that the peak concentration is reduced (level A on figure 7.12) at the expense of longer durations at lower concentrations (level C on figure 7.12). At intermediate levels, some reduction in duration of exceedance is obtained (level B on figure 7.12).

This result is significant. The pond used in this example does significantly reduce peak flow rates, and the duration of overall detention (in the same order as the duration of the inflow event) is consistent with typical practice in quantity control; the pond therefore functions as an effective quantity control device. Present practice commonly assumes without supportive analysis that installing quantity control ponds will have inherent quality benefits as well as intended quantity benefits. This example shows that this assumption may be inaccurate, and unless a comprehensive analysis is undertaken, unintended adverse impacts may actually result from installation of a control pond.

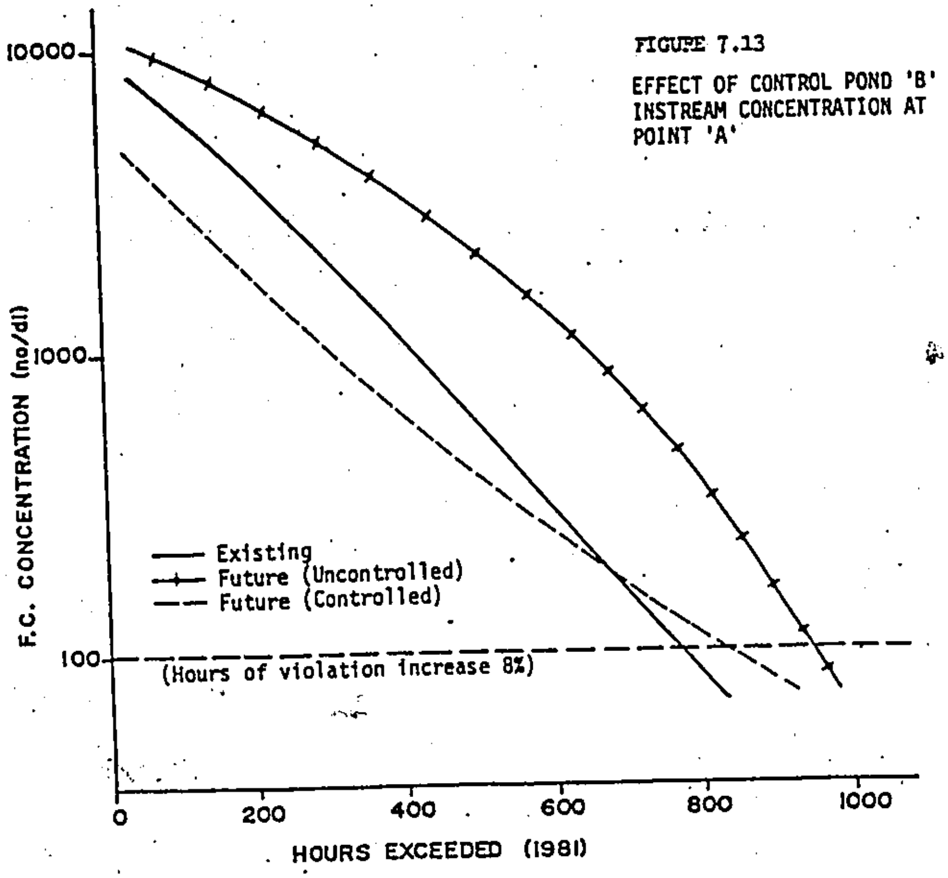
7.6.4 Control Pond Configuration ''B'

The second control pond tested was equivalent to a volume of 10mm over the tributary basin, but was otherwise identical to pond 'A'. As shown in figure 7.13, this pond is large enough that it provides a dieoff period (pond detention time) which controls quality of outflows to approximately pre-developed conditions.

This result is in agreement with results of the Rideau Study which derived a required pond control volume of 10mm to 20mm to achieve control of pollution increases associated with urbanization in the Ottawa area. As noted in chapter 1, however, the Rideau Study required three separate models to achieve the same result and was not able to incorporate a complete examination of instream impact of the pond.

FIGURE 7.13

EFFECT OF CONTROL POND 'B' - INSTREAM CONCENTRATION AT POINT 'A'



CHANGE IN NUMBERS OF EXCEEDANCES

LEVEL (no/dl)	100	1000	10000
Existing Number	16	14	0
Future Number	20	23	22
Number with Pond	18	13	0

POND CHARACTERISTICS

Volume — Equivalent to 10 mm over basin
Depth — 2 m to spillway
Outlet — 300mm concrete pipe; unrestricted spillway.
Inlet — Unrestricted.
Operation — Continuous, completely mixed.

7.6.5 Control Pond Configuration 'C'

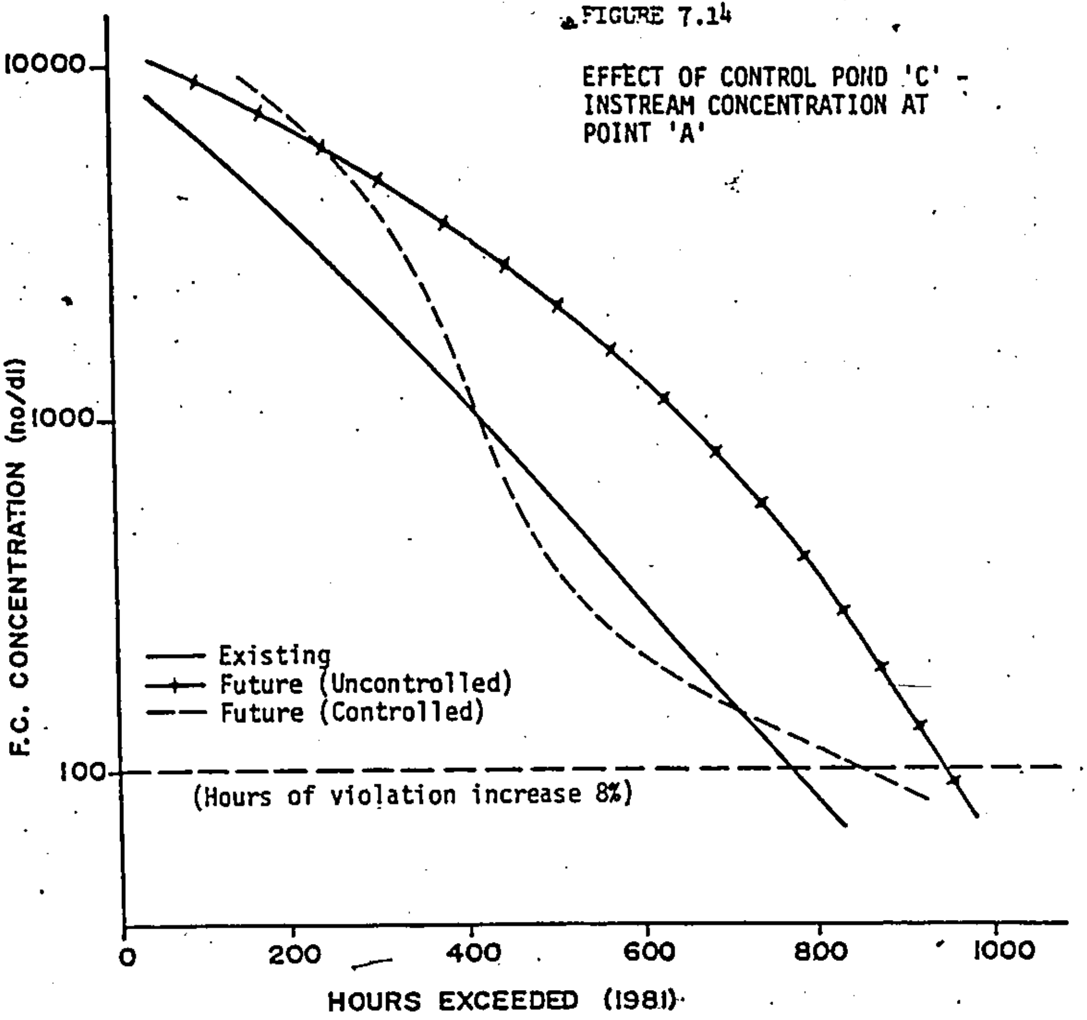
This control pond was identical in size and outflow structure to control pond 'A', but was designed to incorporate an unusual inlet control device. In this case, the pond inlet was designed to inlet flows only if the approach flow was less than 0.84 cms. Approach flows less than this rate were admitted entirely, flows greater than this were not admitted at all. The effect of the pond is therefore to treat only small events and the beginning and end of large events. The precise inflow control structure behaviour was determined after successive trials with different cutoff rates.

The principle of operation in this case is that events which would cause a violation of the 100 no/dl standard in the existing situation are not treated at all, and the events which would not previously cause a violation before development are prevented from causing a violation after development. This is a concept which has not been previously tested.

As indicated in figure 7.14, the effect of the pond with this inlet control at low concentration levels is to control conditions and to effectively maintain violations at existing levels. At high levels, however, the conditions are not improved at all. In terms of the 100 no/dl objective concentration, this pond is

FIGURE 7.14

EFFECT OF CONTROL POND 'C' - INSTREAM CONCENTRATION AT POINT 'A'



CHANGE IN NUMBERS OF EXCEEDANCES

LEVEL (no/dl)	100	1000	10000
Existing Number	16	14	0
Future Number	20	23	22
Number with Pond	24	24	18

POND CHARACTERISTICS

Volume — Equivalent to 3 mm over basin.
Depth — 2 m. to spillway.
Outlet — 300mm concrete pipe; unrestricted spillway.
Inlet — By-pass, operating for larger flows.
Operation — Continuous, completely mixed.

essentially equivalent to the much larger pond 'B'.

Of interest to the present work is that this analysis suggests that pond inflow/ bypass characteristics may be as important in obtaining specified target conditions instream as pond volume and outflow characteristics. Previous practice has not accounted for this effect in water quality control pond analysis. It is interesting to note that the analysis which leads to this observation, conducted with the proposed model, would be difficult or impossible with SWMM or HSPF.

7.7 Implications for Regulatory Policy

Existing Policy

Ponds 'B' and 'C' present interesting implications in terms of policy for specification of instream criteria. At present, in Ontario the MOE standard of 100 no/dl (PWQO) is the only criterion for assessment of control ponds and instream conditions. For other jurisdictions and other pollutants, other standards apply; but as discussed above these still tend to be single valued.

From the perspective of a single valued criterion of 100 no/dl, it appears that pond 'C' is as effective as pond 'B'. However, pond 'C' has less than a third of the volume of pond 'B' which suggests a smaller land area and hence cost savings. The difficulty with this approach is that, although the 100 no/dl standard is met equally with either pond, pond 'C' does not do an adequate job of controlling pollutants at other levels.

Where risk is proportional to concentration, as is usually assumed in the case of indicator bacteria, this is an unacceptable situation. Present legislation and guidelines tend to require only demonstration of the degree to which a single valued criterion is met or not met. In the ponds 'B' and 'C' above, it is evident that one pond is much different from the other in terms of overall effectivenes, but results reported in terms of the single valued indicator level of 100 no/dl would not distinguish between the two. It has therefore been demonstrated that the single valued expression of an instream management criterion, even if seasonally based, does not provide an adequate measure of protection.

It is concluded that there is a need to examine pollutant control devices on the basis of more than one instream concentration level; it appears that the entire range of concentrations instream should be considered.

A Possible Alternative Policy

An alternative policy for specification of instream control requirements may be proposed if certain assumptions are made as to the impact of instream pollutants, namely that:

a.) lower pollutants concentrations do not represent worse conditions than higher concentrations;

b.) shorter durations at a given concentration level do not represent worse conditions than longer durations;

c.) existing conditions are acceptable; and

d.) a worsening in existing conditions, represented by an increase in instream duration at any concentration, is unacceptable.

This may be viewed as somewhat conservative as it does not recognise any residual assimilative capacity, but is otherwise generally consistent with current concepts as discussed in chapter 1.

In such a case, the solution space illustrated in 7.15a, defines an appropriate minimum criterion for assessing instream impacts. Any scheme which provides an instream concentration/ duration curve lying within the shaded area represents an acceptible alternative, in accordance with the above assumptions.

In terms of the specific problem faced in differentiating ponds 'B' and 'C', this approach would still determine that both of the ponds meet the existing single valued criterion; in addition, however,

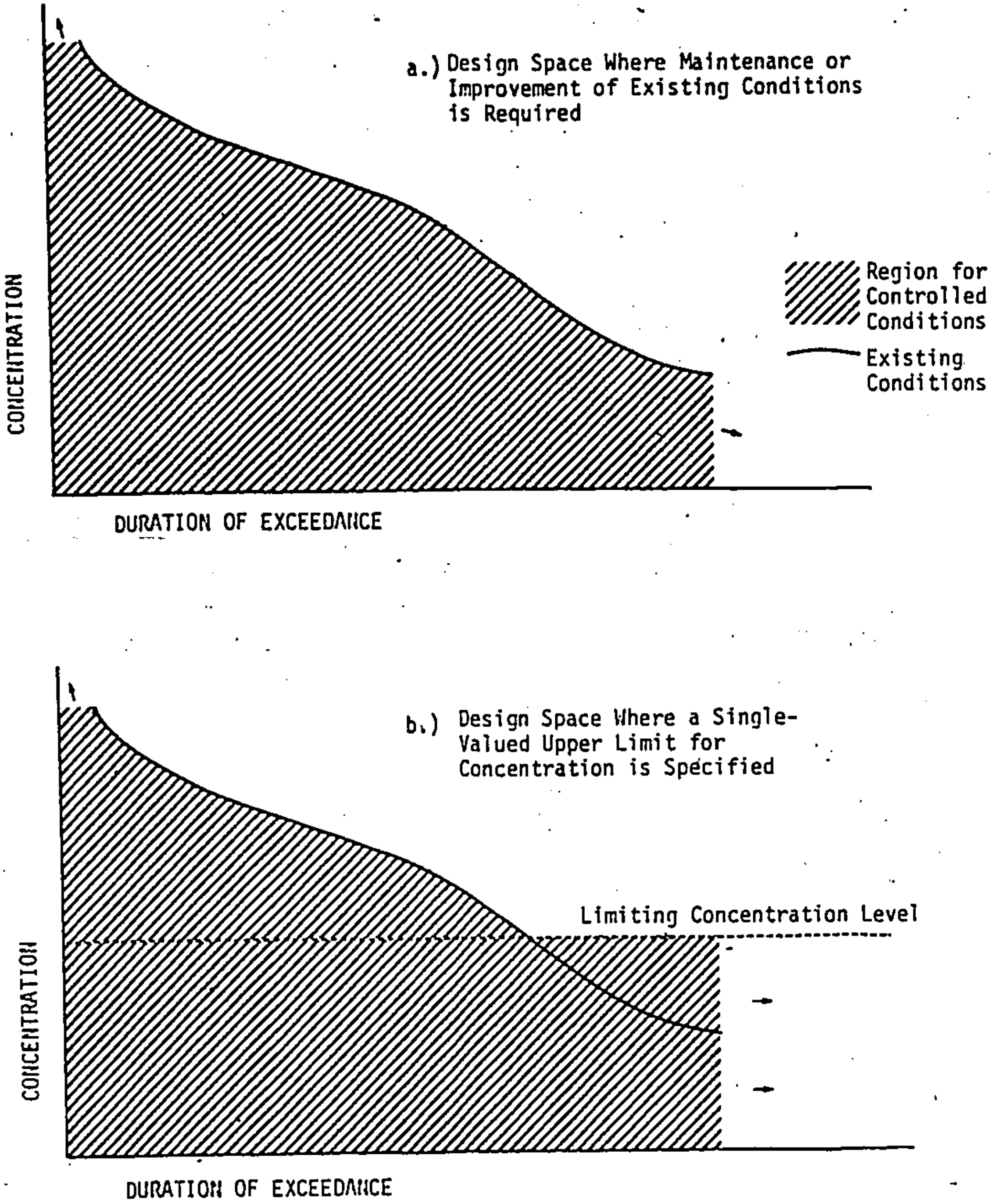


Figure 7.15 TWO POSSIBLE APPROACHES TO DEFINITION OF AN ALLOWABLE DESIGN SPACE FOR CONTROLLED CONCENTRATION/DURATION CURVES

this approach would establish that pond 'C' is not as effective as pond 'B'. This approach would therefore be superior to a single valued criterion in this instance.

Variations on this approach can be specified. For example in figure 7.15b, a solution space in shown for a system otherwise similar to that in 7.15a, but where an additional assumption is made, namely that:

e.) there exists a limiting concentration level below which negative impacts are not an issue.

This limiting level might occur if there is a threshold limit for damage, below which the pollutant is innocuous. It also might occur if a single valued criterion has been specified for the river, and that any condition below that criterion will meet regulatory requirements. Figure 7.15b would resolve either situation and still provide a better basis for assessment of alternatives.

It is recognised that the approach described above is still limited in that it does not permit any weighting of the problem according to degree of damage. The approach therefore does not allow selection of a 'best' choice when two alternatives are acceptible according to the above criterion, but are different. Such information could in principle be incorporated, in a manner analogous to that used in benefit/ cost analysis, by integrating the product of a damage/ concentration/ duration curve with the simulated concentration/ duration curve to produce a net 'index of damage'. The lesser degree of damage would then be preferable.

Unfortunately, the environmental basis for such a refinement has not yet been established. The above approach to criteria specification, supported by a case by case judgement for alternatives which are otherwise acceptible, is therefore the best alternative which can be proposed at this time. Further work in this area is clearly needed.

8.0 Conclusion

One of the most costly problems facing urban development in North America in this decade is control of the pollution, potential and actual, of receiving waters by urban areas (McNeil, 1976). A major component of that pollution is associated with stormwater runoff, and analysis of stormwater pollution at a planning level has been the central concern of this research. This chapter provides general and specific conclusions on the overall results of research. Possible avenues of future research are also outlined.

8.1 General

An examination of the problem of planning analysis for management of stormwater pollution has shown that planning should be done at a regional level, and has resolved some major characteristics which a simulation model created for that purpose should have. These are listed in chapter 1. Investigation of available simulation models in light of these characteristics has shown that existing approaches to simulation are not suited to analysis of stormwater pollution management problems.

The primary reason for this appears to be that existing models were generally not developed specifically for regional planning analysis as it is presently practiced, but are based on techniques developed for and borrowed from other problems. Specific available models which were assessed have characteristics which support this conclusion. For example: STORM is limited in scope of simulation and unable to analyse systems which include an instream transport component; HSPF relies for runoff simulation on complex and seldom applied algorithms which are not suitable for a regional analysis at a planning level; SWMM is oriented towards detailed urban stormwater quantity management.

This work has contributed to this area of engineering by creating a simulation model which is specifically oriented towards the problem of regional stormwater quality management planning in river basins. The model is structured in a way which is different from other models, and contains algorithms which are different in concept from existing approaches used in continuous simulation of stormwater quality at a regional level.

Test applications of the model have verified that currently applied single valued approaches to specification of instream water quality criteria are unable to adequately discriminate between management alternatives, and an alternative criterion has been proposed as a result.

8.2 Specific Conclusions

1. There is a need for an alternative regional simulation model for water quality management planning. Specific general qualities are identified which make a model appropriate this purpose, and it has been found that existing models do not meet these criteria.

2. A model has been developed for regional stormwater quality management planning analysis, which meets the qualities defined herein for such a model. The model is therefore an improvement over existing approaches.

3. A method of instream quality routing has been developed. This method is consistent with many existing approaches in (i) its fundamental assumptions about the nature of pollutant transport and (ii) results, but differs from existing approaches in the solution technique which is applied. In the conditions tested, the proposed routing method was shown to produce results which correspond to those of more complex two-dimensional models, and did so at much less cost. The method may therefore be useful in practical applications where existing alternatives are inappropriate.

5. An alternative to existing algorithms for continuous simulation of runoff volume has been developed. The proposed method uses an Antecedant Precipitation Index to update loss parameters and thereby avoids the uncertainty and difficulty of continuous accounting for soil moisture storage. The concept could be used to achieve a continuous result from a variety of loss functions, but for practical reasons has in this work been coupled with a variant of the SCS method. As such, the algorithm is fundamentally different from existing approaches.

6. The proposed runoff volume algorithm has been shown to perform well in the specific catchment locations tested, which were of three types:

- small urban catchments,
- a large mixed urban and rural catchment, and,
- a large, predominantly rural catchment.

7. The proposed runoff volume algorithm has been shown to provide a simulation result superior to that which can be achieved by STORM, and at much less cost. Since STORM is generally accepted as appropriate for continuous runoff simulation, it is concluded that the proposed method constitutes an improvement which will be acceptible for that

purpose also.

8. The model has been shown in a test case to produce quality simulation results for both catchment runoff and control pond analysis which are consistent with those achieved using HSPF. The proposed model therefore represents an effective alternative to HSPF.

9. A test case which was evaluated using the proposed model has demonstrated that the use of a single valued criterion for stormwater quality management would be unsuitable in that case. This is demonstrated by the failure of the single valued criterion to discriminate adequately among alternative management scenarios. The test case in which this was demonstrated was representative of typical stormwater quality management planning problems. It is therefore concluded that there is an adequate basis for stating more generally that:

a.) A single valued criterion for instream concentration, even if frequency based, does not always provide an adequate measure of or basis for instream quality management.

b.) Management policy in Ontario and other locations where a single valued criterion is used should be reviewed to determine whether an alternative policy should be adopted. One possible alternative criterion has been presented herein.

8.3 Future Research

1. The proposed model, and its functional algorithms, should be further tested in a wider variety of catchment and management conditions. This would provide further supportive data for use of the model, and would contribute towards a data base which could facilitate use of the model in uncalibrated applications.

2. Further work should be done to develop an alternative approach to specification of criteria for stormwater quality management. The model developed herein would be appropriate for use in such an investigation.

3. In support of the need for improved water quality managment criteria, further work should be done to better establish the kind and degree of impact which typical pollutants in urban stormwater have on receiving waters. An emphasis of this work should be to determine what relationships exist between damage and pollutant

concentration, duration, and frequency of occurence.

4. It is noted that some of the conclusions in this work are based on theoretical assumptions about the nature of pollutant removal in control ponds; these assumptions are an unavoidable consequence of the present state of the art of simulation control pond removal mechanisms. Further research should be conducted to establish exactly how control ponds function to remove pollutants, and to determine whether an alternative removal model should be used.

6. This work has shown that control pond inlet structure operation can have as large an impact on pond performance as pond outlet or size. Further work should be carried out, using the proposed model, to establish to what degree this effect may be important in practical applications. Similarly, further work should be carried out, using the proposed model, to determine what pond operation and size characteristics are most effective in pollutant control in typical stormwater quality management applications.

5. This work has shown that the proposed model appears to produce results equivalent to a much more sophisticated model, HSPF, in control pond planning analysis; it further appears that these results are achieved at much less cost than with HSPF. It therefore appears that a model of the degree of sophistication of HSPF is not required for planning purposes, and should therefore not be used. It is concluded that research should be conducted to determine in what circumstances, if any, a model of the degree of sophistication of HSPF is required.

6. Additional routines should be added to the model to permit simulation of erosion and soil loss in agricultural or undeveloped areas.

APPENDIX A

POLLUTANT AND RUNOFF GENERATION

APPENDIX A

POLLUTANT AND RUNOFF GENERATION

A-1 Introduction

This appendix documents the algorithms used in the model for runoff quantity and quality simulation. Some duplication of information in the main text exists in this appendix. This has been accepted in order to express the relations used in terms consistent with the program code, and to make this appendix comprehensive.

A-2 Runoff Quantity Analysis

As described in the main text, the model is a linear unit hydrograph model. Runoff volume is determined using two procedure, one (for pervious areas) based on the SCS method (Soil Conservation Service, 1969) and the other (for impervious areas) using a volumetric coefficient approach. Runoff rate is calculated by a convolution of runoff volume by either or both of two unit hydrograph shapes.

A-2-1 Runoff Volume- Pervious Areas

As noted in the main text of this volume, the model reads hourly rainfall records from a disc or tape specified by the user. Excess rainfall in pervious areas is calculated by the model using the SCS relation:

$$Q = \frac{(P\text{-}ABSPER)*(P\text{-}ABSPER)}{(P\text{-}ABSPER+S^*)} \qquad (A1)$$

where Q is cumulative depth of runoff (mm),
P is cumulative depth of precipitation (mm),
ABSPER is an initial abstraction (mm),
and S* is a loss parameter (mm).

S* and ABSPER are updated by the model for each event, to provide an accounting for initial moisture conditions. In the case of S*, this is accomplished by expressing S* as a function of a variable Antecedent Precipitation Index, the API. The API is determined from the following relation:

$$API2 = APIK*API1 + P1 \qquad (A2)$$

where APIK is a coefficient,
P1 is precipitation in the previous time step,
and the API subscripts refer to conditions at the beginning

(1) and end (2) of the
previous time step

The relationship which is used to relate S* and API is:

$$S^* = SMIN + (SMAX-SMIN)*\exp(SK*API) \qquad (A3)$$

where SMIN and SMAX represent
the range of S*,
and SK is a calibration
parameter.

If adequate data exists, the parameters SMIN, SMAX and SK can all be adjusted for a best fit to observations. It is noted that in this process, the value of the APIK parameter assumes only minor importance, since the adjustment of equation A3 tends to compensate for changes in the API which result from different values of APIK. (It is also noted that where data is limited, SMAX and SMIN can in principle be estimated from the SCS AMC I and AMC III values of CN. This leaves the single value of SK in equation A3 as the primary calibration parameter.)

Use of the API as a means of adjusting the SCS loss parameter has been investigated and tested in the IMPSWM program, and the pseudo-continuous model achieved using the above equations appears to be an approach suited to planning studies of the type contemplated here. Although there will need to be more Ontario experience in the use of this algorithm for continuous simulation, it appears that the method is potentially useful for multi-event simulation of the type anticipated in this methodology. One advantage to the approach, aside from its simplicity, is that hydrologic parameters required by the model are generally well understood. The basic SCS loss relations have been widely applied in Ontario and elsewhere; the main concept introduced by the model is that of an internal link between the SCS equation (A2) and the loss parameter equation (A3). In practice, therefore, the model introduces at a maximum three, and in some cases one, parameter to achieve a continuous or multi-event representation of runoff from pervious areas.

The second adjustment made in the model to provide a continuous capability relates to the value of the available initial abstraction, ABSPER. The user inputs a maximum value of the ABSPER to the model, and this is thereafter adjusted automatically according to the following conventions:

- ABSPER is reduced by the amount of precipitation falling until it reaches zero, where it remains until a dry period occurs.
- In dry periods, ABSPER is increased until the maximum value is reached. This occurs at a rate such that 24 dry hours will raise ABSPER from zero to maximum.

Thus, the initial abstraction will recover to normal conditions in a period of about one day after an event which is a reasonable but essentially arbitrary value. The user can change this with appropriate code changes if required. It was not included as an input condition in the present version of the model in an effort to minimise the number of input decisions required by the user.

Another pre-determined assumption in the model is that of the interevent time which defines a new event. The use of the SCS method by definition requires that losses are computed on the basis of an 'S*' value which is chosen at the beginning of each event. The model code applies a new 'S*' value (using equation A3) to each rainfall period preceded by at least four dry hours. If a dry period is less than four hours, the previously used 'S*' value will be applied. Similarly, if a dry period is less than four hours, subsequent rainfall and flow volumes are considered to be part of the previous event; this is equivalent to assuming that there is one event with a short intra-event period of no rain. This is also a factor which the user can change at a code, but not data input, level.

A-2-2 Runoff Volume- Impervious Areas

The model allows the user to specify some fraction of the basin as an impervious area (using parameter FRIMP in the GENERATE command). It is assumed that the volumetric runoff coefficient in such areas is essentially fixed. The model calculates impervious area runoff as:

$$Q = (P-ABSIMP)*RIMP \tag{A4}$$

where RIMP is a constant volumetric runoff coefficient
ABSIMP is the impervious area initial abstraction,
and other parameters are as defined above.

The initial abstraction in impervious areas is updated in exactly the same way as for pervious areas. Inter-event definitions are immaterial in the calculation of impervious area runoff.

It is noted that the basin can be coded as entirely impervious, or not at all, or any fraction in between.

A-2-3 Runoff Rate

The model calculates flow rates from flow volume by convolution. Two different unit hydrograph shapes are available for this; since runoff from pervious areas is convoluted separately from impervious areas, these can use the same unit hydrograph shapes, or can use two different shapes.

One unit hydrograph available to the user is the original unit hydrograph shape designed by Williams et al (1973) for use in HYMO. Parameters to determine the unit hydrograph characteristics in this case are, optionally, basin height and length or unit hydrograph K and Tp (using input parameters KORH and TPORL in the GENERATE command). If height and length are specified, the Williams empirical relation from HYMO is used to calculate K and Tp. It is noted that although the use of the Williams relation in impervious areas has not been prevented, this may be an inappropriate alternative for small impervious areas in practice.

The second option for a unit hydrograph shape is to use a Nash unit hydrograph (described by Wisner et al, 1983). When this is used, the user inputs N, the number of linear reservoirs, and Tp (using input parameters KORH and TPORL in the GENERATE command); the model the calculates a Nash unit hydrograph for use in the convolution. (This is the same approach as in the OTTHYMO COMPUTE NASHYD algorithm.)

A-2-4 Base Flow Rate

The model has a relatively simple algorithm for calculating base flow. This consists of a single reservoir representing groundwater storage. Mass balance within the reservoir is achieved by calculating net inflow and outflow from the reservoir at each time step. Inflow to this soil reservoir is taken as the difference between precipitation and runoff, minus any losses to initial abstraction. In the absence of infiltration, outflow from the soil reservoir is calculated by a rate averaged over the time step:

QOave = (SVOL1+SVOL2)*SLOSK1/2 (A5)

where QOave is average outflow from the reservoir over a time step, per unit area (mm/sec),
SVOL is the reservoir volume at the beginning (SVOL1) and end (SVOL2) of the time step (mm),
SLOSK1 is the base flow recession constant 7mm/sec/mm).

Note on Input

The constant SLOSK1 can be estimated from stream gauge records for dry (no rainfall) periods. The assumption inherent in equation A5 is that base flow from the reservoir behaves as follows:

QO = SLOSK1*SVOL (A6)

where QO is the instantaneous rate of outflow from the reservoir per unit area (mm/sec),

which, since QO is assumed equal to the rate of outflow from reservoir of unit area, means that during dry periods:

dSVOL/dt = -SLOSK1*SVOL (A7)

This can be solved during dry periods to the form:

$$SVOL/SVOLo = \exp(-SLOSK1*T) \qquad (A8)$$

where SVOLo is an initial value for storage volume,

and T is time elapsed,

and which by substitution of A6 results in:

$$Q/Qo = \exp(-SLOSK1*T) \qquad (A9)$$

where Qo is an initial value for flow and Q is a flow observed at time T.

By regression, or by plotting flows on log paper, the user can estimate SLOSK1 from observations using equation A9. (It may be necessary to provide a constant amount BASMIN as described below to do this.)

The above relations (A5 and A6) are not used by the model to provide based flow directly, since three additional factors are accounted for. First, QO must be multiplied by the basin area to convert it from unit area outflow to total basin outflow rate. Second, a factor SLOSK2 is input to account for losses to deep storage and to provide an adjustment in the actual rate of outflow; SLOSK2 is a constant factor by which the above QO is multiplied. Third, the user can specify a minimum rate of base flow (BASMIN) as a constant value which is added to the product of QO, AREA and SLOSK2, and which used to facilitate calibration if a relatively constant base flow exists in the basin. Thus, baseflow rate is calculated at any time as:

$$\text{Baseflow} = \text{BASMIN} + (\text{AREA}*\text{SLOSK1}*\text{SLOSK2}*\text{SVOL})/1000 \tag{A10}$$

where BASMIN is a minimum rate of base flow (cubic meters per second),
AREA is total basin area (square meters),
SLOSK1 is a parameter (mm per second per mm),
SLOSK2 is a parameter (no units),
and SVOL is volume of water in the baseflow storage reservoir (mm).

SVOL is found at any time from its value at the beginning of the previous time step as:

$$\text{SVOL} = (\text{RINF} + \text{SVOLo}*(1-\text{SLOSK1}*\text{T}'))/(1+\text{SLOSK1}*\text{T}') \tag{A11}$$

in which RINF is the volume infiltrated over the time step (rain less runoff and initial abstraction) (mm),
SVOLo is the reservoir volume at the beginning of the time step (mm),
and T' is one half of the duration of a time step (sec).

Equation A11 arises from mass balance requirements, as a finite approximation of equations A6 and A7. (SVOL = SVOLo + RINF - (Q + Qo)*T)

Note on Input

The user must supply the model with an initial value for SVOL. Regardless of this value, the model will converge to the same base flow rate after some elapsed time from the start of the simulation, the length of which is dependent on SLOSK1 and to some extent on rainfall. Sensitivity runs with SVOL as a variable will show whether or not the starting value is important during the period of interest. It is probably better, however, to arrive at a starting value by means of equation A10; once other parameters are determined by calibration to a later part of the observed flow series, SVOL can be established from the flow observed at the start of the simulation period.

A-3 Runoff Quality Analysis

Washoff Method 1 (IWM=1 on the POLLUTANT RATES card)

If the user chooses this washoff method, the model calculates pollutant washoff rate directly for any time step according to the following relation:

$$WRATE = PCO*Q**PEX \qquad (A12)$$

where PCO is a rate constant,

PEX is a rate constant,
Q is flow rate (cubic meters per second,
and WRATE is a pollutant washoff rate (mass per second).

The rate of pollutant washoff is calculated separately for pervious areas, impervious areas, and baseflow and then summed to provide a total pollutant rate for the catchment. Note that basin area does not appear in the relation.

Note on Input

The user selects this method using IWM=1 on the POLLUTANT RATES card; parameters which the user must supply are PCO and PEX. WRATE must have units of (mass per second), and Q is supplied by the model in units of (cubic meters per second). Therefore PCO should be input in nominal units of (mass per cubic meter), with units of mass arbitrary (for example numbers of bacteria, kilograms of sediment etc.). PEX has no units. (Note that exact units of PCO must be (mass/ meters**(3*PEX)* seconds**(PEX-1) to produce WRATE in (mass/ second), but that values seldom appear this way in the literature.)

Since the pollutant component is calculated separately by the model for pervious area, impervious area, and base flow contributions, the user must input PCO and PEX values for each of these components using the POLLUTANT RATES cards.

Further, since there are two possible pollutants (first order and sediment) simulated by the model, the user must input values of PCO and PEX for each if both are to be simulated. (This corresponds to ICASE 1 through 6 on the POLLUTANT RATES card; a total of up to six PCO and six PEX values can therefore be input to the model.)

Washoff Method 2 (IWM=2 on the POLLUTANT RATES card)

The second available means of calculating washoff is to use:

WRATE = AREA * PBUILD * PCO * Q ** PEX (A13)

where AREA is provided by the model (square meters),
and PBUILD is available pollutant mass on the watershed surface (mass per square meter).

This relation can be applied to either impervious area or pervious area (but not base flow) runoff components. Thus, all of the terms in equation A13 pertain to one or the other, but not both, of these areas.

Note on Input

The user selects this approach using IWM=2 on the POLLUTANT RATES card. AREA is calculated by the model for pervious or impervious areas from information in the GENERATE command, as is Q; the user must input PCO and PEX. In addition, if this approach is selected, build-up parameters must be supplied as described below.

The model provides Q for each time step in units of (cubic meters per second), and AREA in (square meters). PBUILD is calculated by the model as described below in units of (mass per square meter). The user inputs PCO in nominal units of (per cubic meter) and PEX has no units. (Of course in both of the above relations, PCO technically must have units of (seconds**(PEX-1)/ meters**(3*PEX)) to provide a dimensionally correct relation, but values are seldom reported this way in the literature.)

As with washoff method 1 (equation A12), the user inputs the appropriate values of PCO and PEX for pervious or impervious areas separately in the POLLUTANT RATES cards. No base flow component can be calculated using washoff method 2 (equation A13), however, so there can be a possible total of only 4 POLLUTANT RATES cards using washoff method 2. These would correspond to ICASE 1 through 4; base flows (ICASE= 5 and 6) must always be simulated with washoff method 1 (equation A12; IWM=1).

A-3-3 Pollutant Build-up on the Land Surface

The value of PBUILD, pollutant build-up per square meter of land surface, is increased by the model during dry periods according to one of two possible methods described below. It is assumed that no build-up occurs during wet periods; when runoff exists, PBUILD is reduced according to the washoff equation (A14) described above. Reductions of PBUILD during runoff periods are calculated using a finite approximation:

$$\text{PBUILD2}=\text{PBUILD1}-\frac{(\text{WRATE2}+\text{WRATE1})T'}{\text{AREA}} \qquad \text{(A14)}$$

where T' is one half of the time step (seconds),
and subscripts refer to the beginning (1) and end (2) of the time step.

Substituting and rearranging,

$$\text{PBUILD2}=\frac{(\text{PBUILD1}-\text{WRATE1}*T'/\text{AREA})}{1+\text{PCO}*\text{Q2}**\text{PEX}*T'} \qquad \text{(A15)}$$

where Q2 is the flow rate for the present time.

All of the terms on the right hand side of equation A15 are known from the present or previous time step calculations.

Build-up Method 1 (IBM=1 on the POLLUTANT RATES card)

The first build-up method is expressed as a power function written here as:

PBUILD=PK1*Tc**PK2 ; PBUILD < PMAX (A16)

where Tc is the build-up time (days)
PBUILD is pollutant build-up on the catchment surface (mass per square meter),
PMAX is maximum allowable value of PBUILD (mass per square meter),
PK1 is a rate constant,
and PK2 is a rate constant.

Note on Input

This method is selected by setting IBM=1 on the POLLUTANT RATES card. Build-up is calculated separately for pervious and impervious areas, so the parameters PK1, PK2, and PMAX must be specified on two POLLUTANT RATES cards for each pollutant (sediment or arbitrary first order) simulated; this corresponds to the comments on numbers of cards made in the above notes on washoff method 2. The user must supply an initial value of Tc (DAYSIN), nominally the starting time since the last event, which provides the model with an initial pollutant build-up on the catchment.

PBUILD must have units of (mass per square meter), PK2 has no units, and T is provided by the model in units of (days).

PK1 therefore has nominal units of (mass per square meter per day). (Again, units of PK1 are (mass per square meter per day**PK2) to be dimensionally consistent, but values seldom appear this way in the literature.) PMAX has units of (mass per square meter). All mass units must be consistent with those used in the washoff equation.

As noted above, the initial value for Tc is input by the user. Thereafter, Tc is calculated using the inverse function of equation (A17):

```
Tc=(PBUILD/PK1)**(1/PK2)                          (A17)
```

and incremented each time step (DELTAT in days) by:

```
Tc = Tc + DELTAT                                  (A18)
```

Tc is therefore an equivalent time, and forces the build-up to begin after any event at a rate consistent with the amount of pollutant remaining on the catchment. The change of Tc during dry periods is the same as the change in clock time.

Build-up Method 2.

