Development and Application of A Simultaneous Routing Model For Dual Drainage Systems

by

Atef M. A. Kassem

A thesis presented to the School of Graduate Studies and Research in partial fulfillment of the requirements for the degree of Doctor of Philisophy in Department of Civil Engineering University of Ottawa

OTTAWA, 1982

I hereby declare that I am the sole author of this thesis.

I authorize University of Ottawa to lend this thesis to other institutions or individuals for the purpose of scholarly research.

Atef M. A. Kassem

I further authorize University of Ottawa to reproduce this thesis by photocopying or by other means, in total or in part, at the request of other institutions or individuals for the purpose of scholarly research.

Atef M. A. Kassem

University of Ottawa requires the signatures of all persons using or photocopying this thesis. Please sign below, and give address and date.

ABSTRACT

A comprehensive mathematical model (Dual Drainage Simultaneous Routing Model - DDSRM) has been developed for the design/analysis of urban storm drainage systems considering infrequent as well as frequent storms. The model is based on the dual drainage principle (major-minor systems). Surface runoff is routed simultaneously in the two interconnected networks formed by the storm sewer system and the streets. Special arrangements proposed for the design of the dual drainage system, and considered in DDSRM, include limitation of flow access into the sewers (inlet control) for the purpose of limiting sewer surcharge, and separate storage facilities for the minor and the major systems (dual storage). DDSRM has several options which allow the flexibility to handle a variety of design conditions. Flow charts for various sub-programs and a user's manual have been provided.

The application of DDSRM for the design of dual drainage/ storage systems is demonstrated through an example for a residential subdivision. Input data and computer output for the numerical example are explained. The model has also been utilized to investigate the role of inlet efficiency

and spacing in the design and operation of dual drainage systems and recommendations for the practical implementation of inlet control have been derived. DDSRM is also suitable for application in master drainage studies.

Another aspect of this study is the development of improved routing methods for unsteady flow under both free surface and pressurized (surcharged) conditions. For free surface flow, a numerical solution scheme for a specially-weighted finite difference formulation of the kinematic wave model, with discharge-dependent wave celerities, has been developed for flow routing in the sewer and street networks. Sewer surcharge simulation is based on a modified Hardy-Cross method, incorporated into an existing dynamic routing model for sewer networks.

The validity of the routing sub-models of DDSRM has been investigated through several numerical experiments. In all the experiments conducted (with slopes greater than 0.1%), the simplified routing method for free surface flow has been found to give results (time-variation of flows, peak flows, time-to-peak and mass conservation) which are not significantly different from those obtained by the complete solutions of the Saint Venant equations for unsteady gradually varied flow. The method for the simulation of sewer surcharge has been verified under transient and steady surcharged conditions. It has been found to predict surcharge levels and durations, and sewer flows with reasonable accuracy for practical purposes.

DDSRM, like other urban drainage models, does not simulate energy losses at the sewer junctions, nor can it simulate hydraulic jumps. The kinematic wave routing and design sub-model cannot simulate backwater effects. However, the surcharge sub-model is capable of representing backwater in the sewer network. The model also assumes a continuous major system and cannot simulate systems with low points where water ponding may occur.

ACKNOWLEDGEMENTS

I wish to express my sincere appreciation to my supervisor Dr. Paul Wisner for his encouragement, sound advice and above all his stimulating comments during the course of this study.

I would also like to acknowledge with thanks the helpful suggestions and comments provided by Dr. Larry Roesner, of Camp Dresser and McKee, Inc. regarding improvements to EXTRAN model.

The co-operation of the Department of Public Works of the City of Toronto for making the HVM model available for the comparative analysis with DDSRM is gratefully acknowledged. In particular, appreciation is expressed to Mr. R. Bremner, Commissioner, for his approval to use the city computer facilities, and to Mr. C. Kitchen, Systems Manager and Mr. W. Wichman, for their co-operation.

Thanks are also due to Miss Susan Glockling and Miss Nicole Renaud for typing this thesis.

This study was made possible through grants provided by the National Sciences and Engineering Research Council of Canada, and by several consultants, municipalities and governmental agencies, in the frame of the IMPSWM program, to whom I extend my sincere thanks.

TABLE OF CONTENTS

Appendix

LIST OF FIGURES

LIST OF TABLES

Chapter I

INTRODUCTION AND RESEARCH OBJECTIVES

1.1 URBAN HYDROLOGY

In 1800 only 1% of the world's population lived in cities of 10,000 and more. This proportion has increased to 20% in 1960 (Lazaro, 1979). The trend towards urbanization is continuing in many countries throughout the world. In Canada the urban population which comprised just over one half of the total population in 1950, is at present over 80% (Ontario Ministry of the Environment, 1977). Urbanization has resulted in increasingly difficult problems in various areas of urban water management, such as water supply, waste water management, urban drainage, flood control and water-related recreational activities, etc.

Hydrological problems associated with urban growth cannot be solved without a scientific approach to urban water resources management. In the last decade, hydrologic research traditionally concerned with rural and large watersheds has increasingly been concerned with smaller urbanized watersheds. Urban hydrology has emerged as a distinct branch of hydrology. Some areas of urban hydrologic research are:

1. application of advanced hydrologic modelling principles for flow simulation in urban drainage systems; and
2. development of new techniques for mitigation of storm water effects and eventual beneficial use. This aspect is known, in North America, as STORM WATER MANAGEMENT (SWM).

Despite the recent progress in urban hydrology and the use of sophisticated urban drainage models in design, widespread flooding of basements and roadways in urban areas occurs on a regular basis all over North America and other parts of the world. The cost of damage due to basement flooding may range from several hundred to several thousand dollars per household. However, the cost of relief sewers for relatively new subdivisions is very high. Recent examples from the City of Ottawa showed that the cost of relief sewers amounted to over one million dollars per subdivision (McNeely, Ltd., 1981). In some cases basement flooding is also accompanied by structural damage. The pictures in Figs. 1.1 and 1.2 show two examples of basement damage and street flooding in urban areas.

While the causes of these problems may differ from one location to another, they can be traced back to the inadequacy of the general design and modelling practices currently used. The present research stems from the urgent need to

Fig. 1.1 Typical Basement Damage Caused By Sewer Surcharge

Fig. 1.2 An Example of Roadway Flooding

explore means of avoiding these flooding problems, which should have national priorities.

1.2 DUAL DRAINAGE SYSTEM

A physical reality of urban storm drainage systems is the existence of two separate and distinct systems: (a) a "minor" (or "convenience") system, and (b) a "major" system. The minor system, formed in most cities by underground conduits (storm sewers), is intended to provide traffic convenience. It requires sufficient capacity to collect and transport runoff from a storm that might be expected to occur once in a 2 to 10 year period. Runoff from storms with return periods of, say, 25 to 100 years is conveyed by the major system, which is formed by: (a) streets which carry overland flow in excess of the minor system capacity, and (b) various creeks, artificial and natural channels.

Traditional design of urban storm drainage systems has, in general, considered only the minor system. Flooding which might occur during severe storms has been accepted as unavoidable or an "act of God".

The inclusion of both the "minor" and "major" systems (dual drainage concept) in the design of urban storm drainage is apparently described for the first time in the drainage criteria manual for the city of Denver (Wright-McLaughlin, 1968). The objective is to increase the level of protection against local (basement and street) flooding, by incorporating, in the design, rare storm events which were previously considered only in flood-plain management. In this manual, however, recommendations for the implementation of the major system in design are limited to providing proper street grading for the conveyance of street overland flow and restrictions regarding the maximum flow depth (or spread) on the streets. The same general recommendations can also be found in other recent publications (Theil, 1978; American Iron and Steel Institute, 1980).

The inclusion of the major-minor systems in urban drainage design is now required by some Canadian regulatory agencies such as Ontario Ministry of the Environment (1977). A detailed analysis of the dual drainage system is required by several Canadian municipalities (Town of Markham, 1978; Town of Oakville, 1979; Borough of Scarborough, 1980).

Failure to provide proper conveyance of street overland flow has resulted in frequent occurrences of basement flooding in many locations including Baltimore City (John Hopkins University, 1956), Metropolitan Toronto (Theil, 1978), the city of Nepean (Kostuch, Ltd., 1981), Gloucester (McNeely, Ltd., 1981), the city of Edmonton (Ahmad, 1980), and many others. In many of these cases, basement flooding is caused by the accumulation of street overland flow at low points resulting in localized high inflow, via the inlets, into the sewer system. This, in turn, generates backwater effects, sewer surcharge and eventual flooding by back-up through the foundation drains or other house connections.

1.3 URBAN STORM DRAINAGE MODELS

There is a wide spectrum of techniques available for the design and evaluation of urban storm drainage systems. They range from the simple traditional Rational method to complex computer models. A review of the state-of-the-art of these models (Brandstetter, 1976) shows "a tremendous diversity in scope and purpose, mathematical detail, system elements and hydrological phenomena being modeled, size of system that can be handled, etc.. This diversity, of course, is a result of the varying conditions and objectives which govern the design and evaluation of individual sewerage systems".

Urban drainage models may be divided into three broad categories: planning, analysis/design, and operations. They may also be classified as single event and continuous simulation, distributed and lumped. Analysis/design models which are by necessity of the distributed type are the most sophisticated of the urban drainage models since they require the detailed analysis of discrete elements of the drainage system such as sub-catchments, sewers, etc.

An example of the main components of a complex "distributed" urban storm drainage model is shown schematically in Fig. 1.3. Aside from some features included in some of the models such as water quality simulation or storage computation, urban drainage models are usually structured to simulate two main phenomena: (a) catchment runoff and (b) flow routing in the sewer system. The models in this category have been reviewed elsewhere: MacLaren (1975), Colyer and Pethick (1976), Brandstetter (1976), Yen et al (1976), Yen (1978a), McPherson (1975, 1979), and others. Some of the models which are frequently used in Canada, U.S., and Europe are: EPA - Storm Water Management Model - SWMM (Metcalfe & Eddy, Inc. et al, 1971); Illinois Urban Drainage Area Simulator - ILLUDAS (Terstriep and Stall, 1974) and Hydrograph Volume Method - HVM (Klym, et al 1972).

Distributed urban storm drainage models differ in capabilities, mathematical formulation, etc. Nevertheless, the procedures used for their validation have, in general, been carried out in a similar manner. Model developers usually compare observed with simulated hydrographs for a recorded rainfall event. This process always involves "calibration" of model parameters (mainly hydrologic parameters) in order to obtain the "best fit". A criticism of these procedures is the objectivity used in the calibration process (McPherson, 1979). Perhaps the most critical remark on this process of

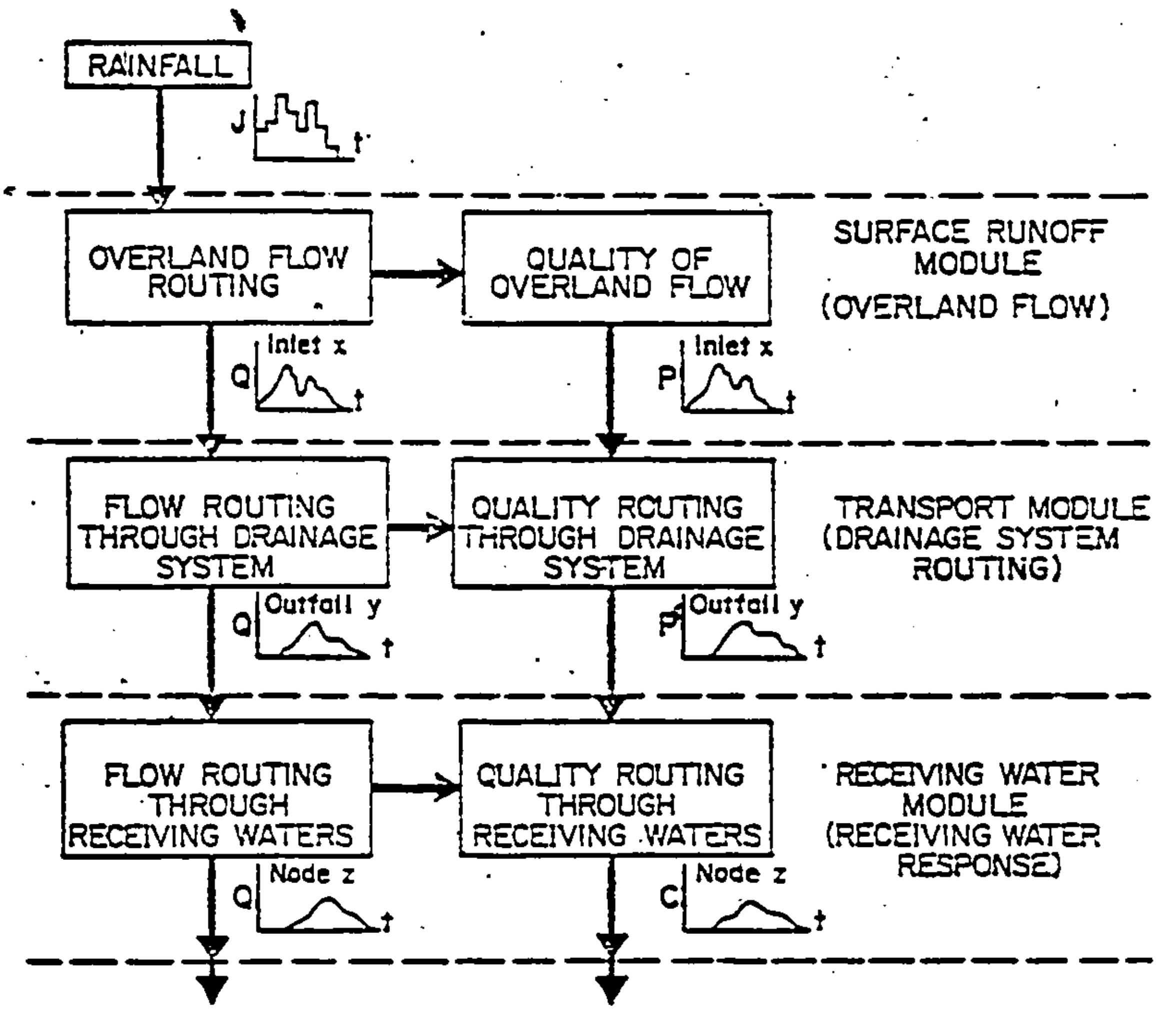


Fig. 1.3 Overview of a Complex Urban Drainage Model
(EPA - Storm Water Management Model)

model validation is that given by Desbordes (1981) "...It would not be surprising, due to the present mathematical development, that one may correctly reproduce an hydrograph recorded in London, from rainfall recorded in New York, using a Japanese catchment".

Moreover, almost all sewer application model validation has been for the total catchment response, at the outfalls. That is, "under contemporary conditions, a distributed system model deteriorates into a lumped system model for all practical purposes" (McPherson, 1979). It has also been noted that "...until each internal sub-model of the overall model can be independently verified, the model remains strictly a hypothesis with respect to its internal locations and transformations..." (Gburek, 1971).

1.4 SCOPE AND OBJECTIVES OF RESEARCH

Increasing the level of protection against local flooding in urban areas is one of the main objectives of applying the dual principle in the design of drainage systems. However, the implementation of the dual drainage concept is not a simple street grading problem as suggested in some publications (section 1.2). An independent analysis conducted at the initial phase of the present study (Wisner and Kassem, 1980a) shows that the design should also consider, among other things:

1. inlet efficiency and spacing;
2. the limitation of flow access into the sewers during infrequent storms;
3. the minimization of the inconvenience associated with the occurrence of street overland flow during infrequent rainfalls; and
4. design of storage facilities for the purpose of runoff control.

The ultimate objective is to provide drainage systems for new subdivisions which are free from basement flooding by considering rare storm events in the design, and to extend runoff control to include infrequent, as well as frequent, storms.

A tacit assumption in all existing urban storm drainage models is that all the catchment runoff is transferred directly (through "hypothetical" inlets) into the underground sewers. This simplistic conceptual approach considers only one component (minor system) of what, in reality, is a "dual drainage" system. The validity of this assumption will depend on several factors, the most important of which is the storm severity (design storm frequency). Existing models have been mainly applied in conjunction with storms of return frequencies of 2 to 10 years. If the dual drainage principle is to be applied, the design should consider rare storm events, such as the one in one-hundred years as well.

Under these circumstances the assumption of total interception of runoff by the storm inlets is no longer applicable. Existing urban storm drainage models are, therefore, not suitable for the design of dual drainage systems. A realistic model for these conditions should be based on the simultaneous analysis of the two interconnected networks formed by the storm sewers and the streets. Under these circumstances (a) the inclusion of the hydraulic efficiency of storm inlets in the model may be a key consideration; (b) simulation of sewer surcharge may be necessary; and (c) detention storage facilities, if any, should be designed/analyzed in accordance with the major-minor systems.

Therefore, one of the main objectives of the present research is to develop a mathematical model for the design/analysis of urban drainage systems and the associated storage facilities, based on the dual drainage principle. The proposed model will be called "Dual Drainage Simultaneous Routing Model - DDSRM".

The proposed model would deal primarily with:

1. the computation of rainfall-runoff transformation;
2. flow routing in the street and sewer networks; and
3. flow routing through storage facilities.

In the present research, the methodology for the computation of rainfall-runoff transformation will be borrowed from an existing model which has been adequately tested.

While the technique used in the runoff block of the EPA-Storm Water Management Model (Huber et al, 1980) will be adopted here, the proposed model can accommodate other techniques as well.

The mathematical formulation of DDSRM will therefore focus on the correct representation of flows in the drainage networks rather than the determination of the runoff hydrographs. The two flow conditions considered in DDSRM are:

1. Free surface flow in the major system and in the sewers, in connection with design. In this case, the routing technique should be capable of accurately representing wave attenuation and translation properties as well as conservation of mass. This aspect is also critical for sizing storages.
2. Surcharged flow in the sewers: in this case, the routing technique should provide the following information as accurately as possible: (a) the increased carrying capacity of the sewers under pressurized conditions, and (b) surcharge levels and durations at the sewer junctions.

It is considered in the present research that the storm sewers are designed on the basis of free surface flow, for the minor system design storm runoff. Surcharge flow conditions would be considered for less frequent storm events such as the major system design storm.

Routing Sub-Models For DDSRM:

The development of the unsteady flow routing sub-models of DDSRM for both free surface and surcharged flow conditions will be carried out after an extensive literature search. The use of dynamic routing models for free surface flow conditions will be avoided mainly because they are too expensive in terms of computer time. It will be attempted, however, to have a reliable and simplified routing technique which, for urban drainage conditions, can provide results not significantly different from those obtained by exact (complete) solutions of the unsteady gradually-varied flow equations.

For the simulation of sewer surcharge, a literature search will be made for an available method in an existing sewer network model. The selected method will then be analyzed and further improvements made, if necessary.

Validation:

The procedures usually used for the validation of urban storm drainage models, by comparisons with measurements, have been criticized as indicated in section 1.2. On the other hand, the application of DDSRM will involve rare storm events, and there are no measurements available in urban areas under these conditions. Yen (1981) has also suggested that "...In storm drainage, the overland surface part and sewer part are two separate subsystems. Hence their evaluation should be made separately despite the fact that most simulation models contain both".

In the present research, the validation of DDSRM will be carried out by verifying the accuracy of the routing techniques, for both free surface and surcharged flow conditions, without further evaluation of the rainfall-runoff transformation part. Considerations will also be given to model verification in network situations.

1.5 SUMMARY OF RESEARCH OBJECTIVES

The principal objectives of the present research are: (a) to re-examine the requirements for safe and efficient design of dual drainage systems for new subdivisions, including the design of storage facilities; and (b) to develop, verify, and apply a comprehensive mathematical model for the analysis of the system so devised.

The detailed research objectives are as follows:

1. To examine the requirements for an urban dual drainage system that can accommodate rare storm events such as the one in one-hundred years, and which has the following features:
   a) limitation of sewer surcharge with the objective of reducing the risk of basement flooding in systems which have direct house connections to the sewers;

b) limitation of flow depth on the streets with the minimum feasible inconvenience to traffic; and

c) runoff control extended for infrequent as well as frequent storms.

2. To develop a mathematical model (Dual Drainage Simultaneous Routing Model - DDSRM) for the design/analysis of urban dual drainage systems with the following capabilities:

   a) simultaneous routing of unsteady flow (net rainfall) in the two networks formed by the streets and storm sewers;

   b) modelling of the interconnection (storm inlets) between the two networks;

   c) design/analysis of storage facilities, with consideration to both minor and major rainfall events; and

   d) simulation of sewer surcharge.

3. To develop and incorporate in the proposed model a numerical scheme for free surface flow routing in the sewer and street networks, simpler than dynamic routing but with compatible accuracy for practical purposes;

4. To select, review and, if necessary, improve an existing methodology for sewer surcharge simulation, and to incorporate it in the proposed model;

5. To validate the model by verification of the routing techniques, rather than only by comparisons of outlet hydrographs;
6. To have the model operational and to illustrate its applications in practical design of dual drainage/ storage systems for new subdivisions;
7. To apply the model to the investigation of some of the aspects related to the practical design of dual drainage systems.

Chapter II

AN IMPROVED URBAN DRAINAGE SYSTEM

2.1 INTRODUCTION

The implementation of the dual drainage principle in the design of urban storm drainage systems requires a new approach with regard to some of the elements of the drainage system.

In this chapter, special arrangements proposed for the design of storm inlets, major system, and detention storage facilities will be discussed. The advantages of the new scheme and the inadequacy of available models for its analysis will be pointed out. The main features of a new mathematical model for the design of the proposed dual system will also be discussed.

2.2 SYSTEM COMPONENTS AND MAIN OBJECTIVES

The urban (dual) drainage system considered here is visualized as being composed of:

1. subcatchments, consisting of roofs, driveways, lawns, etc;
2. storm inlets;
3. storm sewers (minor system);

4. roadways (major system); and
5. in addition, the system may also include detention storage facilities, for the purpose of runoff control.

These components of the drainage system are illustrated in Fig. 2.1. The basic flow process on an urban catchment is illustrated in Fig. 2.2. In this system, the street inlets represent the main link between the major and the minor systems. Following a storm event, the net rainfall runs off the catchment surface as lateral inflow into the streets. The storm sewers collect and transport runoff which is intercepted by the storm inlets. Runoff which exceeds the capacities of the storm inlets is carried by the street network. Proper street grading should therefore be provided for flow conveyance without ponding.

The design of this dual drainage system requires the specification of two storm frequencies, one for the minor system and the other for the major system. The former may range from 2 to 10 years, while 25 to 100 years might be considered for the latter.

The following broad objectives will be considered in the design of the dual drainage system:

1. for the minor system design frequency, the sewers should be utilized at or near capacity, with as little flow as possible on the streets;

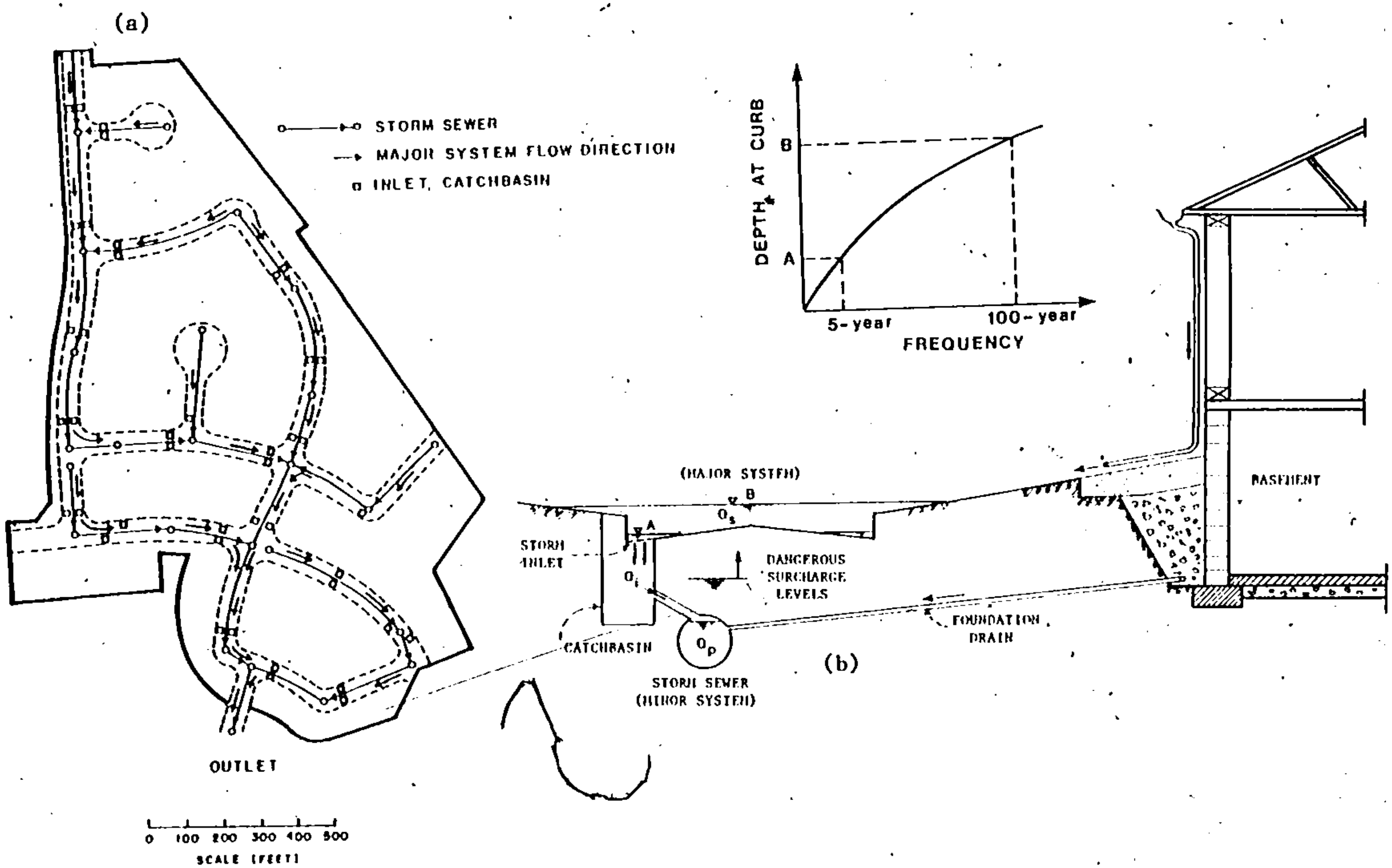


Fig. 2.1 Components of An Urban (Dual) Drainage System

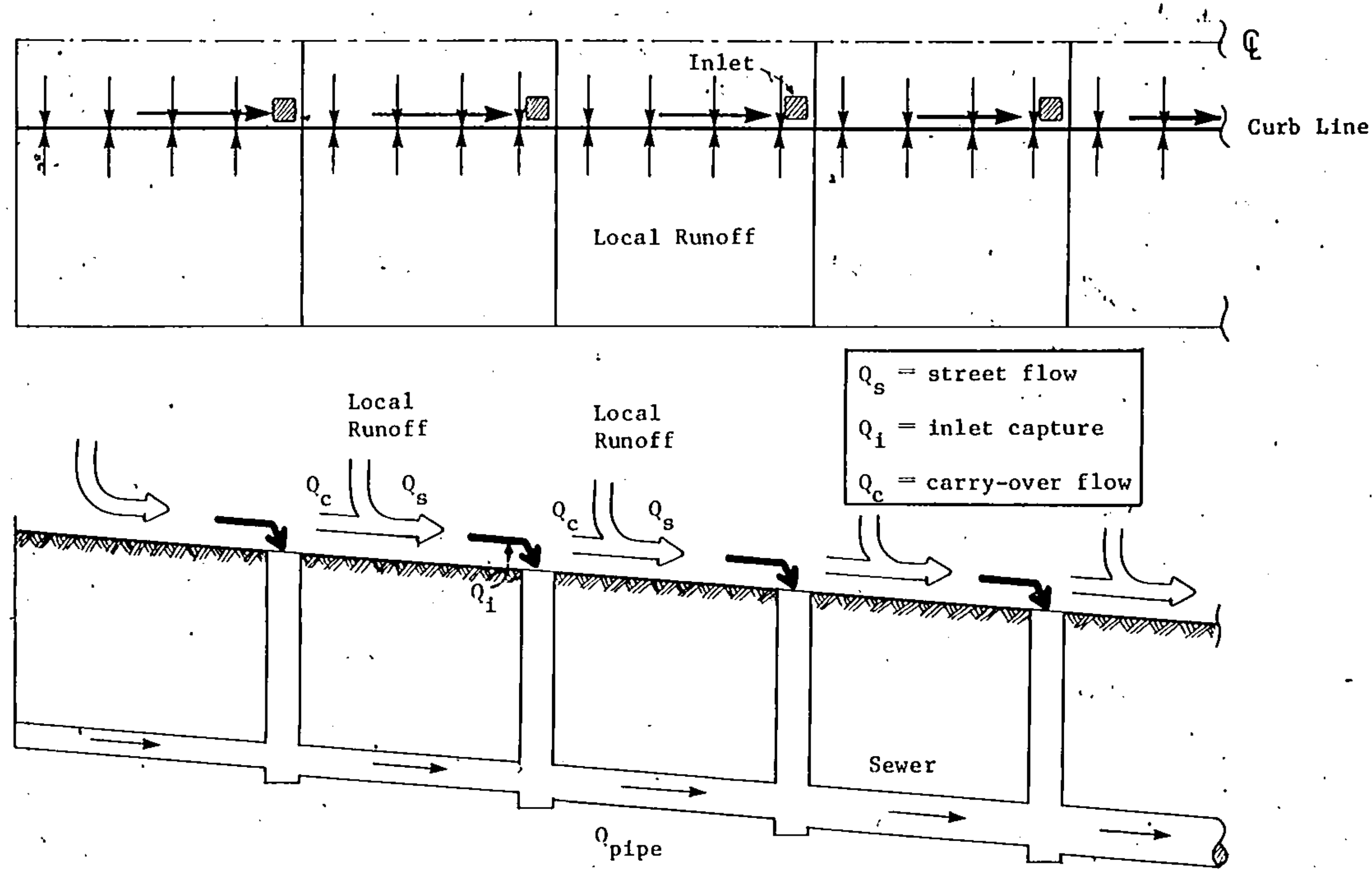


Fig. 2.2 Basic Flow Process on An Urban Catchment

2. for more severe storms:
   a) the minor system flow should not exceed the sewer capacities or at least surcharge levels should be controlled. As shown in Fig. 2.1b surcharge levels above basement elevation may result in basement flooding, caused by back-up through the foundation drains (or direct house connections), if they persist for a sufficient duration.
   b) the major system should be designed such that inconvenience to traffic and pedestrians is minimized.
3. runoff control provided for both minor and major rainfall events (that is, for both pipe flow and street overland flow).

2.3 PROPOSED FEATURES

In order to achieve the main system objectives described above, there are three design aspects of the drainage system which require special consideration. These three aspects are: (a) inlet control, (b) major system discharge, and (c) separate storage facilities for the minor and major systems.

2.3.1 Inlet Control

The hydraulic analysis of storm inlets requires the determination of inlet capacity, that is the hydraulic capture under a given set of conditions. The efficiency of an inlet (see Fig. 2.2) is defined as the ratio of discharge intercepted by the inlet (Q_i) to the approach street (gutter) flow (Q_s). The discharge that by-passes the inlet is termed "carry-over" (Q_c).

There are basically three types of storm inlets: (a) grate inlets, (b) curb opening inlets, and (c) combination inlets. A large number of inlet configurations in the three classes have been investigated in hydraulic laboratory studies (John Hopkins University, 1956; Guillou, 1959; Brune et al, 1975; FHWA, 1977, 1978; U.S. Dept. of Transportation, 1979; and Marsalek, 1980). In highway and road design, inlet type and spacing are selected in order to limit the flow spread for the minor system design frequency (Ministry of Transportation and Communications, 1981). An upper limit for inlet spacing is usually set by the local municipality or regulatory agency in order to achieve this objective. Considerations are also given to road freezing problems in winter. Previous studies on the hydraulic performance of inlets have therefore been concerned with increasing their capture efficiency.

Regulations for spacing of inlets differ considerably from one location to another. The average distance between inlets may range from 150 to 250 ft (45 to 75 m) (MacLaren, 1975). Nevertheless, it is always assumed in urban storm drainage models that the storm inlets will capture 100% of the runoff for the minor system design frequency and hence their hydraulic performance is excluded from the computation. It also seems that the only drainage practices which include inlet hydraulics are those conducted for highway drainage.

Some investigators (Jens, 1975; U.S. Department of Transportation, 1979), however, have referred to the need for including the hydraulic performance of storm inlets in the design of sewerage systems based on the grounds that "many costly storm drains flow at less than design capacity because the storm runoff cannot get into the drains".

The inclusion of inlet capture in the design of storm drainage systems may be debatable for minor storm events. The degree of discrepancy from the assumption of total interception of runoff by the storm inlets will be governed by several factors such as inlet type and configuration, spacing between inlets, catchment imperviousness and design frequencies, in addition to street longitudinal slope, cross slope and gutter slope and configuration. However, if the dual drainage concept is to be implemented, the inlet efficiency becomes an important parameter that cannot be excluded.

For the proposed dual drainage system it is considered that an optimum design of inlets should satisfy two conflicting requirements (Fig. 2.3):

1. for frequent storms up to the design frequency of the minor system, the inlets should capture as much runoff as possible with a minimum of inconvenience to traffic (usually the inlets are spaced such that the flow spread does not exceed a maximum value for the design storm runoff), and
2. for rare storm events, these inlets should work as a buffer to prevent overloading of the storm sewers, by restricting the captured flows. This latter feature was not considered previously in urban drainage design. It will be further referred to as "inlet control". Inlet control may be achieved either by the gratings themselves or special flow constricting devices.

Available data on inlet hydraulics are mainly related to point (1) above, and thus are not adequate for the implementation of the dual drainage concept. Consequently a full scale hydraulic model study[1] was generated at the University

---

[1] This study was carried out in a separate effort under the direction of Prof. R. Townsend, head of the hydraulics laboratory at the University of Ottawa.

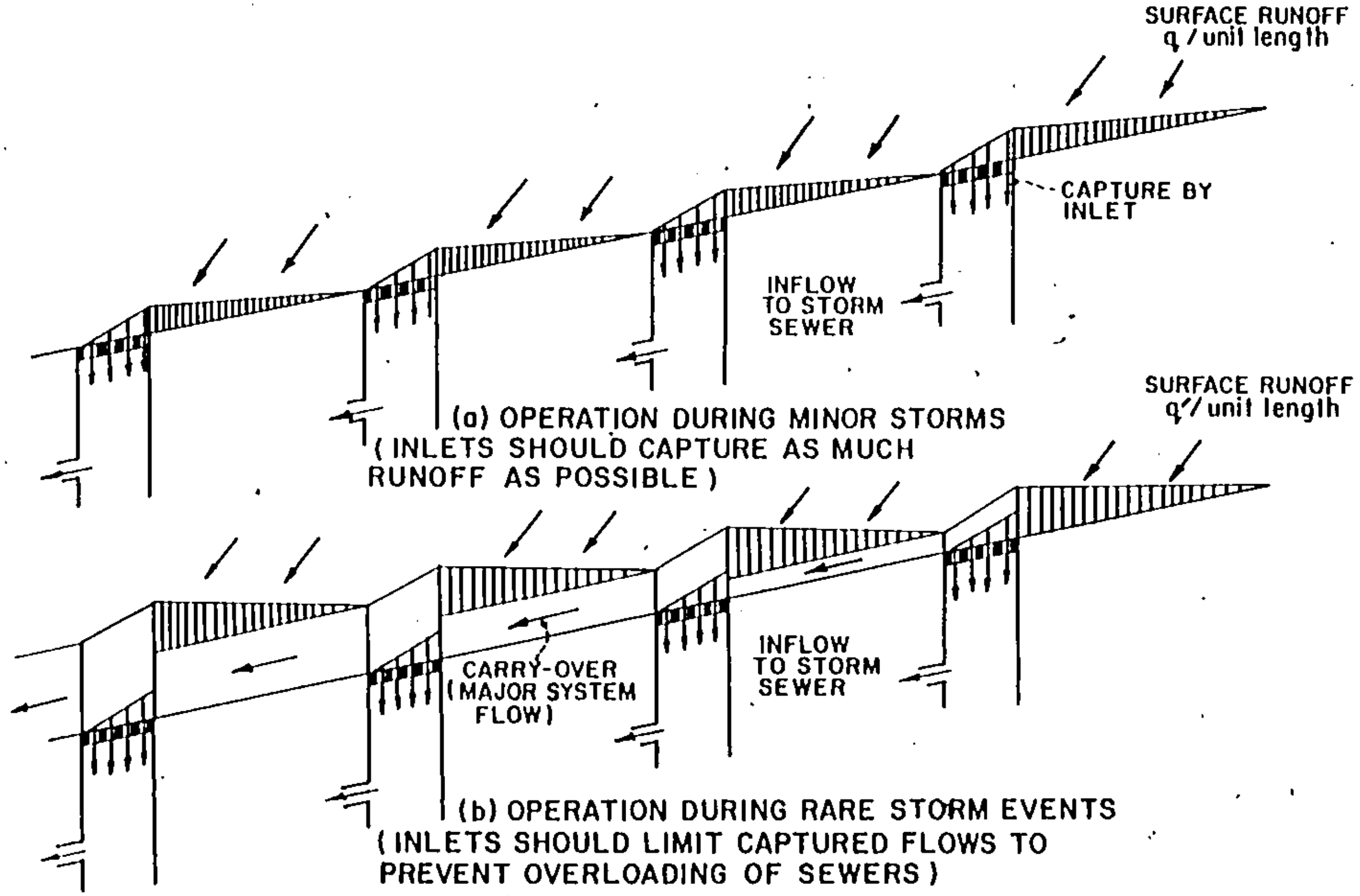


Fig. 2.3 Requirements for An Optimum Design of Storm Inlets for the Implementation of the Dual Drainage Principle

of Ottawa (Townsend et al, 1980). Unlike other studies, this research focussed on "inlet control" from the viewpoint of the dual principle. The main objective of the investigation was twofold: (a) to explore the hydraulic performance of several types and configurations of catchbasin covers used in Canadian practices, which have not been studied previously; and (b) to examine various alternatives for constricting the inlet capture, including observations on clogging by debris, etc. and other maintenance problems.

It was found out that a diamond-shaped orifice plate mounted at the entrance of the pipe connecting the catchbasin and the sewer could provide a reliable flow constricting device with a minimum of maintenance requirements.

Integration of these results, and of other studies on inlet hydraulics, in the design of "dual" drainage systems in new subdivisions can be used to investigate the basic inlet control alternatives, namely: (a) selection of type and spacing of inlets, (b) inlet constriction, and (c) a combination of the above, as will be discussed in Chapter VII.

2.3.2 Discharge of Major System Flow

Inlet control (including inlet constriction) may be utilized in order to have free surface flow in the sewers for all storm frequencies up to (say) 100 years. However, limited sewer surcharge may be allowed during severe storm events. The objective in this case is threefold: (a) to utilize the increased carrying capacity of surcharged sewers, (b) to limit the amount and number of inlet constrictions, and (c) to limit the flow on the streets, that is, minimize inconvenience. In this case, surcharge levels and durations must be carefully analysed in order to avoid basement flooding which may result from surcharge levels exceeding the basement elevation.

The depth of flow on the streets during major storms will also be governed by the configuration of the street network (Wisner and Kassem, 1980a). The increased inconvenience may result from flow accumulation from tributary areas. If this inconvenience is to be minimized, consideration should be given to providing conveniently located outlets for the major system. This can be done at crossings of creeks by provision of swales, etc. Another viable alternative is to discharge the major system flow into "depressed" park areas, which can be utilized for temporary detention of runoff, as discussed below.

2.3.3 Dual Storage For Runoff Control

Runoff control from new developments by means of Storm Water Management (SWM) measures has become increasingly accepted in North America. Its role is to minimize the effects of hydrologic changes such as downstream flooding, erosion, and impairment of water quality. Whenever possible, it is attempted to derive benefits for recreation, irrigation, etc. (Tourbier and Westmacott, 1974). Many regulations assume that it is possible to achieve these objectives by means of simple rules based on "zero runoff increase" above pre-development levels, applied for design storms with one or several recurrence intervals (Fig. 2.4a).

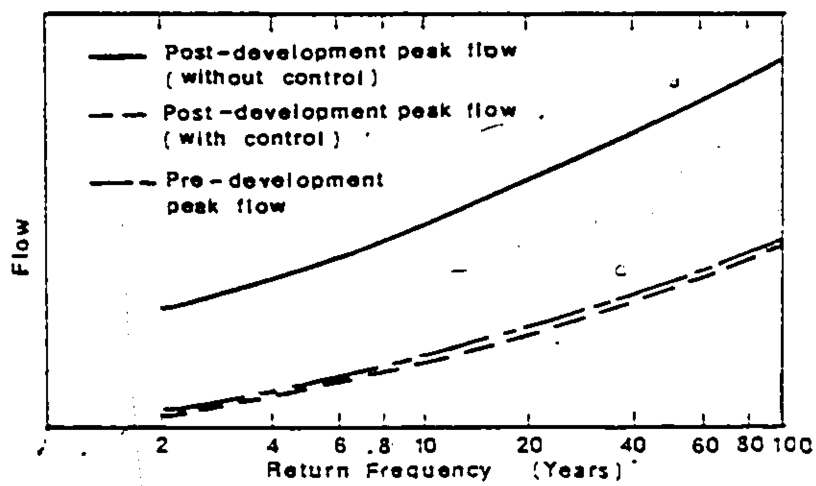
The most common approach for the control of urban runoff is by means of detention or retention storage facilities (Poertner, 1974). Storage facilities provided for minor floods are usually associated with the objectives of protection against erosion (Leopold, 1968), for example in Montgomery County, U.S. (Davis, 1974), or the inadequate capacity of the downstream sewers (MacLaren, 1978a). Dry and wet ponds are usually used in this case. However, in areas where land is expensive, underground storage is preferred (MacLaren, 1978b).

On the other hand, when runoff control is required for both minor and major floods (i.e. for both pipe flow and street overland flow), storage facilities usually consist of a single large storage (dry or wet ponds), ideally with multiple outlets (Wisner and Kassem, 1979).

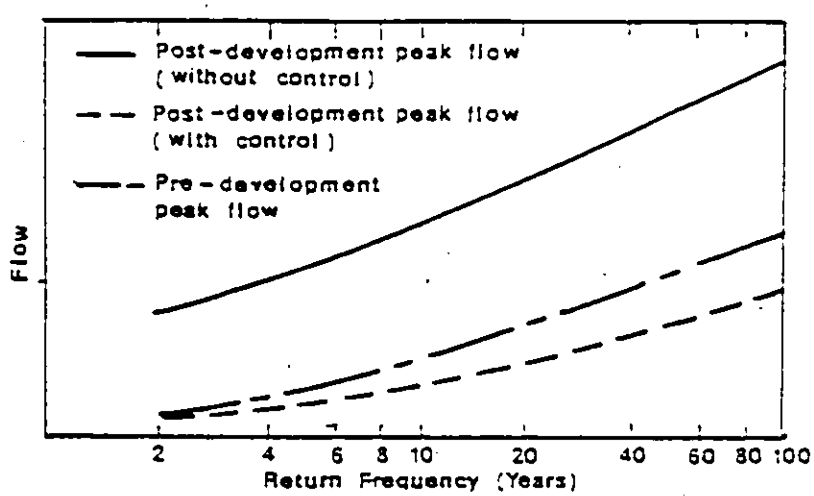
Two main criticisms of the large, dry and wet, ponds for the control of major system flow are:

Maintenance: From the municipalities' viewpoint

Land Requirements: From the developers' viewpoint



(a) Zero Runoff Increase



(b) Overcontrol

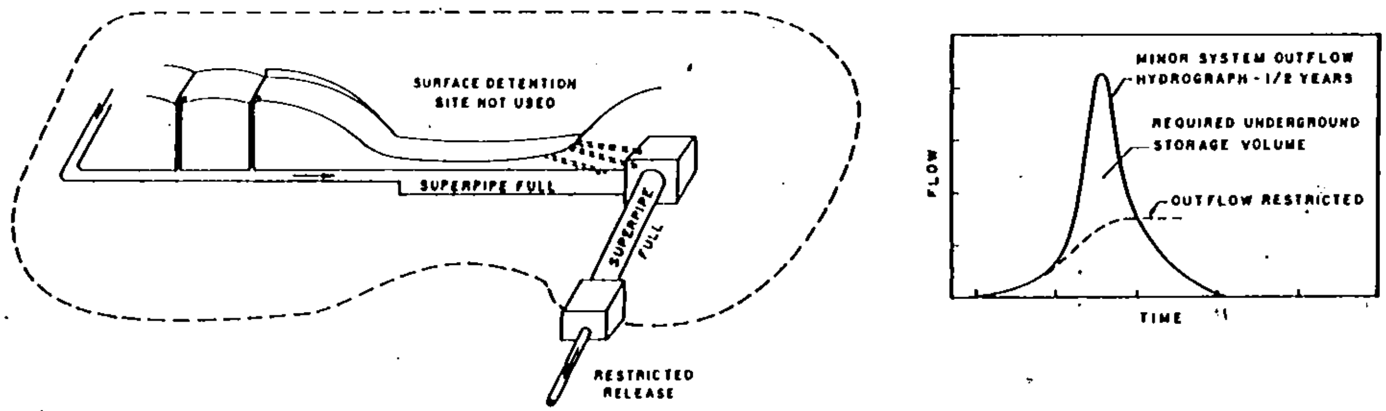
Fig. 2.4 Illustration of Zero Increase and Overcontrol of Runoff For Storms of Several Return Frequencies

One of the main features of the proposed dual drainage scheme is that runoff storage in new developments can be accommodated, for both the minor and major systems, in two separate units (Fig. 2.5): (a) storage for minor system flow, consisting of, for example, oversized pipe (superpipe), and (b) temporary storage of street overland flow from the major system, which can be accommodated, in this case, in depressed park areas. This storage scheme can therefore be designated as "dual storage" (Wisner and Kassem, 1980b).

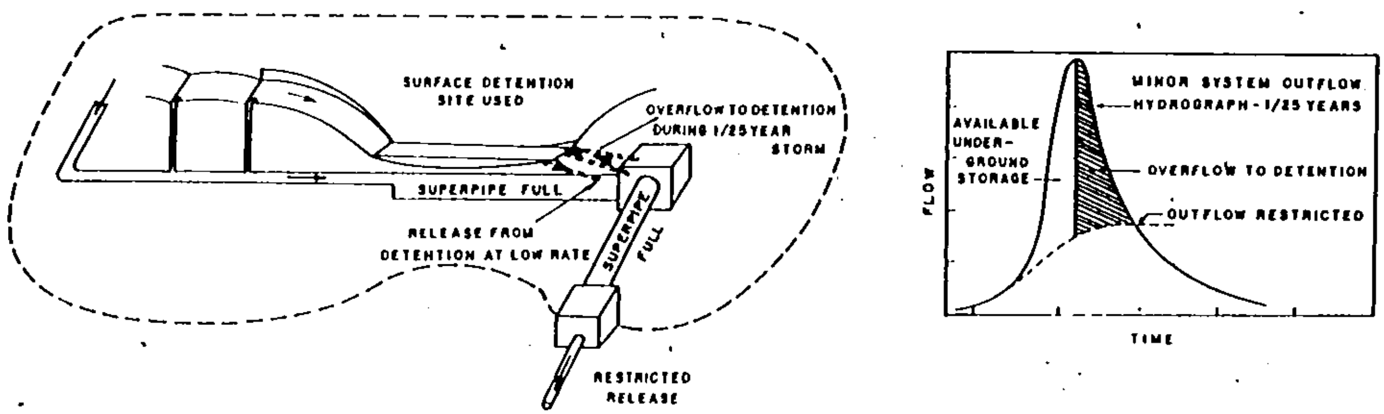
2.3.3.1 Park Storage

The implementation of park storage in new developments is closely related to the dual drainage principle. The major system should be designed with street low points at locations where flows can be directed to the park(s) by shallow open channels. One advantage of utilizing park area for runoff detention is that the depth of flow on the streets can be limited even if discharge into creeks or other open channels is not possible. This, however, requires establishing the location(s) of park(s) at early stages of planning of a subdivision.

Park flooding is temporary and rare. It occurs only when the capacity of the minor system (or more appropriately the capacity of storm inlets) is exceeded. For the approval



OPERATION OF DUAL STORAGE AT 1/2 YEAR FLOW



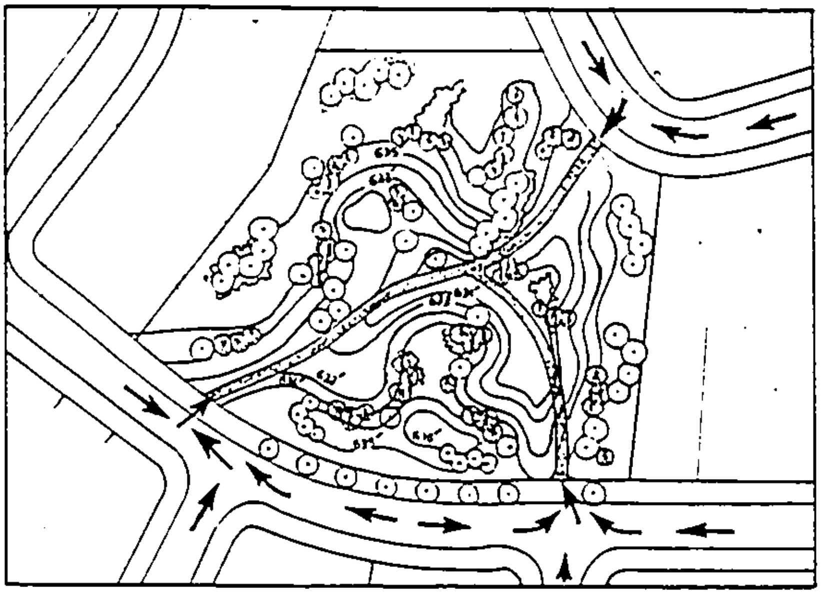
OPERATION OF DUAL STORAGE AT 1/25 YEAR FLOW

Fig. 2.5 Proposed Dual Storage System - Illustration of Operation at Minor and Major Storms of park utilization for runoff detention, safety requirements are also important. They have to be compatible with activities of children, locations of schools and playgrounds. The parks have to be landscaped to accommodate the required detention volume for the major storm design frequency. Examples of the implementation of the proposed park storage are given in Fig. 2.6. Playgrounds in the depressed park can be located at a higher elevation in order to reduce the frequency of flooding for these relatively more expensive facilities, from (say) one in 5 years, to one in 10 or 15 years (Fig. 2.6b). Arrangements should be made for the runoff, which is temporarily stored in the park area, to be released at slow rate, for example, through the sewer system, after the termination of the storm (see Fig. 2.5).

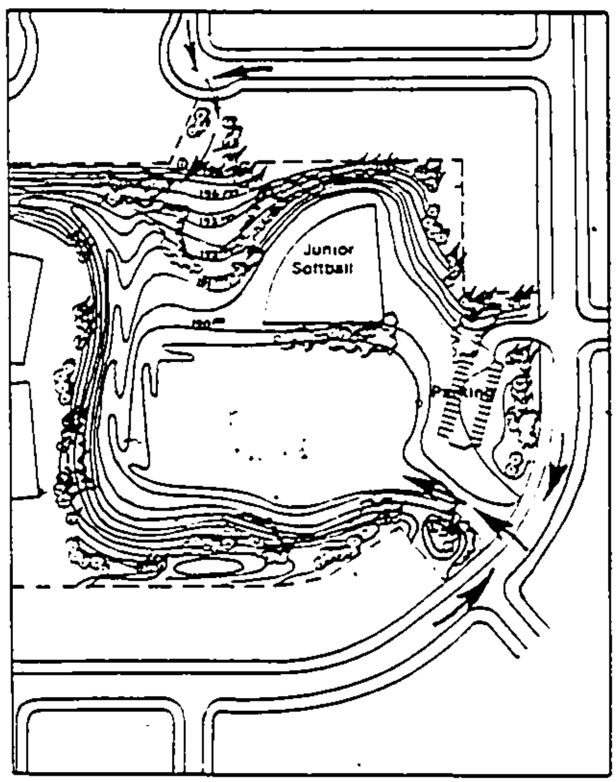
2.3.3.2 Dual Storage

The dual storage system shown schematically in Fig. 2.5 is proposed if runoff control is required for both minor and major storms. It operates as follows:

During frequent storms, the underground storage facility will operate and reduce the peak outflow conveyed by the minor system while the park remains dry. During less frequent storms, both park and underground storage will be utilized and the peak outflows from the major and minor systems will be reduced correspondingly. The peak inflow into the



(a) Landscaping of (Depressed) Park Area for Runoff Detention



(b) Park Storage With Recreational Facilities

Fig. 2.6 Examples of the Implementation of Park Storage in New Subdivisions storm sewers will be controlled by the inlet grates or with special flow constricting devices as discussed in section 2.3.1.

While inlet control can be a very effective method for controlling the peak inflow admitted into the storm sewers, this is not true for the runoff volume. Due to the prolonged duration of peak inflow, via the inlets, the volume of water admitted will increase significantly, to as much as 2.5 times in some situations (Wisner and Kassem, 1979). Increasing the storage capacity will not be economical, and in order to relieve the minor system, overflow facilities are required. They can be located and connected in such a way as to discharge the excess volume into the park. The park will now have to accommodate flows from the major system and overflows from the underground storage during rare storms. The operation of this dual storage system is illustrated in Fig. 2.5, in which it is assumed that the minor and major system storages are designed to accommodate the 2 and 25-year flows, respectively.

2.3.4 Advantages of The Proposed Dual Drainage/Storage Scheme

The implementation of the dual drainage/dual storage system, with the proposed features, in the design of new subdivisions has significant advantages:

1. protection against basement flooding during rare storm events;
2. disconnection of foundation drains is not required;
3. economic benefits: potential for reduction in pipe sizes. The design frequency of storm sewers would be selected for convenience of traffic and not for protection against basement flooding.
4. The inconvenience associated with major storm events is temporary and would occur anyway. In the proposed scheme this inconvenience is controlled and minimized, particularly if park storage is implemented.
5. Surface detention in the parks does not require any additional land. Since park flooding is rare and temporary, maintenance problems are negligible. The utilization of park storage also has significant advantages when compared to other storage alternatives such as wet and dry ponds as indicated in table 2.1.
6. Implementation of dual storage may be utilized for flow reduction for major storms, to less than pre-development levels. This overcontrol (Fig. 2.4b) may be desirable to alleviate flooding problems in existing developments built downstream in the same watershed.
7. Overcontrol of runoff has potential benefits for application as a flood control alternative with the ob-

jective of long term reduction in flood levels in overtaxed streams.

8. The extra cost in the implementation of the proposed system is negligible. The only price to be paid is a more sophisticated design.

Table 2.1 Comparison of Park Storage with Dry and Wet Ponds

	Wet Pond	Dry Pond	Park Storage
Storage	continuous	frequent	rare
Aesthetics	very imp.	very. imp.	less imp.
Maintenance	high	moderate	very low
Accident prob.	moderate	low	very low
Facility cost	moderate	moderate	low
Land cost	high	high	none
Landscaping cost	high	medium	medium
Planning	very imp.	very imp.	very imp.
Experience	adequate	adequate	limited
Modelling	medium	medium	complex

* imp. = important

2.3.5 Implementation

The proposed drainage concept has developed in parallel with new regulations by Canadian municipalities requiring an increased level of protection in the design of urban storm drainage systems to include storms up to the 100-year frequency. The proposed features presented above, namely inlet control, park and dual storage, and considerations regarding an efficient utilization of streets for drainage, have been presented in several papers in the last years (Wisner and Kassem, 1979, 1980a, 1980b, 1981). Some of the proposed features have also been adopted in updated drainage manuals (Town of Oakville, 1979; Borough of Scarborough, 1980).

Two examples of recent applications of dual drainage/storage in practical design of new subdivisions are presented in Figs. 2.7 and 2.8.

1. Fig. 2.7 presents one of the first applications. In this example the minor and major systems are intended for the 5 and 25-year storm frequencies, respectively. The downstream development has also experienced frequent basement flooding problems. Consequently overcontrol of runoff from the new development is desirable. This could be achieved by applying a dual storage system as indicated in Fig. 2.7. Proper street grading is provided with low points which are connected to two parks for temporary detention of runoff during major storms. Note that, in spite of the complexity involved in the detailed design, approval of the project by the downstream jurisdiction is a simple matter since the flow release is controlled by the size of the outlet from the new subdivision.

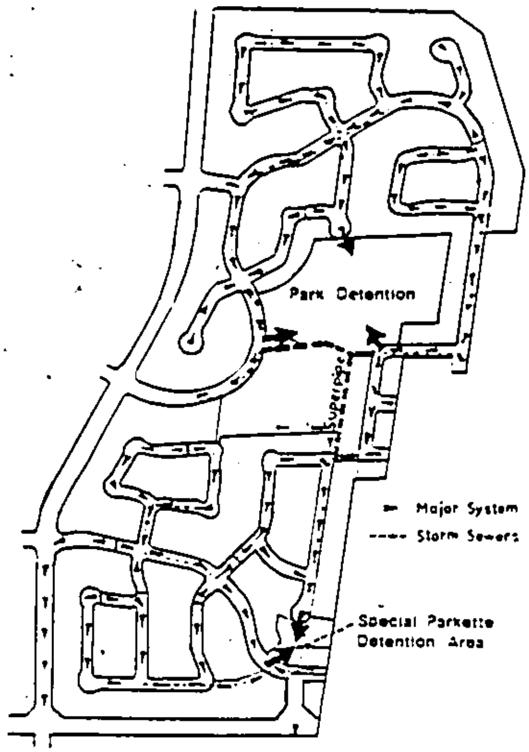


Fig. 2.7 A Subdivision With Dual Drainage and Dual Storage

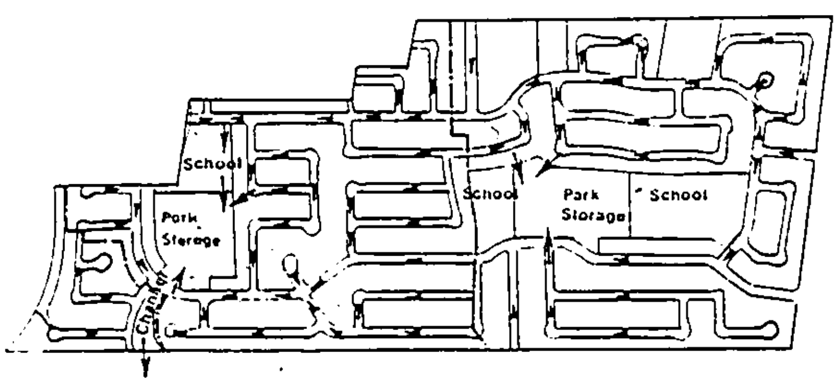


Fig. 2.8 A Subdivision With Park Storage for Major System Flow

2. Another example is presented in Fig. 2.8 in which runoff control is required only for the major system. In this application the minor and major drainage systems are intended for the 5 and 100-year storm frequencies, respectively. The storm sewers are discharged, without control, into an open channel which has sufficient capacity to convey the 5-year post-development flow. The major system is therefore intended to convey flows from the less frequent storms, to two separate parks.

Despite the implementation of the dual drainage concept in recent design of the drainage systems for new subdivisions, the design methods utilized have been based on rather crude assumptions as demonstrated below.

2.4 ATTEMPTS AT MODELLING DUAL SYSTEMS

Several attempts have been made for modelling dual drainage. Two directions have been followed in these efforts: (a) the use of simplified hand computation methods, and (b) ad hoc procedures using existing computer models.

Hand computation methods are described in the drainage criteria manual for the city of Denver (Wright-McLaughlin, 1968), using the Rational method. The same procedures have also been applied (Ontario Ministry of the Environment, 1977; Theil, 1978) using the Rational method for the minor system and the SCS-TR55 (U.S. Department of Agriculture, 1975) for the major system. Recently modelling of a ficti-tious major system has been considered in the "Penn State Urban Runoff Model" (Aron et al, 1979).

In general, all modelling efforts of dual systems have, more or less, followed the Denver procedures where the sizes of storm sewers are first determined as in traditional design by using the Rational method (or a hydrograph meth-od). The major system is next analyzed as follows:

1. compute peak flows for the selected major system de-sign frequency, in conjunction with an "imaginary" major system (usually assuming a wide rectangular channel);
2. subtract flows corresponding to the pipe capacities, in different segments of the drainage network, to ob-tain the major system flows;
3. compute flow depth, and hence determine the minimum elevations at which the buildings should be set.

Although these procedures do not explicitly consider inlet control, they include a tacit assumption that the ac-cummulated capture by the inlets is equal to the sewer ca-pacities. Obviously this approach is rather simplistic and does not reflect the actual operation of the major-minor systems. The inadequacy of these procedures has been demon-strated through systematic analysis (using ad hoc procedures, without flow routing ,but including inlet capture) on several subdivisions (Wisner and Kassem, 1980a). An example is presented in Fig. 2.9.

This example shows that the actual (accumulated) inlet capture is substantially greater than the available capacity of the minor system. Under these circumstances the sewers will surcharge which may result in occurrence of basement flooding. On the other hand, due to the increased carrying capacity of surcharged sewers, the major system flows may be highly overestimated.

If the same procedures are extended for the purpose of sizing storages the consequences are: (a) storage for the major system (e.g. park storage) will not operate as intended, and (b) underground storage, if any, may be seriously overloaded.

Moreover, the Denver method has been mainly applied in conjunction with drainage systems where there is no concern regarding sewer surcharge. It should not be used when the foundation drains are connected to the storm sewers (a practice mostly used at present and assumed in the present study) or in the case of combined systems. Furthermore, the method cannot be extended for the design of storage facilities.

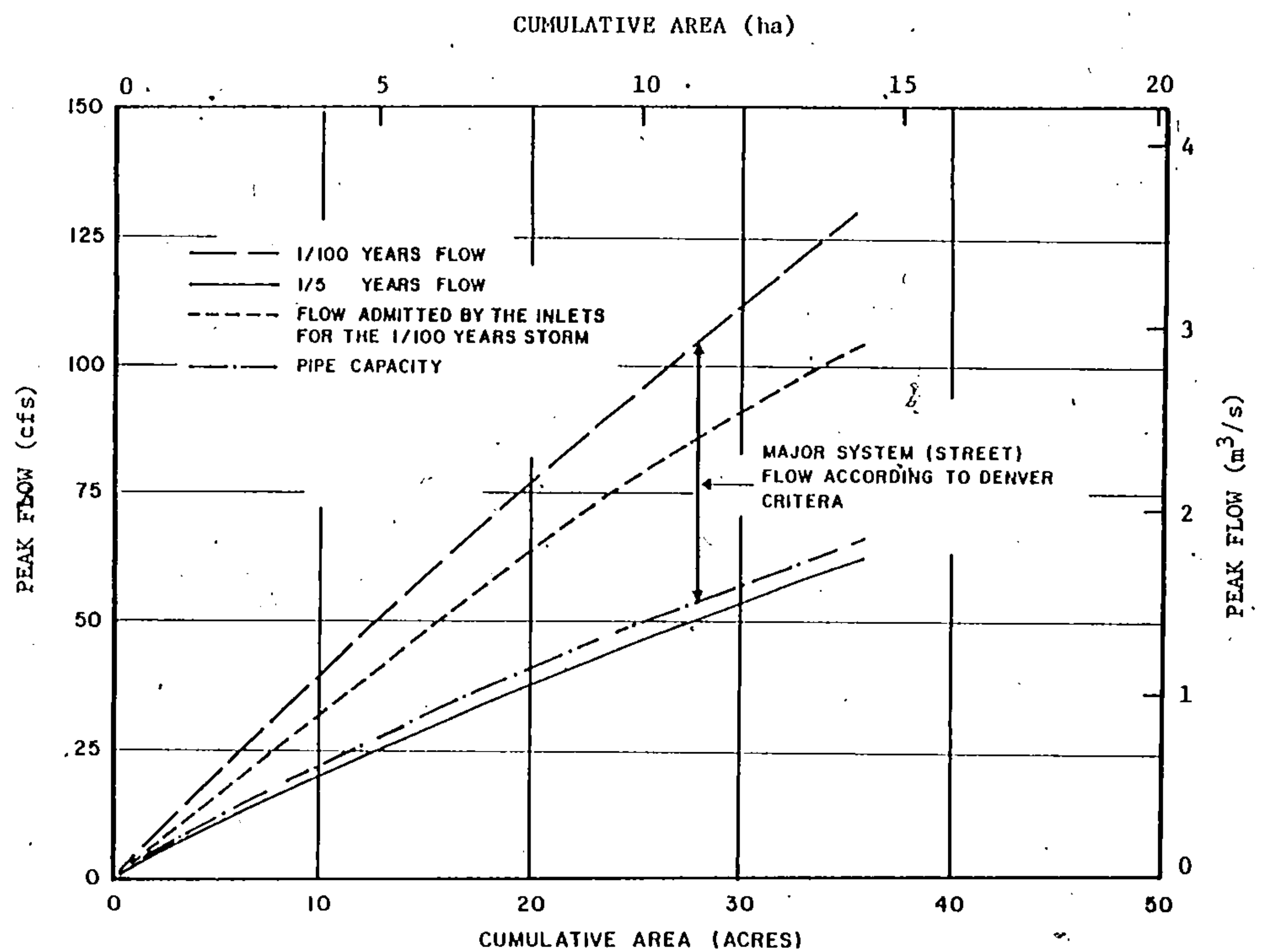


Fig. 2.9 Illustration of the Role of Inlet Control In the Analysis of Dual Drainage

2.5 COMPONENTS AND MAIN FEATURES OF THE PROPOSED MODEL FOR DUAL SYSTEMS

The development of a comprehensive mathematical model (DDSRM) for the design/analysis of dual drainage/storage systems is one of the prime objectives of the present research. The main components of the proposed model are illustrated in Fig. 2.10, which follows the system devised in this chapter. Fig. 2.10 also shows possible flow transfers among various sub-systems. A comparison with Fig. 1.3 shows the difference, in the logical arrangements, between the proposed model and existing urban storm drainage models.

The following are some of the features which will be considered in DDSRM:

1. the street and sewer networks need not be parallel or flow in the same direction;
2. it should apply for the design/analysis of new (dual) drainage systems (subdivisions) considering frequent as well as infrequent rainfall events. For design, the model should determine sewer sizes (based on free surface flow for the minor system design storm runoff), storage volumes, etc. For less frequent storms, the model should have the capability of simulating sewer surcharge;
3. storage facilities should be modelled separately for the minor and for the major system;

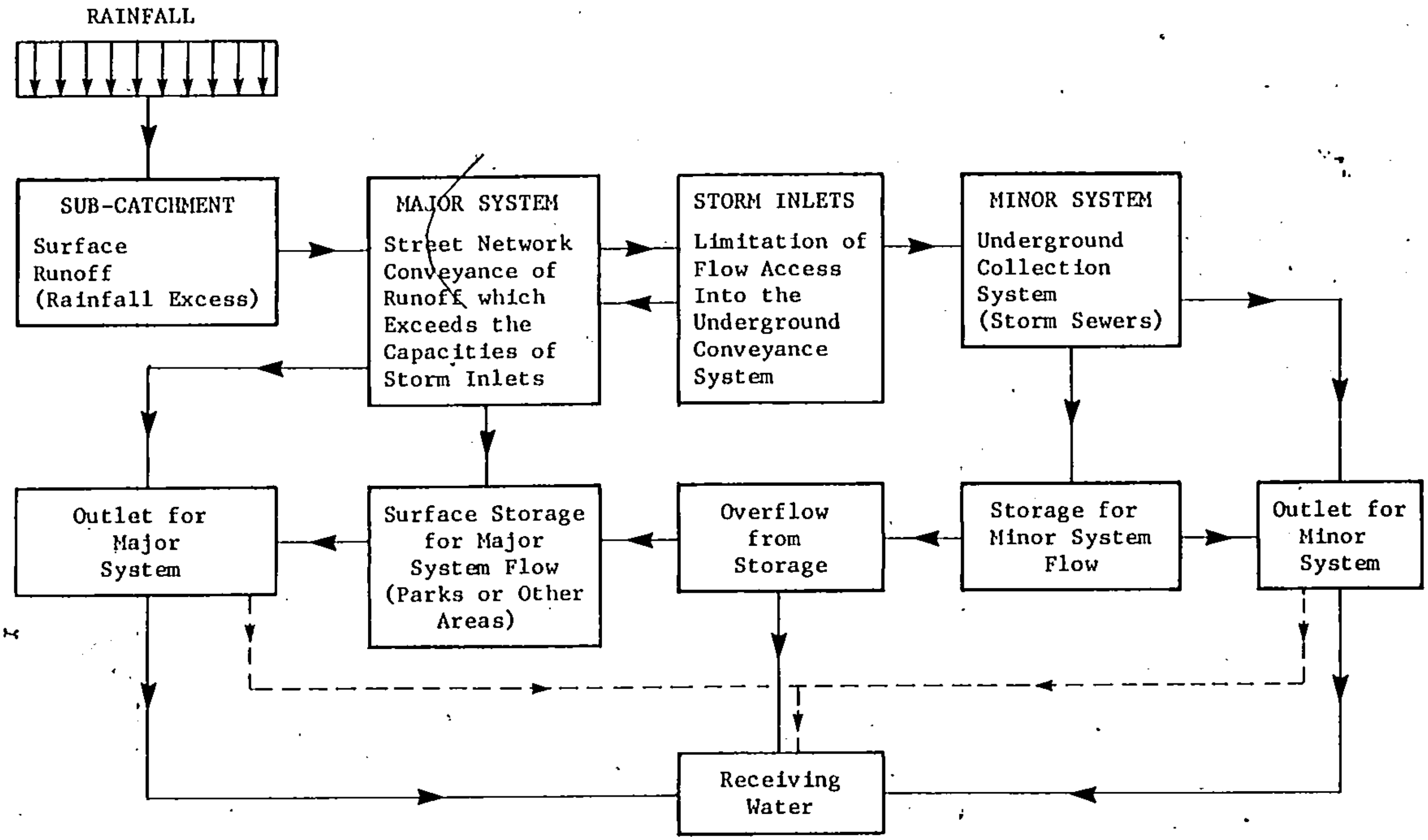


Fig. 2.10 Main Components of the Proposed Model For Dual Systems

4. the model should be written in as general a form as possible and have the flexibility to handle various design conditions;
5. it should determine inlet constriction requirements, if any;
6. it should be computationally efficient and at the same time retain reasonable accuracy; and
7. computer output should provide sufficient information regarding all parameters pertinent to practical design.

A detailed description of the new model, together with its main assumptions, will be given in chapter VI.

Chapter III

REVIEW OF STORM DRAINAGE ROUTING MODELS

3.1 INTRODUCTION

A primary requirement in the mathematical formulation of the proposed "Dual Drainage Simultaneous Routing Model-DDSRM" is the development of sub-models for flow routing in the street and sewer networks, routing through storage facilities, and simulation of sewer surcharge.

This chapter presents a review of the routing theory for unsteady, gradually-varied, free surface flow, numerical techniques for the solution of the complete equations, and various levels of approximate solutions. The applications of flow routing models in storm drainage systems will be discussed together with a review of the state of the art of existing sewer network models.

This review will set the base for the development and validation of the routing sub-models for DDSRM.

3.2 ROUTING THEORY

Flood routing may be defined as a mathematical method (model) for the description of water movement through a channel/conduit or a reservoir. During this process the flood wave is usually attenuated, that is, its peak is lowered and its base extended. It is the task of flood routing methods to mathematically reproduce these effects.

The channel/conduit flow is usually unsteady and non-uniform. The governing equations consist of the equations of conservation of mass and conservation of momentum which are known as Saint Venant equations. The derivation of these equations is described in several textbooks (Chow, 1959; Henderson, 1966):

Continuity Equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \qquad 3.1$$

or

$$A\frac{\partial v}{\partial x} + vB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = q \qquad 3.2$$

Momentum Equation:

$$\frac{1}{A}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + \frac{1}{A}\frac{\partial Q}{\partial t} + \frac{q}{B}\frac{\partial A}{\partial x} - g(S_o - S_f) = -\frac{vq}{A} \qquad 3.3$$

or

$$v\frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} + g\frac{\partial y}{\partial x} - g(S_o - S_f) = -\frac{vq}{A} \qquad 3.4$$

In Eqs. 3.1 through 3.4, the two unknown variables are the flow rate Q (or velocity v) and the flow cross sectional area A (or flow depth y); x is the space coordinate; t is the time coordinate; B is the surface width; q is the lateral inflow per unit length of channel; S_o and S_f are the channel slope and friction slope, respectively, and g is the acceleration due to gravity. These non-linear differential equations are hyperbolic and their solution is subject to two boundary conditions (one upstream and one downstream), and initial state at time $t=t_o$.

The terms on the left hand side of the continuity equation (Eq. 3.2) represent prism storage, wedge storage, and rate of rise respectively. The terms on the left hand side of the momentum equation (Eq. 3.4) represent convective acceleration, local acceleration, pressure force, gravity force and friction force respectively.

The flow resistance can be approximated from normal (steady) flow equations. Thus the friction slope S can be expressed in terms of Manning's n, Chezy's C or Darcy-Weisbach's f. When the channel geometry is known, the only parameter involved in the solution of the Saint Venant equations is the roughness coefficient.