The second build-up calculation available to the user is the exponential type of relation expressed here as:

```
PBUILD=PMAX(1-exp(-PK1*Tc))                       (A19)
```

where PMAX is the limiting amount of pollutant on
the watershed (mass per square meter),
Tc is build-up time (days),
PBUILD is pollutant build-up on the
catchment surface (mass per square meter),
and PK1 is a calibration parameter (per day).

As with the first build-up method, build-up is assumed to occur only during dry periods. The initial value of Tc is input by the user, and thereafter calculated by the inverse function of equation A19:

$$Tc=\ln(1-PBUILD/PMAX)/(-PK1) \quad (A20)$$

and incremented each time step (DELTAT in days) as:

$$Tc = Tc + DELTAT \quad (A18)$$

Note on Input

This build-up method is selected with IBM=2 on the POLLUTANT RATES card. The user must supply an initial value for T (DAYSIN), nominally the starting time since the last event, which provides the model with an initial pollutant build-up on the catchment. The user also inputs PK1 and PMAX.

PBUILD is required in units of (mass per square meter), and T is provided by the model in (days), so the user must input PK1 in units of (per day) and PMAX as (mass per square meter). As with the other build-up method, the units of mass used must be consistent with those assumed in the washoff data input by the user.

APPENDIX B

SIMULATION OF CONTROL PONDS

APPENDIX B

SIMULATION OF CONTROL PONDS

B-1 Background

Pond Structure

The physical concept of the control pond used in this model has been kept quite general. This reflects the anticipated use of the model in a planning environment where a variety of alternatives may be tested. It also reflects the present lack of experience in installing control ponds for stormwater quality control in Ontario, and the consequent need to allow analysis of a wide variety of possible configurations; in the future, there will probably evolve a typical control pond installation, and the analysis will become more restricted. At present, however, there are few assumptions made as to the structural design of the control pond.

The physical nature of the pond is illustrated in figure B-1. As shown, the pond is assumed to have a single inlet (although the use of an ADD command prior to routing through a pond can circumvent this restriction), and a single outlet. There is provision in the model to allow simulation of a high level bypass, or all flows can be forced through the pond. Between events, a residual volume can be permitted, or the pond can be dry. In addition, a base

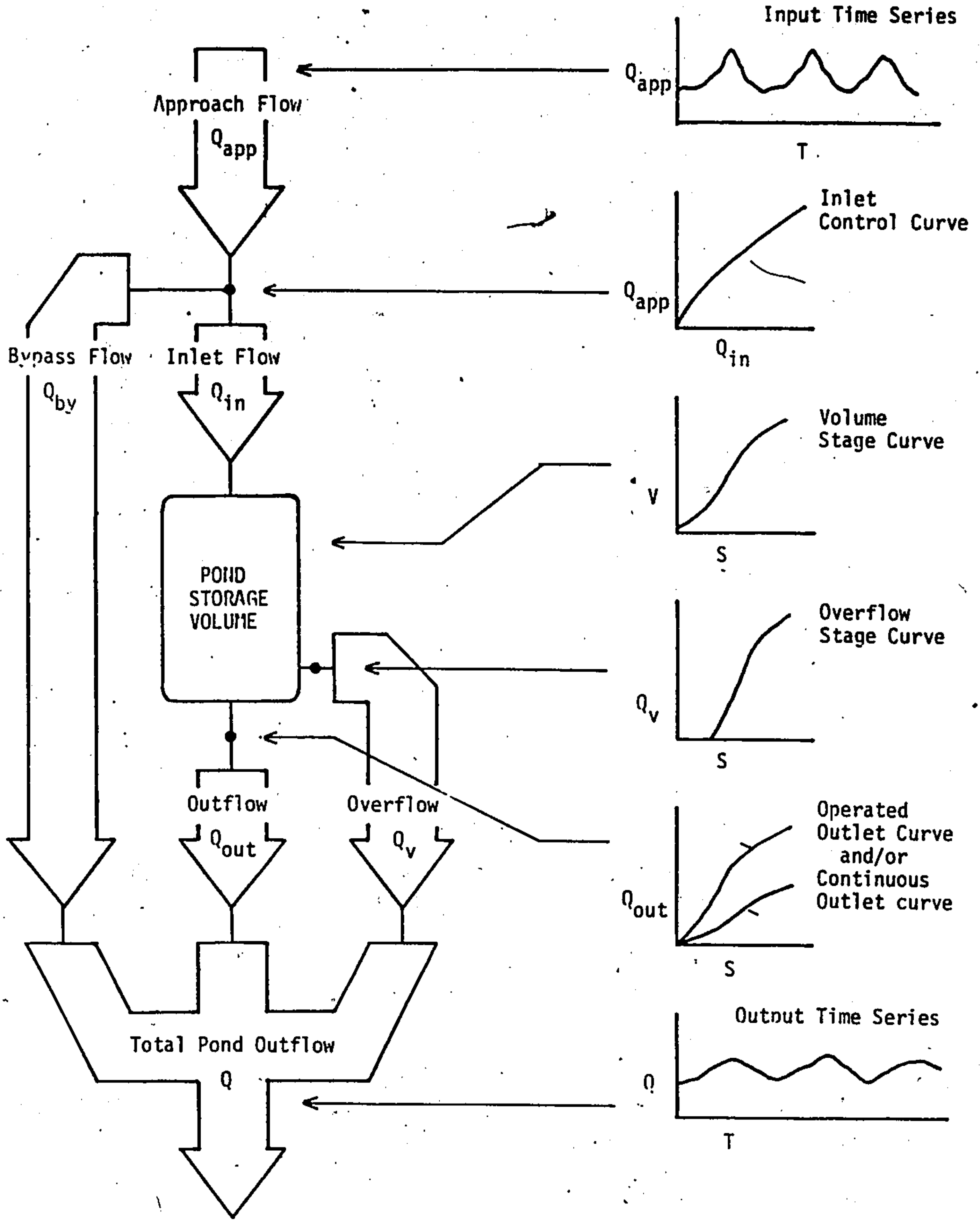


FIGURE B-1 PHYSICAL CONCEPT OF CONTROL POND FLOW ROUTING through-flow rate can be maintained in dry weather. It is possible to represent any gravity type pond outlet structure where backwaters are not of concern, since the outlet structure is input as a flow/stage curve. Similarily, an over-flow curve can optionally be input to the model, to represent weir flow or any other over-flow condition in the event that the pond maximum storage capacity is exceeded. To reflect possible operation as a batch facility, there is provision to incorporate a detention time in the model; flows not exceeding the maximum storage capacity will be retained until there is a dry period longer than the stated detention time, and then released.

Pollutant Removal

The model presently contains two mechanisms for reducing the amount of pollutant eliminated in the detained water volume. These are removal by first order decay, and removal by discrete settling. In both cases, removal is computed using the assumption of complete mixing, but the user is given the capability of dividing the control pond into any number of contiguous compartments. This capability allows the user to approximate the effects of an imperfect reactor, and in addition to approximate a plug flow reactor by using a large number of sub-compartments.

B-2 Quantity Analysis

Flows to be routed through the pond are contained in a file created by the model in a previous GENERATE, STORE, REACH, POND, or ADD command, and may or may not contain quality as well as quantity (flow rate) information.

B-2-1 Pond Flow Components

The routing of water volumes through the control pond is achieved using a routing scheme consistent with the pond as depicted in figure B-1. As shown, there are five characteristic curves and several parameters which can be used to simulate flows through the pond. The most important of these are:

1. Inlet Control Curve (Input as QAPP(i) and QIN(I) pairs on the POND cards.)

This curve is user-specified, and allows flows approaching the pond to be split, with part of the flow bypassing the pond and part entering the pond. The user can omit this curve, in which case all approach flows are fed to the pond. If an approach curve is input, it can be used to specify either a constant or variable flow split. Also, the approach curve can specify complete capture up to some limiting flow, and entire or partial bypass of flows exceeding this rate. The reverse case, with no capture at all until some limiting flow is achieved, can also be simulated.

The purpose of this curve is to allow the user to simulate conditions where the pond is designed to have excess flows bypass the pond, or where flows less than some minimum are not controlled, or where other considerations lead to control over the approach flow. An example application of an inlet control is contained in Volume 1.

2. Volume/Stage Curve. (Either input as STAGE(I) and VOLUME(I) pairs on the POND cards, or calculated from STAGE(I) and AREA(I) pairs input on the POND cards.)

This curve provides the storage information used in simulations. If input as a volume/stage curve linear, interpolation or extrapolation is used to calculate pond volume at any stage. If the user inputs only an area/stage curve, a volume stage curve is calculated as:

$$V(S1) = 0 \qquad (B1a)$$

$$V(S2) = (S2-S1)*(A1+A2)/2 \qquad (B1b)$$

$$V(S3) = (S3-S2)*(A2+A3)/3 + V(S2) \qquad (B1c)$$

and so on over the range of S and A,

where Sj is an input stage (m.),
Aj is the area input for a stage of Sj (sq. m.),
and V(Sj) is the area calculated for a stage of Sj (cu. m.).

3. Operated Outlet Curve (STAGE(I) and FLOW(I) pairs input in the POND cards.)

This curve, which is mandatory, represents the primary outflow structure from the pond. This curve can be used to maintain a continuous throughflow from the pond (if TDET is set to zero) or can be used to represent a gated structure which is operated according to a design batch detention time (if TDET is non-zero) as described below.

Note that to allow simulation of 'wet' ponds, the starting elevation of both this and the continuous outlet curve described below (point 5.) can be above the minimum point on the stage/volume curve.

4. Detention Time (Input at TDET on the POND cards.)

If desired, the user can incorporate a design detention time in the simulation; this allows simulation of batch operated ponds (as opposed to the continuous operation otherwise assumed). Batch operation is defined here as a procedure whereby volumes of water entering the pond are held for a specified time before release.

To do this, the model maintains a 'timer' which records the age (time since end of last event) of water in the pond. When the 'timer' shows all the water in the pond has been there for at least a period of TDET, a release of water through the operated control curve (see point three above) is possible. The following operation rules are applied to govern this process:

A. Provided that the 'timer' registers greater than TDET, when flows enter the pond at a rate less than a specified minimum (QBAS on the POND cards) the operated outlet curve (see point 3. above) is 'opened'. Flows can therefore leave the pond by means of this curve.

B. When flows enter the pond at a rate in excess of QBAS, the operated outlet curve is 'closed' and the timer is set to zero. Outflows from the pond are therefore not possible through the operated outflow curve.

C. When flows entering the pond drop to less than QBAS after a period where QBAS was exceeded, the 'timer' is started. The operated outlet curve is left 'closed'.

Regardless of the status of the operated outlet curve, it is possible for flows to leave the pond through either or both of the overflow curve or the continuous outflow curve if stored volume is sufficient and if these are coded into the POND data cards.

5. Continuous Outlet Curve. (Input as FLOW(I) and STAGE(I) pairs on the POND cards.)

This curve represents a secondary outflow structure, which may omitted if desired. Outflows from the pond are calculated using this curve alone if the operated outlet curve is 'closed'; if the operated curve is 'open', flows are released through both the operated and the continuous outlet curves.

This secondary detention curve allows the user to specify some small throughflow regardless of the detention period, since it is conceivable that there will be a small discharge permitted at all times. This is particularly the case if the receiving water has some assimilative capacity, or if a significant baseflow is present.

6. Overflow/Stage Curve. (Input as FLOW(I) STAGE(I) pairs on the POND cards.)

This curve allows the user to simulate a pond overflow condition in the event that the available storage volume in the pond is exceeded. The curve functions, if input, regardless of the status of flows from the operated and the continuous outlet curves.

Where any of the above curves are exceeded, a linear extrapolation is made based on the extreme two points on the curve as coded.

B-2-2 Flow Routing Calculations

The model solves the continuity routing equation:

$$dV/dt = QIN - QO1 - QO2 - QV \qquad (B2)$$

where V is the pond volume,
QIN is the total inflow rate,
QO1 is the outflow rate through the operated outlet structure,

QO2 is the outflow rate through the continuous outlet structure,
and QV is the overflow rate.

The above three outflow components (QO1, QO2, and QV) can combine in at most two distinct ways, depending on what the user has input. These are:

$$QOUTA = QO2 + QV \qquad (B3a)$$

and

$$QOUTB = QO1 + QO2 + QV \qquad (B3b)$$

where QOUTA is the outflow if the operated gate is closed,
and QOUTB is the outflow if the operated gate is open.

Equation B3 is true even if some of the components (i.e. QO2 and/or QV) are zero. The model therefore calculates a combined outflow curve for each of these two cases, and uses whichever one applies at any time step.

The combined curves are calculated by the model and contain every stage value input by the user. The model first obtains and orders every STAGE which appears on any of the input curves (stage/area, stage/flow (operated), stage/flow (continuous), stage/volume, stage/overflow) supplied by the user, dropping duplicates. Flow and volume data are then found by linear interpolation of each of those curves for each stage. Finaly, a total QOUTA and QOUTB at any STAGE is found by summation. The exact form of the user input data is therefore preserved in the combined outflow curves.

The solution of equation B2 is otherwise done in a fairly typical way. Taking a finite approximation of B2:

$$2*(V2 - V1)/T = (QIN1+QIN2) - (QOUT1+QOUT2) \qquad (B4)$$

where V is volume (cu. m.),
T is the time step (secs.),
QIN is inflow rate to pond (cu. m. per sec.),
QOUT is outflow rate (QOUTA or QOUTB as appropriate) from pond (cu. m per sec.),
and subscripts refer to the beginning (1) and end (2) of a time step.

This is done by defining a parameter SI which is a function of stage:

$$SI = V/T + QOUT/2 \qquad (B5)$$

where V and QOUT are as defined above,
and all terms apply to the same stage.

Solution of pond outflow is then done at any time by solving:

$$SI2 = SI1 + (QIN1 + QIN2)/2 - QOUT1 \qquad (B6)$$

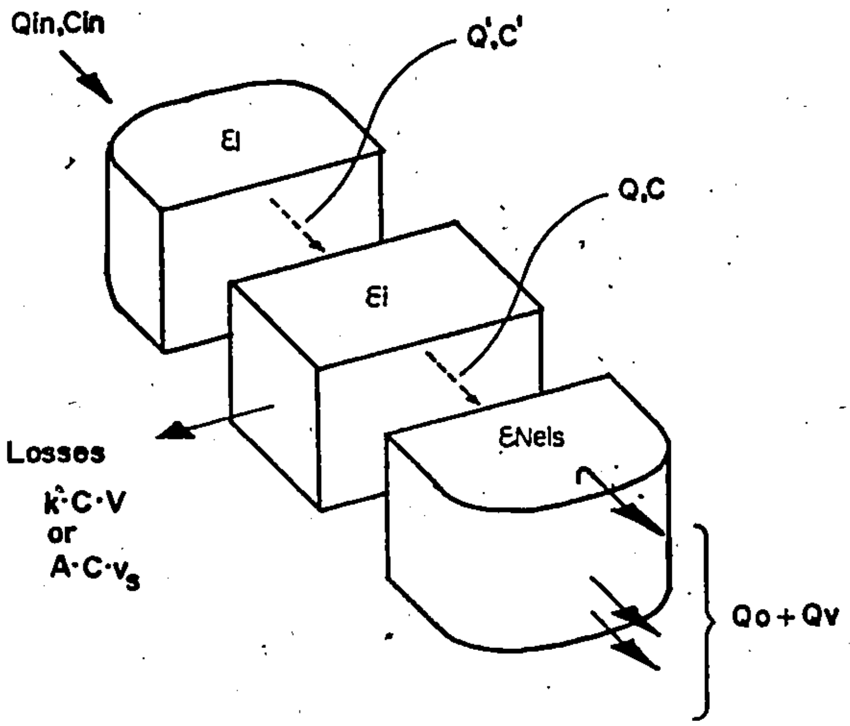
and then finding the stage corresponding to SI2 by interpolation of the SI curve (B5). This stage allows calculation of pond outflow rate and volume at the end of the time step by interpolation of the QOUT and volume/ stage curves maintained by the model.

This solution technique ('storage indication') is popular in simulation of reservoirs for hydrologic studies, and essentially implies that the effect of surface slope has no impact on outflows from the reservoir. For the relatively small ponds which are envisaged here, this is likely to be a reasonable assumption.

B.3 Quality Analysis

The control pond model simulates pollutant removal by routing constituents through a series of well mixed reactors of equal volume and depth (figure B-2). The user specifies the number of reactor elements, and the model determines individual element characteristics from the curves specified for the overall pond. Routing through each element is achieved by a numerical solution a conservation equation for a completely mixed reactor, which is written here as:

$$dVC/dt * C = QiCi - QoC - Losses \qquad (B7)$$



(Each cell 'ε' has

- Volume = V
- Area = A (horizontal)
- Concentration = C)

Fig. B-2 Segmentation of Control Pond Into Similar Reactors where V is the reactor volume,

C is the concentration in the reactor and outflow,

Qi is the inflow rate,

Ci is the inflow concentration,

Qo is the outflow rate,

and Losses are due to sedimentation or decay of pollutants in the reactor.

In this model, a variable volume reactor is assumed, and flows and volumes at any time 't' are determined from the water quantity calculations. Losses may be calculated in one of two ways. For a first order decay constituent:

$$Losses = k*C \qquad (B8)$$

where k is a decay constant.

For the case of discrete settling, losses are also expressed in a first order form:

$$Losses = vs*C*A \qquad (B9)$$

where vs is the effective particle settling velocity,

and A is the horizontal surface area of the pond element.

The principle difference between these two equations is that in B8, the coefficient k is a constant. In B9, vs is constant for a given size fraction of sediment, but A is a variable which is a function of depth of water in the pond.

Note on Sedimentation

Equation B9 results from an assumption of complete mixing, and is somewhat different from some alternatives commonly proposed. Often, sedimentation is simulated in a form which implies conditions equivalent to quiescent settling; all particles move uniformly downward at a constant velocity, and concentration in the vertical plane is therefore either zero (above the particle which started at the uppermost postition in the water column) or constant (below that particle). Sediments remaining in the water flowing from the pond are therefore those particles which have not had time to fall past the depth of the outlet structure by the time they have travelled the length of the pond to the outlet end. This mechanism, familiar to any engineer involved in sedimentation basin design for waste water treatment, is effective and reasonable in relatively quiescent conditions, or where particles have relatively high fall velocities.

At the other end of the scale, it is possible to suppose complete mixing of particles in the vertical plane; removal of sediments can then be conceptualized as occuring when particles impinge on the bottom without re-suspension. Removal in this case is a function of the type of mixing and loss equations represented here by equations B7 and B9.

In reality, the process of sediment removal in a pond is likely to be more involved than either of the above approaches would suggest; there are a number of sediment transport models, for instance, which approach this problem in a much more complex manner than is described here. For the purposes of the present version of the model, it was felt that adopting a complex transport model to simulate sediment behaviour in a pond was not justified either by objectives (a planning level analysis of many alternatives) or by experience (there is as yet an incomplete understanding of ~~basic~~ aspects of pond behaviour).

The completely mixed approach used for sediment removal simulation was chosen for several reasons:

- It represents hydraulic conditions consistent with the completely mixed assumptions used for the first order decay pollutant.

- The pond is likely to experience a high degree of turbulence when subjected to inflows of the magnitude and variability anticipated from stormwater runoff, which makes quiescent settling a questionable assumption.

- Finer fractions in particular will tend to react more to turbulence, and may be better approximated as completely mixed than in a quiescent settling situation. Since the finer fractions may be considered to represent a large if not major part of associated pollutants, a completely mixed assumption may be preferable.

o In a situation of otherwise similar conditions, the vertical mixing assumption is conservative compared to the quiescent settling assumption.

The question of what constitutes an effective settling velocity is difficult no matter what mechanism is postulated for simulation purposes, since the physical situation in the pond is not likely to be well represented by the equipment typically used to measure fall velocities in the lab. The effective settling velocity is represented by the difference between a downward fall velocity and an upward tendancy which is a function of turbulence and concentration gradients; other factors such as hindering enter the problem as well. As a result, the effective fall velocity may not only be less than the discrete settling velocity, but may be time varying. (Note that the vertical mixing assumption used here will result in a removal rate which decreases as the mass of sediment in a given vertical section decreases; this may be preferable to the discrete settling mechanism which provides a constant areal deposition rate from an slug of initially uniform concentration.) Until more data is available to support improved alternative assumptions, it appears that the most reasonable alternative for simulation in the context of the above model is to use a settling velocity 'vs' equal to the discrete settling velocity, and to test the sensitivity of model results to changes in 'vs'.

Solution of the Quality Routing Equations

Writing equation B7 in a finite form for one element:

$$\begin{aligned} 2*(V2*C2 - V1*C1) = {} & (C1'*Q1' + C2'*Q2')*DELT \\ & -(C1*Q1 + C2*Q2)*DELT \\ & -(C1*L1 + C2*L2)*DELT \end{aligned} \qquad (B10)$$

where V is the element volume,
C is concentration,
L is a loss term,
Q is flow,
DELT is the time step size,
subscripts refer to the beginning (1) and end (2) of the time step,
and primes (') refer to the upstream face of the element.

Terms not primed refer to the downstream face of the element (Q,C) or to the interior volume of the element (V,L,C); note that since a completely mixed element is assumed, C is the same at the interior and downstream face of the element. The left hand side of B10 represents changes in storage. Other terms from left to right are inflow from upstream, outflow to downstream, and removal by sedimentation or decay.

The term L in B10 depends on the substance. For settleable material,

$$L = AREA*vs \qquad (B11)$$

where AREA is the horizontal
area of the element,
and vs is the effective settling
velocity of the sediment,

For the first order decay pollutant,

$$L = VOL*k$$

where VOL is the volume of the
element,
and k is a constant parameter.

k and vs are provided by the user in the START card as DECAYK and SEDSET respectively. The element volume and area are calculated by the model each time step as (1/NELS) times the pond volume and area; NELS is provided by the user on the POND cards.

Note on Input

The model obtains overall pond surface area either from an AREA/ STAGE curve input by the user, or by calculating an area/ stage curve from the VOLUME/ STAGE curve which was input. In the latter event, the inverse of equation B1 is used. This relation was chosen for consistency with the approach used in calculating volume from AREA, described in section B.2. In some cases, the input data can cause the model to calculate an area which is very large at one of the stages. This large area can sometimes force a decrease in area calculated at the next higher stage; if this occurs, the model uses the last calculated area instead, so as to prevent a pond area curve which decreases with elevation. A warning message "ADJUSTED AREA" is printed if this has happened. If this is a problem, the cure is to (i) use a finer elevation step or (ii) input an AREA/ STAGE curve directly.

Flow into and out of each element is distributed evenly among the elements so that flow rates and volumes overall match the information computed in the pond flow routing. It is possible for instance to have zero flow rate at one end of the pond, and a positive or negative flow rate at the other; an example of this would be if the outlet to the pond is closed, and flows are entering. This is accounted for in distributing flows into and out of each element.

Equation B11 is solved in the model for first order pollutants as:

$$C2 = (KA*C1 + (Q1'*C1' + Q2'*C2')*DELT)/KB \qquad (B12)$$

$$\text{where } KA = (2-DECAYR)*VOL1 - Q1*DELT$$

$$\text{and } KB = (2+DECAYR)*VOL2 + Q2*DELT$$

$$\text{in which } DECAYR = \exp(DECAYK*DELT)$$

using consistent units.

For settleable material, B11 is solved as:

```
    C2 = ((KA -AREA/vs*DELT)*C1 (B13) + (Q1'*C1' +
             Q2'*C2')*DELT)/ (KB + AREA*vs*DELT)
           where KA = 2*VOL1 - Q1*DELT
             and KB = 2*VOL2 + Q2*DELT
```

for each size fraction, using consistent units.

In both of the above, if the divisor is zero (which can only happen if the element volume and flow are both zero) C2 is set to zero. Equation B11 and/or B12 as appropriate are solved in sequence for all elements in the pond, starting at the top end.

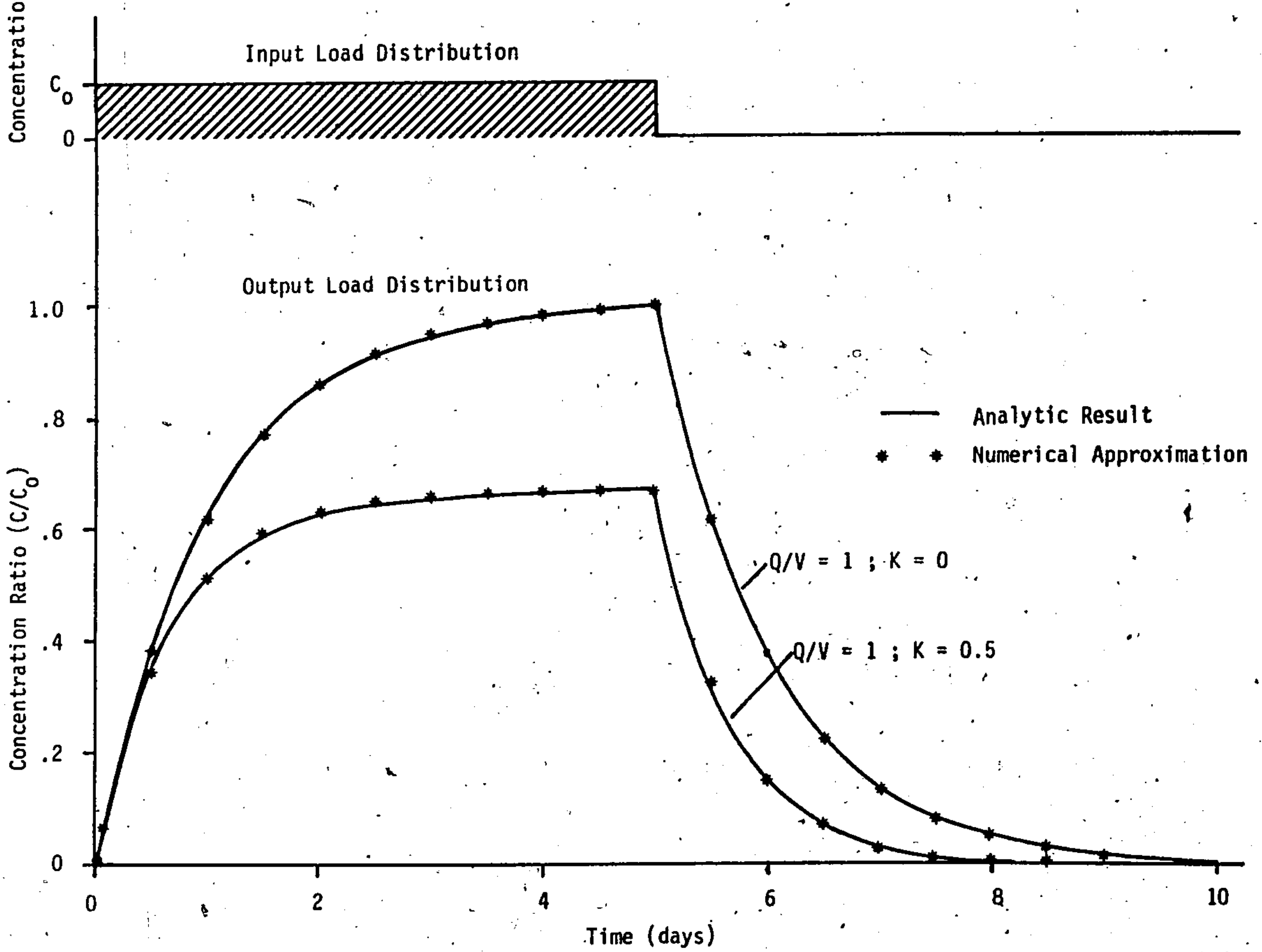
The solution technique applied here can be used to simulate a number of mixing conditions. For a single element reactor, the solution is simply that of a completely mixed reactor. For several elements, the solution approximates that of the imperfect advective/dispersive reactor, while for a large number of elements, the solution approaches that of the plug flow reactor. Other mixing conditions, such as short circuiting, are possible but were not considered in the present version of the model. The model code as established, however, would be amenable to extension to include short circuiting or possibly other mixing conditions should the need arise. Alternatively, the pond bypass curve described above could be used to approximate a short circuiting effect by having an appropriate fraction of flow bypass the pond.

Testing

Testing of the quality routing algorithm was accomplished by comparing the model result to a known analytic solution for a special case of pond inflow. The case chosen was that of a time varied input concentration, in the form of a step function, routed during a period of constant throughflow. The object of this testing was to check that the finite difference solution as coded in the model does in fact represent the reactor cell which was assumed in the model development. The case used for testing was chosen because it represents one of the few time varied conditions for which an analytic solution exists.

Results of the testing are presented in figure B-3. As shown, the model was checked against a routing of both a conservative and a non-conservative (first order decay) substance. The results in both cases agree very closely, and it is concluded that the model solution algorithm for the pond is appropriate for its intended purpose, which was to solve for concentration changes which would be engendered by one or more completely mixed reactors in series.

FIGURE B-3 VERIFICATION OF CONTROL POND NUMERICAL SOLUTION AGAINST EXACT SOLUTION



APPENDIX C

RIVER ROUTING

APPENDIX C

RIVER ROUTING

C-1 Introduction

This appendix describes river channel flow routing, and also documents a method of instream quality routing not described in the main text. This additional routine, provided for analysis of transport and modification of pollutants by the receiving stream, is essentially a cell model. Pollutants are routed along a single river reach composed of a series of reactor cells or elements.

This alternative was chosen as a basis of instream simulation for several reasons. The model is capable of solving the transport and modification of settleable or decaying quality constituents in a stream reach of arbitrary section, and is amenable to possible future extensions to include processes other than first order settling and decay. The method is economical in terms of computer storage and computation time, and similar transport models have been applied in water quality analysis in the past. The model can also be operated with a mimium amount of data about the physical system. Thus, the overall level of sophistication and data requirement, as well as cost, was deemed appropriate for and consistent with the general level of detail adopted for the model. Finally, as can be seen below, choice of the river routing schemes applied allowed use of many of the solution algorithms developed in the pond routing scheme, since the basic equations are of the same form (although application in simulation may differ). As with any of the simulation routines contained in the model, however, the user can supply an alternative river routing algorithm if necessary.

C-2 Flow Routing

The model uses a kinematic approximation to route flows along a river reach. The basic scheme is depicted in figure C-1. As shown, the model treats the entire river reach as a single storage volume represented by a characteristic depth volume curve. Hydraulics in the reach are represented either by a Manning flow equation:

Qo = Ax * 1.49/RN*SF**0.5*R**0.6667 (C1)

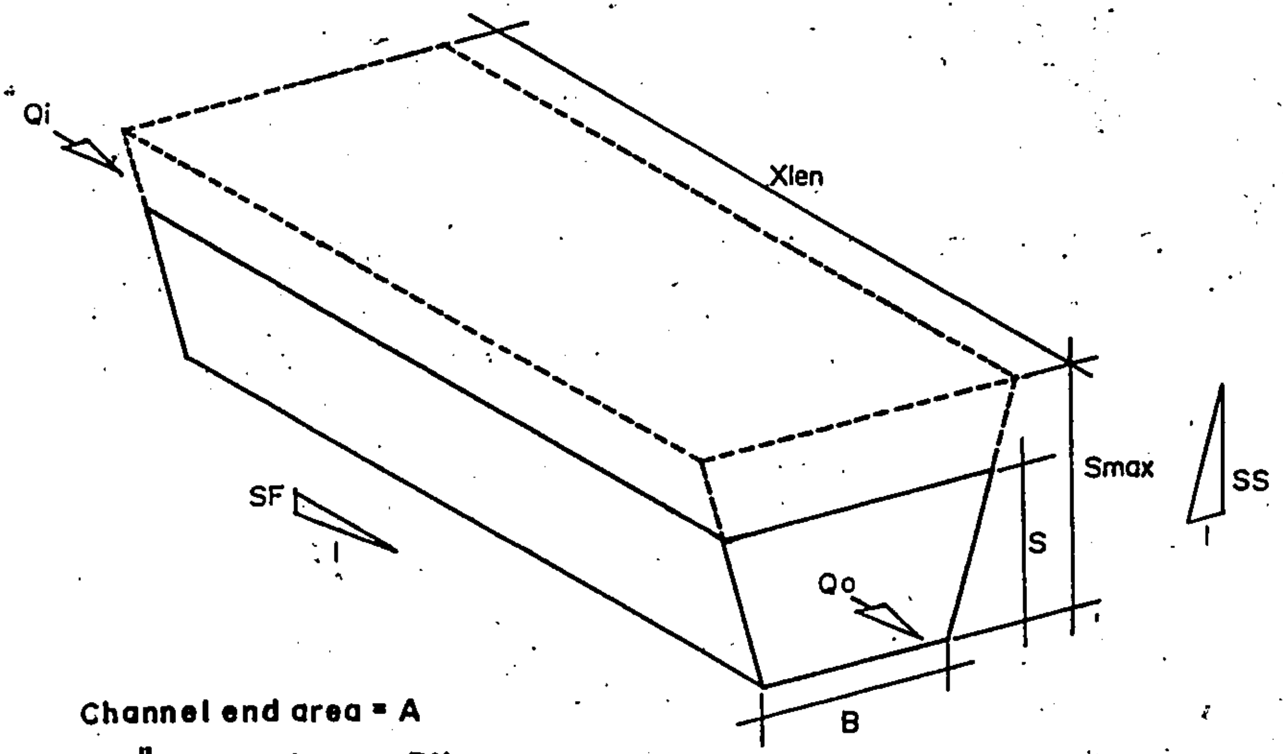
where SF is the channel friction slope,
RN is the Manning roughness 'n',
R is the hydraulic radius,
and Ax is the channel end area,

or by a rating curve:

Qo = COEF*D**EXPON (C2)

where D is depth of flow in the channel,
COEF is a coeficient,
and EXPON is an exponent.

(Channel section may be trapezoidal or arbitrary)



Channel end area = A
" roughness = RN
" hydraulic radius = R

$$Qo = A \cdot R^{2/3} \cdot SF^{1/2} \cdot RN^{-1}$$

— or —

$$Qo = COEF \cdot S^{EXPON}$$

Fig. C-1 Definition Sketch for Channel Flow Routing

Where the Manning relation is used, a trapezoidal channel is assumed and the user supplies bottom width (B), reach length (XLEN), Manning roughness (Rn), and side slope (SS) data in the REACH cards.

In either case, the model calculates depth/flow velocity and depth/section area curves for the channel, and uses these curves in the subsequent flow routing.

Note on Input

The second relation (C2) has been provided primarily for use in situations where the user has determined flow relationships by some means outside of the model. Available HEC-2 data, for example, might lead to the use of the equation C2 in preferance to C1. If Equation C2 is used, the user inputs the required coeficient (COEF) and exponent (EXPON). The user also provides the model with volume and area curves explicitly as (AREA(i), STAGE(i)) and/or (VOLUME(i), STAGE(i)) pairs. Calculation of AREA/STAGE curves from VOLUME/STAGE curves or vice versa is done in exactly the same way as in the pond routing described in appendix B if only one or the other is input. End area is defined as volume divided by length.

Flow routing through the channel is achieved by resolving the flow/stage relation with the storage balance in the channel. The model selects a depth and flow rate at the end of a time interval such that the change in volume in the reach over the interval corresponds to the flow into and out of the reach over the interval.

The mass balance which is applied is:

$$2*(V2 - V1) = (I2 + I1 - O2 - O1)*\ DELT \qquad (C3)$$

where V is the volume in the reach,
I is the inflow to the reach (all sources),
O is the outflow at the downstream end of the reach,
DELT is the time step,
and subscripts refer to the beginning (1) and end (2) of the time step.

Where flow is expressed according to Manning's equation and the channel is restricted to a trapezoidal shape, or in fact in most cases where functional relations are used to describe flow in the channel, mathematical techniques can solve for equation C3 by direct operation on the functions. This has been done in SWMM and other models. However, if the flow curve is arbitrary, other methods must be used. Due to this factor, to economize on computer time, and because of the mathematical consistency between equtions C3 and B2, the model uses a different approach.

In the proposed model, the solution technique for the river routing is exactly similar to that applied in the pond routing. The model in fact uses the same algorithm to solve for flows in both the POND and REACH routines. (This has been described in section B.2 above.)

This approach to routing is possible because (i) the fundamental differential equation assumed in both cases is the same, and (ii) the difference between the two cases therefore reduces to different volume/depth and flow rate/depth relations. The model can use the same techniques for solving either case, and does. (It is interesting to note that the overland runoff routines used by SWMM are also of this type, and could equally well be solved using the same routines.) This does not of course mean that the pond routing and reach routings will have the same effect on flows, or that the two are physically the same. It simply means that the conventional uniform flow assumption and reservoir routing assumptions have certain characteristics in common; it is the choice of rating curves which provides the mathematical difference between the two in common usage.

C-3 Pollutant Routing

The technique applied to pollutant routing is similar to the pond model techniques, except that the equations applied to solve constituent changes in an element incorporate the possibility of lateral sources. The basic scheme is illustrated in figure C-2, and the fundamental equation solved is:

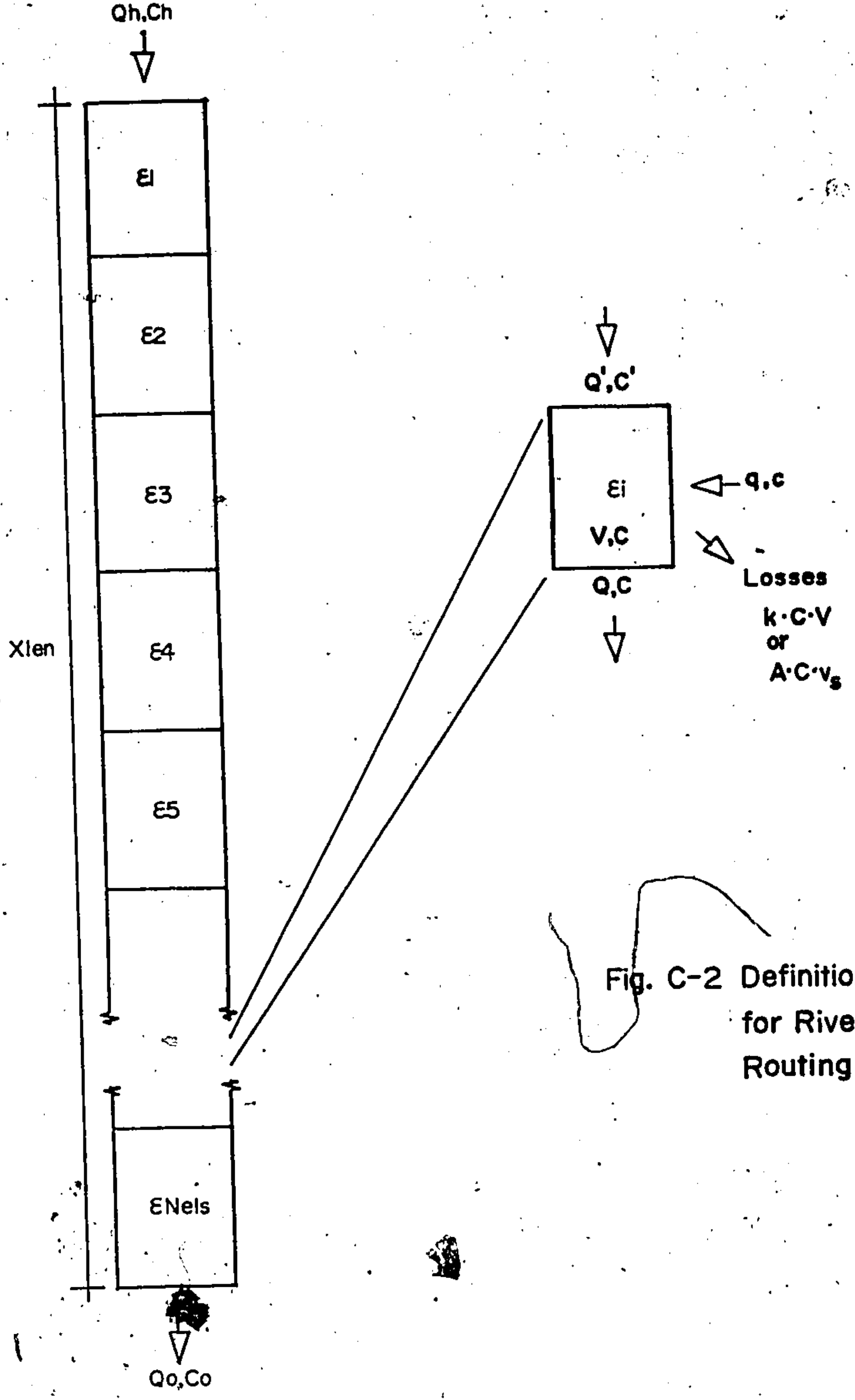


Fig. C-2 Definition Sketch for River Quality Routing

```
2*(V2*C2 - V1*C1) = (C1'*Q1' + C2'*Q2')*DELT      (C4)
                     -(C1*Q1 + C2*Q2)*DELT
                       -(C1*L1 + C2*L2)*DELT
                         +(c1*q1 + c2*q2)*DELT
```

where V is the cell volume,
C is concentration,
Q is flow along the cell,
q is lateral flow into the cell,
c is lateral flow concentration,
DELT is the time step,
subscripts refer to the end (2) and beginning (1) of the time step,
and prime (') refers to the upstream face of the cell.

The loss terms, L, are as defined in appendix B. Also as defined there, the unprimed terms C,V and L refer to values within the cell, and the other terms Q and C refer to values leaving the cell; each cell is completely mixed. The added terms q and c represent lateral inflows along the reach.

As with the flows, the intentional similarity of the pollutant routing algorithms in the POND and REACH routines makes it possible to use common computer code for solution. The primary mathematical difference between the two is that in the case of the REACH command, lateral inflows to each cell can exist. Other factors, such as lack of residual volume in the reach, no control of outflows by detention curves, and no bypasses also differentiate the two.

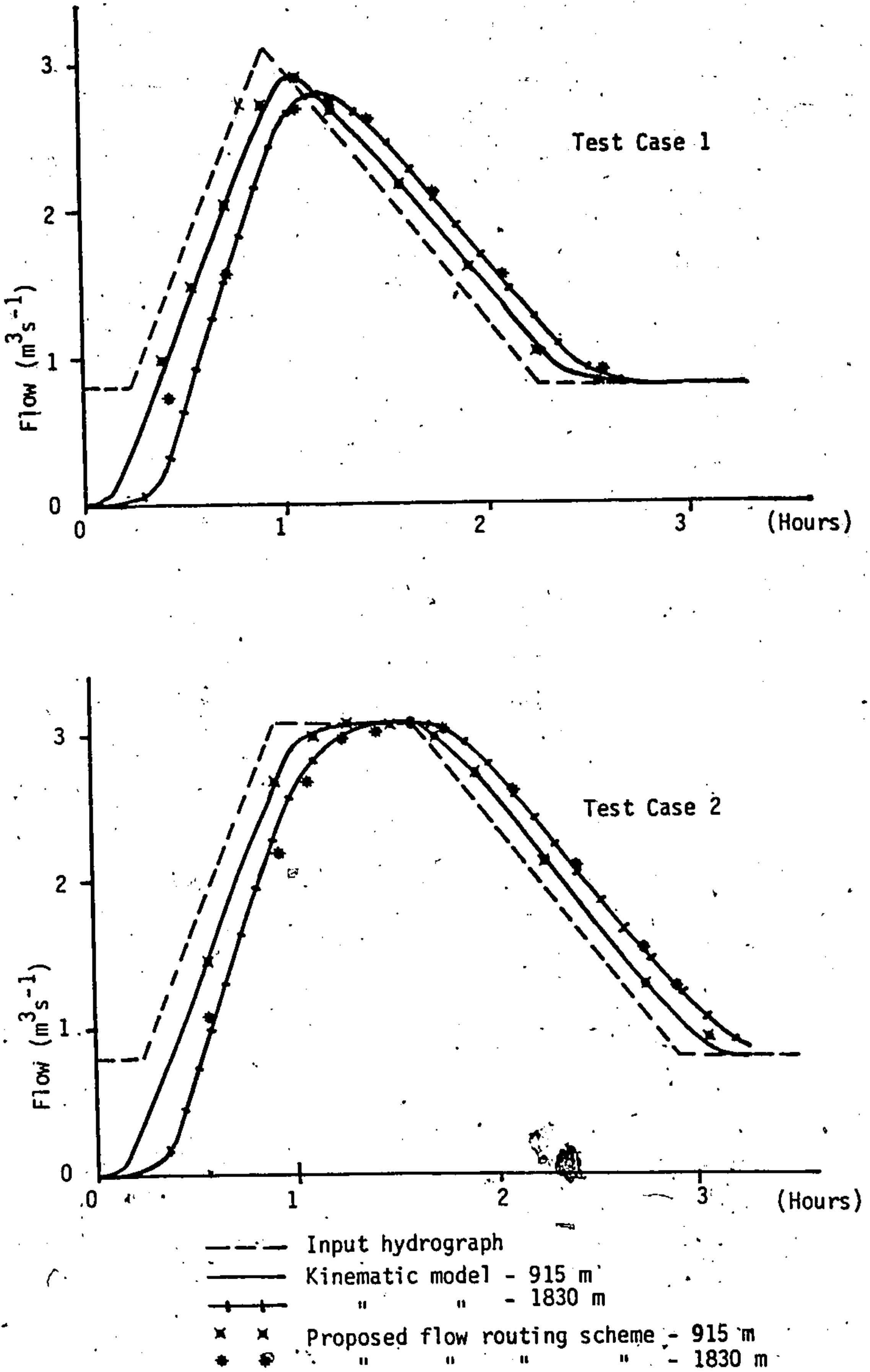
As in the quantity routing, this does not mean that there is no difference between the river reach and the control pond. The choice of input data will differentiate between the two. There is precedent for use of the cell model in both river and pond routing, and this will not be discussed here. It is noted, however, that in the case of river routing the choice of numbers of cells will reflect the anticipated (or observed) longitudinal dispersion behaviour in the river, while in the case of pond routing, the number of cells will reflect the mixing behaviour within the pond. It is expected that ponds will typically be simulated as either having one (perfect mixing), a few (imperfect mixing) or many (plug flow) cells, while the river will have an intermediate number of cells (advective/ dispersive transport). In the event that the mixing model presented above is not appropriate for the system being considered, other models can be substituted by the user.

Testing

To ensure that the flow routing scheme in this algorithm produces appropriate results, the model was tested against simulations made with a proven kinematic routing scheme. The object of the testing was primarily to establish whether the flow routing algorithm as coded in the model does in fact produce results consistent with the fundamental equations assumed in model development.

To do this, two test cases were chosen. One used a triangular input hydrograph, and the other a trapezoidal hydrograph; both tests were carried out in a regular channel of circular section, sloped at 0.001 m/m, with a nominal diameter of 1.83m. The routing scheme used as a benchmark test was a four-point implicit finite difference scheme developed by Kassem (1982). The distance of the routing was two kilometers; results of the two models were compared at the 915m and 1830m points. Figure C-3 provides the results of the testing. As shown, the two models are in close agreement at each of the two points depicted in both of the cases tested. It is concluded that the chosen routing scheme will produce results consistent with other flow routing models of the same type.

FIGURE C-3 VERIFICATION OF FLOW ROUTING SCHEME AGAINST KINEMATIC SCHEME BY KASSEM (1982)



APPENDIX D

MODEL STRUCTURE AND INPUT DATA

APPENDIX D

MODEL STRUCTURE AND INPUT DATA

D-1 Introduction

This appendix provides details of model structure, function, and input requirements not included in the main body of this thesis.

D-2 Scope of the Model

The overall scope of the model is discussed below. A detailed accounting of the model capabilities and limitations is made in section D-6 where the various model functions are presented, and in Appendices A through C where some of the basic algorithms are documented.

(i) The model is capable of continuous simulation of rainfall/runoff and of detention pond or river routing over a period of indefinite length.

(ii) Constituents which can be simulated are:

1. stormwater runoff,

and optionally one, none, or both of

2. one pollutant exhibiting first order decay,
3. sediments (in up to 5 size fractions)

The above conventions were adopted for reasons described in chapter 3. Essentially, they maintain model simplicity and economy without unduely restricting its usefulness; other pollutants (especially heavy metals) can be related to sediments on a mass fraction basis, so simulation of sediments alone indirectly provides scope for assessment of a variety of pollutant impacts. It is noted that pollutant or flow records obtained elsewhere (monitoring or other simulation models) can be routed through ponds or river reaches by the model provided that discrete settling or first order decay apply.

(iii) Components of a physical system which can be simulated are:

1. catchments,
2. detention ponds,
3. river reaches and junctions, and
4. bifurcations (flow splits).

(iv) In principle, a system of any combination and any number of the above components can be simulated, provided that:

1. each component of the system and the system as a whole can be described as 'one-way' or 'non-feedback'.
2. at most six components are simultaneously active.

Looping networks, or ponds where outflow depends on water depth in the receiving channel, are examples of situations which might not be well represented in this model. A dendritic river network with channels adequately represented by uniform flow conditions will typically be appropriate for application of the model.

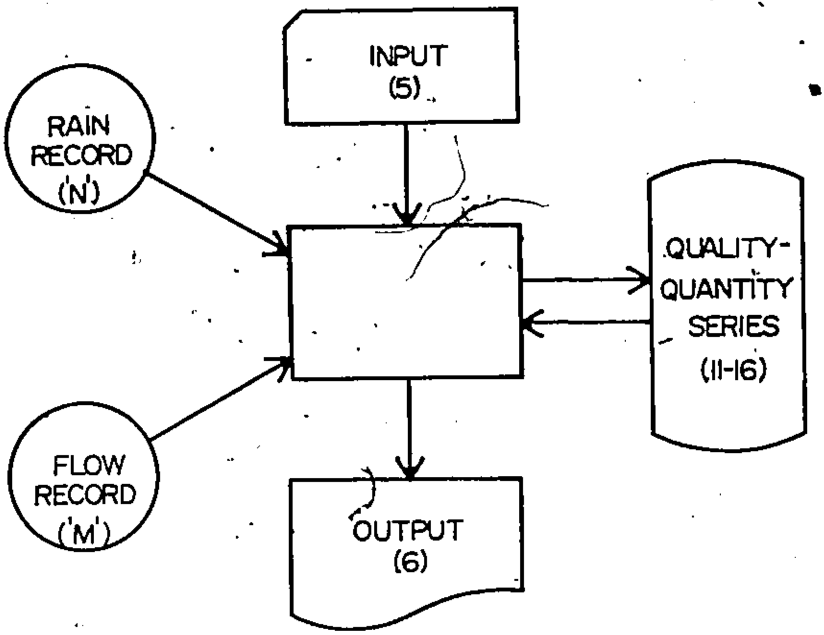
The restriction to six components is the same limitation imposed by HYMO and OTTHYMO, and is not likely to be a problem in practice. A reader familiar with HYMO and OTTHYMO will note that the above points are similar to, but less constraining than, the limitations imposed on those two models. The widespread use of HYMO and OTTHYMO in medium and large river basins in Ontario suggests that the proposed model will be suitable for use in simulation of many river systems in Ontario. In any case, the output flow series created by the model can be kept on disk for subsequent use, so in principle the ultimate limitation on the number of components becomes the amount of available disc or tape space.

D-3 Computer System Requirements

The model code is written in FORTRAN, and the model was developed on an AMDAHL computer under a CMS (VM/370) environment using a FORTRAN IV G compiler. No unusual FORTRAN extensions were used in this development, but some conventions used may not apply directly to all installations. In particular, some of the IBM scientific subroutines are invoked; further, there are no file opening or closing statements in the code.

The code itself consists of a main program and 27 subroutines, and depending on the system it is installed in, requires about 130000 bytes of code and data space to run. In addition, the model requires direct access disk or tape units to contain the synthesised runoff and pollutant time series. This direct access storage must be organised into sequential access files designated as FORTRAN devices 11 through 16. (With minor changes, any six files numbered in a sequence of equal increments could be used with minor code changes in the model.) The files are written to and read from as ASCII files, record length 80 bytes, with file length approximately one record per time step simulated. For a typical run of, say, 200 days at an hourly time step this will mean 6 files each of size 400k. Two additional devices, arbitrarily designated by the user, may be required for rainfall and flow gauge record input. Finally, the standard input and output devices are taken to be 5 and 6 respectively. Figure D.1 depicts the relationship between the model and the various input/output devices.

The number of file operations in the model will



((X)-FORTRAN Device number)

Fig. D-1 Model Data Units

require a varying degree of effort in initially setting up the model, but will not be excessive in most installations since no special file characteristics or file operations are used. Extensive overlays will not be required on most machines since the computer code requirements are modest. In fact, the 130k noted above could be reduced significantly if necessary. The model has been used on mini- and micro- computers, although limitations in 'floppy' disk space in the latter restricted run spans somewhat.

D-4 Data Input Sructure

As described in chapter 4, the model has been written as a series of separate commands which the user can invoke as necessary, in any order, to simulate a particular physical system.

The free format input routines which interpret these commands are the same as those developed by Williams for use in HYMO, and as used in OTTHYMO. These were adopted because they are simple, effective for the model concept used herein, and familiar to many modelers in Canada and the United States.

The input deck for a simulation can be viewed as consisting of two parts. First, there is a series of definition cards which provide the model with information about the commands which will be used. Second, there are the command cards which contain the sequence of instructions coded to simulate the particular basin of interest.

Definition Cards

These cards serve to define the numerical characters recognised by the model, and define the table of commands used by the model. The input format of these cards is fixed, and the content of the definition cards can be regarded as fixed as well. (Note: careful examination of the model code will show that there is some latitude available to the user in this regard, but for simplicity, the cards will normally be the same in every run.) Figure D-2 shows the definition cards which should preface every input deck. It is noted that these cards are analogous to the cards placed in front of a HYMO deck, and that these are the only fixed format cards in an input deck for the proposed model.

Command Cards

All of the model commands are formatted in the following manner:

Figure D-2 Fixed Format Cards Prefacing Every Input Deck

```
--------------------------------------------------
 col.|12345678901234567890123456789012345678901234
     |         1         2         3         4
--------------------------------------------------
 card|         contents
--------------------------------------------------

   1 |1234567890 *./-
   2 |16
   3 |START                 1 25
   4 |STORE                 2  4
   5 |GENERATE              3 25
   6 |PRINT SPAN            4 10
   7 |PLOT SPAN             5 10
   8 |ADD SERIES            6  4
   9 |POND                  7310
  10 |REACH                 8310
  11 |CALIBRATE             9310
  12 |POLLUTANT RATES      10  9
  13 |SPLIT SERIES         11310
  14 |DUMP PRINT           12  1
  15 |EXCEEDANCE CURVES    13310
  16 |DUMP PLOT            14  9
  17 |FINISH               15  0
  18 |CONVOLUTE            16310
--------------------------------------------------
```

COMMAND NAME ((text)(parameter))((text)(parameter))...

where COMMAND NAME is the exact string of characters representing the command to be invoked, beginning in column 1 of the record.

(text) is arbitrary and may be omitted, and must not contain any numerical characters, dashes, or decimal points.

(parameter) is provided according to the requirements of the command. The parameter list must be provided in the correct order but parameters are otherwise in free format. Extra trailing parameters will be ignored.

and the ((text)(parameter)) list must occupy only columns 21 to 79 of any record, but may occur on as many records following the COMMAND NAME as necessary.

The COMMAND cards follow directly after the definition cards, in whatever sequence is selected by the user. The first command must always be a START. The last command, and hence the last card in an input deck, should be a FINISH card. Chapter 3 provides details on the various commands, and on the way in which the commands are assembled to represent a basin.

D-5 Model Application

This section provides basic information on how the model is used to represent a physical system.

Coding A Basin

As already indicated, the basic principle in providing input data for the model is to assemble a series of commands or instructions from the library which the model can interpret.

The model will read and act on each command in the order given, so the sequence of the commands is important. The basin must be represented in a 'top down' manner, with information needed in any command provided or generated before the command is entered.

It is noted that it is up to the user to ensure that the model commands are appropriate for the physical situation they simulate. The model will provide some sort of result for most valid combinations of input parameters, but this does not mean that the result is always physically reasonable.

An Example of Basin Coding

The use of the model in simulating a basin will be presented in this section by a simplified example. To facilitate this, most of the parameters used with individual commands will not be presented in this section. Instead, the choice of what commands to use to represent the whole system will be illustrated. Detailed information on how to code each command is left to later sections of this appendix; detailed examples of model application are presented in the main text.

The sample system is shown in figure D-3. As indicated, there are four sub-basins joined by a forked stream in this system.

(1.) Conceptual Representation of the System

The first step in the simulation, once the sub-basins have been defined, is to represent the basin as a simple conceptual model. This determines which physical components will be included in the model, how they will be represented, and how they are related to one another. Figure D-4 shows one way that the basin in Figure D-3 could be represented. Note that in this case, two river reaches are deemed necessary for routing and that it is chosen to input sub-basins 302 and 303 at the lower end of the reaches to which they are

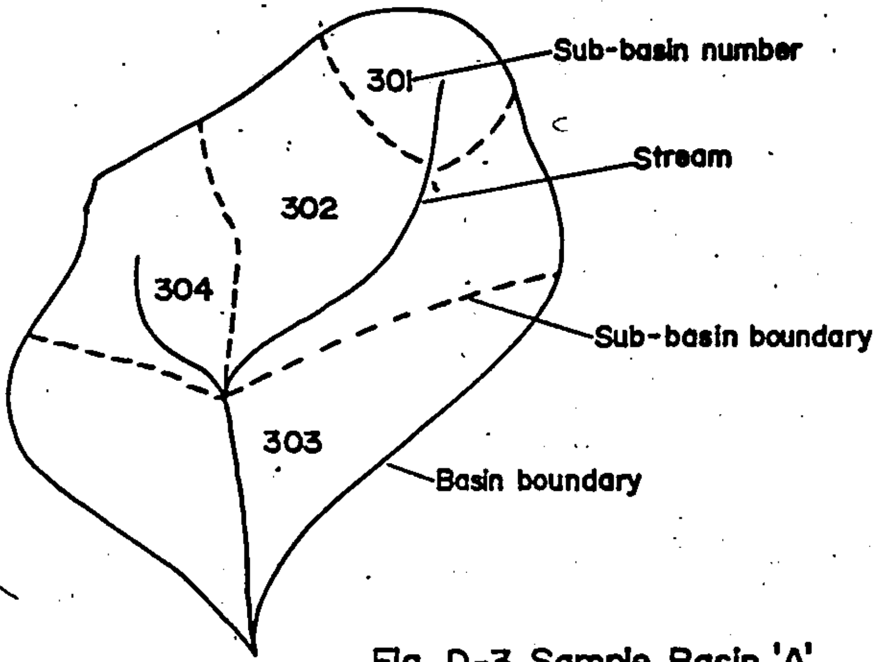


Fig. D-3 Sample Basin 'A'

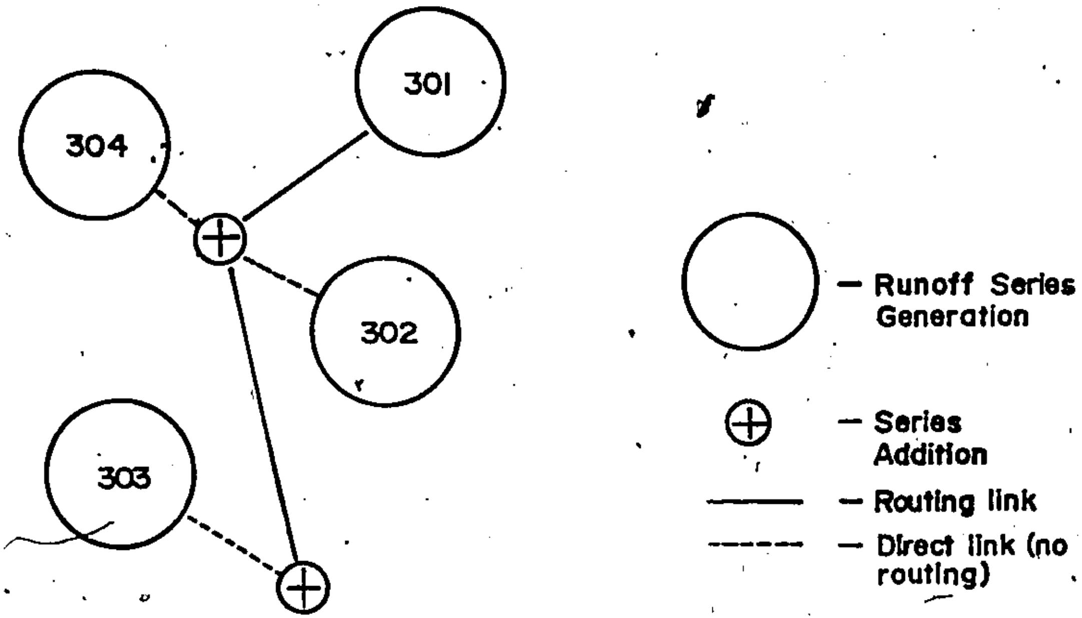


Fig. D-4 One Conceptual Representation of Basin 'A' tributary. The following points summarise the chosen conceptual scheme:

- o flows from area 301 are generated and routed
- o flows from areas 302 and 304 are generated, and the result added to the routed flows from area 301
- o the above totaled flows are routed, and added to those generated by area 303

The simulation will therefore consist of four generations, two routings, and several additions linked in a particular order. Other schemes are possible, and could increase or decrease the number or sequence of any of these operations.

(2.) Specify Connectivity

To follow the steps outlined above, the model must be able to identify each component of the system so that the intended time series are correctly identified at any time. This is done by numbering each input and output series generated or routed by the model. There are two sets of identification numbers available for this purpose in the model. One set, the Series numbers, serves to provide the user with an additional identifier for each time series, and might be regarded as titles which are attached to a series after each operation. The second set of numbers, the ID numbers, are critical in representing the basin connectivity. Each output series must have a unique ID number; it is this number which is used by the model as a reference to any input or output time series.

The Series number can be any integer number, but should be systematic to avoid confusion. Duplicate Series numbers, for example, are not useful. In contrast, the ID numbers must conform to the following rules:

- o They must be integer values between 1 and 6 inclusive.
- o They may be re-used any number of times in a simulation, but must be unique at any one time.

Since each time series must have an ID number, the consequence of these rules is that only six time series may exist at one time in the memory of the model. Once all six ID numbers have been allocated, a new series can only be generated by occupying the space previously held by an old series. Printing or plotting a series prior to reusing a number, or adding series at a junction, are two ways of providing more space without losing information. In summary, the Series number is for the convenience of the user while the ID number controls the simulation.

One appropriate numbering scheme for the example case is shown in figure D-5. The generated flows are all given series numbers which correspond to the original arbitrary sub-basin numbering, while the routed and added flows are given numbers in sequence starting at 400. As mentioned, these numbers are entirely a matter of convenience.

The ID numbers, however, are more carefully chosen. In this case, as few numbers were used as possible, as follows:

- o the outflow from area 301 was given ID number 1
- o when routed the outflow was given ID number 2
- o flows generated from area 304 were assigned ID number 1. This is possible since the flows originally held in this number (from area 301) will have been routed before flows from area 304 are generated, leaving ID 1 available for other purposes. If it was decided to generate flows from area 304 before routing flows from area 301, this would not be possible.
- o flows with ID numbers 1 and 2 are added, and the result is given ID number 3
- o flows from area 302 are generated, and given the

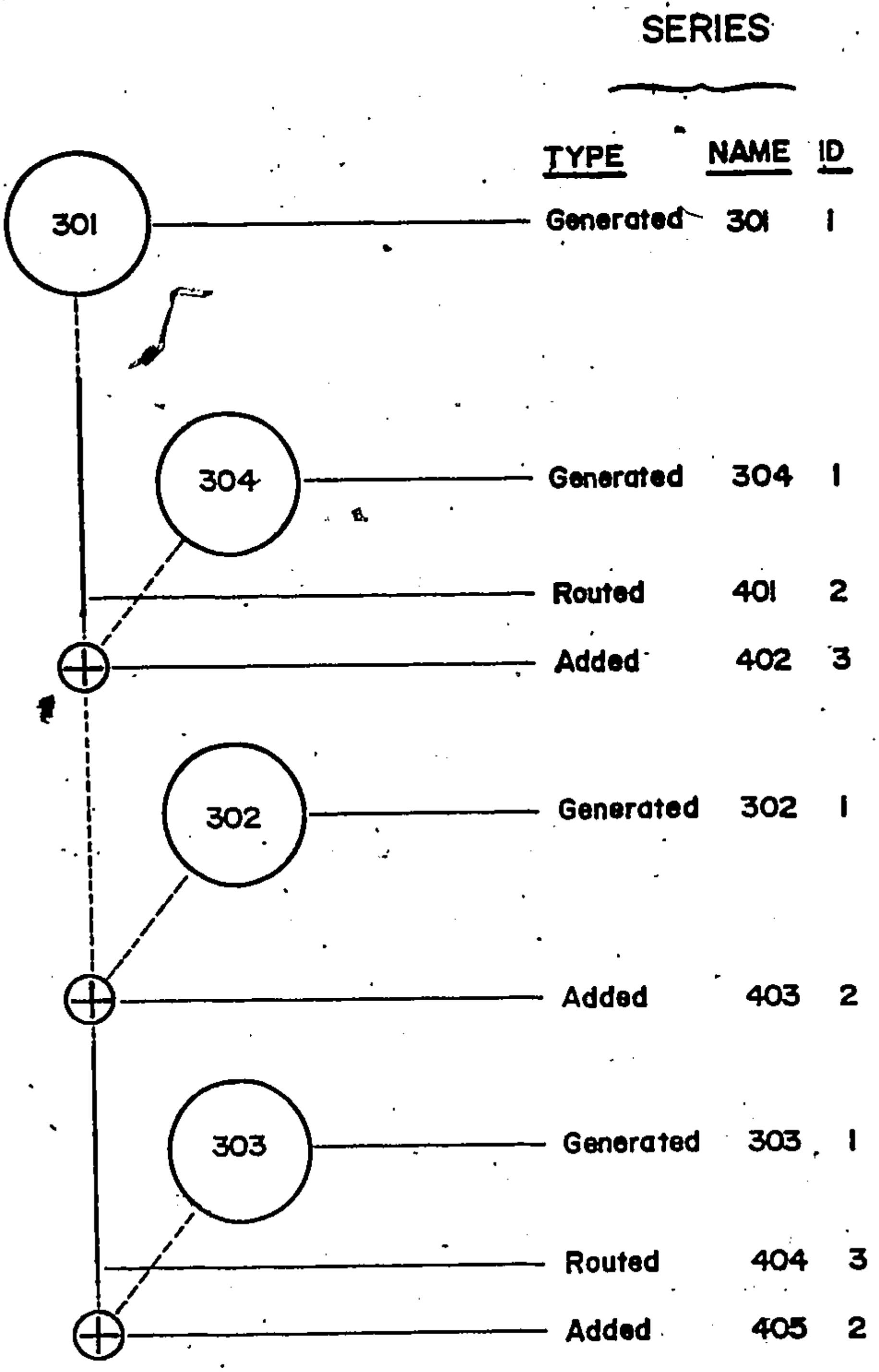


Fig. D-5 One Numbering Scheme for Basin 'A' number 1 (number 1 and 2 are both available since the above addition has been performed)

o flows from ID number 3 and number 1 are added, and given ID number 2 o flows with ID number 2 are routed and given ID number 3 (which again leaves numbers 1 and 2 available)

o flows from area 303 are generated with the number 1, and added to ID number 3 which results in a final series with ID number 2

Again, there are a number of ways to achieve the proper connectivity with the six available ID numbers. The choice of the scheme used depends on the disk space available to the user, on the points where output information is required, and on user preference. As a further example, note that it is possible in this case to generate all of the flow series from areas 301 through 304 at the beginning of the run using ID numbers 1 through 4. These series could then be routed and added as required using the remaining available numbers 5 and 6.

(3.) Code the Numbered Conceptual Scheme

Once the basic components and connectivity of the system to be simulated have been defined, the commands representative of each operation must be assembled. Taken together the commands are essentially a word statement of the various components identified in the conceptual model, presented in an order consistent with the chosen numbering scheme.

The commands required in this example are:

GENERATE	- to create the flow series from each basin
ADD	- to combine flow series
REACH	- to route flows through a river reach

The first parameters of each command are the Series and ID numbers. The details of these or the other parameters will not be accounted here. Figure D-6, however, shows how each command would be coded to conform with the above numbering scheme in the absence of other parameters.

As indicated above, the first cards in the deck are the Definition cards. The first COMMAND card is the START card which has no ID or series numbers. There are then the nine functional commands which create and route the flow series through the system. Finally, there is a PRINT and PLOT command

FIGURE D-6 CODE SEQUENCE CORRESPONDING TO FIGURE D-5

```
CARD| CONTENTS
--------------------------------------------------------
 1  |1234567890 *./-
 2  |16
 3  |START                  1 25
 4  |STORE                  2  4
 5  |GENERATE               3 25
 6  |PRINT SPAN             4 10
 7  |PLOT SPAN              5 10
 8  |ADD SERIES             6  4
 9  |POND                   7310
10  |REACH                  8310
11  |CALIBRATE              9310
12  |POLLUTANT RATES       10  9
13  |SPLIT SERIES          11310
14  |DUMP PRINT            12  1
15  |EXCEEDANCE CURVES     13310
16  |DUMP PLOT             14  9
17  |FINISH                15  0
18  |CONVOLUTE             16310
19  |START
20  |GENERATE              ID=1, SERIES=301
21  |REACH                 ID=1, TO BECOME ID=2,
22  |                      SERIES=401
23  |GENERATE              ID=1, SERIES=304
24  |ADD                   ID=1+2 TO BECOME ID=3,
25  |                      SERIES=402
26  |GENERATE              ID=1, SERIES=302
27  |ADD                   ID=1+3 TO BECOME ID=2,
28  |                      SERIES=403
29  |GENERATE              ID=1  SERIES=303
30  |REACH                 ID=2 TO BECOME ID=3,
31  |                      SERIES=404
32  |ADD                   ID=1+3 TO BECOME ID=2,
33  |                      SERIES=405
34  |DUMP PRINT            ID=2
35  |DUMP PLOT             ID=2
36  |FINISH
```

which provide the user with information about the calculated flows, and a FINISH command which terminates the run.

There is less latitude at this stage than in the previous steps, primarily because most of the choices to be made by the modeler will have been made by the time coding begins. However, there is still often some variation in code possible even with a specific conceptual model and numbering scheme. For example, if several GENERATE commands are issued one after the other (ie no routings or additions in between) the order in which this is done does not matter. Similarly, PRINTing or PLOTing can be done at any time provided that the series to be output still exists.

D-6 Detailed Description of Commands

All commands have the same format, as described above. The command name starts in column 1, and parameters interspersed with optional descriptive text appear in columns 21 to 79 of the command card on as many subsequent cards as necessary. The following sections list the commands available in this version of the model, and describe the parameters and use of each command. Parameters are listed in the order in which they should appear following the command name.

D-6-1 START

Purpose

The START command is used to initialise major controlling variables for the simulation.

Parameters

Name	Description
GY1	Start year of simulation. If <= 0, whole period of record is simulated.
GM1	Start month of simulation - omit if GY1 <=0
GD1	Start day of simulation ''
GY2	End year of simulation ''
GM2	End month of simulation ''
GD2	End day of simulation ''
IRAIN	FORTRAN file number of input rainfall data. Use dummy value if rain not required.
IFLOW	FORTRAN file number of input flow records. Use dummy value if flows are not input.
IFDECA	Flag. If non-zero, a first order decay substance will be simulated; if zero, a first order decay substance will not be simulated.
DECAYK	Decay rate (per day) for use in simulation first order decay substance. Omit if IFDECA=0.
IFSEDT	Flag. If non-zero, sediment will be simulated. If zero, sediment will not be simulated.
SEDSET(J) J=1 to 5	Effective settling velocities (m/s) size fractions - omit if IFSEDT=0
SEDDIS(J) J=1 to 5	Sediment size fraction (Mass fraction of total sediment.) corresponding to each SEDSET(J). - omit if IFSEDT=0

Notes

The START command must appear before all of the other commands, but may reappear as many times as necessary within the run to modify parameters.

All 5 fractions and settling velocities must be input if IFSEDT is non-zero, but some of the fractions can be zero if fewer than five fractions are desired. The fractions can be input in any order, provided that the information in SEDDI6 and SEDSET are in the same order. Note that fractions are mass fractions, not cumulative or percent less than totals. Allowable values of IRAIN and IFLOW may depend on the computer installation.

Sample START Command:

```
----------------------------------------------------------------
START               SIMULATE FROM 81 4 1 TO 81 9 30
                    RAIN RECORDS ON FILE 9
                    WATER SURVEY FLOW RECORDS ON FILE 10
                    SIMULATE F.C. BACTERIA (IFDECAY=1)
                     WITH DECAY RATE 3.6 PER DAY
                    SIMULATE SEDIMENT (IFSEDT=1)
                     SETTLING VELOCITIES ARE
                     .05 .10 .20 .25 AND .3 M PER SEC
                     FOR SIZE FRACTIONS
                     .10 .15 .46 .20 AND .09
----------------------------------------------------------------
```

The above cards specify a run to span the period April 1, 1981 to September 30, 1981. Rainfall records are on FORTRAN device 9, and flows are on FORTRAN device 10. The parameters IFDECA and IFSEDT flag that both a first order pollutant and sediment will be simulated. The first order decay rate is given as 3.6 per day, and five size fractions are specified according to settling velocity and fractional composition.

D-6-2 GENERATE

Purpose

Calculates a rainfall and pollutant series of indefinite length.

Parameters

Name	Description
IDOUT	ID number of generated series.
ISER	Series number of generated series.
DT(ID)	Output time step of series.
DA(ID)	Basin area (hectares).
AB	Flag. If =1, unit hydrographs are printed. If =0, unit hydrographs are not printed.
FRIMP	Fraction of impervious land.
	IMPERVIOUS AREA DATA NEXT 5 PARAMETERS ++++ ++++++ OMIT IF FRIMP=0.0 ++++++++++++++++++
AA	Flag. If =1, Nash unit hydrograph used on paved area. If = 2, Williams '' '' '' '' ''
KORH	Hydrograph shape parameter If >0, taken as Nash 'n' or Williams 'k' depending on AA. (no units) If <0, taken as height for use in Williams hydrograph parameter equation. (height in meters)
TPORL	Hydrograph shape parameter If >0, taken as Nash or Williams 'Tp' depending on AA. (no units) If <0, taken as length for use in Williams hydrograph parameter equation. (length in kilometers)
ABSIMP	Impervious area initial abstraction (mm)
RIMP	Impervious area volumetric runoff coefficient.

	PERVIOUS AREA DATA NEXT 8 PARAMETERS ++++++ ++++++ OMIT IF FRIMP=1.0 +++++++++++++++++
AA	Flag. If =1, Nash unit hydrograph used on pervious area. If =2, Williams " " " " "
KORH	Hydrograph shape parameter If >0, taken as Nash 'n' or Williams 'k' depending on AA. (no units) If <0, taken as height for use in Williams hydrograph parameter equation. (height in meters)
TPORL	Hydrograph shape parameter If >0, taken as Nash or Williams 'Tp' (time in hours) If <0, taken as length for use in Williams hydrograph parameter equation. (length in kilometers)
SMIN	Minimum value for S (mm).
SMAX	Maximum value for S (mm).
SK	Variable relating S to API, SMIN and SMAX. (units per mm).
APIK	API calculation parameter, usually= 0.9 per day.
API1	Starting API. (mm).
ABSPER	Pervious area initial abstraction.
	BASEFLOW DATA NEXT 4 PARAMETERS ++++++++++++++++ ++++++ ALWAYS REQUIRED +++++++++++++++++++++++++
SVOL	Starting soil moisture reservoir content (mm).
SLOSK1	Base flow recession constant. (mm/sec/mm)
SLOSK2	Base flow reduction factor. (no units)
BASMIN	Minimum base flow rate. (cu. m. per sec.)

Notes

The start and finish dates of the run are governed by the preceding START command.

Pollutant generation is undertaken by this command if the immediately preceding START command has flagged this function 'on'. If so, the rate parameters for pollutant generation must have been set in preceding

POLLUTANT RATES commands.

Soil moisture accounting parameters are discussed in Appendix A.

Sample GENERATE Command:

```
GENERATE                IDOUT=5, ISER=301, DT=1 HOUR, AREA=2504 HA
                        AB=1 (DO NOT PRINT UNIT HYDS)
                        IMPERVIOUS DATA *****
                         FRACTION IMPERVIOUS = 0.12
                         AA=1 (NASH HYDROGRAPH IS TO BE USED)
                         2 SLRS WITH TP=0.5 HOURS
                         INITIAL ABSTRACTION= 1 MM
                         RUNOFF COEFF= 0.95
                        PERVIOUS DATA *****
                         AA=2 (WILLIAMS HYDROGRAPH IS TO BE USED)
                         K=3 AND TP=1.2 HOURS
                         SMIN=25 MM, SMAX=400 MM AND SK=2.2 PER MM
                         KAPI IS 0.9 PER DAY
                         INITIAL API IS 45 MM
                         INITIAL ABSTRACTION IS 3 MM
                        BASE FLOW DATA *****
                         INITIAL SOIL VOLUME IS 25 MM
                         BASE RECESSION CONST IS 0.00134 PER SEC
                         BASE FLOW REDUCTION FACTOR IS 0.6
                         MINIMUM BASE FLOW RATE IS 0.0 CMS
```

Most of the parameters input in this example are self evident. Essentially, the simulation will occur over the period defined in the preceding START command, using the rainfall defined there, and the resulting flow/pollutant series will have ID number 5 and serial number 301. The total basin area is 2504 hectares, and 12 percent of this is considered to be impervious. Runoff parameters are given for both the impervious areas and pervious areas. Note that different unit hydrograph shapes are chosen for the two areas. The base flow parameters as specified show that no minimum base flow amount is specified, and that only 60 percent of the calculated base flow will be added to the basin out flows (this is indicated by the base flow reduction factor SLOSK2 of 0.6); the remaining 40% represents a loss from the system.