3.3 ROUTING TECHNIQUES

The objective of any flood routing method is to trace the wave modification along a channel/conduit, that is to determine the following functions which are both time and space variant (Fig. 3.1):

flow rate	$Q = Q(x,t)$	
flow depth	$y = y(x,t)$	
flow velocity	$v = v(x,t)$	

Flood routing techniques can be broadly classified into three categories (Fig. 3.2): (1) hydraulic routing, (2) hydrologic routing, and (3) routing methods based on purely empirical or steady flow relations. Hydraulic routing methods can also be divided into (a) complete (dynamic) models, which are based on solutions of the full Saint Venant equations, and (b) approximate hydraulic routing models based on conservation of mass and simplified forms of the momentum equation. Simplifications of the momentum equation are usually obtained by omitting or linearizing some of the terms

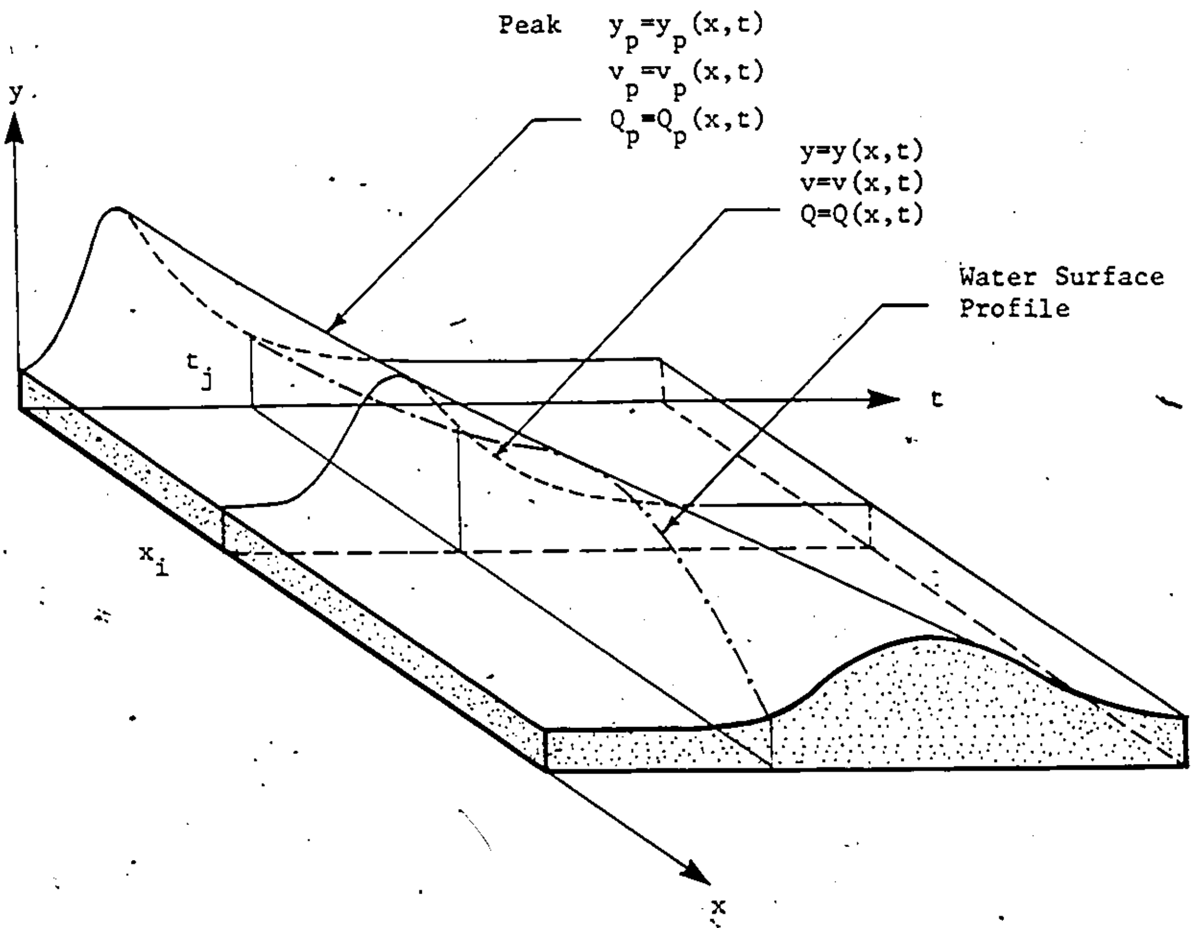


Fig. 3.1 Illustration of Wave Modification Along A Channel

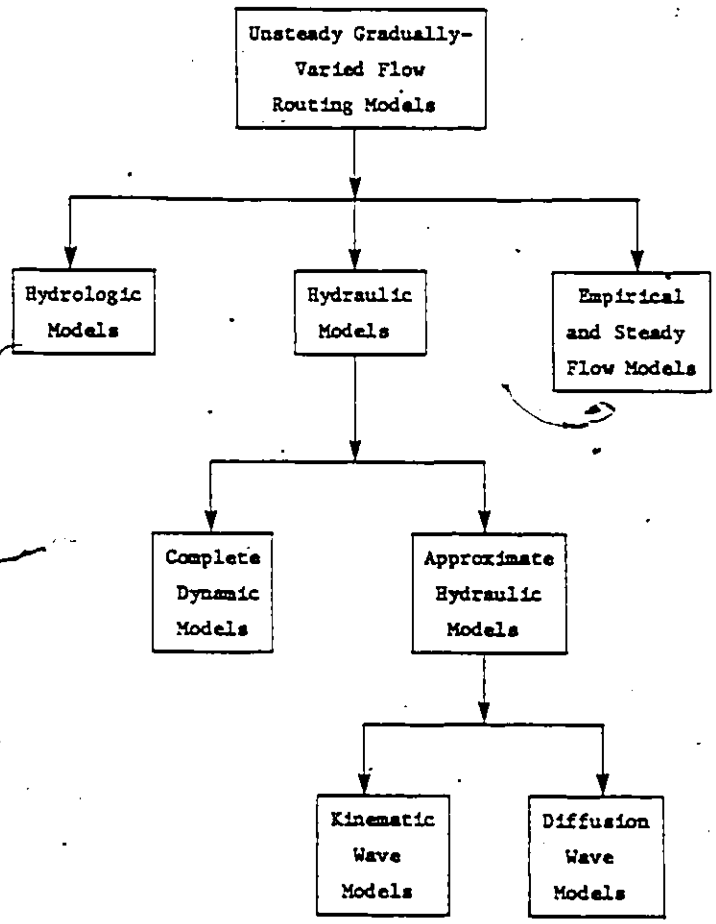


Fig. 3.2 Classification of Unsteady Gradually-Varied Flow Routing Techniques

in Eq. 3.3 (or Eq. 3.4). The most well known of the approximate hydraulic routing models are the kinematic and diffusion wave models.

3.3.1 Hydraulic Routing.

3.3.1.1 Dynamic Wave Models

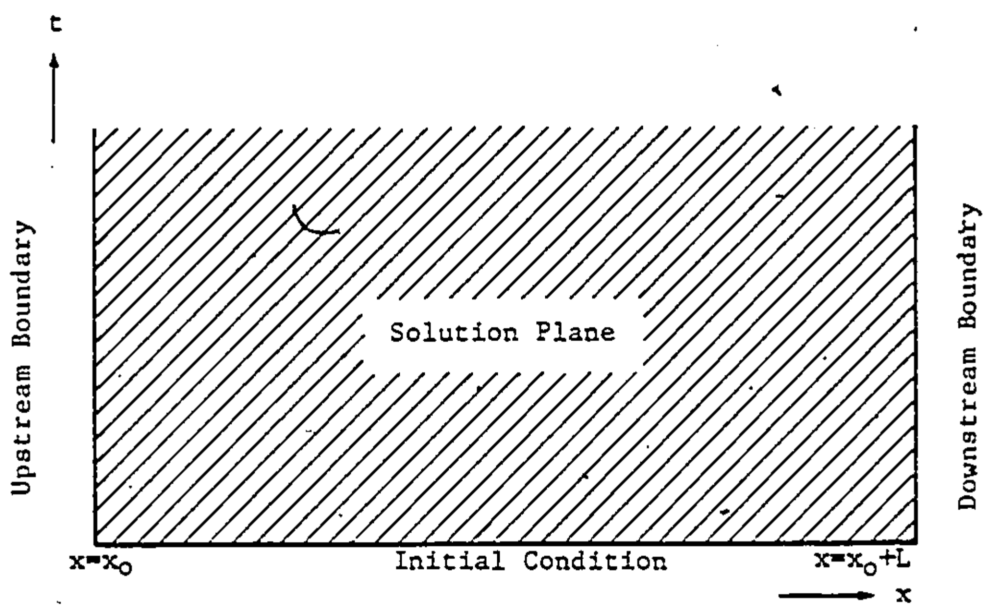


Fig. 3.3 x-t Plane for Numerical Solutions of Saint Venant Equations

Routing methods which are based on the full equations of continuity and momentum are usually referred to as complete (or dynamic) wave models. Since there is no direct mathematical solution for these equations, they have to be solved numerically by replacing the differential equations with approximating difference equations. The solution is therefore obtained at a set of discrete points in the space and time domains (x-t grid)-see Fig. 3.3. The finite-difference solution techniques can be broadly classified into three categories:

1. The method of characteristics which is based upon transforming the two partial differential equations of continuity and momentum into four ordinary differential equations. Neglecting the lateral inflow, the characteristics equations can be expressed by:

$$\frac{dv}{dt} + \sqrt{gA/B}\,\frac{dy}{dt} - g(S_o - S_f) = 0 \qquad 3.5$$

which applies along the positive characteristic curve, defined by

$$\frac{dx}{dt} = \xi^{+} = v + \sqrt{gA/B} \qquad 3.6$$

and

$$\frac{dv}{dt} - \sqrt{gA/B}\,\frac{dy}{dt} - g(S_o - S_f) = 0 \qquad 3.7$$

which applies along the negative characteristic curve, defined by

$$\frac{dx}{dt} = \xi^- = v - \sqrt{gA/B} \qquad 3.8$$

There are basically two numerical techniques for solving the characteristics equations:

a) Finite-difference scheme that solves the characteristics equations for the two dependent parameters Q (or v) and A (or y) at x-t points defined by the intersection points of the (curvilinear) characteristics grid (Fig. 3.4): Amein (1966), Liggett and Woolhiser (1967), Baltzer and Lai (1968), Wylie (1969), Strelkoff (1970), Abbott and Verwey (1970), Wylie (1970), and Sivaloganathan (1978);

b) Explicit finite-differencing of the characteristics equations using a rectangular x-t grid(Fig. 3.5): Streeter and Wylie (1967), Baltzer and Lai (1968), Mozayeny and Song (1969), Yevjevich and Barnes (1970a), Wylie (1970), Jolly and Yevjevich (1974), Kassem (1976) and Sivaloganathan (1974,78,79);

2. explicit finite-difference schemes of the original equations using a rectangular x-t grid: Liggett and Woolhiser (1967), Dronkers (1969), Garrison et al (1969), Strelkoff (1970), Yevjevich and Barnes (1970);

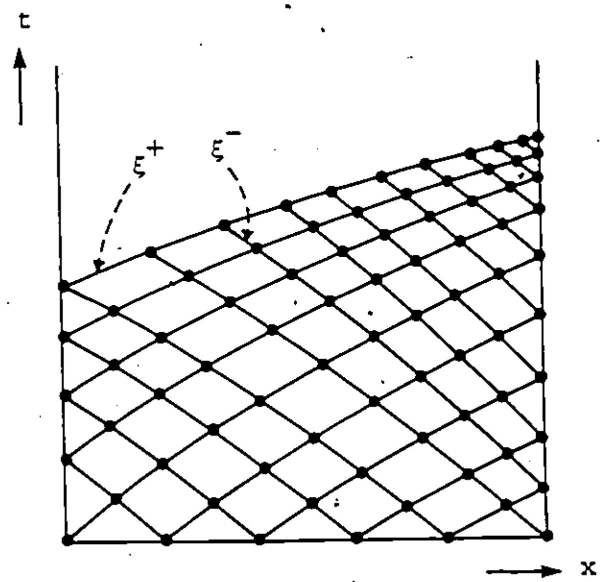


Fig. 3.4 Characteristics Grid

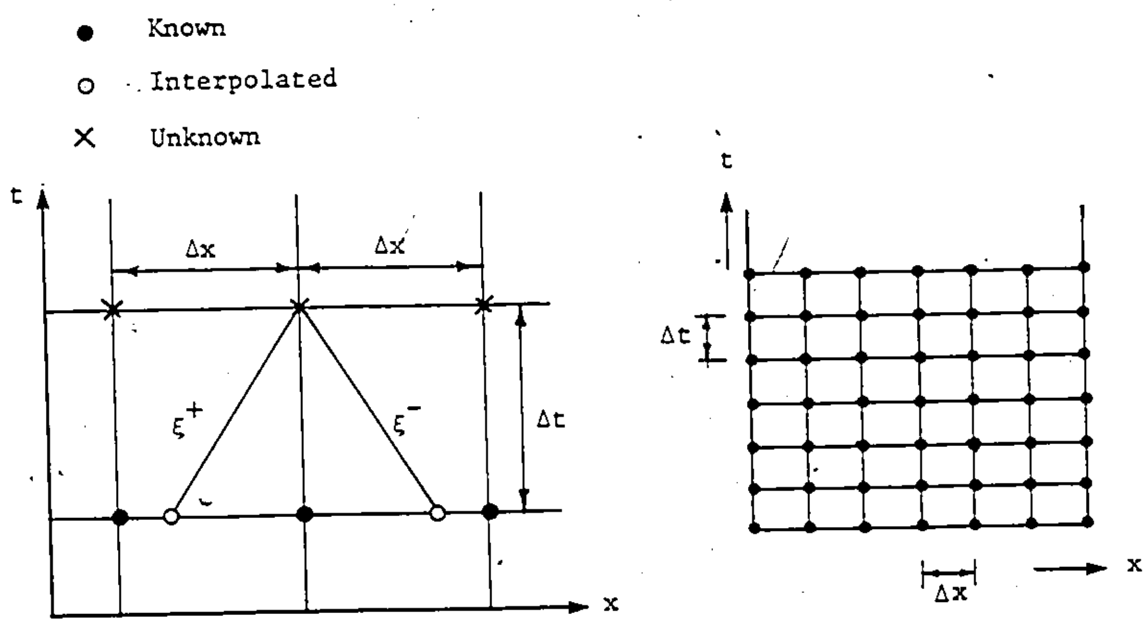


Fig. 3.5 Fixed Grid for Solution By the Method of Characteristics

3. implicit finite-difference schemes of the original equations using a rectangular x-t grid: Abbott and Ionescu (1967), Baltzer and Lai (1968), Dronkers (1969), Strelkoff (1970), Amein and Fang (1970), Fread (1974), Amein and Chu (1975), Ponce et al (1978a).

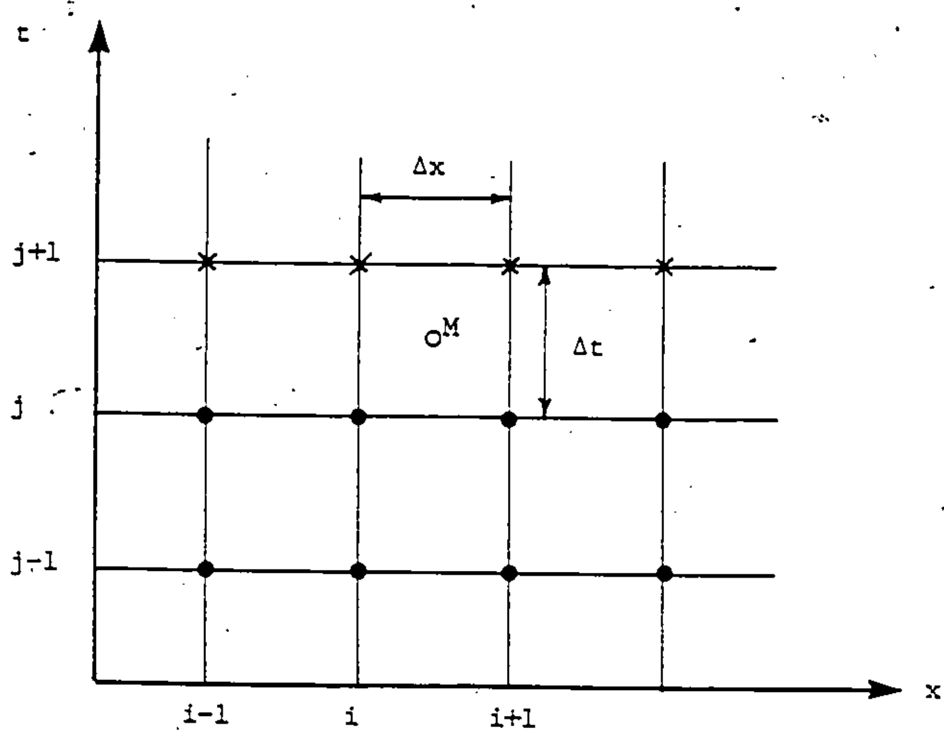


Fig. 3.6 Computational Grid of the Four-Point Implicit Scheme

Among the implicit schemes, the four-point scheme has received particular attention by several investigators: Amein and Fang (1970), Fread (1974), Price (1974), Amein and

Chu (1975), and Ponce et al (1978a) have all reported the advantages of the four-point implicit scheme over other schemes.

In the four-point implicit difference scheme the time derivatives are approximated by a forward difference operator centered between the ith and i+1 points along the x-axis (Fig. 3.6):

$$\frac{\partial \eta}{\partial t} = \frac{(\eta_{i+1}^{j+1} + \eta_i^{j+1}) - (\eta_{i+1}^{j} + \eta_i^{j})}{2\ \Delta t} \qquad 3.9$$

where η represents the variable. The space derivatives are approximated by a forward difference operator positioned between the jth and j+1 points according to a weighting factor θ

$$\frac{\partial \eta}{\partial x} = \theta \left(\frac{\eta_{i+1}^{j+1} - \eta_i^{j+1}}{\Delta x} \right) + (1-\theta)\left(\frac{\eta_{i+1}^{j} - \eta_i^{j}}{\Delta x} \right) \qquad 3.10$$

The average value of the function η is represented by

$$\eta = \theta \left(\frac{\eta_i^{j+1} + \eta_{i+1}^{j+1}}{2} \right) + (1-\theta)\left(\frac{\eta_i^{j} + \eta_{i+1}^{j}}{2} \right) \qquad 3.11$$

Note that θ = 1/2 yields a centered four-point implicit scheme.

The four point implicit finite difference formulation of the continuity and momentum equations of Saint Venant results in a system of two nonlinear algebraic equations in four unknowns. Two unknowns are common at any two contiguous rectangular grids. Combination of all rectangular grids provides 2(N-1) equations in 2N unknowns. Two additional equations are therefore required, which are represented by the upstream and downstream boundary conditions.

The characteristics and explicit schemes are relatively simple when compared to the implicit schemes. However, a restriction is imposed on the size of the computational time step. For numerical stability of explicit schemes

$$\Delta t \leq \frac{\Delta x_m}{v_m + \sqrt{gA_m/B_m}} \qquad 3.12$$

known as the Courant Stability Criterion (Strelkoff, 1970), where B_m and A_m are the width of water surface and cross-sectional area of flow in the m^{th} cross section, and Δx_m is the m^{th} finite difference distance interval. Explicit schemes also require the use of equal distance intervals, and consequently small time steps.

In contrast to explicit schemes, the implicit schemes are stable and independent of the size of the time and distance intervals (Fread, 1974), and thus permit numerical solution over large time steps. However, they require the solution of large sets of equations simultaneously at each time step.

A solution of the complete equations of continuity and momentum is expected to produce the most accurate and reliable results. Good agreements between routed hydrographs using complete (dynamic) models, computed by the method of characteristics (Amein and Fang, 1970; Baltzer and Lai, 1968), implicit finite- difference schemes (Amein and Chu 1975, Baltzer and Lai, 1968), and explicit finite-difference schemes (Amein and Fang, 1970) are obtained in studies conducted in natural channels. However, due to the complexity inherent in the numerical solution of the complete equations, simplified routing methods have been developed and have received particular interest among hydrologists especially in connection with rainfall-runoff models.

3.3.1.2 Approximate Hydraulic Routing Models

The kinematic and diffusion wave models are two well known approximate solutions of the Saint Venant equations. The basic principle behind these models and similar approximate models can be demonstrated by following Henderson's (1966) approach of normalizing the momentum equation with a steady uniform discharge. After neglecting the lateral inflow and rearrangement of the normalized form of Eq. 3.4, the following equation is obtained

$$Q = Q_N \left[1 - \frac{1}{S_o} \left(\frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} \right) \right]^{\frac{1}{2}} \quad 3.13$$

Kinematic Wave (first term: 1)

Diffusion Wave (through $\frac{\partial y}{\partial x}$)

Complete Dynamic Wave

where Q_N is the normal flow discharge. Equation 3.13 represents a general expression for a rating curve during the passage of a flood as shown in Fig. 3.7. Note that the rating curve is not only a function of channel geometry but also depends on the wave characteristics. The "looped" rating curve is caused by the secondary "dynamic" terms in the momentum equation: pressure force ($\partial y / \partial x$), convective acceleration ($\partial v / \partial x$), and local acceleration ($\partial v / \partial t$). These terms are illustrated in Fig. 3.7. The width of the loop and hence the reliability of the simplified model will depend on the relative importance of the dynamic terms, as compared to the channel slope S_o.

A. Kinematic Wave Model

If the dynamic terms in the momentum equation are sufficiently small as compared to the gravity force (S_o), the momentum equation can be approximated by the steady uniform flow equation

$$Q = Q_N = a \cdot A^{\beta} \quad 3.14$$

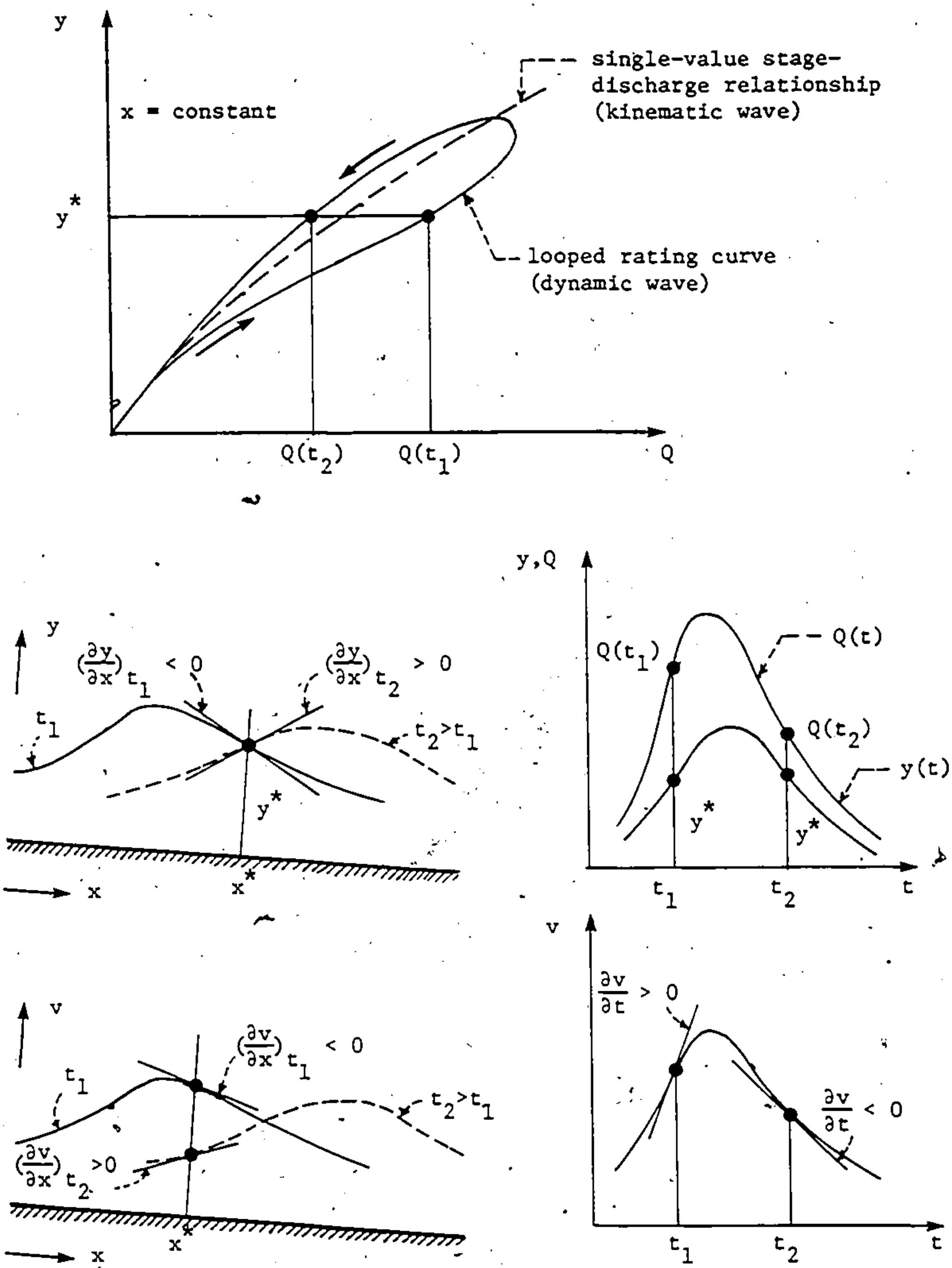


Fig. 3.7 Illustration of the Dynamic Terms in the Momentum Equation - Single and Double Valued Rating Curves .

where a and β are the model parameters whose values depend on the the shape and roughness of the channel. This implies that

$$S_f = S_o \tag{3.15}$$

which is the basis for the kinematic wave models. Assuming a wide channel, a kinematic model would describe unsteady flow conditions for which the discharge can be expressed as a single-valued function of flow depth as illustrated in Fig. 3.7.

Assuming no lateral inflow (q=0), Eq. 3.14 together with the continuity equation (Eq. 3.1) can be combined (Lighthill and Whitham, 1955) to give a general expression for a kinematic wave model

$$\frac{1}{c}\frac{\partial Q}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{3.16}$$

c is the wave celerity or kinematic wave speed which can be evaluated from the Kleitz-Seddon law:

$$c = \left.\frac{dQ}{dA}\right|_{x\,=\,const.} = \left.\frac{1}{B}\frac{dQ}{dy}\right|_{x\,=\,const.} \tag{3.17}$$

Eq. 3.16 is non linear since c is also a function of discharge (Q). If c is considered as a constant this constitutes a linear kinematic model.

The same techniques described previously for the solution of the full equations can still be used for the numerical solution of the kinematic model. The stability criterion for explicit schemes of the kinematic wave is also reduced to

$$\Delta t \leqslant \frac{\Delta x}{c} \tag{3.18}$$

A kinematic wave has only one characteristic curve (forward characteristic). While this results in simplifying the solution by omitting the need for a downstream boundary condition, a kinematic model cannot produce upstream propagation of the wave (backwater effect).

The basic theoretical work on kinematic wave is attributed to Lighthill and Whitham (1955). Much of the theory and finite difference solution techniques have been given by Kibler and Woolhiser (1970), Smith and Woolhiser (1971), Schaake (1971), Woolhiser et al (1971), Overton (1972), Overton and Meadows (1976), Resource Analysis, Inc. (1975), Huang (1978), Ponce et al (1978b), and others.

Theoretically an exact solution of the kinematic wave equation does not allow any wave attenuation to be present, since, by definition, a kinematic model only describes the translation phenomenon of the wave without distortion. The wave attenuation observed in finite-difference formulations of the kinematic wave is caused by numerical diffusion, the amount of which is directly proportional to the size of the computational grid (Ponce and Yabusaki, 1979).

B. Diffusion Wave Model

The diffusion model (or diffusive kinematic wave) offers some improvements over the (non-diffusive) kinematic model by including the pressure term in the momentum equation. Thus for no lateral inflow, the momentum equation (Eq. 3.4) is approximated by

$$\frac{\partial y}{\partial x} = S_o - S_f \qquad 3.19$$

which when combined with the continuity equation gives the diffusion wave equation

$$\frac{\partial Q}{\partial t} + c\,\frac{\partial Q}{\partial x} = \varepsilon\,\frac{\partial^2 Q}{\partial x^2} \qquad 3.20$$

where c is the celerity of discharge Q, and ε is the diffusion coefficient, which is evaluated from

$$\varepsilon = \frac{Q}{2BS_o} \qquad 3.21$$

The diffusion wave is kinematic in nature since it does not include the inertia terms. However, unlike the non-diffusive kinematic wave it has a mechanism for wave attenuation embodied in the diffusion coefficient.

3.3.2 Hydrologic Routing

Hydrologic routing methods are all founded upon the storage balance equation

$$\frac{dS}{dt} = Q(x,t) - Q(x + \Delta x,t) \qquad 3.22$$

in which dS/dt is the rate of change of water volume S in a reach Δx length due to the difference in the volumetric fluxes into and out of it: $Q(x,t)$ and $Q(x + \Delta x,t)$, respectively.

Routing methods which are based on storage considerations when applied to regular channels coincide with the kinematic approach since they too neglect the dynamic effects (Silvic, 1969). For instance the linear storage function (Eq. 3.22) can be considered as a linear approximation of the continuity equation and from a hydraulic viewpoint routing methods using this approach can be termed as "linear kinematic models" (Yen et al, 1976).

There are several routing techniques based on storage considerations. Many of these models include coefficients to approximate the complex relation between storage in a reach and flow characteristics such as inflow, outflow, stage, or water surface slope and are known collectively as the coefficient methods. Several of the coefficient methods have been presented by Miller and Cunge (1975).

It is important to note here that one can distinguish between hydrologic models and hydraulic models based on the method utilized in determining the model parameters (Weinmann and Laurenson, 1979). When these parameters are determined from hydrologic observations, the method can be considered as a "hydrologic model". On the other hand, if the parameters are related to the physical and hydraulic characteristics of the channel the routing method can be referred to as a "hydraulic model".

One of the best-known and most widely used of the hydrologic routing models is the Muskingum method. The method is based on a one-to-one relationship between storage S and a combination of inflow, $Q(x,t)$ and outflow, $Q(x + \Delta x,t)$:

$$S = K\,[\theta Q(x,t) - (1-\theta)\,Q(x + \Delta x,t)] \qquad 3.23$$

where K and θ are the model parameters, and Δx is the reach length (space interval).

The Muskingum model is usually expressed in the well known form:

$$Q_{i+1}^{j+1} = c_1 Q_i^j + c_2 Q_i^{j+1} + c_3 Q_{i+1}^j \quad 3.24$$

$$c_1 = \frac{K\theta + \Delta t/2}{K(1-\theta) + \Delta t/2} \quad 3.24a$$

$$c_2 = \frac{-K\theta + \Delta t/2}{K(1-\theta) + \Delta t/2} \quad 3.24b$$

$$c_3 = \frac{K(1-\theta) - \Delta t/2}{K(1-\theta) + \Delta t/2} \quad 3.24c$$

$$c_1 + c_2 + c_3 = 1 \quad 3.24d$$

In Eq. 3.24, subscripts i and i+1 refer to the beginning and end of the reach, Δx length, respectively; superscripts j and j+1 refer to time t and $t + \Delta t$, respectively.

The model parameters K and θ are assumed constants. In practice they are usually determined from past flood records, using a trial and error procedure, and utilizing the linear relation between storage and flows (Eq. 3.23).

3.3.3 Some Special Approximate Routing Models

From the analysis of a finite difference formulation of the kinematic wave equations, Cunge (1969) obtained a solution which can be expressed in a form which is identical to the coefficients equation of the Muskingum model (Eq. 3.24). By taking K and θ as constants Cunge further showed that if the parameter θ is evaluated from

$$\theta = \frac{1}{2} - \frac{K\varepsilon}{(\Delta x)^2} \qquad 3.25$$

where ε is the diffusion coefficient, defined from Eq. 3.21, his solution and hence the Muskingum model can be considered as an approximate solution of the diffusion wave model (Eq. 3.20). He also indicated that the wave attenuation in this case closely approximates the Saint Venant equations. The expression for θ (Eq. 3.25) was obtained from a Taylor series expansion of $Q(x_i, t_i)$ in the finite difference solution of Eq. 3.16.

The analogy between the Muskingum model and the diffusion wave model was also demonstrated by Koussis (1978), who combined Eqs. 3.22 and 3.23 into one equation of the diffusion-convection type:

$$\frac{\partial Q}{\partial t} + \frac{\Delta x}{K}\frac{\partial Q}{\partial x} = [\Delta x(1-\theta)c - \frac{1}{2}\frac{(\Delta x)^2}{K}]\frac{\partial^2 Q}{\partial x^2} \qquad 3.26$$

By taking K as

$$K = \frac{\Delta x}{c}$$

and comparing Eq. 3.26 with the diffusion wave equation (Eq. 3.20) it can be seen that the parameter θ will be defined by Eq. 3.25.

K can be interpreted as the propagation time of discharge. The weighting factor θ is now related to the hylic and physical characteristics of the channel. In order for the wave to attenuate, the diffusion coefficient has to be positive. Therefore it follows from Eq. 3.25 that $\theta \leq 0.5$. The physical concept behind Eq. 3.23 also requires that $\theta \geq 0$. Thus

$$0 \leq \theta \leq 0.5 \qquad 3.27$$

A more exact solution of Eq. 3.16 was proposed by Koussis (1976), which can be expressed by a coefficient equation of the Muskingum type (Eq. 3.24), with the model parameters calculated from

$$c_1 = \frac{K}{\Delta t}\,(1-e^{-\frac{\Delta t}{K(1-\theta)}}) - e^{-\frac{\Delta t}{K(1-\theta)}} \qquad 3.28a$$

$$c_2 = 1 - \frac{K}{\Delta t}\,(1-e^{-\frac{\Delta t}{K(1-\theta)}}) \qquad 3.28b$$

$$c_3 = e^{-\frac{\Delta t}{K(1-\theta)}} \qquad 3.28c$$

$$c_1 + c_2 + c_3 = 1 \qquad 3.28d$$

Koussis (1976) also demonstrated that this model is a second order approximation of the diffusion wave model if θ is estimated from

$$\theta = 1 - \frac{r}{\ln(\frac{\lambda+1+r}{\lambda+1-r})} ; \quad 0 \leq \theta \leq 0.5 \qquad 3.29$$

$$r = c\,\frac{\Delta t}{\Delta x}$$

$$\lambda = \frac{Q}{BS_o c\Delta x}$$

The difference between this model and that of Cunge is the expressions used for the calculation of the routing coefficients c_1, c_2 and c_3 and the parameter θ. The value of θ estimated from Eq. 3.25, however, differs only slightly from the value calculated from Eq. 3.29 (Weinmann and Laurenson, 1979), since Eq. 3.29 can be approximated by

$$\theta = \frac{1}{2}(1-\lambda) \tag{3.30}$$

which is identical to Eq. 3.25 if we substitute for ε from Eq. 3.21. Thus the simpler expression for by Eq. 3.25 is adequate.

Note that Eqs. 3.28a through 3.28d reduce to the model proposed by Kalinin and Miljukov (Miller and Cunge, 1975), with θ =0 and

$$\tau = K = \frac{\Delta x}{c} \tag{3.31}$$

τ is assumed constant and represents the time of propagation of discharge. The Kalinin-Miljukov model requires the use of a specific reach length $\Delta x = L$, associated with θ =0. The use of θ =0 eliminates the physical diffusion from the model. It can be shown (Cunge, 1969) that θ =0 will correspond to fully concentrated or reservoir type action, that is maximum attenuation.

The Cunge model has recently received particular attention in the literature. It has been favourably reported in studies conducted in natural channels (Koussis, 1975,1978; Ponce, 1979; Ponce and Yevjevich, 1978). Although the model parameters have been derived under the assumption of constant parameters it can be applied with variable parameters. Applications of Cunge model, to date, whether with constant or variable parameters have aimed at reproducing a measured flow event (Koussis, 1978; Ponce, 1979).

The Cunge model appears to have significant advantages over direct solutions of the kinematic and diffusion equations: (1) it allows for the attenuation effects that the kinematic model cannot include, and (2) unlike the diffusion wave model it does not require the specification of a downstream boundary condition and thus the computational efforts are substantially reduced.

The Cunge model is best suited for applications along well defined channels where the model parameters can be easily estimated (Price, 1979). The accuracy of the numerical solutions of the Cunge model with those of the full Saint Venant equations remains to be compared (Price, 1980).

3.4 APPLICABILITY OF KINEMATIC AND DIFFUSION MODELS

As indicated above, all the approximate hydraulic and hydrologic routing methods are essentially variations of the kinematic and diffusion wave analogies. The approximate methods neglect the inertia effects. Their accuracy will therefore depend on the relative importance of the inertia terms in the momentum equation.

Miller and Cunge (1975) presented a review of several criteria for the applicability of the kinematic flow theory: Using a dimensionless form of the kinematic equations (Woolhiser and Liggett, 1967), two parameters appear which have significance in determining when the kinematic flow theory is valid. These parameters are the Froude number

$$F_o = \frac{v_o}{\sqrt{g\ H_o}} \qquad 3.32a$$

and the kinematic flow number

$$k = \frac{S_o\ L_o}{F_o^{\ 2}\ H_o} \qquad 3.32b$$

where v_o is the steady normal velocity at the end of some plane or channel length L_o, H_o is the corresponding normal depth, and S_o is the plane or channel slope. It was found that (Liggett and Woolhiser, 1967) when flows are supercritical or when the kinematic flow number is large, the Chezy kinematic model is an accurate model. Another criterion for the validity of the kinematic model which involves the Froude number was obtained by Lighthill and Whitham (1955). By setting the kinematic and dynamic wave velocities equal, the Froude number will be equal to 3/2 or 2, depending on whether the Manning's formula or Chezy's formula are used for the resistance. These Froude numbers determine the conditions at which dynamic and kinematic waves are of equal importance.

Henderson (Miller and Cunge, 1975, pp. 198) used the bottom slope as the criterion for the applicability of kinematic, diffusion and dynamic wave models. He classified the slopes as gentle, intermediate and steep. According to Henderson classification, kinematic models apply to steep slopes, dynamic models to gentle slopes, and diffusion models are applicable to intermediate slopes.

A criterion for the applicability of kinematic and diffusion wave models accounting for the wave period as well as the channel slope was recently obtained by Ponce et al (1978). The range of applicability of these approximate models was analysed theoretically using a linearized form of the Saint Venant equations. The propagation characteristics of sinusoidal perturbations to the steady uniform flow for the kinematic, diffusion and dynamic models were compared. An inequality criterion was determined for the applicability of kinematic and diffusion wave models, as follows:

$$\frac{T_o S_o v_o}{F_o y_o} = T_o S_o \sqrt{\frac{g}{y_o}} > \xi \qquad 3.33$$

where T_o is the wave period; y_o is the uniform flow depth; v_o, F_o and S_o are as defined earlier. It was determined that the kinematic and diffusion wave models should be applicable where ξ equals 171 and 30, respectively.

Eq. 3.33 indicates that the wave period (that is hydrograph duration) and bed slope are the important physical parameters for the applicability of the approximate routing models. The diffusion wave model also has a wider range of applications than the kinematic wave model.

It should be noted that the criterion of Eq. 3.33 was obtained from an analytical solution. The limiting values of ξ were based on 5% error in peak flow after one period of propagation. The application of the kinematic and diffusion wave models may, however, involve numerical errors introduced by the approximating difference equations.

3.5 FLOW ROUTING AND SEWER NETWORKS

The selection of the most suitable flood routing technique is a key aspect in building an efficient and reliable sewer network model. The review presented above dealt with the state of the art of routing techniques for free-surface, gradually varied and unsteady flow in open channels. In this section we will discuss applications of flow routing techniques in sewer networks.

3.5.1 Basic Considerations

The following remarks should be considered for the selection or development of routing models for sewer networks systems:

1. Most storm sewer networks are dendritic (tree type) in pattern. However, they may also form closed-loop networks. In the latter case the routing model should account for the mutual dynamic effects of all sewers at every time step in the calculation. Under

these circumstances, a representative sewer network model should be based on dynamic routing since simplified routing methods can be applied only in a sequential manner.

2. A sewer network model is usually developed with one of two objectives: (a) design of new systems, or (b) analysis/ evaluation of existing systems. In the case of design, the main objectives of the model are usually to determine sewer sizes, storage volumes, etc. The common approach in sizing storm sewers is to select the smallest commercial pipe diameter which can carry the design flow without surcharge. It is therefore desirable that the routing method provides accurate prediction of the peak flows, which is reflected by the model capability of representing wave attenuation and translation properties. Conservation of mass is important for the design of storage facilities. The design of storm sewers is usually based on the assumption of straight water surfaces. Where the sizes of conduits are sufficiently large or other needs for a higher degree of accuracy exist, backwater or drawdown curves should be calculated (Wright-McLaughlin, 1969).

3. If a sewer network model is developed for the evaluation of existing systems, a different problem arises,

that is, the possibility that the sewer capacities are exceeded. Under these circumstances, the model should have a capability of simulating surcharged (pressurized) flow conditions. Because sewer surcharge usually occurs only in a part of the system, this may have an effect on flow conditions in the upstream sewers. Therefore modelling of surcharged flow conditions should be considered in conjunction with a dynamic routing model.

4. In the case of sewer design models, dynamic routing is not appropriate since this would require the specification of a downstream boundary condition, and hence a prior knowledge of sewer sizes. The use of dynamic routing in this case can be carried out only in the frame of a trial and error procedure which is expected to be extremely expensive in terms of computational time. An example of computer time required for sewer design utilizing dynamic models has been reported in an application of the Illinois Storm Sewer System Simulation "ISS" model, which uses the method of characteristics (Yen, 1973). In this application a conservative estimate of the computer time required for the design of a sewer system composed of 54 pipes was one hour!

5. The analysis of sewer network systems involves, besides flow routing, a number of special problems such as energy losses at the junctions (for both free surface and surcharged flow conditions), and the possibility of the occurrence of backwater caused by the conditions at the junctions, or in the case of the existence of tidal conditions at the outlet from the sewer system, and similar situations. Under design conditions, the pipe sizes are usually selected such that they carry the design flows at, or close to, their full flow capacities. Consequently, backwater effects, if any, would, most probably, result in the occurrence of pressurized (surcharged) flow conditions. Under these circumstances, backwater effects are not expected to have a significant effect on the flows, since the remaining storage in the pipes, which is the main reason for wave attenuation, would be insignificant. If a sewer network system is designed, neglecting the backwater effects (e.g. using a kinematic wave routing model), these effects can be analysed by using a dynamic routing model, which, however, should have the capability of simulating surcharged flow conditions.

3.5.2 State of the Art of Sewer Network Models

Table 3.I presents some of the currently available sewer network models and the routing technique used in each. An inspection of the table shows that a wide variety of routing methods has been utilized. The simplest of all routing techniques used in storm sewer design is the steady state time-lag methods, such as: (1) hydrograph shifting by the flow time without distortion, used in the University of Cincinnati Urban Runoff Model - UCURM (Papadakis and Preul 1972), and in the Queen's University Urban Runoff Model - QUURM (Watt, 1975); and (2) moving average time-lag method (Harris, 1970). The use of such simplistic assumptions is usually justified by the uncertainty in the estimation of surface runoff.

At the other extreme, some of the models apply sophisticated routing techniques based on solutions of the full dynamic equations (see table 3.1). This is usually justified by: (1) the recent improvement in runoff prediction techniques (Yevjevich, 1975); (2) the development in the size and capacity of computers which makes such solutions possible (Yen, 1973) and (c) the desire to improve the design, in light of the billions of dollars spent annually in storm drainage systems.

It is well known, according to the present state of the art, that dynamic routing models cannot simulate supercriti-

Table 3.1 - Routing Methods in Selected Urban Storm Drainage Models

Model	Routing Method	References
Los Angeles Hydrograph Method	storage routing	Hicks (1944)
Chicago Hydrograph Method	storage routing	Tholin and Keifer (1960), Yen and Sevuk (1975)
TRRL	reservoir routing lagged by time of travel	Watkins (1962,63)
HSP	diffusion wave	Crawford (1971)
EPA-SWMM	non-linear kinematic wave corrected for dynamic effects	Metcalfe & Eddy, Inc. et al. (1971), Huber et al (1975), Huber et al (1980)
UCURM	time-lag	Papadakis and Preul (1972)
HVM	dynamic (4-pt. implicit scheme)	Klym et al (1972)
MIT	kinematic wave	Leclerc and Shaake (1973)
CAREDAS	dynamic (implicit)	Sogreah (1973)
ISS	dynamic (method of characteristics)	Sevuk et al (1973), Yen and Sevuk (1975)
ILLUDAS	storage routing	Terstriep and Stall (1974)
WRE-TRANSPORT	dynamic (explicit)	Kibler et al (1974) Kassem and Roesner (1979) Roesner, Kassem and Wisner (1980)
QUURM	time-lag	Watt (1975)
SIIS	dynamic (6-pt. implicit)	Hoff-Clausen et al (1981)
-	dynamic (method of characteristics)	Joliffe (1981)

cal flow conditions, unless special and inefficient numerical solution procedures are followed (Fread, 1981). This limitation is known in existing sewer network models such as the ISS model (Yen, 1973) and the German HVM model (Wichman, 1980). In sewer network systems, where supercritical flow is expected to occur, this fact represents a severe limitation of the use of dynamic routing models in practical applications. It should be noted, however, that the EXTRAN model, described briefly in Chapter V, does not suffer from numerical instability problems in the case of supercritical flow.

There have been some studies comparing various numerical methods for flow routing along a pipe (Yevjevich and Barnes, 1970a; Yen, 1973 and Bettess and Price, 1976). These studies compare numerical solutions of the complete equations, kinematic and diffusion wave models. The only experimental study for the evaluation of flow routing along a pipe seems to be the one conducted by Yevjevich and Barnes (1970b) on a prototype pipe 822 ft long, 8.0 ft diameter. They showed that a numerical solution of the characteristics equations could describe the wave propagation along a pipe for a wide variety of flow conditions with reasonable accuracy. None of the available studies were conducted, however, on sewer network systems.

Nevertheless, in most existing urban storm drainage models, it seems that the merits of using a particular routing method have not been analyzed since model validations have almost always been based on comparing the outlet hydrographs with measurements. This process mainly involves calibration of the hydrologic parameters and excludes the effect of the routing technique. It seems probable therefore that complex routing methods have been utilized without justifying the increased computational cost by, for example, improvements in the design. On the other hand, many of the approximate routing methods have been used without adequate analysis of their limitations.

3.5.3 Sewer Surcharge

Flood routing theory has been developed to deal with unsteady flow problems where the flow always has free surface (open channels). In sewer network systems, however, pressurized (surcharged) flow can occur if the sewer capacities are exceeded. Most existing sewer network models such as ILLUDAS (Terstriep and Stall, 1974); SWMM (Metcalfe & Eddy, Inc., 1971) and ISS (Yen, 1973) avoid the calculation of sewer surcharge, by means of ad hoc assumptions. In many cases, however, sewer surcharge is a reality that should be analyzed.

At the present time the only sewer network models which are known to have a capability of simulating surcharged conditions are: (1) EXTRAN model (Kibler et al, 1974), (2) the German HVM model (Klym et al, 1972) and the French CAREDAS model (Sogreah, 1973). The last two models are proprietory, and the methodologies they use for simulating surcharge are not known. Recently, simulation of sewer surcharge has received interest by several investigators (Sjoberg, 1981; Hoff-Clausen, 1981; Martin and King, 1981; Price, 1981; Pansic and Yen, 1981 and Toyokuni, 1981).

The simulation of free surface/surcharged flow in storm sewers systems is a complex problem which requires special techniques. It should also be noted that there have been no studies on the evaluation of methodologies for the simulation of surcharge in sewer systems.

The transition from free surface to surcharged flow, and vice versa, may be accompanied by a transient pressure (hydraulic instability) similar to water hammer waves in forcemains (Hamam and McCorqudale, 1980). The occurrence of a hydraulic jump is another source of hydraulic (physical) instability. The problem of hydraulic instabilities of storm sewer flows is disussed elsewhere (Yen, 1978b) and is beyond the scope of the present study.

3.5.4 Energy Losses at Sewer Junctions

In sewer network systems, junction manholes are typically placed where two or more pipes join together, or where the pipe grade, size or alignment change. The design of sewers is based on the equations of mass conservation and energy consevation. In network systems, the energy conservation would require consideration of not only the skin friction losses, but also the "minor" losses occuring at the junctions, bends, etc.

Energy losses at the junctions is a phenomenon that involves numerous variables, such as the number of adjoining pipes, the angles of intersection, the shape and slope of the conduits/pipes, the direction and discharges of flow, the rounding of the corners at the junctions, etc. (Chow, 1959).

The usual method of expressing energy losses at sewer junctions, h_e, is in terms of a constant, K, times the velocity head of the conduit in question

$$h_e = K\ (v^2/2g) \qquad 3.34$$

A difficulty in design is the determination of the value of K.

Chow (1959) discussed energy losses at open channel junctions on the basis of works by Taylor (1944) and Bowers

(1950). He concluded that the flow through a junction was such a complicated problem that its generalization was not possible and the best solution would be found through a model study.

A number of studies dealing with energy losses at sewer junctions have been reported in the literature. A recent review of these studies (Masalek, 1981) indicates that experimental energy losses were reported only for a few selected junction geometries and that the majority of the studies were conducted for junctions flowing under surcharge. It seems that the most extensive tests of energy losses at surcharged junctions are those conducted by Sangester et al (1959). Other studies on junction energy losses include those by Acker (1959), Yevjevich and Barnes (1970c), Townsend and Prins (1978) and Acker et al (1978).

The drainage criteria manual for the City of Denver (Wright-McLaughlin, 1969) includes a set of design charts, compiled from various sources, for the estimation of energy losses at sewer junctions. The charts apply only to surcharged flow conditions. The manual also presents an example of the computation of the maximum hydraulic grade line in a sewer system, using hand computation and assuming steady flow conditions.

Marsalek (1981) found that energy losses at straight-flow-through junctions, in the case of free surface flow conditions, are fairly small and can be neglected in practical calculations. However, energy losses under surcharged conditions are significantly larger than for free surface flow conditions. Energy losses at junctions with laterals will be higher than in the case of straight-flow-through junctions. Arrangements can be made, however, to minimize these losses (e.g. Wright-McLaughlin, 1969; Townsend and Prins, 1978). Excessive losses at the junctions in a sewer system may lead to: (a) if the design is for free surface flow, the minor losses at the junctions may result in the sewer system becoming surcharged, and (b) if the design is for surcharged conditions, these losses may result in hydraulic grade lines which are higher than anticipated with the risk of the occurrence of basement flooding. Energy losses at the sewer junctions will mainly affect the hydraulic grade line calculation and should not significantly affect the flows, which are based on continuity considerations.

The difficulties involved in simulating energy losses at the junctions in sewer network routing models are obvious. The fact that none of the existing models can simulate these losses reflects these difficulties. It may be possible, however, to somewhat compensate for these losses by an increase in the roughness coefficient and by providing additional drops in the pipe inverts. Some "rules of thumb" in this regard can be found in the literature (Wright-McLaughlin, 1969).

3.6 SUMMARY

There is a host of usable routing techniques available in the literature with various levels of sophistication. The development of routing models has been mainly related to flood propagation problems in rivers, with few studies conducted on sewer systems. Dynamic routing models have been extensively tested and are found to give reliable results for most flood routing applications. They are, however, the most demanding in terms of computer time and are usually subject to numerical stability constraints. Moreover they cannot simulate supercritical flow conditions unless special techniques are used.

Applications of flow routing techniques in existing urban storm drainage models have utilized very complex as well as very simplistic methods, often without adequate justification of the selected method. Dynamic models are the only acceptable ones if representation of backwater effects is important. Also, if surcharge simulation is considered in a sewer network system, the routing technique should be based on dynamic models.

The approximate model proposed by Cunge (section 3.3.3) does seem to be well suited for (free-surface) flow routing in urban drainage applications. Nevertheless, it seems that this model has not been applied in any of the existing urban storm drainage models. A numerical solution scheme of the Cunge model, which is appropriate for urban drainage systems needs to be investigated.

Last, but not least, a conscious process for the validation of approximate routing models should be based not only on comparisons against measurements or dynamic models for specific conditions, but should also include investigation of the effect of channel slope as well as the wave period (hydrograph shape) on the validity of model results. The limitations for model applications can thus be established. Present practices do not follow these procedures for model validations.

3.7 ROUTING SUB-MODELS FOR DDSRM

The routing methods for the proposed "Dual Drainage Simultaneous Routing Model - DDSRM" will be presented in chapters IV and V for free surface and surcharged flow conditions, respectively. The free surface flow routing sub-model is intended mainly for design purposes. The surcharge sub-model, based on dynamic routing, can also simulate backwater effects in the sewer networks. It can, therefore, be utilized for the analysis of a sewer network system for storm runoff which is larger than the design flows.

Chapter IV

AN IMPROVED KINEMATIC MODEL FOR URBAN DRAINAGE SYSTEMS

4.1 INTRODUCTION

Two main objectives will be considered in the unsteady flow routing technique of DDSRM, namely, efficiency of the computation and reasonable accuracy. It is also desirable to use the same routing technique for the analysis of the sewer and street networks. Dynamic models will be avoided not only because they are too demanding on computer time but also because free surface flow in DDSRM involves sizing of the sewers which, as discussed in section 3.5.1, makes the use of dynamic models inappropriate.

While existing urban storm drainage models use, in general, either too simplistic or too complex routing techniques, it is attempted in the present research to have an intermediate level of sophistication. The development of the flow routing sub-models for DDSRM will be undertaken in conjunction with two basic assumptions for the networks: (1) there is no significant backwater effect, and (2) the networks are dendritic and converge towards the downstream outlet.

It follows from section 3.3.3 that a solution based on the Cunge model is most suitable for DDSRM. In spite of the apparent advantages of this solution, over other approximate routing methods, it seems that it has not been utilized in any of the existing urban storm drainage models. As indicated in section 3.3.3, the Cunge model has been developed under the assumption of constant parameters. However, because of the type of (peaky) design storms usually used in urban drainage studies, this assumption is no longer appropriate. The wave celerity will be strongly variable which necessitates the variation of the model parameters with discharge. The use of a variable wave celerity, $c = c(Q) = c(t)$, has been recently referred to as "crucial" for routing accuracy (Koussis, 1978).

In this chapter a numerical solution scheme of the kinematic wave model is described whereby the model parameters (calculated according to the Cunge model) will be allowed to vary with time. The routing procedures and calculation of model parameters will be described, with special reference to the channel geometries represented by circular pipes and street sections.

A thorough investigation of the model validity will be conducted in comparison with, previously verified, dynamic models through several numerical experiments, including verification in a sewer network situation. The effect of two key parameters, namely channel slope and wave period, on the validity of the model results will also be investigated and the model limitations will thus be established.

4.2 MAIN EQUATIONS

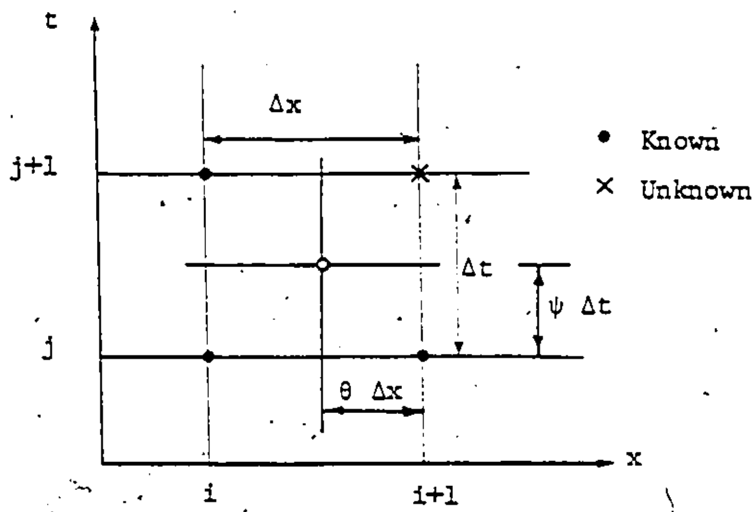


Fig. 4.1 Computational Grid for DDSRM

The kinematic wave model can be expressed by Eq. 3.16, which for no lateral inflow gives

$$\frac{1}{c}\frac{\partial Q}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad 4.1$$

where c(Q) is the wave celerity or kinematic wave speed, Q(x,t) is the discharge, and x and t are the space and time coordinates, respectively.

With reference to Fig. 4.1, Eq. 4.1 can be expressed in finite-difference form as

$$\frac{1}{c}\,\frac{\theta(Q_i^{j+1} - Q_i^j) + (1-\theta)(Q_{i+1}^{j+1} - Q_{i+1}^j)}{\Delta t} + \frac{\Psi(Q_{i+1}^{j+1} - Q_i^{j+1}) + (1-\Psi)(Q_{i+1}^j - Q_i^j)}{\Delta x} = 0 \qquad 4.2$$

in which θ and Ψ are weighting factors. Taking $\Psi = 1/2$, and substituting

$$K = \frac{\Delta x}{c} \qquad 4.3$$

the finite-difference equation (Eq. 4.2) can be written in a form which is identical to the classical form of the Muskingum model

$$Q_{i+1}^{j+1} = c_1\,Q_i^j + c_2\,Q_i^{j+1} + c_3\,Q_{i+1}^j \; ; \qquad 4.4$$

where

$$c_1 = \frac{K\theta + \Delta t/2}{K(1-\theta) + \Delta t/2} \; ; \quad c_2 = \frac{-K\theta + \Delta t/2}{K(1-\theta) + \Delta t/2} \; ; \quad c_3 = \frac{K(1-\theta) - \Delta t/2}{K(1-\theta) + \Delta t/2}$$

In the above scheme the wave celerity c is assumed constant. Δx is the reach length and K represents the travel time of discharge, which is also assumed constant. $\psi = 1/2$ will guarantee the stability of the finite difference scheme (Cunge, 1969).

As indicated in section 3.2.2, if θ is calculated from Eq. 3.23, or alternatively

$$\theta = \frac{1}{2}\left(1 - \frac{Q}{BS_o c\Delta x}\right) \; ; \quad 0 \leq \theta \leq 0.5 \qquad 4.5$$

where B is the surface width of flow , the specially weighted finite difference scheme of the kinematic model (Eq. 4.2) can be considered as a second order approximation of the diffusion wave model.

Now if we write

$$\phi = \frac{Q}{BS_o c\Delta x} \qquad 4.6$$

and

$$\alpha = \frac{c\Delta t}{\Delta x} \qquad 4.7$$

Eq. 4.2 can be expressed as

$$Q_{i+1}^{j+1} = \frac{1 + \alpha - \phi}{1 + \alpha + \phi} Q_i^j + \frac{-1 + \alpha + \phi}{1 + \alpha + \phi} Q_i^{j+1} + \frac{1 - \alpha + \phi}{1 + \alpha + \phi} Q_{i+1}^j \qquad 4.8$$

ϕ represents a type of cell Reynolds number; that is, the ratio of physical to cell diffusivity. α is the Courant number, representing the ratio of wave celerity to cell celerity (Ponce, 1979).

The advantage of the above scheme is that the solution is carried out using an algebraic equation rather than a differential equation. Furthermore the model parameters can be related to the physical and hydraulic characteristics of the channel under consideration.

4.3 MODEL PARAMETERS AND STABILITY CRITERIA

There are two basic parameters involved in the above model, namely the grid Reynolds number, ϕ, and the Courant number, α, as defined by Eqs. 4.6 and 4.7. As indicated in Eq. 4.5, the value of θ is restricted between $0 \leq \theta \leq 0.5$. Thus it follows that:

$$0.0 \leq \phi \leq 1.0 \qquad 4.9$$

Note that the physical concept behind Eq. 4.2 requires that $\theta > 0$. (that is $\phi < 1$.). On the other hand, $\phi < 0.0$ (that is $\theta > 0.5$) will result in numerical instability, causing the wave to amplify, that is the discharge increasing from one point to the next (Cunge, 1969). Therefore Eq. 4.9 will satisfy the stability requirements with respect to ϕ.

Given the space interval, Δx, the theoretical upper limit for Δt is given by the stability condition of the kinematic model, Eq. 3.18. Weinmann and Laurenson (1979) have reported, however, that, in practical applications, values of Δt several times larger than the value given by Eq. 3.18 have been used without any apparent stability problems. In the present research, the conservative upper limit for Δt given by Eq. 3.18 will be used, that is

$$\alpha \leq 1.0$$

It should be recalled that the model parameters have been derived assuming constant celerity, which implies slow rising flood and hence small changes in discharge. This assumption is not adequate for urban drainage systems (DDSRM) since the wave celerity or kinematic wave speed, c, is strongly variable. Therefore the numerical scheme adopted in the present research utilizes wave celerities which vary with discharge (Ponce and Yevjevich, 1979). The numerical solution and routing procedures are presented below.

4.4 NUMERICAL SOLUTION AND ROUTING PROCEDURES

An inflow hydrograph into a sewer/channel can be routed along a reach Δx length according to Eq. 4.8 using the four-point grid shown in Fig. 4.1. For a given discharge the depth of flow and hence other hydraulic properties can be calculated from the steady flow rating curve for the given channel. The wave celerity c can be calculated at every time step from Eq. 3.17 ($c = dQ/dA$).

Let us assume that the inflow hydrograph is specified at time increments of Δt, for example from the surface runoff sub-model. In the case of a pipe (or street) network such as the case at hand the space interval, Δx, will be taken as the total pipe (or street segment) length.

An inflow hydrograph into a channel/sewer specified at time intervals of Δt is thus routed to the downstream end as follows:

A computational time step Δt_1, is first calculated which satisfies the condition $\alpha<1.0$. Given the reach length, Δx, Δt_1 is calculated from

$$\Delta t_1 = \frac{\Delta x}{c_{max}}$$

where c_{max} is the maximum celerity of the inflow, computed according to Eq. 3.17. The inflow hydrograph specified at time increments of Δt is thus interpolated at the time intervals Δt_1 before routing.

The calculation starts at time t and proceeds in increments of Δt_1. The inflow and outflow at time t: Q_i^j and Q_{i+1}^j, respectively are known from the initial conditions (base flow) or from the previous time step. Now in order to calculate the outflow Q_{i+1}^{j+1} corresponding to the inflow Q_i^{j+1} at time $t + \Delta t_1$, the celerity c and top width B are calculated based on the flow conditions at the grid points (i,j), (i+1,j), and (i,j+1).

A three-point average of Φ and α can be calculated as (Ponce and Yevjevich, 1978):

$$\Phi = (\Phi_i^j + \Phi_i^{j+1} + \Phi_{i+1}^j) / 3 \qquad 4.10$$

$$\alpha = (\alpha_i^j + \alpha_i^{j+1} + \alpha_{i+1}^j) / 3 \qquad 4.11$$

The outflow Q_{i+1}^{j+1} is thus calculated using Eq. 4.8 with Φ and α estimated from Eqs. 4.10 and 4.11. Replacing Q_{i+1}^j by Q_{i+1}^{j+1} this process is repeated, advancing in time increments of Δt_1 until the entire hydrograph is routed. The routing procedures for a single channel/sewer are illustrated in Fig. 4.2. The outflow hydrograph is then interpolated back to Δt time intervals.

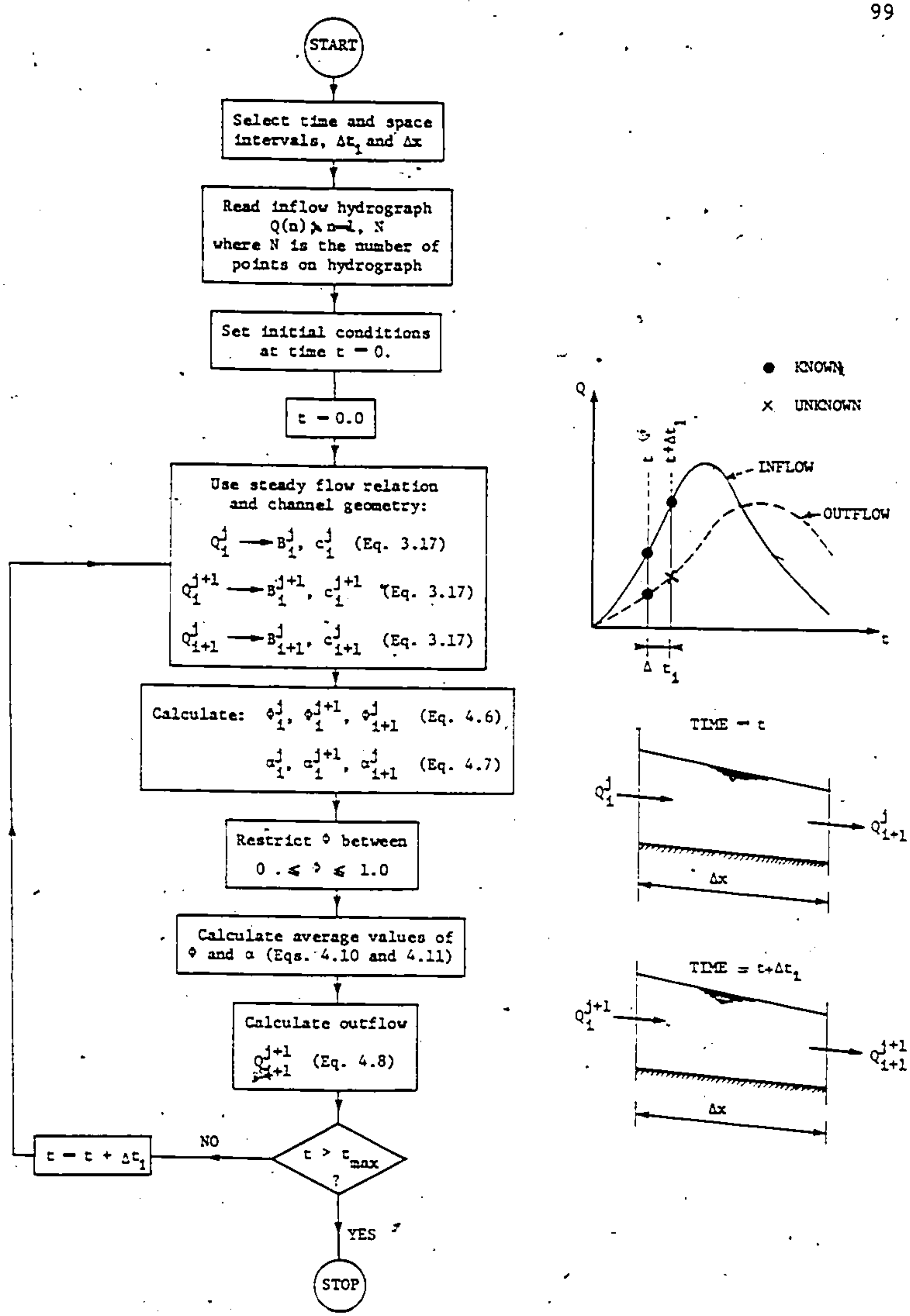


Fig. 4.2 Flow Chart of the Unsteady Flow Routing Model of DDSRM

A computer program based on the above scheme is written for unsteady flow routing in the two channel geometries represented by: (a) a circular pipe, and (b) a street section. The program will be utilized as a sub-model of the Dual Drainage Simultaneous Routing Model - DDSRM (Chapter VI).

In DDSRM the sewer and street systems form dendritic networks. The calculation is therefore performed starting from the uppermost segments and proceeds downstream in a sequential manner. For each segment the entire hydrograph is routed before proceeding downstream. The conditions at the junctions are represented by the continuity equation

$$\Sigma\ (Q_j)_t = 0.0 \qquad 4.12$$

where $\Sigma\ (Q_j)_t$ is the algebraic sum of all flows entering and leaving the junction at time t. The inflow into a sewer is composed of the inlet flow plus the outflow from the immediate upstream pipe(s). The inflow into a street segment is composed of direct runoff from the subcatchment (assumed concentrated at the upstream end of the street segment) plus any carry-over flow from the immediate upstream street segment(s). More details on the network model will be given in chapter VI.

4.5 RATING CURVE

In the above scheme the wave celerity, c, is considered variable with depth (nonlinearity). However it is calculated as a function of depth only. This implies a single valued stage-discharge relationship. The attenuation effects are obtained by adopting a value of $\phi > 0$ (that is $\theta < 0.5$). In the present study Manning's formula is utilized for depth-discharge conversion and, hence, for the calculation of wave celerity.

4.5.1 Considerations For Circular Pipes

The Manning's relationship between depth and discharge in a circular conduit indicates that the maximum capacity is greater than the full flow capacity and will occur somewhere before the pipe is completely full. This is illustrated in Fig. 4.3 (curve 1). The associated abrupt decrease in the hydraulic capacity near the crown of the pipe will be accompanied by hydraulic instability, that is a transient pressure in the system (Hamam and McCorquodale, 1980). These transient effects will be neglected here.

In order to avoid numerical instability caused by the existence of two values of depth for a given discharge (near the pipe crown) appropriate assumptions must be made. Schaake (1971) assumes a linear relationship between the cross sectional area and discharge (curve 3 in Fig. 4.3a).

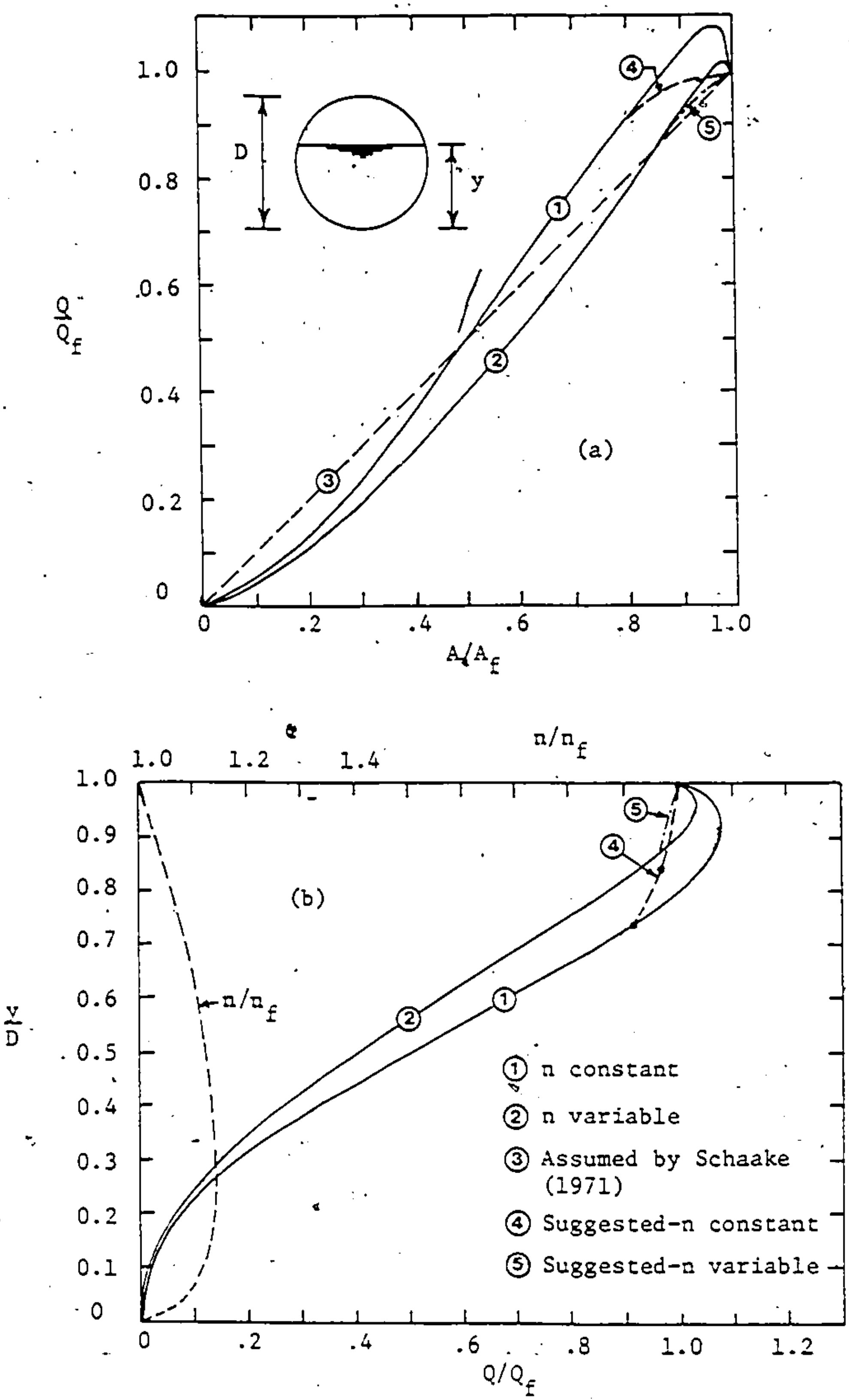


Fig. 4.3 Stage-Discharge Relationship for a Circular Channel

In the present study it will be assumed that the maximum capacity is equal to the full flow capacity. Accordingly, the rating curve should be adjusted as suggested in Fig. 4.3 (curve 4). This relation is based on the assumption that Manning's coefficient is constant. It has been found out, however, that Manning's coefficient will change with the flow depth (ASCE and WPCF, 1970), as indicated in Fig. 4.3b. The associated rating curve is given in Fig. 4.3 (curve 2).

In the computer program the relationship between depth and discharge for circular pipes is input directly in dimensionless form (as block data). This will allow for including the effect of varying Manning's coefficient (curve 5 in Fig. 4.3). During the calculation for any given discharge, the associated depth is computed by a parabolic interpolation from the given depth-discharge relationship. Other parameters such as surface width and cross sectional area are computed from the sewer geometry. The wave celerity, c, is calculated accordingly from Eq. 3.17.

4.5.2 Street Section

Some previous studies (Larson, 1948; Tholin et al, 1960) indicate that Manning's relation cannot be used for the calculation of flow in a street section without modification since the hydraulic radius cannot adequately describe the cross section in this case. Therefore Manning's formula is first used to calculate the flow, dQ, for an incremental area, depth h, and width dx (Fig. 4.4). By integrating from where the depth is zero at the edge of the surface to y at the curb, the total flow Q(cfs) is given by (Tholin et al, 1960):

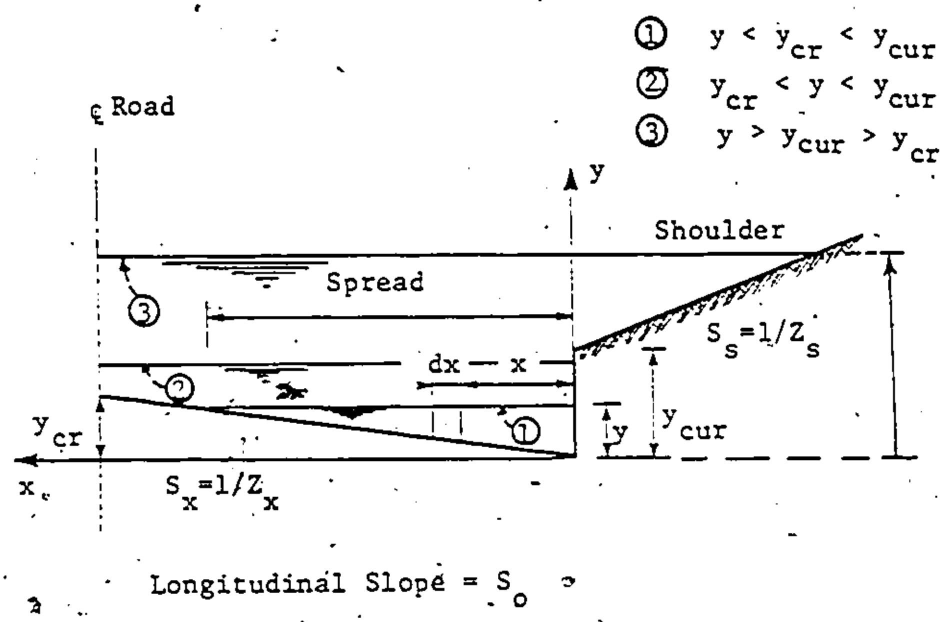


Fig. 4.4 Cross Section of a Road Drainage Channel

$$Q = \frac{0.557}{n_x} S_o^{1/2} Z_x y^{8/3} \qquad 4.13$$

where

n_x = Manning's coefficient for the pavement;

S_o = longitudinal slope;

Z_x = reciprocal of the pavement cross slope;

y = depth at curb, in ft.

Eq. 4.13 is developed for low flows where the depth does not exceed the road crown, and is usually referred to as "gutter" flow. This equation has been used for the calibration of the roughness coefficient in the hydraulic model study on inlet control (Townsend, 1979).

With reference to Fig. 4.4, Eq. 4.13 can be extended to compute the street carrying capacity for higher stages of flow as follows:

For $y_{cr} < y < y_{cur}$

$$Q = \frac{1.114}{n_x} S_o^{1/2} Z_x \, [y^{8/3} - (y - y_{cr})^{8/3}] \qquad 4.14$$

where

y_{cr} = height of road crown, measured from the edge of the curb;

y_{cur} =height of curb.

For $y > y_{cur} > y_{cr}$

$$Q = 1.114\, S_o^{\frac{1}{2}} \left[\frac{Z_x}{n_x} \{ y^{8/3} - (y - y_{cr})^{8/3} \} + \frac{Z_s}{n_s} (y - y_{cur})^{8/3} \right] \quad 4.15$$

where

n_s = Manning's coefficient for the shoulder;

Z_s = reciprocal of the shoulder cross slope.

Eqs. 4.14 and 4.15 compute the carrying capacity of the whole street section, while Eq. 4.13 computes the flow in half the section. In SI units the constants 0.557 (in Eq. 4.13) and 1.114 (in Eqs. 4.14 and 4.15) should be replaced by 0.375 and 0.75, respectively for Q in m^3/sec. and y in meters.

Eqs. 4.13 through 4.15 can be used to develop a stage-discharge relationship for a given street section. At the same time a similar relationship between discharge and celerity can be obtained, using Eq. 3.17. This relationship can thus be used to compute c, α, and ϕ in a street section for any inflow conditions.

An inspection of Fig. 4.5, which represents an example of a rating curve for a street section, shows that a small change in depth will correspond to a substantial increase in hydraulic capacity.

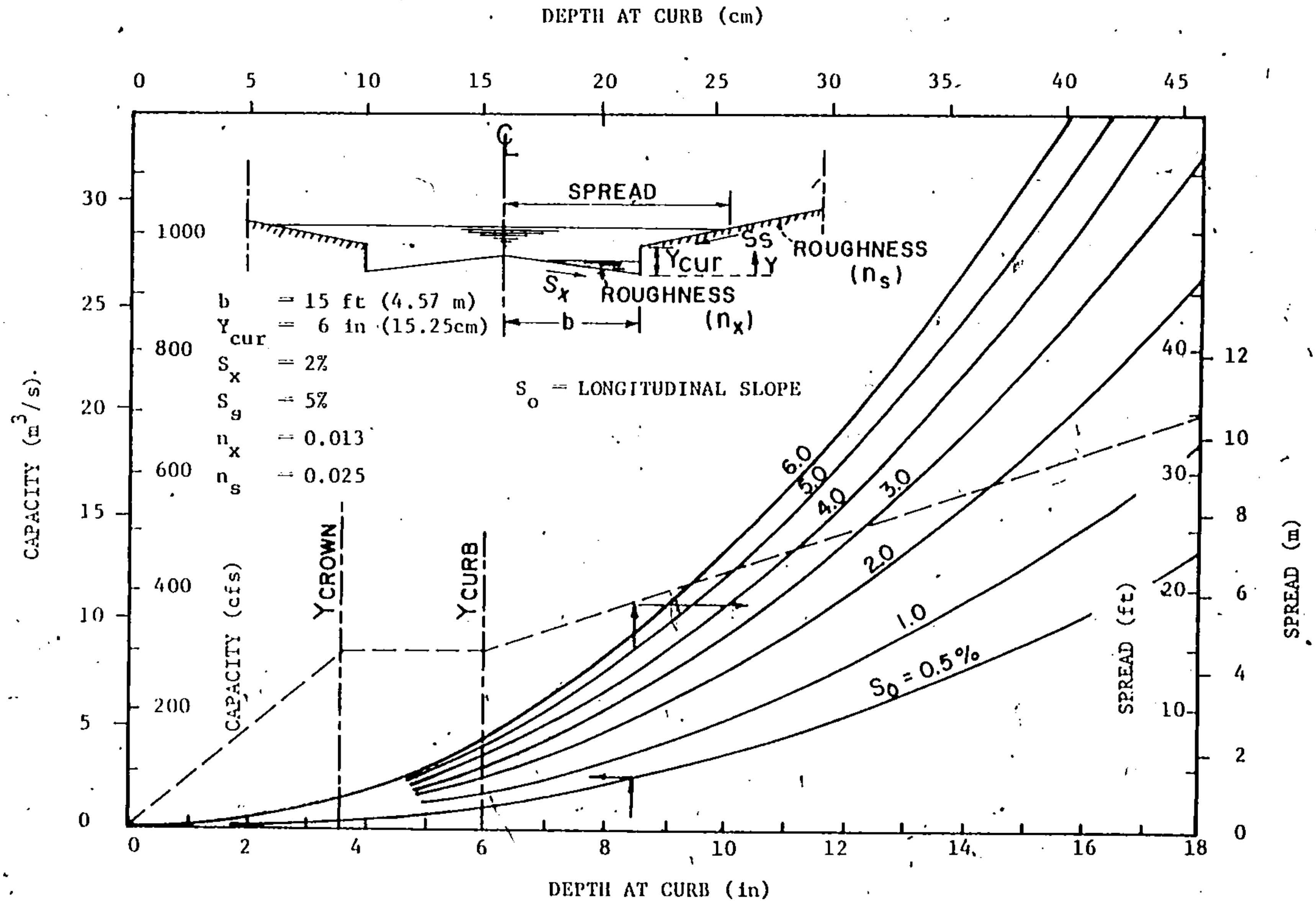


Fig. 4.5 Example of Stage-Discharge Relationship of a Road Drainage Channel

4.6 MODEL VALIDATION

The validation of the model presented above, for the purpose of the present study, should be based on its capability of representing the attenuation and translation properties of the wave, as well as conservation of mass (the wave being represented by the inflow hydrograph). This task will be undertaken in three steps:

1. validation of the numerical solution scheme for conditions corresponding to urban drainage systems;
2. investigation of the range of model applicability, by evaluating the effect of channel slope and wave period on the validity of results; and
3. model validation in a network situation.

As indicated in section 3.6, dynamic models have been extensively tested and are found to provide the most accurate results. Therefore the validation of the routing submodel of DDSRM will be based on comparisons with complete dynamic models.

4.6.1 Validation of Numerical Solution

The validation of the numerical solution scheme presented above is conducted on hypothetical triangular and trapezoidal hydrographs in conjunction with hypothetical pipe systems. The routed hydrographs using the proposed model are compared at selected locations with those obtained by means of dynamic models. Two dynamic models are used for this purpose: (1) an explicit finite-difference scheme of the characteristics equations using a rectangular x-t grid (Yevjevich and Barnes, 1970a), and (2) a centered four-point implicit scheme (Klym et al., 1972). The first model has been previously verified in a hydraulic model study on a prototype pipe (822 ft long, 3 ft diameter covering various slopes).

The results of five numerical experiments (referred to as Schemes 4.1 through 4.5) are presented in Figures 4.7 through 4.9. Description of the pipe systems and test arrangements is given in table 4.1. Reference should also be made to Fig. 4.6 which shows the configuration of inflow. The pipe slope in all the tests is 0.001. The results obtained by the proposed model are referred to as "DDSRM".

Fig. 4.7 compares the modification of a triangular hydrograph obtained by DDSRM with those obtained by means of the method of characteristics (Scheme 4.1). The comparison is made at a distance of 5,000; 18,000 and 30,000 ft (1,525; 5,490 and 9,150 m) down the pipe. This test is identical to the one used to verify the sewer flow routing technique of the transport block of the EPA-SWM model (Metcalfe & Eddy, Inc., 1971). Fig. 4.8 presents a similar comparison between DDSRM and the four-point implicit scheme (Schemes 4.2 and 4.3). The comparison is made at a distance of 3000 and 6000

Table 4.1 Validation of the Unsteady Flow Routing Model of DDSRM - Description of Test Arrangements and Inflows

Scheme	Inflow Hydrograph[a]							Pipe System		Figure No.
	Base Flow cfs(m^3/s)	Q_1 cfs(m^3/s)	Q_2 cfs(m^3/s)	D_1 min.	D_2 min.	D_3 min.	D_4 min.	L ft (m)	D ft (m)	
4.1[b]	28(0.79)	0	80(2.26)	0	40	0	40	30000(9150)	6(1.83)	4.7
4.2[b]	0	28(0.79)	108(3.06)	15	40	0	30	6000(1830)	6(1.83)	4.8a
4.3[b]	0	28(0.79)	108(3.06)	15	40	40	80	6000(1830)	6(1.83)	4.8b
4.4[c]	0	5.6(0.16)	18.6(0.53)	15	40	0	30	4x1500 (4x457.5)	3.5(1.07) 4.5(1.37) 5.0(1.52) 5.5(1.68)	4.9a
4.5[c]	0	5.6(0.16)	18.6(0.53)	15	40	40	80	4x1500 (4x475.5)	3.5(1.07) 4.5(1.37) 5.0(1.52) 5.5(1.68)	4.9b

(a) refer to Figure 4.6 for the configuration of inflow

(b) inflow at upstream end of pipe

(c) pipes are in series - identical inflows at the upstream end of each pipe

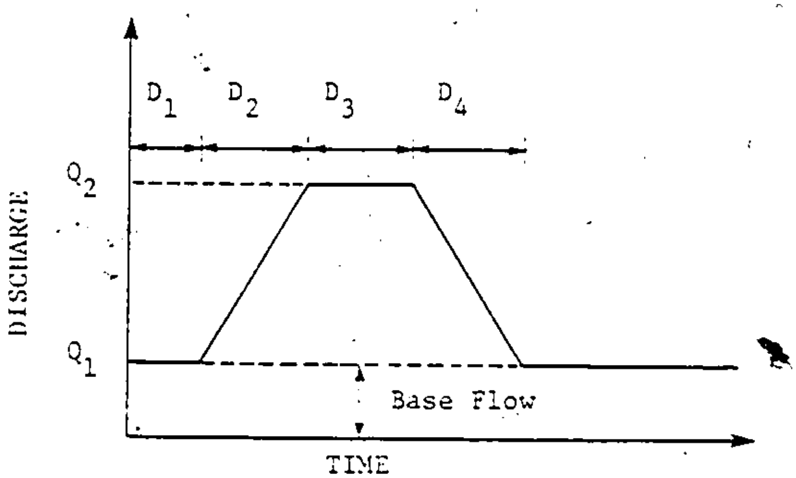


Fig. 4.6 Validation of the Unsteady Flow Routing Model of DDSRM - Configuration of Inflow Hydrograph

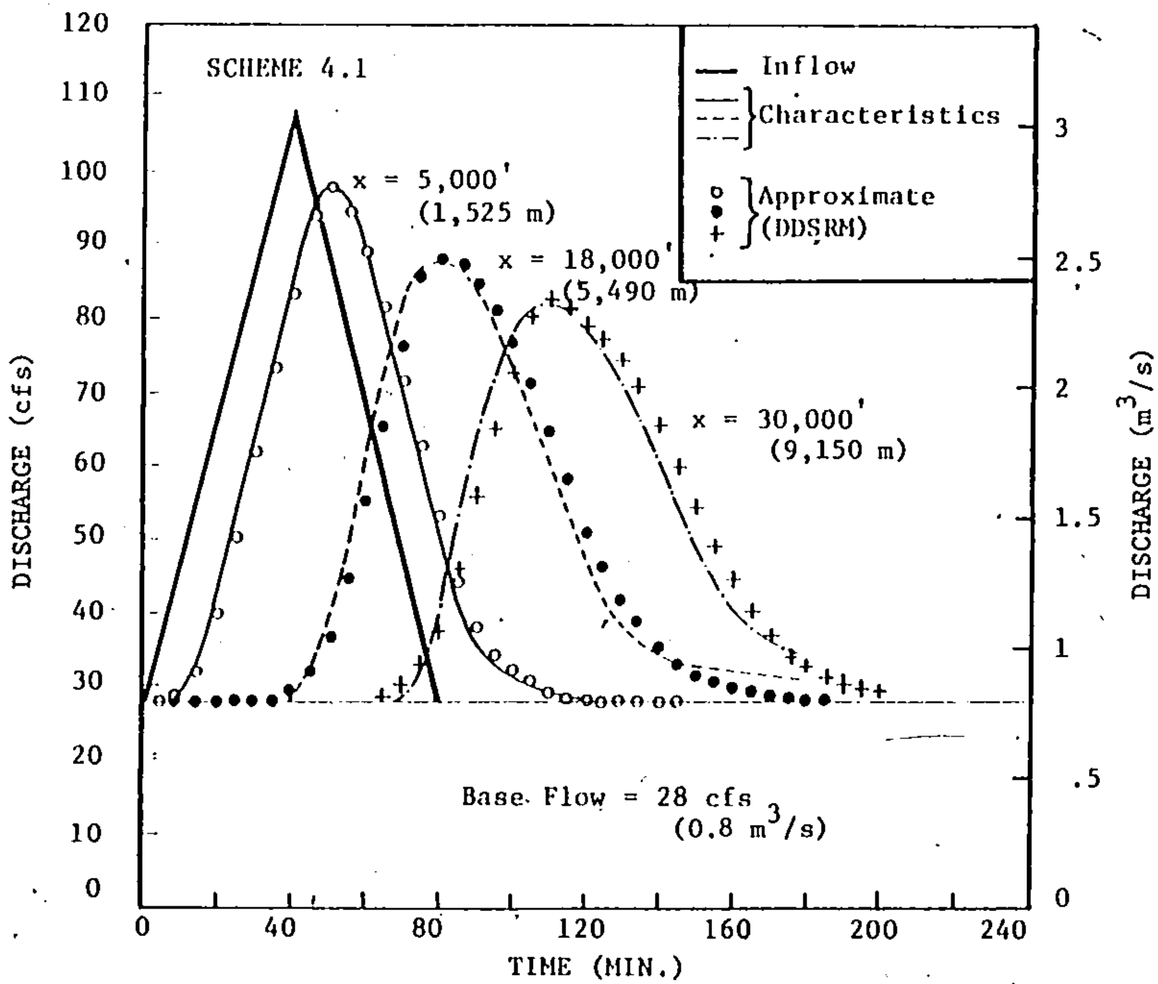
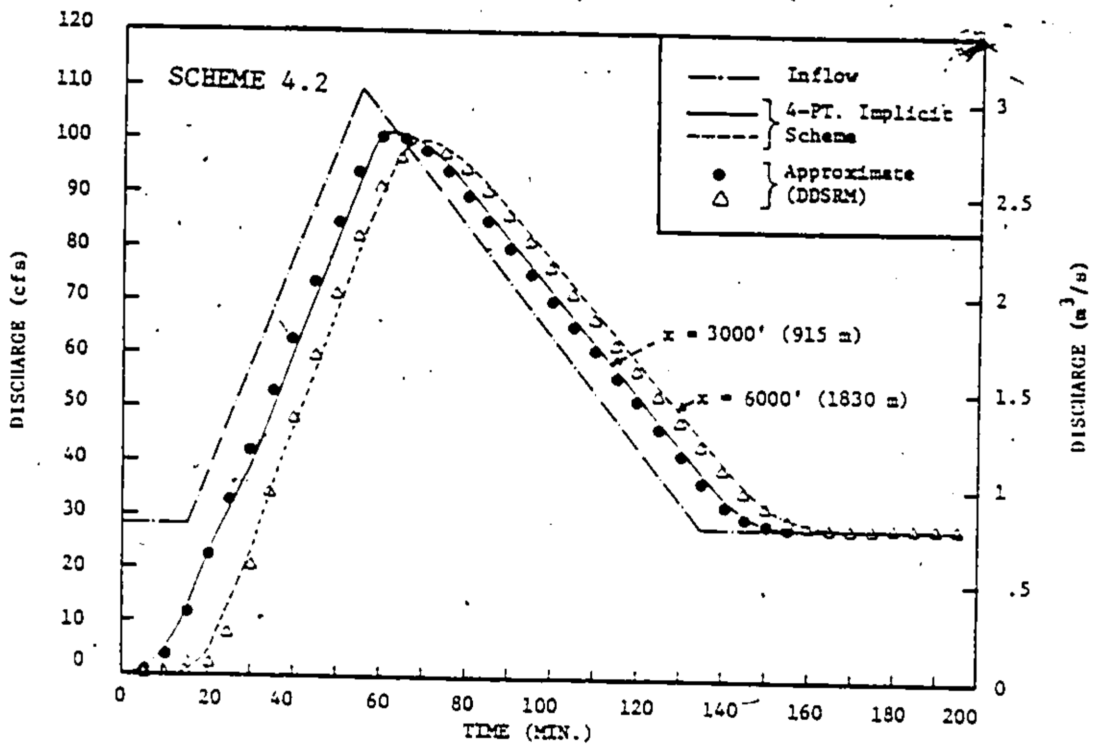


Fig. 4.7 Validation of the Unsteady Flow Routing Model of DDSRM - Comparison With Dynamic Routing By the Method of Characteristics - Scheme 4.1



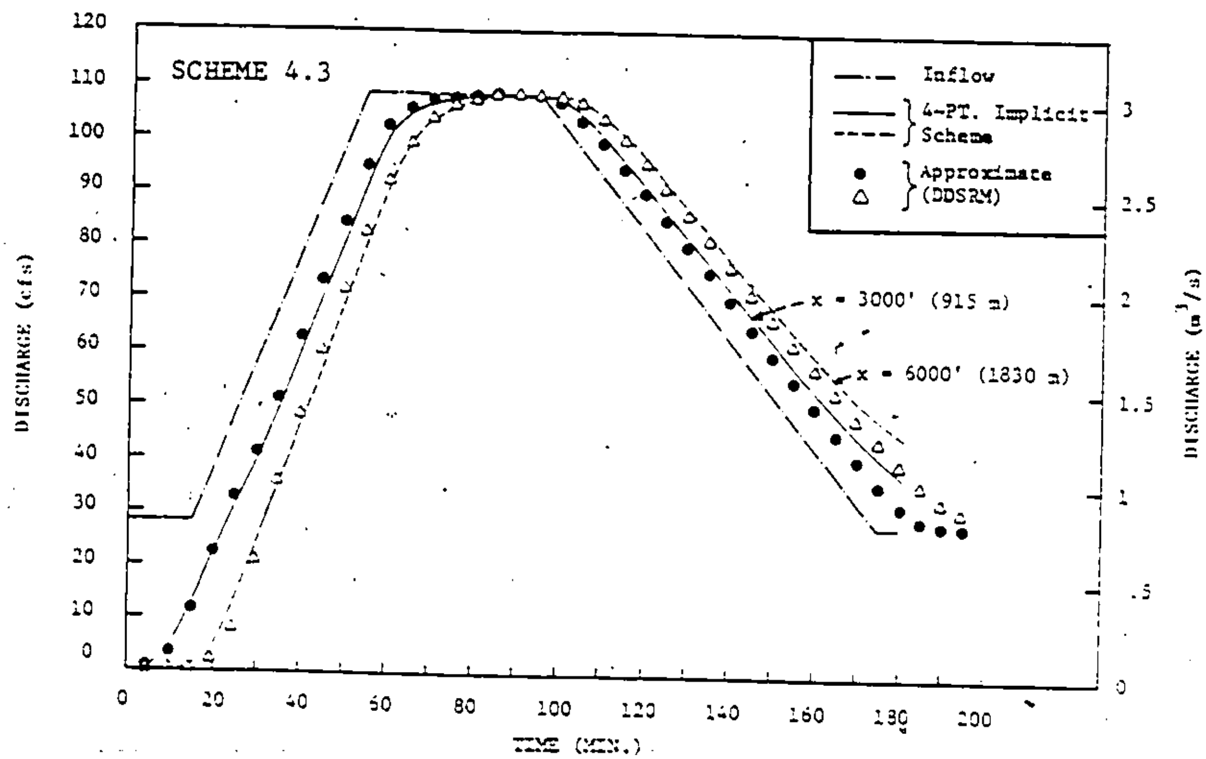
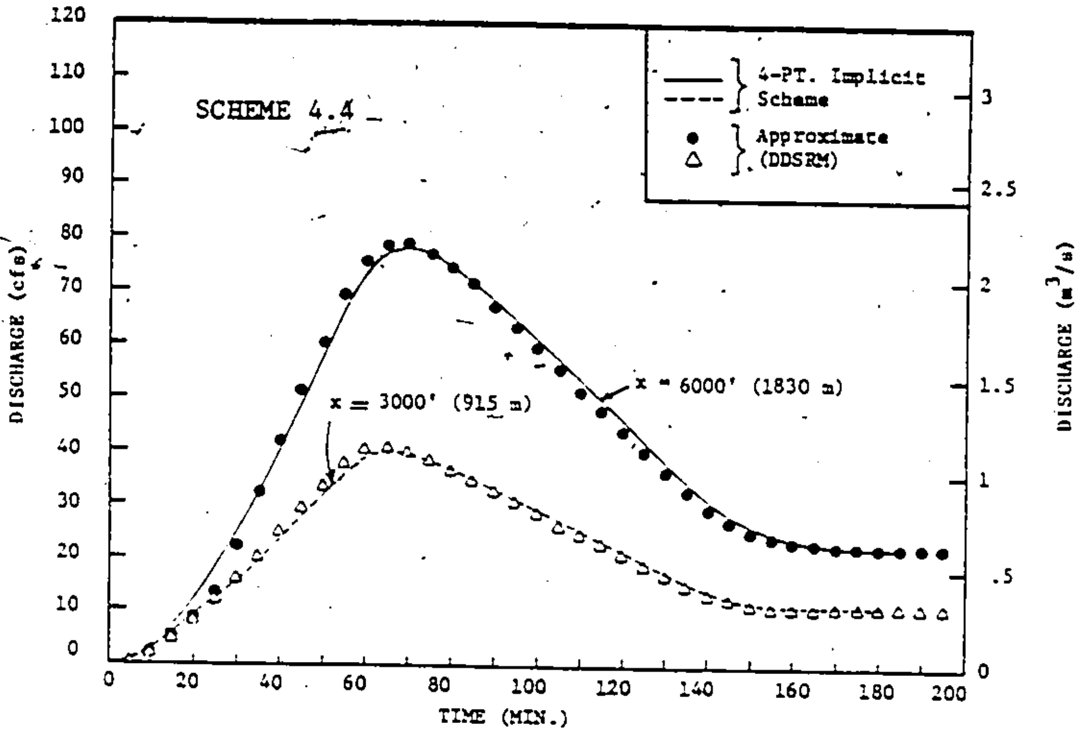


Fig. 4.8 Validation of the Unsteady Flow Routing Model of DDSRM - Comparison With Dynamic Routing By a Centered Four-Point Implicit Solution - Schemes 4.2 and 4.3



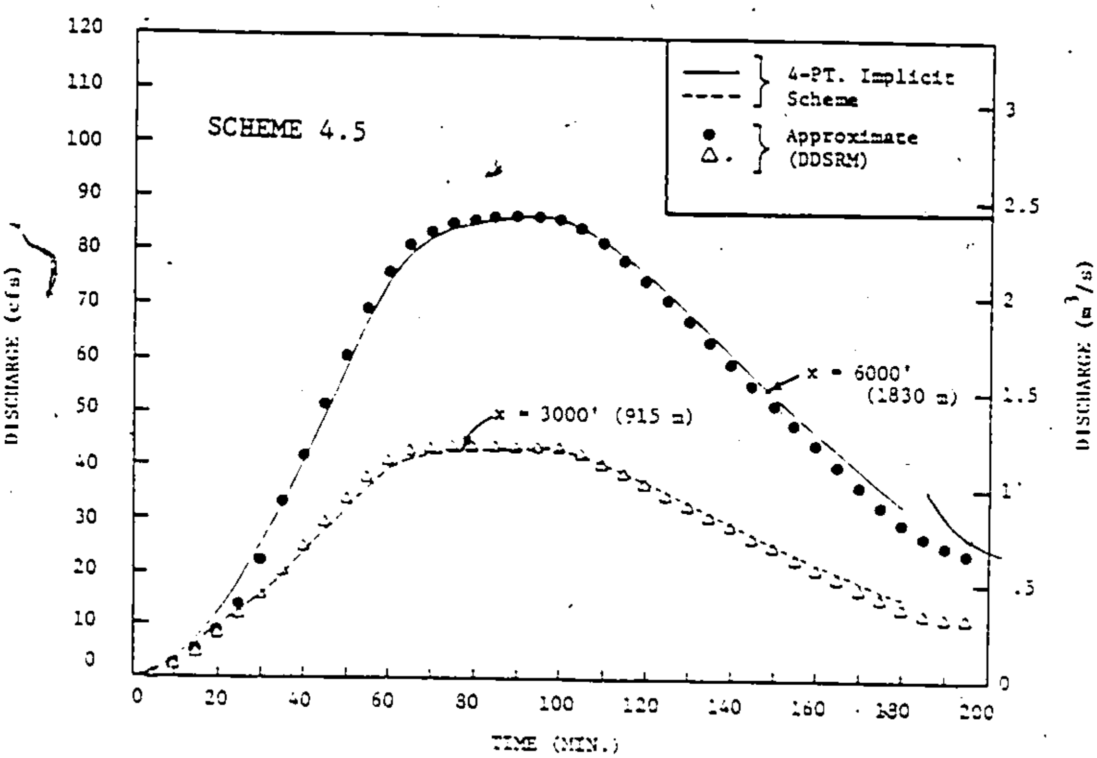


Fig. 4.9 Validation of the Unsteady Flow Routing Model of DDSRM - Comparison With Dynamic Routing By a Centered Four-Point Implicit Solution - Schemes 4.4 and 4.5

ft (915 and 1830 m). In Figs. 4.8a and 4.8b the inflows are of triangular and trapezoidal shapes, respectively and are input at the upstream end of the pipe. Fig. 4.9 shows a comparison with the four-point implicit scheme in the case of several inflows at different locations along the pipe (Scheme 4.4 and 4.5). In Scheme 4.4 all inflow hydrographs are of triangular shape, while in Scheme 4.5 they are trapezoidal.

Inspection of figures 4.7 through 4.9 indicates that the results obtained by DDSRM are in close agreement with the dynamic models even for the extreme case presented by the rather long pipe (Scheme 4.1). The simplified routing method of DDSRM conserves mass and appears to present the translation and attenuation properties of the wave with reasonable accracy.

4.6.2 General Validity of the Model

The above experiments have been conducted for one slope (0.001) and one general shape of inflow hydrograph. It will be attempted next to check the model validity for a wider range of conditions. As indicated in section 3.4 there are two significant parameters which are critical for the applicability of approximate routing models, namely channel slope and wave period (hydrograph duration).

Let us consider two hypothetical circular channels, 8000.0 ft (2440 m) long and 8.0 ft (2.44 m) diameter each, with two different slopes, namely 0.002 and 0.0005. A sinusoidal hydrograph is input into each channel (see Fig. 4.10). The flow is given by

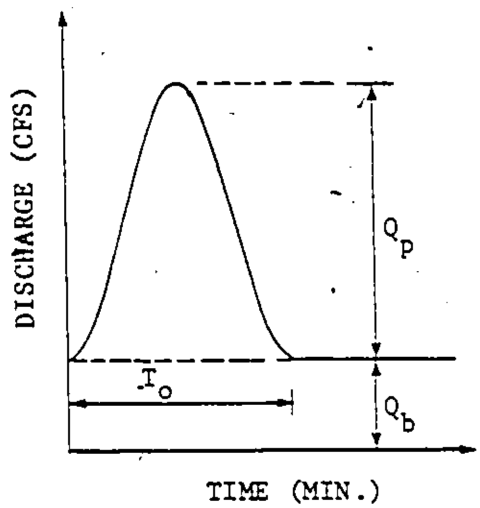


Fig. 4.10 Sinusoidal Wave Used for the Investigation of the General Validity of the Routing Technique of DDSRM

$$I(t) = Q_b + \frac{Q_p}{2}\left(1 - \cos\frac{2\pi t}{T_o}\right); \quad 0 < t \leq T_o$$

$$I(t) = Q_b; \quad t > T_o \tag{4.16}$$

where $I(t)$ is the inflow, Q_b is the base flow, Q_p is the peak of the inflow hydrograph, t is the time in minutes, and T_o is the duration of inflow in minutes.

Taking Q_b = 20 cfs (0.57 m^3/s), and Q_p = 100 cfs (2.83 m^3/s), the inflow at any time t will be given by

$$\left.\begin{aligned} I(t) &= 70 - 50 \cos\left(\frac{2\pi t}{T_o}\right) \; ; \; 0 < t \leq T_o \\ I(t) &= 20 \; ; \; t > T_o \end{aligned}\right\} \text{for } I(t) \text{ in cfs} \qquad 4.17a$$

$$\left.\begin{aligned} I(t) &= 1.985 - 1.415 \cos\left(\frac{2\pi t}{T_o}\right) \; ; \; 0 < t \leq T_o \\ I(t) &= 0.57 \; ; \; t > T_o \end{aligned}\right\} \text{for } I(t) \text{ in } m^3/s \qquad 4.17b$$

Three different durations (T_o) of the inflow are assumed, namely 20, 40 and 100 minutes. The corresponding inflow for each duration is thus routed in each of the two channels by means of: (a) DDSRM, and (b) the EXTRAM model, an explicit dynamic routing model, which has been previously verified (Kassem and Wisner, 1980)- see section 5.3 for examples of the verification tests of EXTRAN.

Fig. 4.11 compares the routed hydrographs at a distance of x/D = 625 for the channel slope of 0.002 and for the three different hydrograph durations. Fig. 4.12 is a similar plot for the channel slope of 0.0005. Fig. 4.13 compares the peak attenuation along each channel by the dynamic and approximate models for the different conditions tested.

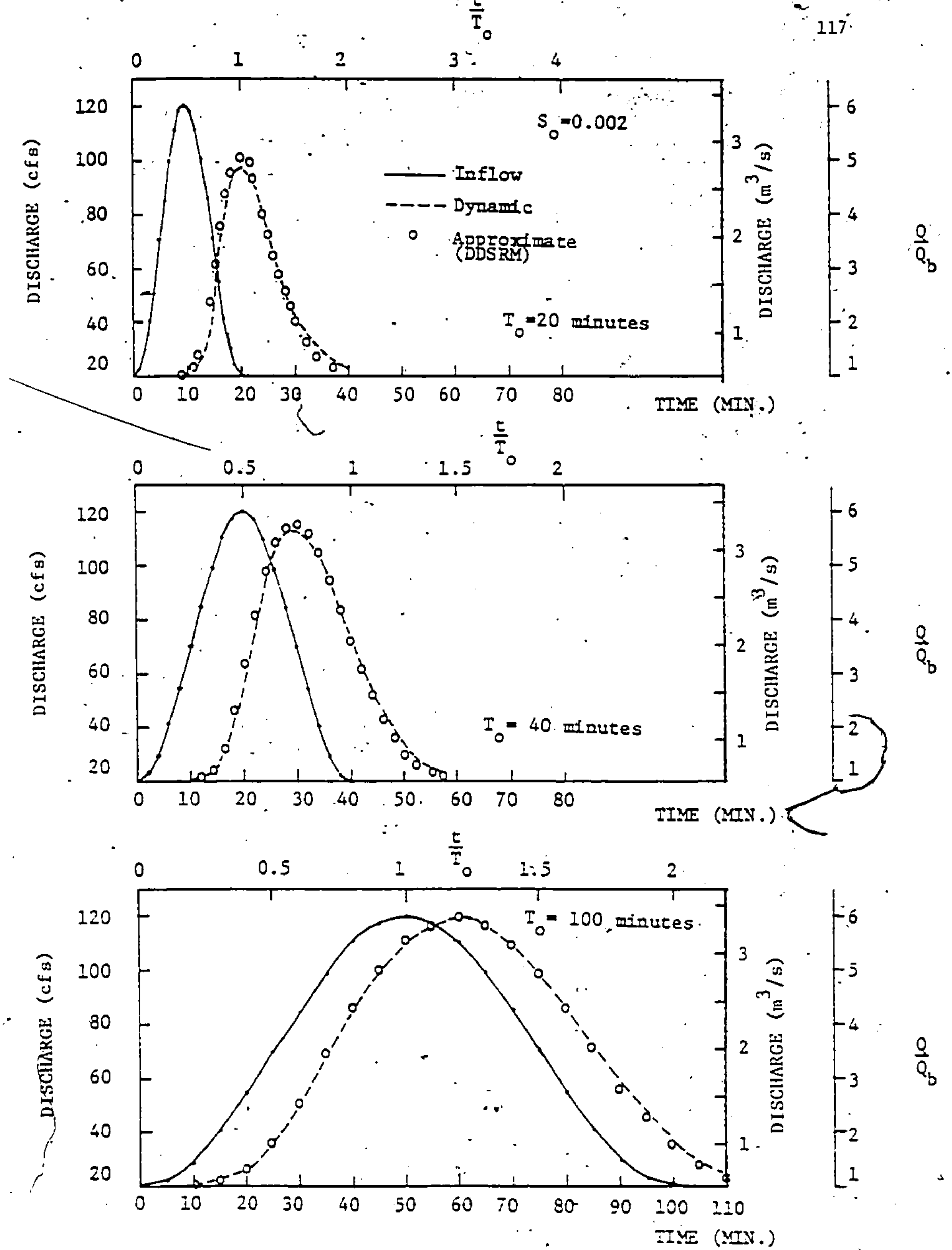


Fig. 4.11 Effect of Wave Period on the Validity of the Routing Technique of DDSRM in a Steep Channel

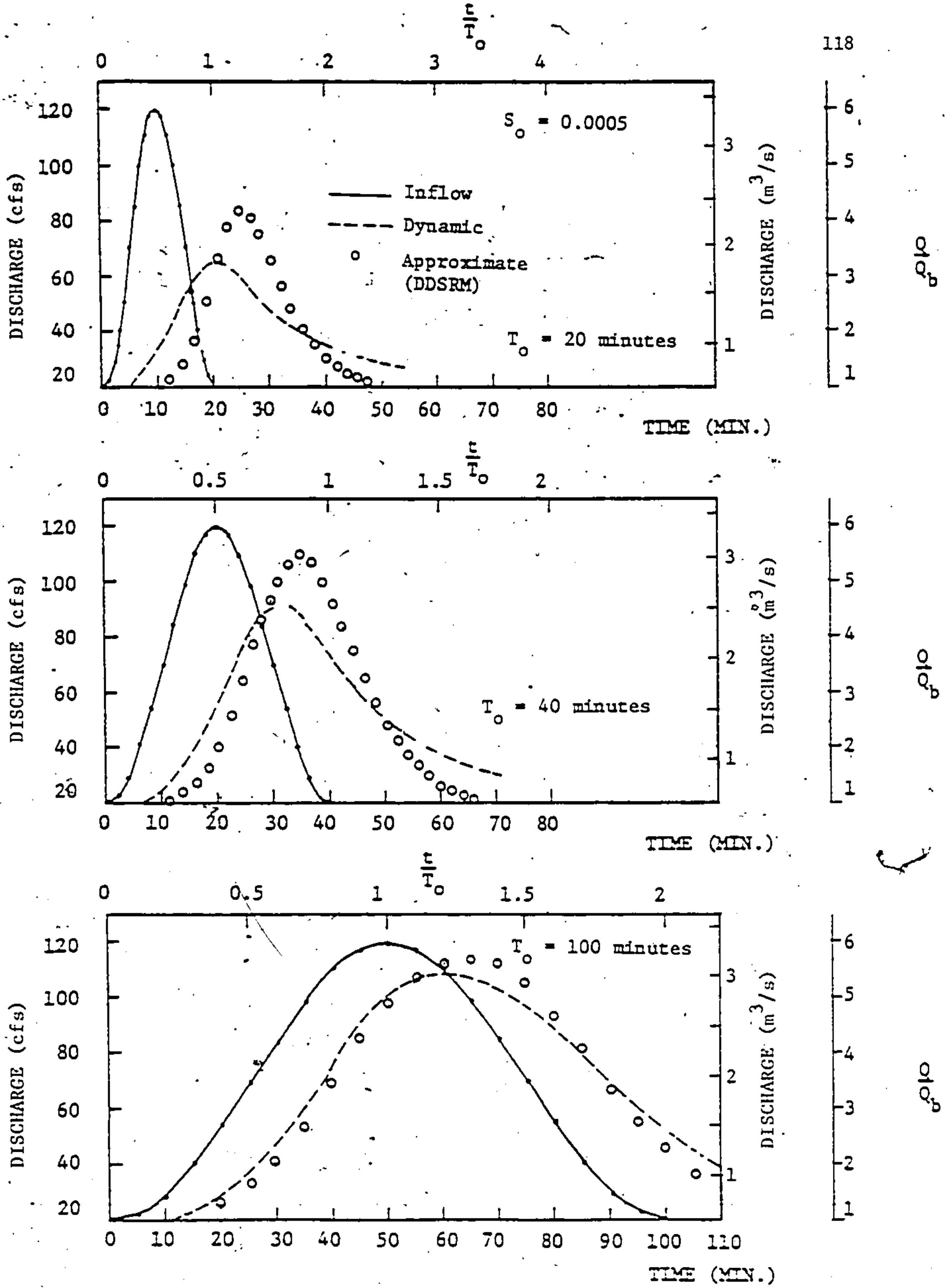


Fig. 4.12 Effect of Wave Period on the Validity of the Routing Technique of DDSRM in a Mild Channel

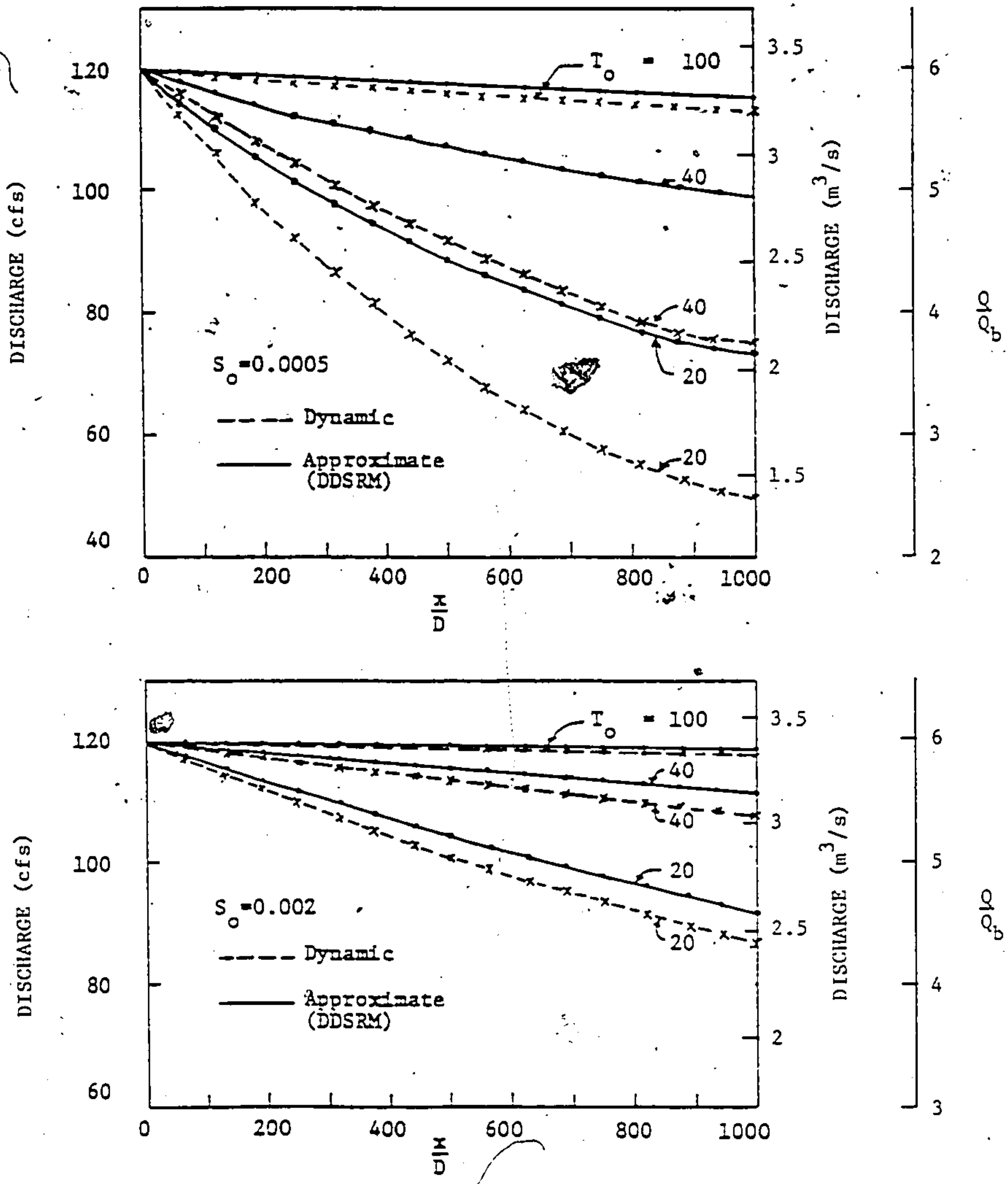


Fig. 4.13 Peak Flow Attenuation In Steep and Mild Channels In Terms of Wave Period - Comparison Between A Dynamic Model and DDSRM

4.6.2.1 Discussion of Results

The results of the above experiments show that:

1. For the steeper slope (S_o = 0.002):

   a) the results obtained by DDSRM are in very close agreement with the dynamic model. The peak flow simulated by DDSRM is slightly higher than that obtained by the dynamic model in the case of the sharp rising hydrograph (T_o = 20 minutes). The difference, however, is very small. The difference in peak flow approaches zero as the duration of hydrograph increases. The time to peak is predicted perfectly for all hydrograph durations.

   b) for all hydrograph durations, the error in continuity is very small for practical purposes.

2. For the milder slope (S_o = 0.0005) the agreement between DDSRM and the dynamic model is not good. The maximum deviation in flows (DDSRM predicts larger flows) is obtained when the mild slope is combined with a sharp rising hydrograph. This deviation, however, is reduced as the hydrograph duration increases. A similar trend can also be observed with respect to the time to peak.

3. It appears from the above results that the channel slope is a more significant parameter than the wave period where the applicability of DDSRM is concerned.

These results are substantiated by the dynamic and kinematic wave theories.

Based on the above experiments, the validity of the simplified routing method will be discussed in terms of the minimum pipe slopes used in urban storm drainage systems.

TABLE 4.2 Slopes for Full-Pipe Velocity of 2 fps (0.6 m/s) and 3 fps (0.9 m/s)

Pipe Diam.		Approximate Pipe Slope - Per Thousand					
		v = 2 fps (0.6 m/s)			v = 3 fps (0.9 m/s)		
in	mm	n=.013	n=.014	n=.015	n=.013	n=.014	n=.015
12	300	1.9	2.2	2.6	4.4	5.1	5.8
18	450	1.1	1.3	1.5	2.55	3.0	3.4
24	600	0.8	0.9	1.0	1.7	2.0	2.3
36	900	0.45	0.5	0.6	1.0	1.2	1.4
48	1200	0.3	0.35	0.4	0.7	0.8	0.9
60	1500	0.2	0.25	0.30	0.5	0.6	0.7

In storm sewer systems, the pipe slopes are limited by two factors: (a) minimum velocity to prevent deposition, and (b) maximum velocity to prevent erosion. Commonly, the minimum slope is calculated such that, when flowing full, the velocity is 2 fps (0.6 m/s) (Wright-McLaughlin, 1969) to

3 fps (0.9 m/s) (WPCF and ASCE, 1970). The slopes required for minimum velocity will depend on the shape and roughness of the conduit. Table 4.2 gives the pipe slopes required for full-flow velocities of 2 fps (0.6 m/s) and 3 fps (0.9 m/s), for a range of pipe diameters, based on Manning's formula with n values of 0.013, 0.014 and 0.015. Steeper slopes may be required to obtain the desired velocities in the upper reaches of lateral sewers which will flow only partly full even for the ultimate design flow (WPCF and ASCE, 1970). Steeper slopes may also be required to obtain self cleaning velocities for flows smaller than the design flows.

Table 4.2 shows that, theoretically speaking, slopes much smaller than the 0.2%, for which good agreement between the simplified and dynamic models was found (Fig. 4.11), are possible. However, in practical design of storm sewers, steep slopes will always be desirable. By increasing the slopes, smaller pipe sizes can be selected to carry the design flow, with the result of saving in the cost of the sewers. The maximum slopes will be governed by the maximum allowable velocity. In storm sewer systems, the maximum velocity can be quite high without causing erosion problems (WPCF and ASCE, 1970) since the maximum flows, by their nature, occur infrequently. However, slopes greater than the critical slope will increase the likelihood of the occurrence of hydraulic jumps. Steep slopes with flows which have Froude numbers greater than 2 may result in the occurrence of roll waves.

The Borough of Scarborough (1979) has put a limit on the maximum slope "that all future sewer designs be set at slopes less than critical slope". The Borough also implied that slopes less than 0.2% shall not be used. It is not known to the writer, however, if other Canadian municipalities have similar regulations. Several new subdivisions in Ontario have been examined and it was found that the pipe slopes were much greater than the minimum slopes. In fact, no slopes less than 0.2% were found in the sewer systems examined.

It can, therefore, be stataed that, for normal design conditions, the model will give reasonably accurate results. In the case of very mild slopes (say less than 0.1%), which may be used in flat areas, one should expect a conservative design.

Note that the above experiments have been conducted for only one pipe size (a rather large, 8 ft (2.44 m) diameter), and the comparisons of the discharge hydrographs are made at a fairly long distance of 5000 ft (1525 m) down the pipe. In practical applications, where the pipes are much shorter, the results should improve. Also, for smaller pipe diameters the accuracy will improve (NWS, 1981).

4.6.3 Validation in a Sewer Network

It is shown above that the free surface flow routing technique of DDSRM is capable of accurately representing wave propagation along a channel/sewer for the conditions usually encountered in urban drainage problems. However, it is important to verify the model performance in a network situation. This can be based on comparisons with a dynamic model for sewer networks which has been previously verified. An improved version of the EXTRAN model (Kassem and Roesner, 1979a; Roesner, kassem and Wisner, 1980) is selected for this purpose. The boundary condition at the junctions in EXTRAN is essentially the same as in DDSRM, that is, continuity considerations, neglecting the energy losses at the junctions.

A hypothetical test scheme (Testville 4.1) indicated in Fig. 4.14 is used for the comparison. Testville 4.1 is derived from an existing storm sewer system. In this scheme the sewer network is dendritic and converging towards the downstream outlet. The test is conducted by first generating surface runoff from the subwatersheds using a 5-year "Chicago" storm distribution as simulated by the runoff block of SWM model. These flows are then admitted directly (unrestricted) into the sewer network at the sewer junctions, as shown in Fig. 4.14, and routed in the sewer network using the two models. The sewer sizes are chosen such that free surface flow prevails all through the pipe system.

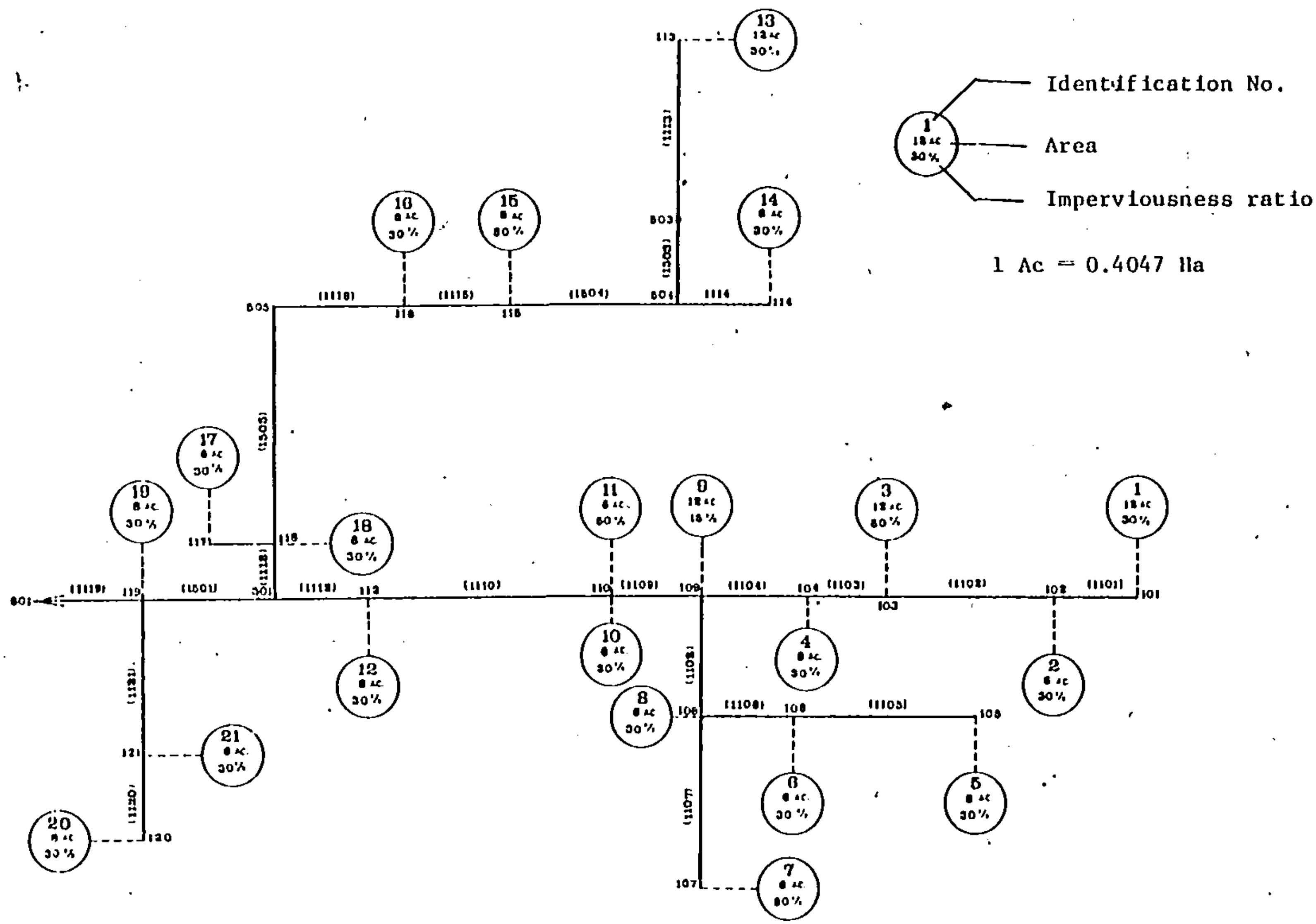
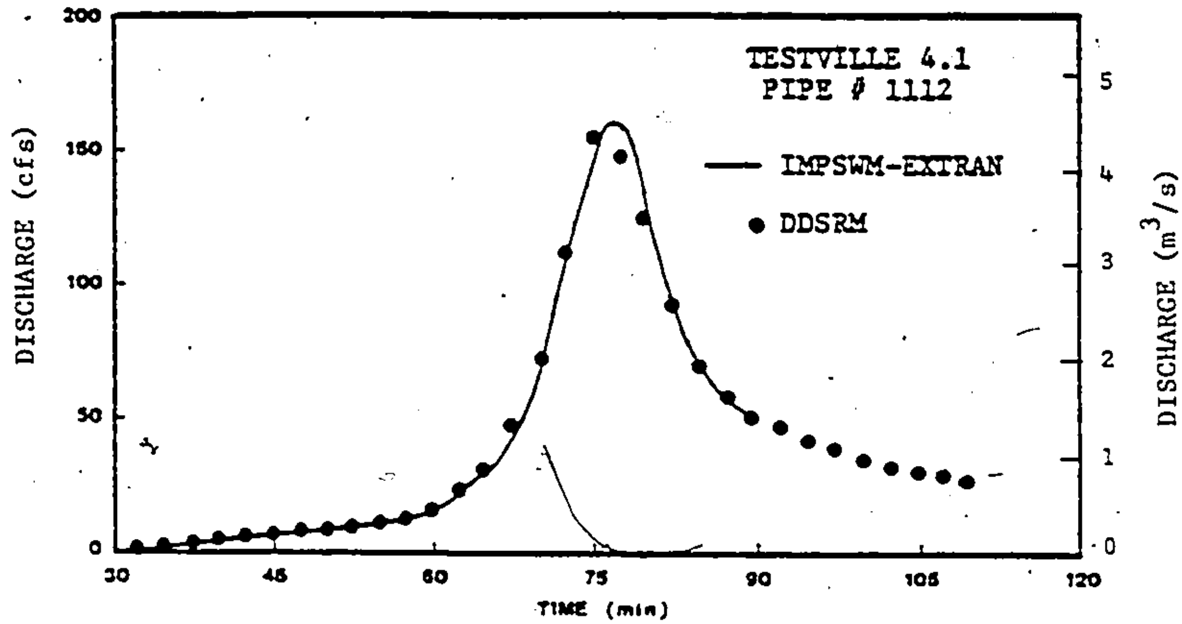


Fig. 4.14 Schematic Layout of Testville 4.1

The time history of sewer flow obtained by the two models is compared, at two locations, in Fig. 4.15. It can be seen that the results obtained by DDSRM are in very good agreement with the dynamic model with respect to both peak flow and time to peak. Similar agreements have also been obtained throughout the network. Since the two models are tested under identical conditions and have similar assumptions for the network, the comparisons given in Fig. 4.15 reflect the effect of routing. These results should, therefore, provide a proof of the accuracy of the network routing sub-model of DDSRM.

4.7 ADVANTAGES AND LIMITATIONS

The numerical experiments presented in section 4.6 show that, for the conditions expected in urban storm drainage systems, the routing model presented in this chapter will give results which are very close to those obtained by dynamic models. The solution is carried out by using an algebraic equation rather than differential equations. The model is therefore computationally fast. For example, the execution time for the simulation carried out for Testville 4.1 (Fig. 4.14), on the IBM 360/370 machine, is less than 14 seconds, as compared to more than one minute required for the dynamic model.



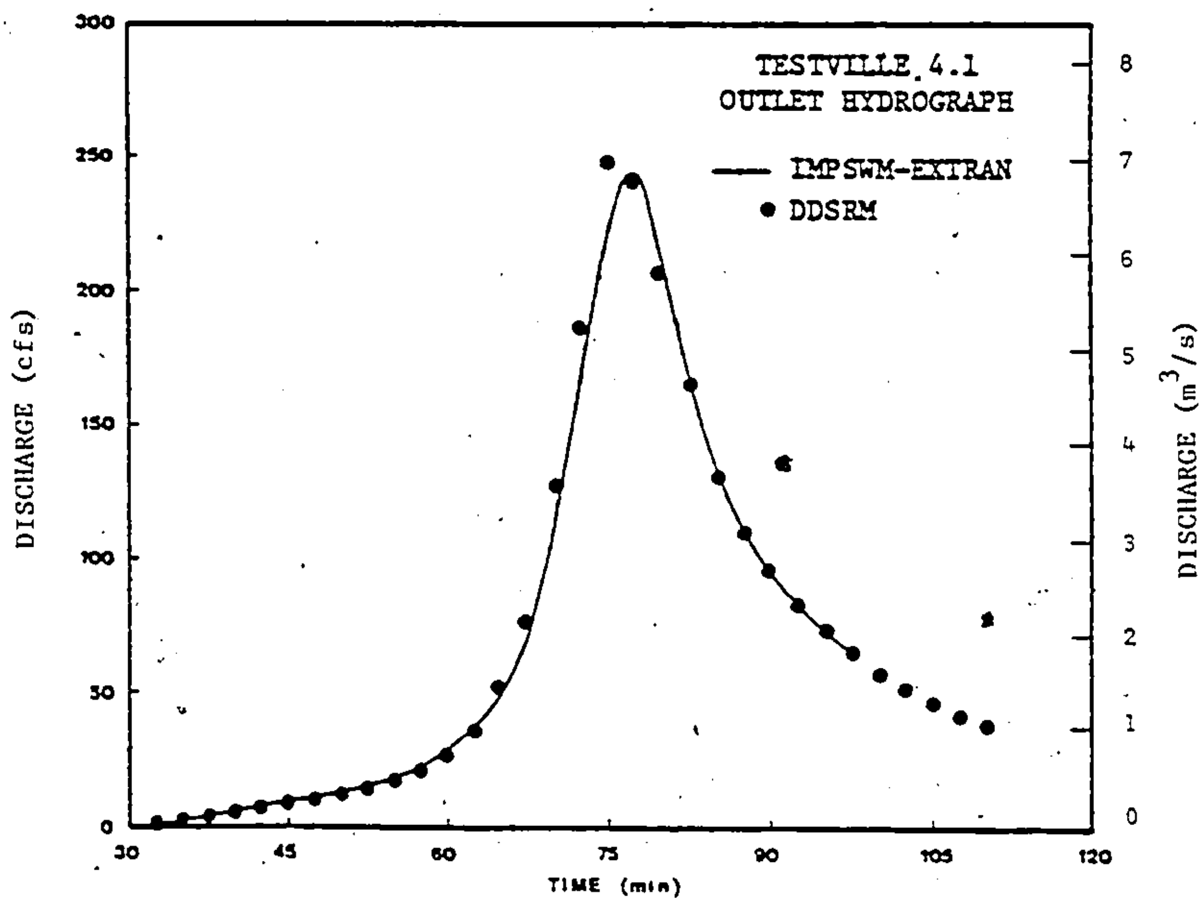


Fig. 4.15 Validation of DDSRM for Sewer Networks - Comparison With A Dyanmic Model

The proposed model allows the wave attenuation to be represented which cannot be done by direct solutions of the kinematic wave equations. The solution scheme is as simple as the Muskingum method, while its parameters are related to the physical and hydraulic characteristics of the channel. Furthermore an advantage of the numerical solution is that it allows the model parameters (in particular the wave celerity) to vary with discharge. Although the model has been tested only for circular channels, the results should be equally valid for other channel geometries.

It has been recently recognized that storms of short duration are critical for urban drainage studies (Wisner, Gupta and Kassem, 1980). It follows from section 4.6.2 that the associated runoff may undergo significant attenuation. This observation may be utilized to justify the use of dynamic routing models for urban drainage applications. Due to the substantially higher computational cost of dynamic models, a more appropriate alternative is the model proposed here. It should also be noted that oversimplification in the routing models, such as the use of time-lag methods incorporated in some of the existing models will, most probably, overestimate the flows.

The utilization of this improved routing method in DDSRM will also make the model suitable for applications in master drainage plans where the effect of routing may be very significant, due to the increased size of the drainage system.

Although the proposed model has been addressed here mainly for applications in urban storm drainage systems, it should have significant advantages for other applications, provided that the combination of channel slope and hydrograph shape are not critical. Additional analysis of the model may also attempt to combine the channel slope and hydrograph duration into a single parameter similar to that of Eq. 3.32. Specific criteria for model applicability can then be established.

The model, however, cannot simulate backwater effects nor can it simulate energy losses at the junctions in a network system. If backwater effects are considered significant, the dynamic routing sub-model presented in the next chapter can be utilized (only for the sewer system), which also has the capability of simulating sewer surcharge.

Chapter V

DYNAMIC FLOW ROUTING IN SURCHARGED SEWERS

5.1 INTRODUCTION

One of the requirements of the present research is a methodology for simulating sewer surcharge. As discussed in section 3.5.1, surcharge simulation should be undertaken in conjunction with a dynamic routing model. Rather than developing a wholly new model for these conditions, it is considered that this goal can be achieved by improvements of an existing model.

The EXTRAN model (Kibler et al, 1974) is chosen for this purpose for three basic reasons:

1. being the only accessible operational and non-proprietory dynamic routing model for sewer networks (at the time of commencing this study);
2. possibility of obtaining assistance from one of the model authors,[2] if required; and
3. possibility of easy implementation of eventual improvements because of familiarity by engineers with its principles in Canada.

---

[2] Dr. Larry A. Roesner from Camp Dresser and McKee, Inc., Springfield, VA.

Experience with EXTRAN in Canada and U.S. shows that one of its main drawbacks is the occurrence of numerical instability problems. Yen (1977) has referred to the model as "notoriously known for its numerical instability".

A research program for improvement of EXTRAN is therefore undertaken. The objective of this research is twofold: (1) to establish the sources of numerical instability; (2) to revise some aspects of the model such that an improved, numerically stable and verified version of the model can be produced.

A series of systematic numerical experiments with EXTRAN showed that numerical instability occurred mainly (a) in the case of surcharge, (b) with flow diversions by orifices and weirs and (c) in the case of the existence of large drops. This research also discovered that the existing methodology in EXTRAN for the simulation of sewer surcharge is inadequate and may in many cases produce misleading results, as will be discussed below.

5.2 SCOPE OF PRESENT INVESTIGATION

5.2.1 Background and Basic Principles

The EXTRAN model was developed for the analysis of complex sewer network systems including looped sewers, flow diversions by weirs, orifices, and pumps, and backwater effects. The original version was developed by CDM/Water Resources Engineers (Kibler et al, 1974) under the name "San Francisco Urban Storm Water Model". During subsequent years revisions were made by the model developers and by other organizations. The main objectives of these efforts focussed on improvements and additions to the model capabilities of simulating specific conditions or particular systems. At the present time there are several versions of the model under a variety of names. The basic technique in all these model versions, however, remains the same.

A conceptual representation of EXTRAN model is shown in Fig. 5.1. The model is based on a link-node description of the sewer system. Links represent pipes and flow diversions. The nodes correspond to manholes or pipe junctions in the physical system. Sewer flow routing is based on the Saint Venant equations of momentum and continuity. The numerical solution scheme for unsteady flow routing by EXTRAN is given in Appendix B.

5.2.2 Surcharge Simulation in Previous Versions of EXTRAN

All versions of EXTRAN simulated sewer surcharge based on an analogy with a surge tank. The surcharged sewer junction (node) was therefore considered as a storage element which had relatively large storage volume. While all previous versions of the model applied the same concept for sewer surcharge calculation, each assumed different configuration

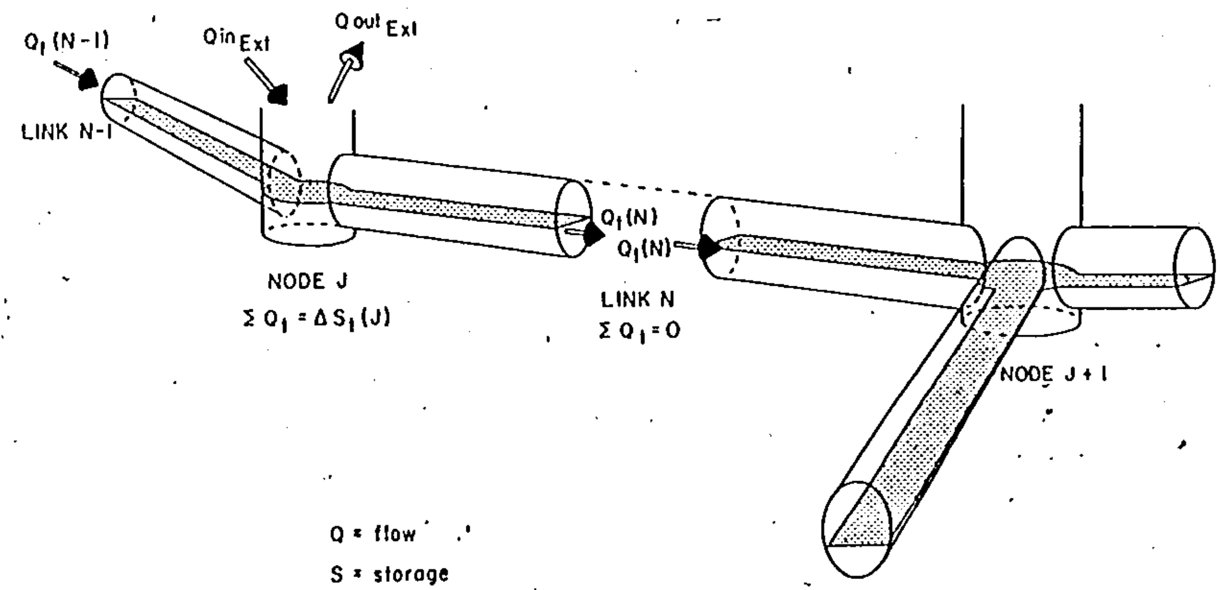


Fig. 5.1 Conceptual Representation of EXTRAN Model

of the "artificial" storage node geometry. Examples of storage node assumptions are presented in Fig. 5.2. Flow routing under surcharge was carried out in exactly the same manner as for free surface flow conditions. The artificial storage node was used to obtain some "junction surface area" to be used in the numerical solution scheme developed for free surface flow (Appendix B).

The above scheme is based on unrealistic assumptions which have no physical or theoretical basis. Notably the geometry of the "storage node" was related to the ground elevation (Fig. 5.2). On the other hand, it appears that the main objectives of the various attempts at altering the shape of storage node aimed mainly at avoiding numerical instability rather than accuracy. Not surprisingly, it followed that various versions of the model predicted different surcharge levels when tested under identical conditions (Kassem and Roesner, 1979b), as can be seen in Fig. 5.3.

5.2.3 An Improved Version of EXTRAN

The present research resulted in a new improved version of EXTRAN (Roesner, Kassem and Wisner, 1980). The main features of the improved model are: (1) a new algorithm for the simulation of sewer surcharge, including a methodology for the simulation of the transition from free surface to pressurized conditions; and (2) new algorithms for the simu-

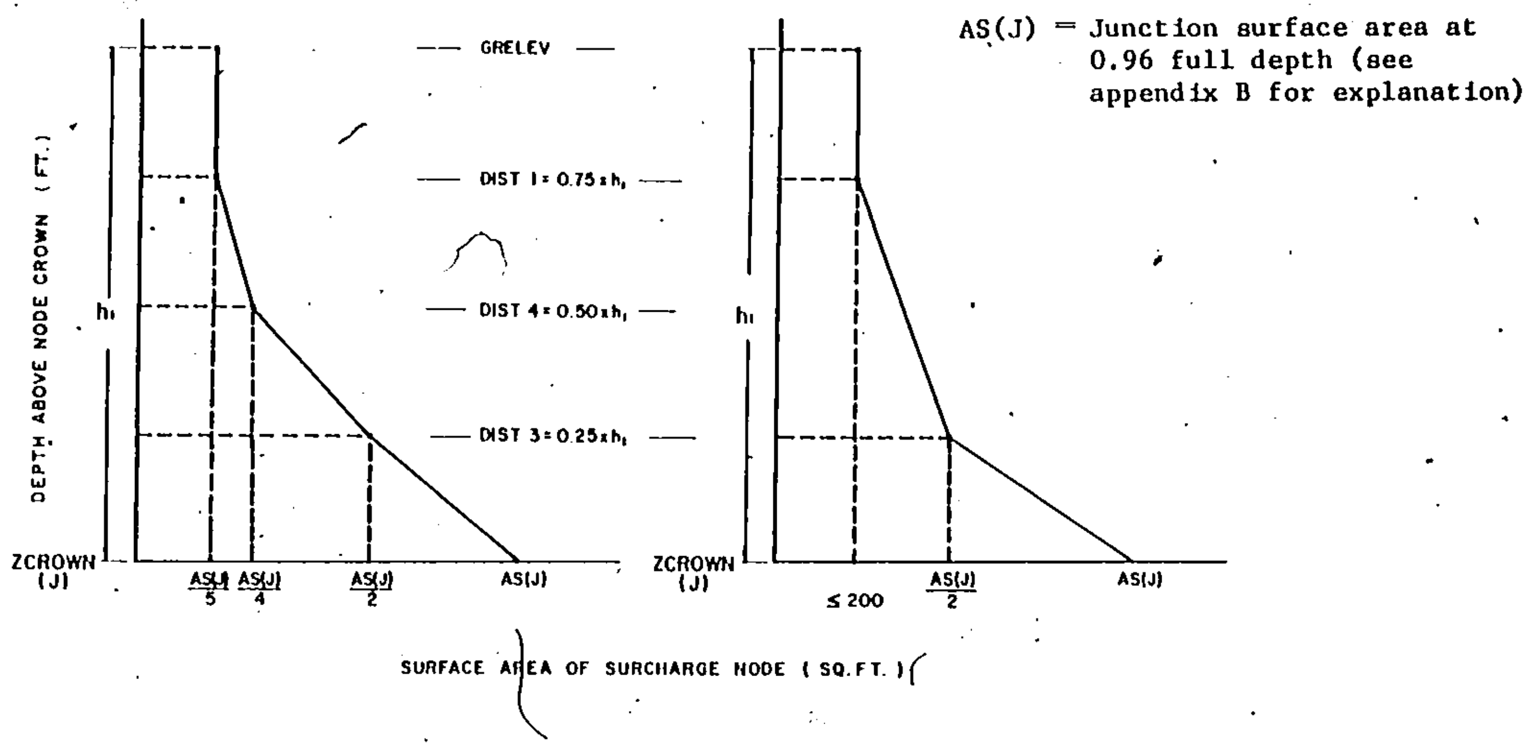


Fig. 5.2 Examples of Storage Node Assumptions for Surcharge Simulation in Previous Generations of EXTRAN Model

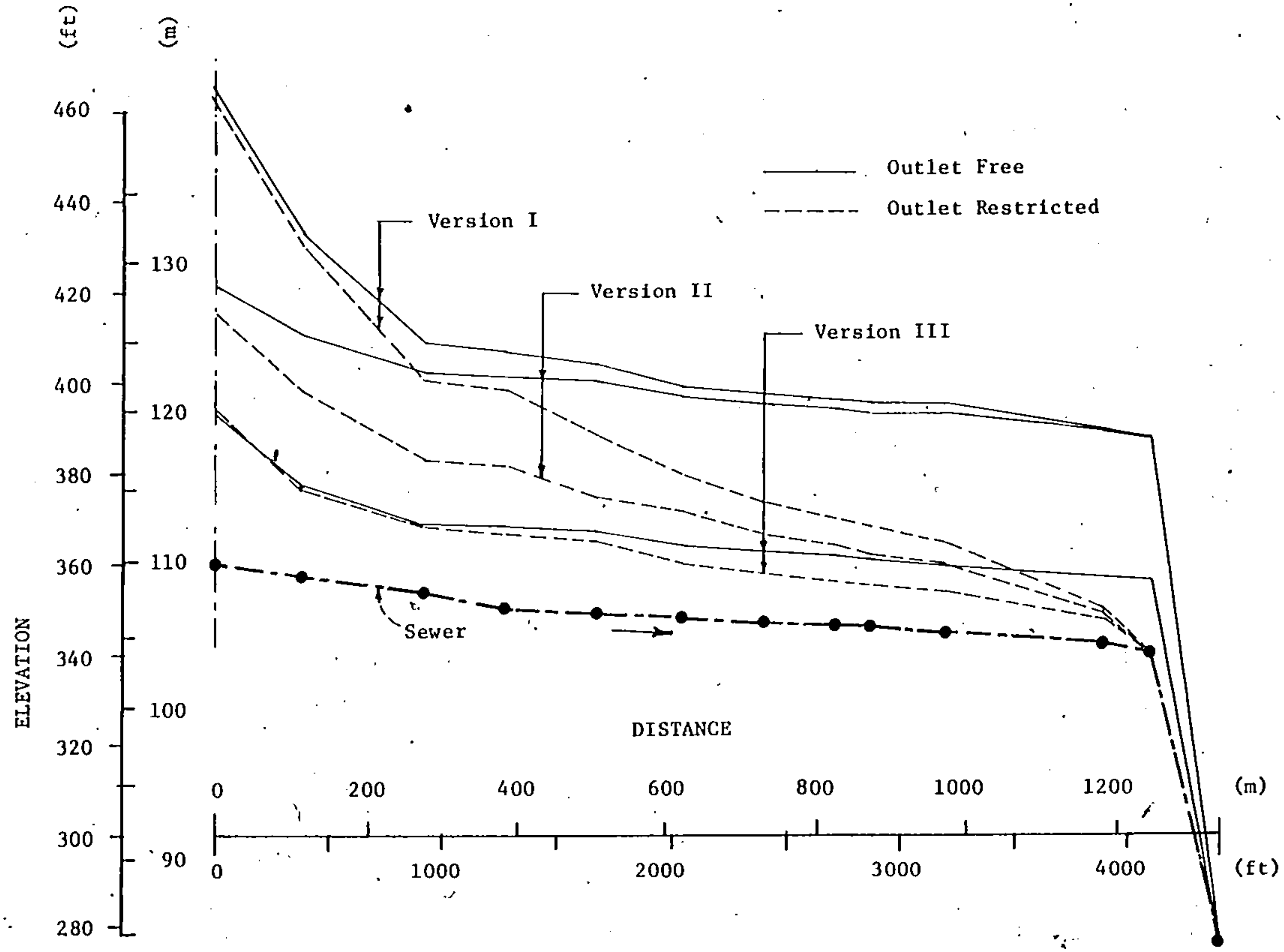


Fig. 5.3 Prediction of Maximum Surcharge Levels Obtained By Three Different Assumptions for Storage Node Geometry

lation of flow diversions by orifices and weirs. The possibility of the occurrence of numerical instability is also considerably reduced. In addition, users' oriented error messages and improved print-out have also been incorporated into the model.