D-6-3 POLLUTANT RATES

Purpose

The POLLUTANT rates cards is necessary to simulate pollutants, but does not perform any simulation operations when invoked. It actually sets parameters and pollution generation methods which will be used in the GENERATE command. Unless the GENERATE command is subsequently used, this command has no function.

Parameters

Name	Description
ICASE	Identifies the part of the system which the POLLUTANT RATES command applies to, as follows:

ICASE value	pollutant defined	applicable source
1	first order	impervious area
2	''	pervious area
3	sediment	impervious area
4	''	pervious area
5	first order	base flow
6	sediment	base flow

Name	Description
IWM	chosen washoff method. if =1, a rating curve is used. if =2, a buildup/washoff method is used. - omit if ICASE=5 or ICASE=6
PCO	washoff coefficient (units mass per cu. m.)
PEX	washoff exponent (no units)
IBM	chosen build-up method - omit this and all subsequent parameters if IWM=1 or if ICASE=5 or if ICASE=6 if IBM=1, power linear build-up is used if IBM=2, exponential build-up is used
DAYSIN	days accumulation of pollutant at start of first event in series.
PMAX	limiting amount of build-up on surface. (units mass per sq. m.)
PK1	build-up rate parameter no. 1. (units mass per sq. m. per day)
PK2	build-up rate parameter no. 2.-omit if

IBM=2 (no units)

Notes

POLLUTANT RATES commands need only be issued for the pollutant being simulated. For example if no sediments are simulated (IFSEDT=0 on the START command), three POLLUTANT RATES cards (ICASE 3,4, and 6) can be omitted; similarly, if no arbitrary first order pollutant is simulated (IFDECA=0 on the START command), POLLUTANT RATES cards for ICASE 1,2 and 5 can be omitted. Note that if ICASE is 5 or 6 (base flows) then the user is restricted to use of a rating curve approach; IWM is omitted.

The meaning and function of the above rate parameters are discussed in Appendix A. (NOTE: Some of the above units are nominal only. See Appendix A. for details.)

Sample POLLUTANT RATES Command:

```
-------------------------------------------------------------------
POLLUTANT RATES     ICASE=3 (SEDIMENT FROM IMPERVIOUS AREA)
                    WASHOFF METHOD IS 2 (BUILDUP WASHOFF)
                    WASHOFF RELATION IS
                     WRATE= BUILD*AREA*.045(PER CU M)*Q**.85
                    BUILDUP METHOD IS 1 (POWER LINEAR)
                    INITIAL ACCUMULATION=30 DAYS
                    MAXIMUM ACCUMULATION IS 500 KG PER HECTARE
                    BUILDUP RELATION IS
                     BUILD= 10 (KG PER HA PER DAY)*T(DAYS)**1
-------------------------------------------------------------------
```

This POLLUTANT RATES card is specifying data for an impervious area, for sediment. A build-up washoff method is used, and the user has elected to input the parameters in the form of a function. This has only a mnemonic value, since it is the values which are input which are important, rather than the format which is used. However, this type of presentation may serve to facilitate subsequent interpretation of the model outputs and may therefore be useful.

D-6-4 POND

Purpose

This command routes a flow series through a storage detention pond. If pollutant simulation has been selected on the START command, associated pollutants will also be routed through the pond.

Parameters

Name	Description
IDOUT	ID number of outflow series from pond.
ISER	Series number.
IDH	ID number of inflow series to pond.
TDET	Detention time in a batch pond. (Hours) Zero if a continuous pond is simulated.
NELS	Number of CSTRs used in routing pollutants. (range = 1 through 99 but use dummy value if no pollutants are to be routed)
RTINC	Time increment for use in flow (but not pollutant) routing. (Hours).
QBAS	Base flow value which will be allowed through the pond unhindered in batch operation. Meaningless in continuous operation of pond.(cu. m. per sec.)
	APPROACH FLOW CURVE DATA +++++++++++++++++
NPTQQ	Number of points in approach flow curve. (range = 2 through 25 but 0 is allowed) (omit QAPP(i),QIN(i) pairs ir NPTQQ=0)
QAPP(1)	First approach flow point.
QIN(1)	First inlet flow point.
...	...
QAPP(NPTQQ)	Last approach flow point.
QIN(NPTQQ)	Last inlet flow point.
	CONTINUOUS OUTFLOW CURVE DATA +++++++++++++
NPTSQ1	Number of points on lower stage outflow curve. (range = 2 through 25 but 0 is allowed) (omit CONTINUOUS outflow curve STAGE(i),FLOW(i) pairs if NPTSQ1=0)
STAGE(1)	First stage on continuous outflow curve.

FLOW(1)	First flow on continuous outflow curve.
...	...
STAGE(NPTSQ1)	Last stage on continuous outflow curve.
FLOW(NPTSQ1)	Last flow on continuous outflow curve.

OPERATED OUTFLOW CURVE DATA +++++++++++++++

NPTSQ2	Number of points on upper stage outflow curve. (range = 2 through 25) (this curve must be input)
STAGE(1)	First stage on operated outflow curve.
FLOW(1)	First flow on operated outflow curve.
...	...
STAGE(NPTSQ2)	Last stage on operated outflow curve.
FLOW(NPTSQ2)	Last flow on operated outflow curve.

OVERFLOW CURVE DATA +++++++++++++++++++++++

NPTSQV	Number of points on overflow curve. (range = 2 through 25 but 0 is allowed) (omit OVERFLOW curve STAGE(i), FLOW(i) pairs if NPTSQV=0)
STAGE(1)	First stage on overflow curve
FLOW(1)	First flow on overflow curve
...	...
STAGE(NPTSQV)	Last stage on overflow curve.
FLOW(NPTSQV)	Last flow on overflow curve.

POND VOLUME DATA +++++++++++++++++++++++++

NPTSV	Number of points on stage volume curve. (range = 2 through 25 but 0 is allowed provided that NPTSA is not zero) (omit STAGE(i),VOLUME(i) pairs if NPTSV=0)
STAGE(1)	First stage on volume curve.
VOLUME(1)	First volume on volume curve.
...	...
STAGE(NPTSV)	Last stage on volume curve.
VOLUME(NPTSV)	Last volume on volume curve.

POND HORIZONTAL AREA DATA +++++++++++++++

NPTSA	Number of points on stage area curve. (range = 2 through 25 but 0 is allowed provided that NPTSV is not zero) (omit STAGE(i),AREA(i) pairs if NPTSA=0)
STAGE(1)	First stage on area curve.
AREA(1)	First area on area curve.
...	...
STAGE(NPTSA)	Last stage on area curve.

AREA(NPTSA)	Last area on area curve.
SBEGIN	Starting elevation in pond.

Notes

Appendix B describes the interrelationships between the various curves described above, and also documents the hydraulic and pollutant routing schemes used in the model. Note that all flows are in cubic meters per second, areas in square meters, and stages in meters.

The minimum area, volume, and flow in each of the above curve should be zero. However, the stages used can be taken from any arbitrary datum, provided that they are mutually consistent.

In the event that the Stage/Area curve is omitted, it is estimated from the Stage/Volume curve. Conversely, the Stage/Area curve is used to estimate the Stage/Volume curve if that has not been supplied.

Note that the starting concentration of pollutants in the pond is always zero. The output series time step is the same as the input series time step regardless of RTINC.

Sample POND Command:

```
---------------------------------------------------------------
POND                IDOUT=4, ISER=332, AND INFLOW ID IS 5
                    DETENTION TIME IS 0 (BATCH POND)
                    1 CSTR IS USED IN POLLUTANT SIMULATION
                    FLOW ROUTING TIME STEP IS 0.5 HOURS
                    DUMMY BASE FLOW VALUE IS 0
                    ZERO (0) POINTS ON APPROACH CURVE
                    ZERO (0) POINTS ON LOWER OUTFLOW CURVE
                    5 POINTS ON UPPER OUTFLOW CURVE ARE
                         STAGE(M)   RATE(CU M PER SEC)
                           0               0
                           1               .5
                           3               1.2
                           4               3.4
                           7               6.2
                    ZERO (0) POINTS GIVEN FOR OVERFLOW CURVE
                    3 POINTS ON STAGE VOLUME CURVE ARE
                         STAGE(M)    VOLUME(CU M)
                           0                0
                          1.5             11000
                          8.0             45000
                    ZERO (0) POINTS ON POND AREA CURVE
                    STARTING ELEVATION = 0 M
---------------------------------------------------------------
```

This example is one of a very simple pond configuration. Since no inflow curve is specified, all flows will enter the pond. Only one outflow curve is specified, and no overflow curve is given. Flows in excess of the maximum value on the outflow curve will be routed by extrapolation of that curve. One reactor is specified for routing of chemical constituents, and a computation time step for flows of a half hour is requested.

D-6-5 REACH

Purpose

This command routes a flow series through a river reach. If pollutant simulation has been selected on the START command, associated pollutants will also be routed through the pond.

Parameters

Name	Description
IDOUT	ID number of outflow series from reach
ISER	Series number
NIDH	Number of sources tributary to reach at headwater. (range = 1 through 6 but can be zero is NIDL is not zero) (if NIDH=0 omit all IDH(i) values)
IDH(1)	ID number of first source tributary to reach headwater.
...	...
IDH(NIDH)	ID number of last source tributary to reach headwater.
NIDL	Number of sources tributary to reach distributed along sides. (range = 1 through 6 but can be zero if NIDH is not zero) (if NIDL=0 omit all IDL(i) values)
IDL(1)	ID number of first series tributary to reach. distributed along sides.
...	...
IDL(NIDL)	ID number of last series tributary to reach, distributed along sides. Note: if NIDL is zero, omit all IDL.
IFAORM	Flag. If =1, arbitrary channel input is expected. If =2, trapezoidal channel expected.
NELS	Number of CSTRs used in routing pollutants. (range = 1 through 99 but use dummy value if no pollutants are routed.
SMAX	Maximum expected depth of flow in channel (units meters)
XLEN	Reach length. (meters)
RTINC	Time increment for use in flow (but not

	pollutant) routing. (hours)
	ARBITRARY CHANNEL INPUT DATA ++++++++++++ ++++++ OMIT IF IFAROM =2 +++++++++++++++
COEF	Coefficient in flow/stage curve. (units cu. m. per sec. per m.)
EXPON	Exponent in flow/stage curve. (no units)
	REACH VOLUME DATA +++++++++++++++++++++++
NPTSV	Number of points on stage volume curve. (range = 2 through 25 but can be zero provided NPTSA is non-zero) (omit STAGE(i), VOLUME(i) pairs if NPTSV = 0)
STAGE(1)	First stage on volume curve.
VOLUME(1)	First volume on volume curve.
...	...
STAGE(NPTSV)	Last stage on volume curve.
VOLUME(NPTSV)	Last volume on volume curve.
	REACH HORIZONTAL AREA DATA ++++++++++++++
NPTSA	Number of points on stage area curve. (range = 2 through 25 but can be zero provided NPTSV is non-zero) (omit STAGE(i), AREA(i) pairs if NPTSA = 0)
STAGE(1)	First stage on area curve.
AREA(1)	First area on area curve.
...	...
STAGE(NPTSA)	Last stage on area curve.
AREA(NPTSA)	Last area on area curve.
	TRAPEZOIDAL CHANNEL INPUT DATA ++++++++++ ++++++ OMIT IF IFAORM = 1 +++++++++++++++
RN	Channel Manning 'n'.
SF	Channel friction slope. (m/m)
SS	Channel side slopes. (m/m)
B	Channel bottom width. (units meters)

Notes

Appendix C documents the algorithms used in the REACH command, and provides details on the various parameters described above. Note that all volumes are cubic meters, flows are cubic meters per second, areas are square meters, and lengths are meters.

When the non-trapezoidal channel is input, the minimum area and volume in the above curves should be zero. The Stages, however, can start at any arbitrary datum. Note that whatever elevation datum is used, it should be consistent with that which applies to the flow/ stage curve.

In general, the estimated maximum stage in the channel should be as low as possible provided that the flows do not exceed this limit. This improves the outflow estimation. Note that even if the stated maximum stage is exceeded, the model will usually provide a solution by extrapolation. This occurs with no warning, and the user should examine the peak flows calculated to determine whether an extrapolation occurred.

In the event that the Stage/Area curve is omitted, it is estimated from the Stage/Volume Curve. Conversely, the Stage /Area curve is used to estimate the Stage/Volume curve if that has not been supplied.

All of the input series (IDHs and IDLs) must have the same time step; the output series has the same time step as the input series regardless of the value of RTINC.

Sample REACH Command:

```
REACH                   IDOUT IS 6, SERIES NAME IS 452
                        1 INPUT HEADWATER SERIES, ID=4
                        AND 2 LATERAL SERIES, ID 5 AND 1
                        25 ELEMENTS IN QUALITY ROUTING
                        IFAORM=2 (TRAPEZOIDAL CHANNEL)
                        MAX EXPECTED DEPTH OF FLOW IS 6 M
                        LENGTH OF CHANNEL IS 10000M
                        FLOW CALCULATION TIME STEP IS 1.0 HOURS
                        CHANNEL ROUGHNESS IS 0.035
                                BOTTOM SLOPE IS .0001 M PER M
                                SIDE SLOPE IS 0.35 M PER M
                            AND BOTTOM WIDTH IS 10 M
```

In this example, a reach is defined which has one tributary headwater (ID4) and two distributed lateral sources (ID 5 and 1). Other parameters self explanatory. Note that the requested flow calculation time step of one hour will be used in routing flows through the reach provided that it is shorter than the time steps on the input time series (IDs 4, 5 and 1). However, the time step at which quality routing will occur, and the time step for storing data under this reach ID (7) will be the same as for the input IDs. Note also that the input IDs must all have the same time step for a routing to be possible.

D-6-6 ADD SERIES

Purpose

The ADD command combines two flow series. If pollutant data is present and if the START command has called for pollutant simulation, these will be combined as well.

Parameters

Name	Description
IDOUT	ID number of combined flow/pollutant series.
ISER	Series number.
ID1	ID number of one series to be added.
ID2	ID number of the other series to be added.

Notes

ID1, ID2, and IDOUT should all have different values.

If the two input series are at different time steps, the shorter time step is used for the output series. Values from the longer time step series are linearly interpolated to a shorter time step.

Sample ADD SERIES Command:

```
------------------------------------------------------------
ADD SERIES          IDOUT is 3, SERIES NAME IS 233
                    INPUT IDS ARE 4 AND 2
------------------------------------------------------------
```

D-6-7 SPLIT SERIES

Purpose

This command allows the user to split a series into two separate series in a manner dependant on flow.

Parameters

Name	Description
IDIN	ID number of the series to be split.
IDOUT1	ID number of one of the split series.
ISER1	Series number of IDOUT1.
IDOUT2	ID number of the other of the split series.
ISER2	Series number of IDOUT2.
NPTQQ	Number of points in the flow split curve. (range = 2 through 25)
QAPP(1)	First approach flow point.
QTO1(1)	First diverted flow point.
...	...
QAPP(NPTQQ)	Last approach flow point.
QTO1(NPTQQ)	Last diverted flow point.

Notes

The approach flow/ diverted flow curve splits flow according to the approach flow, by interpolating values from the curve. The amount QTO1 which corresponds to the approach flow is diverted to series IDOUT1. The remainder (approach flow less flow diverted to IDOUT1) is sent to the series IDOUT2. All flow units are cubic meters per second.

Sample SPLIT SERIES Command:

```
-------------------------------------------------------------
SPLIT SERIES        INPUT ID IS 2
                    IDOUT A IS 4, NAMED 752
                    IDOUT B IS 5, NAMED 753
                    2 POINTS ON FLOW SPLIT CURVE,
                     APPROACH FLOW   FLOW TO SERIES A
                     CU M PER SEC      CU M PER SEC
                        0                 0
                        100               50
-------------------------------------------------------------
```

This example splits series 751 into two other series, each of which has half of the original flow and pollutant load.

D-6-8 FINISH

Purpose

This command terminates a run.

Parameters

None

Notes

The FINISH card should be the last card in every deck. If it is not included, the run will continue normally until the last command is reached but results at that point are unpredictable. In some cases, the last command will terminate properly, but in other cases it will not; either way, an error message will be generated because an 'end of file' will be detected by the system.

D-6-9 PLOT SPAN

Purpose

The PLOT command is used to provide a plot of part of one or two series on a single one-page graph.

Parameters

Name	Description
IWHICH	Number of parameter to be plotted.
ID1	ID of first series to be plotted.
IFY	Beginning year of time span to be plotted.
IFM	Beginning month '' '' '' '' '' ''
IFD	Beginning day '' '' '' '' '' ''
ITY	Ending year '' '' '' '' '' ''
ITM	Ending month '' '' '' '' '' ''
ITD	Ending day '' '' '' '' '' ''
DT2	Time step of points to be plotted.
ID2	ID of second series to be plotted. May be omitted or set to zero.

Notes

The value of IWHICH depends on the parameter to be plotted. The following convention applies:

IWHICH	parameter
1	flow
2	first order pollutant
3	fraction 1 of sediment
4	'' 2 ''
5	'' 3 ''
6	'' 4 ''
7	'' 5 ''

The PLOT command can be issued to plot time spans in any order. However, if the time spans are coded in chronological sequence, and if they do not overlap, a large amount of computer time can be saved by making the value of IWHICH negative for all but the first PLOT.

No more than 300 points can be PLOTed. The scale of the plot will be cubic meters per second for flow, and mass per second for pollutants; both are plotted against hours.

Sample PLOT SPAN Command:

```
----------------------------------------------------------------
PLOT SPAN            FLOW (IWHICH=1) FROM SERIES 5
                     FROM 81 6 5 TO 81 6 6 IN 1 HOUR STEPS
----------------------------------------------------------------
```

D-6-10 PRINT SPAN

Purpose

This command is used to provide a one-page printout of part of one series.

Parameters

Name	Description
IWHICH	Number of parameter to be printed.
ID1	ID number of series to be printed.
DT2	Time step of points to be printed.
IFY	Beginning year of time span to be printed.
IFM	Beginning month '' '' '' '' '' ''
IFD	Beginning day '' '' '' '' '' ''
ITY	Ending year '' '' '' '' '' ''
ITM	Ending month '' '' '' '' '' ''
ITD	Ending day '' '' '' '' '' ''
NPK	Flag. If non-zero, only the peak value of the parameter is printed. If zero, all values in the span are printed. May be omitted if zero.

Notes

For valid values of IWHICH, see section 3.2.9 PLOT.

The PRINT command can be issued to plot time spans in any order. However, if the time spans are coded in chronological sequence, and if they do not overlap, a large amount of computer time can be saved by making the value of IWHICH negative for all but the first PRINT.

No more than 300 points can be included in the time span PRINTed. Units are cubic meters per second for flow, and mass per second for pollutants.

Sample PRINT SPAN Command:

```
---------------------------------------------------------------
PRINT SPAN          FIRST ORDER POLLUTANT (IWHICH=2)
                    FROM SERIES ID 6
                    SPAN IS 83 7 15 TO 83 7 21
---------------------------------------------------------------
```

D-6-11 CALIBRATE

Purpose

This command provides some basic statistics comparing two series, and is intended to be used in model calibration.

Parameters

Name	Description
NUMINT	Number of time spans over which to compare the two series. (range = 1 through 25)
ID1	ID number of one series to compare.
ID2	ID number of other series to compare.
IFY(1)	Beginning year, first span.
IFM(1)	'' month, '' ''
IFD(1)	'' day, '' ''
ITY(1)	Ending year, '' ''
ITM(1)	'' month, '' ''
ITD(1)	'' day, '' ''
...	...
IFY(NUMINT)	Beginning year, last span.
IFM(NUMINT)	'' month, '' ''
IFD(NUMINT)	'' day, '' ''
ITY(NUMINT)	Ending year, '' ''
ITM(NUMINT)	'' month '' ''
ITD(NUMINT)	'' day '' ''

Notes

This command will compare flows in the two series, and if START has initiated pollutant simulation, will compare pollutant loads as well.

The CALIBRATE will calculate the total mass for both series for each span, and will provide the RMS difference between the masses in the two series for all spans. The command can specify spans in any order, but if the time spans are coded in chronological sequence, and if they do not overlap, a large amount of computer time can be saved by making the value of IFY negative for all but the first PRINT. Output units are cubic meters and cubic meters per second for flow; mass and mass per second are used for pollutants.

Sample CALIBRATE Command:

```
-----------------------------------------------------------------
CALIBRATE           4 TIME SPANS FROM ID 2 AND ID 3
                    79 6 15 79 6 17
                    79 6 23 79 6 30
                    79 5 15 79 6 10
                    79 3 21 79 3 21
-----------------------------------------------------------------
```

This example compares mass and rates of output between two series for four different periods, specified in arbitrary order. Computer time could be saved in this example by arranging the spans in chronological order as follows:

```
79 3 21 79 3 21
79 5 15 79 6 10
79 6 15 79 6 17
79 6 23 79 6 30
```

D-6-12 EXCEEDANCE CURVES

Purpose

This command allows the user to determine the numbers and durations for which flow or pollutant concentration exceed certain levels. The levels which are assessed are determined by the user.

Parameters

Name	Description
NINQ	Number of points on flow exceedance curve (range = 1 through 10)
QLEVEL(1)	First flow level to be assessed. Omit if NINQ=0
...	...
QLEVEL(NINQ)	Last flow level to be assessed.
NIND	Number of points on concentration exceedance curve for arbitrary first order pollutant. (range = 1 through 10)
DLEVEL(1)	First concentration level to be assessed. Omit if NINQ=0
...	...
DLEVEL(NIND)	Last concentration level to be assessed.
NINS	Number of points on exceedance curve for total sediment concentration. (range = 1 through 10)
SLEVEL(1)	First sediment concentration level to be assessed. Omit if NINS=0
...	...
SLEVEL(NINS)	Last sediment concentration level to be assessed.
NUMINT	Number of time spans over which to compute exceedances of the specified QLEVEL, DLEVEL, and SLEVEL curves. (range = 1 through NMAX where NMAX = INT(305/12)
ID	ID number of series to be assessed.
IFY(1)	Beginning year, first span.
IFM(1)	'' month, '' ''
IFD(1)	'' day, '' ''
ITY(1)	Ending year, '' ''
ITM(1)	'' month, '' ''
ITD(1)	'' day, '' ''
...	...

IFY(NUMINT)	Beginning	year,	last	span.
IFM(NUMINT)	''	month,	''	''
IFD(NUMINT)	''	day,	''	''
ITY(NUMINT)	Ending	year,	''	''
ITM(NUMINT)	''	month,	''	''
ITD(NUMINT)	''	day,	''	''

Notes

This command is useful for determining the number and duration of times over any given period that a series exceeds a given objective level of concentration or flow rate. This may have applications in assessing the number of violations of a given standard, or in comparing controlled and uncontrolled series. Units of flow are cubic meters per second; units of concentration are mass per cubic meter.

Sample EXCEEDANCE CURVES Command:

```
------------------------------------------------------------------
EXCEEDANCE CURVES  5 POINTS ON FLOW CURVE ARE,
                   10, 20, 50, 100, AND 250 CFS
                   0 POINTS ON FIRST ORDER CURVE AND
                   0 POINTS ON SEDIMENT CURVE
                   ID NUMBER OF SERIES IS 3
                   1 INTERVAL WILL BE EXAMINED
                   FROM 76 4 1 TO 76 9 30 (ALL SUMMER)
------------------------------------------------------------------
```

In this case, a flow exceedance curve only is requested, for all of one summer.

D-6-13 STORE

Purpose

The STORE command allows the user to convert a WSC gauging record to the file format used by QUALHYMO. This is useful for calibration of flow simulation to observed records.

Parameters

Name	Description
ID	ID number assigned to the gauge station records.
ISER -	Series number for ID.
DT(ID)	Time step of the flow records
IFMET	Flag. If less than or equal to zero, the gauge records are assumed imperial and are converted from cubic feet per sec to cubic meters per second. Can be omitted if zero.

Notes

The disk file location (FORTRAN device number) for the flow series to be read is given in the START command. It is assumed that the flow series has hourly records ordered as follows:

```
IYEAR,IMONTH,IDAY,(FLOW(J),J=1,6)
IYEAR,IMONTH,IDAY,(FLOW(J),J=7,12)
IYEAR,IMONTH,IDAY,(FLOW(J),J=13,18)
IYEAR,IMONTH,IDAY,(FLOW(J),J=19,24)
```

ie 6 hours per card, 4 cards per day.

The format expected for the information on each card is:

```
(8X,3I2,2X,6F10.0)
```

The model will check the first 8 columns on the first card for a station identification code, and will echo this on the output if present.

The model will check that the dates on adjacent cards match properly, but will not recognise special characters indicating ice jams, non-operation, and so on. Negative flows will be set to zero, and the user will be informed of the number of times where this occured.

D-6-14 DUMP PRINT

Purpose

This command allows the user to print out any entire series in a legible format.

Parameters

Name	Description
ID	ID number of file to be dumped.

Notes

DUMP will print the entire contents of the file, including space allocated to quality variables, regardless of the START command or other controlling variables. The user should therefore take care that dummy fields are recognised and discounted. The content of the DUMP information is:

```
IYEAR,IMONTH,IDAY,HOUR,FLOW,DPARM,(SPARM(J),J=1,5)
```

where DPARM is the first order decay pollutant (mass per second),

and SPARM(J) is sediment of size J (mass per second).

The format of the DUMP is:

```
(3I2,F6.0,6E12.4)
```

Units of flow are cubic meters/second. Units of DPARM and SPARM are mass per second, consistent with the units used in the POLLUTANT RATES card. Similarly, the sediment size J is consistent with the definition used in the START card.

D-6-15 DUMP PLOT

Purpose

This command allows the user to plot data for the entire duration of the series, rather than for a span of 300 points as in the PLOT SPAN command. The user has the option of comparing one parameter (flow, the first order decay pollutant, or total sediment) for two series, or three parameters (flow, first order and sediment) for one series.

Parameters

Name	Description
ICASE	Flag. If=1, three parameters from one series are plotted. If=2, one parameter for two series is plotted.
ICTRL	Flag. If=1 or 2, mass rate is plotted. If=3 or 4, concentration is plotted. If=1 or 3, linear scale is used. If=2 or 4, log scale is used.
ID1	ID number of one series to be plotted.
ID2	ID number of second series to be plotted. Omit if ICASE=1.
IPARAM	Parameter to be plotted. Omit if ICASE=1. If IPARAM=1, flow is plotted. If IPARAM=2, the first order pollutant is plotted. If IPARAM=3, total sediment is plotted.
QMIN	Minimum value of flow to be plotted.
QMAX	Maximum value of flow to be plotted.
DMIN	Minimum value of first order pollutant to be plotted
DMAX	Maximum value of first order pollutant to be plotted.
SMIN	Minimum value of sediment to be plotted.
SMAX	Maximum value of sediment to be plotted.

Note: If ICASE=1, QMIN, QMAX, DMIN, DMAX, SMIN, and SMAX are all required. If ICASE=2, only input one pair as appropriate for IPARAM. (i.e. if IPARAM=1 input QMIN and QMAX only, if IPARAM=2 input DMIN and DMAX if IPARAM=3 input SMIN and SMAX)

Notes

This plotting routine is useful primarily for gaining an overall insight into the flow series and for determining during initial runs which time spans will be examined in the CALIBRATE, EXCEEDANCE, or other commands. Values in the series which exceed the specified maximums are truncated along the largest axis of the graph.

Rate units are cubic meters per second for flow, or mass per second for pollutants. Concentration is in units of mass per cubic meter.

Sample DUMP PLOT Command:

```
-----------------------------------------------------------------------
DUMP PLOT          ICASE=2, ICTRL=4, IPARAM=3 (LOG PLOT OF
                    SEDIMENT CONCENTRATION FROM TWO SERIES)
                   SERIES IDS ARE 2 AND 3
                   SEDIMENT RANGE IS .01 TO 1000 KG PER CU M
-----------------------------------------------------------------------
```

In this example, two series will be plotted for one parameter. The time span of the plot will be the same as on the preceding START command. Note that the units of the scale are arbitrary, and depend on the units used in setting up the quality constituent parameters in the POLLUTANT RATES cards.

If, for example, the appropriate parameters in the POLLUTANT RATES cards had specified sediment build-up in grams instead of kilograms, the last line of the above sample command should have read:

```
              SEDIMENT RANGE 0 TO 1000000 G PER CU M
```

to achieve the same range.

D-6-16 CONVOLUTE

Purpose

This command routes a flow series through a river reach. If pollutant simulation has been selected on the START command, associated pollutants will also be routed through the pond.

Parameters

Name	Description
IDOUT	ID number of outflow series from reach
ISER	Series number
NIDH	Number of sources tributary to reach at, headwater. (range = 1 through 6 but can be zero is NIDL is not zero) (if NIDH=0 omit all IDH(i) values)
IDH(1)	ID number of first source tributary to reach headwater.
...	...
IDH(NIDH)	ID number of last source tributary to reach headwater.
IFAORM	Flag. If =1, arbitrary channel input is expected. If =2, trapezoidal channel expected.
SMAX	Maximum expected depth of flow in channel (units meters)
XLEN	Reach length. (meters)
RTINC	Time increment for use in flow (but not pollutant) routing. (hours)
	ARBITRARY CHANNEL INPUT DATA ++++++++++++ ++++++ OMIT IF IFAROM =2 +++++++++++++++
COEF	Coefficient in flow/stage curve. (units cu. m. per sec. per m.)
EXPON	Exponent in flow/stage curve. (no units)
	REACH VOLUME DATA ++++++++++++++++++++++
NPTSV	Number of points on stage volume curve. (range = 2 through 25 but can be zero provided NPTSA is non-zero) (omit STAGE(i), VOLUME(i) pairs if NPTSV = 0)

STAGE(1)	First stage on volume curve.
VOLUME(1)	First volume on volume curve.
...	...
STAGE(NPTSV)	Last stage on volume curve.
VOLUME(NPTSV)	Last volume on volume curve.
	REACH HORIZONTAL AREA DATA ++++++++++++++
NPTSA	Number of points on stage area curve. (range = 2 through 25 but can be zero provided NPTSV is non-zero) (omit STAGE(i), AREA(i) pairs if NPTSA = 0)
STAGE(1)	First stage on area curve.
AREA(1)	First area on area curve.
...	...
STAGE(NPTSA)	Last stage on area curve.
AREA(NPTSA)	Last area on area curve.
	TRAPEZOIDAL CHANNEL INPUT DATA ++++++++++ ++++++ OMIT IF IFAORM = 1 ++++++++++++++
RN	Channel Manning 'n'.
SF	Channel friction slope. (m/m)
SS	Channel side slopes. (m/m)
B	Channel bottom width. (units meters)
	CONVOLUTION CURVE DATA +++++++++++++++++
QRESP	Flow for which the convolution curve applies
NCONV	Number of points in convolution curve
TIME(1)	Time lag of first fraction
MFRAC(1)	Value of first fraction
...	...
TIME(NCONV)	Time lag of last fraction
MFRAC(2)	Value of last fraction

Notes

The algorithms for flow rate used in the CONVOLUTE command are the same as those in the REACH command, documented in Appendix D. Note that all volumes are cubic meters, flows are cubic meters per second, areas are square meters, and lengths are meters.

When the non-trapezoidal channel is input, the minimum area and volume in the above curves should be zero. The Stages, however, can start at any arbitrary datum. Note that whatever elevation datum is used, it should be consistent with that which applies to the flow/ stage curve.

In general, the estimated maximum stage in the channel should be as low as possible provided that the flows do not exceed this limit. This improves the outflow estimation. Note that even if the stated maximum stage is exceeded, the model will usually provide a solution by extrapolation. This occurs with no warning, and the user should examine the peak flows calculated to determine whether an extrapolation occurred.

In the event that the Stage/Area curve is omitted, it is estimated from the Stage/Volume Curve. Conversely, the Stage /Area curve is used to estimate the Stage/Volume curve if that has not been supplied.

All of the input series (IDHs) must have the same time step; the output series has the same time step as the input series regardless of the value of RTINC. The convolution curve can be input at an arbitrary input step, as the model converts the input curve to a curve consistent with RTINC before beginning the routing.

Sample CONVOLUTE Command

```
-------------------------------------------------------------
CONVOLUTE           IDOUT IS 6, SERIES NAME IS 452
                    1 INPUT HEADWATER SERIES, ID=4
                    IFAORM=2 (TRAPEZOIDAL CHANNEL)
                    MAX EXPECTED DEPTH OF FLOW IS 6 M
                    LENGTH OF CHANNEL IS 10000M
                    FLOW CALCULATION TIME STEP IS 1.0 HOURS
                    CHANNEL ROUGHNESS IS 0.035
                            BOTTOM SLOPE IS .0001 M PER M
                            SIDE SLOPE IS 0.35 M PER M
                        AND BOTTOM WIDTH IS 10 M
                    CONVOLUTION CURVE BASED AT 5 CMS
                    AND HAS 5 POINTS:
                           TIME       FRACTION
                          (HOURS)      (MASS)
                            0            0
                            1           .3
                            2           .6
                            3           .1
                            4            0
-------------------------------------------------------------
```

In this example, a reach is defined which has one tributary headwater (ID4). Other parameters self explanatory. Note that the requested flow calculation time step of one hour will be used in routing flows through the reach provided that it is shorter than the time steps on the input time series (IDs 4, 5 and 1). However, the time step at which quality routing will occur, and the time step for storing data under this reach ID (7) will be the same as for the input IDs. Note also that the input IDs must all have the same time step for a routing to be possible.

APPENDIX E

MODEL OUTPUT

APPENDIX E - Model Output

This section contains examples of model outputs to illustrate the format of some of the principle commands, and to provide further documentation of the runs discussed in the main text of this report. The sections in this appendix contain:

1. a calibration to Sawmill Creek flow data;
2. a calibration to Wixon Creek flow data;
3. a calibration to Sawmill Creek bacteria data; and
4. an output plot of the observed and simulated Sawmill creek bacteria data.

All of this information corresponds to the calibrations described in the main text of this report.