In the context of the present study, the improved version of EXTRAN will be referred to as "IMPSWM[3] -EXTRAN". IMPSWM-EXTRAN will be used as a "surcharge" sub-model manipulated by the main program of DDSRM (see chapter VI).

For the purpose of the present study the emphasis will be given to the new methodology for the simulation of sewer surcharge, together with a series of numerical experiments for its validation. However, it is important first to validate the basic routing technique of EXTRAN.

5.3 VALIDATION OF ROUTING TECHNIQUE

Validation of the link-node solution for the sewer flow routing technique of EXTRAN model (Appendix B) is carried out for free surface flow conditions. Several tests are conducted on hypothetical pipe systems and hypothetical inflow hydrographs. The routed hydrographs obtained by means of IMPSWM-EXTRAN are compared at selected locations down the

[3] IMPSWM is a research program for the "Implementation of Storm Water Management Models", organized at the University of Ottawa. The writer acted as an assistant coordinator in the program which started early in 1979. IMPSWM is supported by a group of governmental agencies, municipalities and consultants.

pipe with other dynamic routing models.

Fig. 5.4a presents a comparison against a solution by the method of characteristics (Yevjivich and Barnes, 1970). In this application the pipe is a 6.0 ft (1.83 m) diameter, 30,000.0 ft (9150 m) long on a slope of 0.001 and Manning's n of 0.012. The test arrangements are identical to Scheme 4.1 used for the validation of the approximate flow routing model of DDSRM - Section 4.6.1. The pipe is divided into 30 reaches, 1000.0 ft (300 m) each. Fig. 5.4a compares the routed hydrographs at a distance of 5,000.0, 18,000.0, and 30,000.0 ft (1,525, 5,490 and 9,150 m) down the pipe, which indicate an excellent agreement. Similar agreements have also been obtained in comparison with a four-point implicit scheme.

The model is further tested for more critical condition by routing a hypothetical sinusoidal wave through a 8000 ft (2440 m), 6 ft (1.83 m) diameter pipe on a small slope of 0.0006. The inflow hydrograph has a small duration and is represented by

$$\left.\begin{aligned} I(t) &= 50 - 30 \cos(0.314\, t) \quad ; \quad 0 < t \leq 20 \\ I(t) &= 20 \quad ; \quad t > 20 \end{aligned}\right\} \text{for } I(t) \text{ in cfs} \qquad 5.1a$$

$$\left.\begin{aligned} I(t) &= 1.416 - 0.85 \cos(0.314\, t) \quad ; \quad 0 < t \leq 20 \\ I(t) &= 0.566 \quad ; \quad t > 20 \end{aligned}\right\} \text{for } I(t) \text{ in } m^3/s \qquad 5.1b$$

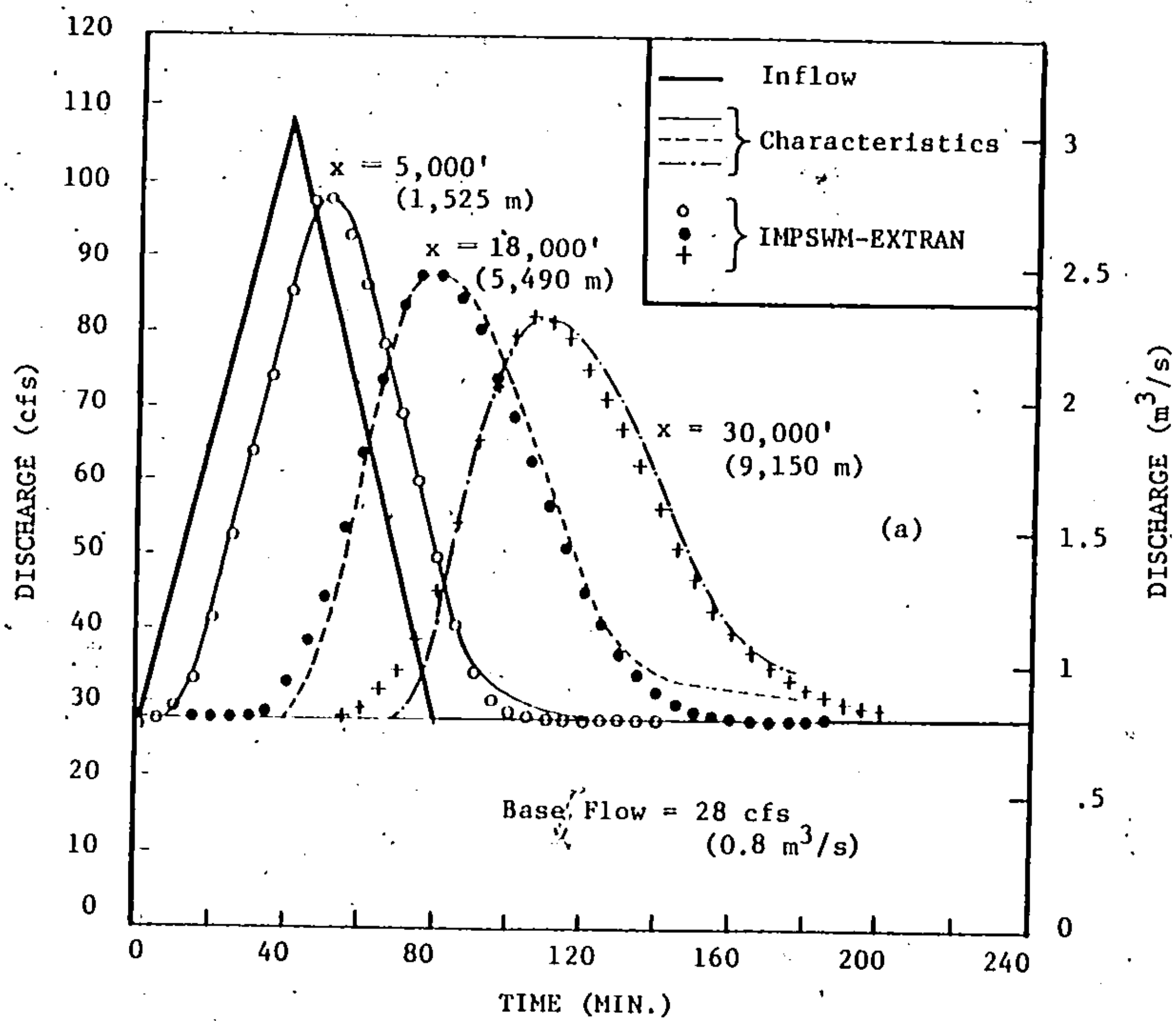
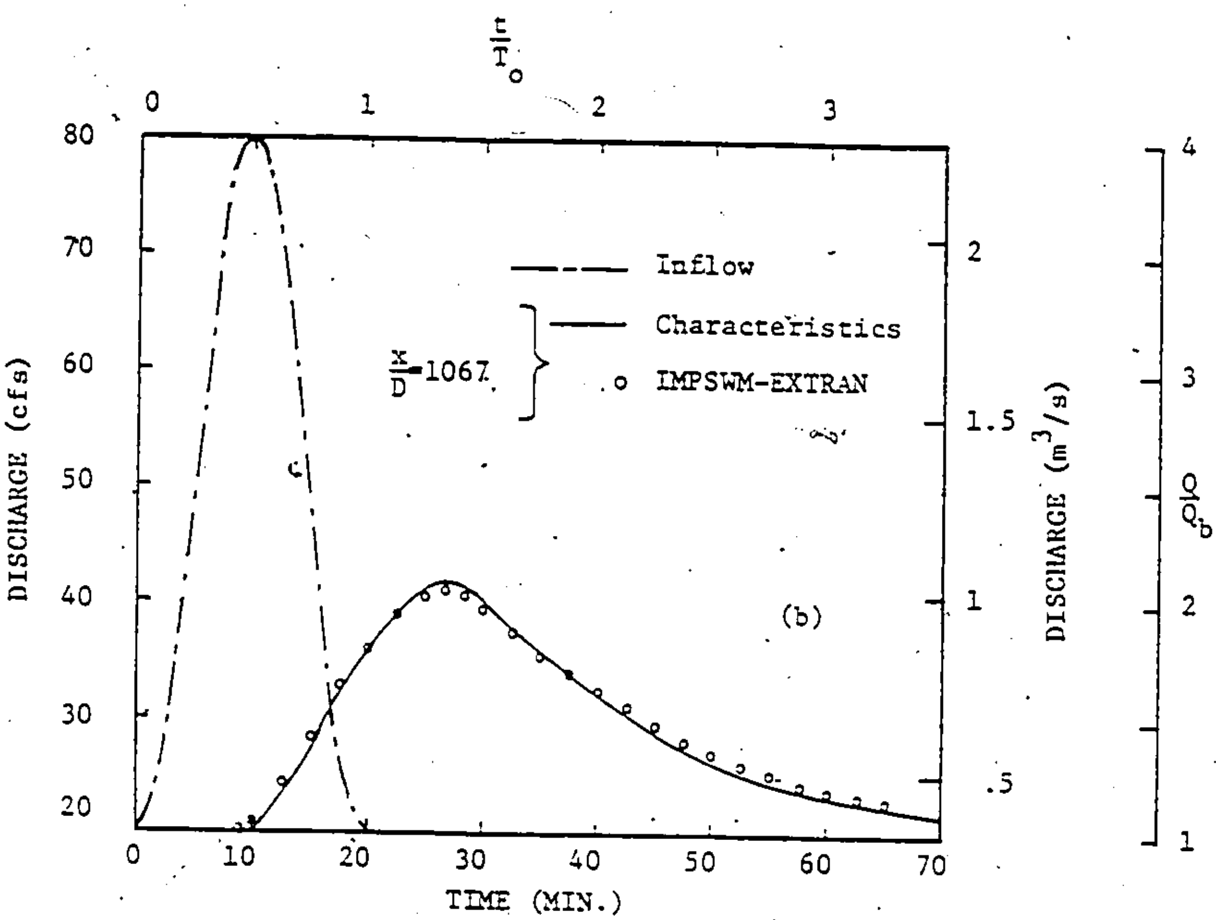


Fig. 5.4 Validation of the Routing Technique of EXTRAN Model For Free Surface Flow Conditions - Comparison With the Method of Characteristics



t/T_o
0
1
2
3
80
70
60
50
40
30
20
DISCHARGE (cfs)
Inflow
Characteristics
IMPSWM-EXTRAN
x/D=1067
(b)
2
1.5
1
.5
DISCHARGE (m^3/s)
4
3
2
1
Q/Q_b
0
10
20
30
40
50
60
70
TIME (MIN.)

Fig. 5.4 Cont'd

where I(t) is the inflow in cfs and t is the time in minutes. Fig. 5.4b compares the routed hydrograph obtained by IMPSWM-EXTRAN with that obtained by the method of characteristics (Yen, 1973) at a distance of 6400 ft (1950 m) down the pipe.

These tests provide a proof of the accuracy of the routing technique of EXTRAN model for free surface flow conditions.

5.4 METHODOLOGY FOR SEWER FLOW ROUTING UNDER SURCHARGE

A new methodology for extending the free surface flow routing technique of EXTRAN to account for surcharged conditions is developed to replace the unrealistic storage node assumptions adopted in previous generations of the model (section 5.2.2). The new solution scheme is based on a modified Hardy-Cross method. The basic methodology was first suggested by Roesner (1979). Significant effort was, however, required for its incorporation into the model. The new method is presented below prior to the description of a series of numerical experiments for its validation.

In the new scheme a junction is assumed to be surcharged when the nodal water level exceeds the crown elevation of the highest pipe entering or leaving the junction. Energy losses at the junction and other minor losses are neglected. The junction water level is therefore represented by the piezometric head, which, in turn, is assumed to represent the piezometric head of all pipes at the junction (Fig. 5.5).

During surcharge, the continuity equation for node j at time t is

$$\Sigma Q\,(t) = 0 \qquad 5.2$$

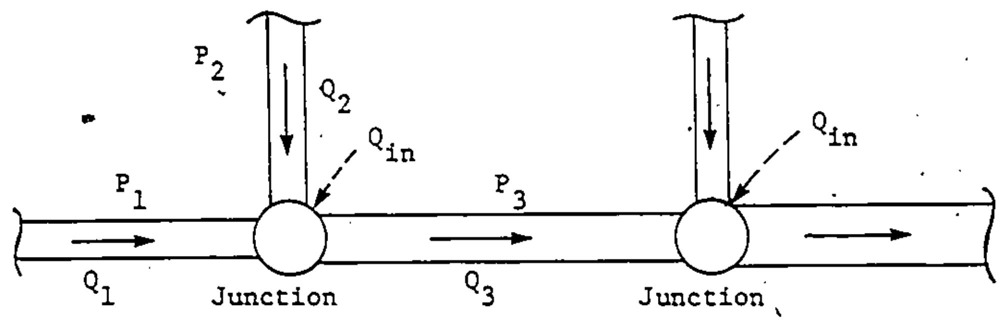
where $\Sigma\, Q(t)$ is the algebraic sum of all inflows into and ouflows from the node (see Fig. 5.5).

Since the momentum and continuity equations are not solved simultaneously (Appendix B), the flows computed in the links connected to node j may not satisfy Eq. 5.2. However, by computing $\partial Q/\partial H$ for each link connected to node j, a head adjustment can be obtained such that the continuity equation is satisfied. Rewriting Eq. 5.2 in terms of the adjusted head gives:

$$\Sigma[Q(t) + \frac{\partial Q(t)}{\partial H_j}\,\Delta H_j(t)] = 0 \qquad 5.3$$

which can be solved for ΔH_j as

$$\Delta H_j(t) = -\Sigma Q(t)/\Sigma\,\frac{\partial Q(t)}{\partial H_j} \qquad 5.4$$



$$Q_1 + Q_2 + Q_{in} - Q_3 = 0.$$

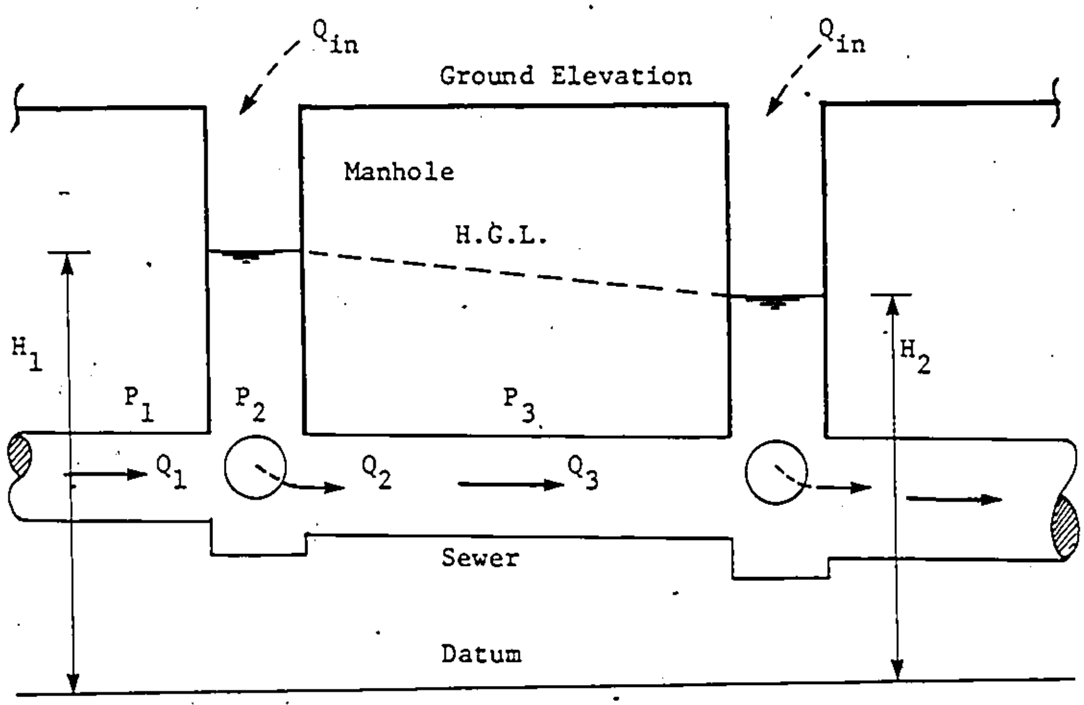


Fig. 5.5 Surcharged Flow In Storm Sewers

This adjustment is made by half-steps during surcharge so that the half-step correction is given as:

$$H_j(t + \frac{\Delta t}{2}) = H_j(t) + \Delta H_j(t + \frac{\Delta t}{2}) \qquad 5.5$$

where $\Delta H_j\ (t + \frac{\Delta t}{2})$ is given by Eq. 5.4, while the full-step head is computed as:

$$H_j(t + \Delta t) = H_j(t + \frac{\Delta t}{2}) + \Delta H_j(t) \qquad 5.6$$

where $\Delta H_j\ (t)$ is described by Eq. 5.4.

For a conduit connected to a node, $\partial Q/\partial H$ is computed as:

$$\frac{\partial Q(t)}{\partial H_j} = \frac{32.2}{1-K(t)} \quad \Delta t\ (\frac{A(t)}{L}) \qquad 5.7$$

where

$$K(t) = -\Delta t\ \frac{32.2\ n^2}{2.208\ R^{4/3}}\ |v(t)| \qquad 5.8$$

If the surcharge level at a junction reaches the ground elevation, the nodal water depth is set at this level. Inflows in excess of the pipe capacity under these conditions will be lost and will not be recovered.

5.4.1 Transition From Gravity to Surcharge Flow

The above scheme does not account for flow transition from gravity to pressure flow, that is, the dynamic process whereby the flow condition in a given pipe reverts between open channel and surcharge flow. This process will be accompanied by hydrodynamic instability (surges) as discussed elsewhere (Hamam and McCorquodale, 1980). These effects will be neglected here.

In order to avoid the occurrence of numerical instability at the beginning of sewer surcharge, a smooth transition from free surface to surcharge computations has been assumed (Roesner, Kassem and Wisner, 1980). The transition function is given by

$$\Delta H_j(t) = \frac{\Sigma Q(t)}{\delta} \qquad 5.9$$

where δ is given by

$$\delta = \Sigma \frac{\partial Q(t)}{\partial H_j} + \left(\frac{A_s}{\Delta t} - \Sigma \frac{\partial Q(t)}{\partial H_j}\right) e^{\frac{-15(y_j - D_j)}{D_j}} \qquad 5.10$$

and

A_s = the nodal surface area at 0.96 full depth

D_j = pipe diameter

y_j = water depth.

The exponential function causes equation 5.9 to converge within 2 percent of Eq. 5.4 by the time the water depth is 1.25 times the full flow depth.

With reference to Fig. 5.6, the computation of junction depth is performed using Eq. 5.9 from the time the junction first surcharges until surcharge level reaches 1.25 times the full flow depth. For higher surcharge levels, the head computation is carried out using Eqs. 5.5 and 5.6.

It is interesting to note here that, very recently, other researchers (Sjoberg, 1981; and Hoff-Clausen, 1981) have adopted (independently, in conjunction with different routing models) a similar transition function to simulate the flow transition from free surface to surcharge conditions and vice versa, in order to avoid numerical instability.

5.5 VALIDATION OF SEWER SURCHARGE SIMULATION

The difficulty in verifying sewer surcharge simulation is obvious since there are no measurements available under these conditions. There are, however, two basic considerations in a surcharged sewer system:

1. the outflow from a (surcharged) pipe should be equal to the inflow into the pipe, with the response almost instantaneous, that is "no time lag"; and

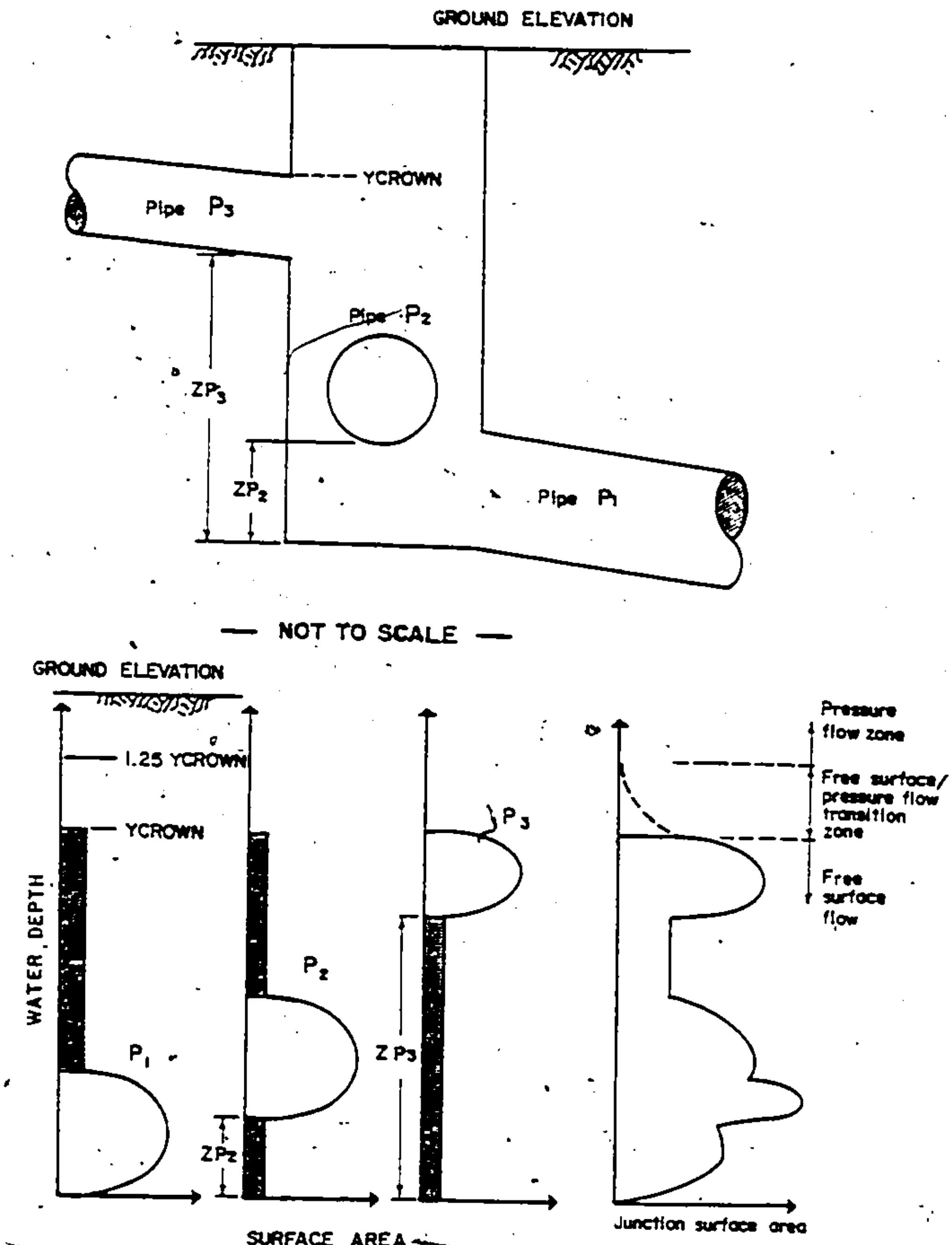


Fig. 5.6 Junction Water Depth vs. Junction Surface Area and Assumption for Transition from Gravity to Surcharge Flow

2. at any instance, the flow along a (surcharged) pipe should correspond to the instantaneous slope of the hydraulic grade line, which in turn is governed by the instantaneous surcharge levels at the upstream and downstream junctions.

Flow routing in sewer networks, under surcharge, also includes transition from free surface flow to surcharge conditions, and vice versa. It is also important to predict the duration of surcharge, that is the time-history of junction water level.

It is therefore considered in the present study that the assessment of the scheme outlined above for dynamic flow routing of free surface/surcharged flow in sewers can be obtained through a combination of the following:

1. comparisons with another sewer network model which has a capability of simulating sewer surcharge. The "proprietory" HVM model (Klym et al, 1972) was acquired for this purpose through an agreement with the Dorsch Consult in West Germany and the City of Toronto;
2. careful design of numerical experiments in order to enable a quantitative inspection of the performance of the two models. In this regard, the inclusion of steady inflow conditions makes comparisons with hand computations of the maximum hydraulic grade line possible; and

3. testing of various boundary conditions in a surcharged sewer system.

5.5.1 Description of Numerical Experiments

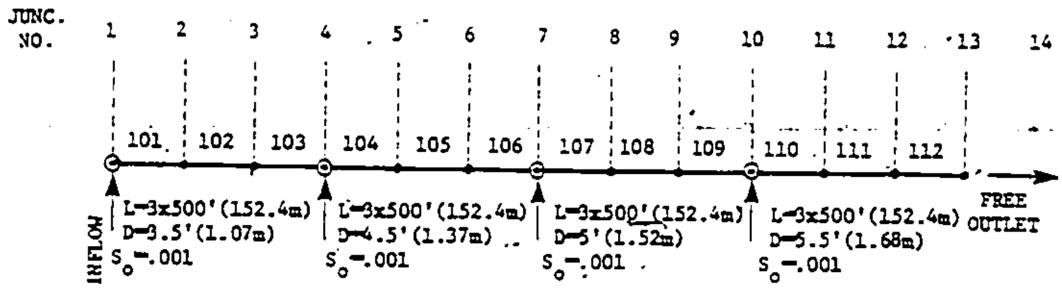
The validation tests of surcharge simulation are conducted on the hypothetical pipe systems indicated in Fig. 5.7. Four situations are considered, which will be referred to as Schemes 5.1 through 5.4.

Scheme 5.1 (Fig. 5.7 a): presents a situation where inflows are greater than the free flow capacities of the pipes.

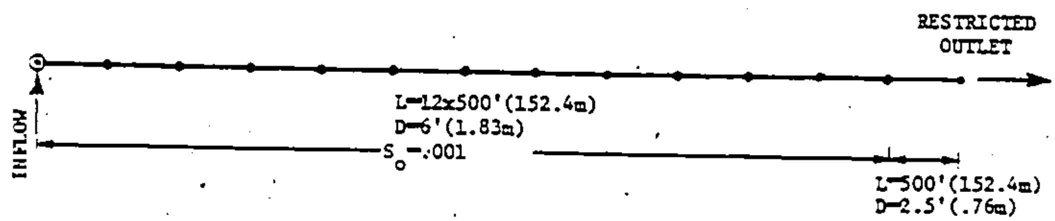
Scheme 5.2 (Fig. 5.7 b): presents a situation where sewer surcharge is forced by a constriction near the outlet of the pipe system.

Schemes 5.3 and 5.4 (Figs. 5.7 c and d): sewer surcharge is caused by a constriction in the pipe system created downstream of a 32.8 ft (10.0 m) drop. The size of the constriction is chosen such that: (a) only the pipe system downstream of the drop is surcharged (Scheme 5.3), and (b) the pipes both upstream and downstream of the drop are surcharged (Scheme 5.4).

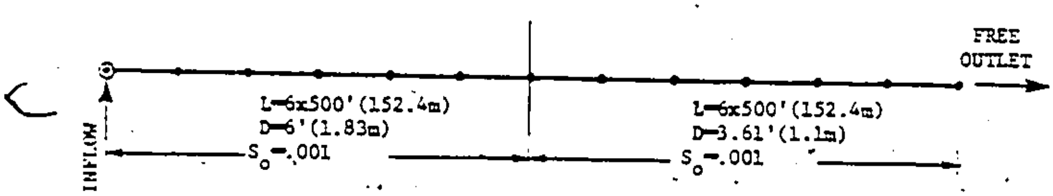
These arrangements should cover most situations in which sewer surcharge would occur. In all experiments the



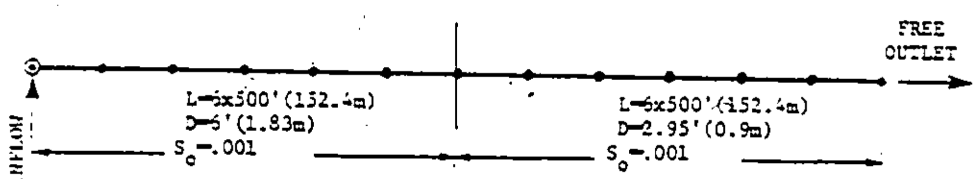
(a) SCHEME 5.1



(b) SCHEME 5.2



(c) SCHEME 5.3



(d) SCHEME 5.4

Fig. 5.7 Validation of Sewer Surcharge Simulation - Description of Test Arrangements outlet of the pipe system is free, that is, surcharge caused by submergence of the outlet is not considered.

Three different configurations of the inflow hydrographs are used for testing each scheme: TRG1, TRG2, and TRG3 as indicated in Fig. 5.8. The peak inflow used for testing each scheme is given in table 5.1. For the location(s) of inflows, reference should be made to Fig. 5.7. TRG2 and TRG3 include steady flow conditions so that comparisons with hand computations of the maximum (steady-state) hydraulic grade lines can be made.

5.5.2 Discussion of Results

The results of the above numerical experiments are presented in Figs. 5.9 through 5.12 for two shapes of inflow: TRG1 and TRG2. The results for TRG3 are given in Appendix C. For each scheme a comparison is made between the simulation results obtained by (1) the scheme outlined in section 5.4 (referred to in Figs. 5.9 through 5.12 as "DDSRM"), and (2) HVM model. These comparisons are given for (a) time history of junction water level at selected junctions, and (b) time history of pipe flow at selected pipes. Reference should be made to Fig. 5.7 for the locations of junctions and pipes.

The results of these experiments show that:

Table 5.1 Validation of Sewer Surcharge Simulation by IMPSWM-EXTRAN (DDSRM) - Specification of Inflows

Scheme	Inflows[(a)]	
	Q_1 cfs (m^3/s)	Q_2 cfs (m^3/s)
5.1	14 (0.4)	54 (1.53)
5.2	28 (0.8)	108 (3.06)
5.3	28 (0.8)	108 (3.06)
5.4	28 (0.8)	108 (3.06)

(a) refer to Fig. 5.7 for the location(s) of inflow

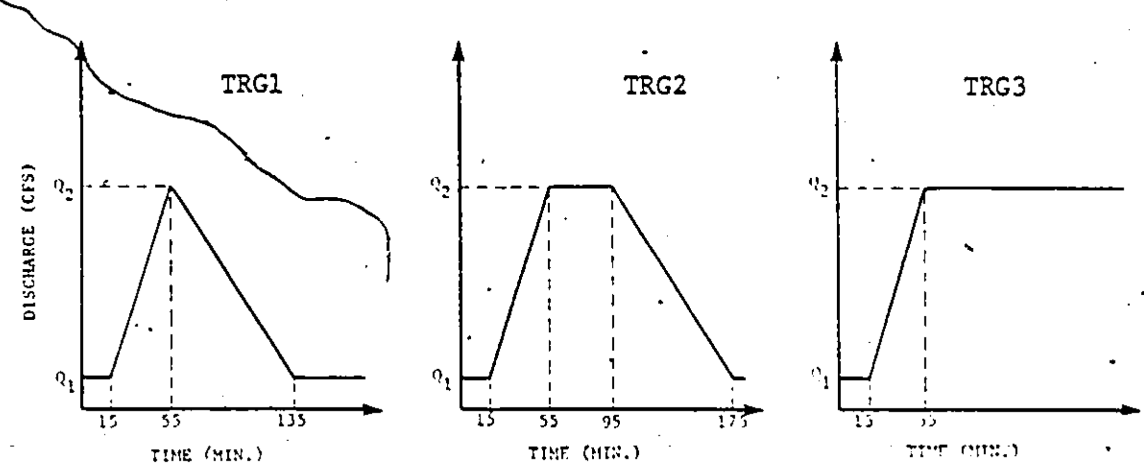


Fig. 5.8 Configuration of Inflow Hydrographs Used For the Validation of Sewer Surcharge Simulation

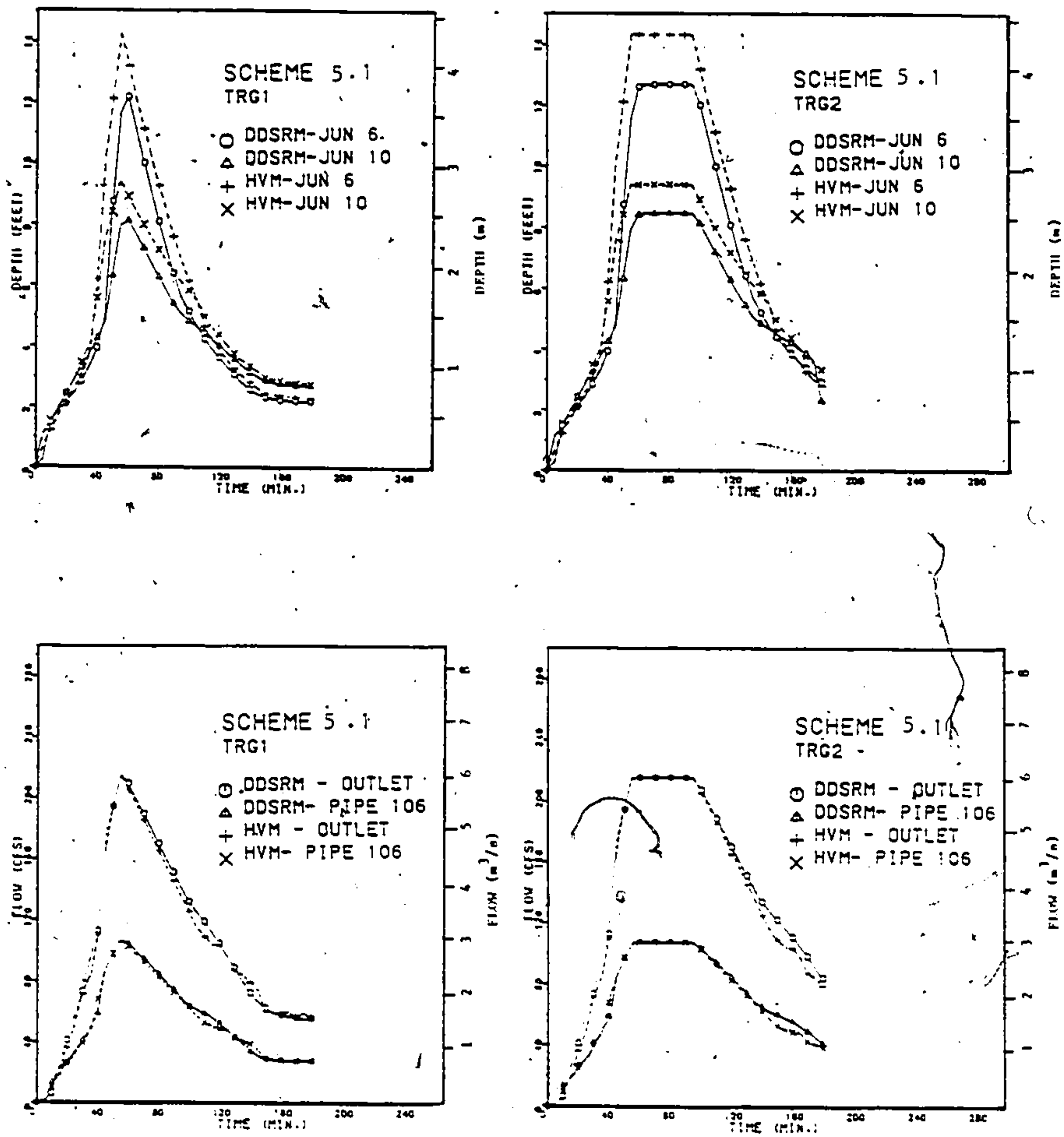


Fig. 5.9 Prediction of the Time Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.1; TRG1, TRG2

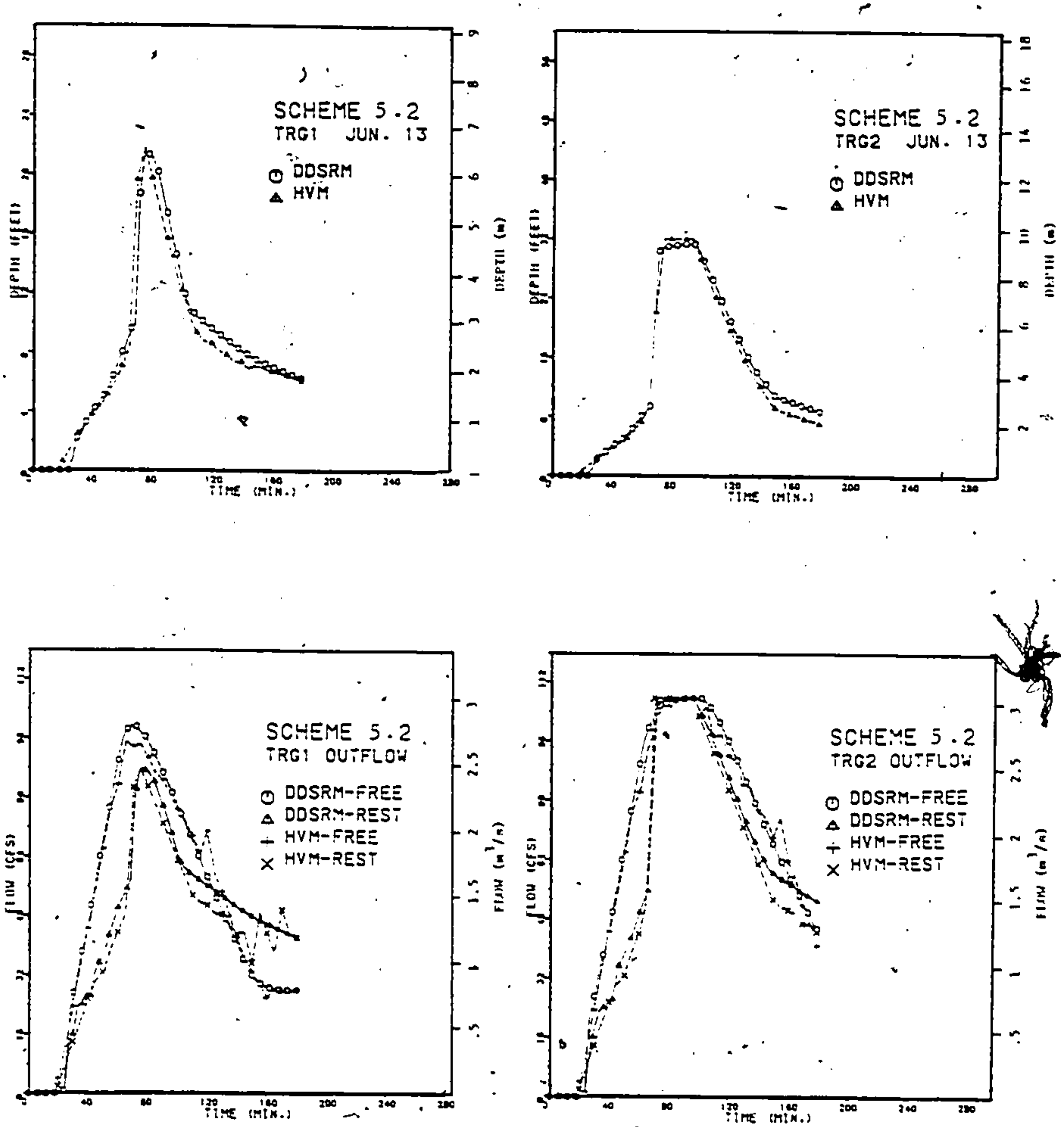


Fig. 5.10 Prediction of the Time Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.2; TRG1, TRG2

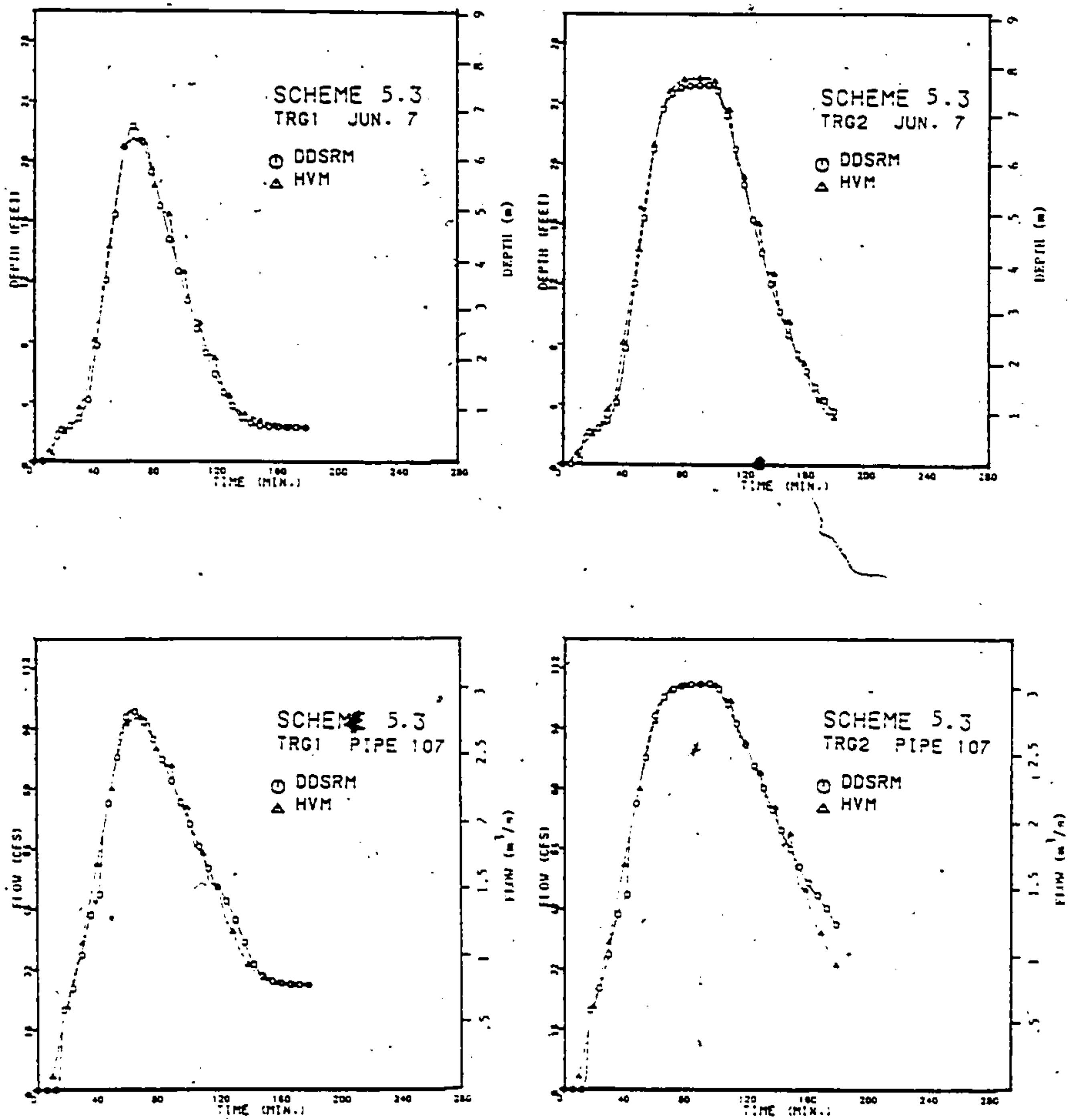


Fig. 5.11 Prediction of the Time Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.3; TRG1, TRG2

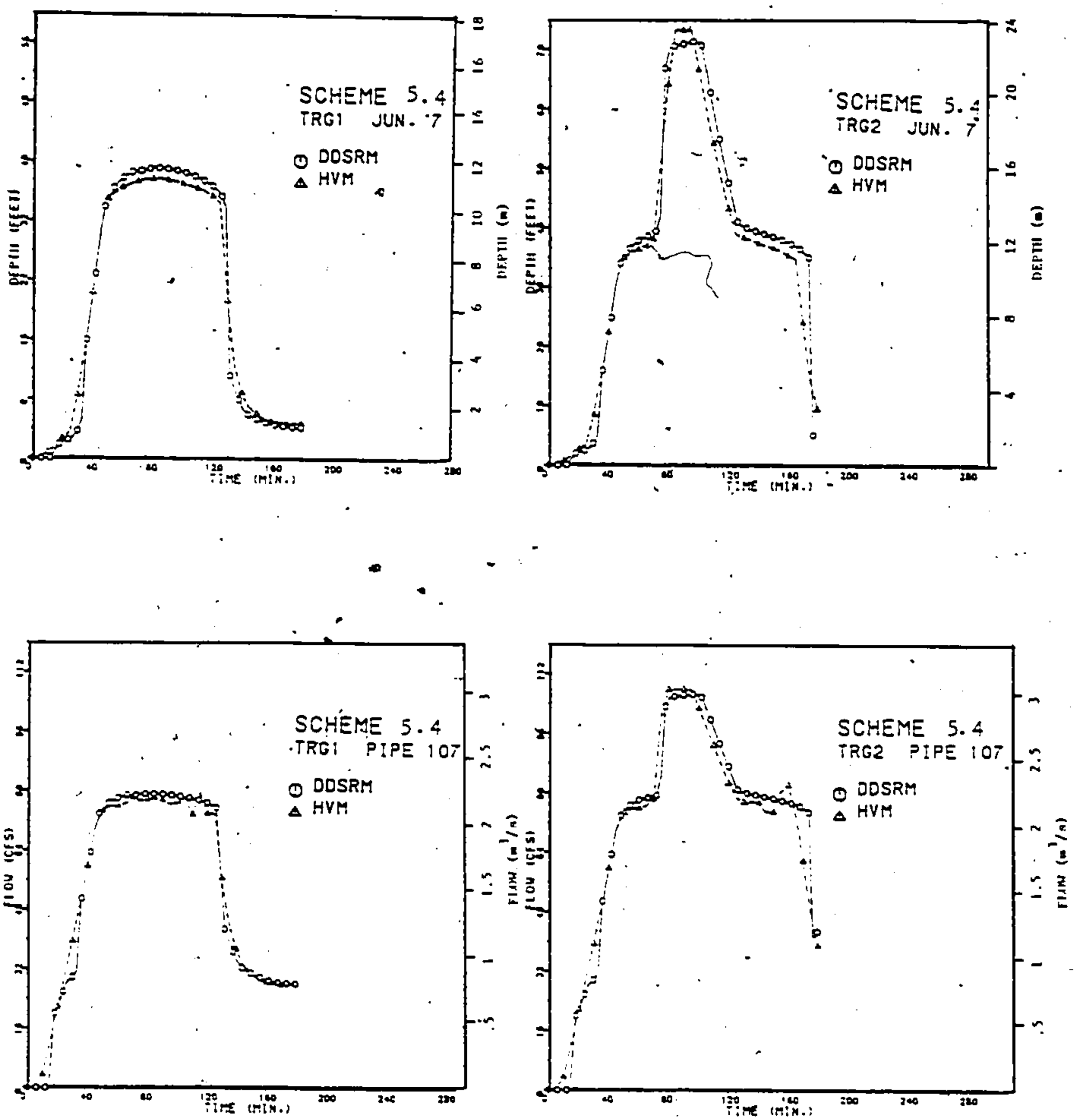


Fig. 5.12 Prediction of the Time Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.4; TRG1, TRG2

1. in all the tests the agreement between the simulated flows by the two models is excellent. Note that in Scheme 5.2 (Fig. 5.10), the comparison also includes a plot of outflow hydrographs without the constriction.
2. in the case of TRG2 and TRG3 (which include steady flow conditions), the slope of the maximum hydraulic grade line predicted by the two models is the same as would have been predicted by hand computation. Note that in both models the friction slope is expressed by Manning's equation. Thus for fully pressurized flows

$$S_f = \frac{n^2Q^2}{.2145\ D^{16/3}} \qquad 5.11a$$

using imperial units (D in ft and Q in cfs), or

$$S_f = \frac{n^2Q^2}{.097\ D^{16/3}} \qquad 5.11b$$

using SI units (D in m and Q in m^3/s)
3. with the exception of Scheme 5.1, the time history of junction water elevations and maximum surcharge levels predicted by IMPSWM-EXTRAN are in very good agreement with the results obtained by HVM model.

Note that in storm sewer systems, damage caused by surcharge will occur if, and only if, two conditions are satisfied: (a) surcharge levels are above basement elevation, and (b) these levels are sustained for a sufficient duration.

4. In Scheme 5.1 (Fig. 5.9) where the sewers have different sizes, flows, and velocities, a difference in the hydraulic grade line predicted by the two models can be observed. The discrepancy in results is due to the different assumptions at the junctions made in each model. In HVM the computation distinguishes between the total energy line and the hydraulic grade line. In IMPSWM-EXTRAN, on the other hand, the difference in kinetic energy is neglected. The maximum error to be introduced in the simulation of surcharge levels (between two points in a sewer network) by IMPSWM-EXTRAN due to this assumption will be equal to the absolute value of the difference in the velocity heads. This difference should be small for practical purposes.

5.6 CONCLUSIONS

It can be concluded from the above numerical experiments that the scheme outlined in section 5.4 is adequate for the simulation of sewer surcharge, including transition from free surface to pressurized conditions. The new scheme performs as well as the proprietory HVM model. It is numerically stable provided that the time increment, Δt, satisfies the Courant criterion.

The improved model (IMPSWM-EXTRAN) will be used as a "surcharge" sub-model of DDSRM - chapter VI. It is, however, considerably more expensive in terms of computer time than the free surface flow routing model of DDSRM presented in the previous chapter. Therefore it should be used only when it is necessary to calculate sewer surcharge.

IMPSWM-EXTRAN neglects the difference between the hydraulic gradient and the total energy lines. Further improvements of the model may consider accounting for this difference. There are also two basic limitations of the model: (1) the friction slope is calculated based on constant roughness coefficient (Manning's n), and (2) the energy losses at the junctions are neglected. Future improvements of the model may therefore consider varying the roughness coefficient. As regards the losses at the junction, their inclusion in the model is a more difficult task. No attempts are made herein to investigate this aspect.

Chapter VI

DEVELOPMENT OF THE DUAL DRAINAGE SIMULTANEOUS ROUTING MODEL (DDSRM)

6.1 INTRODUCTION

This chapter presents a detailed description of the "Dual Drainage Simultaneous Routing Model - DDSRM". The basic principles, the main assumptions of the model and the various sub-models will be discussed. The input data to DDSRM and the computer output will also be explained. The flow routing sub-models for free surface and surcharged conditions are presented in chapters IV and V, respectively. The listing of the computer program is given in Appendix F and a users' manual in Appendix D.

6.2 ASSUMPTIONS

The basic assumptions underlying the development of DDSRM can be summarized as follows:

1. the major system is formed by the streets;
2. the major system network follows a dendritic pattern and converges to a downstream outlet. The minor system on the other hand can be either dendritic or looped;

3. the sewer network and the street network are not necessarily parallel or flow in the same direction;
4. the major system should be continuous, without water ponding occurring on the streets. Changes in the street grades are small such that flow transitions and possible backwater effects can be neglected;
5. storm inlets are connected to sewer junctions. If an inlet is connected directly to a pipe, the captured flows will be assigned to the upstream junction;
6. the layout and the slopes of the streets and sewers are known a priori;
7. the characteristics of sub-areas on both sides of a street are similar resulting in the same runoff on each side.

The assumption of dendritic sewer system is applicable for free surface flow in connection with new designs. A looped sewer system is considered for surcharged conditions, in connection with the evaluation of an existing sewer system.

The only flow access into the sewers is assumed to be through the storm inlets. If the roofs are directly connected to the sewers (a practice which is seldom used at present), they can still be simulated separately for direct (unrestricted) access into the connecting sewers. Flows from backlot catchbasins can also be handled in a similar manner if their hydraulic performance is known.

6.3 MODEL OVERVIEW

An overview of the Dual Drainage Simultaneous Routing Model "DDSRM" is shown schematically in Fig. 6.1. The model is composed of four main sub-models:

1. surface runoff sub-model, which calculates the catchment supply rate by deducting rainfall abstractions (depression storage and infiltration losses), and then routes the rainfall excess on the catchment surface to the drainage channels;
2. inlet sub-model, which calculates runoff captured by the storm inlets, that is, inflow into the sewers, and the by-pass flow (carry-over), which represents the major system flow. Considerations are also given to the restriction of the capture capacity of the inlets;
3. minor system sub-model, which routes the inlet flows through the sewer network; and
4. major system sub-model, which routes the flows exceeding the capacity of the storm inlets through the street network.

In addition, sub-models for the design/analysis of storage facilities have been provided for the minor and major systems flows. Existing urban storm drainage models, in contrast to DDSRM, consider only items (1) and (3) above.

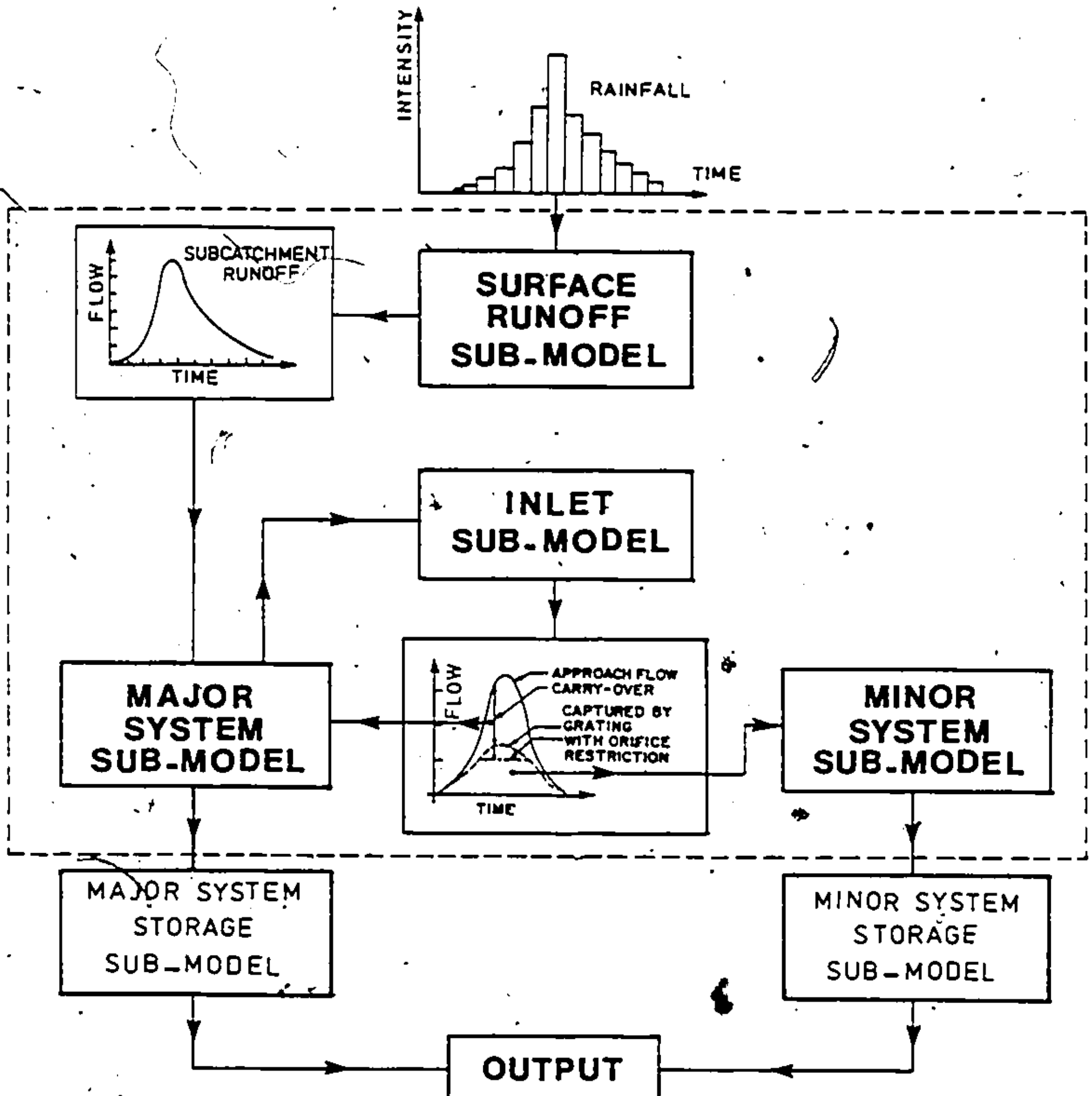


Fig. 6.1 An Overview of the Dual Drainage Simultaneous Routing Model (DDSRM)

As a design tool, the mathematical and logical formulation of DDSRM provides, among other things, the following information:

1. sewer sizes;
2. maximum flow depth (or spread) on the streets;
3. the locations of storm inlets which may need additional constrictions; and
4. storages for the minor and major system flows for the given outlet conditions.

Several options are provided in DDSRM for the evaluation of existing urban storm drainage systems, including the simulation of sewer surcharge.

A new parameter is also introduced in DDSRM. This parameter is the density of inlets, which can be expressed in terms of the number of inlets per unit area, or more appropriately the number of inlets per unit of impervious area. The new parameter would reflect the spacing of the inlets. It can be utilized at the initial phase of new designs, and an optimum spacing between the inlets can then be determined.

DDSRM, however, has a number of limitations, in addition to the basic assumptions mentioned in Section 6.2: (a) it neglects the energy losses at the sewer junctions, (b) it cannot simulate hydraulic jumps, and (c) it cannot simulate roll waves which may occur for flows with large Froude numbers (approximately greater than 2).

6.4 MODEL STRUCTURE

A master flow chart of DDSRM is presented in Fig. 6.2, which shows the overall structure of the model. The program is divided into sub-programs, each of which serves the purpose indicated in Fig. 6.2. Some of the sub-programs call other auxiliary routines which perform certain specific computations. This increases the flexibility of the program by allowing additional features to be added with minimum effort and without the need to change other parts of the program.

6.4.1 Input Data and Organization

The program execution is controlled through the input data supplied via subroutine "DATA". Subroutine "DATA" serves the following purposes:

1. selection of the appropriate routines to be called by the main program based on the problem under consideration and the objectives of the simulation;
2. conversion of units and the basic calculation of constants, parameters, etc;
3. calculation of rating curves for the major system and the sorting of inlet capture characteristics;
4. provision of connectivity matrices for

   subareas - major system segments

   major system network

   inlets - storm sewers

   minor system network

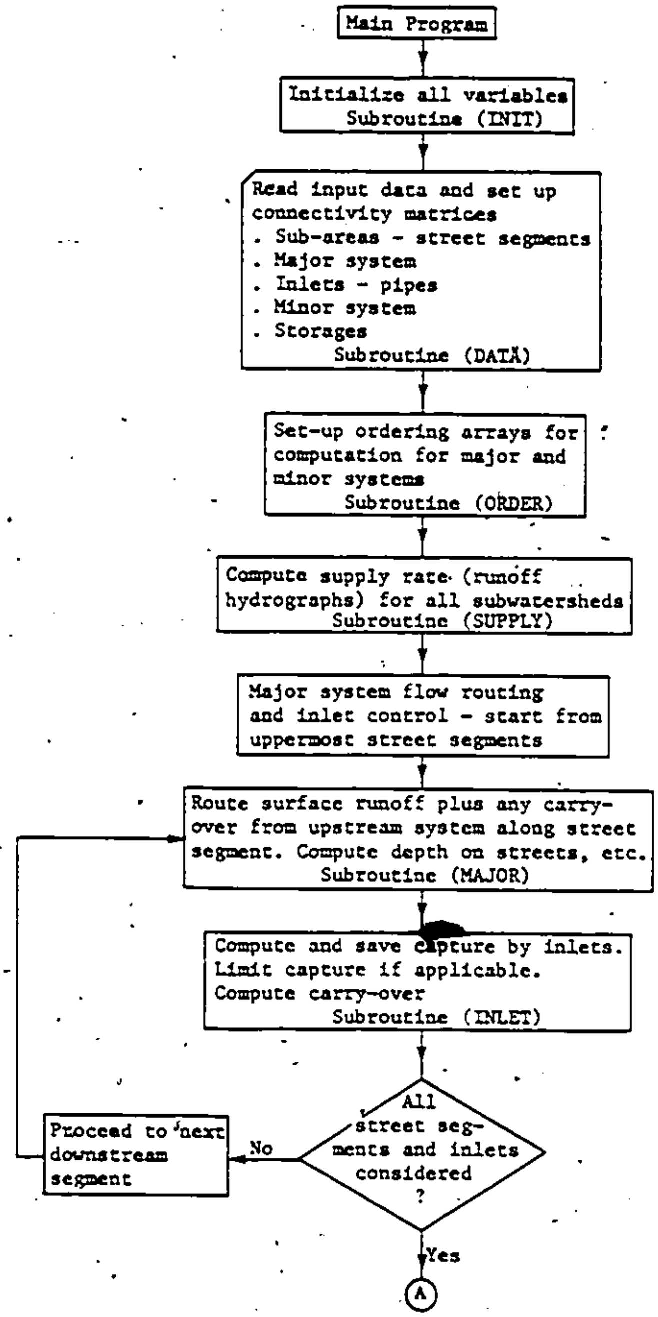


Fig. 6.2 Master Flow Chart of DDSRM

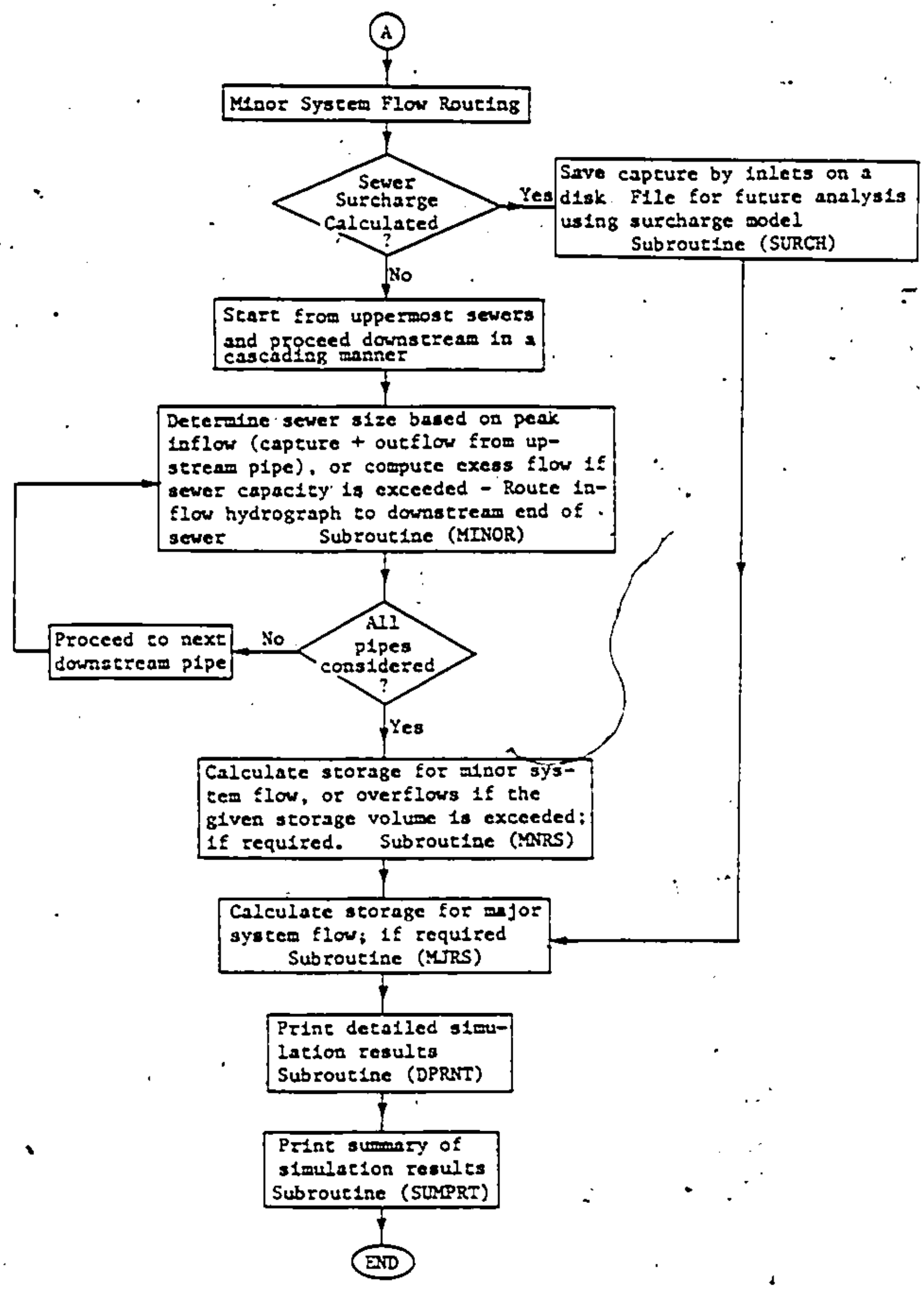

A
Minor System Flow Routing
Sewer Surcharge Calculated ?
Yes
Save capture by inlets on a disk File for future analysis using surcharge model Subroutine (SURCH)
No
Start from uppermost sewers and proceed downstream in a cascading manner
Determine sewer size based on peak inflow (capture + outflow from upstream pipe), or compute exess flow if sewer capacity is exceeded - Route inflow hydrograph to downstream end of sewer Subroutine (MINOR)
All pipes considered ?
No
Proceed to next downstream pipe
Yes
Calculate storage for minor system flow, or overflows if the given storage volume is exceeded; if required. Subroutine (MNRS)
Calculate storage for major system flow; if required Subroutine (MJRS)
Print detailed simulation results Subroutine (DPRNT)
Print summary of simulation results Subroutine (SUMPRT)
END

Fig. 6.2 Cont'd

major system - surface storage(s)

minor system - underground storage(s)

Subroutine "ORDER" sorts out the connectivity matrices for the major and for the minor system (only if sewer surcharge calculation is not required) networks and re-arranges them into an order which is converging towards the downstream outlet. These "ordered" arrays are used by the routing sub-programs.

An exception is the case of the evaluation of an existing sewer network system which requires the simulation of surcharge. In this case, the sewer data are supplied directly to the surcharge sub-model. Subroutine "DATA" in this case will only allocate the inlet flows into the sewer network. The ordering of the sewer system in a sequential manner is no longer applicable since the calculation is performed for all sewers at every time step.

Fig. 6.3 shows the main card groups for the input data to DDSRM.

6.4.2 Surface Runoff Sub-Model

The surface runoff sub-model provides the basic input to DDSRM, that is, catchment supply rate. The problem of selection or development of a new relation for the hydrological transformation of rainfall into runoff is not considered in the present study, as discussed earlier. Rather,

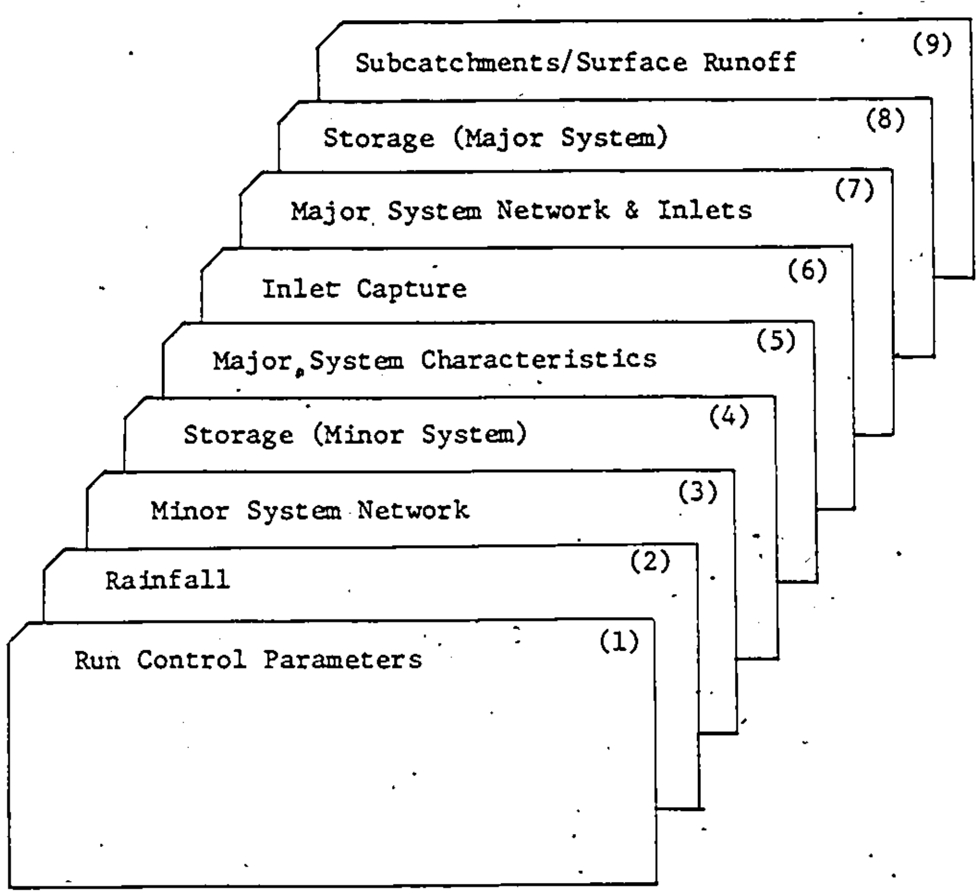


Fig. 6.3 Main Card Groups For Input Data to DDSRM

the calculation of catchment supply rate is borrowed from a revised version of EPA-SWM Model (Huber et al, 1980). Catchment runoff is considered as an inflow into the street network.

DDSRM will also accept surface runoff hydrographs input directly (e.g. as calculated by another model). In this case, IWS on card group 1B (Appendix D) should be set equal to an integer other than zero (any number between 1 and 99999). The surface runoff hydrographs must then be input on card group 9C for all sub-areas in the same order as they appear on card group 9B.

6.4.3 Major System Sub-Model

Fig. 6.4 is a flow chart of the major system sub-model. Its primary purpose is to route the major system flow through the street network. It is assumed that the street network is dendritic and converging towards the downstream outlet. The calculation starts from the uppermost street segments and proceeds downstream. The sorting of the network in this order is done in subroutine "ORDER". For each segment the entire hydrograph is routed before proceeding to the next downstream segment. The inflow into a major system segment is composed of: (a) direct runoff from the sub-catchment; and (b) flows which by-pass the inlets (carry-over) from the immediate upstream segment(s). If a major

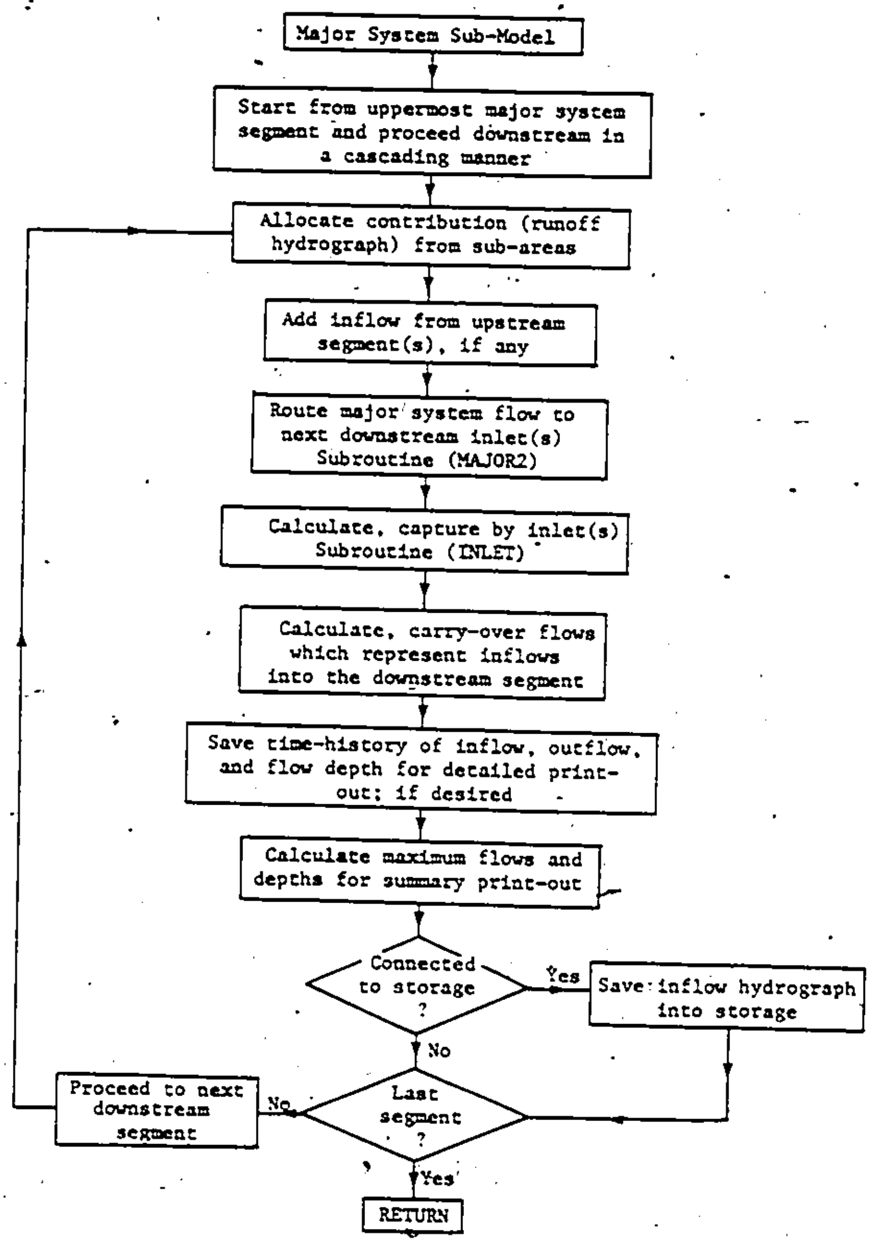


Fig. 6.4 Flow Chart of the Major System Sub-Model

system segment is connected to a storage unit the outflow hydrograph is assigned to that storage location and saved for future routing through storage. Up to five major system segments can be connected to each storage location.

The input data for the major system is supplied in card group 7. Every major system segment is given an identification number (any integer between 1 and 99999). The data also includes the number of storm inlets within the major system segment, the identification number for the inlet type and an identification number for the storm sewer receiving the inlet flow.

If a major system segment is connected to a storage unit, NSTS(I) on card group 7 must be assigned a number corresponding to IDSTS (identification number of storage unit) on card group 8. Otherwise it should be left blank. If printing and plotting of the time history of flow and depth for a major system segment is required, IPRTS(I) must be given an integer between 1 and 99999 or left blank.

6.4.4 Inlet Sub-Model

Fig. 6.5 is a flow chart of the inlet sub-model, which serves the following purposes: (a) computes flows captured by the storm inlets, and hence the by-pass flow (carry-over), and (b) establishes inlet restriction requirements, if any. Data representing the hydraulic performance of the

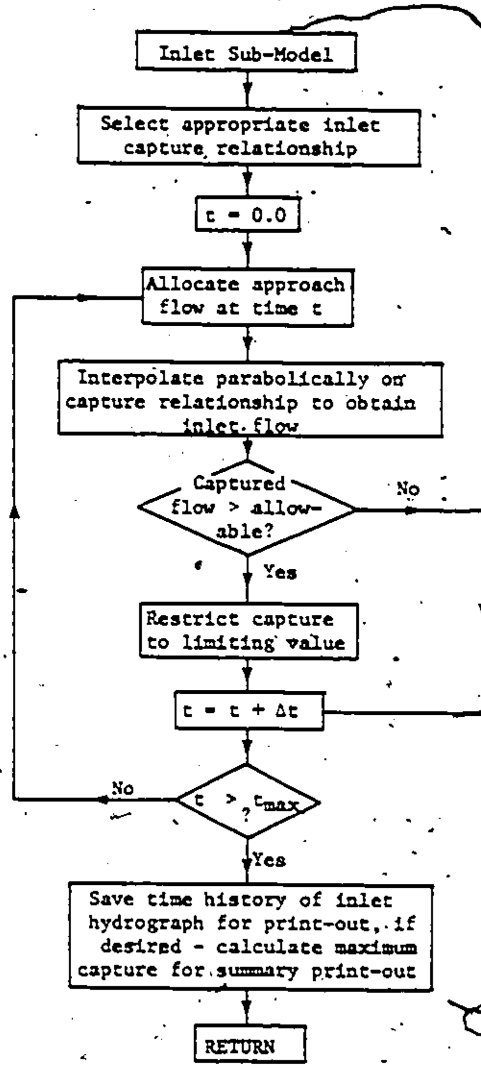


Fig. 6.5 Flow Chart of the Inlet Sub-Model

inlets are supplied to the model (in terms of approach flow versus inlet flow) by subroutine "DATA" - card group 6. Up to 10 different inlet types can be used in a single simulation.

If inlet restriction is considered, the desired limiting capture should be supplied to the model (QLIMIT on card group 1B). An option is, however, provided to assign different limiting captures for individual inlets. This can be done by specifying the desired inlet restriction on each major system segment (QLIM(I) on card group 7), which will override the value specified on card group 1B (QLIMIT). If QLIM(I) for any segment is left blank, the value assigned to QLIMIT will be assumed by the model.

The model can determine the locations of storm inlets which may require flow constricting devices. Another option for determining inlet restriction requirements is to limit the inlet flow in order to satisfy the criterion that free-surface flow prevails throughout the sewer system. This solution is carried out by the minor system sub-model.

Note that DDSRM can also be applied in the same way as existing urban storm drainage models by assuming 100% interception of runoff by the storm inlets. This can be done by specifying a one-to-one relationship between approach flow and captured flow on card group 6. In this case, IRTS on card group 1B must be assigned a number other than zero (any integer between 1 and 99999).

6.4.5 Minor System Sub-Model

Fig. 6.6 is a flow chart of the minor system sub-model. There are two basic flow conditions in the sewers considered here: (a) free surface flow, and (b) surcharge flow.

For the case of free surface flow the sewer system is assumed to be dendritic and converging towards the downstream outlet. The computations start at the uppermost pipes and proceed downstream. The sorting of the sewers in this order is also done in subroutine "ORDER". For each sewer the entire hydrograph is routed before proceeding to the next downstream sewer. The inflow into a sewer is composed of: (a) flows captured by the connecting inlets, and (b) outflows from the immediate upstream pipe(s). For the case of combined sewers the dry-weather flow should be calculated separately and is supplied as a base flow (QB(I) on card group 3B).

If the simulation is for a new design, IEVAL on card group 1B must be set equal to zero. The model will then determine the smallest commercial sizes of the sewers for free surface flow, based on the peak inflow into the sewer. Sewers slopes and roughness coefficients must be known a priori.

Three options have been provided regarding the evaluation of an existing sewer network system:

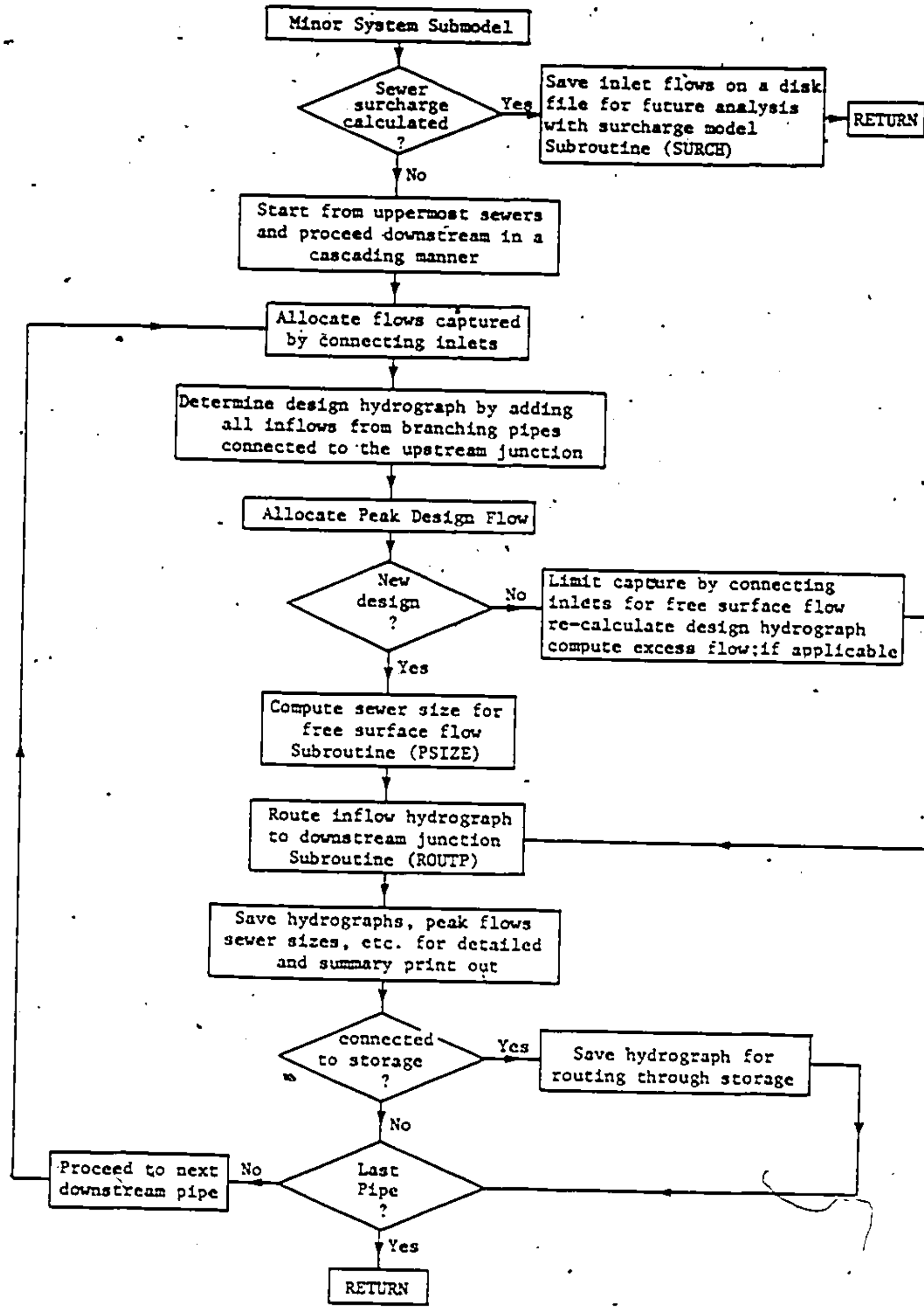


Fig. 6.6 Flow Chart of the Minor System Sub-Model

1. The first option is to determine any additional inlet constrictions such that free surface flow prevails throughout the sewer system. In this case, IEVAL on card group 1B must be set equal to 2, and the model will limit the captured flows such that the pipe capacities are not exceeded. The excess flow in this case will not, however, be added to the major system and additional simulations may be required before achieving the final design.
2. By specifying IEVAL = 1 on card group 1B, the model will re-size the pipes, if required, in order to have free surface flow conditions. This option is useful in new designs of dual systems. For example, if a specified minimum inlet restriction is found to be inadequate to limit the inlet flows to the available capacities of the minor system, minor increases in the pipe sizes may provide a reliable protection during the major system design storm frequency, as will be illustrated in section 7.2.
3. In the case that sewer surcharge needs to be simulated, IEVAL on card group 1B must be set equal to 3. No data will be required on card group 3B. Inlet flows will then be stored on a disk file for future analysis by means of the surcharge sub-model (Chapter V). In this case, looped sewer systems are allowed.

If a sewer is connected to a storage unit, the outflow hydrograph is assigned to that storage location and saved for routing through storage. Up to five sewers can be connected to each storage location.

The input data for the storm sewer network is supplied in card group 3B. Every sewer is given an identification number (any integer between 1 and 99999). The data also include sewer length, slope, dry weather flow, if any, and pipe diameter (only for the case of evaluating an existing system).

If a storm sewer is connected to a storage unit, NPPS(I) on card group 3B must be assigned a number corresponding to IDSTP (identification number for storage unit) on card group 4 or left blank. If printing and plotting of the time history of flows for a storm sewer is required, IPRTP(I) must be assigned an integer between 1 and 99999 or left blank.

6.4.6 Dual Storage

Storage of runoff is modelled separately for the major and the minor systems. This dual storage system is described in section 2.3.3.2 - also refer to Fig. 2.5. The storage of the minor system flow is accommodated, for example, in an oversized pipe (superpipe), while surface storage (e.g. utilizing park areas) is considered for the major system. Up to five storage locations of each type can be specified. As many as five storm sewers can be connected to each underground storage and five street segments can be connected to each surface storage unit.

If storage computation for the minor system flow is required, JPP on card group 1B must be set equal to an integer between 1 and 99999, otherwise it should be left blank. The same is true for JSS on card group 1B in the case of storage computation for the major system flow. Every storage unit for the minor system will be given an identification number (card group 4B, 4C or 4D). Similarly every storage unit for the major system will be given an identification number (card group 8).

Fig. 6.7 is a flow chart of the minor system storage routine. If IOPTSP(I) on card group 4A is set equal to 1, the model will determine the required storage volume(s) for a given maximum release rate(s). On the other hand, if the storage volume is specified, flows will be routed through the storage unit(s).

If the storage-outflow relationship is not known, IOPTSP on card group 4A should be set equal to 2. In this case, a constant outflow rate equal to the maximum allowable will be assumed by the model. If the storage-outflow relationship is known, IOPTSP should be set equal to 3, and the storage-outflow relationship must be input on card group 4E.

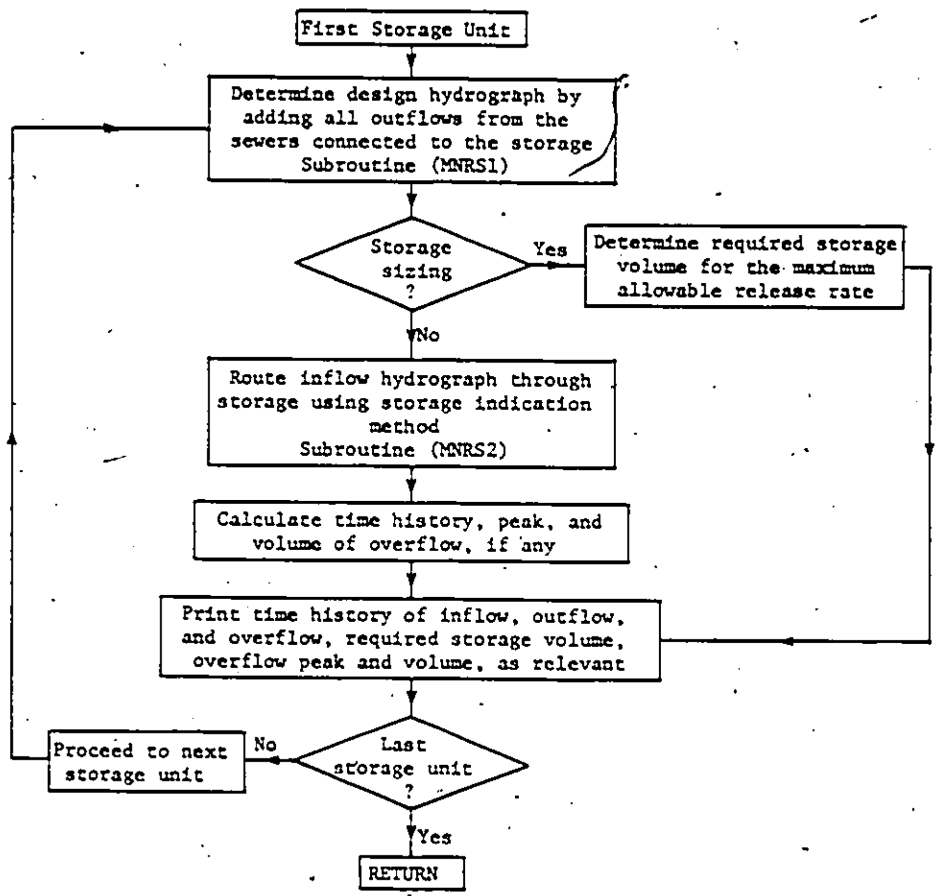


Fig. 6.7 Flow Chart of the Minor System Storage Routine

Flow routing through storage in this case is based on the storage indication method (Gray and Wigham, 1970). Once the given storage volume is exceeded, overflows will be calculated (IOPTSP = 2 or 3).

Modelling of the major system storage is carried out in basically the same manner as for the minor system, except that overflows are not considered. The main output from the major system storage routine is the required surface storage volume. If an overflow from an underground storage is discharged into a surface storage unit (Fig. 2.5b), the total surface storage volume is obtained by adding the overflow volume to the contribution from the major system.

The model will also handle situations in which a single storage unit is considered for both the major and minor system flows.

6.5 COMPUTER OUTPUT

The basic computer output of DDSRM provides the following information:

1. a print-out of the input data as well as a summary statistics of the watershed: number of subareas, sewers, storage units, total drainage area, etc., density of inlets (number of inlets per unit area), average distance between inlets, etc;

2. required sizes of sewers for free surface flow conditions;
3. inlet control requirements, that is locations of inlets which may need flow constricting devices, and limiting capacities if the latter is not specified;
4. detailed simulation results for specified elements, in printed and plotted forms:
   a) time history of surface runoff
   b) time history of major system flows and depths
   c) time history of inlet flows
   d) time history of sewer flows
5. a summary of simulation results including maximum flows and depths at various locations as requested;
6. storages:
   a) required surface storage volume for major system flow
   b) required storage volume for minor system flow, or volume of overflow if a given storage volume is exceeded
   c) inflow, outflow, and overflow hydrographs for storages, in printed and plotted forms
   d) total storage volume required for the given outlet conditions
7. in addition, if the surcharge sub-model is utilized for sewer flow routing the following information is obtained:

a) print-out summary for junctions, which shows:

   i) maximum computed depth and time of occurrence

   ii) maximum computed surcharge

   iii) minimum depth below ground elevation

   iv) duration of surcharge

b) print-out summary for conduits which shows:

   i) design flows

   ii) maximum computed flows and velocities and time of occurrence

   iii) ratio of maximum to design flows

   iv) maximum depth above inverts at conduit ends.

An example to illustrate the application of DDSRM for the design of dual drainage/storage for a real subdivision is presented in the next chapter.

Chapter VII

APPLICATIONS

7.1 INTRODUCTION

The Dual Drainage Simultaneous Routing Model - DDSRM presented in the previous chapter has been applied for the design of the drainage systems of several subdivisions. In this chapter an example for a real subdivision is presented together with discussions of the results of the computer simulations. DDSRM will also be used to investigate the effect of inlet efficiency and spacing on the design/operation of dual drainage. It will also be illustrated how a municipality can utilize DDSRM to develop planning diagrams regarding inlet control for the implementation of the dual principle in the design of new subdivisions.

7.2 EXAMPLE INPUT-OUTPUT FOR DDSRM

7.2.1 Study Area

In order to illustrate the application of DDSRM let us consider Fig. 7.1 which represents a small residential development (Testville 7.1) extracted from an existing (real) system. Fig. 7.1a shows sub-areas, street layout, locations of inlets, manholes, pipes, direction of major system flow,

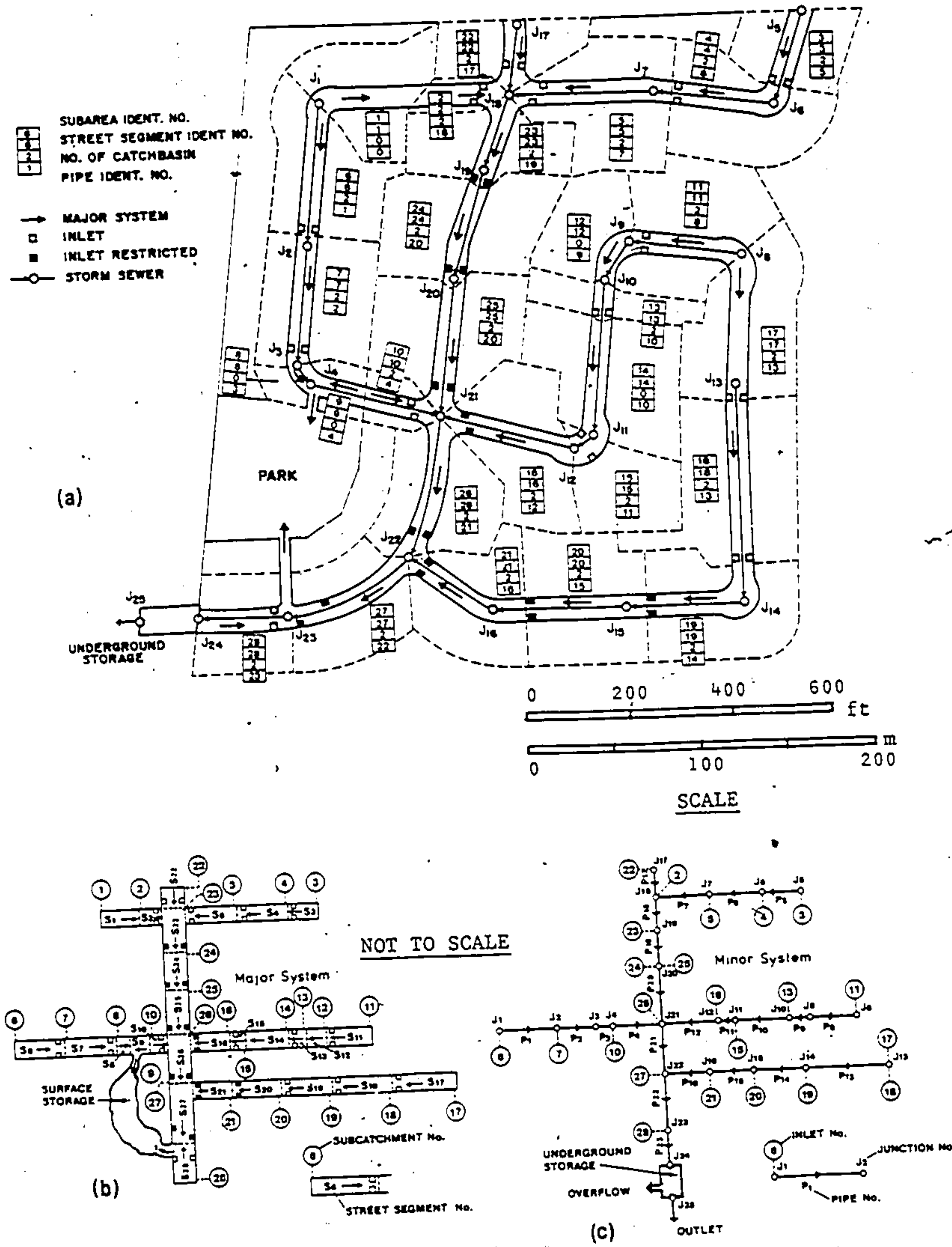


Fig. 7.1 A Subdivision With Dual Drainage/Dual Storage for Illustration of the Application of DDSRM

etc. Flow transfers from various sub-areas into the major system are shown in Fig. 7.1b, while inlet inflows into the sewer system are shown in Fig. 7.1c.

An underground storage unit is located near the outlet from the storm sewers to control the flow release from the minor system. A park area is utilized for temporary detention of the major system flow. An overflow from the underground storage is provided which discharges into the park area. The storm inlets are of the Fishbone cover type and are located according to the requirements of the local municipality. The hydraulic performance of the storm inlets is shown in Fig. 7.2.

Data relevant to the development are:

Total area	26 acres (10.5 ha)
Imperviousness	35%
Manning's coefficient	.013 storm sewers .013 pavement (smooth asphalt) .025 shoulder
Design frequency	5 years (minor system) 100 years (major system)
Longitudinal slope(streets)	1%
Street width(between curbs)	30 ft (9.1 m) -local street 40 ft (12.2 m) -main collector
Maximum allowable release	10 cfs (283 l/s)

The anaysis for the design of the drainage system of the subdivision for the two specified storm frequencies is carried out by DDSRM in two main steps (see Fig. 7.3 for the design storms):

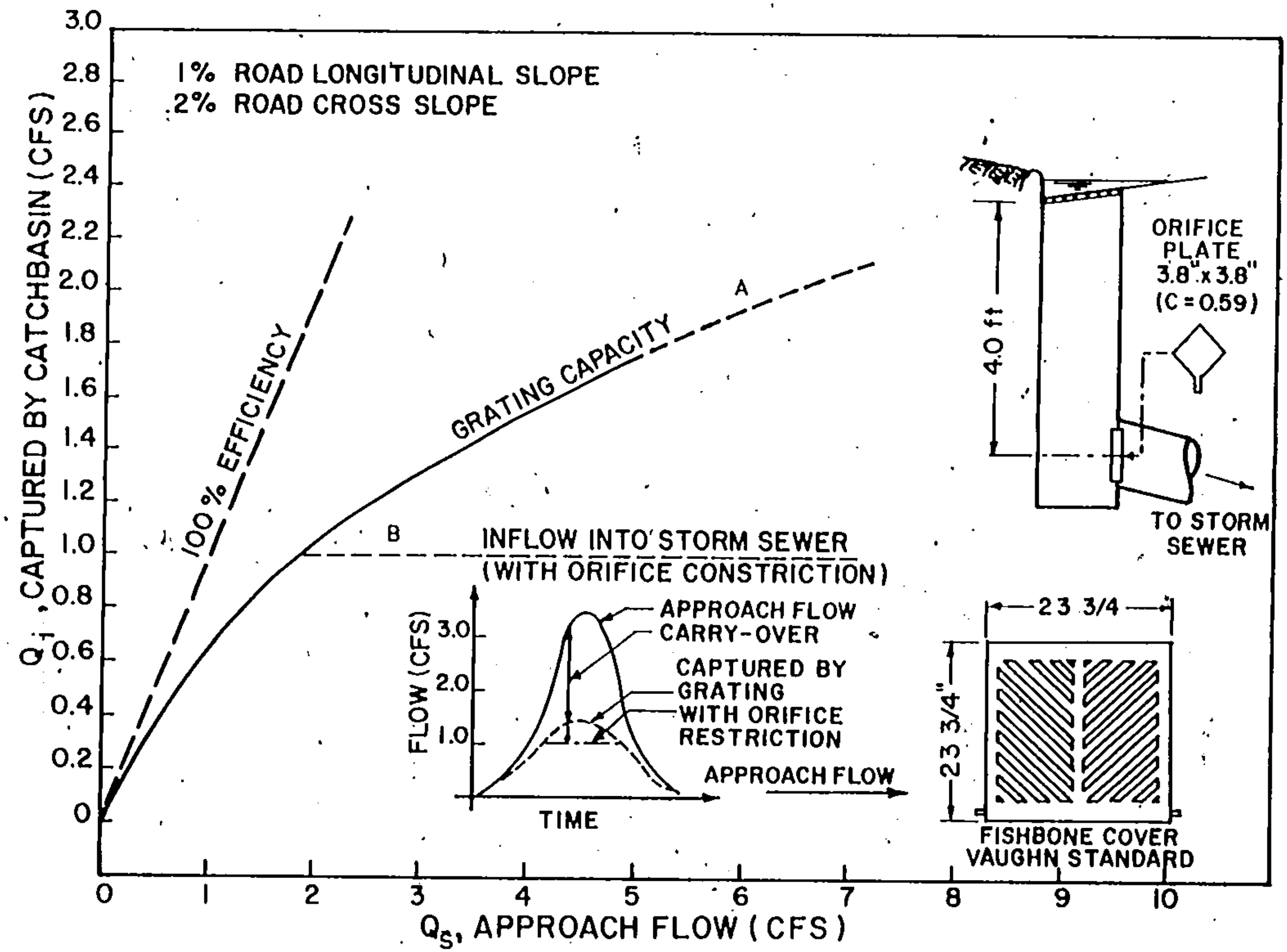


Fig. 7.2 Hydraulic Performance of the Fishbone Catchbasin Cover (After Townsend, Wisner and Moss, 1980)

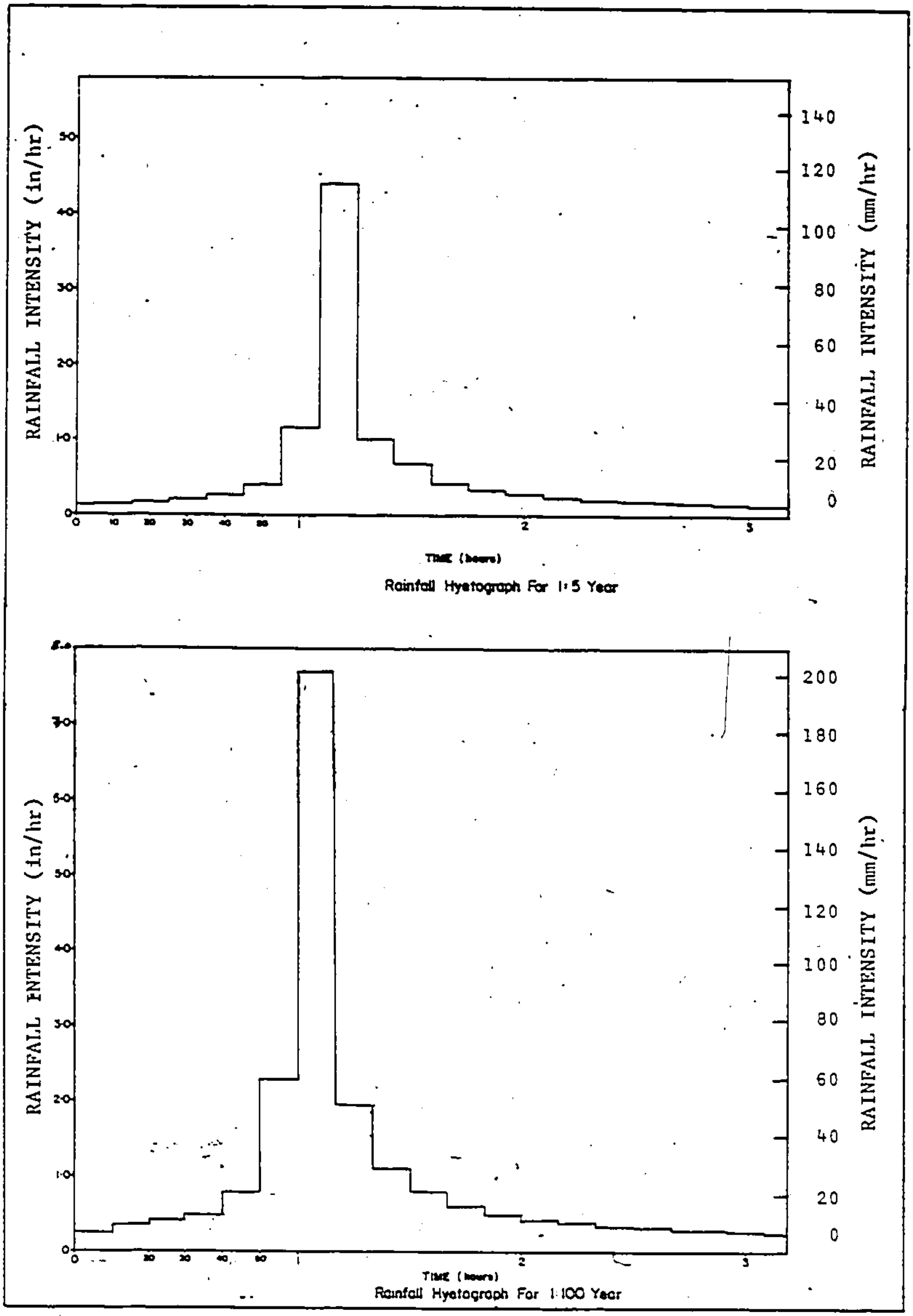


Fig. 7.3 Five and One Hundred-Year (Chicago) Design Storms Used in Example Problem

1. For the minor system design frequency (five years in this case), determine sewers sizes and the size of the underground storage for the specified maximum release rate.

2. Analyze the system for the major system design frequency (one hundred years in this case). This involves:

   a) determination of inlet restriction requirements, if any with the objective of protecting the minor system from high surcharge levels. Several options are provided in DDSRM regarding this aspect as discussed earlier (also see Appendix D);

   b) determination of flows and depths on the streets; and

   c) calculation of required storage volume for the major system flow, overflow from the minor system storage facility and hence the total storage volume.

For this example only a summary of simulation results for the 5-year storm will be presented. Complete simulation results will be provided for the 100-year design conditions.

7.2.2 Analysis for the Minor System Design Frequency

Computer simulation by DDSRM for the 5-year storm shows that the inlets will capture most of the runoff (Note that despite the use of efficient inlets, spaced at short distances apart, not all the runoff will be captured as will be shown in section 7.3). In this example the sizes of the storm sewers and the underground storage are designed on the basis of the assumption of total interception of runoff by the storm inlets. This can be simulated by DDSRM assuming a one-to-one relationship between approach flow and capture (card group 6C in Appendix D).

The required pipe sizes, as obtained by DDSRM, are given in Fig. 7.4. These sizes are the smallest commercially available pipes which can carry the design flow with free surface. Due to the discrete sizes of commercial pipes, many of the pipes will not be flowing full. With few exceptions, it is found that in this example the pipe sizes will not change if the design is based on the actual inlet capture. This finding may not, however, apply in other situations as will be discussed in section 7.3.

The required underground storage volume is found to be 29,600 cu.ft. (840 m^3) assuming total interception of runoff by the inlets. Simulation accounting for the capacity of inlets showed that only 26,700 cu.ft. (760 m^3) of storage will be sufficient and that approximately 6,500 cu.ft. (185

MINOR SYSTEM

SUMMARY OF SIMULATION RESULTS

	SEWER NO	SLOPE (FT/FT)	MANNING N	MAX. FLOW (CFS)	SEWER DIAM. (IN)	SEWER CAPACITY (CFS)	Q/QFULL (RATIO)
1	1	0.00500	0.0130	1.78	12.00	2.52	0.70
2	2	0.00500	0.0130	3.62	15.00	4.57	0.79
3	3	0.00500	0.0130	3.51	15.00	4.57	0.77
4	4	0.00500	0.0130	4.05	15.00	4.57	0.89
5	5	0.00500	0.0130	1.33	12.00	2.52	0.53
6	6	0.00500	0.0130	3.35	15.00	4.57	0.73
7	7	0.00500	0.0130	4.51	15.00	4.57	0.99
8	8	0.00500	0.0130	2.53	15.00	4.57	0.55
9	9	0.00500	0.0130	2.48	12.00	2.52	0.98
10	10	0.00500	0.0130	4.75	18.00	7.43	0.64
11	11	0.00500	0.0130	8.64	21.00	11.21	0.77
12	12	0.00500	0.0130	10.82	21.00	11.21	0.97
13	13	0.00500	0.0130	5.48	18.00	7.43	0.74
14	14	0.00500	0.0130	7.69	21.00	11.21	0.69
15	15	0.00500	0.0130	10.00	21.00	11.21	0.89
16	16	0.00500	0.0130	11.70	24.00	16.00	0.73
17	17	0.00500	0.0130	1.28	12.00	2.52	0.51
18	18	0.00500	0.0130	8.15	21.00	11.21	0.73
19	19	0.00500	0.0130	9.39	21.00	11.21	0.84
20	20	0.00500	0.0130	15.87	24.00	16.00	0.99
21	21	0.00500	0.0130	31.61	33.00	37.41	0.85
22	22	0.00500	0.0130	44.23	36.00	47.18	0.94
23	23	0.00500	0.0130	44.79	36.00	47.18	0.95

Fig. 7.4 Required Sizes of Storm Sewers For 100% Capture of the 5-Year Design Storm As Obtained by DDSRM

m^3) will reach the surface detention area for the 5-year design storm. In this example the size of underground storage is chosen as 30,000 cu.ft. (850 m^3).

7.2.3 Analysis for the Major System Design Frequency

In this example it will be assumed that the minimum allowable restriction of inlets is 1.5 cfs (42.5 l/s). Since the inlet locations are pre-specified, the computer simulation will determine the locations of inlets which may require flow constricting devices. This may not, however, be sufficient for protecting the minor system from high surcharge levels. Two options are available in DDSRM for the analysis of the minor system in this case, as discussed in section 6.4.5. They are (a) to analyze surcharge levels, or, (b) to re-size the storm sewers based on the 100-year capture with inlet control. The second option is adopted in this example. Simulations by DDSRM are therefore carried out with IEVAL = 1 on card group 1B (Appendix D).

7.2.4 Input Data and Computer Output for the 100-Year Design Storm

The input data to DDSRM for the 100-year design storm are shown in Fig. 7.5. The computer output is provided in Appendix E. Refer also to Appendix D for the input data specification and description of input variables.

```
D D S R M - EXAMPLE INPUT - OUTPUT  - TESTVILLE 7.1
100 - YEAR STORM
  5   40    1    1    1    0  1.5    0
  5   60
 .26  .26  .31  .32  .38  .39  .51  .51  .79  .79 2.29 2.29 7.71 7.71 1.93 1.93
1.10 1.10  .78  .78  .62  .62  .51  .51  .44  .44  .39  .39  .35  .35  .32  .31
 .29  .28  .27  .26  .25  .24  .24  .23  .22  .22  .21  .21  .20  .20  .19  .19
 .18  .18  .17  .17  .16  .16  .15  .15  .14  .14  .13  .13
  0
  1    2  280 .005 .013   12    0    0    0
  2    3  225 .005 .013   15    0    0    0
  3    4   51 .005 .013   15    0    0    0
  4   21  246 .005 .013   15    0    0    0
  5    6  180 .005 .013   12    0    0    0
  6    7  225 .005 .013   15    0    0    0
  7   18  278 .005 .013   15    0    0    0
  8    9  213 .005 .013   15    0    0    0
  9   10   89 .005 .013   12    0    0    0
 10   11  294 .005 .013   18    0    0    0
 11   12   41 .005 .013   21    0    0    0
 12   21  258 .005 .013   21    0    0    0
 13   14  395 .005 .013   18    0    0    0
 14   15  228 .005 .013   21    0    0    0
 15   16  254 .005 .013   21    0    0    0
 16   22  172 .005 .013   24    0    0    1
 17   18  149 .005 .013   12    0    0    0
 18   19  147 .005 .013   21    0    0    0
 19   20  210 .005 .013   21    0    0    0
 20   21  270 .005 .013   24    0    0    1
 21   22  276 .005 .013   33    0    0    0
 22   23  265 .005 .013   36    0    0    0
 23   24  165 .005 .013   36    0  201    1
 (BLANK CARD)
  3
201      30000     6
        0          0   6000.00       2.00  12000.00        4.00  18000.0        6.00
24000.00         8.0  30000.00      10.00
 (BLANK CARD)
2
  1   15  .02    .5 .013  .01  .05 .025  1.5
  2   20  .02    .5 .013  .01  .05 .025  1.5
  1
  1   13
  0    0    1  .67    2 1.04    3 1.30    4 1.55    5 1.75    6 1.90    7  2.0
  9 2.10   20 2.10  100 2.10  200 2.10  300 2.10
  1    2  195    1    0    1        18         0
  2   23  160    1    2    1        18         0
  3    4  145    1    2    1         5         0
  4    5  200    1    2    1         6         0
  5   23  310    1    2    1         7         0
  6    7  255    1    2    1         1         0
  7    8  255    1    2    1         2         0
  8    9   50    1    0    1         4         0
  9   29  100    1    0    1         4  301    0
```

Fig. 7.5 Input Data to DDSRM for the 100-Year Design Storm

```
10   26  140    1    2    1        4          0
11   12  220    1    2    1        8          0
12   13   90    1    0    1        8          0
13   14   80    1    2    1       10          0
14   15  210    1    0    1       10          0
15   16   40    1    2    1       11          0
16   26  260    1    2    1       12          0
17   18  290    1    2    1       13          0
18   19  290    1    2    1       13          0
19   20  260    1    2    1       14          0
20   21  230    1    2    1       15          0
21   27  260    1    2    1       16          1
22   23  150    2    2    1       17          0
23   24  180    2    2    1       19          0
24   25  200    2    2    1       20          0
25   26  240    2    2    1       20          1
26   27  260    2    2    1       21          0
27   29  250    2    2    1       22  301     1
28   29  180    2    2    1       23  301     0
   (BLANK CARD)
301
   (BLANK CARD)
     3      .52  .00115
 1     1  1.11   35 .013   .25   .02   390 .062 .184    0
 2     2   .16   35 .013   .25   .02   320 .062 .184    0
 3     3   .67   35 .013   .25   .02   290 .062 .184    0
 4     4  1.04   35 .013   .25   .02   400 .062 .184    0
 5     5   .68   35 .013   .25   .02   620 .062 .184    0
 6     6   .86   35 .013   .25   .02   510 .062 .184    0
 7     7   .92   35 .013   .25   .02   510 .062 .184    0
 8     8   .17   35 .013   .25   .02   100 .062 .184    0
 9     9   .25   35 .013   .25   .02   200 .062 .184    0
10    10   .25   35 .013   .25   .02   280 .062 .184    0
11    11  1.31   35 .013   .25   .02   440 .062 .184    0
12    12   .79   35 .013   .25   .02   180 .062 .184    0
13    13   .44   35 .013   .25   .02   160 .062 .184    0
14    14  1.43   35 .013   .25   .02   420 .062 .184    0
15    15   .74   35 .013   .25   .02    80 .062 .184    0
16    16  1.11   35 .013   .25   .02   520 .062 .184    0
17    17  1.24   35 .013   .25   .02   580 .062 .184    0
18    18  1.53   35 .013   .25   .02   580 .062 .184    0
19    19  1.19   35 .013   .25   .02   520 .062 .184    0
20    20  1.28   35 .013   .25   .02   460 .062 .184    0
21    21   .98   35 .013   .25   .02   520 .062 .184    0
22    22   .64   35 .013   .25   .02   300 .062 .184    0
23    23   .68   35 .013   .25   .02   360 .062 .184    0
24    24  1.79   35 .013   .25   .02   400 .062 .184    0
25    25  1.79   35 .013   .25   .02   480 .062 .184    0
26    26  1.01   35 .013   .25   .02   520 .062 .184    0
27    27   .99   35 .013   .25   .02   500 .062 .184    0
28    28   .74   35 .013   .25   .02   360 .062 .184    1
   (BLANK CARD)
```

Fig. 7.5 Cont'd

7.2.5 Discussion of Simulation Results for the 100-Year Design Storm

The computer output of DDSRM given in Appendix E provides a clear picture of the operation of the drainage system. The results are given in, easy to interpret, printed and plotted forms. Note that the detailed simulation results have been requested only for a few segments of each subsystem. Similar information at other locations can also be obtained, if desired. The results of the computer simulation for the 100-year design frequency are discussed below.

7.2.5.1 Inlet Control

In this example inlet types, locations, and minimum allowable restrictions have been pre-specified. The summary of simulation results regarding inlet control can be depicted from the computer output on page E.19 (Appendix E). It can be seen that the constriction of inlets will be required primarily on the main street collector (also see Fig. 7.1 for the locations of inlets requiring flow constricting devices). This is due to the increased approach flow on main collector caused by the accumulation of major system flow (flows by-passing the inlets) from the tributary (local) streets. On the local streets, the gratings themselves will provide sufficient control of capture.

7.2.5.2 Major System

The time history of flows and depths at three locations on the major system are shown (in printed and plotted forms) on pages E.11 through E.13. Detailed simulation results at these locations have been requested in the input data by specifying IPRTS(I) ≠ 0 on card group 7 (Appendix D). The summary table on page E.19 gives the maximum flows and depths throughout the major system network. The absolute maximum flow in the major system is 65.72 cfs (1.86 m^3/s) at 5.97 inches (15.2 cm) depth, near the main inlet into the park detention area. This depth is considerably smaller than that usually allowed by Canadian municipalities. For example, in the drainage criteria manual for the Town of Oakville, Ontario (1980) flow depths on the streets up to 18 inches (45 cm) are allowed, for the 100-year storm. The peak inflow into the park can be utilized for sizing the entrance channel. High velocity may, however, result at the entrance into the park which may require provisions for protection against erosion.

7.2.5.3 Minor System

The summary of simulation results for the minor system network is given on page E.20 (Appendix E). It shows for each sewer pipe, the peak flow, the original and revised pipe size (if applicable) and the corresponding ratios between design flows and pipe capacities. The time history of flows in three pipes (in printed and plotted forms) are given on pages E.14 through E.16. Detailed simulation results at these locations have been requested in the input data (by specifying IPRTP(I) ≠ 0 on card group 3B (Appendix D)).

It can be seen from the summary table (page E.20) that in order to have free surface flow, some of the pipe sizes will need to be increased (as compared to the sizes required for the 5-year flows). This increase in sizes, however, seems to be minor and is required mainly on the main sewer collector. It might not have been necessary if more restrictions on inlet capture are imposed, or if less efficient inlets are used (Section 7.3.1). Another alternative to avoid increasing the pipe sizes is to set the inlets at greater distances, as will be examined in Section 7.3.2.

Another alternative for the analysis of the minor system for the 100-year flow conditions is to simulate sewer surcharge in conjunction with the original pipe sizes. In this case, IEVAL on card group 1B should be set equal to 3. The analysis of the minor system is thus carried out by means of the surcharge sub-model of DDSRM. Revisions of the pipe sizes will be required only if surcharge levels are found to be dangerously high. The investigation with sewer surcharge is not, however, carried out in this example.

7.2.5.4 Dual Storage

A. Underground Storage: The results of simulation pertinent to the underground storage are shown on page E.18. The computer output of DDSRM gives the time history of inflow into the storage facility, outflow and overflow (in printed and plotted forms) and overflow volume. In this example, a linear relationship between storage and outflow is assumed as indicated in Fig. 7.4 - card group 4E (also see page E.4 of the computer print-out in Appendix E), with the maximum outflow not exceeding 10 cfs (283 l/s). Note that the maximum outflow from the storage will be governed by the size of the outlet and the maximum head, which is controlled by the elevation of the overflow.

The simulation shows that, in spite of inlet control, the excess volume of water reaching the underground storage facility has increased to 83,470 cu.ft. (2360 m^3)-see page E.18 of the computer print-out in Appendix E, as compared to 26,700 cu.ft. (760 m^3) for the 5-year storm (approximately 3.1 times the 5-year volume). The balance between the excess volume of water reaching the underground storage facility and the available detention volume (30,000 cu.ft. = 850 m^3) is 53,470 cu.ft. (1510 m^3) which will overflow into the park area. The peak overflow is approximately 43.9 cfs (1.24 m^3/s) which can be utilized for the design of the overflow structure.

B. Park Storage: The results of simulation of park storage for the 100-year design storm are shown on page E.17. The plot gives the total inflow into the park area (in this example, the summation of inflows from the two street inlets into the park - see Fig. 7.1). These flows contribute 1.23 ac.ft. (1520 m^3) of detention volume in the park. The total required storage volume in the park is 2.46 ac.ft. (3030 m^3) which is obtained by adding the overflow volume (53,470 cu.ft. = 1.23 ac.ft. = 1510 m^3) to the contribution from the major system. The area allotted for the park in this example is 1.0 acre (4050 m^2) -approximately 4% of the total area of the subdivision. The average detention depth in the park for the 100-year storm is therefore 2.46 ft (0.75 m). The park should be landscaped accordingly.

7.3 EFFECT OF INLET EFFICIENCY AND SPACING ON THE OPERATION OF DUAL DRAINAGE

The above example is presented to demonstrate one application of DDSRM in practical design of dual drainage/storage for a new subdivision. This example is for specific conditions with the inlet type and locations pre-specified. In this section DDSRM will be utilized to investigate the effect of: (1) inlet type (efficiency) and (2) spacing of inlets on the design/operation of dual drainage systems.

Two basic assumptions will be made: (1) the design frequencies of the minor and major systems are 5 and 100 years, respectively, and (2) the sewer sizes are determined based on the total runoff (100% inlet capture) for the 5-year design storm. The design storms used in the analysis are those given in Fig. 7.3. Runoff for the 100-year storm may result, however, in flows in the minor system which are greater than the sewer capacities, with the consequence of the occurence of sewer surcharge. The inclusion of surcharge computation in this investigation will obviously complicate the analysis. Therefore for the sake of examining the effect of inlet capture (efficiency and spacing) the computer simulations will be based on the assumption that the storm sewers have adequate capacities to transport any runoff captured by the storm inlets. DDSRM will therefore be applied in conjunction with IEVAL = 0 on card group 1B, (Appendix D). The resulting flows can then be used to analyze the role of inlet efficiency and spacing in the design of dual drainage.

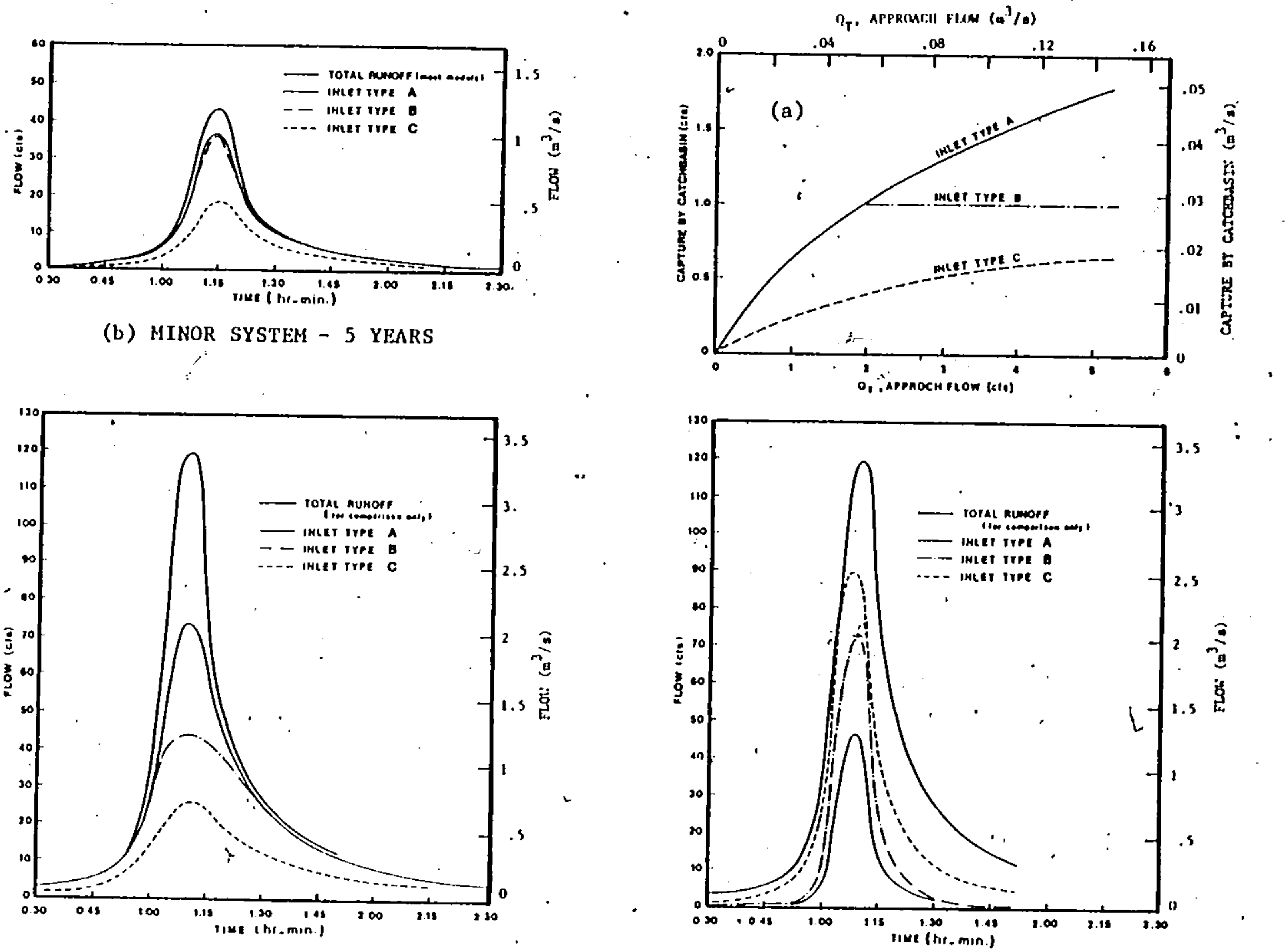
7.3.1 Inlet Efficiency

In order to examine the effect of inlet efficiency on the operation/design of dual drainage systems, the case study of Fig. 7.1 is analyzed again by DDSRM assuming three alternate inlets: A, B and C. The inlet locations are fixed and are as shown in Fig. 7.1. Inlet type A is the fishbone cover used in the previous section (Fig. 7.2). Inlet type B is again a fishbone cover with an orifice plate mounted at the entrance from the catchbasin into the storm sewer. The size of the orifice constriction is such that its maximum capacity is 1.0 cfs (28.3 l/s) - see Fig. 7.2. Inlet type C is a combination inlet (City of Edmonton standards). The capture characteristics of the three inlets, as determined from the hydraulic model study at the University of Ottawa (Townsend et al, 1980), are compared in Fig. 7.6a.

The drainage system of the subdivision is analyzed by DDSRM for the two design storm frequencies, with each of the three inlets assumed to be used exclusively. The results of simulations are presented in Figs. 7.6b,c,d, and are given only at the outlets from the minor and major systems.

With reference to Fig. 7.6 it can be seen that:

1. Although the density of inlets in this example is very high (1.78 inlets per acre (4.4 inlets per ha) @ 35% imperviousness), none of the inlets will capture all runoff for the 5-year design storm.
2. While inlet type A will capture the majority of runoff for the 5-year storm (see Fig. 7.6b), some carry-over will still occur. On the other hand, during the 100-year storm, the inlet capture will substantially exceed the capacity of the minor system.



(b) MINOR SYSTEM - 5 YEARS

(c) MINOR SYSTEM - 100 YEARS

(d) MAJOR SYSTEM - 100 YEARS

Fig. 7.6 Effect of Inlet Type on the Design/Operation of Dual Drainage

Since the sewer sizes are determined on the basis of the 5-year flows, the result is the occurrence of sewer surcharge with the potential for basement flooding during less frequent storm events.

3. Inlet type C exerts such a severe control on flows admitted into the minor system that the storm sewers will not be utilized to their capacity even for the 100-year storm! While this will prevent the occurrence of sewer surcharge, the consequences are: (a) large investments in the cost of "oversized" sewers, and (b) increased inconvenience to traffic and pedestrians on a more frequent basis.

4. Inlet type B appears to achieve an optimum design. It captures the majority of runoff during frequent storms, up to the 5-year frequency, and at the same time will provide adequate control of flows admitted into the minor system during more severe rainfall events. By comparing Fig. 7.6b with Fig. 7.6c it can be seen that the 100-year capture will not exceed the minor system capacity. Note also that the volume of water admitted into the minor system has substantially increased. This aspect is, however, important only if underground storage is used for runoff control as discussed earlier.

This analysis confirms that the inlet type (efficiency) will play a key role in the operation of dual drainage. It also shows that the assumption of total interception of runoff by the storm inlets (for the minor system), inherent in existing urban storm drainage models, may be seriously invalid for some of the inlets currently used.

An important conclusion regarding the implementation of inlet control can also be derived from the above analysis. It may be a good practice to combine storm inlets which have high efficiency with inlet constriction devices. Efficient inlets will minimize inconvenience during the more frequent storms. Inlet restriction by means of an orifice plate at the entrance from the catchbasin into the storm sewers (or similar arrangements) will act as a "second line of defence" for protecting the minor system against high surcharge levels during less frequent storm events. This "dual purpose" inlet gives a reliable design of dual drainage. The orifice plate constriction has been tested for clogging by debris, etc. in a full-scale hydraulic model study (Townsend, et al, 1980) and is found to require minimum maintenance. A number of experimental orifice constrictions has been in operation for more than one year in a new subdivision (Keliar, 1981), and there have been no complaints regarding clogging.

DDSRM will give the designer a tool for a "rational" allocation of inlet constrictions. The number of constrictions can be minimized if they are combined with a rational spacing as will be discussed in section 7.3.2.

In the above example an optimum solution with 1.0 cfs (28.3 1/s) inlet constriction in conjunction with the fishbone cover is just a coincidence and does not necessarily apply in other situations. On the other hand, inlet constriction will be required for almost all inlets. Reducing the number of constrictions will depend upon the spacing (or density) of inlets besides other factors.

7.3.2 Spacing of Inlets

In order to study the effect of inlet spacing on the design/ operation of dual drainage, DDSRM is applied on a hypothetical subdivision, 100 acres in size with the inlets using the fishbone cover (Fig. 7.2). In order to make the conclusion as general as possible two parameters are varied, namely distance between inlets (or number of inlets per unit area) and imperviousness ratio. These two parameters can be combined into a single parameter, that is, "density of inlets" expressed in terms of number of inlets per unit of impervious area.

Fig. 7.7 presents the relationship between the density of inlets and the peak flows in the minor and major drainage systems for the 5 and 100-year design storms, derived from several simulations by DDSRM. It can be seen that in order

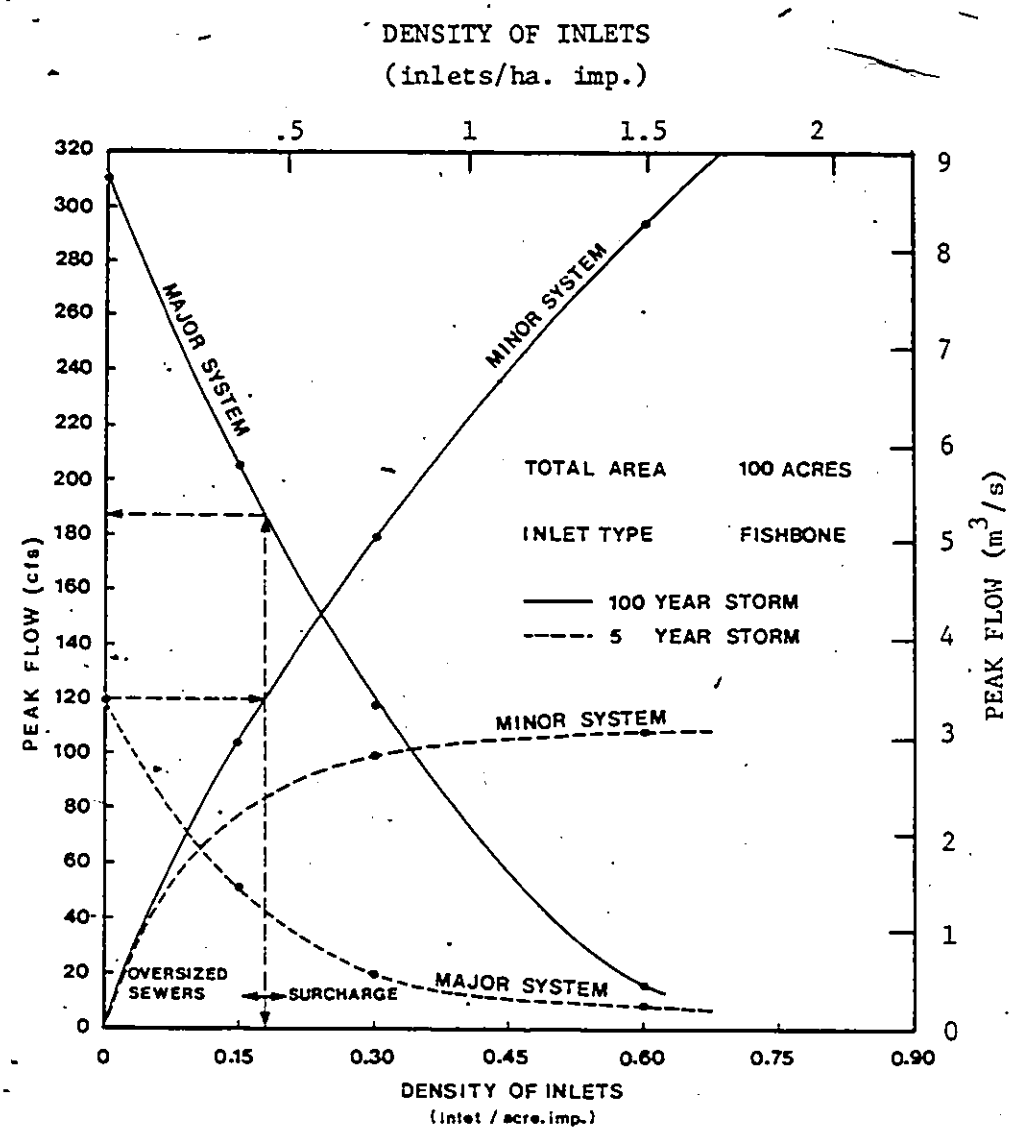


Fig. 7.7 Illustration of the Effect of Inlet Density (Spacing) On the Operation of Dual Drainage, For the Fishbone Cover

to (practically) capture all runoff for the minor system design frequency (5 years) the density of inlets should be significantly high. The use of such a large number of inlets will, however, result in a substantial increase in capture during the less frequent storm events with the consequences of overloading the minor system, occurrence of sewer surcharge and the possibility of basement flooding. Unless severe constriction of inlets is implemented it is impossible, in this case, to achieve the two conflicting requirements for proper operation of the minor and major drainage systems.

By inspection of Fig. 7.7 it also appears that in order to limit the captured runoff during the 100-year storm to the sewer capacities without the need for flow constricting devices it is necessary to select the inlet density as 0.16 inlets/acre. imp. (0.4 inlets/ha. imp.). The use of this very low density of inlets will result in the sewers not being utilized to their capacities for the 5-year storm (in this example approximately 70% of runoff captured) and minor inconvenience will occur.

This solution may not, however, be practically feasible. Assuming that the imperviousness ratio is 35%, the use of inlet density of 0.16 inlets/acre. imp. (0.4 inlets/ha. imp.) means an average of 0.46 inlets per acre (1.14 inlets per ha.). For typical residential areas this means that the inlets should be spaced at approximately 400 ft (120 m) apart (instead of the 200 ft (60 m) which is the current practice). Such a large space between inlets may not be acceptable by municipalities since it may interfere with other objectives of storm inlets such as reducing the potential for freezing problems in winter. Note also that the inlets should be spaced at even longer distances if the catchment imperviousness is higher.

An analysis similar to that of Fig. 7.7 can be followed in order to determine: (a) the adequacy of a particular type of storm inlet to achieve the design objectives of a (dual) drainage system (besides the inlet efficiency this will also depend on the design frequencies of the minor and major systems and catchment imperviousness), and (b) an optimum spacing between inlets, and hence the number of constrictions can be minimized.

7.4 CONCLUSIONS ON PRACTICAL DESIGN OF DUAL SYSTEMS

The analyses presented in the previous section demonstrate that the type and spacing of storm inlets will have a profound impact on the design/operation of dual drainage. However, there is no universal conclusion to be applied at all locations regarding a "best solution". Detailed simulations for individual projects are therefore necessary. DDSRM has been designed for this purpose.

It is, however, possible to apply DDSRM in conjunction with some representative developments in order to develop "planning" diagrams similar to that of Fig. 7.7. Practical rules for the (initial) selection of inlet type and spacing for various applications can thus be derived. Similar diagrams regarding storages can also be derived.

Based on the analysis carried out in section 7.3, municipalities should be encouraged to re-examine the type of storm inlets currently in use and the regulations for their spacing.

There are several factors, other than storm inlets, affecting the design of dual drainage which can be investigated by DDSRM. The most significant of these factors are:

1. area and shape;
2. catchment imperviousness; and
3. criteria for the design frequencies (minor and major systems).

Additional investigation by DDSRM should also utilize the surcharge sub-model to examine the possible cost savings in the design of dual systems by allowing "limited" sewer surcharge.

Chapter VIII

CONCLUSIONS

8.1 CONCLUSIONS

1. The special arrangements proposed for the design of urban (dual) drainage systems, namely (a) inlet control, with selective constrictions of inlets; (b) separate storage facilities for the pipe flow and the street overland flow (dual storage); and (c) utilization of depressed park areas for temporary detention of runoff, should assure protection against basement and street flooding and runoff control for storms of all frequencies, up to the 1 in 100 years.

2. A comprehensive mathematical model "DDSRM" for the design/analysis of the proposed dual drainage/storage system has been developed, and is now operational for practical applications, subject to the limitations given below. It can be interfaced with any of the existing hydrologic models for rainfall-runoff transformation, and is essentially a routing and design tool. DDSRM has several options which allow the flexibility to handle a variety of design conditions. The computer print out of DDSRM is provided in, easy-to-interpret, printed and plotted forms.

3. The specially-weighted finite difference formulation of the kinematic wave for (free surface) flow routing will give, in most practical applications where there is no backwater effect present, results (time-history of flows and mass conservation) which are compatible with those obtained by complete (dynamic) models. The numerical solution scheme has the advantages of simplicity and efficiency of the computation, and its parameters, varying with discharge-dependent wave celerities, are derived from the physical and hydraulic characteristics of the channels.
4. The numerical scheme for simulating sewer surcharge and the methodology for the simulation of the transition between free surface and pressurized flow will provide results (time variation of sewer flows and junction water levels) which are comparable with the the proprietory HVM model, an implicit dynamic model. The results can be considered accurate if the energy losses at the junctions are neglected.
5. DDSRM has the following limitations:
   a) the model assumes a continuous major system and cannot simulate negative slopes or systems with low points where water ponding may occur
   b) it cannot simulate roll waves which may occur for flows of high Froude numbers (approximately greater than 2).

c) it cannot simulate backwater effects in the major system and may not be suitable for very mild slopes

d) it neglects the energy losses at the junctions for both free surface and surcharged flow conditions

e) it cannot simulate hydraulic jumps.

6. The following conclusions regarding inlet control can also be derived from several simulations conducted using DDSRM:

a) The assumption of total interception of runoff by the storm inlets for the design frequency of the minor system, inherent in existing urban storm drainage models, may be seriously violated for some of the inlets currently used. The discrepancy from this assumption will mainly depend on inlet efficiency and spacing, design storm and catchment imperviousness. There is a need to check this assumption for certain applications; otherwise misleading results may be obtained.

b) It will not be feasible in most cases to meet the two conflicting requirements of traffic convenience and protection against basement flooding for the minor and major systems design frequencies, respectively, merely by the spacing of inlets.

c) It is, in general, a good practice to combine storm inlets of high capture efficiency with flow constricting devices in the design of dual drainage systems. An analysis similar to that of Fig. 7.7 can be utilized for the selection of an optimum spacing between inlets, and hence the number of constrictions can be minimized.

7. DDSRM can also be applied in the same manner as existing urban storm drainage models. However, it has an improved and efficient routing method, and is considerably more flexible than many of the existing models. DDSRM is also well suited for applications in master drainage studies.

8.2 RECOMMENDATIONS FOR FUTURE RESEARCH

There are several aspects stemming from this study which require additional investigations:

1. One of the main assumptions underlying the development of DDSRM is the existence of a continuous major system, and that backwater effects in the major system are not significant. Application of DDSRM for old developments, which usually do not have adequately designed major systems, will require a method to simulate water ponding on the streets, and possible backwater effects. This may also require additional improvements to the sewer surcharge sub-model to simulate manhole flooding. Of interest in this case is

the catchment response to severe storm events. The outflow from the watershed under these conditions is of crucial importance to flood control studies. A new routing sub-model for the major system which can account for backwater effects can be easily incorporated in DDSRM. Additional improvements of the model may also attempt to simulate energy losses at the junctions, for free surface and surcharged flow conditions.

2. Hydraulic model studies on surcharged flow in storm sewers are lacking in the literature, and are therefore recommended. Such experimental studies can be utilized to improve the simulation of sewer surcharge. Additional studies are also required to investigate the possible savings in the design of storm sewers by allowing "limited" surcharge.

3. For larger watersheds, additional investigations are required to study the possible benefits of over-control of runoff (by means of the dual storage system proposed in this study) for applications in flood control studies.

4. Experimental studies may also be required on storm inlets whose hydraulic performance is not known. Available studies on inlet efficiency have been conducted for relatively small flows. Further investigations for larger flows are recommended.

5. It is also recommended to install instruments to monitor the operation of urban storm drainage systems designed according to the dual drainage principle. This may include observations on possible clogging of inlet constriction devices by debris, etc., freezing problems in winter, operation of park storage and traffic convenience.
6. DDSRM can be applied for various design conditions for the purpose of deriving some "planning" diagrams for the purpose of the preliminary design of dual systems. These diagrams can be related to the selection of inlet type and spacing, flow depths on streets, storage volumes, etc.

REFERENCES

Abbott, M.B. and Ionescu, F.H., "On the Numerical Computation of Nearly Horizontal Flows," Journal of Hydraulic Research, Vol. 5, No. 2, 1967, pp. 97-117.

Abbott, M.B. and Verwey, A., "Four-Point Method of Characteristics," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 96, No. HY12, December 1970, pp. 2549-2564.

Acker, P., "An Investigation of Head Losses at Sewer Manholes," Civil Engineering and Public Works Review, Vol. 54, Nos. 637 and 638 (Parts I and II), August-September 1959, pp. 882-884 and 1033-1036.

Acker, P., Bettes, F. and Colyer, P.J., "Head Losses and Air Entrainment at Surcharged Manholes," Report No. IT185, Hydraulic Research Station, Wallingford, U.K., November 1978.

Ahmad, Maqbool (City of Edmonton), Personal Communications, 1980.

Amein, M., "Streamflow Routing on Computer by Characteristics," Water Resources Research, Vol. 2, No. 1, 1966, pp. 123-130.

Amein, M. and Chu, H.L., "Implicit Numerical Modelling of Unsteady Flows," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 101, No. HY6, June 1975, pp. 717-731.

Amein, M. and Fang, C., "Implicit Flood Routing in Natural Channels," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 96, No. HY12, 1970, pp. 2481-2500.

American Iron and Steel Institute, Modern Sewer Design, Library of Congress Catalog Card No. 79-56206, Washington, D.C., 1980.

American Society of Civil Engineers and the Water Pollution Control Federation, Design and Construction of Sanitary and Storm Sewers, ASCE Manuals and Reports on Engineering Practice No. 37, 1970.

Aron, G., Radziul, J.V., Lakatos, D.F. and Blair, D., "Penn State Urban Runoff Model to Pinpoint Flood Peak Source Locations," Water Resources Bulletin, American Water Resources Association, Vol. 15, No. 5, October 1979.

Baltzer, R.A. and Lai, C., "Computer Simulation of Unsteady Flows in Waterways," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 94, No. HY4, 1968, pp. 1083-1117.

Bettess, R. and Price, R.K., "Comparison of Numerical Methods For Routing Flow Along a Pipe," Hydraulic Research Station, Wallingford, Oxon, Report No. IT 162, October 1976.

Borough of Scarborough, Scarborough Works Department, "Sewer Design Critical Slopes," November 1979.

Borough of Scarborough, Scarborough Works Department, "Storm Drainage Policies and Criteria," 1980.

Bowers, C.E., "Studies of Open Channel Junctions," Technical Paper No. 6, Series B, University of Minnesota, St. Anthony Falls Hydraulic Laboratory, January 1950.

Brandstetter, A., "Assessment of Mathematical Models for Urban Storm and Combined Sewer Management," Environmental Protection Technology Series, EPA-600/2-76-175a, Municipal Environmental Research Laboratory, U.S. EPA, May 1976.

Brune, A.W., Graf, W.H., Appel, E. and Yee, P.P., "Performance of Pennsylvania Highway Drainage Inlets," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 101, No. HY12, December 1975, pp. 1519-1536.

Chow, V.T., Open Channel Hydraulics, McGraw-Hill, Inc., New York, N.Y., 1959.

Colyer, P.J. and Pethick, R.W., "Storm Drainage Design Methods: A Literature Review," Report No. INT 154, Hydraulic Research Station, Wallingford, Oxfordshire, England, March 1976.

Crawford, N.H., "Studies in the Application of Digital Simulation to Urban Hydrology," Hydrocomp International, Palo Alto, California, 1971.

Cunge, J.A., "On the Subject of a Flood Propagation Computation Method (Muskingum Method)," Journal of Hydraulic Research, Vol. 7, No. 2, 1969, pp. 205-230.

Davis, W.J., "Watershed Management and Sediment Control in Montgomery County," Proceedings, National Symposium on Urban Rainfall and Runoff and Sediment Control, University of Kentucky, July 1974.

Desbordes, M., "Data and Fitting Problems in Urban Storm Drainage Models," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

Dronkers, J.J., "Tidal Computations for Rivers, Coastal Areas and Seas," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 95, No. HY1, January 1969, pp. 29-77.

FHWA, Bicycle-Safe Grate Inlet Study, Vol.1 and Vol.2, "Hydraulic and Safety Characteristics of Selected Grate Inlets on Continuous Grades," Report No. FHWA-RD-77-4, June 1977; Final Report April 1978.

Fread, D.L., "Numerical Properties of Implicit Four-Point Finite Difference Equations of Unsteady Flow," NOAA Technical Memorandum NWS HYDRO-18, Office of Hydrology, Washington, D.C. March 1974.