CALIBRATION TO SAWMILL CREEK FLOW DATA

CALIBRATION TO SAWMILL CREEK FLOW
DATA -
STATISTICS FOR CALIBRATION,
VERIFICATION, AND TOTAL SERIES
OF EVENTS, 1981.

```
      ZALFA = 1234567890 +./-

            COMMAND TABLE

      START                      1 25
      STORE                      2310
      GENERATE                   3310
      PRINT SPAN                 4  2
      PLOT SPAN                  5 10
      ADD SERIES                 6  4
      POND                       7310
      REACH                      8310
      CALIBRATE                  9310
      POLLUTANT RATES           10  8
      SPLIT SERIES              11  5
      DUMP PRINT                12  9
      EXCEEDANCE CURVES         13310
      DUMP PLOT                 14  5
      FINISH                    15  0
 START                    FROM ALL RECORD (-1)
                           INPUT RAIN FROM DEVICE 8
                           DUMMY FLOW DEVICE # 5
                          NO DECAY OR SEDT SIMULATION
 GENERATE                 IO= 4 SERIES NO=444 DT=1.0 DA=8080 ACRES
                          NP=0.0 (DO NOT PRINT UNIT HYDS)
                          0.14 IS IMPERVIOUS AREA FRACTION
                          NW=1.0 (NASH HYD)
                           1.05 SLR AND TP=.2
                          MAX ABSTRACTION IMP AREA IS  .16
                          RUNOFF COEFF IMP AREA IS 1.00
                          NW=1.0 (NASH HYD)
                           1.05 SLR AND TP=.5
                          SMIN= 1 INS SMAX= 2000 INS K=3.5
                          API CONSTANT IS .9 PER DAY
                          API INITIAL VALUE= 1.5INS
                          MAX ABSTRACTION PER AREA= .17NS
                          SVOL INITIAL VALUE =0.66
                          BASE RESS CONST=.00134
                          AREA REDUC FACTOR=0.58
                          BASMIN=0.0

      IMPERVIOUS AREA UNIT HYDROGRAPH DATA:

           SHAPE CONSTANT, N =  1.050
           UNIT PEAK, QP =      178.12CFS

           SUM OF THE UNIT HYDROGRAPH ORDINATES =      0.880

        PERVIOUS AREA UNIT HYDROGRAPH DATA:

           SHAPE CONSTANT, N =  1.050
           UNIT PEAK, QP =      437.67CFS

           SUM OF THE UNIT HYDROGRAPH ORDINATES =      0.993
 API REDUCTION IS = 0.996E 00= 0.900E 00= PER STEP/DY
```

```
  CALIBRATE            8 EVENTS FROM FILES 4 AND 8
                        -81 7 13 81 7 14
                        -81 7 18 81 7 19
                        -81 7 28 81 7 29
                        -81 8  4 81 8  7
                        -81 8 11 81 8 11
                        -81 8 15 81 8 17
                        -81 8 23 81 8 24
                        -81 8 30 81 8 31
      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 713 81 714  444  TOTAL   0.780E 06 -0.0          0.0       0.0       0.0       0.0       0.0
                444  RATE    0.100E 02 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.112E 07 0.269E 13 0.0       0.0       0.0       0.0       0.0
                301  RATE    0.380E 02 0.178E 09 0.0       0.0       0.0       0.0       0.0

      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 718 81 719  444  TOTAL   0.290E 07 0.0           0.0       0.0       0.0       0.0       0.0
                444  RATE    0.948E 02 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.208E 07 0.405E 13 0.0       0.0       0.0       0.0       0.0
                301  RATE    0.731E 02 0.191E 09 0.0       0.0       0.0       0.0       0.0

      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 728 81 729  444  TOTAL   0.756E 06 0.0           0.0       0.0       0.0       0.0       0.0
                444  RATE    0.162E 02 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.963E 06 0.642E 12 0.0       0.0       0.0       0.0       0.0
                301  RATE    0.168E 02 0.384E 08 0.0       0.0       0.0       0.0       0.0

      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 8 4 81 8 7  444  TOTAL   0.369E 07 0.0           0.0       0.0       0.0       0.0       0.0
                444  RATE    0.280E 03 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.185E 08 0.654E 14 0.0       0.0       0.0       0.0       0.0
                301  RATE    0.535E 03 0.333E 10 0.0       0.0       0.0       0.0       0.0

      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 811 81 811  444  TOTAL   0.633E 06 0.0           0.0       0.0       0.0       0.0       0.0
                444  RATE    0.187E 02 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.763E 06 0.143E 12 0.0       0.0       0.0       0.0       0.0
                301  RATE    0.204E 02 0.579E 08 0.0       0.0       0.0       0.0       0.0

      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 815 81 817  444  TOTAL   0.546E 07 0.0           0.0       0.0       0.0       0.0       0.0
                444  RATE    0.808E 02 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.452E 07 0.552E 12 0.0       0.0       0.0       0.0       0.0
                301  RATE    0.989E 02 0.676E 08 0.0       0.0       0.0       0.0       0.0

      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 823 81 824  444  TOTAL   0.338E 07 0.0           0.0       0.0       0.0       0.0       0.0
                444  RATE    0.157E 02 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.116E 07 0.424E 12 0.0       0.0       0.0       0.0       0.0
                301  RATE    0.224E 02 0.720E 08 0.0       0.0       0.0       0.0       0.0

      SPAN       NAME                          QUANTITY
   FROM    TO                FLOW       POL1        SED1      SED2      SED3      SED4      SED5
-81 830 81 831  444  TOTAL   0.239E 07 0.0           0.0       0.0       0.0       0.0       0.0
                444  RATE    0.743E 02 0.0           0.0       0.0       0.0       0.0       0.0

                301  TOTAL   0.167E 07 0.0           0.0       0.0       0.0       0.0       0.0
                301  RATE    0.379E 02 0.0           0.0       0.0       0.0       0.0       0.0

 ***** TOTAL MASS *****
  SERIES  444        0.240E 08 0.0         0.0       0.0       0.0       0.0       0.0

  SERIES  301        0.308E 08 0.751E 14 0.0       0.0       0.0       0.0       0.0

1***** SUMMARY COMPARISON OF SERIES   444 AND   301 *****

 AVE (MASS RATIO)           1.017     0.0       0.0       0.0       0.0       0.0       0.0
 AVE (RATE RATIO)           0.944     0.0       0.0       0.0       0.0       0.0       0.0

 RMS (MASS RATIO-1)         0.309     1.000     1.000     1.000     1.000     1.000     1.000
 RMS (RATE RATIO-1)         0.482     1.000     1.000     1.000     1.000     1.000     1.000
    NOTE: RATIO = MASS OR RATE FROM SERIES    444         DIVIDED BY MASS OR RATE FROM SERIES
```

```
CALIBRATE              9 EVENTS FROM FILES 4 AND 6
                        81 5 11 81 5 14
                       -81 5 15 81 5 18
                       -81 5 26 81 5 27
                       -81 5 28 81 5 31
                       -81 6  6 81 6  7
                       -81 6  8 81 6  9
                       -81 6 12 81 6 13
                       -81 6 15 81 6 16
                       -81 7  9 81 7 10
      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
 81 511 81 514 444   TOTAL   0.905E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.872E 02 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.821E 07 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.797E 02 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 515 81 518 444   TOTAL   0.753E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.140E 03 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.964E 07 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.151E 03 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 526 81 527 444   TOTAL   0.322E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.120E 03 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.277E 07 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.995E 02 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 528 81 531 444   TOTAL   0.113E 08 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.193E 03 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.132E 08 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.195E 03 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 6 6 81 6 7 444   TOTAL   0.259E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.784E 02 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.232E 07 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.600E 02 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 6 8 81 6 9 444   TOTAL   0.161E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.274E 02 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.155E 07 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.329E 02 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 612 81 613 444   TOTAL   0.212E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.542E 02 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.161E 07 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.323E 02 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 615 81 616 444   TOTAL   0.832E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.233E 03 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.801E 07 0.0              0.0         0.0        0.0        0.0        0.0
               301   RATE    0.287E 03 0.0              0.0         0.0        0.0        0.0        0.0

      SPAN        NAME                              QUANTITY
   FROM    TO                  FLOW         POL1         SED1        SED2       SED3       SED4       SED5
-81 7 9 81 710 444   TOTAL   0.130E 07 0.0              0.0         0.0        0.0        0.0        0.0
               444   RATE    0.400E 02 0.0              0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.187E 07 0.140E 14 0.0                0.0        0.0        0.0        0.0
               301   RATE    0.879E 02 0.123E 10 0.0                0.0        0.0        0.0        0.0

***** TOTAL MASS *****
  SERIES  444          0.451E 08 0.0         0.0        0.0        0.0        0.0        0.0

  SERIES  301          0.480E 08 0.140E 14 0.0          0.0        0.0        0.0        0.0

1***** SUMMARY COMPARISON OF SERIES   444 AND   301 *****

 AVE (MASS RATIO)            0.997      0.0        0.0        0.0        0.0        0.0        0.0
 AVE (RATE RATIO)            0.972      0.0        0.0        0.0        0.0        0.0        0.0

 RMS (MASS RATIO-1)          0.181      1.000      1.000      1.000      1.000      1.000      1.000
 RMS (RATE RATIO-1)          0.243      1.000      1.000      1.000      1.000      1.000      1.000
    NOTE: RATIO = MASS OR RATE FROM SERIES    444                 DIVIDED BY MASS OR RATE FROM SERIES   3
```

```
CALIBRATE            17 EVENTS FROM FILES 4 AND 5
                       81 5 11 81 5 14
                      -81 5 15 81 5 18
                      -81 5 26 81 5 27
                      -81 5 28 81 5 31
                      -81 6  6 81 6  7
                      -81 6  8 81 6  9
                      -81 6 12 81 6 13
                      -81 6 15 81 6 16
                      -81 7  9 81 7 10
                      -81 7 13 81 7 14
                      -81 7 18 81 7 19
                      -81 7 24 81 7 25
                      -81 8  4 81 8  7
                      -81 8 11 81 8 11
                      -81 8 15 81 8 17
                      -81 8 23 81 8 24
                      -81 8 30 81 8 31
     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
 81 511 81 514 444   TOTAL    0.905E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.872E 02 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.821E 07 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.797E 02 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 515 81 518 444   TOTAL    0.753E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.140E 03 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.864E 07 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.151E 03 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 526 81 527 444   TOTAL    0.322E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.120E 03 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.277E 07 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.835E 02 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 528 81 531 444   TOTAL    0.113E 08 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.193E 03 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.132E 08 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.195E 03 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 6 6 81 6 7 444   TOTAL    0.258E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.764E 02 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.232E 07 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.660E 02 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 6 8 81 6 9 444   TOTAL    0.161E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.274E 02 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.155E 07 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.328E 02 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 612 81 613 444   TOTAL    0.212E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.342E 02 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.161E 07 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.523E 02 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 615 81 616 444   TOTAL    0.632E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.233E 03 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.609E 07 0.0          0.0         0.0         0.0         0.0         0.0
               301   RATE     0.287E 03 0.0          0.0         0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 7 9 81 710 444   TOTAL    0.138E 07 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.400E 02 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.167E 07 0.140E 14 0.0        0.0         0.0         0.0         0.0
               301   RATE     0.879E 02 0.123E 10 0.0        0.0         0.0         0.0         0.0

     SPAN        NAME                               QUANTITY
  FROM    TO                    FLOW        POL1         SED1        SED2        SED3        SED4        SED5
-81 713 81 714 444   TOTAL    0.780E 06 0.0          0.0         0.0         0.0         0.0         0.0
               444   RATE     0.100E 02 0.0          0.0         0.0         0.0         0.0         0.0

               301   TOTAL    0.112E 07 0.269E 13 0.0        0.0         0.0         0.0         0.0
               301   RATE     0.360E 02 0.178E 08 0.0        0.0         0.0         0.0         0.0
```

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      SPAN         NAME                                 QUANTITY
   FROM    TO                      FLOW        POL1          SED1         SED2         SED3         SED4         SED5
 -81 718 81 719 444   TOTAL      0.280E 07 0.0              0.0          0.0          0.0          0.0          0.0
                444   RATE       0.946E 02 0.0              0.0          0.0          0.0          0.0          0.0

                301   TOTAL      0.208E 07 0.409E 13 0.0                 0.0          0.0          0.0          0.0
                301   RATE       0.731E 02 0.191E 08 0.0                 0.0          0.0          0.0          0.0

      SPAN         NAME                                 QUANTITY
   FROM    TO                      FLOW        POL1          SED1         SED2         SED3         SED4         SED5
 -81 728 81 729 444   TOTAL      0.756E 06 0.0              0.0          0.0          0.0          0.0          0.0
                444   RATE       0.182E 02 0.0              0.0          0.0          0.0          0.0          0.0

                301   TOTAL      0.583E 06 0.842E 12 0.0                 0.0          0.0          0.0          0.0
                301   RATE       0.168E 02 0.364E 08 0.0                 0.0          0.0          0.0          0.0

      SPAN         NAME                                 QUANTITY
   FROM    TO                      FLOW        POL1          SED1         SED2         SED3         SED4         SED5
 -81 8 4 81 8 7 444   TOTAL      0.869E 07 0.0              0.0          0.0          0.0          0.0          0.0
                444   RATE       0.280E 03 0.0              0.0          0.0          0.0          0.0          0.0

                301   TOTAL      0.185E 08 0.884E 14 0.0                 0.0          0.0          0.0          0.0
                301   RATE       0.535E 03 0.333E 10 0.0                 0.0          0.0          0.0          0.0

      SPAN         NAME                                 QUANTITY
   FROM    TO                      FLOW        POL1          SED1         SED2         SED3         SED4         SED5
 -81 811 81 811 444   TOTAL      0.833E 06 0.0              0.0          0.0          0.0          0.0          0.0
                444   RATE       0.187E 02 0.0              0.0          0.0          0.0          0.0          0.0

                301   TOTAL      0.763E 06 0.143E 12 0.0                 0.0          0.0          0.0          0.0
                301   RATE       0.204E 02 0.375E 08 0.0                 0.0          0.0          0.0          0.0

      SPAN         NAME                                 QUANTITY
   FROM    TO                      FLOW        POL1          SED1         SED2         SED3         SED4         SED5
 -81 815 81 817 444   TOTAL      0.546E 07 0.0              0.0          0.0          0.0          0.0          0.0
                444   RATE       0.906E 02 0.0              0.0          0.0          0.0          0.0          0.0

                301   TOTAL      0.492E 07 0.952E 12 0.0                 0.0          0.0          0.0          0.0
                301   RATE       0.909E 02 0.876E 08 0.0                 0.0          0.0          0.0          0.0

      SPAN         NAME                                 QUANTITY
   FROM    TO                      FLOW        POL1          SED1         SED2         SED3         SED4         SED5
 -81 823 81 824 444   TOTAL      0.116E 07 0.0              0.0          0.0          0.0          0.0          0.0
                444   RATE       0.157E 02 0.0              0.0          0.0          0.0          0.0          0.0

                301   TOTAL      0.116E 07 0.424E 12 0.0                 0.0          0.0          0.0          0.0
                301   RATE       0.224E 02 0.728E 08 0.0                 0.0          0.0          0.0          0.0

      SPAN         NAME                                 QUANTITY
   FROM    TO                      FLOW        POL1          SED1         SED2         SED3         SED4         SED5
 -81 830 81 831 444   TOTAL      0.239E 07 0.0              0.0          0.0          0.0          0.0          0.0
                444   RATE       0.743E 02 0.0              0.0          0.0          0.0          0.0          0.0

                301   TOTAL      0.187E 07 0.0              0.0          0.0          0.0          0.0          0.0
                301   RATE       0.379E 02 0.0              0.0          0.0          0.0          0.0          0.0

 ***** TOTAL MASS *****
   SERIES  444         0.891E 08 0.0          0.0          0.0          0.0          0.0          0.0

   SERIES  301         0.798E 08 0.892E 14 0.0          0.0          0.0          0.0          0.0
1***** SUMMARY COMPARISON OF SERIES   444 AND   301 *****

 AVE (MASS RATIO)           1.008        0.0          0.0          0.0          0.0          0.0          0.0
 AVE (RATE RATIO)           0.959        0.0          0.0          0.0          0.0          0.0          0.0

 RMS (MASS RATIO-1)         0.249        1.000        1.000        1.000        1.000        1.000        1.000
 RMS (RATE RATIO-1)         0.375        1.000        1.000        1.000        1.000        1.000        1.000
    NOTE: RATIO = MASS OR RATE FROM SERIES  444                    DIVIDED BY MASS OR RATE FROM SERIES   3
```

CALIBRATION TO WIXON CREEK FLOW DATA

CALIBRATION TO WIXON CREEK
FLOW DATA -
MODEL OUTPUT

```
     ZALFA = 1234567890 +./-

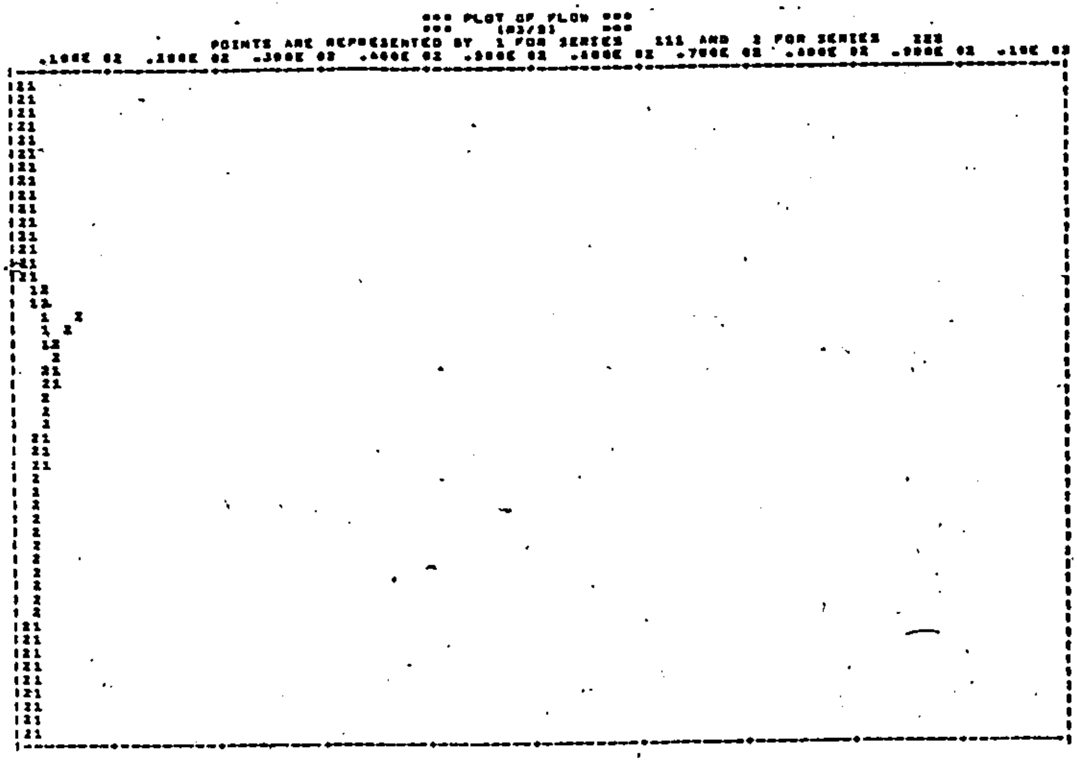
            COMMAND TABLE

     START                 1 25
     STORE                 2310
     GENERATE              3310
     PRINT SPAN            4  2
     PLOT SPAN             5 10
     ADD SERIES            6  4
     POND                  7310
     REACH                 8310
     CALIBRATE             9310
     POLLUTANT RATES      10  9
     SPLIT SERIES         11  5
     DUMP PRINT           12  5
     EXCEEDANCE CURVES    13310
     DUMP PLOT            14  4
     FINISH               15  0
START                  FROM ALL FILE (-1)
                        INPUT RAIN FROM DEVICE 9
                        FLOW DEVICE=17
                       NO FIRST ORDER POLLUTANT
                       NO  SEDIMENT SIMULATION
GENERATE               IDN 1 SERIES NO=111 DT=1.0 DA=1002 ACRES
                       NP=0.0 (DO NOT PRINT UNIT HYDS)
                       0.00 (NONE) IMPERVIOUS AREA
                       NU=1.0 (NASH HYD)
                        2.0 SLR AND TP=4.5
                       SPIN= 3 INS SMAX= 15.0 INS SK=.4
                       API CONSTANT IS .8 PER DAY
                       API INITIAL VALUE= 1.5INS
                       MAX ABSTRACTION PER AREA= .04NS
                       EVOL INITIAL VALUE =10
                       BASE RESS CONST=.005
                       AREA REDUC FACTOR=0.2
                       BASKIN=1.5

      PERVIOUS AREA UNIT HYDROGRAPH DATA:

         SHAPE CONSTANT, N =  2.000
         UNIT PEAK, QP =    200.83CFS

         SUM OF THE UNIT HYDROGRAPH ORDINATES =      0.998
API REDUCTION IS = 0.995E 00= 0.900E 00= PER STEP/DT
 START                 FROM 76 06 01 TO 76 08 30
                        INPUT RAIN FROM DEVICE 9
                        FLOW DEVICE=17
                       NO FIRST ORDER POLLUTANT
                       NO  SEDIMENT SIMULATION
 DUMP PLOT             ICASE=2, ICTRL=1,  ID 1 AND 2
                       IPARAM=1 (FLOW IS PLOTTED)
                       FLOW RANGE IS 0 TO 100 CFS
```



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PAGE BEGINS AT 1976, 6,13
                    .0
DAY HR. I  RATE1  I  RATE2  I
13   7.00I0.337E 01I0.230E 01 I21
13   8.00I0.336E 01I0.230E 01 I21
13   9.00I0.334E 01I0.230E 01 I21
13  10.00I0.332E 01I0.226E 01 I21
13  11.00I0.330E 01I0.226E 01 I21
13  12.00I0.328E 01I0.226E 01 I21
13  13.00I0.327E 01I0.226E 01 I21
13  14.00I0.325E 01I0.222E 01 I21
13  15.00I0.323E 01I0.219E 01 I21
13  16.00I0.321E 01I0.212E 01 I21
13  17.00I0.320E 01I0.208E 01 I21
13  18.00I0.318E 01I0.201E 01 I21
13  19.00I0.316E 01I0.201E 01 I21
13  20.00I0.315E 01I0.209E 01 I21
13  21.00I0.313E 01I0.208E 01 I21
13  22.00I0.311E 01I0.403E 01 I 12
13  23.00I0.374E 01I0.636E 01 I 12
13  24.00I0.417E 01I0.728E 01 I   1  2
14   1.00I0.460E 01I0.628E 01 I   1 2
14   2.00I0.498E 01I0.603E 01 I   12
14   3.00I0.512E 01I0.533E 01 I    2
14   4.00I0.513E 01I0.484E 01 I   21
14   5.00I0.501E 01I0.463E 01 I   21
14   6.00I0.469E 01I0.434E 01 I   2
14   7.00I0.447E 01I0.403E 01 I   2
14   8.00I0.449E 01I0.403E 01 I   2
14   9.00I0.431E 01I0.319E 01 I  21
14  10.00I0.415E 01I0.395E 01 I  21
14  11.00I0.401E 01I0.380E 01 I  21
14  12.00I0.399E 01I0.375E 01 I  2
14  13.00I0.378E 01I0.384E 01 I  2
14  14.00I0.369E 01I0.393E 01 I  2
14  15.00I0.362E 01I0.339E 01 I  2
14  16.00I0.356E 01I0.325E 01 I  2
14  17.00I0.350E 01I0.311E 01 I  2
14  18.00I0.340E 01I0.308E 01 I  2
14  19.00I0.342E 01I0.300E 01 I  2
14  20.00I0.336E 01I0.304E 01 I  2
14  21.00I0.339E 01I0.304E 01 I  2
14  22.00I0.333E 01I0.300E 01 I  2
14  23.00I0.330E 01I0.297E 01 I21
14  24.00I0.326E 01I0.293E 01 I21
15   1.00I0.320E 01I0.290E 01 I21
15   2.00I0.323E 01I0.288E 01 I21
15   3.00I0.321E 01I0.288E 01 I21
15   4.00I0.319E 01I0.290E 01 I21
15   5.00I0.318E 01I0.290E 01 I21
15   6.00I0.316E 01I0.293E 01 I21
15   7.00I0.314E 01I0.297E 01 I21
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                                                                              *** PLOT OF FLOW ***
                                                                              ***    (M3/S)    ***
PAGE BEGINS AT 1976, 6.15                        POINTS ARE REPRESENTED BY  1 FOR SERIES    111 AND  2 FOR SERIES   222
                     .0           .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
DAY HR. I  RATE1   I  RATE2     I-----------------------------------------------------------------------------------------------I
15   8.00I0.313E 01I0.297E 01  I21                                                                                              I
15   9.00I0.311E 01I0.293E 01  I21                                                                                              I
15  10.00I0.309E 01I0.290E 01  I21                                                                                              I
15  11.00I0.308E 01I0.288E 01  I21                                                                                              I
15  12.00I0.306E 01I0.279E 01  I21                                                                                              I
15  13.00I0.305E 01I0.285E 01  I21                                                                                              I
15  14.00I0.303E 01I0.291E 01  I21                                                                                              I
15  15.00I0.302E 01I0.247E 01  I21                                                                                              I
15  16.00I0.300E 01I0.247E 01  I21                                                                                              I
15  17.00I0.299E 01I0.247E 01  I2                                                                                               I
15  18.00I0.297E 01I0.244E 01  I2                                                                                               I
15  19.00I0.296E 01I0.237E 01  I2                                                                                               I
15  20.00I0.294E 01I0.230E 01  I2                                                                                               I
15  21.00I0.293E 01I0.226E 01  I2                                                                                               I
15  22.00I0.291E 01I0.226E 01  I2                                                                                               I
15  23.00I0.290E 01I0.226E 01  I2                                                                                               I
15  24.00I0.289E 01I0.226E 01  I2                                                                                               I
16   1.00I0.287E 01I0.226E 01  I2                                                                                               I
16   2.00I0.286E 01I0.230E 01  I2                                                                                               I
16   3.00I0.284E 01I0.233E 01  I2                                                                                               I
16   4.00I0.283E 01I0.237E 01  I2                                                                                               I
16   5.00I0.282E 01I0.244E 01  I2                                                                                               I
16   6.00I0.281E 01I0.254E 01  I2                                                                                               I
16   7.00I0.279E 01I0.261E 01  I2                                                                                               I
16   8.00I0.278E 01I0.261E 01  I2                                                                                               I
16   9.00I0.277E 01I0.261E 01  I2                                                                                               I
16  10.00I0.275E 01I0.247E 01  I2                                                                                               I
16  11.00I0.274E 01I0.261E 01  I2                                                                                               I
16  12.00I0.273E 01I0.261E 01  I2                                                                                               I
16  13.00I0.272E 01I0.261E 01  I2                                                                                               I
16  14.00I0.273E 01I0.261E 01  I2                                                                                               I
16  15.00I0.272E 01I0.254E 01  I2                                                                                               I
16  16.00I0.271E 01I0.254E 01  I2                                                                                               I
16  17.00I0.270E 01I0.254E 01  I2                                                                                               I
16  18.00I0.268E 01I0.254E 01  I2                                                                                               I
16  19.00I0.267E 01I0.258E 01  I2                                                                                               I
16  20.00I0.266E 01I0.258E 01  I2                                                                                               I
16  21.00I0.265E 01I0.258E 01  I2                                                                                               I
16  22.00I0.264E 01I0.261E 01  I2                                                                                               I
16  23.00I0.263E 01I0.261E 01  I2                                                                                               I
16  24.00I0.261E 01I0.261E 01  I2                                                                                               I
17   1.00I0.260E 01I0.261E 01  I2                                                                                               I
17   2.00I0.259E 01I0.258E 01  I2                                                                                               I
17   3.00I0.258E 01I0.259E 01  I2                                                                                               I
17   4.00I0.257E 01I0.255E 01  I2                                                                                               I
17   5.00I0.256E 01I0.261E 01  I2                                                                                               I
17   6.00I0.255E 01I0.261E 01  I2                                                                                               I
17   7.00I0.254E 01I0.261E 01  I2                                                                                               I
17   8.00I0.253E 01I0.261E 01  I2                                                                                               I
                                I-----------------------------------------------------------------------------------------------I
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                                                                              *** PLOT OF FLOW ***
                                                                              ***    (M3/S)    ***
PAGE BEGINS AT 1976, 6.17                        POINTS ARE REPRESENTED BY  1 FOR SERIES    111 AND  2 FOR SERIES   222
                     .0           .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
DAY HR. I  RATE1   I  RATE2     I-----------------------------------------------------------------------------------------------I
17   9.00I0.252E 01I0.261E 01  I2                                                                                               I
17  10.00I0.251E 01I0.261E 01  I2                                                                                               I
17  11.00I0.250E 01I0.251E 01  I2                                                                                               I
17  12.00I0.249E 01I0.256E 01  I2                                                                                               I
17  13.00I0.248E 01I0.258E 01  I2                                                                                               I
17  14.00I0.247E 01I0.256E 01  I2                                                                                               I
17  15.00I0.246E 01I0.258E 01  I2                                                                                               I
17  16.00I0.245E 01I0.254E 01  I2                                                                                               I
17  17.00I0.244E 01I0.254E 01  I2                                                                                               I
17  18.00I0.243E 01I0.254E 01  I2                                                                                               I
17  19.00I0.242E 01I0.254E 01  I2                                                                                               I
17  20.00I0.241E 01I0.251E 01  I2                                                                                               I
17  21.00I0.240E 01I0.251E 01  I2                                                                                               I
17  22.00I0.239E 01I0.251E 01  I2                                                                                               I
17  23.00I0.238E 01I0.251E 01  I2                                                                                               I
17  24.00I0.238E 01I0.251E 01  I2                                                                                               I
18   1.00I0.237E 01I0.251E 01  I2                                                                                               I
18   2.00I0.236E 01I0.251E 01  I2                                                                                               I
18   3.00I0.235E 01I0.251E 01  I2                                                                                               I
18   4.00I0.234E 01I0.251E 01  I2                                                                                               I
18   5.00I0.233E 01I0.251E 01  I2                                                                                               I
18   6.00I0.232E 01I0.251E 01  I2                                                                                               I
18   7.00I0.231E 01I0.251E 01  I2                                                                                               I
18   8.00I0.231E 01I0.251E 01  I2                                                                                               I
18   9.00I0.230E 01I0.251E 01  I2                                                                                               I
18  10.00I0.229E 01I0.251E 01  I2                                                                                               I
18  11.00I0.228E 01I0.251E 01  I2                                                                                               I
18  12.00I0.228E 01I0.247E 01  I2                                                                                               I
18  13.00I0.227E 01I0.247E 01  I2                                                                                               I
18  14.00I0.226E 01I0.247E 01  I2                                                                                               I
18  15.00I0.225E 01I0.247E 01  I2                                                                                               I
18  16.00I0.225E 01I0.247E 01  I2                                                                                               I
18  17.00I0.224E 01I0.247E 01  I2                                                                                               I
18  18.00I0.223E 01I0.244E 01  I2                                                                                               I
18  19.00I0.222E 01I0.240E 01  I2                                                                                               I
18  20.00I0.222E 01I0.237E 01  I2                                                                                               I
18  21.00I0.221E 01I0.233E 01  I2                                                                                               I
18  22.00I0.220E 01I0.230E 01  I2                                                                                               I
18  23.00I0.220E 01I0.226E 01  I2                                                                                               I
18  24.00I0.219E 01I0.222E 01  I2                                                                                               I
19   1.00I0.218E 01I0.222E 01  I2                                                                                               I
19   2.00I0.218E 01I0.222E 01  I2                                                                                               I
19   3.00I0.217E 01I0.222E 01  I2                                                                                               I
19   4.00I0.216E 01I0.222E 01  I2                                                                                               I
19   5.00I0.216E 01I0.226E 01  I2                                                                                               I
19   6.00I0.215E 01I0.226E 01  I2                                                                                               I
19   7.00I0.214E 01I0.230E 01  I2                                                                                               I
19   8.00I0.218E 01I0.240E 01  I2                                                                                               I
19   9.00I0.213E 01I0.247E 01  I2                                                                                               I
                                I-----------------------------------------------------------------------------------------------I
```
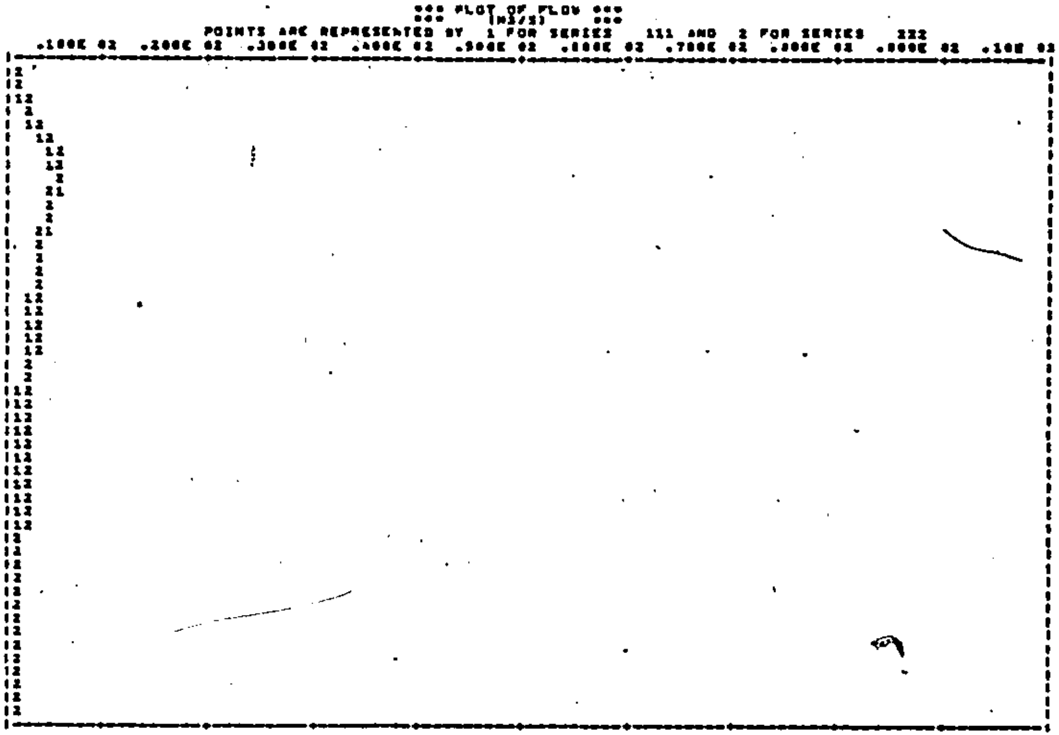

```
PAGE BEGINS AT 1978, 6,19
                        .0
DAY HR. |  RATE1   |  RATE2
19 10.00|0.223E 01|0.250E 01
19 11.00|0.233E 01|0.279E 01
19 12.00|0.286E 01|0.314E 01
19 13.00|0.307E 01|0.349E 01
19 14.00|0.382E 01|0.459E 01
19 15.00|0.478E 01|0.544E 01
19 16.00|0.568E 01|0.607E 01
19 17.00|0.583E 01|0.618E 01
19 18.00|0.615E 01|0.604E 01
19 19.00|0.616E 01|0.572E 01
19 20.00|0.600E 01|0.537E 01
19 21.00|0.571E 01|0.508E 01
19 22.00|0.538E 01|0.481E 01
19 23.00|0.500E 01|0.468E 01
19 24.00|0.465E 01|0.470E 01
20  1.00|0.433E 01|0.459E 01
20  2.00|0.409E 01|0.446E 01
20  3.00|0.381E 01|0.438E 01
20  4.00|0.361E 01|0.427E 01
20  5.00|0.344E 01|0.428E 01
20  6.00|0.330E 01|0.416E 01
20  7.00|0.318E 01|0.403E 01
20  8.00|0.310E 01|0.399E 01
20  9.00|0.302E 01|0.389E 01
20 10.00|0.286E 01|0.378E 01
20 11.00|0.281E 01|0.371E 01
20 12.00|0.287E 01|0.364E 01
20 13.00|0.284E 01|0.357E 01
20 14.00|0.281E 01|0.346E 01
20 15.00|0.276E 01|0.338E 01
20 16.00|0.275E 01|0.332E 01
20 17.00|0.273E 01|0.325E 01
20 18.00|0.271E 01|0.318E 01
20 19.00|0.270E 01|0.311E 01
20 20.00|0.288E 01|0.304E 01
20 21.00|0.287E 01|0.297E 01
20 22.00|0.268E 01|0.290E 01
20 23.00|0.259E 01|0.282E 01
20 24.00|0.264E 01|0.275E 01
21  1.00|0.263E 01|0.279E 01
21  2.00|0.262E 01|0.270E 01
21  3.00|0.260E 01|0.282E 01
21  4.00|0.259E 01|0.281E 01
21  5.00|0.256E 01|0.272E 01
21  6.00|0.257E 01|0.288E 01
21  7.00|0.250E 01|0.288E 01
21  8.00|0.255E 01|0.268E 01
21  9.00|0.254E 01|0.258E 01
21 10.00|0.252E 01|0.268E 01
```



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PAGE BEGINS AT 1978, 6,21
                        .0
DAY HR. |  RATE1   |  RATE2
21 11.00|0.252E 01|0.268E 01
21 12.00|0.251E 01|0.259E 01
21 13.00|0.250E 01|0.261E 01
21 14.00|0.248E 01|0.261E 01
21 15.00|0.249E 01|0.258E 01
21 16.00|0.247E 01|0.254E 01
21 17.00|0.246E 01|0.251E 01
21 18.00|0.245E 01|0.251E 01
21 19.00|0.244E 01|0.251E 01
21 20.00|0.243E 01|0.251E 01
21 21.00|0.242E 01|0.251E 01
21 22.00|0.241E 01|0.251E 01
21 23.00|0.240E 01|0.251E 01
21 24.00|0.240E 01|0.251E 01
22  1.00|0.239E 01|0.251E 01
22  2.00|0.238E 01|0.251E 01
22  3.00|0.237E 01|0.254E 01
22  4.00|0.236E 01|0.261E 01
22  5.00|0.235E 01|0.269E 01
22  6.00|0.234E 01|0.269E 01
22  7.00|0.233E 01|0.269E 01
22  8.00|0.233E 01|0.269E 01
22  9.00|0.232E 01|0.259E 01
22 10.00|0.231E 01|0.259E 01
22 11.00|0.230E 01|0.259E 01
22 12.00|0.229E 01|0.261E 01
22 13.00|0.228E 01|0.261E 01
22 14.00|0.228E 01|0.261E 01
22 15.00|0.227E 01|0.261E 01
22 16.00|0.226E 01|0.251E 01
22 17.00|0.226E 01|0.250E 01
22 18.00|0.225E 01|0.254E 01
22 19.00|0.224E 01|0.251E 01
22 20.00|0.223E 01|0.250E 01
22 21.00|0.223E 01|0.254E 01
22 22.00|0.222E 01|0.254E 01
22 23.00|0.221E 01|0.254E 01
22 24.00|0.220E 01|0.251E 01
23  1.00|0.220E 01|0.254E 01
23  2.00|0.218E 01|0.250E 01
23  3.00|0.218E 01|0.250E 01
23  4.00|0.214E 01|0.261E 01
23  5.00|0.217E 01|0.265E 01
23  6.00|0.216E 01|0.266E 01
23  7.00|0.216E 01|0.266E 01
23  8.00|0.215E 01|0.272E 01
23  9.00|0.214E 01|0.275E 01
23 10.00|0.214E 01|0.279E 01
23 11.00|0.213E 01|0.279E 01
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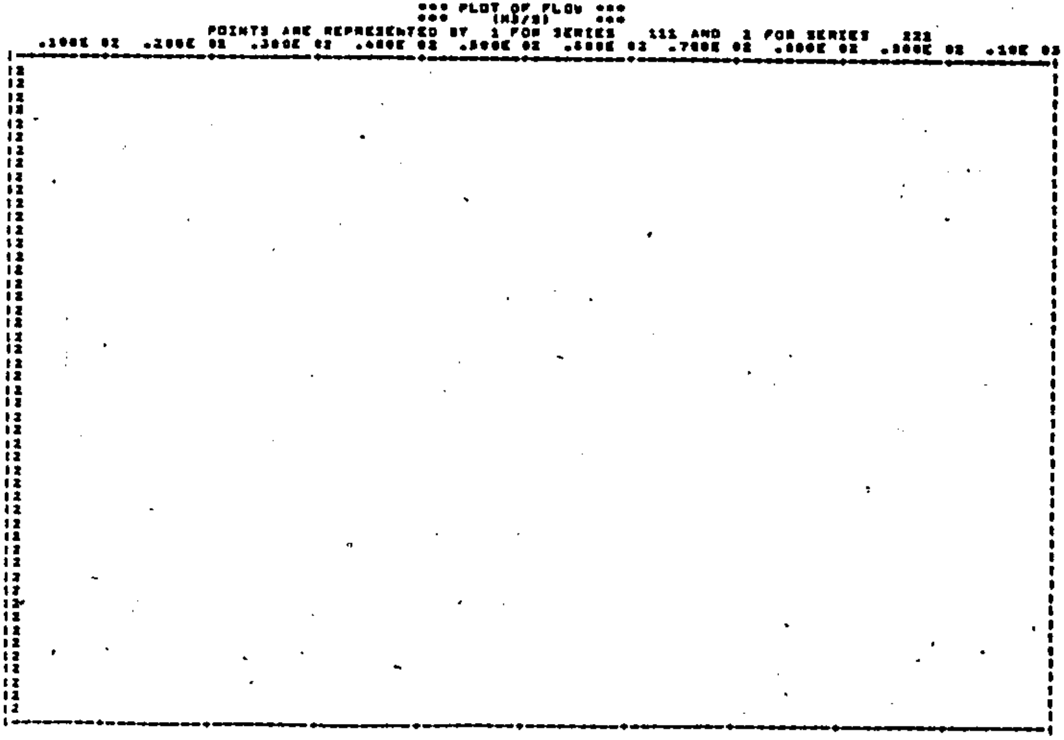