Fread, D.L., "Flood Routing: A Synopsis of Past, Present, and Future Capability," Proceedings, International Symposium on Rainfall-Runoff Modelling, Mississippi State, Miss., May 1981.

Garrison, J.M., Granju, J.P. and Price, J.T., "Unsteady Flow Simulation in Rivers and Reservoirs," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 95, No. HY5, 1969, pp. 1559-1576.

Gburek, W.J., Discussion of "Hydrologic Consequences of Rainfall Augmentation," by Alan M. Lumb and Ray K. Linsley (Proc Paper 8232) July 1971, Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 97, No. HY12, December 1971, pp. 2114-2115.

Gray, D.M. and Wigham, J.M., "Peak Flow - Rainfall Events," Section VIII, Handbook of the Principles of Hydrology, Edited by D.M.Gray, National Research Council of Canada, 1970.

Guillou, J.C., "The Use and Efficiency of Storm Gutter Inlets," Bulletin 450, Engineering Experiment Station, University of Illinois, Urbana, July 1959.

Hamam, M.A. and McCorquodale, J.A., "Transion of Gravity to Surcharge Flow in Sewers," International Symposium on Urban Hydrology, Hydraulics, and Sediment Control, University of Kentucky, Lexington, Kentucky, July 1980.

Harris, G.S., "Real Time Routing of Flood Hydrographs in Storm Sewers," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 96, No. HY6, June 1970, pp. 1247-1260.

Henderson, F.M., Open Channel Flow, The McMillan Co., New York, N.Y., 1966.

Hicks, W.I., "A Method of Comparing Urban Runoff," Trans., ASCE, Vol. 109, 1944.

Hoff-Clausen, N.E., "System 11 Sewer - A Storm Sewer Model," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

Huang, Y.H., "Channel Routing by Finite Difference Method," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY10, October 1978, pp. 1379-1393.

Huber, W.C., Heany, J.P., Medina, M.A., Peltz, W.A., Sheikh, H. and Smith, G.F., "Storm Water Management Model Users' Manual - Version II," Environmental Protection Technology Series EPA-670/2-75-017, U.S. EPA, March 1975.

Huber, C.W., Heany, J.P. and Nix, S.T., "Storm Water Management Model, Version III - Documentation of New Procedures," Department of Environmental Engineering Sciences, University of Florida, Gainsville, Florida, January 1980.

Jens, S.W. and Reitz, H.M., "Some Notes on Changing Approaches to Urban Drainage Management," National Symposium on Urban Hydrology and Sediment Control, University of Kentucky, Lexington, KY., July 1975.

John Hopkins University, "The Design of Storm Water Inlets," Department of Sanitary Engineering and Water Resources, Baltimore, Maryland, 1956.

Joliffe, L.B., "Numerical Modelling of Urban Drainage Networks," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

Jolly, J.P. and Yevjevich, V., "Simulation Accuracies of Gradually Varied Flow," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 100, No. HY7, July 1974, pp. 1011-1030.

Kassem, A.M., Design For Protection Against Water Hammer in Pump-Discharge Lines, M.Eng. Thesis, Department of Civil Engineering, McMaster University, Hamilton, Ontario, March 1976.

Kassem, A.M. and Roesner, L.A., "Updated Documentation of the WRE-IMPSWM Transport Model," Unpublished Report, Department of Civil Engineering, University of Ottawa, Ottawa, October 1979a.

Kassem, A.M. and Roesner, L.A., "Routing in Sewer Systems. Determination of Surcharges. Recent Improvements of EXTRAN and Comparison With Dorsch - HVM," Short Course on Storm Water Management, Department of Civil Engineering, University of Ottawa, October 1979b.

Kassem, A.M. and Wisner, P.E., "Validity of WRE-TRANSPORT Model," Unpublished Report, Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, August 1980.

Keliar, D., (Fred Schaffers Associates), Personal Communications, July 1981.

Kibler, D.F., Monser, J.R. and Roesner, L.A., "San Francisco Storm Water Management Model - User's Manual and Program Documentation," Water Resources Engineers, California, 1974.

Kibler, D.F. and Woolhiser, D.A., "The Kinematic Cascade as a Hydrologic Model," Hydrology Paper No. 39, Colorado State University, March 1970.

Klym, H., Koniger, W., Mevius, F. and Vogel, G., "Urban Hydrological Processes - Computer Simulation," Dorsch Consult, Munich, From a Seminar on Computer Methods in Hydraulics, E.T.H., Zurich, 1972.

Kostuch Engineering, Ltd., Report on Relief Sewers in the Crestview System, City of Nepean (Unpublished), 1981.

Koussis, A.D., "An Improved Approximate Method for the Computation of Flood Waves, Technical Report No. 15, Institute for Hydraulics and Hydrology, Technical University of Darmstadt, W. Germany, 1975.

Koussis, A.D., "An Approximate Dynamic Flood Routing Method," Proceedings of the International Symposium on Unsteady Flow in Open Channels, Paper L1, Newcastle-Upon-Tyne, England, 1976.

Koussis, A.D., "Theoretical Estimation of Flood Routing Parameters," Technical Notes, Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY1, January 1978, pp. 109-115.

Larson, C.L., "Experiments on Flow Through Inlet Gratings for Street Gutters," Highway Research Board, Research Report 6-B, Washington, D.C., 1948.

Lazaro, T.R., Urban Hydrology, A Multidisciplinary Perspective, Ann Arbor Science, Michigan, November 1981.

Leclerc, G. and Schaake, J.C., "Methodology For Assessing the Potential Impact of Urban Development on Urban Runoff and the Efficiency of Runoff Control Alternatives," Ralph M. Parsons Laboratory, Massachusetts Institute of Technology, Report No. 167, 1973.

Leopold, L.B., "Hydrology for Urban Land Planning - A Guidebook on the Hydrologic Effects of Urban Land Use," U.S. Geological Survey Circular 554, 1968.

Liggett, J.A. and Woolhiser, D.A., "Difference Solution of the Shallow-Water Equations," Jnl. Eng. Mech. Div., ASCE, Vol. 95, EM2, 1967, pp. 39-71.

Lighthill, M. and Whitham, G., "On Kinematic Waves--I--Flood Movement in Long Rivers," Proceedings, Royal Society of London, Volume 229, May 1955, pp. 281-316.

MacLaren, James F., Ltd., "Review of Canadian Design Practice and Comparison of Urban Hydrologic Models," Research Report No. 26, Canada-Ontario Agreement Research Program for the Abatement of Municipal Pollution, October 1975.

MacLaren, James F., Ltd., "Design and Operation of the West Risborough Stormwater Management System," (Unpublished), February 1978a.

MacLaren, James F., Ltd., "Stormwater Management Plan for the Newmarket 35-2 Ltd. Development," (Unpublished), December 1978b.

Marsalek, J., "Sewer Inlets Study - Laboratory Investigation of Selected Inlets," (Draft Report) Environmental Hydraulics Section, National Water Research Institute, Burlington, Ontario, July 1980.

Marsalek, J., "Energy Losses at Straight-Flow-Through Sewer Junctions," Research Report No. 111, Ontario Ministry of the Environment and Environment Canada, 1981.

Martin, C. and King, D., "Analysis of Storm Sewers Under Surcharge," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

McNeely Engineering, Ltd., Report on Flood Relief Measures in Beacon Hill, Ottawa, (Unpublished), 1981.

McPherson, M.B., "Urban Mathematical Modelling and Catchment Research in the U.S.A.," Technical Memorandum No. IHP-1, Urban Water Resources Research Program, ASCE, New York, June 1975.

McPherson, M.B., "Overview of Urban Runoff Tools of Analysis," Report EP79-R-20, Department De Genie Civil, Ecloe Polytechnique De Montreal, Quebec, April 1979.

Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, California, "Storm Water Management Model," Vol. I, Report No. 11024DOC07/71, Environmental Protection Agency, Washington, 1971.

Miller, W.A. and Cunge, J.A., "Simplified Equations of Unsteady Flow," Unsteady Flow in Open Channels, Chapter 5, Vol. I, edited by K. Mahmood and V. Yevjevich, Water Resources Publications, Fort Collins, Colorado, 1975.

Ministry of Transportation and Communications, "MTC Drainage Manual, Chapter E, Hydraulic Design of Storm Sewers," Draft by Proctor and Redfern Ltd., April 1981.

Mozayeny, B. and Song, C.S., "Propagation of Flood Waves in Open Channels," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 95, No. HY3, May 1969, pp. 877-892.

NWS, Design and Analysis of Urban Storm Drainage-The Wallingford Procedure, Volume 1: Principles, Methods and Practices, National Water Council, U.K., Standing Technical Committee Reports No. 28, September 1981.

Ontario Ministry of the Environment, "Manual of Practice on Urban Drainage," Draft No. 3, Completed by the Canadian Urban Drainage Committee, 1977.

Overton, D.E., "Kinematic Flow on Long Impermeable Planes," Water Resources Bulletin, American Water Resources Association, Vol. 8, No. 6, December 1972, pp. 1198-1204.

Overton, D.E. and Meadows, M.E., Stormwater Modelling, Academic Press, New York, 1976.

Pansic, N. and Yen, B.C., "Kinematic Wave Modelling of Storm Sewers With Surcharge," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, USA, June 1981.

Papadakis, C.N. and Preul, H.C., "Urban Runoff Characteristics," Report No. 11024 DQU, Environmental Protection Agency, Washington, 1972.

Poertner, H.G., "Practices in Detention of Urban Stormwater Runoff," Special Report No. 43, American Public Works Association, Chicago, 1974.

Ponce, V.G., "Simplified Muskingum Routing Equation," Technical Notes, Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 105, No. HY1, January 1979, pp. 85-91.

Ponce, V.M., Chen, Y.H. and Simons, D.B., "Unconditional Stability in Convection Computations," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 105, No. HY9, September 1979, pp. 1079-1086.

Ponce, V.M., Indlekofer, H. and Simons, D.B., "Convergence of Four-Point Implicit Water Wave Model," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY7, July 1978a, pp. 947-958.

Ponce, V.M., Li, R.M. and Simons, D.B., "Applicability of Kinematic and Diffusion Models," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY3, March 1978b, pp. 353-360.

Ponce, V.M. and Yabusaki, S., Discussion of "Channel Routing By Finite Difference Method," by Huang, Y.H. (Proc. Paper 14088), Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 105, No. HY8, August 1979, pp. 1050-1052.

Ponce, V.M. and Yevjevich, V., "Muskingum-Cunge Method with Variable Parameters," Technical Notes, Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY12, December 1978, pp. 1663-1667.

Price, R.K., "Comparison of Four Numerical Methods for Flood Routing," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 100, No. HY7, July 1974, pp. 879-899.

Price, R.K., Discussion of "Channel Routing By Finite Difference Method," by Huang, Y.H. (Proc. Paper 14088), Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 105, No. HY8, August 1979, pp. 1049-1050.

Price, R.K., Discussion of "Approximate Flood Routing Methods: A Review," by P.E.Weinmann and Eric M. Laurenson (Proc. Paper 15057), Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 106, No. HY11, November 1980, pp. 1947-1948.

Price, R.K., "A Simulation Model for Storm Sewers," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

Resource Analysis, Inc., "MITCAT Catchment Simulation Model - Description and Users' Manual," Version 6, Cambridge, Massachusetts, September 1975.

Roesner, L.A., (Camp Dresser and McKee, Inc.), Personal Communications, 1979.

Roesner, L., Kassem, A. and Wisner, P., "Improvements to the SWMM," Proceedings, SWMM Users Group Meeting, Gainseville, Florida, January 1980.

Sangster, W.M., Wood, H.W., Smerdon, E.T. and Bossy, H.G., "Pressure Changes at Open Junctions in Conduits," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 85, No. HY6, June 1959, pp. 13-44.

Schaake, J.C., "Deterministic Urban Runoff Models," Chapter VIC, Treatise on Urban Water Systems, Colorado State University, Fort Collins, Colorado, July 1971.

Sevuk, A.S., Yen, B.C. and Peterson, G.E., "Illinois Storm Sewer Simulation Model - User's Manual," Research Report No. 73, Water Resources Center, University of Illinois at Urbana - Champaign, October 1973.

Silvio, G.D., "Flood Wave Modification Along Prismatic Channels," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 95, No. HY5, September 1969, pp. 1589-1614.

Sivaloganathan, K., "Free Surface Flow Computations By Characteristics," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY4, April 1974, pp. 543-556.

Sivaloganathan, K., "Flood Routing By Characteristic Methods," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY7, July 1978, pp. 1075-1091.

Sivaloganathan, K., "Channel Flow Computations Using Characteristics," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 105, No. HY7, July 1979, pp. 899-910.

Sjoberg, A., "Sewer Network Models DAGVL-A and DAGVL-DIFF," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

Smith, R.E. and Woolhiser, D.A., "Mathematic Solution of Infiltrating Watersheds," Hydrology Paper No. 47, Colorado State University, January 1971.

Sogreah, "Mathematical Model of flow simulation in Urban Sewerage Systems - CAREDAS Program," Societe Grenobloise d'Etudes et d'Applications Hydrauliques, Grenoble, 1973.

Streeter, V.L. and Wylie, E.B., Hydraulic Transients, McGraw-Hill, New York, 1967.

Strelkoff, T., "Numerical Solution of the Saint-Venant Equations," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 96, No. HY1, 1970, pp. 223-252.

Taylor, E.H., "Flow Characteristics at Regular Open Channel Junctions," Transactions, ASCE, Vol. 109, 1944, pp. 893-903.

Terstriep, M.L. and Stall, J.B., "The Illinois Urban Drainage Area Simulator: ILLUDAS," Bulletin 58, Illinois State Water Survey, Urbana, 1974.

Theil, P.E., "Improved Manual Methods of Design of Urban Storm Drainage Systems," Lecture Notes, APWA Seminar, 1978.

Tholin, A.L. and Keifer, C.J., "Hydrology of Urban Runoff," Trans., ASCE, Vol. 125, 1960, pp. 1308-1355.

Tourbier, J. and Westmacott, R., Water Resources Protection Measures in Land Development - A Handbook, Water Resources Center, University of Delaware, 1974.

Town of Markham, "Storm Drainage Policies and Criteria," Department of Engineering, 1978.

Town of Oakville, " Storm Drainage Policies and Criteria," Department of Public Works, 1979.

Townsend, D.R., "Hydraulic Model Study of Stormwater Catchbasin (Inlet) Control Devices," Preliminary Report, Unpublished, Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, July 1979.

Townsend, R.D. and Prins, J.R., "Performance of Model Storm Sewer Junctions," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 104, No. HY1, January 1978, pp. 99-104.

Townsend, D.R., Wisner, P.E. and Moss, D.J., "Inlet Control Devices for Stormwater Catchbasins: A Laboratory Study," Canadian Hydrology Symposium 80 - Hydrology of Developed Areas, Toronto, Ontario, May 1980.

Toyokuni, E., "Stormwater Runoff With Surcharge in Matsuyama," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

U.S. Department of Agriculture, "Technical Release No. 55, Urban Hydrology For Small Watersheds," Soil Conservation Service, January 1975.

U.S. Department of Transportation, "Design of Urban Highway Drainage - The State of the Art," Federal Highway Administration, Office of Research and Development, (FHWS-TS-79-226), August 1979.

Watkins, L.H., "The Design of Urban Storm Sewers," Technical Paper No. 55, Road Research Laboratory, Department of Science and Industrial Research, Great Britain, 1962.

Watkins, L.H., "A Guide for Engineers to the Design of Storm Sewer Systems," Road Note 35, Road Research Laboratory, Department of Science and Industrial Research, Great Britain, 1963.

Watt, W.E., "QUURM - Queen's University Urban Runoff Model," Second Canadian Hydrology Conference, Burlington, May 1975.

Weinmann, P.E. and Laurenson, E.M., "Approximate Flood Routing Methods: A Review," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 105, No. HY12, December 1979, pp. 1521-1536.

Wichman, W., (Representetive of the Dorsch Consult of W. Germany in Canada), Personal Communications, 1980.

Wisner, P.E., Gupta, S. and Kassem, A.M., "Considerations Regarding the Application of SCS TR55 Procedures For Runoff Computations," Proceedings, SWMM Users' Group Meeting, Toronto, June 1980.

Wisner, P.E. and Kassem, A.M., "Application of the SWM Model for the Planning and Design of Dual Storage," SWMM Users Group Meeting, Montreal, Quebec, Canada, May 1979.

Wisner, P.E. and Kassem, A.M., "Street Overland Flow and Inlet Control," Canadian Society For Civil Engineering, 1980 Annual Conference, Winnipeg, Manitoba, Canada, May 1980a.

Wisner, P.E. and Kassem, A.M., "Overcontrol of Urban Runoff By Park Storage," Canadian Hydrology Symposium: 80, Hydrology of Developed Areas, National Research Council, Toronto, Ontario, Canada, May 1980b.

Wisner, P.E., Kassem, A.M. and Cheung, P., "Parks Against Storms," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

Woolhiser, D., Holland, M., Smith, G. and Smith, R., "Experimental Investigation of Converging Overland Flow," Trans., American Society of Agriculture Engineers, Vol. 14, No. 4, 1971, pp. 684-687.

Woolhiser, D.a. and Liggett, J.A., "Unsteady, One-Dimensional Flow Over a Plane--The Rising Hydrograph," Water Resources Research, Vol. 3, No. 3, 1967, pp. 753-771.

Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual," Denver Regional Council of Governments, Denver, Colorado, 1968.

Wylie, E.B., "Control of Transient Free Surface Flow," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 95, No. HY1, January 1969, pp. 347-361.

Wylie, E.B., "Unsteady Free-Surface Flow Computations," Journal of the Hydraulics Division, Proceedings, ASCE, Vol. 96, No. HY11, November 1970, pp. 2241-2251.

Yen, B.C., "Methodologies For Flow Prediction in Urban Storm Drainage Systems," WRC Research Report No. 72, University of Illinois, Urbana, Illinois, August 1973.

Yen, B.C., Discussion of "Storm Routing Models," by Zaghloul, N.A., Proceedings of the Fort Collins Third International Hydrology Symposium, on Theoretical and Applied Hydrology, Colorado State University, Fort Collins, Colorado, July 1977, pp. 682-683.

Yen, B.C., "Workshop Notes on Storm Sewer Design," Water Resources Center, University of Illinois at Urbana - Champaign, Illinois, 1978a.

Yen, B.C., "Hydraulic Instabilities of Storm Sewer Flows," Proceedings, International Conference on "Urban Storm Drainage," University of Southampton, ed. P.R.Helliwell, Pentech Press, London, April 1978b, pp.282-293.

Yen, B.C., "Some Measures for Evaluation and Comparison of Simulation Models," Proceedings, Second International Conference on Urban Storm Drainage, Urbana, Illinois, June 1981.

Yen, B.C. and Sevuk, A.S., "Design of Storm Sewer Networks," Journal of the Environmental Engineering Division, ASCE, Vol. 101, No. EE4, August 1975, pp. 535-553.

Yen, B.C., Wenzel, H.G., Mays, L.W. and Tang, W.H., "Advanced Methodologies For the Design of Sewer Systems," Research Report No. 112, Water Resources Center, University of Illinois at Urbana - Champaign, Illinois, August 1976.

Yevjevich, V., "Storm Drain Networks," Unsteady Flow In Open Channels, Chapter 16, Vol. II, edited by K. Mahmood and V. Yevjevich, Water Resources Publications, Fort Collins, Colorado, 1975.

Yevjevich, V. and Barnes, A.H., Flood Routing Through Storm Drains, Part I, Solution of Problems of Unsteady Free Surface Flow in Storm Drains, Hydrology Paper No. 43, Colorado State University, Fort Collins, Colorado, November 1970a.

Yevjevich, V. and Barnes, A.H., Flood Routing Through Storm Drains, Part IV, Numerical Computer Methods For Solution, Hydrology Paper No. 46, Colorado State University, Fort Collins, Colorado, November 1970b.

Yevjevich, V. and Barnes, A.H., Flood Routing Through Storm Drains, Part II, Physical Facilities and Experiments, Hydrology Paper No. 44, Colorado State University, Fort Collins, Colorado, November 1970c.

Appendix A

NOTATION

The following symbols are used in this thesis:

a = routing coefficient;

A = cross sectional Area of flow;

A_f = cross sectional area of a pipe flowing full;

A_s = surface area of the junction at 0.96 full depth;

B = surface width of flow;

c = wave celerity or Kinematic wave speed;

c_1, c_2, c_3 = coefficients in Muskingum model;

D = pipe diameter;

F_o = steady uniform flow Froude number;

g = acceleration due to gravity;

H = junction water surface elevation (piezometric head), measured from datum;

H_1 = piezometric head in upstream junction, measured from datum;

H_2 = piezometric head in downstream junction, measured from datum;

I = I(t), inflow into channel/conduit;

i = subscripts, denoting distance interval;

j = subscript denoting junction; superscript denoting time increment;

k = kinematic wave number (dimensionless parameter);

K = propagation time of discharge;

L = conduit/channel length;

n = Manning's coefficient;

n_f = Manning's coefficient for a pipe flowing full;

n_s = Manning's coefficient of shoulder;

n_x = Manning's coefficient of street pavement;

q = lateral inflow per unit length;

Q = discharge;

Q_f = full flow capacity of pipe;

Q_b = base flow;

Q_c = carry-over flow, flow bypassing storm inlet;

$\bar{Q}_i$ = inlet capture;

Q_{in} = external inflow into junction;

Q_N = normal flow discharge;

Q_p = peak flow;

Q_s = approach (street) flow;

R = hydraulic radius;

S = storage volume;

S_f = friction slope;

S_o = longitudinal slope of channel;

t = time;

T_o = wave period, or hydrograph duration;

v = velocity of flow;

v_o = steady uniform flow mean velocity;

x = distance along channel/conduit;

y = depth of flow;

y_{cr} = height of road crown, measured from the edge of the curb;

y_{cur} = height of curb;

y_o = steady uniform flow depth;

Z_s = reciprocal of shoulder cross slope;

Z_x = reciprocal of pavement cross slope;

α = Courant number, ratio of wave to cell celerity;

β = routing coefficient;

ΔH = junction head adjustment;

Δt = time interval, routing period;

Δx = space interval, reach length;

ε = diffusion coefficient;

η = $\eta(x,t)$, a variable denoting flow, velocity, depth, etc.;

ϕ = cell Reynold number, ratio of physical to cell diffusivity;

ψ = a weighting factor;

τ = time of propagation of discharge, used in Kalinin-Miljukov model;

θ = a weighting factor;

ξ = a dimensionless parameter;

ξ^+ = denoting positive characteristics curve;

ξ^- = denoting negative characteristics curve.

Appendix B

SEWER FLOW ROUTING TECHNIQUE BY EXTRAN MODEL

The basic differential equations for the sewer flow problem come from the gradually varied unsteady flow equations for open channels. In EXTRAN these equations are expressed as follows:

Momentum

$$\frac{\partial Q}{\partial t} = -gAS_f + 2v\frac{\partial A}{\partial t} + v^2\frac{\partial A}{\partial x} - gA\frac{\partial H}{\partial x} \qquad \text{B.1}$$

Continuity

$$\left(\frac{\partial H}{\partial t}\right)_t = \frac{\Sigma Q_t}{(A_s)_t} \qquad \text{B.2}$$

where

Q = discharge through the conduit;

v = velocity in the conduit;

A = cross-sectional area of flow;

H = hydraulic head;

S_f = friction slope;

ΣQ = net inflow into node; and

A_s = surface area of node.

The friction slope is defined by Manning's equation, i.e.

$$S_f = \frac{k}{gAR^{4/3}} Q|v| \qquad \text{B.3}$$

where $k = g\ (n\ /\ 1.49)^2$, R is the hydraulic radius, and n is Manning's coefficient.

In EXTRAN the continuity and momentum equations are not solved simultaneously. The momentum equation is considered for conduit flow while the continuity equation is applied to the junctions.

In finite-difference form, equations B.1 and B.2, respectively are written as:

$$Q_{t+\Delta t} = \left[\frac{1}{1 + \frac{k.\Delta t}{\bar{R}^{4/3}} |\bar{v}|}\right] \left[Q_t + 2\bar{v}\ \Delta A + \bar{v}^2 \frac{A_2 - A_1}{L} \Delta t - g\bar{A} \frac{H_2 - H_1}{L} \Delta t\right] \qquad \text{B.4}$$

and

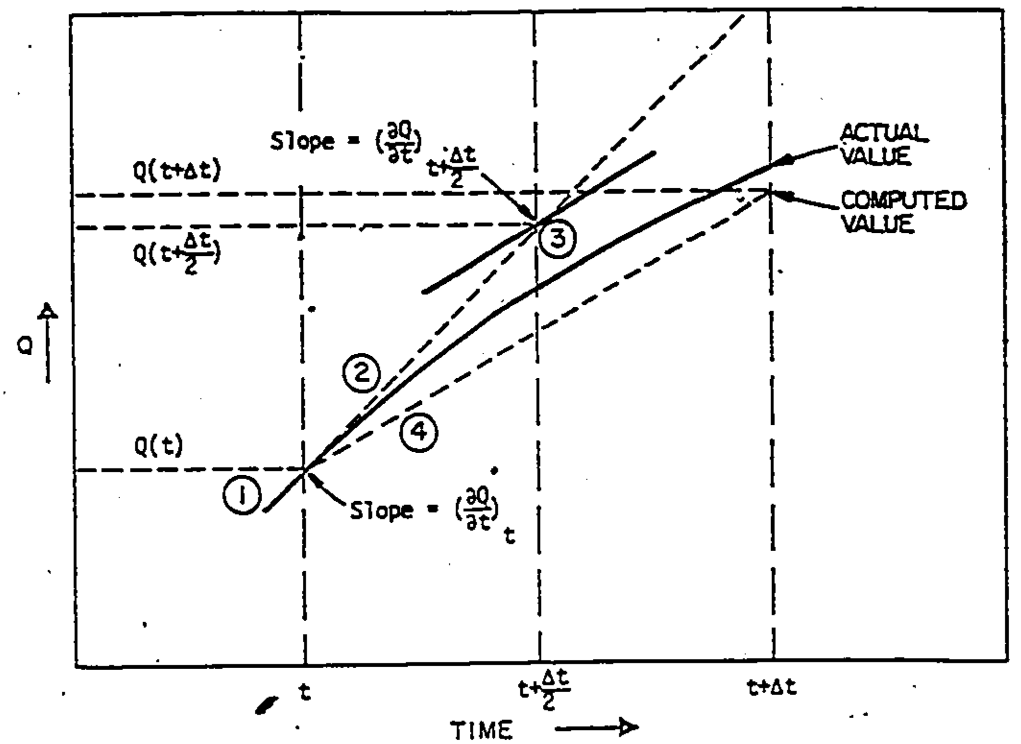
$$H_{t+\Delta t} = H_t + \frac{\Sigma Q_t\ \Delta t}{(A_s)_t} \qquad \text{B.5}$$

In Eq. B.4, the values $\overline{V}$, $\overline{R}$, and $\overline{A}$ are weighted averages of the conduit end values at time t. The surface area of a junction is defined as the surface area of flow in half the lengths of the pipes connected to the junction.

Eqs. B.4 and B.5 are solved sequentially to determine discharge in each link and head at each node over a time step Δt. The numerical integration is accomplished by a modified Euler method. Figure B.1 shows how the process would work if only the discharge equation were involved. The first three operations determine the slope $\partial Q / \partial t$ at the "half-step". This is used in operation (4) to project the "full step" value of discharge. In other words, it is assumed that the slope at time t is the mean slope during the interval. The method is extended easily to more than one equation, although graphic representation is then very difficult. The corresponding half-step and full-step calculations of head are shown below:

Half-step at node j: Time $t + \frac{\Delta t}{2}$

$$H_j\left(t + \frac{\Delta t}{2}\right) = H_j(t) + \left\{ \frac{1}{2} \underset{\text{conduits}}{\Sigma}\left[Q(t) + Q\left(t + \frac{\Delta t}{2}\right)\right] + \underset{\text{diversions}}{\Sigma} Q\left(t + \frac{\Delta t}{2}\right) \right\} / A_s(t) \qquad \text{B.6}$$



(1) Compute $(\frac{\partial Q}{\partial t})_t$ from properties of system at time t

(2) Project $Q(t+\frac{\Delta t}{2})$ as $Q(t+\frac{\Delta t}{2}) = Q(t) + (\frac{\partial Q}{\partial t})_t \frac{\Delta t}{2}$

(3) a. Compute system properties at $t+\frac{\Delta t}{2}$

b. Form $(\frac{\partial Q}{\partial t})_{t+\frac{\Delta t}{2}}$ from properties of system at time $t+\frac{\Delta t}{2}$

(4) Project $Q(t+\Delta t)$ as $Q(t+\Delta t) = Q(t) + (\frac{\partial Q}{\partial t})_{t+\frac{\Delta t}{2}} \Delta t$

Fig. B.1 Modified Euler Solution Method For Discharge Based on Half-Step, Full Step Projection

Full-step at node j: Time $t + \Delta t$

$$H_j(t + \Delta t) = H_j(t) + \{ \frac{1}{2} \sum_{\text{conduits}} [Q(t) + Q(t + \Delta t)] + \sum_{\text{diversions}} Q(t + \Delta t) \} / A_s(t) \qquad B.7$$

Note that the half-step computation of head uses the half-step computation of discharge in all connecting conduits. Similarly, the full-step head computation requires the full-step discharge at time $t + \Delta t$ for all connecting pipes.

A master flow chart of EXTRAN is presented in Fig. B.2, which is developed in the frame of present research. The rules for assigning the junction surface area and weighted averages of velocity, cross sectional area and hydraulic radius are described in Fig. B.4, which is self explanatory. Reference should also be made to Fig. B.3 which shows the variables associated with conduits and junctions.

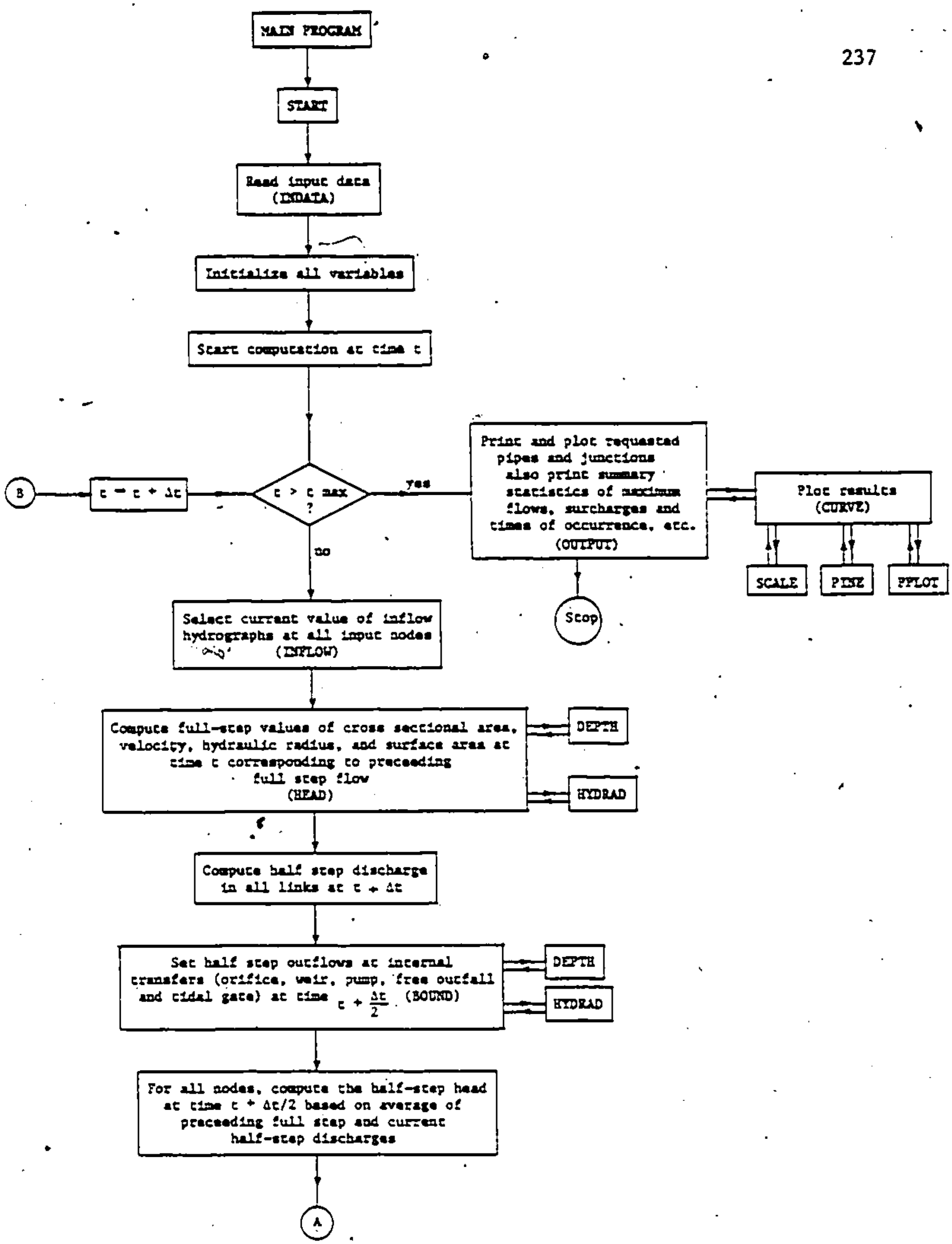


Fig. B.2 Master Flow Chart of EXTRAN Model

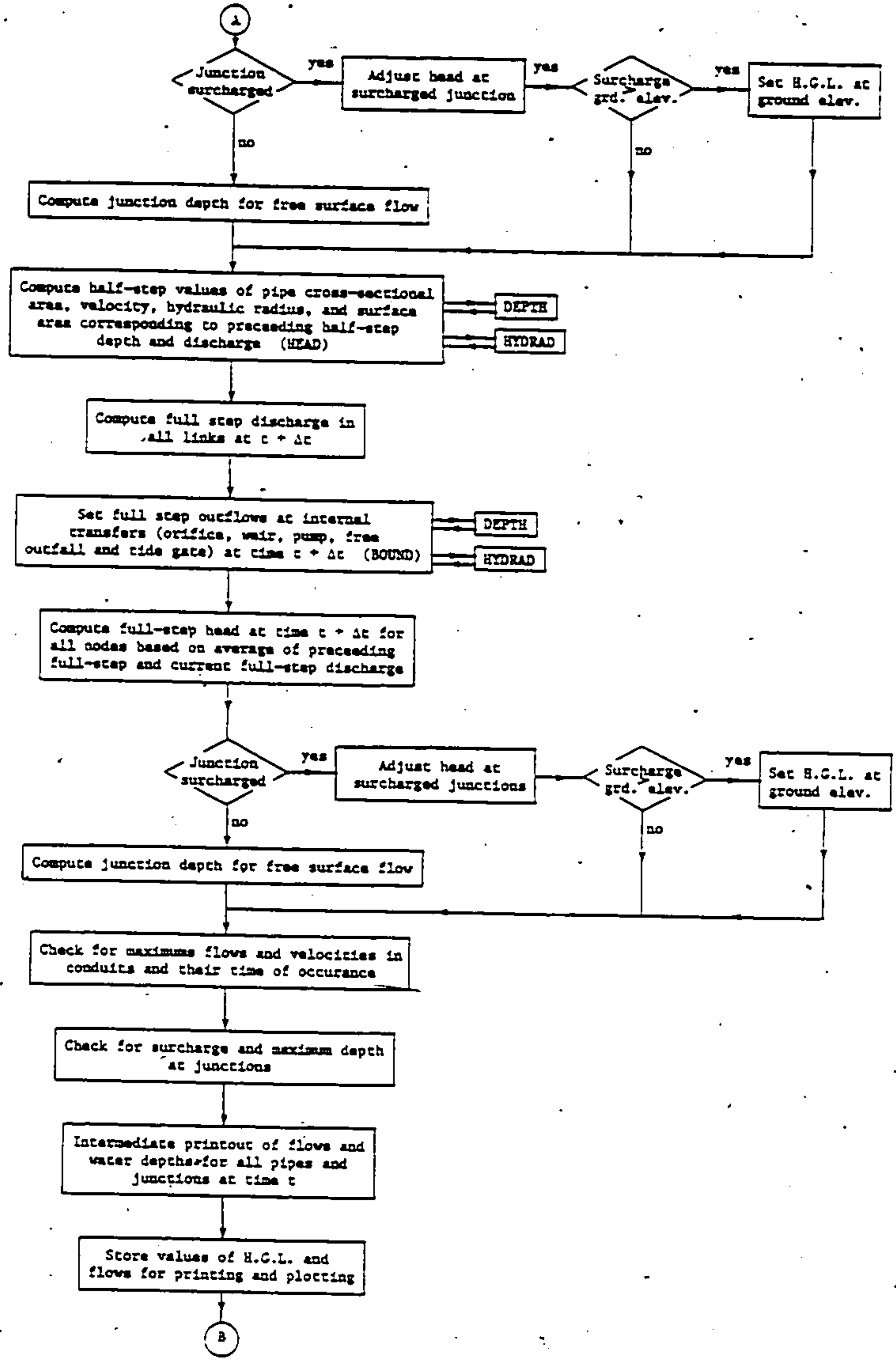
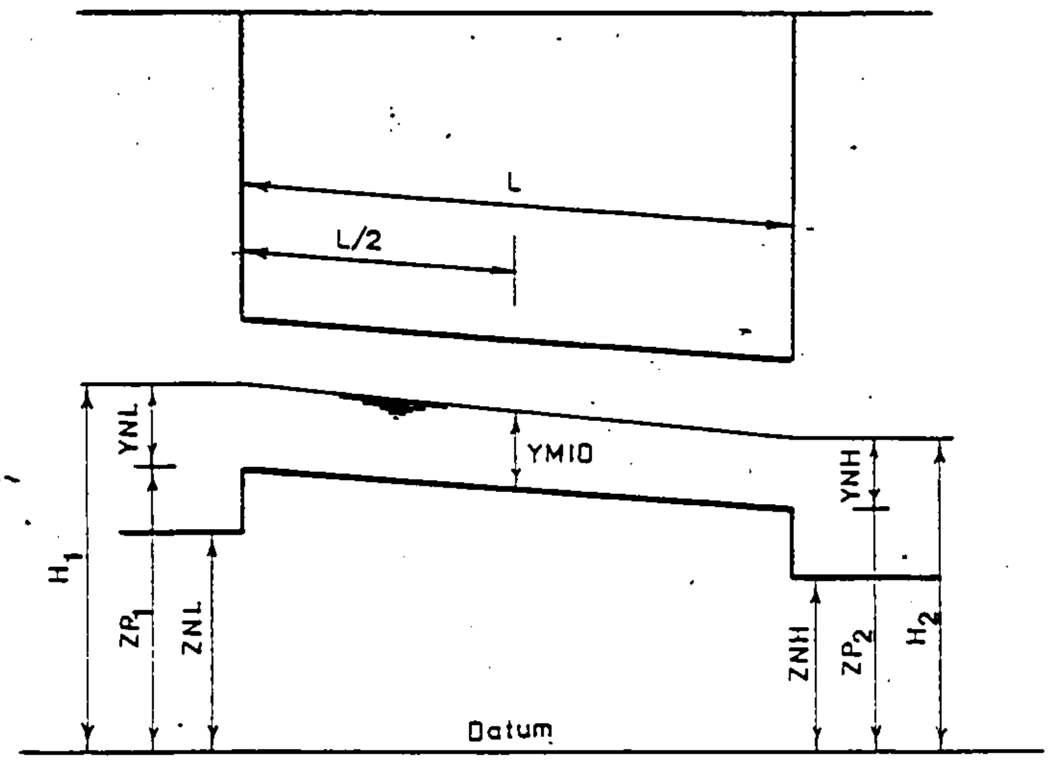

A
Junction surcharged
yes
Adjust head at surcharged junction
yes
Surcharge grd. elev.
yes
Set H.G.L. at ground elev.
no
no
Compute junction depth for free surface flow
Compute half-step values of pipe cross-sectional area, velocity, hydraulic radius, and surface area corresponding to preceding half-step depth and discharge (HEAD)
DEPTH
HYDRAD
Compute full step discharge in all links at t + Δt
Set full step outflows at internal transfers (orifice, weir, pump, free outfall and tide gate) at time t + Δt (BOUND)
DEPTH
HYDRAD
Compute full-step head at time t + Δt for all nodes based on average of preceding full-step and current full-step discharge
Junction surcharged
yes
Adjust head at surcharged junctions
Surcharge grd. elev.
yes
Set H.G.L. at ground elev.
no
no
Compute junction depth for free surface flow
Check for maximums flows and velocities in conduits and their time of occurance
Check for surcharge and maximum depth at junctions
Intermediate printout of flows and water depths for all pipes and junctions at time t
Store values of H.G.L. and flows for printing and plotting
B

Fig. B.2 Cont'd



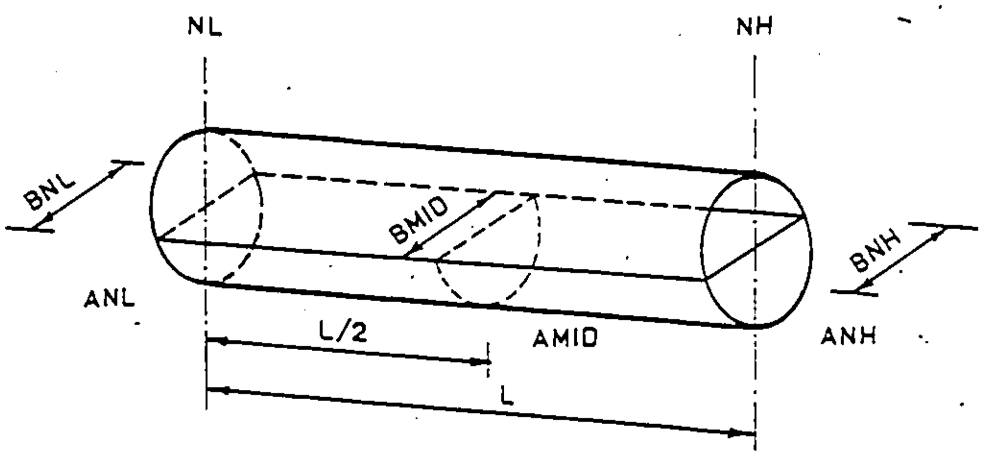


Fig. B.3 Variables Associated With Conduits and Junctions Used By EXTRAN Model

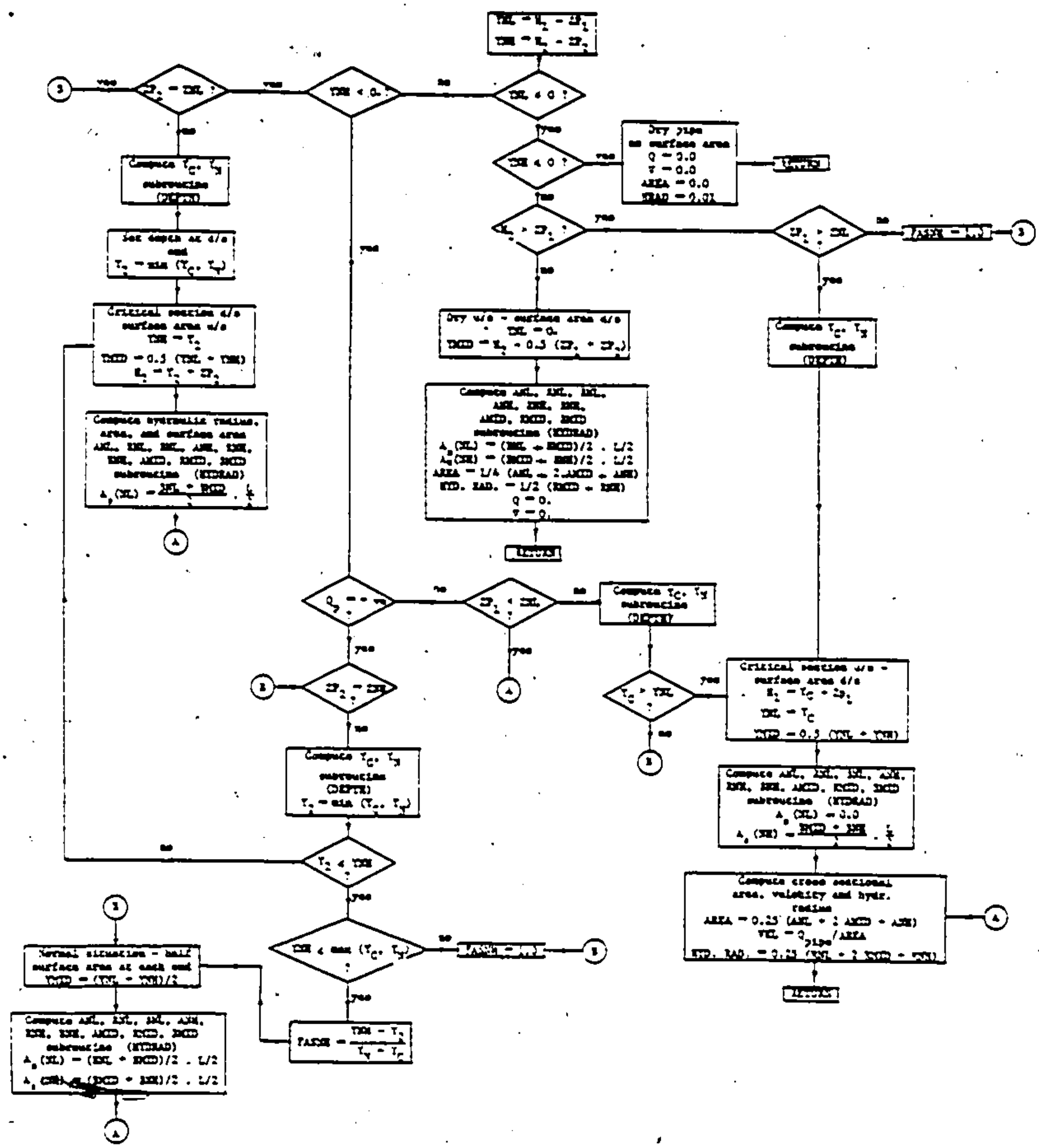
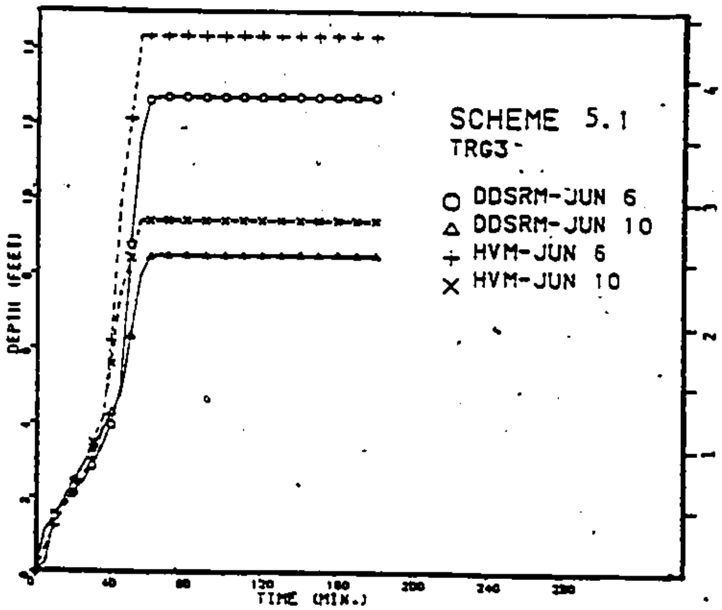


Fig. B.4 Flow Chart for the Rules of Assigning the Junction Surface Area and Weighted Averages of Velocity, Cross Sectional Area and Hydraulic Radius Used By EXTRAN Model

Appendix C

SUPPLEMENTARY FIGURES FOR SECTION 5.6.2



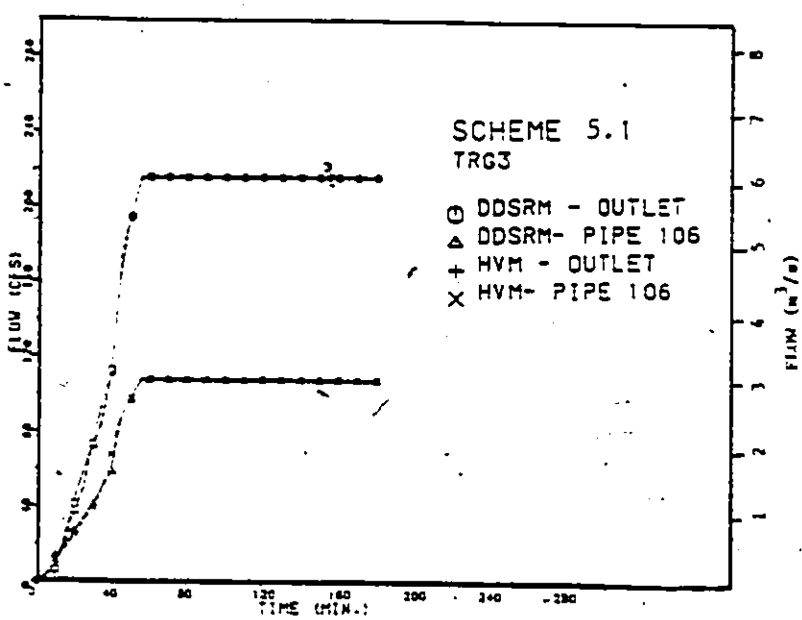


Fig. C.1 Prediction of the Time-Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.1; TRG3

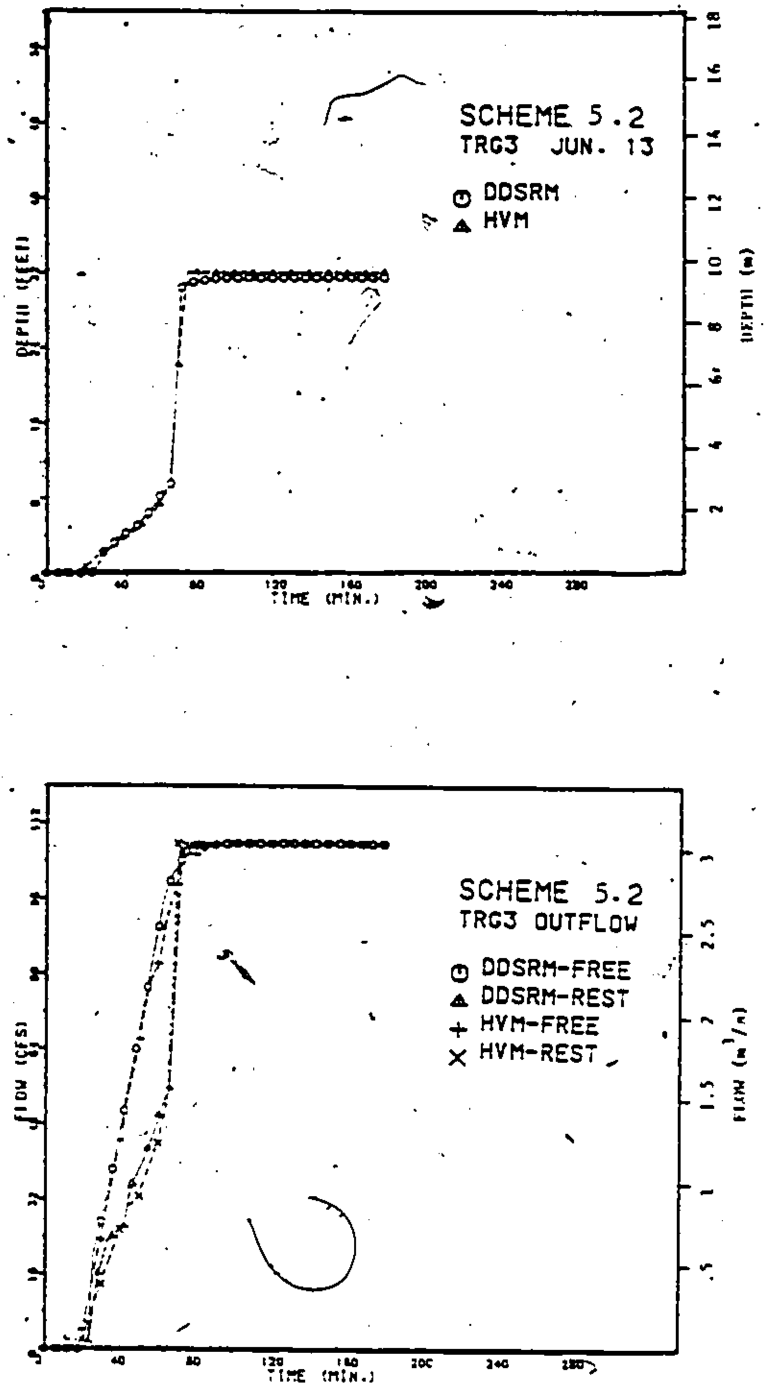
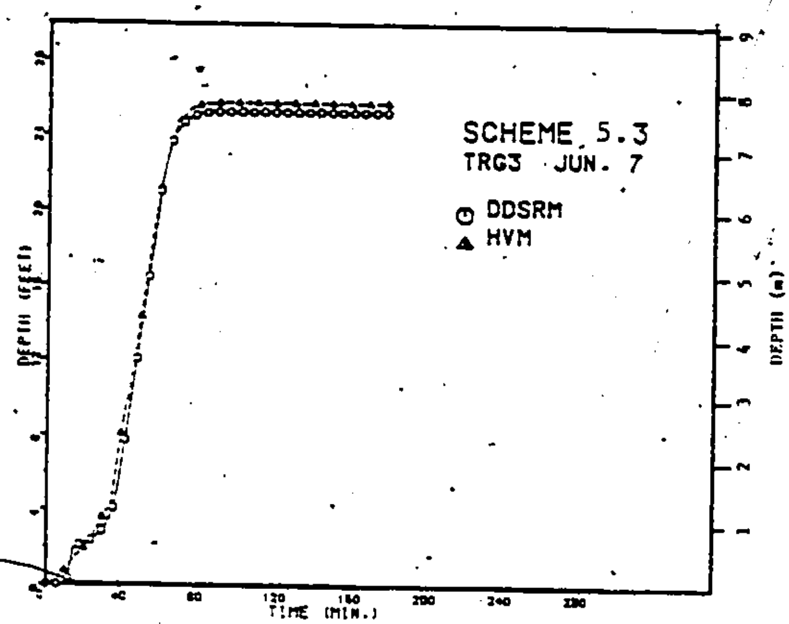


Fig. C.2 Prediction of the Time Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.2; TRG3



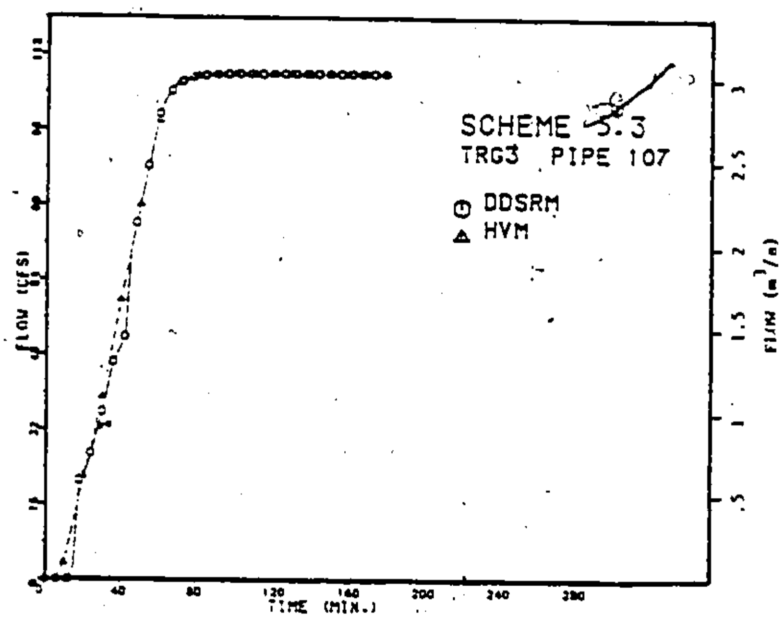
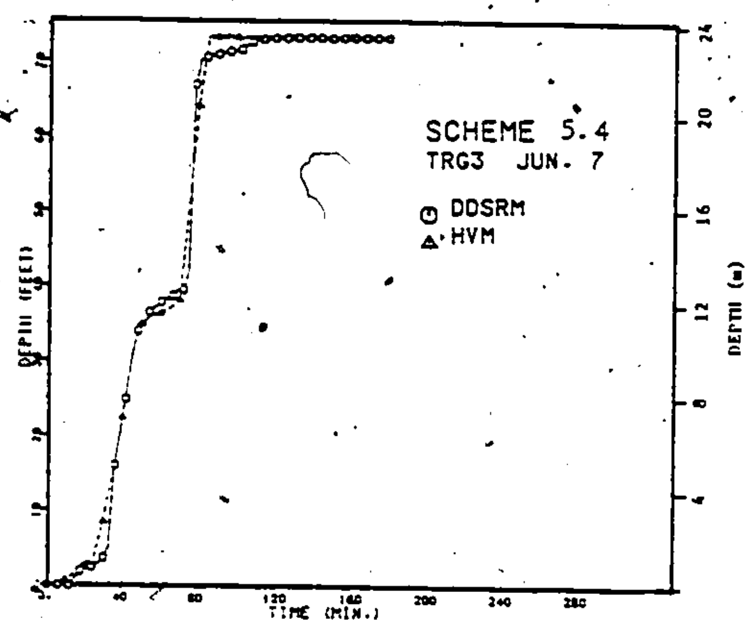


Fig. C.3 Prediction of the Time Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.3; TRG3



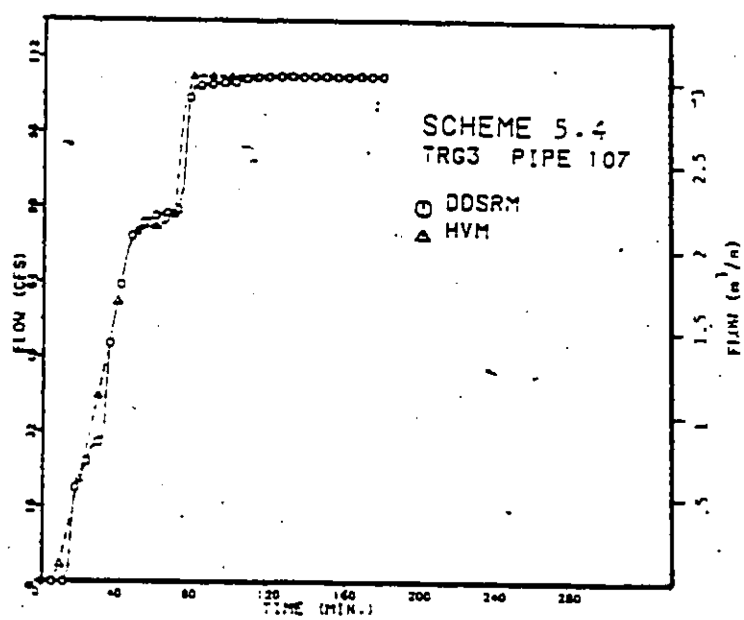


Fig. C.4 Prediction of the Time Histories of Pipe Flow and Junction Water Depth Under Surcharge, by IMPSWM-EXTRAN (DDSRM) and HVM Models - Scheme 5.4; TRG3

Appendix D

INPUT SPECIFICATION AND DESCRIPTION OF THE INPUT VARIABLES FOR DDSRM

Card Group	Format	Card Column	Description	Variable Name
1			Run specification.	
1A	20A4	1-80	Title to be printed with output (2 cards).	ITITL
1B			Control data for run.	
	F5.0	1-5	Time increment for calculation - minutes.	DELTC
	I5	6-10	Number of computational time steps.	NDELTC
	I5	11-15	Options for sewer network analysis. IEVAL = 0, if pipe sizes are to be determined for free surface flow. IEVAL = 1, if pipe sizes are given and it is required to revise these sizes if necessary based on free surface flow. IEVAL = 2, if pipe sizes are given and it is required to determine inlet restriction requirements for free surface flow. IEVAL = 3, surcharge sub-model is used for sewer flow routing - inlet flows will be saved on a disk file.	IEVAL
	I5	16-20	Option for storage computations for major system flow. JSS = 0, if storage computation for major system flow is not required. JSS ≠ 0, if storage computation is required.	JSS
	I5	21-25	Option for storage computation for minor system flow.	JPP

Card Group	Format	Card Column	Description	Variable Name
			JPP = 0, if storage computation for minor system flow is not required.	
			JPP ≠ 0, if storage computation is required.	
	I5	26-30	Option for catchment runoff.	IWS
			IWS = 0, if catchment runoff is to be calculated.	
			IWS ≠ 0, if catchment runoff is input directly.	
	F5.0	31-35	Limiting capacity of inlets, - cfs. Value can be overridden for individual inlets by the values assigned to (QLIM(I)) on card group (7).	QLIMIT
	I5	36-40	IRTS = 0, routing of major system flow is included.	IRTS
			IRTS ≠ 0, no routing of major system flow.	
2			Rainfall data - required only if IWS = 0 on card group (1B).	
2A	F5.0	1-5	Time increment for hyetograph - minutes.	DELTR
	I5	6-10	Number of time increments of hyetograph.	NDELTR
2B	16F5.0	1-80	Rainfall intensities - in/hr. 16 values per card, up to NDELTR.	RINT(I)
3			Minor system data - required only if IEVAL ≠ 3, otherwise skip to card group (5).	
3A	I5	1-5	Option for routing method for minor system flow - leave blank (or zero). Not used at present.	IOPTRP

Card Group	Format	Card Column	Description	Variable Name
3B			One card per sewer pipe.	
	I5	1-5	Sewer identification number.	NP(I)
	I5	6-10	Identification number of downstream sewer.	NPD(I)
	F5.0	11-15	Length of sewer - ft.	XP(I)
	F5.0	16-20	Invert slope - ft./ft.	SP(I)
	F5.0	21-25	Manning's roughness coefficient - sec/ft.$^{1/3}$	CP(I)
	F5.0	26-30	Pipe diameter - inches. Leave blank if IEVAL = 0 on card group (1B).	DP(I)
	F5.0	31-35	Dry weather flow - cfs.	QB(I)
	I5	36-40	Identification number of underground storage. Required only if pipe is connected to storage unit, otherwise leave blank. Number must correspond to (IDSTP) on card group (4).	NPPS(I)
	I5	41-45	IPRTP(I) = integer number, time history of pipe flow will be printed and plotted. IPRTP(I) = 0 or blank, no printing or plotting is provided. Blank card to terminate minor system data.	IPRTP(I)
4			Data for minor system storage. Repeat card group (4) for each storage unit. If no storage for minor system (JPP = 0 on card group (1B)), skip to card group (5).	
4A	I5	1-5	Option for routing through storage.	IOPTSP(I)

Card Group	Format	Card Column	Description	Variable Name
			IOPTSP(I) = 1, storage volume computed based on constant outflow equal to maximum release rate - no overflow.	
			IOPTSP(I) = 2, overflow is calculated for a given storage volume and constant outflow equal to maximum release rate.	
			IOPTSP(I) = 3, flows routed through given storage volume based on storage-outflow relationship - overflows calculated.	
4B			Required only if IOPTSP(I) = 1.	
	I5	1-5	Identification number of storage unit (numbers are those assigned to NPPS on card group (3B).	IDSTP(I)
	F5.0	6-10	Maximum release rate - cfs.	QRLSP(I)
4C			Required only if IOPTSP(I) = 2.	
	I5	1-5	Identification number of storage unit (numbers are those assigned to NPPS on card group (3B).	IDSTP(I)
	F10.0	6-15	Available storage volume - cu.ft.	SPMX(I)
	F5.0	16-20	Maximum release rate - cfs.	QRLSP(I)
4D			Required only if IOPTSP(I) = 3. Must be followed by card group (4E).	
	I5	1-5	Identification number of storage unit (numbers are those assigned to NPPS on card group (3B).	IDSTP(I)
	F10.0	6-15	Available storage volume - cu.ft.	SPMX(I)

Card Group	Format	Card Column	Description	Variable Name
	F5.0	16-20	Number of points defining storage-outflow relationship.	NSP1(I)
4E			Required only if IOPTSP(I) = 3. Must follow card group (4D).	
	8F10.2	1-10	First point: storage volume on storage-outflow relation - cu.ft.	SP1(1)
		11-20	First point: release rate on storage-outflow relation - cfs.	RP1(1)
		21-30	Second point: storage volume on storage-outflow relation - cu.ft.	SP1(2)
		31-40	Second point: release rate on storage-outflow relation - cfs.	RP1(2)
				SP1(NSP1(I)) RP1(NSP1(I))
			Repeat up to NSP1(I) points (8 values per card).	
			Blank card to terminate data for minor system storage.	
5			Major system characteristics data.	
5A	I2	1-2	Number of segment types.	NSSEG
5B			Repeat card group (5B) NSSEG times.	
	I5	1-5	Identification number of segment type.	IS
	F5.0	6-10	Segment width (from curb to road crown) - ft.	W2
	F5.0	11-15	Pavement cross slope - ft./ft.	SX

Card Group	Format	Card Column	Description	Variable Name
	F5.0	16-20	Height of curb - inches.	YCURB
	F5.0	21-25	Manning's roughness coefficient for pavement - sec/ft. 1/3	CNS
	F5.0	26-30	Longitudinal slope - ft./ft.	SO
	F5.0	31-35	Cross slope of shoulder ft./ft.	SROW
	F5.0	36-40	Manning's roughness coefficient for shoulder - sec/ft.1/3	CNROW
	F5.0	41-45	Maximum water depth (measured from bottom of curb) for calculation of street carrying capacity - ft.	YMAX
6			Inlet capture data.	
6A	I5	1-5	Number of inlet types.	NINCAP
			Repeat card groups (6B) and (6C) NINCAP times - card group (6C) must follow card group (6B).	
6B	I5	1-5	Identification number of inlet type.	IDCB
	I5	6-10	Number of points on capture curve.	NJC
6C	16F5.0	1-5	First point: approach flow on capture curve - cfs.	QAPP(J,1)
		6-10	First point: captured flow on capture curve - cfs.	QCAP(J,1)
		11-15	Second point: approach flow on capture curve - cfs.	QAPP(J,2)
		16-20	Second point: captured flow on capture curve - cfs.	QCAP(J,2)
				QAPP(J,NJC) QCAP(J,NJC)

Card Group	Format	Card Column	Description	Variable Name
			Repeat up to NJC points - 16 values per card.	
7			Major system data. One card per segment.	
	I5	1-5	Identification number of street segment.	NS(I)
	I5	6-10	Identification number of downstream segment.	NSD(I)
	F5.0	11-15	Length of segment - ft.	XS(I)
	I5	16-20	Segment type. Number must be identical to (IS) on card group (5B) if they have the same characteristics.	IDSS(I)
	I5	21-25	Number of storm inlets within street segment.	NUMCB(I)
	I5	26-30	Inlet type. Number must be identical to (IDCB) on card group (6B) if they have the same characteristics.	IINC(I)
	F5.0	31-35	Maximum allowable capture - cfs. If left blank (or zero), value specified for (QLIMIT) on card group (1B) will be used.	QLIM(I)
	I5	36-40	Identification number of pipe receiving inlet flow.	NPD(I)
	I5	41-45	Identification number of surface storage unit for segment outflow. Required only if segment is connected to storage, otherwise leave blank. Number must correspond to (IDSTS) on card group (8).	NSTS(I)

Card Group	Format	Card Column	Description	Variable Name
	I5	46-50	IPRTS(I) = integer number, time history of segment flow and depth will be printed and plotted. IPRTS(I) = 0 or blank, no printing or plotting is provided.	IPRTS(I)
			Blank card to terminate major system data.	
8			Major system storage data. One card per storage unit. If no storage for major system flow (JSS = 0 on card group (1B), skip to card group (9).	
	I5	1-5	Identification number of first storage unit (numbers are those assigned to NSTS(I) on card group (7)).	IDSTS(1)
				IDSTS(2)
			Blank card to terminate data for major system storage.	
9			Sub-catchment data.	
			Card group (9A) and card group (9B) are required only if IWS = 0 on card group (1B). If IWS ≠ 0, skip card group (9A). The only data required on card group (9B) is for NW(I).	
9A			Infiltration parameters. Required only if IWS = 0 on card group (1B).	
	F8.0	1-8	Maximum infiltration rate - in/hr.	INFMAX
	F8.0	9-16	Minimum infiltration rate - in/hr.	INFMIN

Card Group	Format	Card Column	Description	Variable Name
	F8.0	17-24	Decay rate - sec^{-1}.	DECAY
9B			One card per subarea. If IWS ≠ 0 on card group (1B), the only data required is for NW(I).	
	I5	1-5	Identification number of sub-watershed.	NW(I)
	I5	6-10	Identification number of street segment for drainage.	NSD(I)
	F5.0	11-15	Area of sub-watershed - acres.	ASW
	F5.0	16-20	Percentage imperviousness.	PIMP(I)
	F5.0	21-25	Manning's roughness coefficient - $sec/ft.^{1/3}$ for impervious area.	CNIMP(I)
	F5.0	26-30	Manning's roughness coefficient - $sec/ft.^{1/3}$ for pervious area.	CNP(I)
	F5.0	31-35	Average slope - ft./ft.	S(I)
	F5.0	36-40	Sub-catchment width - ft.	WLAT(I)
	F5.0	41-45	Detention storage for impervious area - inches.	DETIMP(I)
	F5.0	46-50	Detention storage for pervious area - inches.	DETP(I)
	I5	51-55	IPRTW(I) = integer number, surface runoff hydrograph will be printed and plotted. IPRTW(I) = 0 or blank; no printing or plotting is provided.	IPRTW(I)
			Blank card to terminate sub-catchment data.	
9C			Catchment supply rate. Required only if IWS ≠ 0 on card group (1B).	

Card Group	Format	Card Column	Description	Variable Name
	16F5.0		Catchment supply rate at increments of DELTC for first sub-area - cfs. (NDELTC values: I = 1,2,...NDELTC)	QR(I,1)
			Catchment supply rate at increments of DELTC for second sub-area - cfs. (NDELTC values: I = 1,2,...NDELTC).	QR(I,2)
				QR(I,J)
			Repeat for all sub-areas in the same order as they appear on card group (9B).	