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                                                                              ***          (M3/S)           ***
PAGE BEGINS AT 1976, 6,23                                         POINTS ARE REPRESENTED BY  1 FOR SERIES   111 AND  2 FOR SERIES   222
                       .0                 .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
DAY HR.  I   RATE1   I   RATE2    I-------------------------------------------------------------------------------------------------------I
23 12.00I0.212E 01I0.279E 01  I2                                                                                                      I
23 13.00I0.212E 01I0.279E 01  I2                                                                                                      I
23 14.00I0.211E 01I0.272E 01  I2                                                                                                      I
23 15.00I0.211E 01I0.272E 01  I2                                                                                                      I
23 16.00I0.210E 01I0.268E 01  I2                                                                                                      I
23 17.00I0.209E 01I0.294E 01  I2                                                                                                      I
23 18.00I0.209E 01I0.291E 01  I2                                                                                                      I
23 19.00I0.208E 01I0.291E 01  I2                                                                                                      I
23 20.00I0.208E 01I0.237E 01  I2                                                                                                      I
23 21.00I0.207E 01I0.215E 01  I2                                                                                                      I
23 22.00I0.207E 01I0.215E 01  I2                                                                                                      I
23 23.00I0.206E 01I0.211E 01  I2                                                                                                      I
23 24.00I0.205E 01I0.222E 01  I2                                                                                                      I
24  1.00I0.205E 01I0.230E 01  I2                                                                                                      I
24  2.00I0.204E 01I0.233E 01  I2                                                                                                      I
24  3.00I0.204E 01I0.237E 01  I2                                                                                                      I
24  4.00I0.203E 01I0.237E 01  I2                                                                                                      I
24  5.00I0.203E 01I0.240E 01  I2                                                                                                      I
24  6.00I0.202E 01I0.240E 01  I2                                                                                                      I
24  7.00I0.202E 01I0.244E 01  I2                                                                                                      I
24  8.00I0.201E 01I0.244E 01  I2                                                                                                      I
24  9.00I0.201E 01I0.244E 01  I2                                                                                                      I
24 10.00I0.200E 01I0.237E 01  I2                                                                                                      I
24 11.00I0.200E 01I0.240E 01  I2                                                                                                      I
24 12.00I0.199E 01I0.258E 01  I2                                                                                                      I
24 13.00I0.199E 01I0.279E 01  I2                                                                                                      I
24 14.00I0.198E 01I0.297E 01  I2                                                                                                      I
24 15.00I0.198E 01I0.298E 01  I2                                                                                                      I
24 16.00I0.197E 01I0.282E 01  I2                                                                                                      I
24 17.00I0.197E 01I0.279E 01  I2                                                                                                      I
24 18.00I0.196E 01I0.265E 01  I2                                                                                                      I
24 19.00I0.196E 01I0.285E 01  I2                                                                                                      I
24 20.00I0.195E 01I0.265E 01  I2                                                                                                      I
24 21.00I0.195E 01I0.261E 01  I2                                                                                                      I
24 22.00I0.194E 01I0.261E 01  I2                                                                                                      I
24 23.00I0.193E 01I0.255E 01  I2                                                                                                      I
24 24.00I0.204E 01I0.266E 01  I2                                                                                                      I
25  1.00I0.220E 01I0.304E 01  I12                                                                                                     I
25  2.00I0.263E 01I0.328E 01  I12                                                                                                     I
25  3.00I0.306E 01I0.357E 01  I 2                                                                                                     I
25  4.00I0.335E 01I0.369E 01  I 2                                                                                                     I
25  5.00I0.357E 01I0.410E 01  I  12                                                                                                   I
25  6.00I0.365E 01I0.427E 01  I  12                                                                                                   I
25  7.00I0.363E 01I0.446E 01  I  12                                                                                                   I
25  8.00I0.353E 01I0.478E 01  I  12                                                                                                   I
25  9.00I0.341E 01I0.478E 01  I  12                                                                                                   I
25 10.00I0.327E 01I0.463E 01  I  12                                                                                                   I
25 11.00I0.313E 01I0.499E 01  I  12                                                                                                   I
25 12.00I0.300E 01I0.445E 01  I1 2                                                                                                    I
                              I-------------------------------------------------------------------------------------------------------I

                                                                              *** PLOT OF FLOW ***
                                                                              ***    (M3/S)    ***
PAGE BEGINS AT 1976, 6,25                                         POINTS ARE REPRESENTED BY  1 FOR SERIES   111 AND  2 FOR SERIES   222
                       .0                 .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
DAY HR.  I   RATE1   I   RATE2    I-------------------------------------------------------------------------------------------------------I
25 13.00I0.288E 01I0.438E 01  I1 2                                                                                                    I
25 14.00I0.278E 01I0.424E 01  I1 2                                                                                                    I
25 15.00I0.269E 01I0.413E 01  I1 2                                                                                                    I
25 16.00I0.262E 01I0.399E 01  I12                                                                                                     I
25 17.00I0.256E 01I0.368E 01  I12                                                                                                     I
25 18.00I0.251E 01I0.357E 01  I12                                                                                                     I
25 19.00I0.247E 01I0.350E 01  I12                                                                                                     I
25 20.00I0.243E 01I0.328E 01  I12                                                                                                     I
25 21.00I0.240E 01I0.325E 01  I12                                                                                                     I
25 22.00I0.236E 01I0.325E 01  I12                                                                                                     I
25 23.00I0.236E 01I0.321E 01  I12                                                                                                     I
25 24.00I0.234E 01I0.321E 01  I12                                                                                                     I
26  1.00I0.233E 01I0.318E 01  I12                                                                                                     I
26  2.00I0.231E 01I0.314E 01  I12                                                                                                     I
26  3.00I0.230E 01I0.314E 01  I12                                                                                                     I
26  4.00I0.228E 01I0.312E 01  I12                                                                                                     I
26  5.00I0.227E 01I0.304E 01  I12                                                                                                     I
26  6.00I0.227E 01I0.297E 01  I2                                                                                                      I
26  7.00I0.226E 01I0.286E 01  I2                                                                                                      I
26  8.00I0.225E 01I0.278E 01  I2                                                                                                      I
26  9.00I0.224E 01I0.271E 01  I2                                                                                                      I
26 10.00I0.224E 01I0.279E 01  I2                                                                                                      I
26 11.00I0.223E 01I0.272E 01  I2                                                                                                      I
26 12.00I0.222E 01I0.272E 01  I2                                                                                                      I
26 13.00I0.221E 01I0.272E 01  I2                                                                                                      I
26 14.00I0.221E 01I0.282E 01  I2                                                                                                      I
26 15.00I0.220E 01I0.304E 01  I12                                                                                                     I
26 16.00I0.219E 01I0.279E 01  I2                                                                                                      I
26 17.00I0.219E 01I0.294E 01  I2                                                                                                      I
26 18.00I0.219E 01I0.260E 01  I2                                                                                                      I
26 19.00I0.256E 01I0.285E 01  I2                                                                                                      I
26 20.00I0.275E 01I0.265E 01  I2                                                                                                      I
26 21.00I0.294E 01I0.265E 01  I2                                                                                                      I
26 22.00I0.307E 01I0.265E 01  I21                                                                                                     I
26 23.00I0.311E 01I0.265E 01  I21                                                                                                     I
26 24.00I0.311E 01I0.272E 01  I21                                                                                                     I
27  1.00I0.308E 01I0.278E 01  I21                                                                                                     I
27  2.00I0.298E 01I0.282E 01  I2                                                                                                      I
27  3.00I0.292E 01I0.290E 01  I2                                                                                                      I
27  4.00I0.284E 01I0.213E 01  I2                                                                                                      I
27  5.00I0.277E 01I0.297E 01  I2                                                                                                      I
27  6.00I0.271E 01I0.293E 01  I2                                                                                                      I
27  7.00I0.265E 01I0.213E 01  I2                                                                                                      I
27  8.00I0.260E 01I0.196E 01  I2                                                                                                      I
27  9.00I0.256E 01I0.180E 01  I2                                                                                                      I
27 10.00I0.252E 01I0.286E 01  I2                                                                                                      I
27 11.00I0.249E 01I0.282E 01  I2                                                                                                      I
27 12.00I0.247E 01I0.279E 01  I2                                                                                                      I
27 13.00I0.244E 01I0.279E 01  I2                                                                                                      I
                              I-------------------------------------------------------------------------------------------------------I
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                                                                          *** PLOT OF FLOW ***
                                                                          ***    (M3/S)    ***
PAGE BEGINS AT 1976, 6,27                                   POINTS ARE REPRESENTED BY  1 FOR SERIES   111 AND  2 FOR SERIES   222
                        .0         .100E 02   .200E 02   .300E 02   .400E 02   .500E 02   .600E 02   .700E 02   .800E 02   .900E 02   .100E 03
DAY HR. I  RATE1   I  RATE2       I-------------------------------------------------------------------------------------------------------------
27 14.00I0.242E 01I0.172E 01  I2
27 15.00I0.241E 01I0.272E 01  I2
27 16.00I0.239E 01I0.261E 01  I2
27 17.00I0.238E 01I0.294E 01  I2
27 18.00I0.237E 01I0.244E 01  I2
27 19.00I0.236E 01I0.233E 01  I2
27 20.00I0.239E 01I0.238E 01  I2
27 21.00I0.234E 01I0.238E 01  I2
27 22.00I0.232E 01I0.222E 01  I2
27 23.00I0.231E 01I0.219E 01  I2
27 24.00I0.231E 01I0.219E 01  I2
28  1.00I0.230E 01I0.222E 01  I2
28  2.00I0.229E 01I0.226E 01  I2
28  3.00I0.226E 01I0.236E 01  I2
28  4.00I0.227E 01I0.233E 01  I2
28  5.00I0.227E 01I0.237E 01  I2
28  6.00I0.226E 01I0.237E 01  I2
28  7.00I0.225E 01I0.246E 01  I2
28  8.00I0.224E 01I0.233E 01  I2
28  9.00I0.224E 01I0.230E 01  I2
28 10.00I0.223E 01I0.222E 01  I2
28 11.00I0.222E 01I0.219E 01  I2
28 12.00I0.222E 01I0.209E 01  I2
28 13.00I0.221E 01I0.208E 01  I2
28 14.00I0.220E 01I0.222E 01  I2
28 15.00I0.221E 01I0.212E 01  I2
28 16.00I0.220E 01I0.212E 01  I2
28 17.00I0.218E 01I0.212E 01  I2
28 18.00I0.217E 01I0.212E 01  I2
28 19.00I0.217E 01I0.219E 01  I2
28 20.00I0.218E 01I0.219E 01  I2
28 21.00I0.225E 01I0.228E 01  I2
28 22.00I0.226E 01I0.240E 01  I2
28 23.00I0.233E 01I0.247E 01  I2
28 24.00I0.242E 01I0.251E 01  I2
29  1.00I0.248E 01I0.254E 01  I2
29  2.00I0.252E 01I0.256E 01  I2
29  3.00I0.255E 01I0.281E 01  I2
29  4.00I0.255E 01I0.288E 01  I2
29  5.00I0.253E 01I0.279E 01  I2
29  6.00I0.250E 01I0.288E 01  I2
29  7.00I0.247E 01I0.308E 01  I12
29  8.00I0.244E 01I0.288E 01  I2
29  9.00I0.241E 01I0.277E 01  I2
29 10.00I0.238E 01I0.261E 01  I2
29 11.00I0.236E 01I0.261E 01  I2
29 12.00I0.233E 01I0.256E 01  I2
29 13.00I0.231E 01I0.294E 01  I2
29 14.00I0.230E 01I0.251E 01  I2
                              I-------------------------------------------------------------------------------------------------------------

                                                                          *** PLOT OF FLOW ***
                                                                          ***    (M3/S)    ***
PAGE BEGINS AT 1976, 6,29                                   POINTS ARE REPRESENTED BY  1 FOR SERIES   111 AND  2 FOR SERIES   222
                        .0         .100E 02   .200E 02   .300E 02   .400E 02   .500E 02   .600E 02   .700E 02   .800E 02   .900E 02   .100E 03
DAY HR. I  RATE1   I  RATE2       I-------------------------------------------------------------------------------------------------------------
29 15.00I0.228E 01I0.247E 01  I2
29 16.00I0.240E 01I0.247E 01  I2
29 17.00I0.241E 01I0.251E 01  I2
29 18.00I0.242E 01I0.251E 01  I2
29 19.00I0.243E 01I0.251E 01  I2
29 20.00I0.242E 01I0.254E 01  I2
29 21.00I0.241E 01I0.254E 01  I2
29 22.00I0.240E 01I0.256E 01  I2
29 23.00I0.238E 01I0.258E 01  I2
29 24.00I0.237E 01I0.254E 01  I2
30  1.00I0.235E 01I0.251E 01  I2
30  2.00I0.233E 01I0.261E 01  I2
30  3.00I0.232E 01I0.261E 01  I2
30  4.00I0.230E 01I0.281E 01  I2
30  5.00I0.229E 01I0.281E 01  I2
30  6.00I0.228E 01I0.261E 01  I2
30  7.00I0.227E 01I0.279E 01  I2
30  8.00I0.287E 01I0.308E 01  I12
30  9.00I0.357E 01I0.318E 01  I 2
30 10.00I0.454E 01I0.408E 01  I   2-
30 11.00I0.595E 01I0.621E 01  I     12
30 12.00I0.691E 01I0.764E 01  I       12
30 13.00I0.747E 01I0.826E 01  I         12
30 14.00I0.811E 01I0.851E 01  I           2
30 15.00I0.832E 01I0.858E 01  I           2
30 16.00I0.820E 01I0.822E 01  I           2
30 17.00I0.784E 01I0.792E 01  I         2
30 18.00I0.731E 01I0.689E 01  I       21
30 19.00I0.675E 01I0.646E 01  I       2
30 20.00I0.619E 01I0.604E 01  I       2
30 21.00I0.567E 01I0.561E 01  I     2
30 22.00I0.520E 01I0.519E 01  I     2
30 23.00I0.480E 01I0.491E 01  I   2
30 24.00I0.446E 01I0.477E 01  I   2
 1  1.00I0.417E 01I0.463E 01  I   2
 1  2.00I0.394E 01I0.448E 01  I  12
 1  3.00I0.375E 01I0.434E 01  I  12
 1  4.00I0.359E 01I0.426E 01  I  12
 1  5.00I0.346E 01I0.417E 01  I  12
 1  6.00I0.336E 01I0.417E 01  I  12
 1  7.00I0.328E 01I0.420E 01  I  12
 1  8.00I0.322E 01I0.420E 01  I  12
 1  9.00I0.316E 01I0.424E 01  I  12
 1 10.00I0.312E 01I0.427E 01  I  12
 1 11.00I0.307E 01I0.441E 01  I  12
 1 12.00I0.303E 01I0.452E 01  I  12
 1 13.00I0.301E 01I0.468E 01  I  12
 1 14.00I0.288E 01I0.477E 01  I1  2
 1 15.00I0.289E 01I0.481E 01  I1  2
                              I-------------------------------------------------------------------------------------------------------------
```

```
PAGE BEGINS AT 1976, 7, 1
                                  .0       .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
DAY HR. |  RATE1   |  RATE2       |-----------------------------------------------------------------------------------------------------
 1 16.00|0.294E 01|0.360E 01      |1  2
 1 17.00|0.292E 01|0.396E 01      |1  2
 1 18.00|0.291E 01|0.648E 01      |1    2
 1 19.00|0.289E 01|0.724E 01      |1     2
 1 20.00|0.288E 01|0.794E 01      |1     2
 1 21.00|0.287E 01|0.871E 01      |1      2
 1 22.00|0.285E 01|0.830E 01      |1      2
 1 23.00|0.284E 01|0.770E 01      |1     2
 1 24.00|0.283E 01|0.717E 01      |1     2
 2  1.00|0.281E 01|0.669E 01      |1    2
 2  2.00|0.280E 01|0.657E 01      |1    2
 2  3.00|0.279E 01|0.632E 01      |1    2
 2  4.00|0.277E 01|0.607E 01      |1    2
 2  5.00|0.276E 01|0.582E 01      |1   2
 2  6.00|0.275E 01|0.556E 01      |1   2
 2  7.00|0.274E 01|0.537E 01      |1   2
 2  8.00|0.272E 01|0.519E 01      |1   2
 2  9.00|0.271E 01|0.501E 01      |1   2
 2 10.00|0.270E 01|0.487E 01      |1  2
 2 11.00|0.269E 01|0.480E 01      |1  2
 2 12.00|0.268E 01|0.473E 01      |1  2
 2 13.00|0.266E 01|0.466E 01      |1  2
 2 14.00|0.265E 01|0.459E 01      |1  2
 2 15.00|0.264E 01|0.452E 01      |1  2
 2 16.00|0.263E 01|0.449E 01      |1  2
 2 17.00|0.262E 01|0.436E 01      |1  2
 2 18.00|0.261E 01|0.424E 01      |1  2
 2 19.00|0.260E 01|0.413E 01      |1  2
 2 20.00|0.259E 01|0.403E 01      |1  2
 2 21.00|0.257E 01|0.392E 01      |12
 2 22.00|0.256E 01|0.381E 01      |12
 2 23.00|0.255E 01|0.374E 01      |12
 2 24.00|0.254E 01|0.371E 01      |12
 3  1.00|0.253E 01|0.364E 01      |12
 3  2.00|0.252E 01|0.360E 01      |12
 3  3.00|0.251E 01|0.393E 01      |12
 3  4.00|0.250E 01|0.390E 01      |12
 3  5.00|0.248E 01|0.346E 01      |12
 3  6.00|0.246E 01|0.343E 01      |12
 3  7.00|0.287E 01|0.371E 01      |12
 3  8.00|0.314E 01|0.408E 01      | 12
 3  9.00|0.348E 01|0.438E 01      | 12
 3 10.00|0.379E 01|0.473E 01      | 12
 3 11.00|0.406E 01|0.508E 01      |  12
 3 12.00|0.426E 01|0.537E 01      |  12
 3 13.00|0.434E 01|0.568E 01      |  12
 3 14.00|0.430E 01|0.621E 01      |  1 2
 3 15.00|0.420E 01|0.743E 01      |  1  2
 3 16.00|0.408E 01|0.763E 01      |  1  2
                                  |-----------------------------------------------------------------------------------------------------

                                                                            *** PLOT OF FLOW ***
                                                                            ***    (M3/S)    ***
PAGE BEGINS AT 1976, 7, 3                                 POINTS ARE REPRESENTED BY  1 FOR SERIES   111 AND  2 FOR SERIES   222
                                  .0       .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
DAY HR. |  RATE1   |  RATE2       |-----------------------------------------------------------------------------------------------------
 3 17.00|0.389E 01|0.682E 01      | 1   2
 3 18.00|0.373E 01|0.681E 01      | 1   2
 3 19.00|0.358E 01|0.674E 01      | 1   2
 3 20.00|0.344E 01|0.660E 01      | 1   2
 3 21.00|0.332E 01|0.643E 01      | 1   2
 3 22.00|0.323E 01|0.625E 01      | 1   2
 3 23.00|0.314E 01|0.607E 01      | 1   2
 3 24.00|0.307E 01|0.590E 01      | 1  2
 4  1.00|0.300E 01|0.572E 01      | 1  2
 4  2.00|0.295E 01|0.554E 01      |1   2
 4  3.00|0.291E 01|0.533E 01      |1   2
 4  4.00|0.287E 01|0.516E 01      |1   2
 4  5.00|0.284E 01|0.501E 01      |1   2
 4  6.00|0.281E 01|0.484E 01      |1  2
 4  7.00|0.279E 01|0.470E 01      |1  2
 4  8.00|0.277E 01|0.463E 01      |1  2
 4  9.00|0.275E 01|0.452E 01      |1  2
 4 10.00|0.273E 01|0.445E 01      |1  2
 4 11.00|0.271E 01|0.438E 01      |1  2
 4 12.00|0.270E 01|0.427E 01      |1  2
 4 13.00|0.268E 01|0.417E 01      |1  2
 4 14.00|0.267E 01|0.408E 01      |1  2
 4 15.00|0.266E 01|0.395E 01      |12
 4 16.00|0.265E 01|0.385E 01      |12
 4 17.00|0.264E 01|0.374E 01      |12
 4 18.00|0.263E 01|0.367E 01      |12
 4 19.00|0.261E 01|0.360E 01      |12
 4 20.00|0.260E 01|0.353E 01      |12
 4 21.00|0.259E 01|0.344E 01      |12
 4 22.00|0.258E 01|0.338E 01      |12
 4 23.00|0.257E 01|0.332E 01      |12
 4 24.00|0.256E 01|0.325E 01      |12
 5  1.00|0.255E 01|0.321E 01      |12
 5  2.00|0.254E 01|0.321E 01      |12
 5  3.00|0.253E 01|0.318E 01      |12
 5  4.00|0.252E 01|0.314E 01      |12
 5  5.00|0.251E 01|0.314E 01      |12
 5  6.00|0.250E 01|0.311E 01      |12
 5  7.00|0.248E 01|0.314E 01      |12
 5  8.00|0.248E 01|0.314E 01      |12
 5  9.00|0.247E 01|0.318E 01      |12
 5 10.00|0.246E 01|0.314E 01      |12
 5 11.00|0.245E 01|0.311E 01      |12
 5 12.00|0.244E 01|0.307E 01      |12
 5 13.00|0.243E 01|0.300E 01      |12
 5 14.00|0.242E 01|0.297E 01      |2
 5 15.00|0.241E 01|0.288E 01      |2
 5 16.00|0.240E 01|0.279E 01      |2
 5 17.00|0.238E 01|0.268E 01      |2
                                  |-----------------------------------------------------------------------------------------------------
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PAGE BEGINS AT 1976, 7, 5                                  POINTS ARE REPRESENTED BY  1 FOR SERIES   111 AND  2 FOR SERIES   222
                       .0             .100E 02   .200E 02   .300E 02   .400E 02   .500E 02   .600E 02   .700E 02   .800E 02   .900E 02   .10E 03
DAY HR.  I  RATE1  I  RATE2      I----+----------+----------+----------+----------+----------+----------+----------+----------+----------+----I
 5 18.00I0.233E 01I0.250E 01  I2
 5 19.00I0.233E 01I0.251E 01  I2
 5 20.00I0.237E 01I0.247E 01  I2
 5 21.00I0.235E 01I0.240E 01  I2
 5 22.00I0.235E 01I0.237E 01  I2
 5 23.00I0.234E 01I0.237E 01  I2
 5 24.00I0.233E 01I0.240E 01  I2
 6  1.00I0.233E 01I0.240E 01  I2
 6  2.00I0.232E 01I0.240E 01  I2
 6  3.00I0.231E 01I0.244E 01  I2
 6  4.00I0.230E 01I0.247E 01  I2
 6  5.00I0.228E 01I0.251E 01  I2
 6  6.00I0.228E 01I0.254E 01  I2
 6  7.00I0.228E 01I0.258E 01  I2
 6  8.00I0.227E 01I0.261E 01  I2
 6  9.00I0.226E 01I0.258E 01  I2
 6 10.00I0.225E 01I0.254E 01  I2
 6 11.00I0.225E 01I0.254E 01  I2
 6 12.00I0.224E 01I0.251E 01  I2
 6 13.00I0.223E 01I0.251E 01  I2
 6 14.00I0.222E 01I0.247E 01  I2
 6 15.00I0.222E 01I0.247E 01  I2
 6 16.00I0.221E 01I0.244E 01  I2
 6 17.00I0.220E 01I0.240E 01  I2
 6 18.00I0.220E 01I0.237E 01  I2
 6 19.00I0.219E 01I0.230E 01  I2
 6 20.00I0.218E 01I0.226E 01  I2
 6 21.00I0.218E 01I0.222E 01  I2
 6 22.00I0.217E 01I0.219E 01  I2
 6 23.00I0.216E 01I0.212E 01  I2
 6 24.00I0.216E 01I0.212E 01  I2
 7  1.00I0.215E 01I0.212E 01  I2
 7  2.00I0.214E 01I0.212E 01  I2
 7  3.00I0.214E 01I0.212E 01  I2
 7  4.00I0.213E 01I0.215E 01  I2
 7  5.00I0.212E 01I0.219E 01  I2
 7  6.00I0.212E 01I0.219E 01  I2
 7  7.00I0.211E 01I0.219E 01  I2
 7  8.00I0.211E 01I0.222E 01  I2
 7  9.00I0.210E 01I0.226E 01  I2
 7 10.00I0.209E 01I0.230E 01  I2
 7 11.00I0.209E 01I0.237E 01  I2
 7 12.00I0.208E 01I0.237E 01  I2
 7 13.00I0.208E 01I0.240E 01  I2
 7 14.00I0.207E 01I0.237E 01  I2
 7 15.00I0.206E 01I0.230E 01  I2
 7 16.00I0.205E 01I0.222E 01  I2
 7 17.00I0.205E 01I0.220E 01  I2
 7 18.00I0.232E 01I0.230E 01  I2
                              I----+----------+----------+----------+----------+----------+----------+----------+----------+----------+----I

                                                                *** PLOT OF FLOW ***
                                                                ***     (M3/S)    ***
PAGE BEGINS AT 1976, 7, 7                                  POINTS ARE REPRESENTED BY  1 FOR SERIES   111 AND  2 FOR SERIES   222
                       .0             .100E 02   .260E 02   .300E 02   .400E 02   .500E 02   .600E 02   .700E 02   .800E 02   .900E 02   .10E 03
DAY HR.  I  RATE1  I  RATE2      I----+----------+----------+----------+----------+----------+----------+----------+----------+----------+----I
 7 19.00I0.242E 01I0.237E 01  I2
 7 20.00I0.253E 01I0.244E 01  I2
 7 21.00I0.260E 01I0.285E 01  I2
 7 22.00I0.263E 01I0.290E 01  I2
 7 23.00I0.263E 01I0.307E 01  I12
 7 24.00I0.344E 01I0.318E 01  I 2
 8  1.00I0.420E 01I0.332E 01  I 21
 8  2.00I0.517E 01I0.381E 01  I 2 1
 8  3.00I0.574E 01I0.594E 01  I   2
 8  4.00I0.588E 01I0.576E 01  I   2
 8  5.00I0.594E 01I0.593E 01  I   2
 8  6.00I0.572E 01I0.547E 01  I   2
 8  7.00I0.541E 01I0.515E 01  I   2
 8  8.00I0.507E 01I0.488E 01  I  21
 8  9.00I0.473E 01I0.477E 01  I  2
 8 10.00I0.442E 01I0.449E 01  I  2
 8 11.00I0.414E 01I0.434E 01  I  2
 8 12.00I0.388E 01I0.413E 01  I 12
 8 13.00I0.369E 01I0.388E 01  I 2
 8 14.00I0.351E 01I0.370E 01  I 2
 8 15.00I0.337E 01I0.364E 01  I 2
 8 16.00I0.325E 01I0.346E 01  I 2
 8 17.00I0.316E 01I0.339E 01  I 2
 8 18.00I0.308E 01I0.320E 01  I 2
 8 19.00I0.301E 01I0.314E 01  I 2
 8 20.00I0.298E 01I0.307E 01  I12
 8 21.00I0.292E 01I0.297E 01  I2
 8 22.00I0.288E 01I0.290E 01  I2
 8 23.00I0.285E 01I0.288E 01  I2
 8 24.00I0.282E 01I0.288E 01  I2
 9  1.00I0.280E 01I0.288E 01  I2
 9  2.00I0.276E 01I0.288E 01  I2
 9  3.00I0.275E 01I0.288E 01  I2
 9  4.00I0.273E 01I0.283E 01  I2
 9  5.00I0.272E 01I0.290E 01  I2
 9  6.00I0.271E 01I0.290E 01  I2
 9  7.00I0.270E 01I0.288E 01  I2
 9  8.00I0.269E 01I0.282E 01  I2
 9  9.00I0.267E 01I0.279E 01  I2
 9 10.00I0.266E 01I0.275E 01  I2
 9 11.00I0.265E 01I0.268E 01  I2
 9 12.00I0.264E 01I0.261E 01  I2
 9 13.00I0.263E 01I0.254E 01  I2
 9 14.00I0.262E 01I0.247E 01  I2
 9 15.00I0.261E 01I0.240E 01  I2
 9 16.00I0.259E 01I0.230E 01  I2
 9 17.00I0.258E 01I0.215E 01  I2
 9 18.00I0.257E 01I0.205E 01  I2
 9 19.00I0.256E 01I0.201E 01  I2
                              I----+----------+----------+----------+----------+----------+----------+----------+----------+----------+----I
```

```
 9 20.00I0.255E 01I0.201E 01
 9 21.00I0.254E 01I0.209E 01
 9 22.00I0.253E 01I0.206E 01
 9 23.00I0.252E 01I0.212E 01
 9 24.00I0.251E 01I0.208E 01
10  1.00I0.250E 01I0.208E 01
10  2.00I0.249E 01I0.206E 01
10  3.00I0.248E 01I0.209E 01
10  4.00I0.247E 01I0.208E 01
10  5.00I0.246E 01I0.212E 01
10  6.00I0.245E 01I0.215E 01
10  7.00I0.244E 01I0.219E 01
10  8.00I0.251E 01I0.222E 01
10  9.00I0.253E 01I0.226E 01
10 10.00I0.253E 01I0.230E 01
10 11.00I0.253E 01I0.230E 01
10 12.00I0.253E 01I0.230E 01
10 13.00I0.252E 01I0.228E 01
10 14.00I0.251E 01I0.226E 01
10 15.00I0.250E 01I0.228E 01
10 16.00I0.248E 01I0.228E 01
10 17.00I0.247E 01I0.226E 01
10 18.00I0.246E 01I0.228E 01
10 19.00I0.244E 01I0.226E 01
10 20.00I0.243E 01I0.230E 01
10 21.00I0.242E 01I0.233E 01
10 22.00I0.241E 01I0.247E 01
10 23.00I0.240E 01I0.287E 01
10 24.00I0.238E 01I0.403E 01
11  1.00I0.393E 01I0.591E 01
11  2.00I0.721E 01I0.112E 02
11  3.00I0.118E 02I0.140E 02
11  4.00I0.148E 02I0.140E 02
11  5.00I0.168E 02I0.117E 02
11  6.00I0.170E 02I0.109E 02
11  7.00I0.164E 02I0.960E 01
11  8.00I0.152E 02I0.869E 01
11  9.00I0.136E 02I0.780E 01
11 10.00I0.123E 02I0.710E 01
11 11.00I0.109E 02I0.646E 01
11 12.00I0.962E 01I0.607E 01
11 13.00I0.850E 01I0.565E 01
11 14.00I0.794E 01I0.583E 01
11 15.00I0.674E 01I0.632E 01
11 16.00I0.658E 01I0.638E 01
11 17.00I0.640E 01I0.605E 01
11 18.00I0.634E 01I0.698E 01
11 19.00I0.628E 01I0.607E 01
11 20.00I0.613E 01I0.614E 01
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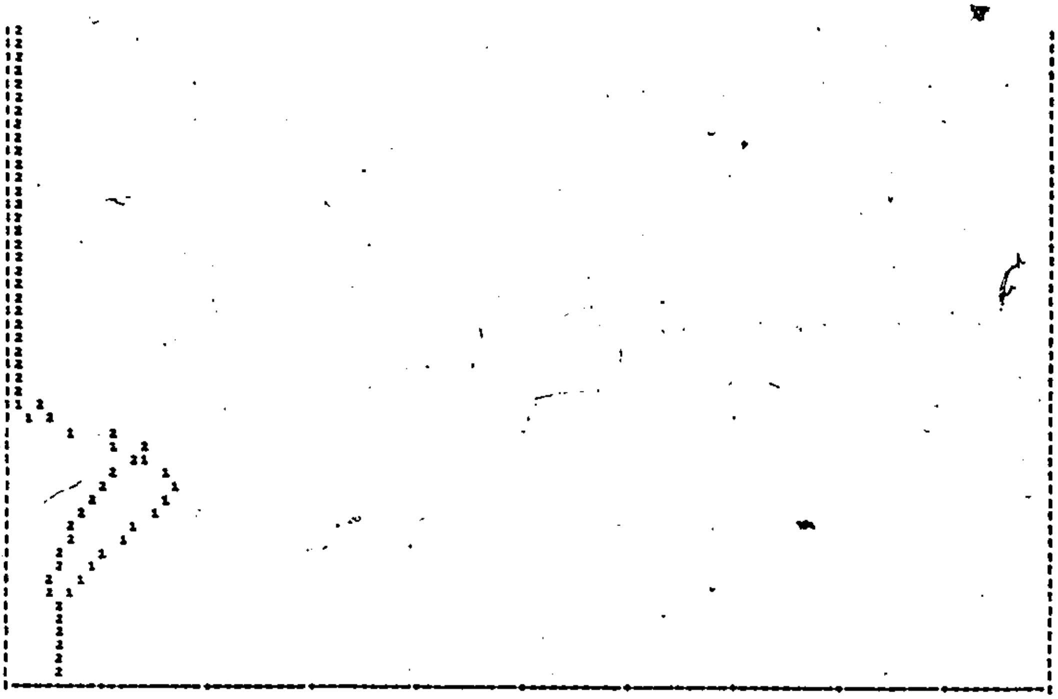

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                      .0
DAY HR. I  RATE1   I  RATE2
11 21.00I0.592E 01I0.583E 01
11 22.00I0.560E 01I0.547E 01
11 23.00I0.541E 01I0.511E 01
11 24.00I0.516E 01I0.481E 01
12  1.00I0.494E 01I0.470E 01
12  2.00I0.475E 01I0.458E 01
12  3.00I0.458E 01I0.448E 01
12  4.00I0.438E 01I0.434E 01
12  5.00I0.426E 01I0.424E 01
12  6.00I0.416E 01I0.413E 01
12  7.00I0.408E 01I0.408E 01
12  8.00I0.402E 01I0.399E 01
12  9.00I0.396E 01I0.392E 01
12 10.00I0.391E 01I0.385E 01
12 11.00I0.387E 01I0.378E 01
12 12.00I0.385E 01I0.367E 01
12 13.00I0.386E 01I0.360E 01
12 14.00I0.384E 01I0.353E 01
12 15.00I0.381E 01I0.346E 01
12 16.00I0.379E 01I0.339E 01
12 17.00I0.376E 01I0.335E 01
12 18.00I0.373E 01I0.328E 01
12 19.00I0.370E 01I0.325E 01
12 20.00I0.368E 01I0.321E 01
12 21.00I0.365E 01I0.318E 01
12 22.00I0.363E 01I0.314E 01
12 23.00I0.361E 01I0.311E 01
12 24.00I0.358E 01I0.312E 01
13  1.00I0.356E 01I0.307E 01
13  2.00I0.354E 01I0.307E 01
13  3.00I0.352E 01I0.307E 01
13  4.00I0.350E 01I0.307E 01
13  5.00I0.348E 01I0.307E 01
13  6.00I0.346E 01I0.304E 01
13  7.00I0.344E 01I0.304E 01
13  8.00I0.342E 01I0.304E 01
13  9.00I0.340E 01I0.300E 01
13 10.00I0.338E 01I0.297E 01
13 11.00I0.336E 01I0.293E 01
13 12.00I0.334E 01I0.290E 01
13 13.00I0.332E 01I0.286E 01
13 14.00I0.331E 01I0.282E 01
13 15.00I0.329E 01I0.279E 01
13 16.00I0.327E 01I0.275E 01
13 17.00I0.325E 01I0.272E 01
13 18.00I0.324E 01I0.268E 01
13 19.00I0.322E 01I0.258E 01
13 20.00I0.320E 01I0.244E 01
13 21.00I0.318E 01I0.240E 01
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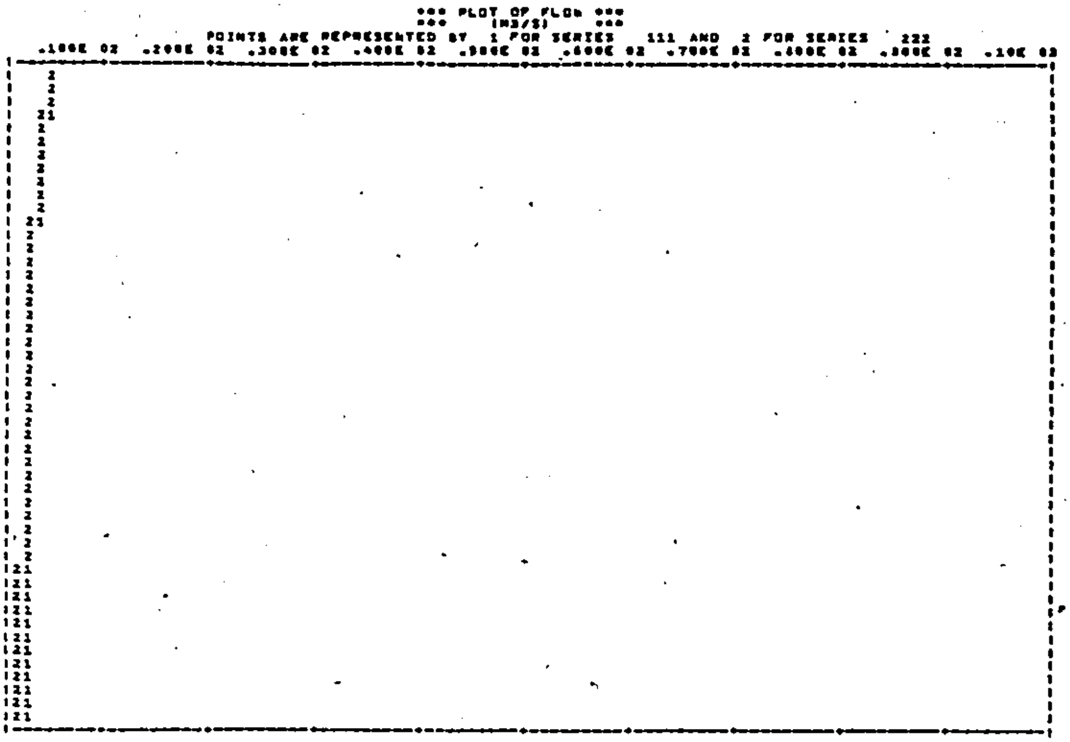