Appendix E

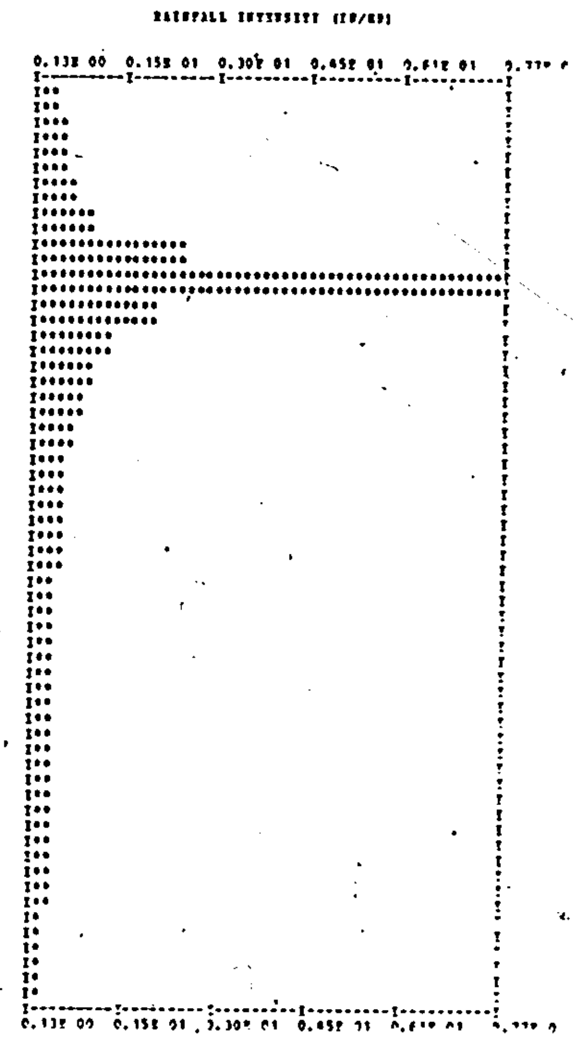
CASE STUDY - TESTVILLE 7.1 - DDSRM COMPUTER OUTPUT FOR THE 100-YEAR DESIGN STORM

```
**************************************************************************
* D U A L   D R A I N A G E   S I M U L T A N E O U S   R O U T I N G   M O D E L *
*                                   D D S R M                                      *
**************************************************************************

D D S R M - EXAMPLE INPUT - OUTPUT  - TESTVILLE 7.1
100 - YEAR STORM

TIME INCREMENT FOR CALCULATION        5.00  MINUTES
NUMBER OF COMPUTATIONAL STEPS           40
LIMITING CAPACITY OF INLETS           1.50  CFS
TOTAL SIMULATION TIME               195.00  MINUTES

PIPE SIZES WILL BE REVISED FOR FREE SURFACE FLOW CONDITIONS

STORAGE COMPUTATION FOR MAJOR SYSTEM FLOW IS REQUESTED

STORAGE COMPUTATION FOR MINOR SYSTEM FLOW IS REQUESTED
```

U D S R R - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

RAINFALL DATA

TIME (MIN)	RAINFALL (IN/HR)
0.0	0.26
5.00	0.26
10.00	0.31
15.00	0.32
20.00	0.38
25.00	0.39
30.00	0.51
35.00	0.51
40.00	0.79
45.00	0.79
50.00	2.29
55.00	2.29
60.00	7.71
65.00	7.71
70.00	1.93
75.00	1.93
80.00	1.10
85.00	1.10
90.00	0.78
95.00	0.78
100.00	0.62
105.00	0.62
110.00	0.51
115.00	0.51
120.00	0.44
125.00	0.44
130.00	0.39
135.00	0.39
140.00	0.35
145.00	0.35
150.00	0.32
155.00	0.31
160.00	0.29
165.00	0.28
170.00	0.27
175.00	0.26
180.00	0.25
185.00	0.24
190.00	0.24
195.00	0.23
200.00	0.22
205.00	0.22
210.00	0.21
215.00	0.21
220.00	0.20
225.00	0.20
230.00	0.19
235.00	0.19
240.00	0.18
245.00	0.18
250.00	0.17
255.00	0.17
260.00	0.16
265.00	0.16
270.00	0.15
275.00	0.15
280.00	0.14
285.00	0.14
290.00	0.13
295.00	0.13



RAINFALL DURATION 300.00 MINUTES

D D S B M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MINOR SYSTEM DATA

NO.	SEWER NUMBER	D/S SEWER	LENGTH (FT)	SLOPE (FT/FT)	MANNING (N)	DIAMETER (IN)	DRY WEATHER FLOW (CFS)	STORAGE ID NUMBER	HYDROGRAPH PRINTED
1	1	2	280.000	0.005	0.013	12.000	0.0	0	NO
2	2	3	225.000	0.005	0.013	15.000	0.0	0	NO
3	3	4	51.000	0.005	0.013	15.000	0.0	0	NO
4	4	20	246.000	0.005	0.013	15.000	0.0	0	NO
5	5	6	180.000	0.005	0.013	12.000	0.0	0	NO
6	6	7	225.000	0.005	0.013	15.000	0.0	0	NO
7	7	18	278.000	0.005	0.013	15.000	0.0	0	NO
8	8	9	213.000	0.005	0.013	15.000	0.0	0	NO
9	9	10	89.000	0.005	0.013	12.000	0.0	0	NO
10	10	11	294.000	0.005	0.013	18.000	0.0	0	NO
11	11	12	41.000	0.005	0.013	21.000	0.0	0	NO
12	12	21	258.000	0.005	0.013	21.000	0.0	0	NO
13	13	14	395.000	0.005	0.013	18.000	0.0	0	NO
14	14	15	228.000	0.005	0.013	21.000	0.0	0	NO
15	15	16	254.000	0.005	0.013	21.000	0.0	0	NO
16	16	22	172.000	0.005	0.013	24.000	0.0	0	YES
17	17	18	149.000	0.005	0.013	12.000	0.0	0	NO
18	18	19	147.000	0.005	0.013	21.000	0.0	0	NO
19	19	20	210.000	0.005	0.013	21.000	0.0	0	NO
20	20	21	270.000	0.005	0.013	24.000	0.0	0	YES
21	21	22	276.000	0.005	0.013	33.000	0.0	0	NO
22	22	23	265.000	0.005	0.013	36.000	0.0	0	NO
23	23	24	165.000	0.005	0.013	36.000	0.0	201	YES

TOTAL NUMBER OF PIPES = 23

```
D D S P M - EXAMPLE INPUT - OUTPUT  - TESTVILLE 7.1
100 - YEAR STORM

                    DATA FOR MINOR SYSTEM STORAGE

 NO. OF STORAGE UNITS       1

1 DETENTION STORAGE ID. NO.      201

DETENTION VOLUME            30000.00 CU. FT.

        STORAGE - OUTFLOW  RELATIONSHIP
          STORAGE           OUTFLOW
          (CU.FT)            (CFS)

             0.0              0.0
          6000.00             2.00
         12000.00             4.00
         18000.00             6.00
         24000.00             8.00
         30000.00            10.00
```

D D S P M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MAJOR SYSTEM RATING CURVE

NO. OF GEOMETRIES 2

TYPE	PAVEMENT WIDTH (FT)	PAVEMENT CROSS SLOPE (FT/FT)	HEIGHT OF CURB (FT)	MANNING (N)	LONG. SLOPE (FT/FT)	SHOULDER CROSS SLOPE (FT/FT)	SHOULDER ROUGHNESS N	MAXIMUM FLOW DEPTH (FT)
1	15.00	0.020	0.50	0.013	0.010	0.050	0.025	1.50

RATING CURVE

DEPTH (IN)	FLOW (CFS)	SPREAD (FT)
0.0	0.0	0.0
1.20	0.92	5.00
2.40	5.86	10.00
3.60	17.28	15.00
4.80	36.29	15.00
6.00	61.62	15.00
7.20	92.64	17.00
8.40	129.52	19.00
9.60	172.43	21.00
10.80	221.53	23.00
12.00	276.98	25.00
13.20	338.96	27.00
14.40	407.64	29.00
15.60	483.19	31.00
16.80	565.80	33.00
18.00	655.69	35.00

TYPE	PAVEMENT WIDTH (FT)	PAVEMENT CROSS SLOPE (FT/FT)	HEIGHT OF CURB (FT)	MANNING (N)	LONG. SLOPE (FT/FT)	SHOULDER CROSS SLOPE (FT/FT)	SHOULDER ROUGHNESS N	MAXIMUM FLOW DEPTH (FT)
2	20.00	0.020	0.50	0.013	0.010	0.050	0.025	1.50

RATING CURVE

DEPTH (IN)	FLOW (CFS)	SPREAD (FT)
0.0	0.0	0.0
1.20	0.92	5.00
2.40	5.86	10.00
3.60	17.28	15.00
4.80	37.22	20.00
6.00	66.56	20.00
7.20	104.06	22.00
8.40	149.45	24.00
9.60	202.69	26.00
10.80	263.78	28.00
12.00	332.77	30.00
13.20	409.76	32.00
14.40	494.84	34.00
15.60	588.14	36.00
16.80	689.79	38.00
18.00	799.92	40.00

D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

INLET DATA

NO. OF INLET TYPES 1

INLET TYPE = 1

NO. OF POINTS 13

APPROCH FLOW (CFS)	INLET FLOW (CFS)
0.0	0.0
1.00	0.67
2.00	1.04
3.00	1.30
4.00	1.55
5.00	1.75
6.00	1.90
7.00	2.00
9.00	2.10
20.00	2.10
100.00	2.10
200.00	2.10
300.00	2.10

D D S B M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MAJOR SYSTEM DATA

NO.	STREET SEGMENT	D/S SEGMENT	LENGTH (FT)	TYPE	NO. OF C.B.	INLET TYPE	INLET RESTRIC.	CONNECTING PIPE	STORAGE ID NO.	FLOW HISTORY
1	1	2	195.0	1	0	1	1.50	18	0	NO
2	2	23	160.0	1	2	1	1.50	18	0	NO
3	3	4	145.0	1	2	1	1.50	5	0	NO
4	4	5	200.0	1	2	1	1.50	6	0	NO
5	5	23	310.0	1	2	1	1.50	7	0	NO
6	6	7	255.0	1	2	1	1.50	1	0	NO
7	7	8	255.0	1	2	1	1.50	2	0	NO
8	8	9	50.0	1	0	1	1.50	4	0	NO
9	9	29	100.0	1	0	1	1.50	4	301	NO
10	10	26	140.0	1	2	1	1.50	4	0	NO
11	11	12	220.0	1	2	1	1.50	8	0	NO
12	12	13	90.0	1	0	1	1.50	8	0	NO
13	13	14	80.0	1	2	1	1.50	10	0	NO
14	14	15	210.0	1	0	1	1.50	10	0	NO
15	15	16	40.0	1	2	1	1.50	11	0	NO
16	16	26	260.0	1	2	1	1.50	12	0	NO
17	17	18	290.0	1	2	1	1.50	13	0	NO
18	18	19	290.0	1	2	1	1.50	13	0	NO
19	19	20	260.0	1	2	1	1.50	14	0	NO
20	20	21	230.0	1	2	1	1.50	15	0	NO
21	21	27	260.0	1	2	1	1.50	16	0	YES
22	22	23	150.0	2	2	1	1.50	17	0	NO
23	23	24	180.0	2	2	1	1.50	19	0	NO
24	24	25	200.0	2	2	1	1.50	20	0	NO
25	25	26	240.0	2	2	1	1.50	20	0	YES
26	26	27	260.0	2	2	1	1.50	21	0	NO
27	27	29	250.0	2	2	1	1.50	22	301	YES
28	28	29	180.0	2	2	1	1.50	23	301	NO

TOTAL NUMBER OF STREET SEGMENTS 28

TOTAL LENGTH OF MAJOR SYSTEM 5500.00

DATA FOR MAJOR SYSTEM STORAGE

NO. OF DETENSION STRUCTURES 1

NO.	ID NO.
1	301

D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

SUB-CATCHMENT DATA

MAX. INFILTRATION	3.00	IN./HR.
MIN. INFILTRATION	0.52	IN./HR.
DECAY RATE	0.001150	1/SEC

	SUBAREA NO.	STREET SEGMENT	AREA (AC.)	IMP. (%)	MANNING (N) (IMP.)	MANNING (N) (PERV.)	SLOPE (FT/FT)	WIDTH (FT)	DEP. STOR. IMP. (IN)	DEP. STOR. PERV. (IN)	FLOW HISTORY
1	1	1	1.1	35.0	0.013	0.250	0.020	390.	0.062000	0.184000	NO
2	2	2	0.2	35.0	0.013	0.250	0.020	320.	0.062000	0.184000	NO
3	3	3	0.7	35.0	0.013	0.250	0.020	290.	0.062000	0.184000	NO
4	4	4	1.0	35.0	0.013	0.250	0.020	400.	0.062000	0.184000	NO
5	5	5	0.7	35.0	0.013	0.250	0.020	620.	0.062000	0.184000	NO
6	6	6	0.9	35.0	0.013	0.250	0.020	510.	0.062000	0.184000	NO
7	7	7	0.9	35.0	0.013	0.250	0.020	510.	0.062000	0.184000	NO
8	8	8	0.2	35.0	0.013	0.250	0.020	100.	0.062000	0.184000	NO
9	9	9	0.3	35.0	0.013	0.250	0.020	200.	0.062000	0.184000	NO
10	10	10	0.3	35.0	0.013	0.250	0.020	280.	0.062000	0.184000	NO
11	11	11	1.3	35.0	0.013	0.250	0.020	440.	0.062000	0.184000	NO
12	12	12	0.8	35.0	0.013	0.250	0.020	180.	0.062000	0.184000	NO
13	13	13	0.4	35.0	0.013	0.250	0.020	160.	0.062000	0.184000	NO
14	14	14	1.4	35.0	0.013	0.250	0.020	420.	0.062000	0.184000	NO
15	15	15	0.7	35.0	0.013	0.250	0.020	80.	0.062000	0.184000	NO
16	16	16	1.1	35.0	0.013	0.250	0.020	520.	0.062000	0.184000	NO
17	17	17	1.2	35.0	0.013	0.250	0.020	580.	0.062000	0.184000	NO
18	18	18	1.5	35.0	0.013	0.250	0.020	580.	0.062000	0.184000	NO
19	19	19	1.2	35.0	0.013	0.250	0.020	520.	0.062000	0.184000	NO
20	20	20	1.3	35.0	0.013	0.250	0.020	460.	0.062000	0.184000	NO
21	21	21	1.0	35.0	0.013	0.250	0.020	520.	0.062000	0.184000	NO
22	22	22	0.6	35.0	0.013	0.250	0.020	390.	0.062000	0.184000	NO
23	23	23	0.7	35.0	0.013	0.250	0.020	360.	0.062000	0.184000	NO
24	24	24	1.8	35.0	0.013	0.250	0.020	400.	0.062000	0.184000	NO
25	25	25	1.8	35.0	0.013	0.250	0.020	480.	0.062000	0.184000	NO
26	26	26	1.0	35.0	0.013	0.250	0.020	520.	0.062000	0.184000	NO
27	27	27	1.0	35.0	0.013	0.250	0.020	500.	0.062000	0.184000	NO
28	28	28	0.7	35.0	0.013	0.250	0.020	360.	0.062000	0.184000	YES

TOTAL DRAINAGE AREA 25.79 ACRES

NUMBER OF SUBAREAS 28

TOTAL NUMBER OF INLETS 46

DENSITY OF INLETS 1.78 C.B./ACRE

AVERAGE DISTANCE BETWEEN INLETS 239.13 FT

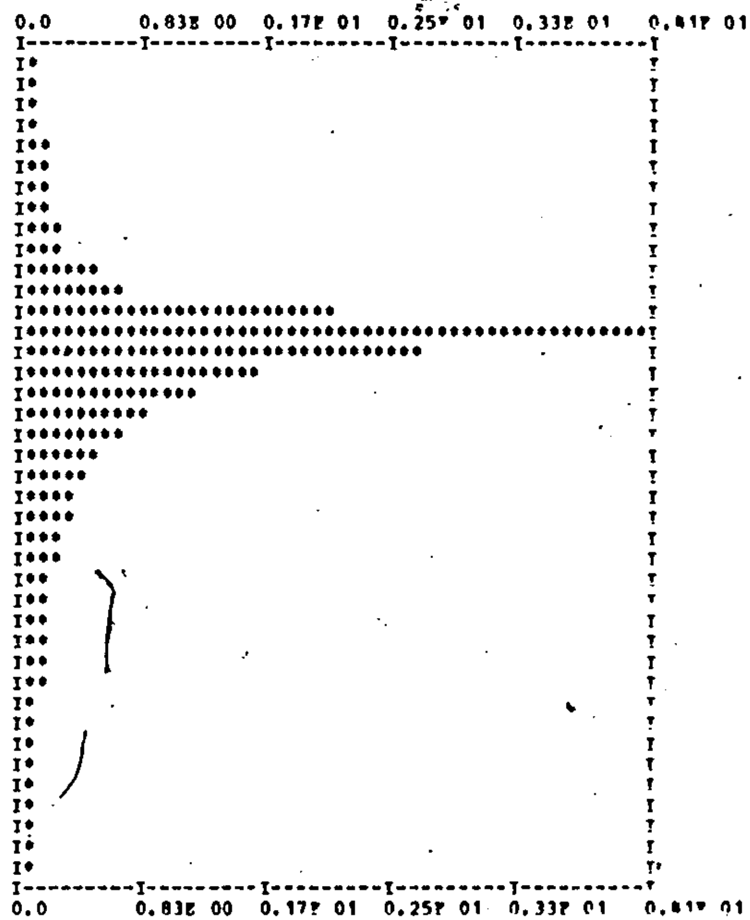
D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

SURFACE RUNOFF - DETAILED SIMULATION RESULTS

SUBAREA NO. 28

TIME (MIN)	FLOW (CFS)
0.0	0.0
5.00	0.0
10.00	0.00
15.00	0.06
20.00	0.10
25.00	0.10
30.00	0.12
35.00	0.14
40.00	0.18
45.00	0.22
50.00	0.49
55.00	0.66
60.00	2.05
65.00	4.13
70.00	2.56
75.00	1.54
80.00	1.12
85.00	0.75
90.00	0.61
95.00	0.45
100.00	0.38
105.00	0.30
110.00	0.25
115.00	0.21
120.00	0.18
125.00	0.15
130.00	0.13
135.00	0.11
140.00	0.10
145.00	0.09
150.00	0.09
155.00	0.08
160.00	0.08
165.00	0.07
170.00	0.07
175.00	0.07
180.00	0.07
185.00	0.06
190.00	0.06
195.00	0.0

FLOW(CFS)

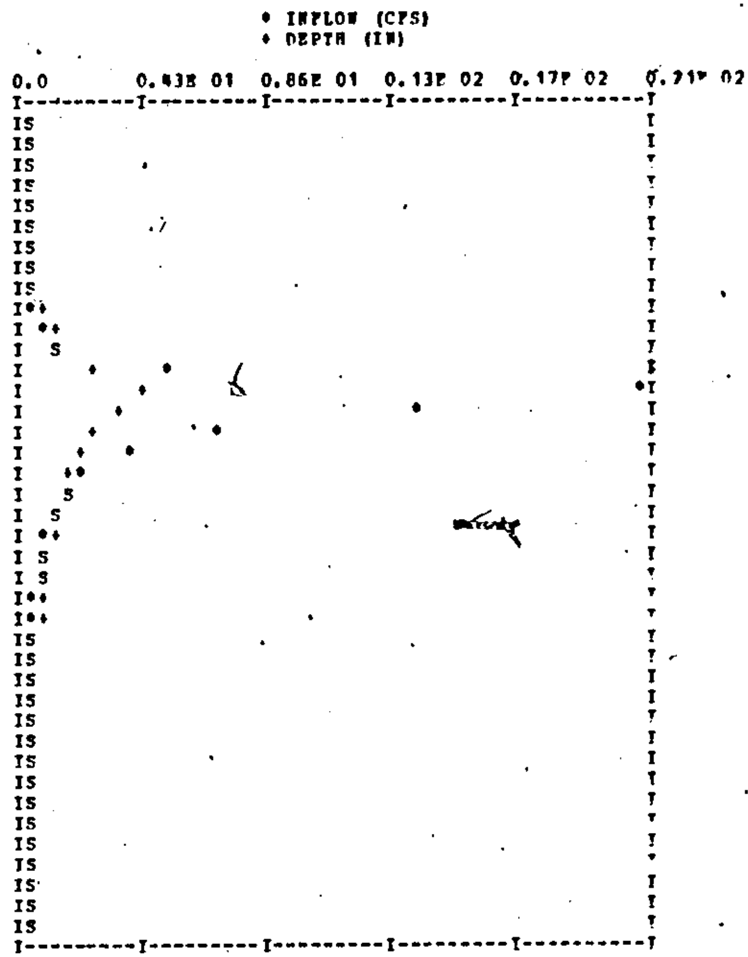


D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MAJOR SYSTEM DETAILED SIMULATION RESULTS

MAJOR SYSTEM SEGMENT NO. 21

TIME (MIN)	INFLOW (CFS)	DEPTH (IN)	OUTFLOW (CFS)
0.0	0.0	0.0	0.0
5.00	0.0	0.00	0.0
10.00	0.00	0.00	0.0
15.00	0.08	0.11	0.0
20.00	0.14	0.20	0.00
25.00	0.15	0.22	0.01
30.00	0.19	0.28	0.02
35.00	0.23	0.32	0.03
40.00	0.30	0.42	0.04
45.00	0.37	0.51	0.06
50.00	0.64	1.10	0.14
55.00	1.23	1.29	0.28
60.00	5.11	2.24	1.98
65.00	21.47	3.89	18.32
70.00	13.69	3.26	12.07
75.00	6.50	2.48	4.99
80.00	3.71	1.93	2.21
85.00	2.08	1.52	0.98
90.00	1.38	1.33	0.48
95.00	0.96	1.21	0.29
100.00	0.76	1.01	0.20
105.00	0.59	0.81	0.15
110.00	0.49	0.67	0.12
115.00	0.40	0.55	0.10
120.00	0.34	0.47	0.08
125.00	0.28	0.40	0.07
130.00	0.24	0.34	0.06
135.00	0.21	0.30	0.05
140.00	0.18	0.26	0.04
145.00	0.17	0.24	0.04
150.00	0.16	0.23	0.03
155.00	0.15	0.21	0.03
160.00	0.14	0.20	0.03
165.00	0.13	0.19	0.03
170.00	0.13	0.18	0.03
175.00	0.12	0.18	0.03
180.00	0.12	0.17	0.02
185.00	0.11	0.16	0.02
190.00	0.11	0.16	0.02
195.00	0.03	0.05	0.03

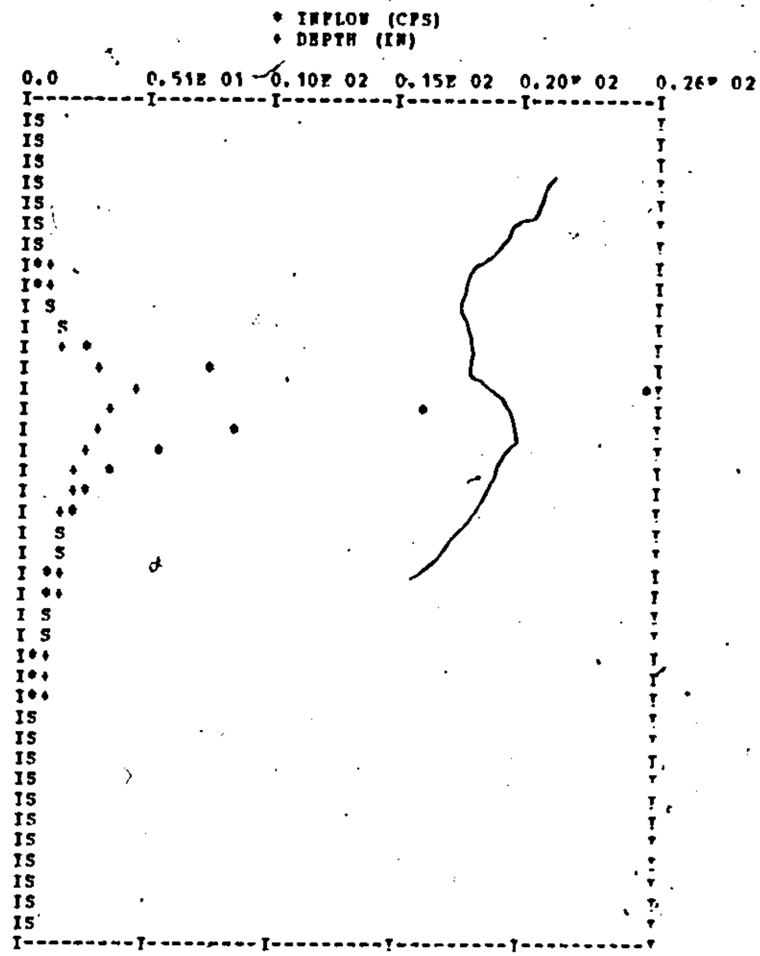


D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MAJOR SYSTEM DETAILED SIMULATION RESULTS

MAJOR SYSTEM SEGMENT NO. 25

TIME (MIN)	INFLOW (CFS)	DEPTH (IN)	OUTFLOW (CFS)
0.0	0.0	0.0	0.0
5.00	0.0	0.00	0.0
10.00	0.00	0.00	0.0
15.00	0.10	0.15	0.00
20.00	0.23	0.32	0.01
25.00	0.28	0.40	0.04
30.00	0.34	0.48	0.06
35.00	0.39	0.54	0.07
40.00	0.51	0.70	0.09
45.00	0.62	0.84	0.12
50.00	1.37	1.32	0.30
55.00	2.07	1.52	0.59
60.00	7.28	2.57	3.53
65.00	25.60	4.14	22.50
70.00	16.16	3.50	14.63
75.00	8.27	2.68	6.70
80.00	5.41	2.31	3.39
85.00	3.42	1.86	1.84
90.00	2.53	1.63	1.11
95.00	1.87	1.46	0.69
100.00	1.50	1.36	0.48
105.00	1.21	1.28	0.36
110.00	1.00	1.22	0.28
115.00	0.82	1.08	0.22
120.00	0.69	0.93	0.18
125.00	0.59	0.80	0.14
130.00	0.50	0.69	0.12
135.00	0.43	0.60	0.10
140.00	0.37	0.52	0.09
145.00	0.32	0.45	0.07
150.00	0.29	0.40	0.07
155.00	0.25	0.36	0.06
160.00	0.24	0.34	0.05
165.00	0.22	0.32	0.05
170.00	0.21	0.31	0.04
175.00	0.21	0.30	0.04
180.00	0.20	0.28	0.04
185.00	0.19	0.27	0.04
190.00	0.19	0.27	0.04
195.00	0.04	0.06	0.04

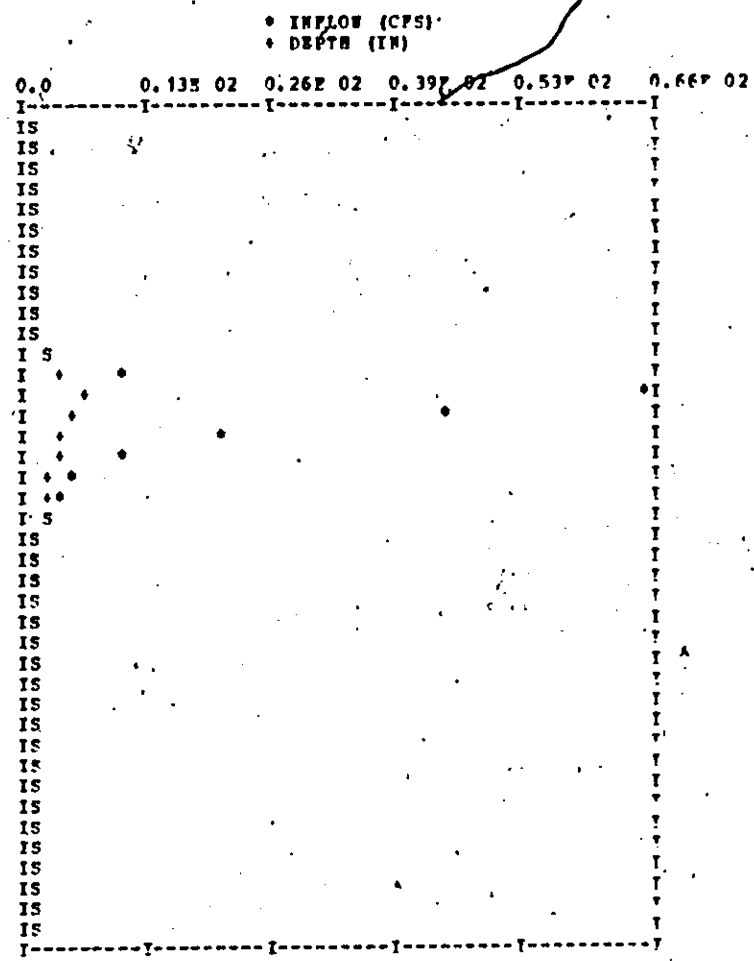


D D S R M - EXAMPLE-INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MAJOR SYSTEM DETAILED SIMULATION RESULTS

MAJOR SYSTEM SEGMENT NO. 27

TIME (MIN)	INFLOW (CFS)	DEPTH (IN)	OUTFLOW (CFS)
0.0	0.0	0.0	0.0
5.00	0.0	0.00	0.0
10.00	0.00	0.00	0.0
15.00	0.08	0.11	0.0
20.00	0.14	0.20	0.00
25.00	0.16	0.23	0.01
30.00	0.21	0.30	0.02
35.00	0.26	0.37	0.04
40.00	0.35	0.49	0.05
45.00	0.44	0.61	0.08
50.00	1.02	1.23	0.19
55.00	1.64	1.40	0.42
60.00	9.46	2.82	4.89
65.00	65.72	5.97	62.38
70.00	43.91	5.09	43.50
75.00	20.34	3.80	20.95
80.00	9.51	2.82	8.50
85.00	4.68	2.15	3.20
90.00	2.67	1.67	1.39
95.00	1.61	1.39	0.62
100.00	1.14	1.26	0.36
105.00	0.86	1.12	0.24
110.00	0.68	0.91	0.18
115.00	0.55	0.75	0.14
120.00	0.45	0.63	0.11
125.00	0.38	0.53	0.09
130.00	0.32	0.45	0.08
135.00	0.28	0.39	0.06
140.00	0.24	0.34	0.06
145.00	0.22	0.31	0.05
150.00	0.20	0.29	0.04
155.00	0.19	0.27	0.04
160.00	0.18	0.26	0.04
165.00	0.17	0.24	0.04
170.00	0.16	0.23	0.03
175.00	0.15	0.22	0.03
180.00	0.15	0.21	0.03
185.00	0.14	0.20	0.03
190.00	0.14	0.20	0.03
195.00	0.06	0.09	0.03

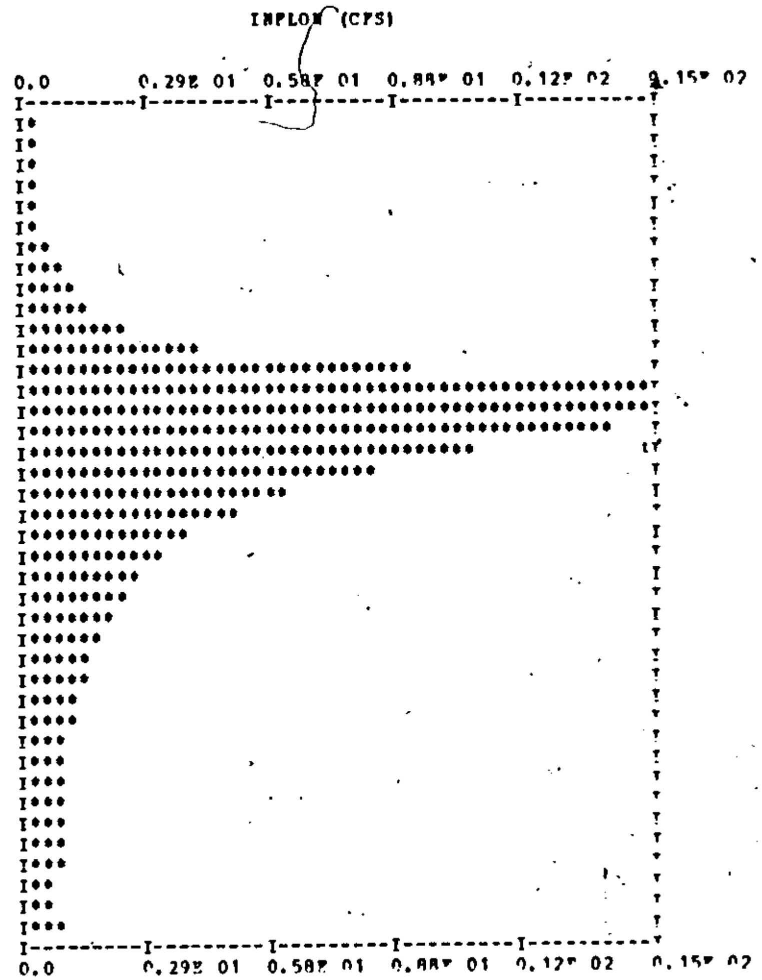


D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MINOR SYSTEM - DETAILED SIMULATION RESULTS

PIPE NO. 16

TIME (MIN)	INFLOW (CFS)	OUTFLOW (CFS)
0.0	0.0	0.0
5.00	0.0	0.0
10.00	0.0	0.0
15.00	0.0	0.0
20.00	0.01	0.01
25.00	0.11	0.05
30.00	0.41	0.34
35.00	0.70	0.65
40.00	0.94	0.91
45.00	1.24	1.20
50.00	2.26	2.14
55.00	3.81	3.66
60.00	8.78	8.36
65.00	14.60	14.57
70.00	14.49	14.49
75.00	13.45	13.53
80.00	10.50	10.75
85.00	7.97	8.19
90.00	6.12	6.29
95.00	4.74	4.87
100.00	3.78	3.87
105.00	3.10	3.17
110.00	2.59	2.65
115.00	2.20	2.24
120.00	1.87	1.91
125.00	1.62	1.65
130.00	1.40	1.43
135.00	1.23	1.25
140.00	1.08	1.10
145.00	0.96	0.98
150.00	0.86	0.88
155.00	0.79	0.80
160.00	0.73	0.74
165.00	0.69	0.70
170.00	0.66	0.66
175.00	0.63	0.63
180.00	0.60	0.61
185.00	0.58	0.59
190.00	0.56	0.57
195.00	0.62	0.61

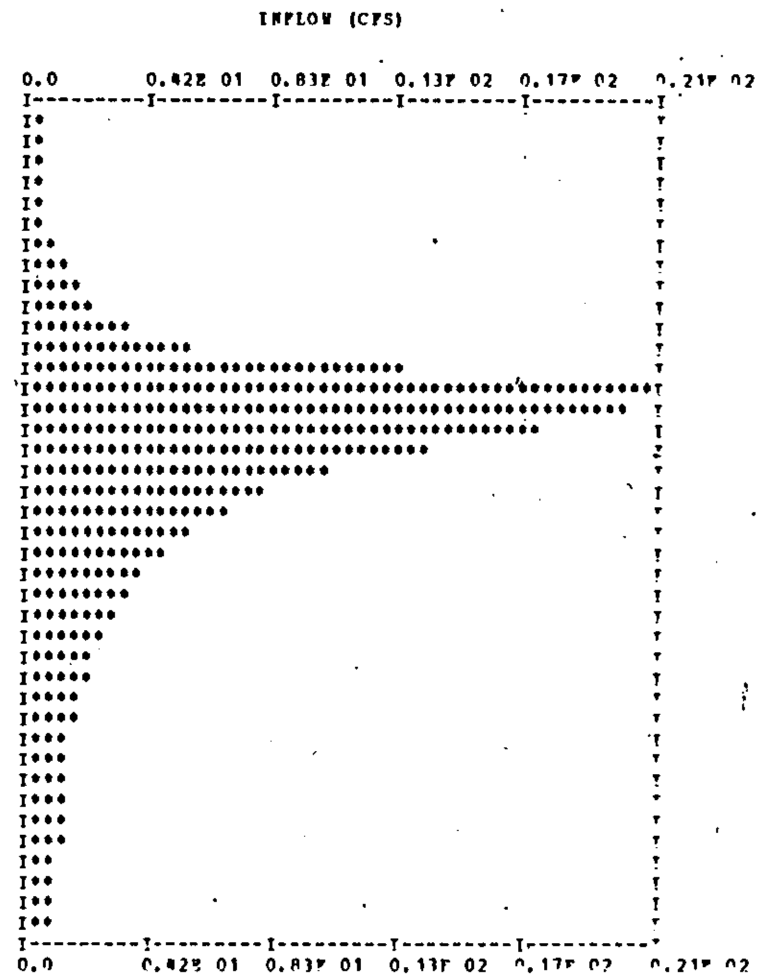


D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MINOR SYSTEM - DETAILED SIMULATION RESULTS

PIPE NO. 20

TIME (MIN)	INFLOW (CFS)	OUTFLOW (CFS)
0.0	0.0	0.0
5.00	0.0	0.0
10.00	0.0	0.0
15.00	0.01	0.01
20.00	0.09	0.01
25.00	0.32	0.20
30.00	0.61	0.52
35.00	0.96	0.87
40.00	1.30	1.23
45.00	1.71	1.63
50.00	3.20	2.95
55.00	5.32	5.01
60.00	12.38	11.50
65.00	20.85	20.62
70.00	19.92	20.02
75.00	17.05	17.39
80.00	13.19	13.67
85.00	10.01	10.42
90.00	7.81	8.11
95.00	6.26	6.47
100.00	5.11	5.28
105.00	4.25	4.39
110.00	3.58	3.69
115.00	3.04	3.13
120.00	2.61	2.69
125.00	2.26	2.32
130.00	1.96	2.02
135.00	1.72	1.77
140.00	1.52	1.56
145.00	1.35	1.39
150.00	1.21	1.24
155.00	1.10	1.13
160.00	1.01	1.03
165.00	0.94	0.96
170.00	0.89	0.90
175.00	0.85	0.86
180.00	0.82	0.83
185.00	0.79	0.80
190.00	0.76	0.77
195.00	0.80	0.79



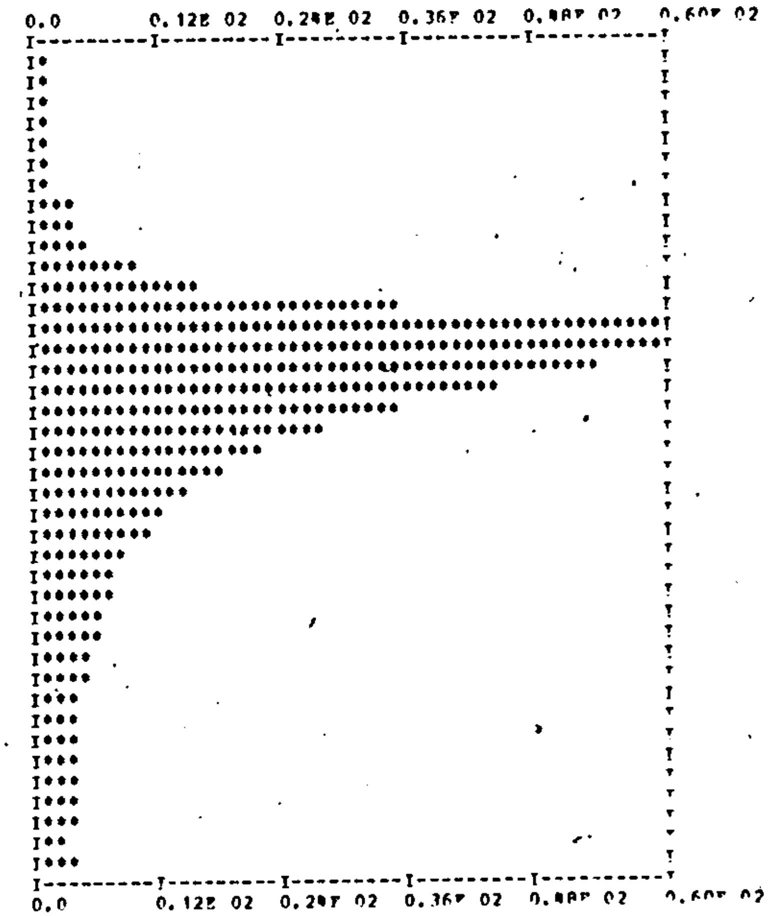
D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MINOR SYSTEM - DETAILED SIMULATION RESULTS

PIPE NO. 23

TIME (MIN)	INFLOW (CFS)	OUTFLOW (CFS)
0.0	0.0	0.0
5.00	0.0	0.0
10.00	0.0	0.0
15.00	0.00	0.0
20.00	0.02	0.01
25.00	0.12	0.03
30.00	1.18	0.99
35.00	2.43	2.28
40.00	3.57	3.45
45.00	4.80	4.68
50.00	8.49	8.20
55.00	14.69	14.29
60.00	34.01	32.92
65.00	60.40	60.21
70.00	59.25	59.30
75.00	53.58	53.87
80.00	44.29	44.79
85.00	34.67	35.22
90.00	26.92	27.38
95.00	21.17	21.53
100.00	16.96	17.24
105.00	13.90	14.11
110.00	11.60	11.77
115.00	9.82	9.95
120.00	8.38	8.49
125.00	7.23	7.32
130.00	6.27	6.35
135.00	5.49	5.56
140.00	4.83	4.89
145.00	4.29	4.34
150.00	3.85	3.89
155.00	3.50	3.53
160.00	3.21	3.24
165.00	2.99	3.01
170.00	2.81	2.83
175.00	2.67	2.68
180.00	2.55	2.56
185.00	2.45	2.46
190.00	2.36	2.37
195.00	2.42	2.41

INFLOW (CFS)



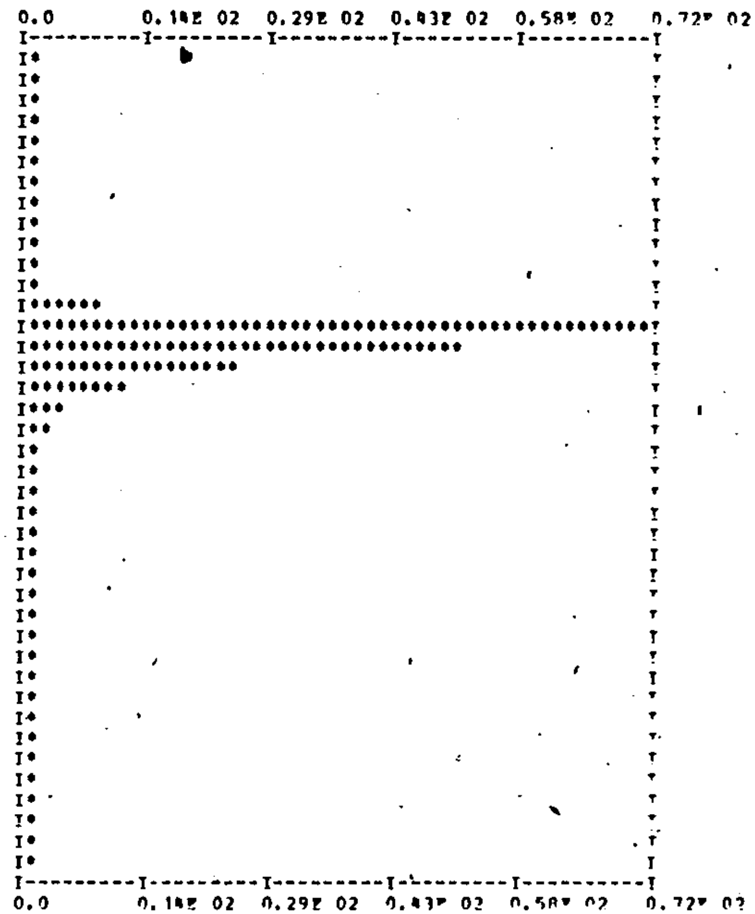
D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

RESULTS OF SIMULATION FOR MAJOR SYSTEM STORAGE

STORAGE UNIT NUMBER 301

TIME (MIN)	FLOW (CFS)
0.0	0.0
5.00	0.0
10.00	0.0
15.00	0.00
20.00	0.01
25.00	0.04
30.00	0.07
35.00	0.11
40.00	0.16
45.00	0.23
50.00	0.53
55.00	1.04
60.00	7.42
65.00	71.88
70.00	49.59
75.00	24.08
80.00	10.25
85.00	4.27
90.00	2.16
95.00	1.20
100.00	0.82
105.00	0.62
110.00	0.49
115.00	0.40
120.00	0.33
125.00	0.28
130.00	0.24
135.00	0.21
140.00	0.19
145.00	0.17
150.00	0.15
155.00	0.14
160.00	0.13
165.00	0.13
170.00	0.12
175.00	0.11
180.00	0.11
185.00	0.10
190.00	0.10
195.00	0.11

FLOW (CFS)



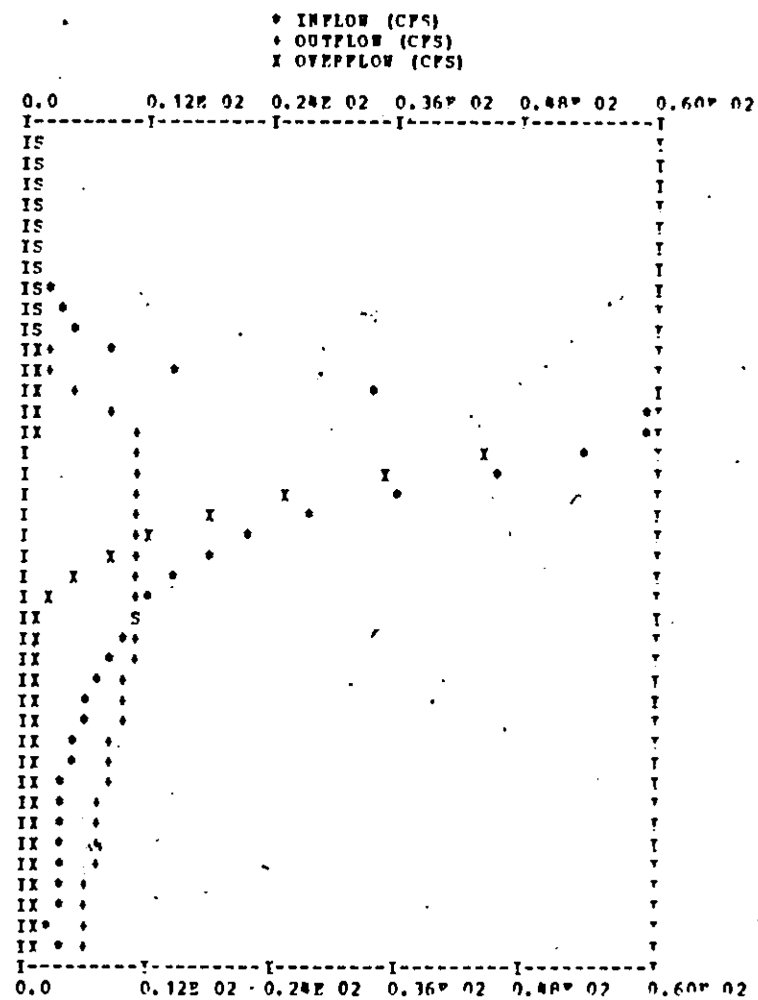
REQUIRED DETENTION VOLUME 1.23 AC.FT

D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

RESULTS OF SIMULATION FOR MINOR SYSTEM STORAGE

STORAGE UNIT NO. 201

TIME (MIN)	INFLOW (CFS)	OUTFLOW (CFS)	OVERFLOW (CFS)
0.0	0.0	0.0	0.0
5.00	0.0	0.0	0.0
10.00	0.0	0.0	0.0
15.00	0.0	0.0	0.0
20.00	0.01	0.00	0.0
25.00	0.03	0.00	0.0
30.00	0.99	0.05	0.0
35.00	2.28	0.20	0.0
40.00	3.45	0.46	0.0
45.00	4.68	0.80	0.0
50.00	8.20	1.34	0.0
55.00	14.29	2.28	0.0
60.00	32.92	4.31	0.0
65.00	60.21	8.34	0.0
70.00	59.30	10.00	0.0
75.00	53.87	10.00	43.87
80.00	44.79	10.00	34.79
85.00	35.22	10.00	25.22
90.00	27.38	10.00	17.38
95.00	21.53	10.00	11.53
100.00	17.24	10.00	7.24
105.00	14.11	10.00	4.11
110.00	11.77	10.00	1.77
115.00	9.95	10.00	0.0
120.00	8.49	9.93	0.0
125.00	7.32	9.73	0.0
130.00	6.35	9.46	0.0
135.00	5.56	9.12	0.0
140.00	4.89	8.75	0.0
145.00	4.34	8.36	0.0
150.00	3.89	7.95	0.0
155.00	3.53	7.55	0.0
160.00	3.24	7.15	0.0
165.00	3.01	6.77	0.0
170.00	2.83	6.40	0.0
175.00	2.68	6.06	0.0
180.00	2.56	5.73	0.0
185.00	2.46	5.42	0.0
190.00	2.37	5.14	0.0
195.00	2.41	4.87	0.0



OVERFLOW VOLUME 53473.14 CU.FT.

D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MAJOR SYSTEM

SUMMARY OF SIMULATION RESULTS

	SEGMENT NO	MAX. FLOW (CFS)	MAX. DEPTH (IN)	MAX. CAPTURE (CFS)	INLET RESTRICTION
1	1	5.68	2.36	0.0	-
2	2	6.87	2.52	2.82	NO
3	3	3.62	1.91	1.95	NO
4	4	7.11	2.55	2.88	NO
5	5	8.63	2.72	3.00	YES
6	6	5.05	2.23	2.35	NO
7	7	7.99	2.65	3.00	YES
8	8	5.96	2.41	0.0	-
9	9	7.51	2.60	0.0	-
10	10	1.70	1.42	1.17	NO
11	11	6.62	2.49	2.76	NO
12	12	7.45	2.59	0.0	-
13	13	9.70	2.84	3.00	YES
14	14	13.65	3.26	0.0	-
15	15	16.42	3.52	3.00	YES
16	16	19.50	3.75	3.00	YES
17	17	6.84	2.52	2.82	NO
18	18	12.00	3.09	3.00	YES
19	19	15.40	3.43	3.00	YES
20	20	18.94	3.72	3.00	YES
21	21	21.47	3.89	3.00	YES
22	22	3.53	1.89	1.92	NO
23	23	15.10	3.40	3.00	YES
24	24	20.16	3.79	3.00	YES
25	25	25.60	4.14	3.00	YES
26	26	45.13	5.15	3.00	YES
27	27	65.72	5.97	3.00	YES
28	28	4.13	2.03	2.11	NO

D D S R M - EXAMPLE INPUT - OUTPUT - TESTVILLE 7.1
100 - YEAR STORM

MINOR SYSTEM

SUMMARY OF SIMULATION RESULTS

	SEWER NO	SLOPE (FT/FT)	MANNING N	MAX. FLOW (CFS)	ORIGINAL SIZE (IN)	AVAILABLE CAPACITY (CFS)	Q/QFULL ORIGINAL SIZE	REQUIRED SIZE (IN)	CAPACITY (CFS)	Q/QFULL REQUIRED SIZE
1	1	0.00500	0.01300	2.35	12.00	2.52	0.93	12.00	2.52	0.93
2	2	0.00500	0.01300	5.29	15.00	4.57	1.16	18.00	7.43	0.71
3	3	0.00500	0.01300	5.19	15.00	4.57	1.14	18.00	7.43	0.70
4	4	0.00500	0.01300	6.33	15.00	4.57	1.39	18.00	7.43	0.85
5	5	0.00500	0.01300	1.95	12.00	2.52	0.78	12.00	2.52	0.78
6	6	0.00500	0.01300	4.80	15.00	4.57	1.05	18.00	7.43	0.65
7	7	0.00500	0.01300	7.72	15.00	4.57	1.69	21.00	11.21	0.69
8	8	0.00500	0.01300	2.76	15.00	4.57	0.60	15.00	4.57	0.60
9	9	0.00500	0.01300	2.73	12.00	2.52	1.08	15.00	4.57	0.60
10	10	0.00500	0.01300	5.69	18.00	7.43	0.77	18.00	7.43	0.77
11	11	0.00500	0.01300	8.62	21.00	11.21	0.77	21.00	11.21	0.77
12	12	0.00500	0.01300	11.60	21.00	11.21	1.04	24.00	16.00	0.72
13	13	0.00500	0.01300	5.82	18.00	7.43	0.78	18.00	7.43	0.78
14	14	0.00500	0.01300	8.73	21.00	11.21	0.78	21.00	11.21	0.78
15	15	0.00500	0.01300	11.66	21.00	11.21	1.04	24.00	16.00	0.73
16	16	0.00500	0.01300	14.60	24.00	16.00	0.91	24.00	16.00	0.91
17	17	0.00500	0.01300	1.92	12.00	2.52	0.76	12.00	2.52	0.76
18	18	0.00500	0.01300	12.28	21.00	11.21	1.10	24.00	16.00	0.77
19	19	0.00500	0.01300	15.11	21.00	11.21	1.35	24.00	16.00	0.94
20	20	0.00500	0.01300	20.85	24.00	16.00	1.30	27.00	21.91	0.95
21	21	0.00500	0.01300	41.33	33.00	37.41	1.10	36.00	47.18	0.88
22	22	0.00500	0.01300	58.56	36.00	47.18	1.24	42.00	71.17	0.82
23	23	0.00500	0.01300	60.40	36.00	47.18	1.28	42.00	71.17	0.85