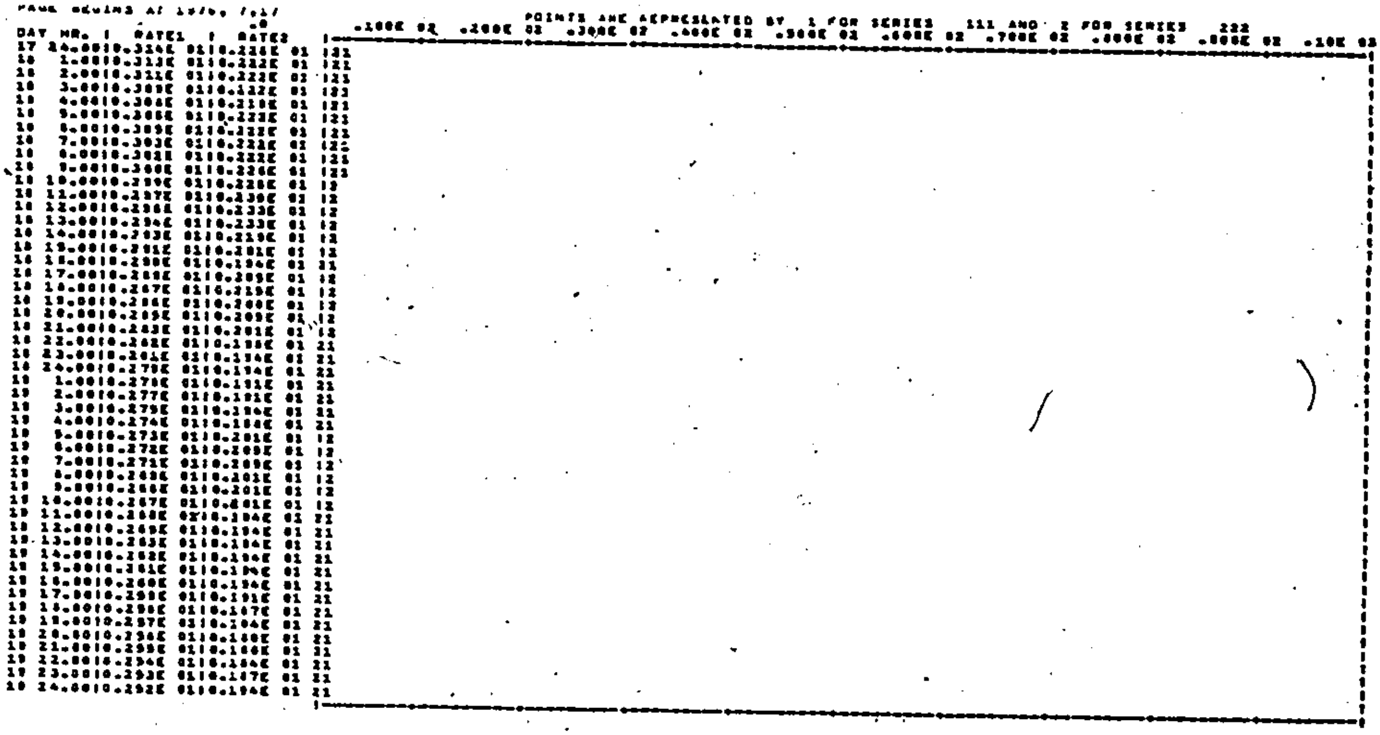

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PAGE BEGINS AT 1976, 7,17
                              .0
DAY HR. I  RATE1  I  RATE2      .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
                                POINTS ARE REPRESENTED BY  1 FOR SERIES  111 AND  2 FOR SERIES  222
17 24.00I0.321E 01I0.222E 01 I21
18  1.00I0.311E 01I0.222E 01 I21
18  2.00I0.311E 01I0.222E 01 I21
18  3.00I0.309E 01I0.222E 01 I21
18  4.00I0.304E 01I0.219E 01 I21
18  5.00I0.306E 01I0.222E 01 I21
18  6.00I0.305E 01I0.222E 01 I21
18  7.00I0.303E 01I0.222E 01 I21
18  8.00I0.301E 01I0.222E 01 I21
18  9.00I0.300E 01I0.226E 01 I21
18 10.00I0.299E 01I0.228E 01 12
18 11.00I0.297E 01I0.230E 01 12
18 12.00I0.296E 01I0.233E 01 12
18 13.00I0.294E 01I0.233E 01 12
18 14.00I0.293E 01I0.219E 01 12
18 15.00I0.291E 01I0.201E 01 12
18 16.00I0.290E 01I0.194E 01 21
18 17.00I0.288E 01I0.205E 01 12
18 18.00I0.287E 01I0.219E 01 12
18 19.00I0.286E 01I0.200E 01 12
18 20.00I0.285E 01I0.205E 01 12
18 21.00I0.283E 01I0.201E 01 12
18 22.00I0.282E 01I0.198E 01 21
18 23.00I0.281E 01I0.194E 01 21
18 24.00I0.279E 01I0.194E 01 21
19  1.00I0.278E 01I0.191E 01 21
19  2.00I0.277E 01I0.191E 01 21
19  3.00I0.275E 01I0.194E 01 21
19  4.00I0.274E 01I0.188E 01 21
19  5.00I0.273E 01I0.201E 01 12
19  6.00I0.272E 01I0.205E 01 12
19  7.00I0.271E 01I0.209E 01 12
19  8.00I0.268E 01I0.201E 01 12
19  9.00I0.266E 01I0.201E 01 12
19 10.00I0.267E 01I0.201E 01 12
19 11.00I0.266E 01I0.194E 01 21
19 12.00I0.265E 01I0.194E 01 21
19 13.00I0.263E 01I0.194E 01 21
19 14.00I0.262E 01I0.194E 01 21
19 15.00I0.261E 01I0.194E 01 21
19 16.00I0.260E 01I0.194E 01 21
19 17.00I0.259E 01I0.191E 01 21
19 18.00I0.258E 01I0.187E 01 21
19 19.00I0.257E 01I0.184E 01 21
19 20.00I0.256E 01I0.180E 01 21
19 21.00I0.255E 01I0.180E 01 21
19 22.00I0.254E 01I0.184E 01 21
19 23.00I0.253E 01I0.187E 01 21
19 24.00I0.252E 01I0.194E 01 21
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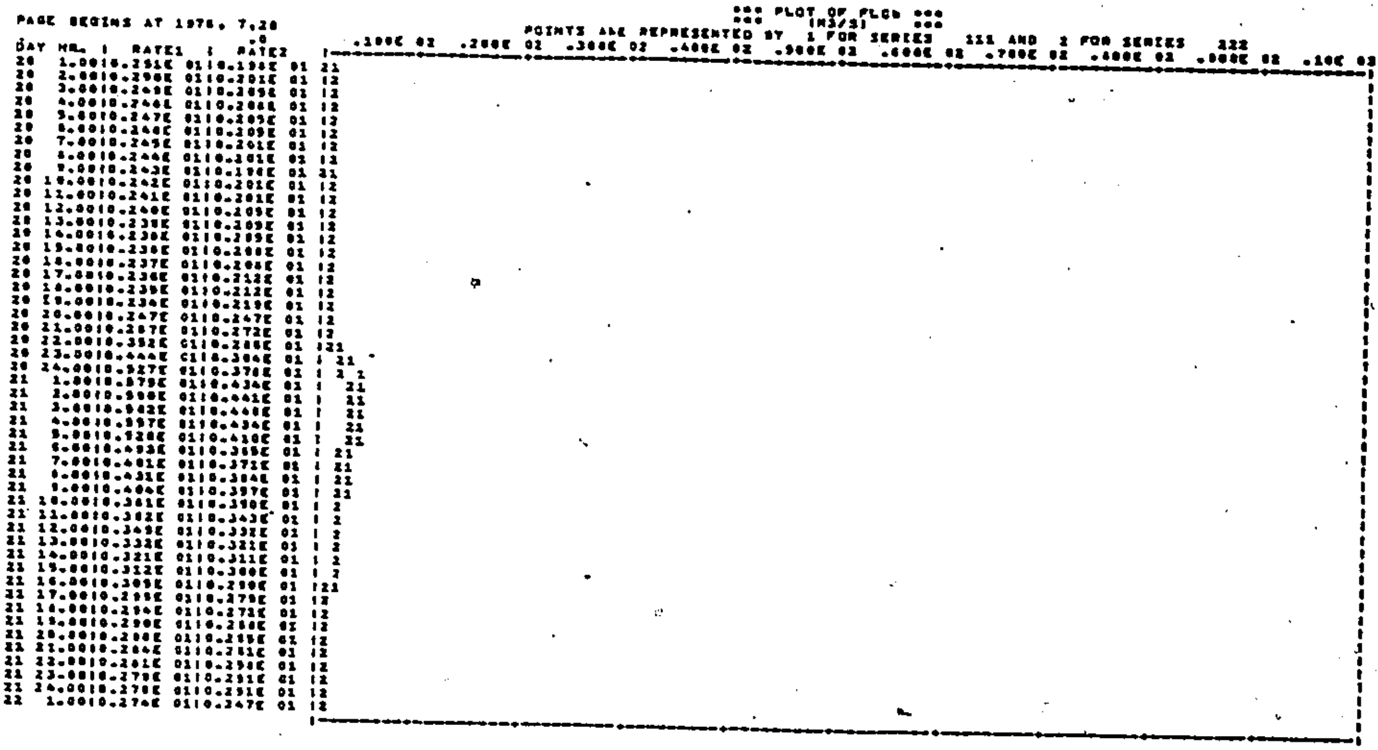

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                                                        *** PLOT OF FLOW ***
                                                        ***    (M3/S)    ***
PAGE BEGINS AT 1976, 7,20       POINTS ARE REPRESENTED BY  1 FOR SERIES  111 AND  2 FOR SERIES  222
                              .0
DAY HR. I  RATE1  I  RATE2      .100E 02  .200E 02  .300E 02  .400E 02  .500E 02  .600E 02  .700E 02  .800E 02  .900E 02  .10E 03
20  1.00I0.251E 01I0.198E 01 21
20  2.00I0.250E 01I0.201E 01 12
20  3.00I0.249E 01I0.205E 01 12
20  4.00I0.248E 01I0.208E 01 12
20  5.00I0.247E 01I0.205E 01 12
20  6.00I0.246E 01I0.209E 01 12
20  7.00I0.245E 01I0.201E 01 12
20  8.00I0.244E 01I0.201E 01 12
20  9.00I0.243E 01I0.198E 01 21
20 10.00I0.242E 01I0.201E 01 12
20 11.00I0.241E 01I0.201E 01 12
20 12.00I0.240E 01I0.205E 01 12
20 13.00I0.239E 01I0.209E 01 12
20 14.00I0.238E 01I0.209E 01 12
20 15.00I0.238E 01I0.208E 01 12
20 16.00I0.237E 01I0.208E 01 12
20 17.00I0.236E 01I0.212E 01 12
20 18.00I0.235E 01I0.212E 01 12
20 19.00I0.234E 01I0.219E 01 12
20 20.00I0.247E 01I0.247E 01 12
20 21.00I0.287E 01I0.272E 01 12
20 22.00I0.352E 01I0.285E 01 I21
20 23.00I0.444E 01I0.304E 01 I  21
20 24.00I0.527E 01I0.378E 01 I  2 1
21  1.00I0.579E 01I0.434E 01 I    21
21  2.00I0.590E 01I0.441E 01 I    21
21  3.00I0.582E 01I0.446E 01 I    21
21  4.00I0.557E 01I0.434E 01 I    21
21  5.00I0.520E 01I0.410E 01 I    21
21  6.00I0.493E 01I0.385E 01 I  21
21  7.00I0.461E 01I0.372E 01 I  21
21  8.00I0.431E 01I0.384E 01 I  21
21  9.00I0.406E 01I0.397E 01 I  21
21 10.00I0.381E 01I0.390E 01 I  2
21 11.00I0.362E 01I0.343E 01 I  2
21 12.00I0.349E 01I0.337E 01 I  2
21 13.00I0.332E 01I0.321E 01 I  2
21 14.00I0.321E 01I0.311E 01 I  2
21 15.00I0.312E 01I0.300E 01 I  2
21 16.00I0.309E 01I0.290E 01 I21
21 17.00I0.299E 01I0.279E 01 12
21 18.00I0.294E 01I0.271E 01 12
21 19.00I0.290E 01I0.260E 01 12
21 20.00I0.288E 01I0.255E 01 12
21 21.00I0.284E 01I0.251E 01 12
21 22.00I0.281E 01I0.250E 01 12
21 23.00I0.279E 01I0.251E 01 12
21 24.00I0.278E 01I0.251E 01 12
22  1.00I0.274E 01I0.247E 01 12
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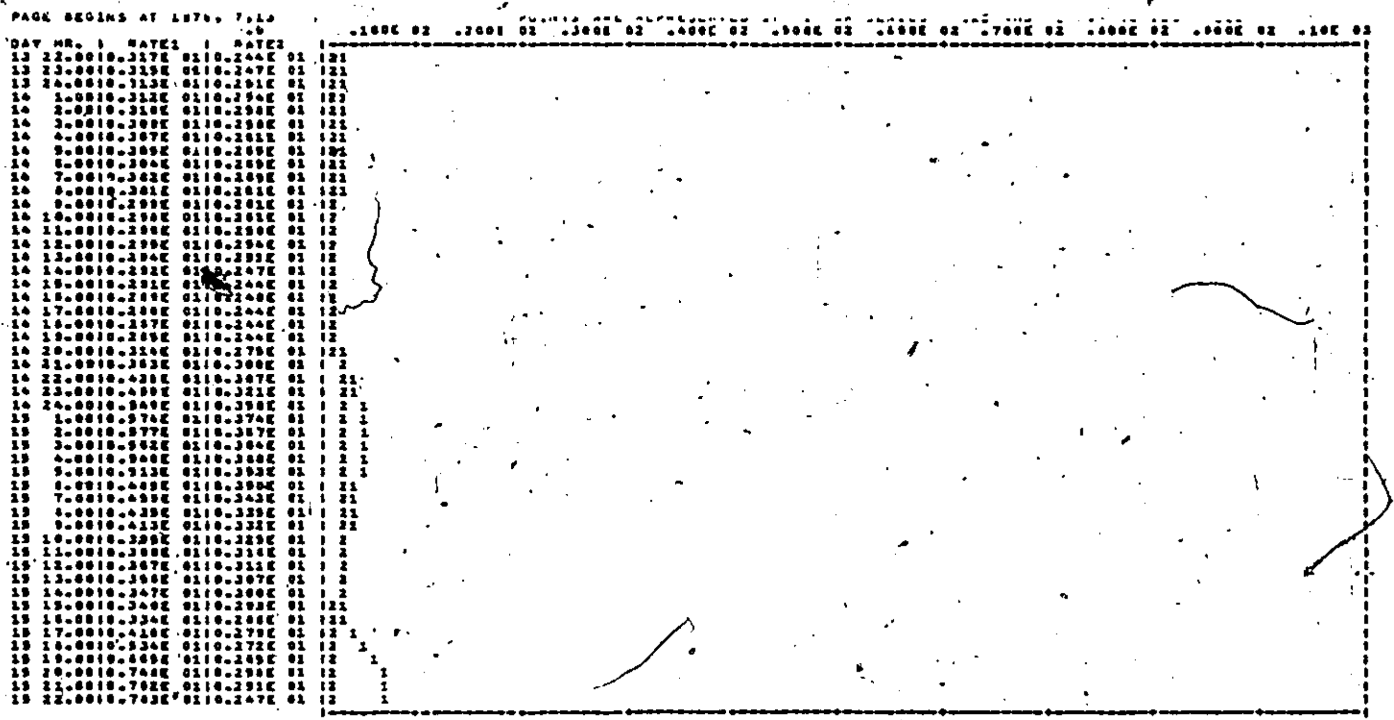
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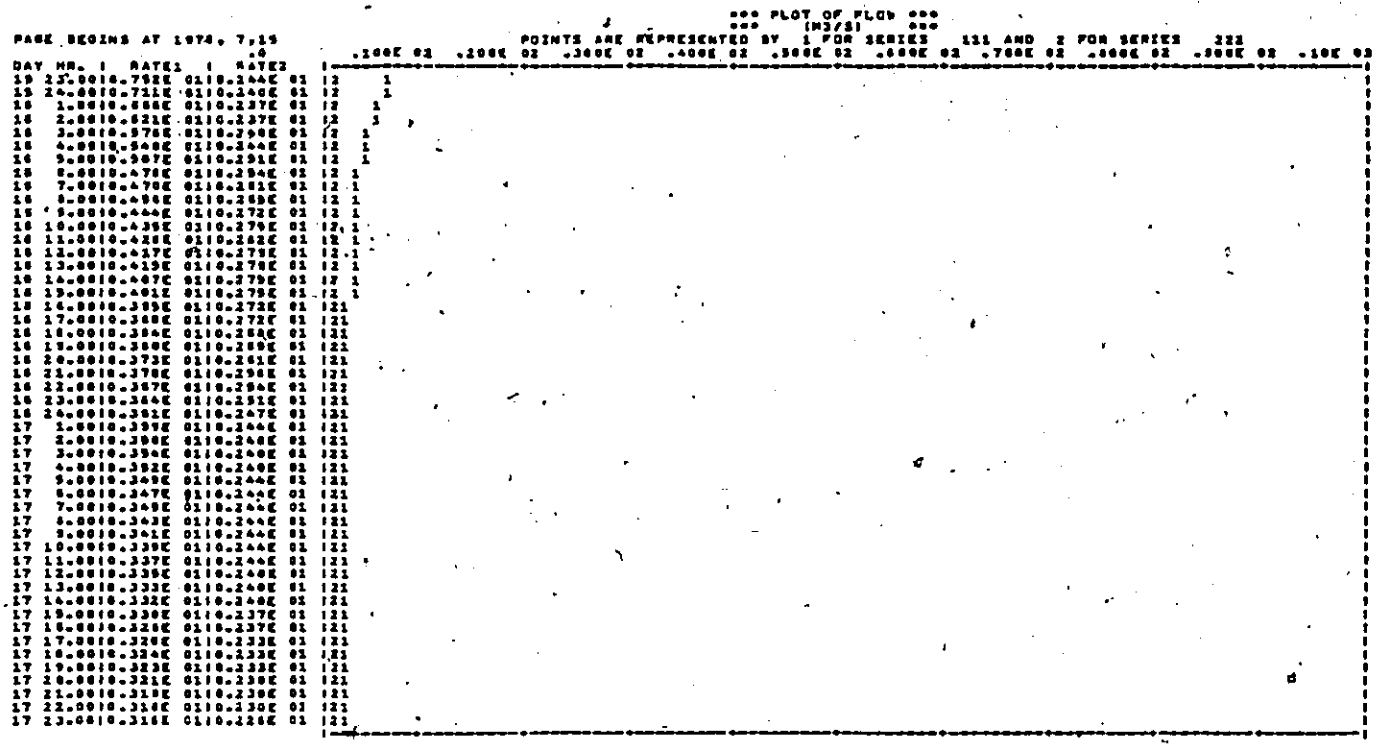
.300E 02
.400E 02
.500E 02
.600E 02
.700E 02
.800E 02
.900E 02
.10E 03


*** PLOT OF FLOW ***
*** (M3/S) ***
POINTS ARE REPRESENTED BY 1 FOR SERIES 111 AND 2 FOR SERIES 222
PAGE BEGINS AT 1974, 7,15
.0
DAY HR. I RATE1 I RATE2
.100E 02
.200E 02
.300E 02
.400E 02
.500E 02
.600E 02
.700E 02
.800E 02
.900E 02
.10E 03

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22   2.00I0.272E 01I0.247E 01
22   3.00I0.271E 01I0.247E 01
22   4.00I0.270E 01I0.244E 01
22   5.00I0.269E 01I0.244E 01
22   6.00I0.266E 01I0.244E 01
22   7.00I0.266E 01I0.240E 01
22   8.00I0.265E 01I0.237E 01
22   9.00I0.264E 01I0.236E 01
22  10.00I0.263E 01I0.228E 01
22  11.00I0.262E 01I0.219E 01
22  12.00I0.261E 01I0.212E 01
22  13.00I0.260E 01I0.206E 01
22  14.00I0.259E 01I0.205E 01
22  15.00I0.257E 01I0.201E 01
22  16.00I0.256E 01I0.198E 01
22  17.00I0.255E 01I0.191E 01
22  18.00I0.254E 01I0.187E 01
22  19.00I0.253E 01I0.145E 01
22  20.00I0.252E 01I0.149E 01
22  21.00I0.251E 01I0.152E 01
22  22.00I0.250E 01I0.162E 01
22  23.00I0.249E 01I0.169E 01
22  24.00I0.248E 01I0.173E 01
23   1.00I0.247E 01I0.177E 01
23   2.00I0.246E 01I0.180E 01
23   3.00I0.245E 01I0.187E 01
23   4.00I0.244E 01I0.191E 01
23   5.00I0.243E 01I0.194E 01
23   6.00I0.243E 01I0.201E 01
23   7.00I0.242E 01I0.205E 01
23   8.00I0.241E 01I0.208E 01
23   9.00I0.240E 01I0.209E 01
23  10.00I0.239E 01I0.202E 01
23  11.00I0.238E 01I0.188E 01
23  12.00I0.237E 01I0.198E 01
23  13.00I0.236E 01I0.190E 01
23  14.00I0.235E 01I0.190E 01
23  15.00I0.235E 01I0.188E 01
23  16.00I0.234E 01I0.194E 01
23  17.00I0.233E 01I0.181E 01
23  18.00I0.232E 01I0.187E 01
23  19.00I0.231E 01I0.191E 01
23  20.00I0.366E 01I0.492E 01
23  21.00I0.801E 01I0.872E 01
23  22.00I0.157E 02I0.201E 02
23  23.00I0.246E 02I0.243E 02
23  24.00I0.316E 02I0.275E 02
24   1.00I0.311E 02I0.208E 02
24   2.00I0.366E 02I0.213E 02
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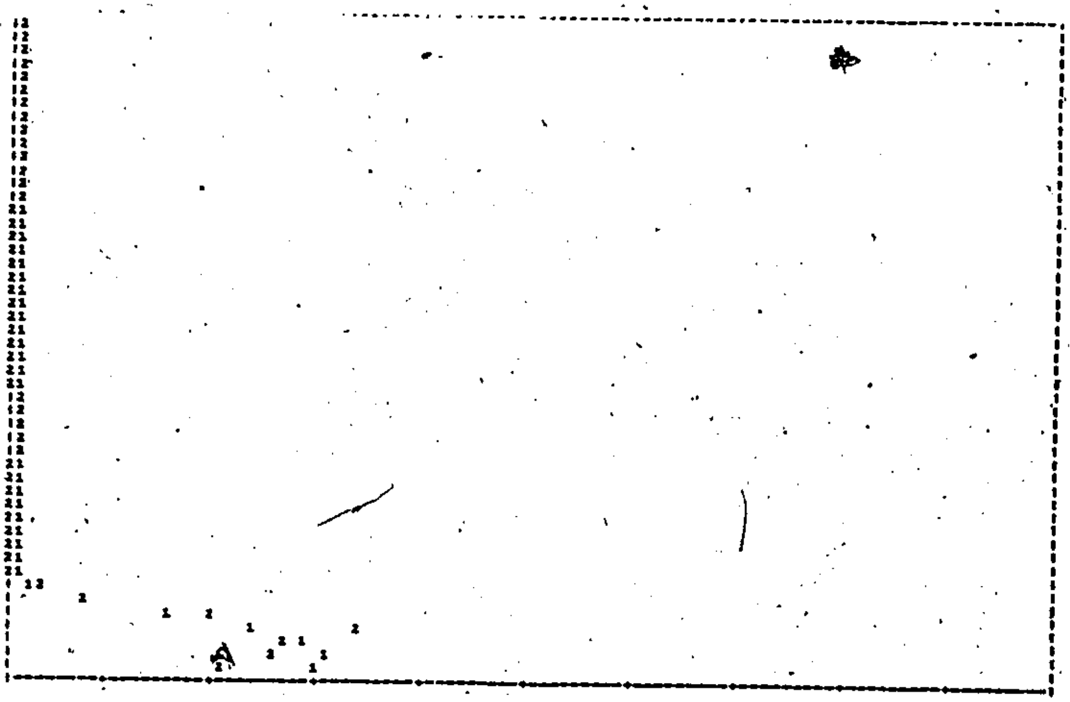

PAGE BEGINS AT 1976, 7,24
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24   4.00I0.234E 02I0.135E 02
24   5.00I0.220E 02I0.111E 02
24   6.00I0.198E 02I0.943E 01
24   7.00I0.171E 02I0.896E 01
24   8.00I0.147E 02I0.781E 01
24   9.00I0.127E 02I0.734E 01
24  10.00I0.109E 02I0.642E 01
24  11.00I0.949E 01I0.638E 01
24  12.00I0.832E 01I0.597E 01
24  13.00I0.739E 01I0.556E 01
24  14.00I0.664E 01I0.519E 01
24  15.00I0.609E 01I0.491E 01
24  16.00I0.556E 01I0.478E 01
24  17.00I0.521E 01I0.449E 01
24  18.00I0.482E 01I0.424E 01
24  19.00I0.470E 01I0.399E 01
24  20.00I0.452E 01I0.388E 01
24  21.00I0.438E 01I0.376E 01
24  22.00I0.428E 01I0.367E 01
24  23.00I0.422E 01I0.353E 01
24  24.00I0.398E 01I0.346E 01
25   1.00I0.393E 01I0.339E 01
25   2.00I0.390E 01I0.328E 01
25   3.00I0.388E 01I0.321E 01
25   4.00I0.386E 01I0.316E 01
25   5.00I0.383E 01I0.307E 01
25   6.00I0.381E 01I0.307E 01
25   7.00I0.379E 01I0.304E 01
25   8.00I0.376E 01I0.300E 01
25   9.00I0.374E 01I0.297E 01
25  10.00I0.372E 01I0.293E 01
25  11.00I0.370E 01I0.293E 01
25  12.00I0.368E 01I0.287E 01
25  13.00I0.365E 01I0.304E 01
25  14.00I0.363E 01I0.304E 01
25  15.00I0.361E 01I0.302E 01
25  16.00I0.359E 01I0.269E 01
25  17.00I0.357E 01I0.244E 01
25  18.00I0.355E 01I0.233E 01
25  19.00I0.353E 01I0.229E 01
25  20.00I0.351E 01I0.222E 01
25  21.00I0.349E 01I0.219E 01
25  22.00I0.347E 01I0.215E 01
25  23.00I0.345E 01I0.219E 01
25  24.00I0.343E 01I0.219E 01
26   1.00I0.341E 01I0.215E 01
26   2.00I0.339E 01I0.222E 01
26   3.00I0.337E 01I0.228E 01
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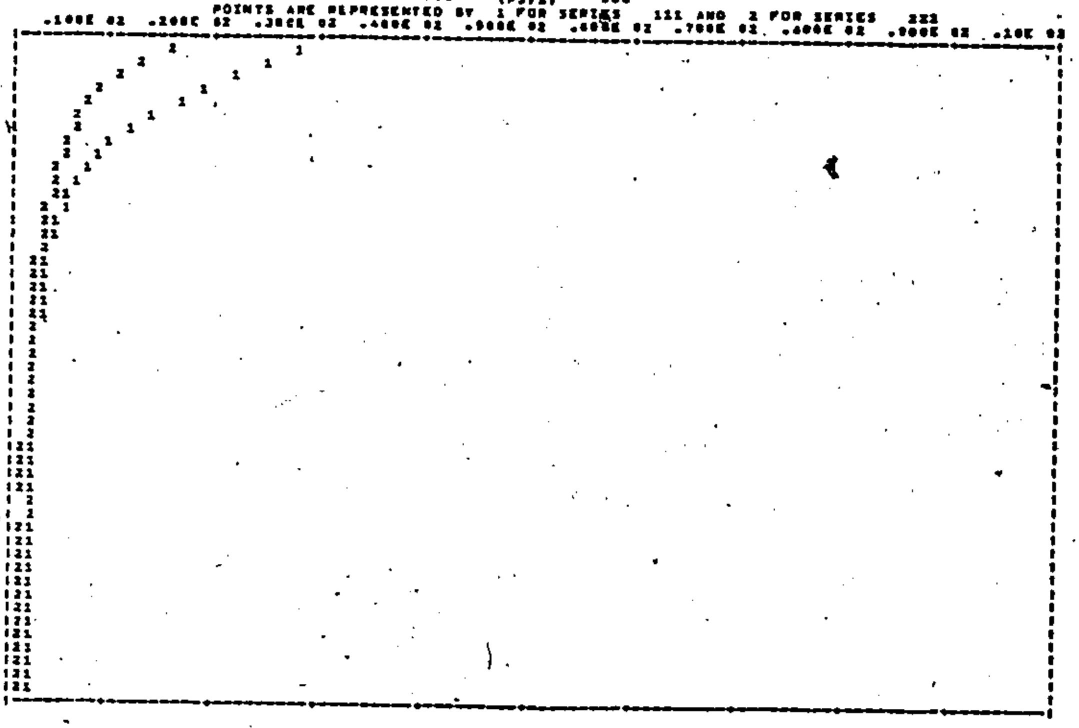
*** PLOT OF FLOW ***
*** (M3/S) ***
POINTS ARE REPRESENTED BY 1 FOR SERIES 111 AND 2 FOR SERIES 222



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DAY HR. |  RATE1   |  RATE2
26   4.00|0.339E 01|0.230E 01
26   5.00|0.334E 01|0.230E 01
26   6.00|0.332E 01|0.230E 01
26   7.00|0.330E 01|0.233E 01
26   8.00|0.328E 01|0.233E 01
26   9.00|0.326E 01|0.233E 01
26  10.00|0.325E 01|0.230E 01
26  11.00|0.323E 01|0.230E 01
26  12.00|0.321E 01|0.230E 01
26  13.00|0.319E 01|0.226E 01
26  14.00|0.318E 01|0.226E 01
26  15.00|0.316E 01|0.226E 01
26  16.00|0.314E 01|0.222E 01
26  17.00|0.313E 01|0.222E 01
26  18.00|0.311E 01|0.219E 01
26  19.00|0.310E 01|0.206E 01
26  20.00|0.308E 01|0.201E 01
26  21.00|0.306E 01|0.194E 01
26  22.00|0.305E 01|0.191E 01
26  23.00|0.303E 01|0.181E 01
26  24.00|0.302E 01|0.191E 01
27   1.00|0.300E 01|0.191E 01
27   2.00|0.299E 01|0.191E 01
27   3.00|0.297E 01|0.190E 01
27   4.00|0.296E 01|0.204E 01
27   5.00|0.294E 01|0.210E 01
27   6.00|0.293E 01|0.230E 01
27   7.00|0.292E 01|0.237E 01
27   8.00|0.290E 01|0.237E 01
27   9.00|0.289E 01|0.240E 01
27  10.00|0.287E 01|0.240E 01
27  11.00|0.286E 01|0.240E 01
27  12.00|0.285E 01|0.233E 01
27  13.00|0.283E 01|0.226E 01
27  14.00|0.282E 01|0.222E 01
27  15.00|0.281E 01|0.215E 01
27  16.00|0.279E 01|0.208E 01
27  17.00|0.278E 01|0.201E 01
27  18.00|0.277E 01|0.205E 01
27  19.00|0.276E 01|0.212E 01
27  20.00|0.274E 01|0.219E 01
27  21.00|0.273E 01|0.218E 01
27  22.00|0.272E 01|0.208E 01
27  23.00|0.271E 01|0.191E 01
27  24.00|0.269E 01|0.190E 01
28   1.00|0.268E 01|0.201E 01
28   2.00|0.267E 01|0.212E 01
28   3.00|0.266E 01|0.208E 01
28   4.00|0.265E 01|0.209E 01
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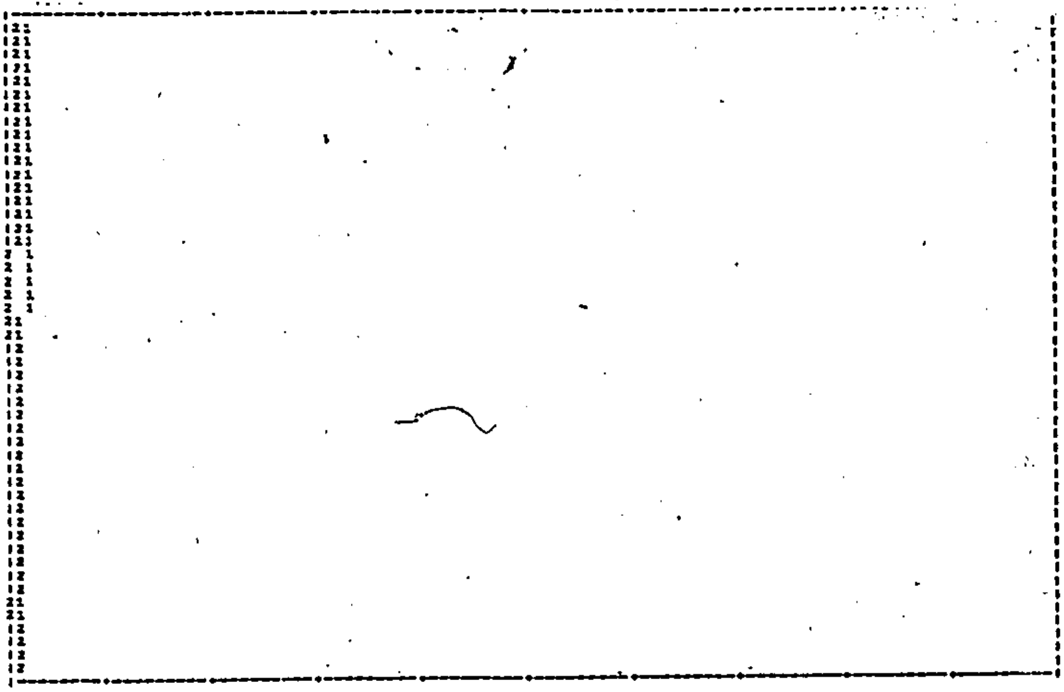

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PAGE BEGINS AT 1976, 7.28
                          .0
DAY HR. |  RATE1   |  RATE2
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28   6.00|0.262E 01|0.196E 01
28   7.00|0.261E 01|0.194E 01
28   8.00|0.260E 01|0.191E 01
28   9.00|0.259E 01|0.207E 01
28  10.00|0.258E 01|0.201E 01
28  11.00|0.257E 01|0.198E 01
28  12.00|0.256E 01|0.201E 01
28  13.00|0.255E 01|0.208E 01
28  14.00|0.254E 01|0.209E 01
28  15.00|0.253E 01|0.209E 01
28  16.00|0.252E 01|0.201E 01
28  17.00|0.251E 01|0.198E 01
28  18.00|0.250E 01|0.188E 01
28  19.00|0.249E 01|0.180E 01
28  20.00|0.248E 01|0.180E 01
28  21.00|0.247E 01|0.188E 01
28  22.00|0.246E 01|0.186E 01
28  23.00|0.245E 01|0.190E 01
28  24.00|0.244E 01|0.194E 01
29   1.00|0.243E 01|0.194E 01
29   2.00|0.242E 01|0.198E 01
29   3.00|0.256E 01|0.201E 01
29   4.00|0.273E 01|0.208E 01
29   5.00|0.287E 01|0.219E 01
29   6.00|0.310E 01|0.230E 01
29   7.00|0.346E 01|0.246E 01
29   8.00|0.427E 01|0.282E 01
29   9.00|0.591E 01|0.339E 01
29  10.00|0.786E 01|0.431E 01
29  11.00|0.962E 01|0.523E 01
29  12.00|0.111E 02|0.632E 01
29  13.00|0.122E 02|0.670E 01
29  14.00|0.137E 02|0.688E 01
29  15.00|0.212E 02|0.158E 02
29  16.00|0.321E 02|0.378E 02
29  17.00|0.402E 02|0.611E 02
29  18.00|0.439E 02|0.703E 02
29  19.00|0.441E 02|0.558E 02
29  20.00|0.417E 02|0.370E 02
29  21.00|0.371E 02|0.238E 02
29  22.00|0.335E 02|0.164E 02
29  23.00|0.291E 02|0.150E 02
29  24.00|0.250E 02|0.132E 02
30   1.00|0.213E 02|0.118E 02
30   2.00|0.181E 02|0.107E 02
30   3.00|0.154E 02|0.969E 01
30   4.00|0.131E 02|0.904E 01
30   5.00|0.113E 02|0.820E 01
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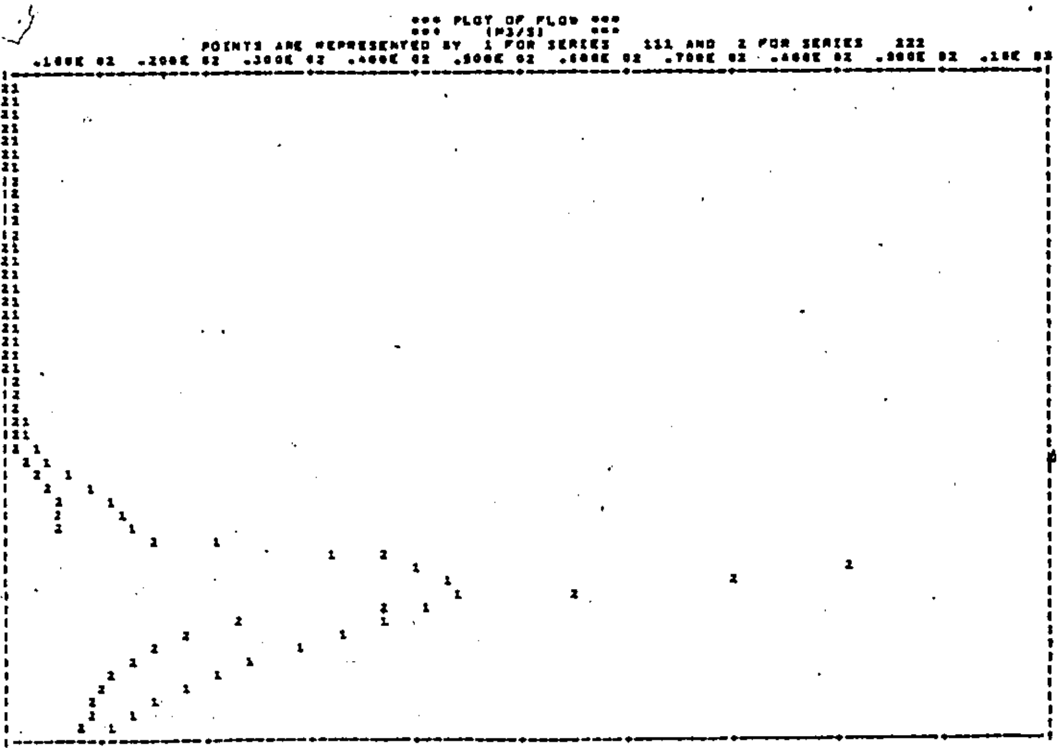

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PAGE BEGINS AT 1978, 7,31
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DAY HR. |  RATE1  |  RATE2
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30   7.00|0.896E 01|0.717E 01
30   8.00|0.746E 01|0.659E 01
30   9.00|0.658E 01|0.653E 01
30  10.00|0.631E 01|0.629E 01
30  11.00|0.543E 01|0.593E 01
30  12.00|0.547E 01|0.561E 01
30  13.00|0.528E 01|0.530E 01
30  14.00|0.497E 01|0.501E 01
30  15.00|0.461E 01|0.460E 01
30  16.00|0.467E 01|0.463E 01
30  17.00|0.443E 01|0.445E 01
30  18.00|0.432E 01|0.427E 01
30  19.00|0.428E 01|0.413E 01
30  20.00|0.426E 01|0.399E 01
30  21.00|0.426E 01|0.389E 01
30  22.00|0.441E 01|0.374E 01
30  23.00|0.418E 01|0.364E 01
30  24.00|0.416E 01|0.393E 01
31   1.00|0.413E 01|0.339E 01
31   2.00|0.410E 01|0.328E 01
31   3.00|0.408E 01|0.328E 01
31   4.00|0.405E 01|0.334E 01
31   5.00|0.403E 01|0.329E 01
31   6.00|0.400E 01|0.328E 01
31   7.00|0.396E 01|0.329E 01
31   8.00|0.395E 01|0.332E 01
31   9.00|0.400E 01|0.339E 01
31  10.00|0.424E 01|0.368E 01
31  11.00|0.458E 01|0.508E 01
31  12.00|0.514E 01|0.649E 01
31  13.00|0.569E 01|0.904E 01
31  14.00|0.675E 01|0.106E 02
31  15.00|0.773E 01|0.108E 02
31  16.00|0.852E 01|0.109E 02
31  17.00|0.896E 01|0.109E 02
31  18.00|0.921E 01|0.108E 02
31  19.00|0.943E 01|0.107E 02
31  20.00|0.924E 01|0.106E 02
31  21.00|0.845E 01|0.925E 01
31  22.00|0.830E 01|0.859E 01
31  23.00|0.760E 01|0.802E 01
31  24.00|0.735E 01|0.750E 01
 1   1.00|0.675E 01|0.727E 01
 1   2.00|0.630E 01|0.674E 01
 1   3.00|0.591E 01|0.640E 01
 1   4.00|0.557E 01|0.621E 01
 1   5.00|0.525E 01|0.587E 01
 1   6.00|0.505E 01|0.568E 01
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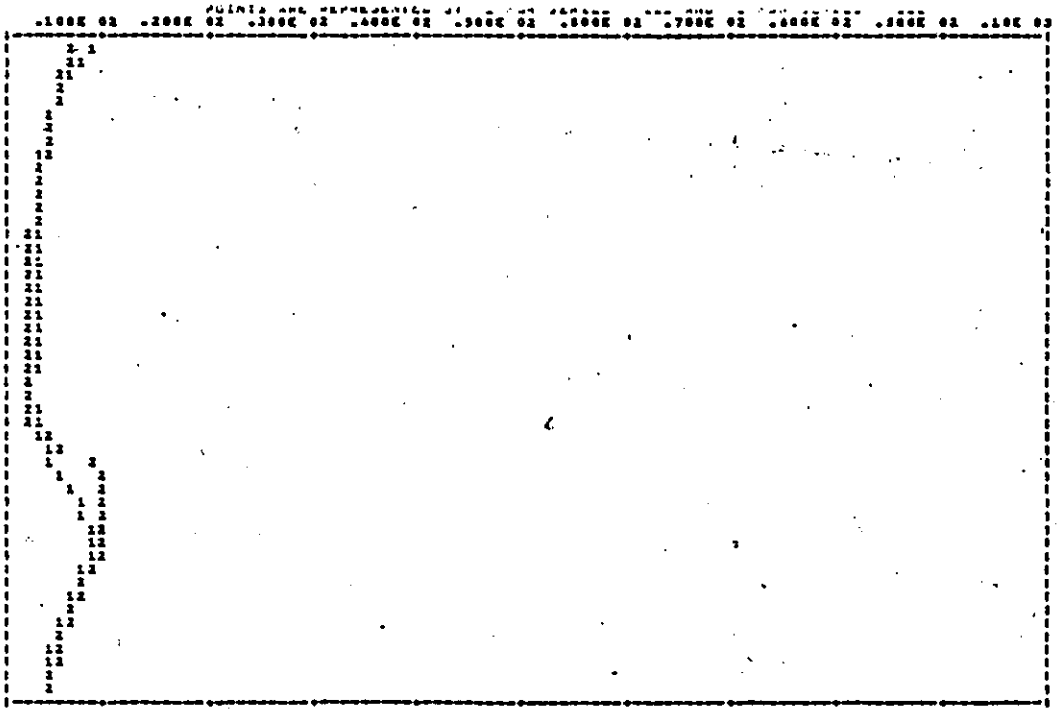

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PAGE BEGINS AT 1978, 8, 1
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 1   8.00|0.470E 01|0.530E 01
 1   9.00|0.457E 01|0.512E 01
 1  10.00|0.446E 01|0.494E 01
 1  11.00|0.438E 01|0.477E 01
 1  12.00|0.430E 01|0.455E 01
 1  13.00|0.423E 01|0.434E 01
 1  14.00|0.418E 01|0.413E 01
 1  15.00|0.412E 01|0.395E 01
 1  16.00|0.408E 01|0.380E 01
 1  17.00|0.404E 01|0.378E 01
 1  18.00|0.401E 01|0.367E 01
 1  19.00|0.395E 01|0.357E 01
 1  20.00|0.395E 01|0.350E 01
 1  21.00|0.392E 01|0.348E 01
 1  22.00|0.390E 01|0.343E 01
 1  23.00|0.387E 01|0.339E 01
 1  24.00|0.385E 01|0.332E 01
 2   1.00|0.383E 01|0.328E 01
 2   2.00|0.380E 01|0.325E 01
 2   3.00|0.378E 01|0.321E 01
 2   4.00|0.375E 01|0.318E 01
 2   5.00|0.374E 01|0.318E 01
 2   6.00|0.371E 01|0.314E 01
 2   7.00|0.369E 01|0.318E 01
 2   8.00|0.367E 01|0.318E 01
 2   9.00|0.365E 01|0.318E 01
 2  10.00|0.363E 01|0.318E 01
 2  11.00|0.361E 01|0.321E 01
 2  12.00|0.358E 01|0.307E 01
 2  13.00|0.356E 01|0.304E 01
 2  14.00|0.354E 01|0.297E 01
 2  15.00|0.352E 01|0.293E 01
 2  16.00|0.350E 01|0.288E 01
 2  17.00|0.348E 01|0.279E 01
 2  18.00|0.348E 01|0.279E 01
 2  19.00|0.344E 01|0.279E 01
 2  20.00|0.342E 01|0.279E 01
 2  21.00|0.341E 01|0.283E 01
 2  22.00|0.339E 01|0.280E 01
 2  23.00|0.337E 01|0.282E 01
 2  24.00|0.335E 01|0.278E 01
 3   1.00|0.333E 01|0.278E 01
 3   2.00|0.331E 01|0.272E 01
 3   3.00|0.328E 01|0.272E 01
 3   4.00|0.328E 01|0.279E 01
 3   5.00|0.326E 01|0.278E 01
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 3   7.00|0.322E 01|0.282E 01
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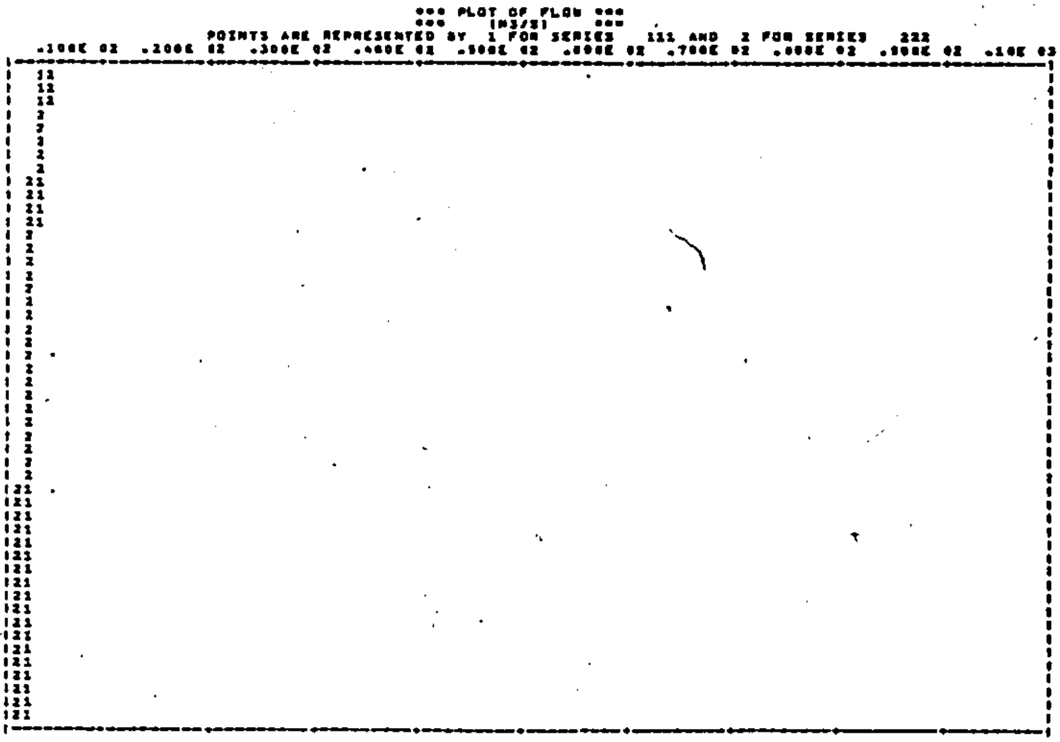


CALIBRATION TO SAWMILL CREEK FECAL COLIFORM DATA

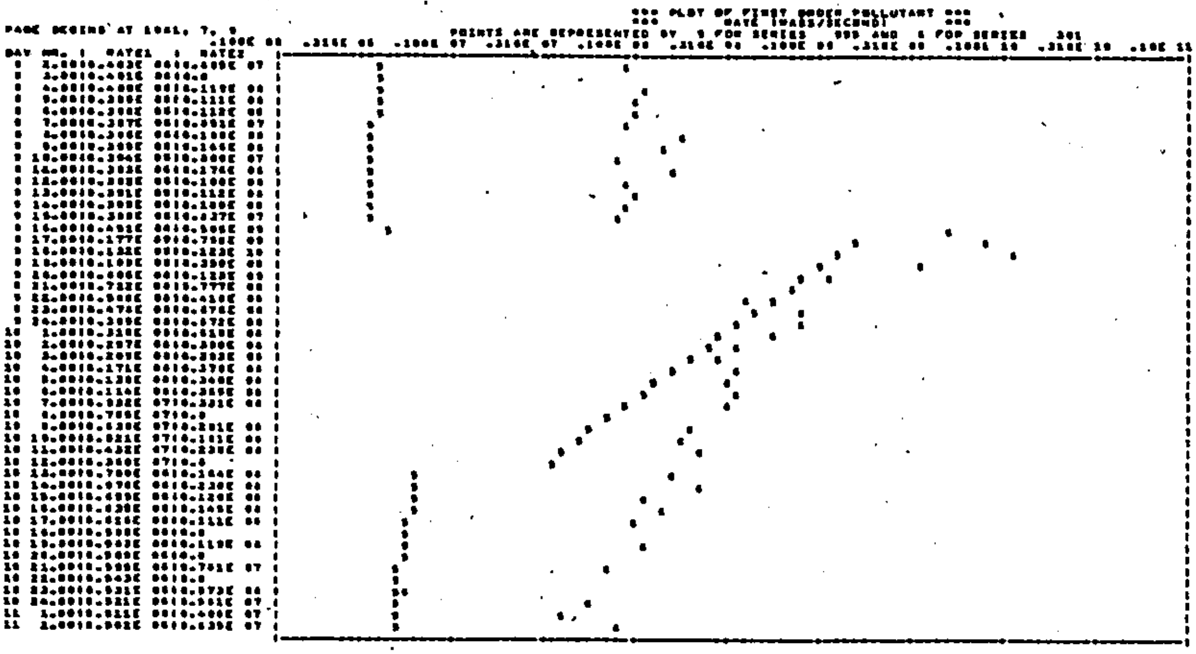
CALIBRATION TO SAWMILL CREEK FECAL COLIFORM DATA - MODEL OUTPUT OF STATISTICS FOR CALIBRATION EVENTS, 1981.

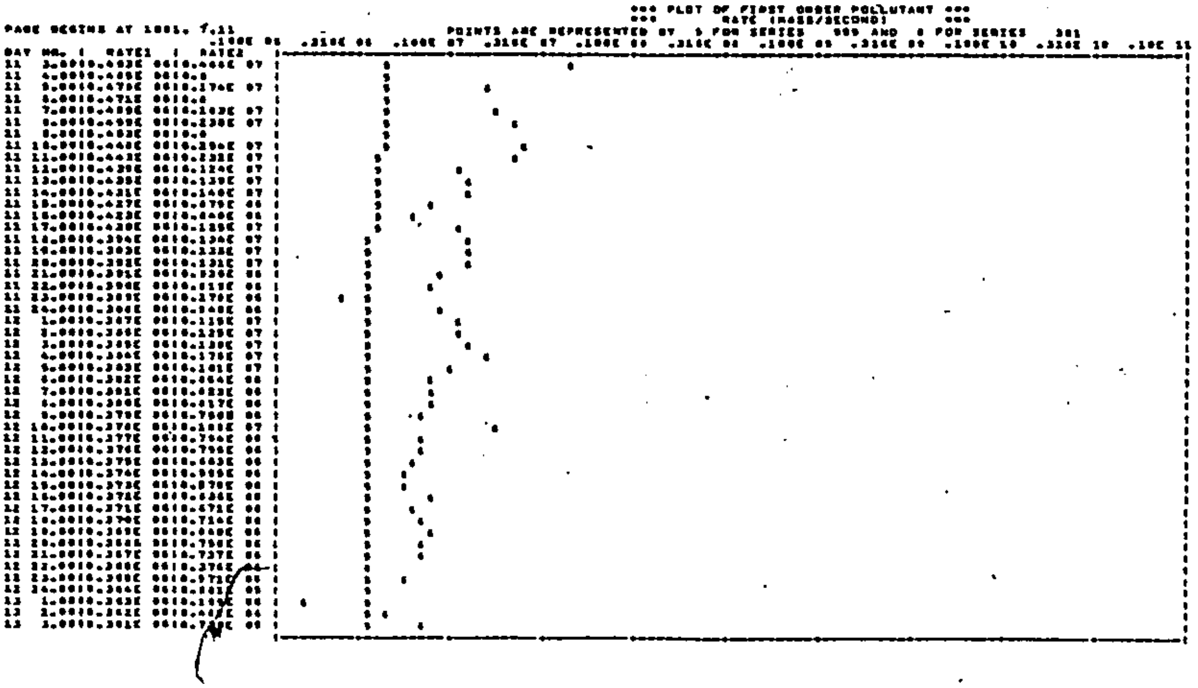
```
       ZALFA = 1234567890 *./-

             COMMAND TABLE

       START                    1 25
       STORE                    2310
       GENERATE                 3310
       PRINT SPAN               4  2
       PLOT SPAN                5 10
       ADD SERIES               6  4
       POND                     7310
       REACH                    8310
       CALIBRATE                9310
       POLLUTANT RATES         10  8
       SPLIT SERIES            11  5
       DUMP PRINT              12  5
       EXCEEDANCE CURVES       13310
       DUMP PLOT               14  6
       FINISH                  15  0
   START                   FROM ALL RECORD (-1)
                            INPUT RAIN FROM DEVICE 8
                            DUMMY FLOW DEVICE = 5
                           SIMULAT FC BACTERI (IFDECA=1) WITH DIEOFF .8 PD
                           NO  SEDIMENT SIMULATION
   POLLUTANT RATES         ICASE=1 IWH=1 IMPERV, FIRST ORDER RATING CURVE
                           FC=8000000 Q** .67
   POLLUTANT RATES         ICASE=2 IWH=2 PERV, FIRST ORDER, RATING CURVE
                           FC=8000000 Q** .67
   POLLUTANT RATES         ICASE=5 BASE FLOW, FIRST ORDER, RATING CURVE
                           FC= 84960  Q**1
   GENERATE                IO= 5 SERIES NO=555 DT=1.0 DA=6000 ACRES
                           NP=0.0 (DO NOT PRINT UNIT HYDS)
                           0.14IS IMPERVIOUS AREA
                           NW=1.0 (NASH HYD)
                            1.05 SLR AND TP=.2
                           MAX ABSTRACTION IMP AREA IS  .16
                           RUNOFF COEFF IMP AREA IS 1.00
                           NW=1.0 (NASH HYD)
                            1.05 SLR AND TP=.5
                           SMIN= 1. INS SMAX= 2000 INS SK=3.5
                           API CONSTANT IS .9 PER DAY
                           API INITIAL VALUE= 1.5INS
                           MAX APSTRACTION PER AREA= .17NS
                           SVOL INITIAL VALUE =.66 INS
                           BASE RESS CONST=.00134
                           AREA REDUC FACTOR=0.58
                           BASMIN=0.0

       IMPERVIOUS AREA UNIT HYDROGRAPH DATA:

            SHAPE CONSTANT, N =  1.050
            UNIT PEAK, QP =     178.12CFS

            SUM OF THE UNIT HYDROGRAPH ORDINATES =     0.890

         PERVIOUS AREA UNIT HYDROGRAPH DATA:

            SHAPE CONSTANT, N =  1.050
            UNIT PEAK, QP =     437.67CFS
            SUM OF THE UNIT HYDROGRAPH ORDINATES =     0.894
   API REDUCTION IS = 0.998E 00* 0.900E 00* PER STEP/DT
```

```
   CALIBRATE              5  EVENTS FROM FILES 5 AND 8
                              81 7  8 81 7 12
                             -81 7 13 81 7 16
                             -81 7 18 81 7 22
                             -81 7 26 81 7 30
                             -81 8  2 81 8  8
                                                  QUANTITY
      SPAN          NAME                                               SED2       SED3       SED4       SED5
   FROM    TO                        FLOW        POL1        SED1
 81 7 8 81 712 555   TOTAL   0.280E 07 0.332E 13 0.0         0.0        0.0        0.0        0.0
               555   RATE    0.400E 02 0.177E 09 0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.311E 07 0.192E 14 0.0         0.0        0.0        0.0        0.0
               301   RATE    0.879E 02 0.123E 10 0.0         0.0        0.0        0.0        0.0
                                                  QUANTITY
      SPAN          NAME                                               SED2       SED3       SED4       SED5
   FROM    TO                        FLOW        POL1        SED1
-81 713 81 716 555   TOTAL   0.140E 07 0.770E 12 0.0         0.0        0.0        0.0        0.0
               555   RATE    0.100E 02 0.377E 08 0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.188E 07 0.248E 13 0.0         0.0        0.0        0.0        0.0
               301   RATE    0.380E 02 0.178E 09 0.0         0.0        0.0        0.0        0.0
                                                  QUANTITY
      SPAN          NAME                                               SED2       SED3       SED4       SED5
   FROM    TO                        FLOW        POL1        SED1
-81 718 81 722 555   TOTAL   0.415E 07 0.101E 14 0.0         0.0        0.0        0.0        0.0
               555   RATE    0.948E 02 0.402E 09 0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.327E 07 0.438E 13 0.0         0.0        0.0        0.0        0.0
               301   RATE    0.731E 02 0.191E 09 0.0         0.0        0.0        0.0        0.0
                                                  QUANTITY
      SPAN          NAME                                               SED2       SED3       SED4       SED5
   FROM    TO                        FLOW        POL1        SED1
-81 726 81 730 555   TOTAL   0.159E 07 0.159E 13 0.0         0.0        0.0        0.0        0.0
               555   RATE    0.182E 02 0.744E 08 0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.204E 07 0.145E 13 0.0         0.0        0.0        0.0        0.0
               301   RATE    0.168E 02 0.364E 08 0.0         0.0        0.0        0.0        0.0
                                                  QUANTITY
      SPAN          NAME                                               SED2       SED3       SED4       SED5
   FROM    TO                        FLOW        POL1        SED1
-81 8 2 81 8 8 555   TOTAL   0.109E 08 0.304E 14 0.0         0.0        0.0        0.0        0.0
               555   RATE    0.280E 03 0.108E 10 0.0         0.0        0.0        0.0        0.0

               301   TOTAL   0.200E 08 0.688E 14 0.0         0.0        0.0        0.0        0.0
               301   RATE    0.535E 03 0.333E 10 0.0         0.0        0.0        0.0        0.0

***** TOTAL MASS *****
  SERIES  555          0.208E 08 0.462E 14 0.0         0.0        0.0        0.0        0.0

  SERIES  301          0.303E 08 0.937E 14 0.0         0.0        0.0        0.0        0.0

***** SUMMARY COMPARISON OF SERIES  555 AND   301 *****
                                                                  0.0        0.0        0.0
AVE (MASS RATIO)          0.634      0.484      0.0        0.0        0.0        0.0        0.0
AVE (RATE RATIO)          0.703      0.364      0.0        0.0
                                                                  1.000      1.000      1.000
RMS (MASS RATIO-1)        0.280      0.803      1.000      1.000      1.000      1.000      1.000
RMS (RATE RATIO-1)        0.476      0.808      1.000      1.000
   NOTE: RATIO = MASS OR RATE FROM SERIES     555          DIVIDED BY MASS OR RATE FROM SERIES    3
```

PLOT OF OBSERVED AND SIMULATED F.C. BACTERIA
(SAWMILL CREEK)

PLOT OF OBSERVED AND SIMULATED F.C. BACTERIA - 9 JULY 1981





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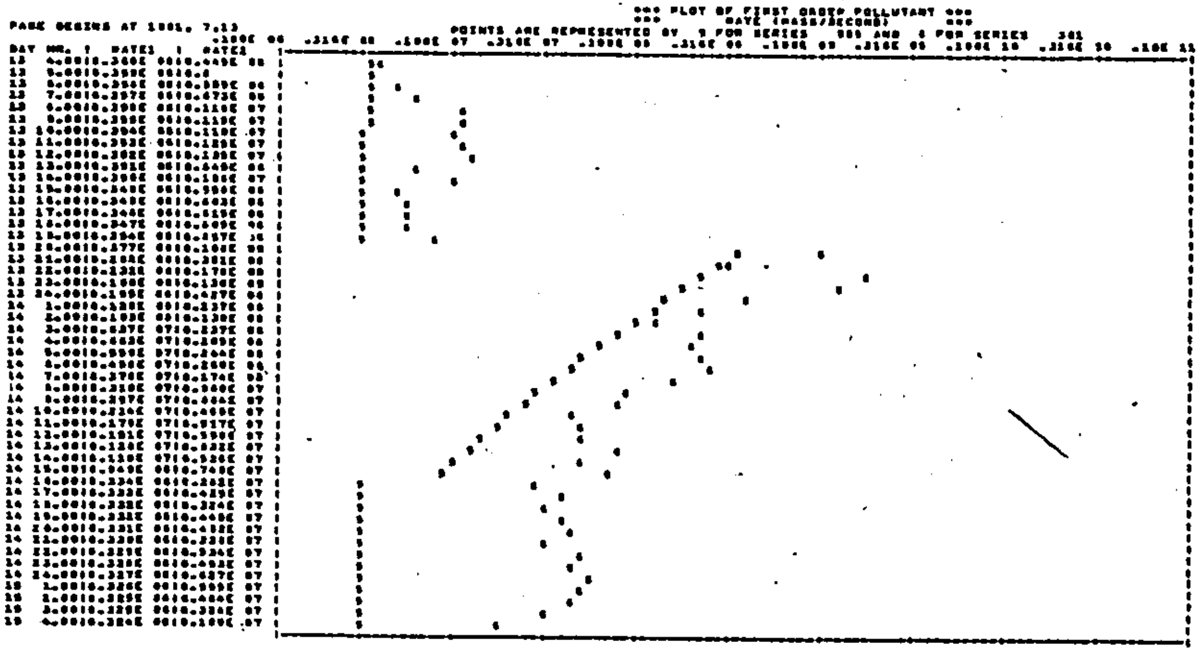

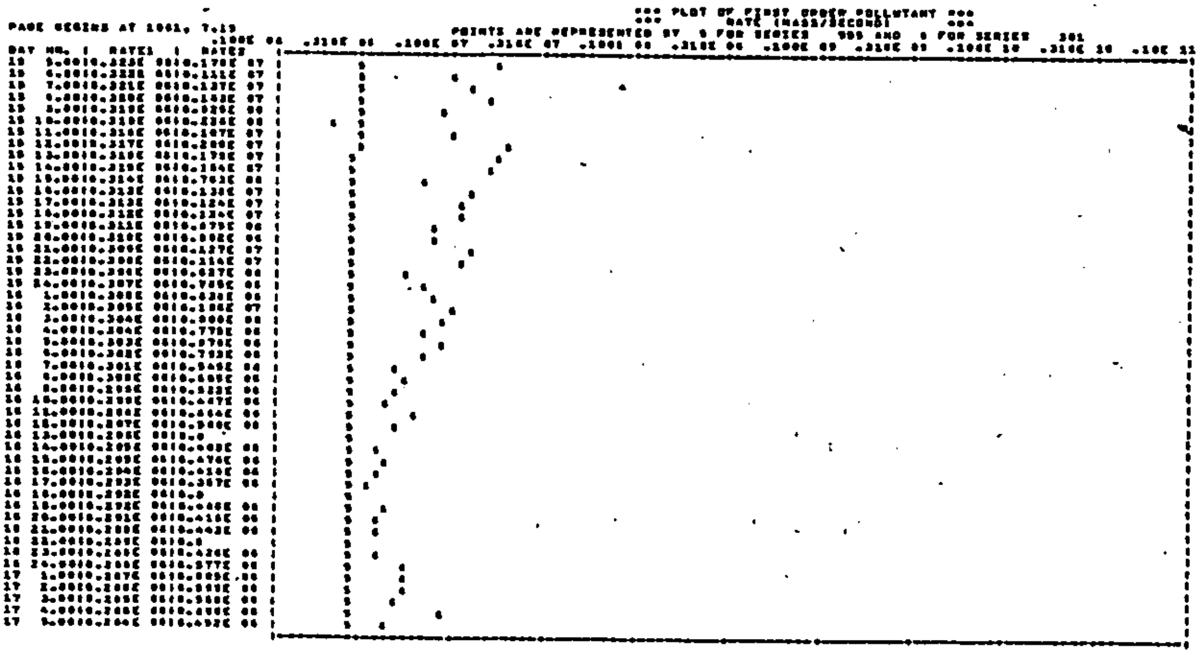

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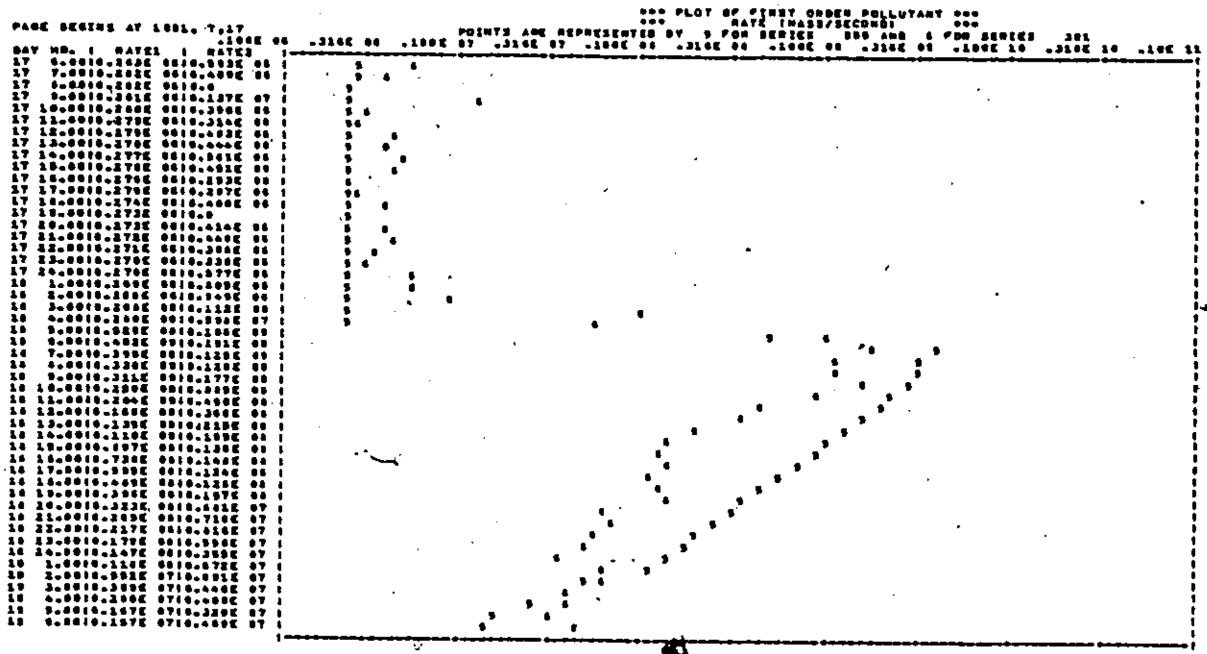

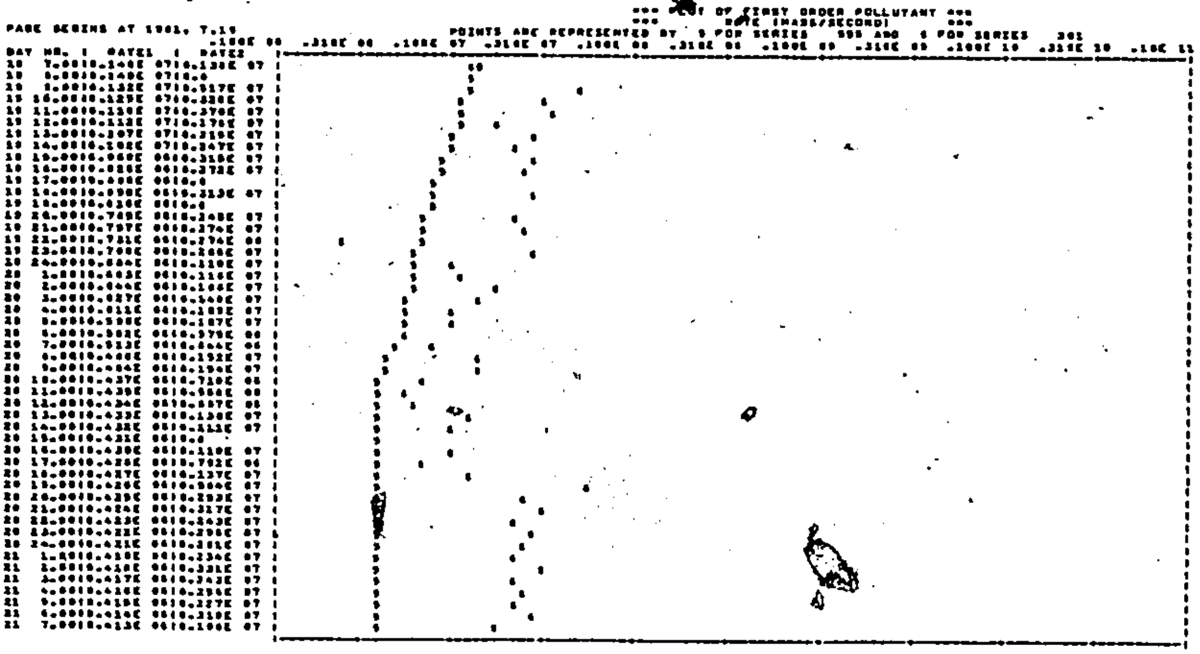

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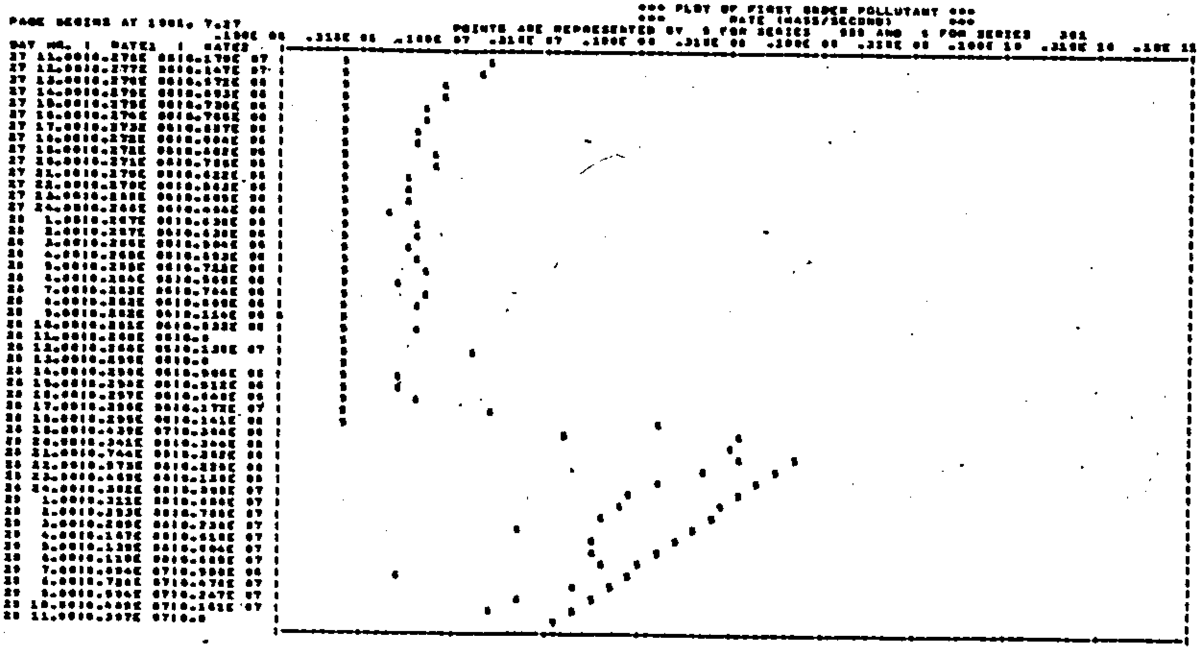
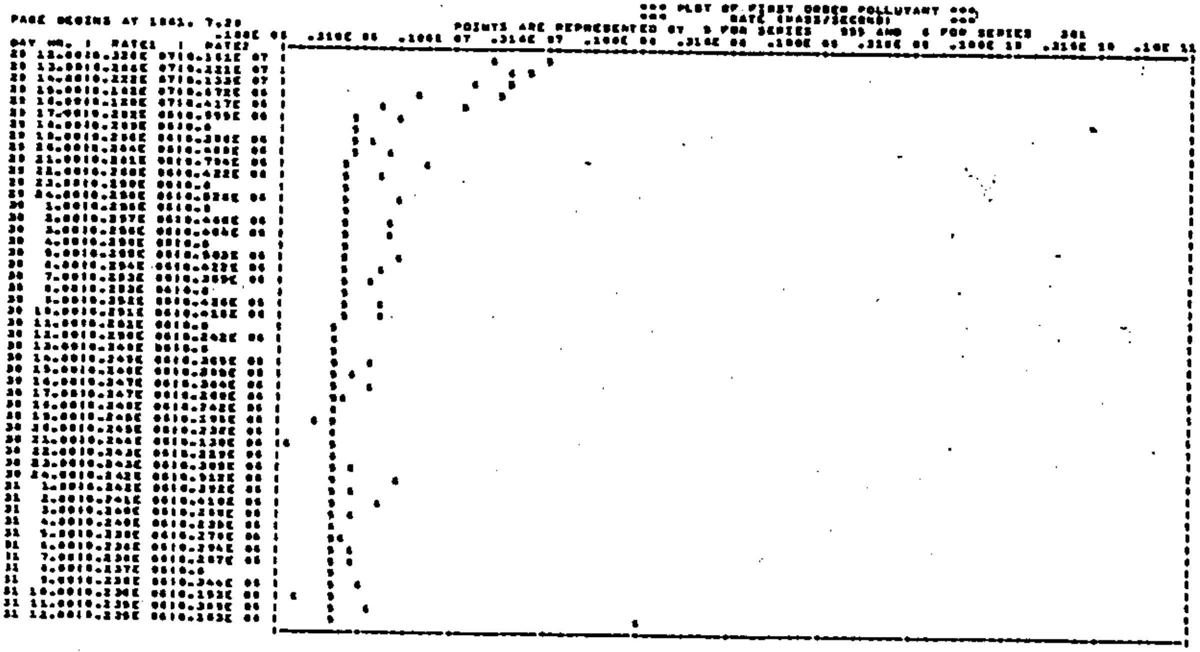
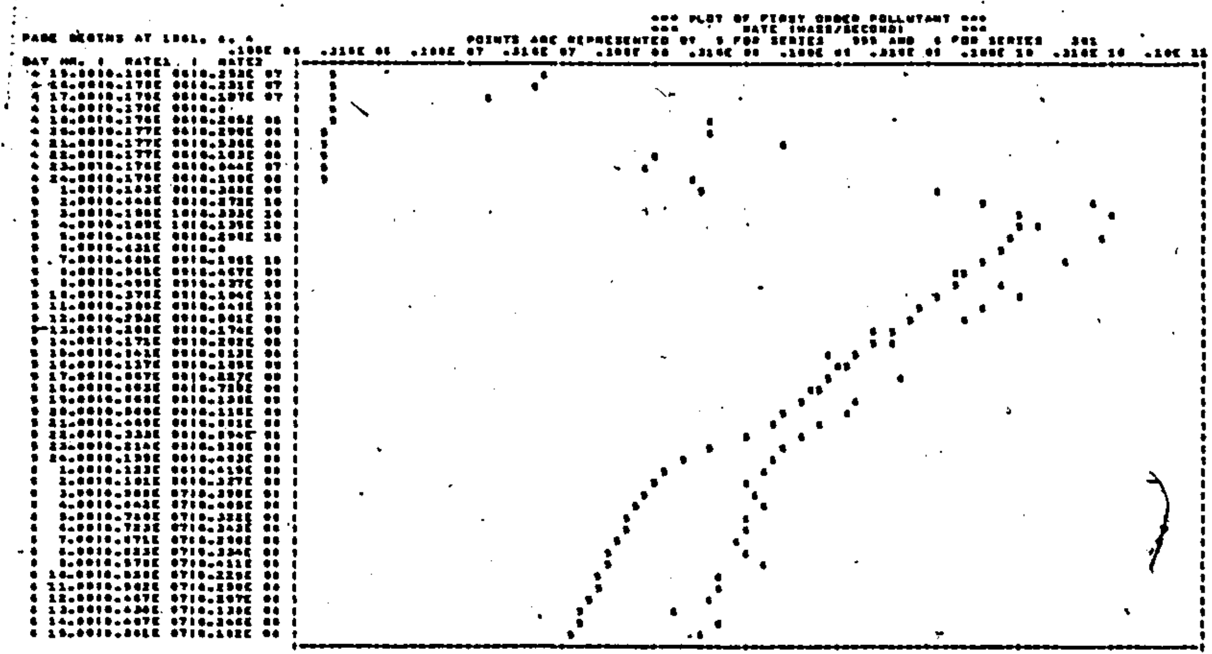
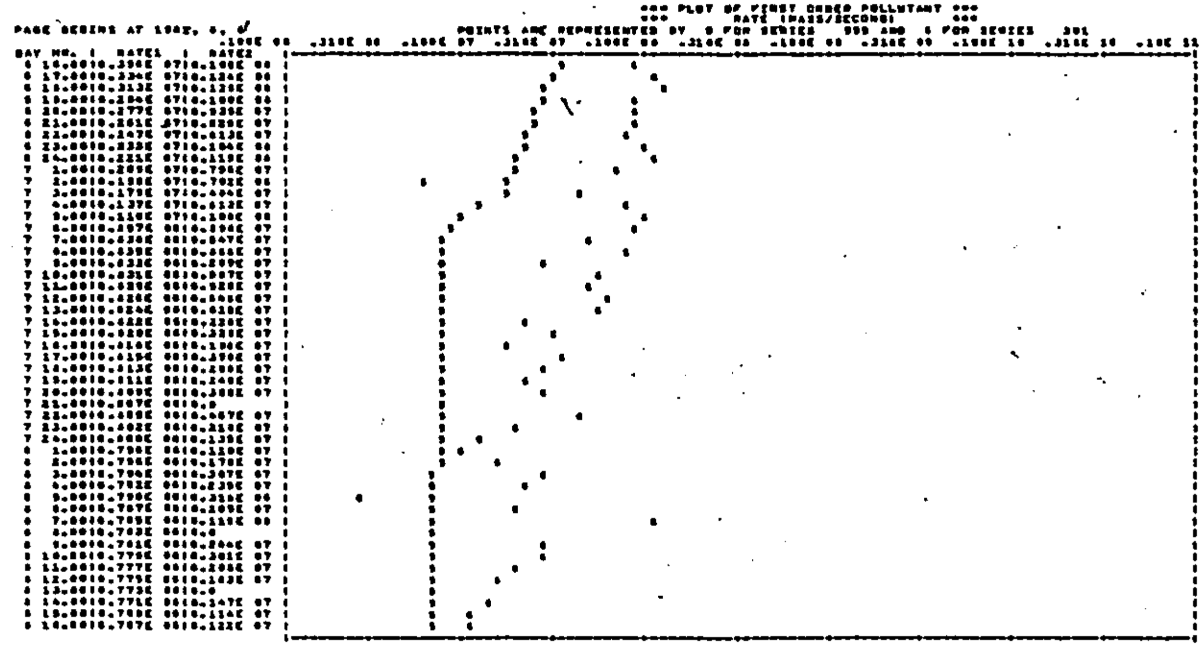




PLOT OF OBSERVED AND SIMULATED F.C. BACTERIA - 5 AUGUST 1981





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