Appendix F

DUAL DRAINAGE SIMULTANEOUS ROUTING MODEL - LISTING OF THE COMPUTER PROGRAM

```
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTM,NI
     1             ,NDELTC,IUS,IRTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1             OP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2             NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),ODQP(200),
     4             IPRTP(200),QB(200),TMAX(200),IOPTPP,QEX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),PP1(5,20),NSP1(5),
     1             QQ1(5,20),QOTPP(5,100),OVFL(5,100),QPINF(5)
      COMMON/  MJR/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1             NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3             SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4             IPRTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1             QINP(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),FORCB(200),
     1             IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2             0),CBMAX(200)
      COMMON/  SUP/ NUSMAX,NINW,NW(200),AW(200),PIMP(200),CNISP(200),
     1             CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2             WCON(200,5),RINT(100),WDEPTH(200,5),WFLOW(200),
     3             FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4             NWS(200,5),SUMQW(200),QB(200,100),IPRTW(200)
      COMMON / TITL/ ITITL(40)
C
C
C
      NREAD=5
      NPRINT=6
C
C
C     INITIALIZATION
C
C
      CALL INIT
C
C
C    INPUT DATA
C
C
      CALL DATA
C
C
C    SET OF ORDERING ARRAYS
C
C
      CALL ORDER
C
C
C     SURFACE RUNOFF COMPUTATION
C
C
      IF(IUS.EQ.0) CALL SUPPLY
C
C
C     FLOW ROUTING -  MAJOR SYSTEM
C
C
      CALL MAJOR
C
C
C
C
C     MINOR SYSTEM
C
C
      IF(IEVAL.NE.3) CALL MINOR
      IF(IEVAL.EQ.3) CALL SUBCH
C
C
C     PRINTING AND PLOTTING OF DETAILED SIMULATION RESULTS
C
C
      CALL OPRINT
C
C
C     MAJOR SYSTEM STORAGE
C
C
      IF(JSS.EQ.1) CALL MAJS1
C
C
C     MINOR SYSTEM STORAGE
C
C
      IF(JPP.EQ.1.AND.IEVAL.NE.3) CALL MINS1
C
C
C     PRINT SUMMARY OF SIMULATION RESULTS
C
C
      CALL SUMPRT
      STOP
      END
C
C
C     SUBROUTINE DATA
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTM,NI
     1             ,NDELTC,IUS,IRTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1             OP(200),NOP,NOPP,IDMN(200),NPOC(5),NPO(200),
     2             NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),ODQP(200),
     4             IPRTP(200),QB(200),TMAX(200),IOPTPP,QEX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),PP1(5,20),NSP1(5),
     1             QQ1(5,20),QOTPP(5,100),OVFL(5,100),QPINF(5)
      COMMON/  MJR/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1             NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
```

```
     2              QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4              IPRTS(200),SDPT(200,100),SPMAX(200),SDPMX(200)
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1              QINF(5,100),NSTMAX,SS(10),CELAN(50,10)
      COMMON/  INL/ NINL,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1              IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2              0),CBMAX(200)
      COMMON/  SUP/ NUSMAX,NINN,NN(200),AW(200),PIMP(200),CNIMP(200),
     1              CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              NCON(200,5),RINT(100),NDEPTH(200,5),NFLOW(200),
     3              FMAX(200),FMIN(200),DEC(200),PET(200,5),NONS,
     4              NNS(200,5),SUMQN(200),OF(200,100),IPRTN(200)
      COMMON / TITL/ ITITL(40)
      REAL INFMAX,INFMIN
      INTEGER ITN(2)
      DATA ITN /'YES','NO'/
      INTEGER ILINE(50)
      REAL GRA(6)
      DATA IIG /'I'/
C
C
C
C
C
C     CONTROL DATA  FOR RUN
C
C
C
      WRITE(NPRINT,1807)
      READ(NREAD,1101)  (ITITL(I),I=1,40)
      WRITE(NPRINT,1808)  (ITITL(I),I=1,40)
      READ(NREAD,1000)  DELTC,NDELTC,IEVAL,JSS,JPP,INS,QLIMIT,INTS
      TTOTC=DELTC*(NDELTC-1)
 1000 FORMAT(F5.0,5I5,F5.0,I5)
 1101 FORMAT(20A4)
      WRITE(NPRINT,1900)  DELTC,NDELTC,QLIMIT,TTOTC
      IF(IEVAL.EQ.0)  WRITE(NPRINT,1901)
      IF(IEVAL.EQ.1)  WRITE(NPRINT,1902)
      IF(IEVAL.EQ.3)  WRITE(NPRINT,1903)
      IF(JSS.GT.0)  WRITE(NPRINT,1904)
      IF(JPP.GT.0)  WRITE(NPRINT,1905)
      DELTCS=DELTC*60.0
      DELTC2=DELTCS/2.0
 1900 FORMAT(//,T5,'TIME INCREMENT FOR CALCULATION  ', T40,F6.2,
     1       '  MINUTES',
     1       //,T5,'NUMBER OF COMPUTATIONAL STEPS     ',T40,I6,//,T5,
     2       'LIMITING CAPACITY OF INLETS    ',T40,F6.2,'  CFS',//,T5,
     3       'TOTAL SIMULATION TIME  ',T40,F6.2,'  MINUTES')
 1901 FORMAT(//,T5,'PIPES WILL BE SIZED FOR FREE SURFACE FLOW CONDITIONS
     1')
 1902 FORMAT(//,T5,'PIPE SIZES WILL BE REVISED FOR FREE SURFACE FLOW CON
     1DITIONS')
 1903 FORMAT(//,T5,'CALCULATION FOR MINOR SYSTEM IS NOT INCLUDED IN THIS
     1RUN')
 1904 FORMAT(//,T5,'STORAGE COMPUTATION FOR MAJOR SYSTEM FLOW IS REQUEST
     1ED')
 1905 FORMAT(//,T5,'STORAGE COMPUTATION FOR MINOR SYSTEM FLOW IS REQUEST
     1ED')
 1906 FORMAT(1H1,//,5X,20A4,/,5X,20A4,/)
 1807 FORMAT(1H1,/////,T25,81(1H*),/,T25,1H*,T106,1H*,/,T25,1H*,
     1       T26,'D U A L   D R A I N A G E   S I M U L T A N E O U S
     2R O U T I N G   M O D E L',T106,1H*,/,T25,1H*,T106,1H*,/,T25,1H*,
     3       T61,'D Q S P M',T106,1H*,/,T25,1H*,T106,1H*,/,T25,81(1H*),
     4       ////)
 1808 FORMAT(//,5X,20A4,/,5X,20A4,/)
C
C
C     RAINFALL DATA
C
C
C
      IF(INS.GT.0) GO TO 91
      READ(NREAD,1100)  DELTR,NDELTR
      TTOTR=DELTR*NDELTR
 1100 FORMAT(F5.0,I5)
      WRITE(NPRINT,1906)  (ITITL(I),I=1,40)
      WRITE(NPRINT,2100)
 2100 FORMAT(//,11X,'RAINFALL DATA',//)
      READ(NREAD,1110)  (RINT(I),I = 1 , NDELTR)
 1110 FORMAT(16F5.0)
      T1=-DELTR
      WRITE(NPRINT,3000)
 3000 FORMAT(T10,'TIME',T20,'RAINFALL',T64,'RAINFALL INTENSITY (IN/HR)',
     1       /,T10,'(MIN)',T20,'(IN/HR)',/)
      INSUB=1
      IVPT=1
      CALL MAXMN1(AMAX,AMIN,RINT,I,NDELTR)
      SCALE=(AMAX-AMIN)/50.0
      CALL VSCAL(SCALE,GRA,NPRINT,AMAX,AMIN)
      DO 50 I = 1 , NDELTR
      T1=T1+DELTR
      IVAL=RINT(I)
      CALL LINE(ILINE,IVAL,SCALE,IVPT,INSUB,I)
      WRITE(NPRINT,3001)  T1,RINT(I),IIG,(ILINE(L),L=1,50),IIG
   50 CONTINUE
      WRITE(NPRINT,3033)
      WRITE(NPRINT,3032)  (GRA(KK),KK=1,6)
 3032 FORMAT(50X,5(E8.2,2X),1X,E8.2)
 3033 FORMAT(50X,5(1HI,9H---------),2H-I)
 3001 FORMAT(T10,F6.2,T20,F6.2,T51,A1,50A1,A1)
      WRITE(NPRINT,2102)  TTOTR
 2102 FORMAT(//,5X,'RAINFALL DURATION  ',F6.2,'  MINUTES')
      DO 90 I = 1 , NDELTR
      RINT(I)=RINT(I)/43200.
   90 CONTINUE
   91 CONTINUE
C
C
C     PIPE DATA
C
C
      IF(IEVAL.EQ.3) GO TO 598
      WRITE(NPRINT,1906)  (ITITL(I),I=1,40)
      WRITE(NPRINT,1810)
 1810 FORMAT(/,T51,'MINOR SYSTEM DATA',/)
      READ(NREAD,1039)  IOPTEP
 1039 FORMAT(I5)
      IF(IEVAL.NE.0)  WRITE(NPRINT,1908)
      IF(IEVAL.EQ.0)  WRITE(NPRINT,1909)
 1908 FORMAT(T5,'NO.',T15,'SEWER',T25,'D/S',T35,'LENGTH',T45,'SLOPE',
     1       T55,'MANNING',T65,'DIAMETER',T75,'DRY WEATHER',T90,
```

```
     2         'STORAGE',T105,'HYDROGRAPH',/,
     3         T15,'NUMBER',T24,'SEWER',T37,'(FT)',T44,'(FT/FT)',T58,'(F)'
     4         ,T67,'(IN)',T75,'FLOW (CFS)',T90,'ID NUMBER',T105,'PRINTED'
     5         //)
 1909 FORMAT(T5,'NO',T15,'SEWER',T27,'D/S',T36,'LENGTH',T48,'SLOPE',
     1         T57,'MANNING',T69,'DRY WEATHER',T84,'STORAGE',T99,
     2         'FLOW',/,
     3         T15,'NUMBER',T26,'SEWER',T38,'(FT)',T47,'(FT/FT)',T58,'(N)
     4',
     5         T69,'FLOW (CFS)',T84,'ID NUMBER',T98,'HISTORY',//)
      DO 600 I = 1 , NPMAX
      READ (NREAD,1040) NP(I),NPD(I),XP(I),SP(I),CP(I),DP(I),QB(I),
     1                  NPPS(I),IPRTP(I)
      IITN=1
      IF(IPRTP(I).EQ.0) IITN=2
      IF (NP(I).EQ.0) GO TO 610
      IF(IEVAL.EQ.1) WRITE (NPRINT,2060) I,NP(I),NPD(I),XP(I),SP(I),
     1                  CP(I),DP(I),QB(I),NPPS(I),ITN(IITN)
      IF(IEVAL.EQ.2) WRITE (NPRINT,2060) I,NP(I),NPD(I),XP(I),SP(I),
     1                  CP(I),DP(I),QB(I),NPPS(I),ITN(IITN)
      IF(IEVAL.EQ.0) WRITE (NPRINT,2061) I,NP(I),NPD(I),XP(I),SP(I),
     1                  CP(I),QB(I),NPPS(I),ITN(IITN)
  600 CONTINUE
  610 CONTINUE
      NOP=I-1
      WRITE(NPRINT,2070) NOP
 1040 FORMAT (2I5,5F5.0,2I5)
 2060 FORMAT(T5,I2,T14,I5,T22,I5,T33,F8.3,T43,F8.3,T54,F8.3,T65,F8.3,
     1         T75,F8.3,T91,I5,T106,A3)
 2061 FORMAT(T5,I2,T14,I5,T25,I5,T35,F8.3,T45,F8.3,T55,F8.3,
     1         T69,F8.3,T85,I5,T100,A3)
 2070 FORMAT (//,5X,'TOTAL NUMBER OF PIPES    =',I4)
      IF(JPP.LE.0) GO TO 603
C
C
C     DATA FOR MINOR SYSTEM STORAGE
C
C
      WRITE(NPRINT,1906) (ITITL(I),I=1,40)
      NNPT=0
      DO 601 I = 1 , NPMAX
      READ(NREAD,1042) IOPTSP(I)
      IF(IOPTSP(I).EQ.1) READ(NREAD,1043) IDSTP(I),QBLSP(I)
      IF(IOPTSP(I).EQ.2) READ(NREAD,1044) IDSTP(I),SPMX(I),QBLSP(I)
      IF(IOPTSP(I).EQ.3) READ(NREAD,1045) IDSTP(I),SPMX(I),NSP1(I)
      NSP11=NSP1(I)
      IF(IOPTSP(I).EQ.3) READ(NREAD,1046) (SP1(I,J),PP1(I,J),J=1,NSP11)
 1042 FORMAT(I5)
 1043 FORMAT(I5,F5.0)
 1044 FORMAT(I5,F10.0,F5.0)
 1045 FORMAT(I5,F10.0,I5)
 1046 FORMAT(8F10.2)
      IF(IDSTP(I).EQ.0) GO TO 602
      NNPT=NNPT+1
  601 CONTINUE
  602 CONTINUE
      WRITE(NPRINT,2071)
      WRITE(NPRINT,2072) NNPT
 2071 FORMAT(//,T30,' DATA FOR MINOR SYSTEM STORAGE ',/)
 2072 FORMAT(//,T10,'NO. OF STORAGE UNITS   ',I5)
      DO 604 I = 1 , NNPT
      IF(IOPTSP(I).EQ.1) WRITE(NPRINT,2073) I,IDSTP(I),QBLSP(I)
      IF(IOPTSP(I).EQ.2) WRITE(NPRINT,2074) I,IDSTP(I),SPMX(I),QBLSP(I)
      IF(IOPTSP(I).EQ.3) GO TO 605
      GO TO 606
  605 CONTINUE
      WRITE(NPRINT,2075) I,IDSTP(I),SPMX(I)
      WRITE(NPRINT,1911)
      NSP11=NSP1(I)
      DO 607 J = 1 , NSP11
      WRITE(NPRINT,2076) SP1(I,J),PP1(I,J)
  607 CONTINUE
  606 CONTINUE
 2073 FORMAT(//,2X,I2,T9,' DETENTION STORAGE NO.',T40,I5,//,T9,
     1         'MAXIMUM ALLOWABLE RELEASE',T35,F10.2,1X,'CFS.')
 2074 FORMAT(//,2X,I2,T9,' DETENTION STORAGE NO.',T40,I5,//,T9,
     1         'AVAILABLE DETENTION VOLUME',T35,F10.0,1X,'CU.FT.',//,T9,
     2         'MAXIMUM ALLOWABLE RELEASE',T40,F5.2,1X,'CFS.')
 2075 FORMAT(//,6X,I2,' DETENTION STORAGE ID. NO.',T40,I5,//,T9,
     1         'DETENTION VOLUME',T35,F10.2,1X,'CU. FT.',//)
 2076 FORMAT(T15,F12.2,T30,F12.2)
 1911 FORMAT(//,15X,'STORAGE - OUTFLOW  RELATIONSHIP',/,19X,'STORAGE',
     1         9X,'OUTFLOW',/,19X,'(CU.FT)',10X,'(CFS)',//)
  604 CONTINUE
  603 CONTINUE
C
C
C     PIPE CONNECTIVITY
C
C
      ICOT=0
      DO 900 I = 1 , NOP
      N=NCP+IOUT
      DO 910 J = 1 , N
      IF (NPD(I).EQ.NP(J)) GO TO 920
  910 CONTINUE
      IOUT=IOUT+1
      J=NCP+IOUT
      NP(J)=NPD(I)
      IDNN(J)=1
      QB(J)=QB(I)
C
C
C     DEFINE OUTLETS OF MINOR SYSTEM AND INLETS TO STORAGES
C
C
      IF(NPPS(I).NE.0) IPTOS(J)=NPPS(I)
      NPCO(IOUT)=J
  920 CONTINUE
      DO 930 K = 1 , NINP
      IF (NPP(J,K).GT.0) GO TO 930
      NPP(J,K)=I
      GO TO 900
  930 CONTINUE
  900 CONTINUE
      NOPT=NOP+ICOT
C
C
C     MINOR SYSTEM/UNDERGROUND STORAGE CONNECTIVITY
C
```

```
C
      DO 906 K = 1 , NPTMAX
      IINSTP=0
      DO 907 M = 1 , NOPP
      IF(IPTOS(M).EQ.IDSTP(K)) GO TO 908
      GO TO 907
  908 CONTINUE
      IINSTP=IINSTP+1
      NNSTP(K,IINSTP)=M
  907 CONTINUE
  906 CONTINUE
      GO TO 599
  598 CONTINUE
      WRITE(NPRINT,2077)
 2077 FORMAT(//,5X,'NO DATA FOR MINOR SYSTEM IS SUPPLIED',//,5X,
     1       'FLOWS CAPTURED BY INLETS WILL BE STORED ON A DISK FILE')
  599 CONTINUE
C
C
C     MAJOR SYSTEM AND INLET DATA
C
C
      READ(NREAD,1010) NSSEG
 1010 FORMAT(I2)
      WRITE(NPRINT,1906) (ITITL(I),I=1,40)
      WRITE(NPRINT,1912)
 1912 FORMAT(//,40X,'MAJOR SYSTEM RATING CURVE',/)
      WRITE(NPRINT,2011) NSSEG
 2011 FORMAT(//,5X,'NO. OF GEOMETRIES ',I5,//)
      DO 5 J = 1 , NSSEG
      READ(NREAD,1020) IS,W2,SX,YCURB,CNS,SO,SNOW,CYPOW,TMAX
 1020 FORMAT(I5,8F5.0)
      IF(J.GT.1) WRITE(NPRINT,2000)
 2000 FORMAT(1H1)
      WRITE(NPRINT,1913)
 1913 FORMAT(T22,'PAVEMENT',T35,'PAVEMENT',/,T9,'TYPE',T22,'WIDTH',
     1       T35,'CROSS SLOPE',T50,'HEIGHT',T65,'MANNING',T80,
     2       'LONG.',T95,'SHOULDER',T110,'SHOULDER',T121,
     3       'MAXIMUM',/,T22,'(FT)',T35,'(FT/FT)',T50,'OF CURB',
     4       T67,'(N)',T80,'SLOPE',T95,'CROSS SLOPE',T110,'ROUGHNESS',
     5       T121,'FLOW DEPTH',/,T80,'(FT/FT)',T95,'(FT/FT)',T115,'N',
     6       T124,'(FT)',T50,'(FT)')
      WRITE(NPRINT,2010) IS,W2,SX,YCURB,CNS,SO,SPOW,CYPOW,TMAX
 2010 FORMAT(//,T6,I5,T22,F5.2,T35,F5.3,T50,F5.2,T65,F5.3,T80,F5.3,T97,
     1       F5.3,T112,F5.3,T123,F5.2,//)
      CALL MAJOR1(W2,SX,YCURB,CNS,SO,SNOW,CYPOW,TMAX)
    5 CONTINUE
C
C
C     READ INLET CAPTURE DATA
C
C
      READ(NREAD,1025) NINCAP
      WRITE(NPRINT,1906) (ITITL(I),I=1,40)
      WRITE(NPRINT,1914)
 1914 FORMAT(//,38X,'INLET DATA')
      WRITE(NPRINT,2012) NINCAP
 2012 FORMAT(//,5X,'NO. OF INLET TYPES ',I5,//)
 1915 FORMAT(T35,'APPROCH      INLET',/,T35,'FLOW          FLOW',/,
     1       T35,'(CFS)',6X,'(CFS)',//)
      DO 92 I = 1 , NINCAP
      READ(NREAD,1026) IDCB,NJC
      J=IDCB
      NPCAP(J)=NJC
      READ(NREAD,1027) ((QAPP(J,K),QCAP(J,K)),K=1,NJC)
      IF(J.GT.1) WRITE(NPRINT,2000)
      WRITE(NPRINT,2013) IDCB,NJC
      WRITE(NPRINT,1915)
 2013 FORMAT(5X,'INLET TYPE ',T20,I5,//,5X,'NO. OF POINTS',T20,I5)
      WRITE(NPRINT,2092) ((QAPP(J,K),QCAP(J,K)),K=1,NJC)
 2092 FORMAT(T35,F6.2,4X,F6.2)
   92 CONTINUE
 1025 FORMAT(I5)
 1026 FORMAT(2I5)
 1027 FORMAT(16F5.0)
      WRITE(NPRINT,1906) (ITITL(I),I=1,40)
      WRITE(NPRINT,1916)
      WRITE(NPRINT,1917)
 1916 FORMAT(50X,'MAJOR SYSTEM DATA',//)
 1917 FORMAT(T10,'STREET',T22,'D/S',T30,'LENGTH',T40,'TYPE',T50,'NO. OF'
     1       ,T2,'NO.'
     1       ,T60,'INLET',T70,'INLET',T85,'CONNECTING',T100,'STORAGE',
     2       T115,'FLOW',/,
     2       T10,'SEGMENT',T20,'SEGMENT',T31,'(FT)',T51,'C.B.',T60,'TYPE',
     3       T70,'RESTRIC.',T88,'PIPE',T100,'ID NO.',T115,'HISTORY',//)
      TOTLG=0.0
      NCBTOT=0
      NINGN=0
C
C
C     MAJOR SYSTEM DATA
C
C
      DO 300 I = 1 , NSMAX
      READ (NREAD,1030) NS(I),NSD(I),XS(I),IDSS(I),NUMCB(I),IINC(I),
     1                  QLIM(I),NPD(I),NSTS(I),IPRTS(I)
      IF (NS(I).LE.0) GO TO 400
      NIN(I)=NS(I)
      IF(QLIM(I).EQ.0.0) QLIM(I)=QLIMIT
      NCBTOT=NCBTOT+NUMCB(I)
      IF (NUMCB(I).NE.0) GO TO 520
      GO TO 530
  520 CONTINUE
      NINGP=NINGN+1
  530 CONTINUE
      IIII=1
      IF(IPRTS(I).EQ.0) IIII=2
      WRITE (NPRINT,2020) I,NS(I),NSD(I),XS(I),IDSS(I),NUMCB(I),
     1                  IINC(I),QLIM(I),NPD(I),NSTS(I),ITN(IIII)
      TOTLG=TOTLG+XS(I)
  300 CONTINUE
  400 CONTINUE
      NOS=I-1
      AVDCB=TOTLG*2./NCBTOT
      WRITE (NPRINT,2030) NOS,TOTLG
 1030 FORMAT (2I5,F5.0,3I5,F5.0,3I5)
 2020 FORMAT(2X,I2,T8,I5,T19,I5,T31,F5.1,T37,I5,T47,I5,T57,I5,T70,F6.2,
     1       T85,I5,T100,I5,T116,A3)
 2030 FORMAT (///,5X,'TOTAL NUMBER OF STREET SEGMENTS ',T50,I5,
     1       //,5X,'TOTAL LENGTH OF MAJOR SYSTEM',T47,F8.2)
```

```
      IF(JSS.LE.0) GO TO 403
C
C
C     MAJOR SYSTEM STORAGE DATA
C
C
      NSST=0
      DO 401 I = 1 , NSTMAX
      READ(NREAD,1031) IDSTS(I)
 1031 FORMAT(I5)
      IF(IDSTS(I).LE.0) GO TO 402
      NSST=NSST+1
  401 CONTINUE
  402 CONTINUE
      WRITE(NPRINT,2032)
      WRITE(NPRINT,2033) NSST
 2032 FORMAT(///,5X,' DATA FOR MAJOR SYSTEM STORAGE '//)
 2033 FORMAT(10X,'NO. OF DETENSION STRUCTURES   ',I5)
      WRITE(NPRINT,1918)
 1918 FORMAT(///,T11,'NO.',T20,'ID NO.',//)
      DO 404 I = 1 , NSST
      WRITE(NPRINT,2034) I,IDSTS(I)
  404 CONTINUE
 2034 FORMAT(T8,I5,T20,I5)
  403 CONTINUE
C
C
C     MAJOR SYSTEM CONNECTIVITY
C
C
      IOUT=0
      DO 800 I = 1 , NOS
      N=NOS+IOUT
      DO 810 J = 1 , N
      IF (NSD(I).EQ.NS(J)) GO TO 820
  810 CONTINUE
      IOUT=IOUT+1
      J=NOS+IOUT
C
C
C     DUMMY SEGMENT FOR OUTLET
C
C
      NS(J)=NSD(I)
      IDMJ(J)=1
      IF(NSTS(I).NE.0) ISTOS(J)=NSTS(I)
      NSOO(IOUT)=J
  820 CONTINUE
      DO 830 K = 1 , NINS
      IF (NSS(J,K).GT.0) GO TO 830
      NSS(J,K)=I
      GO TO 800
  830 CONTINUE
  800 CONTINUE
      NCSS=NOS+IOUT
C
C
C     MAJOR SYSTEM/SURFACE STORAGE CONNECTIVITY
C
C
      DO 801 K = 1 , NSTMAX
      IINSTS=0
      DO 802 M = 1 , NOSS
      IF(ISTOS(M).EQ.IDSTS(K)) GO TO 803
      GO TO 802
  803 CONTINUE
      IINSTS=IINSTS+1
      NSTS(K,IINSTS)=M
  802 CONTINUE
  801 CONTINUE
      IF(IEVAL.EQ.3) GO TO 781
C
C
C     INLET/PIPE CONNECTIVITY
C
C
      DO 750 I = 1 , NOS
      DO 760 J = 1 , NOP
      IF (NPD(I).EQ.NP(J)) GO TO 770
  760 CONTINUE
  770 CONTINUE
      DO 780 K = 1 , NINI
      IF(NIP(J,K).GT.0) GO TO 780
      NIP(J,K)=I
      GO TO 750
  780 CONTINUE
  750 CONTINUE
  781 CONTINUE
C
C
C     SUB-CATCHMENT DATA
C
C
C
C
C     INFILTRATION PARAMETERS
C
C
      WRITE(NPRINT,1906) (ITITL(I),I=1,40)
      WRITE(NPRINT,1919)
 1919 FORMAT(/,50X,' SUB-CATCHMENT DATA',//)
      IF(INS.GT.0) GO TO 101
      READ(NREAD,1200) INFMAX,INFMIN,DECAY
 1200 FORMAT(3F8.0)
      WRITE(NPRINT,2300) INFMAX,INFMIN,DECAY
 2300 FORMAT(15X,'MAX. INFILTRATION',T45,F8.2,T55,'IN./HR.',/15X,
     1       'MIN. INFILTRATION',T45,F8.2,T55,'IN./HR.',/,15X,
     2       'DECAY RATE',T45,F8.6,T55,'1/SEC')
  101 CONTINUE
C
C
C     SUB-AREAS
C
C
      WRITE(NPRINT,2400)
      TAREA=0.0
      DO 100 I = 1 , NUSHAX
      READ(NREAD,1300) NW(I),NSD(I),ASW,PIMP(I),CHIMP(I),
     1                 CNP(I),S(I),WLAT(I),DETIMP(I),DETP(I),IPSTW(I)
 1300 FORMAT(2I5,8F5.0,I5)
```

```
      IF(NW(I).LE.0) GO TO 115
      AW(I)=ASW*43560.0
      FMAX(I)=INFMAX/43200.0
      FMIN(I)=INFMIN/43200.0
      DEC(I)=DECAY
      IITN=1
      IF(IPPTW(I).EQ.0) IITN=2
      WRITE(NPRINT,2401) I,NW(I),NSD(I),ASW,PIMP(I),CNIMP(I),CNP(I),
     1                   S(I),WLAT(I), DETIMP(I),DETP(I),ITN(IITN)
      DETIMP(I)=DETIMP(I)/12.0
      DETP(I)=DETP(I)/12.0
      TAREA=TAREA+ASW
 2400 FORMAT(//,
     1        T20,'SUBAREA',T30,'STREET',T40,'AREA',T50,'IMP.',T60,
     2        'MANNING',T70,'MANNING',T80,'SLOPE',T90,'WIDTH',T100,
     3        'DEP. STOR.',T111,'DEP. STOR.',T122,'FLOW',/,
     4        T23,'NO.',T30,'SEGMENT',T40,'(AC.)',T50,'(%)',T62,
     5        '(N)',T72,'(N)',T80,'(FT/FT)',T90,'(FT)',T100,
     6        'IMP. (IN)',T111,'PERV. (IN)',T122,'HISTORY',/,
     7        T60,'(IMP.)',T70,'(PERV.)',//)
 2401 FORMAT(T6,I2,T18,I5,T28,I5,T39,F5.1,T49,F5.1,T60,F5.3,T70,F5.3,
     1        T80,F5.3,T88,F6.0,T100,F8.6,T111,F8.6,T124,A3)
      IF(PIMP(I).EQ.0.0) GO TO 110
      WCON(I,1)=-1.486*WLAT(I)*SQRT(S(I))*100./(AW(I)*PIMP(I)*CNIMP(I))
  110 CONTINUE
      IF(PIMP(I).EQ.100.0) GO TO 120
      WCON(I,2)=-1.486*WLAT(I)*SQRT(S(I))*100./(AW(I)*(100.-PIMP(I))*CN
     1P(I))
  120 CONTINUE
  100 CONTINUE
  115 CONTINUE
      NOWS=I-1
      CBINT=NCBTOT/TAREA
      WRITE (NPRINT,2051) TAREA,NOWS
      WRITE (NPRINT,2050) NCBTOT,CBINT,AVDCB
 2051 FORMAT(///,T10,'TOTAL DRAINAGE AREA',T45,F8.2,' ACRES',//,
     1           T10,'NUMBER OF SUBAREAS',T45,I8)
 2050 FORMAT (//,T10,'TOTAL NUMBER OF INLETS',T45,I8,//,T10,
     1             'DENSITY OF INLETS',T45,F8.2,' C.B./ACRE',//,T10
     2           , 'AVERAGE DISTANCE BETWEEN INLETS',T45,F8.2,' FT')
      IF(INS.EQ.0) GO TO 102
      DO 106 I = 1 , NOWS
      READ(NREAD,2053)  (QR(I,J),J=1,NDELTC)
 2053 FORMAT(16F5.0)
  106 CONTINUE
  102 CONTINUE
C
C
C     SUBCATCHMENT / MAJOR SYSTEM CONNECTIVITY
C
C
      DO 700 I = 1 , NOWS
      DO 710 J = 1 , NOS
      IF (NSD(I).EQ.NS(J)) GO TO 720
  710 CONTINUE
  720 CONTINUE
      DO 730 K = 1 , NINW
      IF (NWS(J,K).GT.0) GO TO 730
      NWS(J,K)=I
      GO TO 700
  730 CONTINUE
  700 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE INIT
C
C
C
C
C
C
C     INITIALIZATION
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  COM/ INVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,NI
     1              ,NDELTC,INS,INTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4              IPPTP(200),QB(200),TMAX(200),ICPTRP,QPX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),SNPT,
     1              QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),RP1(5,20),NSP1(5),
     1              QQ1(5,20),QOTPP(5,100),OVPL(5,100),QP1SX(5)
      COMMON/  MJR/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1              NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             ,QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4              IPNTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1              QINT(5,100),NSTMAX,SS(10),CELAF(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1              IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2              0),CBMAX(200)
      COMMON/  SUP/ NWSMAX,NINW,NW(200),AW(200),PIMP(200),CNIMP(200),
     1              CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              WCON(200,5),FINT(100),WDEPTH(200,5),WFLOW(200),
     3              FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4              NWS(200,5),SUMQW(200),QW(200,100),IPPTW(200)
C
C
C
      NWSMAX=200
      NSMAX=200
      NPMAX=200
      NSTMAX=5
      NPTMAX=5
      NINS=5
      NINP=5
      NINW=5
      NINI=5
      DO 10 I = 1 , NINS
      NSOO(I)=0
      DO 20 J = 1 , NSMAX
      NSS(J,I)=0
      IDMJ(J)=0
```

```
      NSO(J)=0
      IDSS(J)=0
      IINC(J)=0
      SUMCW(J)=0.0
      NS(J)=0
      NSD(J)=0
      XS(J)=0.0
      NUMCB(J)=0
      QLIM(J)=0.0
      NIN(J)=0
      IPPTS(J)=0
      SPMAX(J)=0.0
      SCMAX(J)=0.0
      CBMAX(J)=0.0
   20 CONTINUE
   10 CONTINUE
      DO 25 I = 1 , NSMAX
      DO 26 J = 1 , 100
      QINS(I,J)=0.0
      SDPT(I,J)=0.0
      QOUTS(I,J)=0.0
      QCAPT(I,J)=0.0
   26 CONTINUE
   25 CONTINUE
      DO 30 I = 1 , NINP
      NPOO(I)=0
      DO 40 J = 1 , NPMAX
      NPP(J,I)=0
      IDNP(J)=0
      SUMQIN(J)=0.0
      NRO(J)=0
      NP(J)=0
      NPD(J)=0
      XP(J)=0.0
      SP(J)=0.0
      CP(J)=0.0
      DP(J)=0.0
      DDP(J)=0.0
      QB(J)=0.0
      QPAX(J)=0.0
      QDQP(J)=0.0
      QEX(J)=0.0
      IPPTP(J)=0
   40 CONTINUE
   30 CONTINUE
      DO 50 I = 1 , NINW
      DC 60 J = 1 , NWSMAX
      NWS(J,I)=0
   60 CONTINUE
   50 CONTINUE
      DO 35 I = 1 , NINI
      DO 45 J = 1 , NPMAX
      NIP(J,I)=0
   45 CONTINUE
   35 CONTINUE
      DC 55 I = 1 , 5
      DO 65 J = 1 , NWSMAX
      NW(J)=0
      AW(J)=0.0
      PIMP(J)=0.0
```

```
DDM0720
DDM0721
DDM0722
DDM0723
DDM0724
DDM0725
DDM0726
DDM0727
DDM0728
DDM0729
DDM0730
DDM0731
DDM0732
DDM0733
DDM0734
DDM0735
DDM0736
DDM0737
DDM0738
DDM0739
DDM0740
DDM0741
DDM0742
DDM0743
DDM0744
DDM0745
DDM0746
DDM0747
DDM0748
DDM0749
DDM0750
DDM0751
DDM0752
DDM0753
DDM0754
DDM0755
DDM0756
DDM0757
DDM0758
DDM0759
DDM0760
DDM0761
DDM0762
DDM0763
DDM0764
DDM0765
DDM0766
DDM0767
DDM0768
DDM0769
DDM0770
DDM0771
DDM0772
DDM0773
DDM0774
DDM0775
DDM0776
DDM0777
DDM0778
DDM0779
```

```
      CNIMP(J)=0.0
      CNP(J)=0.0
      S(J)=0.0
      VIAT(J)=0.0
      DFTINP(J)=0.0
      FMAX(J)=0.0
      FMIN(J)=0.0
      DIC(J)=0.0
      IPPTN(J)=0
      DET(J,I)=0.0
      NDEPTH(J,I)=0.0
      VCCN(J,I)=0.0
   65 CONTINUE
   55 CONTINUE
      DO 70 I = 1 , 100
      DO 80 J = 1 , NWSMAX
      VFLOW(J)=0.0
      QP(J,I)=0.0
      NINT(I)=0.0
   80 CONTINUE
   75 CONTINUE
      DO 15 I = 1 , 50
      DO 16 J = 1 , 10
      NCAP(J)=0
      SS(J)=0.0
      CAPP(J,I)=0.0
      QCAP(J,I)=0.0
      TS(I,J)=0.0
      FLOW(I,J)=0.0
      CFLAB(I,J)=0.0
      SPEED2(I,J)=0.0
      QOSS(I,J)=0.0
   16 CONTINUE
   15 CONTINUE
      DO 90 I = 1 , 100
      DO 95 J = 1 , NPMAX
      QINP(J,I)=0.00
      QOUTP(J,I)=0.0
   95 CONTINUE
   90 CONTINUE
      DC 100 I = 1 , NSTMAX
      DO 101 J = 1 , NINS
      NSTS(I,J)=0
  101 CONTINUE
  100 CONTINUE
      DO 102 I = 1 , NINS
      DO 103 J = 1 , 100
      QINP(I,J)=0.0
  103 CONTINUE
  102 CONTINUE
      DO 104 I = 1 , NPTMAX
      DO 105 J = 1 , NINP
      IDSTP(I)=0
      QPLSP(I)=0.0
      NSTP(I,J)=0
      IOPTSP(I)=0
      SPMX(I)=0.0
      QPINX(I)=0.0
  105 CONTINUE
  104 CONTINUE
```

```
DDM0780
DDM0781
DDM0782
DDM0783
DDM0784
DDM0785
DDM0786
DDM0787
DDM0788
DDM0789
DDM0790
DDM0791
DDM0792
DDM0793
DDM0794
DDM0795
DDM0796
DDM0797
DDM0798
DDM0799
DDM0800
DDM0801
DDM0802
DDM0803
DDM0804
DDM0805
DDM0806
DDM0807
DDM0808
DDM0809
DDM0810
DDM0811
DDM0812
DDM0813
DDM0814
DDM0815
DDM0816
DDM0817
DDM0818
DDM0819
DDM0820
DDM0821
DDM0822
DDM0823
DDM0824
DDM0825
DDM0826
DDM0827
DDM0828
DDM0829
DDM0830
DDM0831
DDM0832
DDM0833
DDM0834
DDM0835
DDM0836
DDM0837
DDM0838
DDM0839
```

```
      DO 106 I = 1 , NINP
      DO 107 J = 1 , 100
      QINPP(I,J)=0.0
      QOTPP(I,J)=0.0
      OVPL(I,J)=0.0
  107 CONTINUE
  106 CONTINUE
      DO 109 I = 1 , NPTMAX
      DO 108 J = 1 , 20
      SP1(I,J)=0.0
      PP1(I,J)=0.0
      QQ1(I,J)=0.0
      NSP1(I)=0.0
  108 CONTINUE
  109 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE ORDER
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ INVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,RI
     1              ,NDELTC,INS,INTS
      COMMON/  BIN/ NPMAX,PP(200),NPD(200),IP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDEN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDQF(200),
     4              IPRTP(200),QB(200),TMAX(200),IOPTRP,QZI(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1              QINPT(5,100),NPTMAX,QBLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMI(5),SP1(5,20),PP1(5,20),NSP1(5),
     1              QQ1(5,20),QOTPP(5,100),OVPL(5,100),QB1MX(5)
      COMMON/  BJN/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1              NOS,NOSS,IDBJ(200),NSOO(5),NSS(200,5),NSO(200),
     2              QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4              IPRTS(200),SDPT(200,100),SPMAX(200),SUMJX(200)
      COMMON/ BJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1              QINP(5,100),NSTMAX,SS(10),CELAF(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),OCAP(10,50),FORCB(200),
     1              IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2              0),CBASE(200)
      COMMON/  SUP/ NWSMAX,NINW,NW(200),AW(200),PIMP(200),CHIMP(200),
     1              CMP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              NCON(200,5),NINY(100),NDEPTH(200,5),NPLCN(200),
     3              PMAX(200),PMIN(200),DEC(200),DET(200,5),NOWS,
     4              NWS(200,5),SUMQW(200),QW(200,100),IPRTW(200)
C
C
C
      DO 100 I = 1 , NISS
      IF(NSOO(I).LE.0) GO TO 110
      NN=NSMAX+1-I
      NSO(NN)=NSOO(I)
  100 CONTINUE
  110 CONTINUE
```

```
      DO 120 I = 1 , NSMAX
      NNN=NSMAX+1-I
      II=NSO(NNN)
      IF(II.LE.0) GO TO 130
      DO 140 J = 1 , NINS
      IF(NSS(II,J).EQ.0) GO TO 130
      NN=NN-1
      NSC(NN)=NSS(II,J)
  140 CONTINUE
  130 CONTINUE
  120 CONTINUE
      NN=0
      DO 150 I = 1 , NSMAX
      IF(NSO(I).EQ.0) GO TO 150
      NN=NN+1
      NSO(NN)=NSO(I)
  150 CONTINUE
 2000 FORMAT(1H1)
      IF(INVAL.EQ.3) GO TO 260
C
C
C     SET UP ORDERING ARRAY-MINOR SYSTEM
C
C
      DO 200 I = 1 , NINP
      IF(NPOO(I).LE.0) GO TO 210
      NN=NPMAX+1-I
      NPC(NN)=NPOO(I)
  200 CONTINUE
  210 CONTINUE
      DO 220 I = 1 , NPMAX
      NNN=NPMAX+1-I
      II=NPC(NNN)
      IF(II.LE.0) GO TO 230
      DO 240 J = 1 , NINP
      IF(NPP(II,J).EQ.0) GO TO 230
      NP=NN-1
      NPO(NN)=NPP(II,J)
  240 CONTINUE
  230 CONTINUE
  220 CONTINUE
      NN=0
      DO 250 I = 1 , NPMAX
      IF(NPO(I).EQ.0) GO TO 250
      NN=NN+1
      NPO(NN)=NPO(I)
  250 CONTINUE
  260 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE SUPPLY
C
C
C
C
C
C     RUNOFF COMPUTATION - SAME AS SWMM-RUNOFF ROUTINE
```

```
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TINEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,RI
     1             ,NDELTC,INS,INTS
      COMMON/  NIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1             DN(200),NOP,NOPP,IDAN(200),NPOO(5),NPO(200),
     2             NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4             IPNTP(200),QB(200),TMAX(200),IOPTPP,OEX(200)
      COMMON/ NIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ NIN2/ IOPTSP(5),SPMX(5),SP1(5,20),PP1(5,20),NSP1(5),
     1             QQ1(5,20),QOTPP(5,100),OTPL(5,100),QP1NT(5)
      COMMON/  NJN/ NSMAX,NINS,NS(200),NSD(200),XS(200),IOSS(200),
     1             NOS,NOSS,IDNJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3             SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4             IPNTS(200),SDPT(200,100),STMAX(200),SDMAX(200)
      COMMON/ NJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1             QINF(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUNCB(200),
     1             IINC(200),QLIN(200),NIN(200),NIP(200,5),QCAPY(200,10
     2             0),CBMAX(200)
      COMMON/  SUP/ NSMAX,NINW,NW(200),AW(200),PINP(200),CNINP(200),
     1             CNP(200),S(200),NLAT(200),DETINP(200),DETP(200),
     2             NCON(200,5),RINT(100),WDEPTH(200,5),WPLOW(200),
     3             FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4             NWS(200,5),SUBQW(200),QR(200,100),IPPTN(200)
      DIMENSION TP (200)
      DATA TP/200*0.0/
C
C
C
      NSEG=2
      DO 150 J = 1 , NOWS
      TINEC=0.0
      DO 135 I = 1 , NDELTC
      TINEC=TINEC+DELTC
      IF(TINEC.GT.TTOTC) GO TO 145
      INDEX=0.99+TINEC/DELTR
      RI=RINT(INDEX)
      DELP=0.0
      WPLOW(J)=0.0
      DO 160 K = 1 , NSEG
      GO TO (10,20),K
   10 CONTINUE
C
C
C     IMPERVIOUS AREA
C
C
      DET(J,K)=DETINP(J)
      ASEG=AW(J)*PINP(J)/100.0
      FLOSS=0.0
      GO TO 170
   20 CONTINUE
C
C
C     PERVIOUS AREA
C
C
      DET(J,K)=DETP(J)
      ASEG=AW(J)*(100.0-PINP(J))/100.0
      FAVAIL=RI+WDEPTH(J,K)/DELTCS
      IF(FAVAIL.GT.0.0) GO TO 180
      FLOSS=0.0
      GO TO 170
  180 CONTINUE
      TLIM=16.0/DEC(J)
      TP1=TP(J)
      IF(TP(J).LT.TLIM) GO TO 190
      FLOSS1=FMIN(J)
      TP(J)=TLIM
      GO TO 200
  190 CONTINUE
      FLOSS1=(CUNINF(J,TP(J)+DELTCS)-CUNINF(J,TP(J)))/DELTCS
  200 CONTINUE
      IF (FAVAIL.LT.FLOSS1) GO TO 210
      FLOSS=FLOSS1
      TP1=TP(J)+DELTCS
      GO TO 220
  210 CONTINUE
      FLOSS=FAVAIL
      IF(TP(J).GE.TLIM) GO TO 230
      CUMI=CUNINF(J,TP(J))
      CUMI=CUMI+FAVAIL*DELTCS
      TP1=TP(J)+DELTC2
C
C
C     NEWTON-RAPHSON ITERATION TO FIND NEW TP
C
C
      DO 240 M = 1 , 11
      EEE=EXP(-TP1*DEC(J))
      DFF=FMIN(J)+(FMAX(J)-FMIN(J))*EEE
      FF=FMIN(J)*TP1+(FMAX(J)-FMIN(J))/DEC(J)*(1.0-EEE)-CUMI
      DELF=FF/DFF
      TP1=TP1-DELF
      IF(ABS(DELF).GT.0.001*DELTCS) GO TO 240
      IF(TP1.LT.TP(J)+DELTCS.AND.TP1.GT.TP(J)) GO TO 220
  240 CONTINUE
      WRITE(NPRINT,2500)
 2500 FORMAT(//,5X,'********* NO CONVERGENCE IN INFILTRATION ROUTINE ')
      TP(J)=TP(J)+0.9*DELTCS
      GO TO 230
  220 CONTINUE
      TP(J)=TP1
  230 CONTINUE
      IF(TP(J).GT.TLIM) TP(J)=TLIM
  170 CONTINUE
      IF(RI-FLOSS+WDEPTH(J,K)/DELTCS.GT.0.0) GO TO 250
      FLOSS=RI+WDEPTH(J,K)/DELTCS
      WDEPTH(J,K)=0.0
      WPLO=0.0
      GO TO 260
  250 CONTINUE
      IF((RI-FLOSS)*DELTCS+WDEPTH(J,K).LE.DET(J,K)) GO TO 270
      DO 280 II = 1 , 11
      DO=WDEPTH(J,K)-DET(J,K)+0.5*DELR
```

```
      IF(DO.LT.0.0) DO=0.0
      F=DELN-DELTCS*(WCON(J,K)*DO**(5./3.)+(RI-RLOSS))
      DF=1.-DELTCS*(.8333333*WCON(J,K)*DO**(2./3.))
      DEL=DELN-F/DF
      IF(II.EQ.1) GO TO 290
      IF((ABS(DEL-DELN)).LT.(ABS(0.01*DELN))) GO TO 300
  290 CONTINUE
      DELN=DEL
  280 CONTINUE
      IF(DELN.LT.0.0001*WDEPTH(J,K)) GO TO 300
      WRITE(NPRINT,2510)
 2510 FORMAT(//,'*********** NO CONVERGENCE IN RUNOFF*************')
  300 CONTINUE
      DCOPN=WDEPTH(J,K)+DEL
      DELN=0.0
C
C
C     COMPUTE AVERAGE FLOW DURING TIME STEP
C
C
      WFLO=(RI-RLOSS)*ASEG-(DCOPN-WDEPTH(J,K))*ASEG/DELTCS
      IF(WFLO.GT.0.0) GO TO 310
  270 CONTINUE
      WFLO=0.0
      DCOPN=WDEPTH(J,K)+(RI-RLOSS)*DELTCS
  310 CONTINUE
      WDEPTH(J,K)=DCOPN
  260 CONTINUE
      WFLOW(J)=WFLOW(J)+WFLO
  160 CONTINUE
      QR(J,I)=WFLOW(J)
  135 CONTINUE
  145 CONTINUE
  150 CONTINUE
  130 CONTINUE
  140 CONTINUE
      RETURN
      END
C
C
C
      FUNCTION  CUMINF(L,TT)
C
C
C
      COMMON/  SUP/ NUSMAX,NINW,NW(200),AW(200),PIMP(200),CNIMP(200),
     1              CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              WCON(200,5),RINT(100),WDEPTH(200,5),WFLOW(200),
     3              FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4              NWS(200,5),SUMQW(200),QR(200,100),IPRTW(200)
C
C
C
      CUMINF=FMIN(L)*TT+(FMAX(L)-FMIN(L))/DEC(L)*(1.0-EXP(-DEC(L)*TT))
      RETURN
      END
C
C
C
      SUBROUTINE MAJOR
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ INTVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,RI
     1              ,NDELTC,IWS,IPTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDHN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDOP(200),
     4              IPRTP(200),QB(200),TMAX(200),IOPTPP,QPI(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTCS(200),NNSTP(5,5),NNPT,
     1              QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),PP1(5,20),NSP1(5),
     1              QQ1(5,20),QOTPP(5,100),OVFL(5,100),OPINX(5)
      COMMON/  MJR/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1              NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2              QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3              SPEED2(50,10),IS,AS(50,10),QQSS(50,10),NPTS,
     4              IPRTS(200),SDPT(200,100),SFMAX(200),SDPMX(200)
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTCS(200),NNSTS(5,5),NNST,
     1              QINP(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1              IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,100
     2              0),CBMAX(200)
      COMMON/  SUP/ NUSMAX,NINW,NW(200),AW(200),PIMP(200),CNIMP(200),
     1              CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              WCON(200,5),RINT(100),WDEPTH(200,5),WFLOW(200),
     3              FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4              NWS(200,5),SUMQW(200),QR(200,100),IPRTW(200)
C
C
C
      DO 100 I = 1 , NOSS
      J = NSO(I)
      DO 110 N = 1 , NDELTC
C
C
C     INPUT FROM SURFACE RUNOFF
C
C
      SUMQW(J)=0.0
      DO 200 K = 1 , NINW
      IF(NWS(J,K).EQ.0) GO TO 210
      NWW=NWS(J,K)
      SUMQW(J)=SUMQW(J)+QR(NWW,N)
  200 CONTINUE
  210 CONTINUE
      QINS(J,N)=SUMQW(J)
C
C
C     INPUT FROM UPSTREAM MAJOR SYSTEM
C
C
      DO 300 K = 1 , NINS
      IF(NSS(J,K).EQ.0) GO TO 310
      NG=NSS(J,K)
      QINS(J,N)=QINS(J,N)+QOUTS(NG,N)
  300 CONTINUE
  310 CONTINUE
```

```
  110 CONTINUE
C
C
C     ROUTE MAJOR SYSTEM FLOW TO INLET(S)
C
C
C
C
      CALL MAJOR2(J)
C
C
C
C
C     CALCULATE  CAPTURE BY INLETS
C
C
C
C
      CALL INLET(J)
C
C
      DO 120 N = 1 , NDELTC
      QOUTS(J,N)=QOUTS(J,N)-QCAPT(J,N)
      IF (QOUTS(J,N).LT.0.0) QOUTS(J,N)=0.0
  120 CONTINUE
  100 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE MAJOR2(IJ)
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,PI
     1             ,NDELTC,INS,INTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),XP(200),SP(200),CP(200),
     1             DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2             NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),ODQP(200),
     4             IPSTP(200),QB(200),TMAX(200),IOPTRP,QTX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QMLSP(5)
      COMMON/ MIN2/ IOPTSP(5),STMX(5),SP1(5,20),PP1(5,20),NSP1(5),
     1             QQ1(5,20),QOTPP(5,100),OTPL(5,100),QPIMX(5)
      COMMON/  MJP/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1             NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3             SPEED2(50,10),IS,AS(50,10),QQSS(50,10),NPTS,
     4             IPPTS(200),SDPT(200,100),SPMAX(200),STMAX(200),
      COMMON/ MJP1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1             QINP(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),OCAP(10,50),NUMCB(200),
     1             IINC(200),QLIE(200),NIN(200),NIP(200,5),QCAPT(200,10
     2             0),CBMAX(200)
      COMMON/  SUP/ NUSMAX,NINN,NN(200),AN(200),PIMP(200),CHIMP(200),
     1             CMP(200),S(200),VLAT(200),DETIMP(200),DEIP(200),
     2             NCON(200,5),RINT(100),NDEPTH(200,5),VFLOW(200),
     3             PMAX(200),PMIN(200),DEC(200),DET(200,5),NONS,
     4             NNS(200,5),SUMQN(200),QP(200,100),IPPTN(200)
      REAL I1,I2
C
C
C
C
C
C     MAJOR SYSTEM FLOW ROUTING
C
C
C
      DIMENSION TQS(100),TT(3000),QII(3000),QOO(3000)
      DIMENSION A(10),B(3),X(3),Y(3)
      J=IDSS(IJ)
      ISS=IS(IJ)
      SSS=SS(J)
      DO 5 I = 1, 100
      TQS(I)= 0.0
    5 CONTINUE
      DO 6 I = 1 , 3000
      TT(I)=0.0
      QII(I)=0.0
      QOO(I)=0.0
    6 CONTINUE
      DO 110 I = 2 , NDELTC
      TQS(I)=DELTC*(I-1)
  110 CONTINUE
C
C
C     COMPUTE MAX. INFLOW AND TIME
C
C
      SPMAX(IJ)=0.0
      NN=NDELTC-1
      DO 1 N = 1 , NN
      IF(QINS(IJ,N+1).GT.SPMAX(IJ)) GO TO 2
      GO TO 1
    2 CONTINUE
      SPMAX(IJ)=QINS(IJ,N+1)
      TPP=TQS(N+1)
    1 CONTINUE
      IF(IDMJ(IJ).EQ.1) GO TO 70
      IF(INTS.GT.0) GO TO 70
      IF(SPMAX(IJ).LE.0.0) GO TO 70
C
C
C      CALCULATE MAX. CELERITY AND DT1
C
C
      QCMC=SPMAX(IJ)
      DO 100 K = 1 , NPTS
      ANS=FLOW(K,J)
      IF(QCMC.LT.ANS) GO TO 101
  100 CONTINUE
  101 CONTINUE
      IF(K.EQ.1) K=K+1
      IF(K.EQ.NPTS) K = K-1
      X(1)=FLOW(K-1,J)
      X(2)=FLOW(K,J)
      X(3)=FLOW(K+1,J)
```

```
      Y(1)=CELAP(K-1,J)
      Y(2)=CELAP(K,J)
      Y(3)=CELAP(K+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      CMAX=B(1)+B(2)*QCMC+B(3)*QCMC**2.0
      DT1=(ISS/CMAX)/60.0
      NDT1=TTOTC/DT1+1
      DT1=TTOTC/(NDT1-1)
      DT1S=DT1*60.0
C
C
C     INTERPOLATE FLOWS AT DT1
C
C
      QII(1)=QINS(IJ,1)
      DO 120 I = 2 , NDT1
      TT(I)=DT1*(I-1)
      TTT=TT(I)
      DO 130 K = 2 , NDELTC
      ANS=TQS(K)
      IF(TTT.LT.ANS) GO TO 131
  130 CONTINUE
  131 CONTINUE
      X1=TQS(K-1)
      X2=TQS(K)
      Y1=QINS(IJ,K-1)
      Y2=QINS(IJ,K)
      CALL LIN(X1,X2,Y1,Y2,TTT,YO)
      QII(I)=YO
  120 CONTINUE
C
C
C     DONT MISS THE PEAK
C
C
      DO 12 I = 2 , NDT1
      TT1=TT(I-1)
      TT2=TT(I)
      IF(TT1.EQ.TMP.OR.TT2.EQ.TMP) GO TO 14
      IF(TT1.LT.TMP.AND.TT2.GT.TMP) GO TO 13
   12 CONTINUE
   13 CONTINUE
      TDIFF1=TMP-TT1
      TDIFF2=TT2-TMP
      IF(TDIFF1.GT.TDIFF2) QII(I)=SPMAX(IJ)
      IF(TDIFF2.GT.TDIFF1) QII(I-1)=SPMAX(IJ)
   14 CONTINUE
C
C
C     ROUTING
C
C
      NNN=NDT1-1
      DO 20 N = 1 , NNN
      I1=QII(N)
      I2=QII(N+1)
      O1=COO(N)
      DO 30 K = 1 , NPTS
      ANS=FLOW(K,J)
      IF(I1.LT.ANS) GO TO 50
   30 CONTINUE
   50 CONTINUE
      IF(K.EQ.1) K=K+1
      IF(K.EQ.NPTS) K=K-1
      X(1)=FLOW(K-1,J)
      X(2)=FLOW(K,J)
      X(3)=FLOW(K+1,J)
      Y(1)=CELAP(K-1,J)
      Y(2)=CELAP(K,J)
      Y(3)=CELAP(K+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      C1=B(1)+B(2)*I1+B(3)*I1**2.0
      Y(1)=SPEED2(K-1,J)
      Y(2)=SPEED2(K,J)
      Y(3)=SPEED2(K+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      SPP1=B(1)+B(2)*I1+B(3)*I1**2.0
      DO 31 K = 1 , NPTS
      ANS=FLOW(K,J)
      IF(I2.LT.ANS) GO TO 53
   31 CONTINUE
   53 CONTINUE
      IF(K.EQ.1) K=K+1
      IF(K.EQ.NPTS) K=K-1
      X(1)=FLOW(K-1,J)
      X(2)=FLOW(K,J)
      X(3)=FLOW(K+1,J)
      Y(1)=CELAP(K-1,J)
      Y(2)=CELAP(K,J)
      Y(3)=CELAP(K+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      C2=B(1)+B(2)*I2+B(3)*I2**2.0
      Y(1)=SPEED2(K-1,J)
      Y(2)=SPEED2(K,J)
      Y(3)=SPEED2(K+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      SPP2=B(1)+B(2)*I2+B(3)*I2**2.0
      DO 32 K = 1 , NPTS
      ANS=FLOW(K,J)
      IF(O1.LT.ANS) GO TO 54
   32 CONTINUE
   54 CONTINUE
      IF(K.EQ.1) K=K+1
      IF(K.EQ.NPTS) K=K-1
      X(1)=FLOW(K-1,J)
      X(2)=FLOW(K,J)
      X(3)=FLOW(K+1,J)
      Y(1)=CELAP(K-1,J)
      Y(2)=CELAP(K,J)
      Y(3)=CELAP(K+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      C3=B(1)+B(2)*O1+B(3)*O1**2.0
      Y(1)=SPEED2(K-1,J)
      Y(2)=SPEED2(K,J)
      Y(3)=SPEED2(K+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      SPP3=B(1)+B(2)*O1+B(3)*O1**2.0
      PHI1=I1/(2.0*SPP1*SSS*C1*ISS)
      PHI2=I2/(2.0*SPP2*SSS*C2*ISS)
      PHI3=O1/(2.0*SPP3*SSS*C3*ISS)
```

```
      PHI=(PHI1+PHI2+PHI3)/3.0
      IF(PHI.LE.0.0) PHI=0.0
      IF(PHI.GE.1.0) PHI=1.0
      ALPH1=C1*DT1S/XSS
      ALPH2=C2*DT1S/XSS
      ALPH3=C3*DT1S/XSS
      ALPH=(ALPH1+ALPH2+ALPH3)/3.0
      ALL=1.+ALPH+PHI
      CE1=(1.+ALPH-PHI)/ALL
      CE2=(-1.+ALPH+PHI)/ALL
      CE3=(1.-ALPH+PHI)/ALL
      QCO(N+1)=CE1*I1+CE2*I2+CE3*O1
      IF(QCO(N+1).LT.0.0) QCO(N+1)=0.0
   20 CONTINUE
C
C
C     ALLOCATE PEAK OUTFLOW AND TIME
C
C
      IF(IDBJ(IJ).EQ.1.OR.IRTS.GT.0) GO TO 52
      QPEAK1=0.0
      DO 51 I = 1 , NDT1
      IF(QCO(I).GT.QPEAK1) GO TO 55
      GO TO 51
   55 CONTINUE
      QPEAK1=QCO(I)
      TMP1=TT(I)
   51 CONTINUE
   52 CONTINUE
C
C
C     INTERPOLATE FLOWS BACK TO DELTC
C
C
   70 CONTINUE
      DO 80 N = 1 , NDELTC
      IF(IDBJ(IJ).EQ.1.OR.IRTS.GT.0) GO TO 81
      IF (SPMAX(IJ).LE.0.0) GO TO 81
      TIMTR=TQS(N)
      DO 90 NN = 1 , NDT1
      ANS=TT(NN)
      IF(TIMTR.LT.ANS) GO TO 95
   90 CONTINUE
   95 CONTINUE
      X1=TT(NN-1)
      X2=TT(NN)
      Y1=QCO(NN-1)
      Y2=QCO(NN)
      CALL LIN(X1,X2,Y1,Y2,TIMTR,YO)
      QOUTS(IJ,N)=YO
      IF(QOUTS(IJ,N).LE.0.0) QOUTS(IJ,N)=0.0
      GO TO 80
   81 CONTINUE
      QOUTS(IJ,N)=QINS(IJ,N)
      IF(QOUTS(IJ,N).LE.0.0) QOUTS(IJ,N)=0.0
   80 CONTINUE
C
C
C     DON'T MISS THE PEAK
C
C
      IF(SPMAX(IJ).LE.0.0) GO TO 66
      IF(IDBJ(IJ).EQ.1.OR.IRTS.GT.0) GO TO 64
      DO 62 I = 2 , NDELTC
      TT1=TQS(I-1)
      TT2=TQS(I)
      IF(TT1.EQ.TMP1.OR.TT2.EQ.TMP1) GO TO 64
      IF(TT1.LT.TMP1.AND.TT2.GT.TMP1) GO TO 63
   62 CONTINUE
   63 CONTINUE
      TDIFF1=TMP1-TT1
      TDIFF2=TT2-TMP1
      IF(TDIFF1.GT.TDIFF2) QOUTS(IJ,I)=QPEAK1
      IF(TDIFF2.GT.TDIFF1) QOUTS(IJ,I-1)=QPEAK1
   64 CONTINUE
C
C
C     COMPUTE DEPTH OF FLOW
C
C
      DO 48 N = 1 , NDELTC
      DO 45 I = 1 , NPTS
      ANS=FLOW(I,J)
      IF(QINS(IJ,N+1).LT.ANS) GO TO 46
   45 CONTINUE
   46 CONTINUE
      X(1)=FLOW(I-1,J)
      X(2)=FLOW(I,J)
      X(3)=FLOW(I+1,J)
      Y(1)=YS(I-1,J)
      Y(2)=YS(I,J)
      Y(3)=YS(I+1,J)
      CALL LESQ(A,B,X,Y,2,3)
      SDPT(IJ,N+1)=B(1)+B(2)*QINS(IJ,N+1)+B(3)*QINS(IJ,N+1)**2.0
   48 CONTINUE
C
C
C     COMPUTE MAX. FLOW DEPTH
C
C
      SDMAX(IJ)=0.0
      DO 65 N = 1 , NDELTC
      IF(SDPT(IJ,N).GT.SDMAX(IJ)) SDMAX(IJ)=SDPT(IJ,N)
   65 CONTINUE
   66 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE LIN(X1,X2,Y1,Y2,XO,YO)
C
C
      IF(Y2.GE.Y1) YO=Y2-(X2-XO)*(Y2-Y1)/(X2-X1)
      IF(Y2.LT.Y1) YO=Y1-(XO-X1)*(Y1-Y2)/(X2-X1)
      RETURN
      END
C
C
C
```

```
      SUBROUTINE INLET(IK)
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ ITVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTN,NI
     1             ,NDELTC,INS,INTS
      COMMON/  NIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1             DP(200),NOP,NOPP,IDMN(200),NPOC(5),NPO(200),
     2             NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),QDOT(200),
     4             IPPTP(200),QB(200),TMAX(200),IOPTPP,QEX(200)
      COMMON/ NIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ NIN2/ IOPTSP(5),SPMX(5),SP1(5,20),EP1(5,20),NSP1(5),
     1             QO1(5,20),QOTPP(5,100),OVFL(5,100),QEINY(5)
      COMMON/  MJN/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1             NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3             SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4             IPPTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1             QINP(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1             IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2             0),CBMAX(200)
      COMMON/  SUP/ NNSMAX,NINW,NW(200),AW(200),PIMP(200),CSIMP(200),
     1             CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2             WCON(200,5),NINT(100),WDEPTH(200,5),WFLOW(200),
     3             FMAX(200),FMIN(200),DEC(200),DET(200,5),NONS,
     4             NWS(200,5),SUMQW(200),QP(200,100),IPPTW(200)
C
C
C
C
C
C     THIS SUBROUTINE COMPUTES FLOWS CAPTURED BY INLET C.B. BASED
C     ON INLET TYPE, APPROACH FLOW,ETC.
C
C
      DIMENSION A(10),B(3),X(3),Y(3)
      IF(IDMJ(IK).EQ.1) GO TO 60
      IC=IINC(IK)
      NPTC=NPCAP(IC)
      DO 50 N = 1 , NDELTC
      IF(QOUTS(IK,N).LE.0.0001) GO TO 30
      DO 10 K = 1 , NPTC
      ABS=QAPP(IC,K)
      Q2=QOUTS(IK,N)/2.
      IF(NUMCB(IK).GT.2)Q2=QOUTS(IK,N)/NUMCB(IK)
      IF(Q2.LE.ABS) GO TO 20
   10 CONTINUE
      QCB=QLIM(IK)
      QCAPT(IK,N)=QCB*NUMCB(IK)
      GO TO 50
   20 CONTINUE
      IF(K.EQ.NPTC) K=K-1
      X(1)=QAPP(IC,K-1)
      X(2)=QAPP(IC,K)
      X(3)=QAPP(IC,K+1)
      Y(1)=QCAP(IC,K-1)
      Y(2)=QCAP(IC,K)
      Y(3)=QCAP(IC,K+1)
      CALL LESQ(A,B,X,Y,2,3)
      QCB=B(1)+B(2)*Q2+B(3)*Q2**2
      IF(QCB.GT.Q2) QCB=Q2
      IF(QCB.GT.QLIM(IK)) QCB=QLIM(IK)
      QCAPT(IK,N)=QCB*NUMCB(IK)
      GO TO 40
   30 CONTINUE
      QCAPT(IK,N)=0.0
   40 CONTINUE
   50 CONTINUE
C
C
C     COMPUTE MAX. CAPTURE
C
C
      CBMAX(IK)=0.0
      DO 70 N = 1 , NDELTC
      IF(QCAPT(IK,N).GT.CBMAX(IK)) CBMAX(IK)=QCAPT(IK,N)
   70 CONTINUE
   60 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE MINOR
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ ITVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTN,NI
     1             ,NDELTC,INS,INTS
      COMMON/  NIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1             DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2             NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),QDOT(200),
     4             IPPTP(200),QB(200),TMAX(200),IOPTPP,QEX(200)
      COMMON/ NIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ NIN2/ IOPTSP(5),SPMX(5),SP1(5,20),EP1(5,20),NSP1(5),
     1             QO1(5,20),QOTPP(5,100),OVFL(5,100),QEINY(5)
      COMMON/  MJN/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1             NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3             SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4             IPPTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1             QINP(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1             IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2             0),CBMAX(200)
      COMMON/  SUP/ NNSMAX,NINW,NW(200),AW(200),PIMP(200),CSIMP(200),
     1             CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2             WCON(200,5),NINT(100),WDEPTH(200,5),WFLOW(200),
     3             FMAX(200),FMIN(200),DEC(200),DET(200,5),NONS,
     4             NWS(200,5),SUMQW(200),QP(200,100),IPPTW(200)
C
```

```
C
C
C     THIS SUBROUTINE ROUTES THE FLOWS CAPTURED BY THE INLETS TO THE
C     OUTLET(S) OF MINOR SYSTEM, OR TO STORAGE LOCATION(S).
C     IT CAN ALSO SIZE PIPES BASED ON PEAK INFLOWS.
C
C
      DO 100 I = 1 , NOPP
      J=NPO(I)
      DO 110 N = 1 , NDELTC
C
C
C     FLOW CAPTURED BY INLETS
C
C
      SUMQIN(J)=0.0
      DO 200 K = 1 , NINP
      IF(NIP(J,K).EQ.0) GO TO 210
      NI=NIP(J,K)
      SUMQIN(J)=SUMQIN(J)+QCAPT(NI,N)
  200 CONTINUE
  210 CONTINUE
      QINP(J,N)=SUMQIN(J)
C
C
C     INFLOW FROM UPSTREAM PIPE(S)
C
C
      DO 300 K = 1 , NINP
      IF(NPP(J,K).EQ.0) GO TO 310
      NPI=NPP(J,K)
      QINP(J,N)=QINP(J,N)+QOUTP(NPI,N)-QB(NPI)
  300 CONTINUE
  310 CONTINUE
  110 CONTINUE
C
C
C    ALLOCATE PEAK INFLOW
C
C
      QPEAK=0.0
      DO 120 N = 1 , NDELTC
      IF(QINP(J,N).GT.QPEAK) GO TO 121
      GO TO 120
  121 CONTINUE
      QPEAK=QINP(J,N)
      TMP=(N-1)*DELTC
  120 CONTINUE
      QMAX(J)=QPEAK+QB(J)
      TMAX(J)=TMP
      IF(IDMN(J).EQ.1) GO TO 132
      IF(IEVAL.EQ.2) GO TO 131
      CALL PSIZE(J)
      GO TO 132
  131 CONTINUE
C
C     CHECK IF INFLOW GREATER THAN CAPACITY
C
C
      D=DP(J)/12.0
      APIP=(22./7.)*D**2.0/4.0
      QPF=APIP*1.486/CP(J)*((D/4.0)**(2./3.))*SP(J)**0.5
      IF(QMAX(J).GT.QPF) GO TO 133
      GO TO 132
  133 CONTINUE
      QEX(J)=QMAX(J)-QPF
      QPAX(J)=QPF
      QALL=QMAX(J)-QB(J)
      DO 135 N = 1 , NDELTC
      IF(QINP(J,N).GT.QALL) QINP(J,N)=QALL
  135 CONTINUE
  132 CONTINUE
      IF(IOPTBP.EQ.0) CALL ROUTP(J)
      IF(IOPTBP.EQ.1) CALL POUTP(J)
  100 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE PSIZE(JJ)
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,PI
     1              ,NDELTC,INS,INTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDOP(200),
     4              IPRTP(200),QB(200),TMAX(200),IOPTBP,QEX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NSPT,
     1              QINTP(5,100),NPTMAX,QALSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),EP1(5,20),NSP1(5),
     1              QQ1(5,20),QOTPP(5,100),OTPL(5,100),QPINT(5)
      COMMON/  MJM/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSE(200),
     1              NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2              QINS(200,100),QOUTS(200,100),YS(50,10),FLOW(50,10),
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4              IPRTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJM1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1              QINT(5,100),NSTMAX,SS(10),CELAY(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),FUNCD(200),
     1              IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2              0),CDMAX(200)
      COMMON/  SUP/ NUSMAX,NINR,NW(200),AW(200),PIMP(200),CNIMP(200),
     1              CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              WCON(200,5),NINT(100),WDEPTH(200,5),WFLCW(200),
     3              FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4              NWS(200,5),SUMQW(200),QP(200,100),IPRTW(200)
      DIMENSION DCONI(20)
      DATA DCONI/ 12.,15.,18.,21.,24.,27.,30.,33.,36.,39.,42.,48.,
     1              54.,60.,66.,72.,78.,84.,90.,96./
C
C
C
C
C     DETERMINE PIPE SIZE
```

```
C
C
      DO 110 I = 1 , 20
      DDES=DCOMI(I)
      APIP=(22./7.)*DDES**2.0/576.0
      QPF=APIP*1.486/CP(JJ)*((DDES/48.)**(2./3.))*SP(JJ)**0.50
      IF(QPF.GE.QMAX(JJ)) GO TO 120
  110 CONTINUE
  120 CONTINUE
      DDP(JJ)=DDES
      QDQF(JJ)=QMAX(JJ)/QPF
      RETURN
      END
C
C
C
      SUBROUTINE ROUTP(KK)
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTH,NI
     1             ,NDELTC,IWS,IPTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),XP(200),SP(200),CP(200),
     1             DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2             NPP(200,5),MINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),QDQF(200),
     4             IPTYP(200),QB(200),TMAX(200),IOPTRP,QEX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QRLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),XP1(5,20),NSP1(5),
     1             QQ1(5,20),QOTPP(5,100),OVFL(5,100),QEMX(5)
      COMMON/  MJN/ NSMAX,MINS,NS(200),NSD(200),XS(200),IDSS(200),
     1             NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3             SPEED2(50,10),IS,AS(50,10),QSS(50,10),NPTS,
     4             IPRTS(200),SDPT(200,100),SFMAX(200),SDMAX(200)
      COMMON/ MJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1             QINF(5,100),NSTMAX,SS(10),CFLAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NORCS(200),
     1             IINC(200),QLIM(200),MIN(200),PIP(200,5),QCAPT(200,10
     2             0),CBMAX(200)
      COMMON/  SUP/ NNSMAX,MINW,NW(200),AW(200),PIMP(200),CNIMP(200),
     1             CMP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2             WCON(200,5),AINT(100),WDEPTH(200,5),WFLOW(200),
     3             FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4             NWS(200,5),SUMQW(200),QW(200,100),IPPTW(200)
      DIMENSION TCP(100),TT(3000),QII(3000),QOO(3000)
      DIMENSION QQF(11),ADAF(11)
      DIMENSION A(10),B(3),X(3),Y(3)
      DATA QQF/0.,.02,.07,.17,.32,.50,.67,.85,.97,.99,1.0/
      DATA ADAF/0.,.1,.2,.3,.4,.5,.6,.7,.8,.9,1.0/
      REAL X1,X2
C
C
C
      CP1=CP(KK)
      SPP=SP(KK)
      XP1=XP(KK)
      DO 5 I = 1 , 100
      TCP(I)=0.0
    5 CONTINUE
      DO 6 I = 1 , 3000
      TT(I)=0.0
      QII(I)=0.0
      QOO(I)=0.0
    6 CONTINUE
      DO 110 I = 2 , NDELTC
      TQP(I)=DELTC*(I-1)
  110 CONTINUE
      IF(IDMN(KK).EQ.1) GO TO 70
      IF(IEVAL.EQ.2) DDP(KK)=DP(KK)
      DDD=DDP(KK)/12.0
      AFULL=(22./7.)*DDD**2.0/4.0
      XO=1.486*SP(KK)**0.5/CP(KK)
      QFULL=XO*AFULL*(DDD/4.0)**(2./3.)
C
C
C     CALCULATE MAX. CELERITY AND DT1
C
C
      QQ=QMAX(KK)
      QQQ=QQ/QFULL
      DO 10 K = 1 , 11
      ANS=QQF(K)
      IF(QOQ.LT.ANS) GO TO 20
   10 CONTINUE
   20 CONTINUE
      IF(K.EQ.1) K=K+1
      IF(K.EQ.11) K=K-1
      X(1)=QQF(K-1)
      X(2)=QQF(K)
      X(3)=QQF(K+1)
      Y(1)=ADAF(K-1)
      Y(2)=ADAF(K)
      Y(3)=ADAF(K+1)
      CALL LESQ(A,B,X,Y,2,3)
      DCF=B(1)+B(2)*QQQ+B(3)*QQQ**2.0
      YF=DCF*DDD
      YFF=0.6*DDD
      IF(YF.LT.YFF) YCELM=YF
      IF(YF.GT.YFF) YCELM=YFF
      PHI=2.0*ARCOS(1.-2.*YCELM/DDD)
      AYS=0.125*DDD**2.0*(PHI-SIN(PHI))
      RYS=0.25*DDD*(1.-SIN(PHI)/PHI)
      BPHI=(SIN(PHI)-PHI*COS(PHI))/(PHI**2.*SIN(PHI/2.0))
      APHI=0.50*DDD*(1.-COS(PHI))/SIN(PHI/2.0)
      X4=RYS**(-0.1333333)
      X3=RYS**(2./3.)
      DQDY=XO*(0.6667*AYS*X4*BPHI+X3*APHI)
      CEL=DQDY/APHI
      DT1=(XP1/CEL)/60.0
      NDT1=TTOTC/DT1+1
      DT1=TTOTC/(NDT1-1)
      DT1S=DT1*60.0
C
C
C     INTERPOLATE FLOWS AT DT1
C
C
```

```
      QII(1)=QINP(KK,1)
      DC 120 I = 2 , NDT1
      TT(I)=DT1*(I-1)
      TTT=TT(I)
      DO 130 K = 2 , NDELTC
      ABS=TQP(K)
      IF(TTT.LE.ABS) GO TO 131
  130 CONTINUE
  131 CONTINUE
      X1=TQP(K-1)
      X2=TQP(K)
      Y1=QINP(KK,K-1)
      Y2=QINP(KK,K)
      CALL LIN(X1,X2,Y1,Y2,TTT,Y0)
      QII(I)=Y0
  120 CONTINUE
C
C
C      DONT MISS THE PEAK
C
C
      THP=THAX(KK)
      QHP=QMAX(KK)-QB(KK)
      DO 12 I = 2 , NDT1
      TT1=TT(I-1)
      TT2=TT(I)
      IF(TT1.EQ.THP.OR.TT2.EQ.THP) GO TO 14
      IF(TT1.LT.THP.AND.TT2.GT.THP) GO TO 13
   12 CONTINUE
   13 CONTINUE
      TDIFF1=THP-TT1
      TDIFF2=TT2-THP
      IF(TDIFF1.GT.TDIFF2) QII(I)=QHP
      IF(TDIFF2.GT.TDIFF1) QII(I-1)=QHP
   14 CONTINUE
C
C
C      ROUTING
C
C
      QB1=QB(KK)+.01
      DO 45 N = 1 , NDT1
      IF(N.EQ.1) QOO(N)=QB(KK)
      IF(N.EQ.1) GO TO 45
      I1=QII(N-1)+QB(KK)
      O1=QOO(N-1)
      I2=QII(N)+QB(KK)
      IF(O1.LT.QB(KK)) O1=QB(KK)
      IF(O1.LE.0.0) O1=.01
      IF(I1.LE.QB1.AND.I2.LE.QB1.AND.O1.LE.QB1) GO TO 39
      CALL COEF(DDD,CP1,SPP,IP1,QFULL,I1,CEL,ALPHA)
      PHI1=ALPHA
      ALPH1=CEL*DT1S/XP1
      CALL COEF(DDD,CP1,SPP,XP1,QFULL,I2,CEL,ALPHA)
      PHI2=ALPHA
      ALPH2=CEL*DT1S/XP1
      CALL COEF(DDD,CP1,SPP,XP1,QFULL,O1,CEL,ALPHA)
      PHI3=ALPHA
      ALPH3=CEL*DT1S/XP1
      PHI=(PHI1+PHI2+PHI3)/3.0
      IF(PHI.GT.1.0) PHI=1.0
      IF(PHI.LT.0.0) PHI=0.0
      ALPH=(ALPH1+ALPH2+ALPH3)/3.0
      ALL=1.0+ALPH+PHI
      C1=(1.0+ALPH-PHI)/ALL
      C2=(-1.0+ALPH+PHI)/ALL
      C3=(1.0-ALPH+PHI)/ALL
      O2=C1*I1+C2*I2+C3*O1
      IF(O2.LT.QB(KK)) O2=QB(KK)
      GO TO 38
   39 CONTINUE
      O2=QB(KK)
   38 CONTINUE
      QOO(N)=O2
      IF(QOO(N).LE.QB(KK)) QOO(N)=QB(KK)
   45 CONTINUE
C
C
C      ALLOCATE PEAK OUTFLOW AND TIME
C
C
      QPEAK1=0.0
      DO 51 I = 1 , NDT1
      IF(QOO(I).GT.QPEAK1) GO TO 55
      GO TO 51
   55 CONTINUE
      QPEAK1=QOO(I)
      TMP1=TT(I)
   51 CONTINUE
C
C
C      INTERPOLATE FLOWS BACK TO DELTC
C
C
   70 CONTINUE
      DO 80 N = 1 , NDELTC
      IF(IDAM(KK).EQ.1) GO TO 81
      TINTR=TQP(N)
      DO 90 NN = 2 , NDT1
      ABS=TT(NN)
      IF(TINTR.LT.ABS) GO TO 95
   90 CONTINUE
   95 CONTINUE
      X1=TT(NN-1)
      X2=TT(NN)
      Y1=QOO(NN-1)
      Y2=QOO(NN)
      CALL LIN(X1,X2,Y1,Y2,TINTR,Y0)
      QOUTP(KK,N)=Y0
      IF(QOUTP(KK,N).LE.QB(KK)) QOUTP(KK,N)=QB(KK)
      GO TO 80
   81 CONTINUE
      QOUTP(KK,N)=QINP(KK,N)+QB(KK)
   80 CONTINUE
C
C
C      DONT MISS THE PEAK
C
C
      IF(IDAM(KK).EQ.1) GO TO 64
```

```
      DO 62 I = 2 , NDELTC
      TT1=TOP(I-1)
      TT2=TOP(I)
      IF(TT1.EQ.TMP1.OR.TT2.EQ.TMP1) GO TO 64
      IF(TT1.LT.TMP1.AND.TT2.GT.TMP1) GO TO 63
   62 CONTINUE
   63 CONTINUE
      TDIFF1=TMP1-TT1
      TDIFF2=TT2-TMP1
      IF(TDIFF1.GT.TDIFF2) QOUTP(KK,I)=QPEAK1
      IF(TDIFF2.GT.TDIFF1) QOUTP(KK,I-1)=QPEAK1
   64 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE COEF(DDD,CP1,SPP,XP1,QFULL,QQ,CEL,ALPHA)
C
C
C
      COMMON/  RIN/ NPMAX,NP(200),NPD(200),XP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDNN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4              IPNTP(200),QB(200),THAX(200),IOPTPP,QEX(200)
      DIMENSION QQP(11),ADAP(11)
      DIMENSION A(10),B(3),X(3),Y(3)
      DATA QQP/0.,.02,.07,.17,.32,.50,.67,.85,.97,.99,1.0/
      DATA ADAP/0.,.1,.2,.3,.4,.5,.6,.7,.8,.9,1.0/
C
C
C
C
C
C     CALCULATE DEPTH OF FLOW
C
C
      QQQ=QQ/QFULL
      DO 10 K = 1 , 11
      ABS=QQP(K)
      IF(QQQ.LE.ABS) GO TO 20
   10 CONTINUE
   20 CONTINUE
      IF(K.EQ.1) K=K+1
      IF(K.EQ.11) K=K-1
      X(1)=QQP(K-1)
      X(2)=QQP(K)
      X(3)=QQP(K+1)
      Y(1)=ADAP(K-1)
      Y(2)=ADAP(K)
      Y(3)=ADAP(K+1)
      CALL LESQ(A,B,X,Y,2,3)
      DDF=B(1)+B(2)*QQQ+B(3)*QQQ**2.0
      YP=DDF*DDD
      YPP=0.2*DDD
      IF(YP.LE.0.0) YP=YPP
C
C
C     CALCULATE ALPHA
C
C
      YPDDD=YP/DDD
      IF(YPDDD.GT.0.95) YPDDD=0.95
      PHI=2.0*ACOS(1.-2.*YPDDD)
      X1=1.486*SPP**0.5/CP1
      AXS=0.125*DDD**2.0*(PHI-SIN(PHI))
      RIS=0.25*DDD*(1.-SIN(PHI)/PHI)
      RPHI=(SIN(PHI)-PHI*COS(PHI))/(PHI**2.*SIN(PHI/2.0))
      APHI=0.50*DDD*(1.-COS(PHI))/SIN(PHI/2.0)
      X2=RIS**(-0.1333333)
      X3=RIS**(2./3.)
      DQDY=X1*(0.6667*AXS*X2*RPHI+X3*APHI)
      CEL=DQDY/APHI
      XK=XP1/CEL
      BW=DDD*SIN(PHI/2.0)
      ALPHA=XK*QQ/(SPP*XP1**2.*BW)
      RETURN
      END
C
C
C
      SUBROUTINE MAJOR1(N2,SI,ICURB,CNS,SO,SPOW,CNPOW,THAX)
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ INVAL,TINCC,DELTC,DELTCS,DELTC2,TTOTC,DELTN,NI
     1              ,NDELTC,INS,INTS
      COMMON/  RIN/ NPMAX,NP(200),NPD(200),XP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDNN(200),NPOC(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4              IPNTP(200),QB(200),THAX(200),IOPTPP,QEX(200)
      COMMON/ RIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1              QINPP(5,100),NPTMAX,QNLSP(5)
      COMMON/ RIN2/ IOPTSP(5),SPMX(5),SP1(5,20),PP1(5,20),NSP1(5),
     1              QQ1(5,20),QOTPP(5,100),OTPL(5,100),OPINX(5)
      COMMON/  RJN/ NSMAX,NINS,NS(200),NSD(200),XS(200),IDSS(200),
     1              NOS,NOSS,IDNJ(200),NSOO(5),NSS(200,5),NSO(200),
     2              QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4              IPNTS(200),SDPT(200,100),SPMAX(200),SDPMX(200)
      COMMON/ RJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTF(5,5),NNST,
     1              QINF(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1              IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2              0),CBMAX(200)
      COMMON/  SUP/ NUSMAX,NINU,NU(200),AW(200),PIMP(200),CNIMP(200),
     1              CNP(200),S(200),WLAT(200),DTIMP(200),DETP(200),
     2              NCON(200,5),NINT(100),NDEPTH(200,5),NFLOW(200),
     3              THAX(200),PNIN(200),DTC(200),DET(200,5),NONS,
     4              NUS(200,5),SUMQU(200),QP(200,100),IPPTU(200)
      REAL INCR
C
C
C
      J=IS
      X=1./SI
      SS(J)=SO
```

```
      Z1=1./SNOW
      TCR=W2/Z
      INCR=0.1
      DO 10 I = 2 , 50
      TS(I,J)=TS(I-1,J)+INCR
      IF(TS(I,J).GT.TMAX) GO TO 60
      IF(TS(I,J)-TCR) 20,20,30
   20 CONTINUE
      AS(I,J)=Z*TS(I,J)**2.0/2.0
      FLOW(I,J)=0.557/CNS*SQRT(SO)*Z*TS(I,J)**(8./3.)
      SPEED2(I,J)=TS(I,J)*Z
      CELAR(I,J)=1./SPEED2(I,J)
      CELAR(I,J)=CELAR(I,J)*(0.557/CNS)*SQRT(SO)*(8./3.)
      CELAR(I,J)=CELAR(I,J)*Z*TS(I,J)**(5./3.)
      GO TO 10
   30 CONTINUE
      IF(TS(I,J)-TCURB) 40,40,50
   40 CONTINUE
      TC=TS(I,J)-TCR
      AS(I,J)=Z*TCR**2.0/2.0/2.0+TC*W2
      FLOW(I,J)=0.557/CNS*SQRT(SO)*Z*(TS(I,J)**(8./3.)-TC**(8./3.))
      SPEED2(I,J)=W2
      CELAR(I,J)=(8./3.)*0.557*SQRT(SO)*Z/(SPEED2(I,J)*CNS)
      CELAR(I,J)=CELAR(I,J)*(TS(I,J)**(5./3.)-TC**(5./3.))
      GO TO 10
   50 CONTINUE
      TC=TS(I,J)-TCR
      TROW=TS(I,J)-TCURB
      AS(I,J)=Z*TCR**2.0/2.0+W2*TC+Z1*TROW**2.0/2.0
      FLOW(I,J)=0.557/CNS*SQRT(SO)*Z*(TS(I,J)**(8./3.)-TC**(8./3.))
      FLOW(I,J)=FLOW(I,J)+0.557/CNROW*SQRT(SO)*Z1*TROW**(8./3.)
      SPEED2(I,J)=W2+TROW*Z1
      CELAR(I,J)=0.557*SQRT(SO)/SPEED2(I,J)
      XXX=(8./3.)*(Z/CNS)*(TS(I,J)**(5./3.)-TC**(5./3.))
      YYY=(Z1*(8./3.)/CNROW)*TROW**(5./3.)
      CELAR(I,J)=CELAR(I,J)*(XXX+YYY)
   10 CONTINUE
   60 CONTINUE
      NPTS=I-1
      WRITE(NPRINT,2010)
 2010 FORMAT(//,T44,'RATING CURVE',////,
     1      T35,'DEPTH',T46,'FLOW',T55,'SPREAD',/,T36,'(IN)',T46,
     1      '(CFS)',T56,'(FT)',//)
      DO 70 I = 1 , NPTS
      TS(I,J)=TS(I,J)*12.0
      FLOW(I,J)=2.0*FLOW(I,J)
      AS(I,J)= 2.0*AS(I,J)
      WRITE(NPRINT,2020) TS(I,J),FLOW(I,J),SPEED2(I,J)
   70 CONTINUE
 2020 FORMAT(T30,3F10.2)
    5 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE BAJS1
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,NI
     1              ,NDELTC,INS,INTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),XP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4              IPRTP(200),QB(200),TMAX(200),IOPTPP,QEX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1              QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),NP1(5,20),NSP1(5),
     1              QQ1(5,20),QOTPP(5,100),OTPL(5,100),ORIMX(5)
      COMMON/  MJN/ NSMAX,NINS,NS(200),NSD(200),XS(200),IDSS(200),
     1              NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2              QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3              SPEED2(50,10),ZS,AS(50,10),QQSS(50,10),NPTS,
     4              IPRTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1              QINF(5,100),NSTMAX,SS(10),CELAR(50,10)
      COMMON/  INL/ NINT,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1              IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2              0),CBMAX(200)
      COMMON/  SUP/ NNSMAX,NINW,NW(200),AW(200),PIMP(200),CHISP(200),
     1              CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              WCON(200,5),RINT(100),WDEPTH(200,5),WFLOW(200),
     3              FMAX(200),FMIN(200),DEC(200),DEY(200,5),NONS,
     4              NNS(200,5),SUMQW(200),QR(200,100),IPPTW(200)
C
C
C
      DO 10 I = 1 , NNST
      DO 20 J = 1 , NDELTC
      SUMINF=0.0
      DO 30 K = 1 , NINS
      KK=NNSTS(I,K)
      IF(KK.NE.0) SUMINF=SUMINF+QOUTS(KK,J)
   30 CONTINUE
      QINF(I,J)=SUMINF
   20 CONTINUE
   10 CONTINUE
      CALL BAJS2
      RETURN
      END
C
C
C
      SUBROUTINE BAJS2
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,NI
     1              ,NDELTC,INS,INTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),XP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4              IPRTP(200),QB(200),TMAX(200),IOPTPP,QEX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1              QINPP(5,100),NPTMAX,QBLSP(5)
```

```
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),RP1(5,20),NSP1(5),       DDM2280
     1              QQ1(5,20),QOTPP(5,100),OTPL(5,100),OP1MX(5)         DDM2281
      COMMON/  MJR/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),       DDM2282
     1              NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),     DDM2283
     2              QINS(200,100),QOOTS(200,100),TS(50,10),FLOW(50,10), DDM2284
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,        DDM2285
     4              IPRTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)      DDM2286
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,   DDM2287
     1              QINP(5,100),NSTMAX,SS(10),CELAP(50,10)              DDM2288
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUHCB(200),   DDM2289
     1              IINC(200),QLIN(200),NIN(200),NIP(200,5),QCAPT(200,10DDM2290
     2              0),CBMAX(200)                                       DDM2291
      COMMON/  SUP/ NUSMAX,NINW,NW(200),AW(200),PINP(200),CNINP(200),    DDM2292
     1              CNP(200),S(200),WLAT(200),DETINP(200),DETP(200),    DDM2293
     2              WCON(200,5),NINT(100),NDEPTH(200,5),NFLOW(200),     DDM2294
     3              FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,       DDM2295
     4              NWS(200,5),SUMQW(200),QP(200,100),IPPTW(200)        DDM2296
      COMMON / TITL/ ITITL(40)                                           DDM2297
      REAL GRA(6)                                                        DDM2298
      INTEGER ILINE(50)                                                  DDM2299
      DATA IIG /'I'/                                                     DDM2300
C                                                                        DDM2301
C                                                                        DDM2302
C                                                                        DDM2303
      NCD=NDELTC-1                                                       DDM2304
      DO 10 I = 1 , NNST                                                 DDM2305
      STSS=0.0                                                           DDM2306
      CALL MAXMN(AMAX,AMIN,QINP,I,NDELTC)                                DDM2307
      T1=-DELTC                                                          DDM2308
      INSUB=1                                                            DDM2309
      IVPT=1                                                             DDM2310
      SCALE=(AMAX-AMIN)/50.0                                             DDM2311
      WRITE(NPRINT,2000) (ITITL(KK),KK=1,40)                             DDM2312
      WRITE(NPRINT,2001)                                                 DDM2313
 2001 FORMAT(//,5X,'RESULTS OF SIMULATION FOR MAJOR SYSTEM STORAGE')     DDM2314
      WRITE(NPRINT,2002) IDSTS(I)                                        DDM2315
      WRITE(NPRINT,2006)                                                 DDM2316
 2006 FORMAT(T10,'TIME',T20,'FLOW',T70,'FLOW (CFS)',/,                   DDM2317
     1       T10,'(MIN)',T20,'(CFS)',/)                                  DDM2318
      CALL PSCAL(SCALE,GRA,NPRINT,AMAX,AMIN)                             DDM2319
      DO 50 K = 1 , NDELTC                                               DDM2320
      T1=T1+DELTC                                                        DDM2321
      XVAL=QINP(I,K)                                                     DDM2322
      CALL LINE(ILINE,XVAL,SCALE,IVPT,INSUB,K)                           DDM2323
      WRITE(NPRINT,2007) T1,QINP(I,K),IIG,(ILINE(L),L=1,50),IIG          DDM2324
   50 CONTINUE                                                           DDM2325
 2007 FORMAT(T10,F6.2,T20,F6.2,T51,A1,50A1,A1)                           DDM2326
      WRITE(NPRINT,2033)                                                 DDM2327
      WRITE(NPRINT,2032) (GRA(KM),KM=1,6)                                DDM2328
 2032 FORMAT(50X,5(E8.2,2X),1X,E8.2)                                     DDM2329
 2033 FORMAT(50X,5(1HI,9H---------),2H-I)                                DDM2330
 2000 FORMAT(1H1,//,5X,20A4,/,5X,20A4,/)                                 DDM2331
      DO 20 J = 1 , NCD                                                  DDM2332
      STSS=STSS+0.5*(QINP(I,J)+QINP(I,J+1))*DELTCS                       DDM2333
   20 CONTINUE                                                           DDM2334
      STSSA=STSS/43560.0                                                 DDM2335
      WRITE(NPRINT,2005) STSSA                                           DDM2336
 2005 FORMAT(//,5X,'REQUIRED DETENTION VOLUME  ',F10.2,' AC.FT')         DDM2337
   10 CONTINUE                                                           DDM2338
 2002 FORMAT(/,5X,'STORAGE UNIT NUMBER  ',I5)                            DDM2339
      RETURN                                                             DDM2340
      END                                                                DDM2341
C                                                                        DDM2342
C                                                                        DDM2343
C                                                                        DDM2344
      SUBROUTINE MNNS1                                                   DDM2345
C                                                                        DDM2346
C                                                                        DDM2347
C                                                                        DDM2348
      COMMON/  TAP/ NREAD,NPRINT                                         DDM2349
      COMMON/  CON/ ITVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,RI       DDM2350
     1              ,NDELTC,INS,IRTS                                     DDM2351
      COMMON/  RIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),      DDM2352
     1              DP(200),NOP,NOPP,IDRN(200),NPOO(5),NPO(200),        DDM2353
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)           DDM2354
     3              ,QOUTP(200,100),DDP(200),QMAX(200),QDOP(200),       DDM2355
     4              IPRTP(200),QB(200),TMAX(200),IOPTPP,OPX(200)        DDM2356
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,   DDM2357
     1              QINPP(5,100),NPTMAX,QBLSP(5)                        DDM2358
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),RP1(5,20),NSP1(5),       DDM2359
     1              QQ1(5,20),QOTPP(5,100),OTPL(5,100),OP1MX(5)         DDM2360
      COMMON/  MJR/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),       DDM2361
     1              NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),     DDM2362
     2              QINS(200,100),QOOTS(200,100),TS(50,10),FLOW(50,10), DDM2363
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,        DDM2364
     4              IPRTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)      DDM2365
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,   DDM2366
     1              QINP(5,100),NSTMAX,SS(10),CELAP(50,10)              DDM2367
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUHCB(200),   DDM2368
     1              IINC(200),QLIN(200),NIN(200),NIP(200,5),QCAPT(200,10DDM2369
     2              0),CBMAX(200)                                       DDM2370
      COMMON/  SUP/ NUSMAX,NINW,NW(200),AW(200),PINP(200),CNINP(200),    DDM2371
     1              CNP(200),S(200),WLAT(200),DETINP(200),DETP(200),    DDM2372
     2              WCON(200,5),NINT(100),NDEPTH(200,5),NFLOW(200),     DDM2373
     3              FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,       DDM2374
     4              NWS(200,5),SUMQW(200),QP(200,100),IPPTW(200)        DDM2375
C                                                                        DDM2376
C                                                                        DDM2377
C                                                                        DDM2378
      DO 10 I = 1 , NNPT                                                 DDM2379
      DO 20 J = 1 , NDELTC                                               DDM2380
      SUMINP=0.0                                                         DDM2381
      DO 30 K = 1 , NINP                                                 DDM2382
      KK=NNSTP(I,K)                                                      DDM2383
      IF(KK.GT.0) SUMINP=SUMINP+QOOTP(KK,J)                              DDM2384
   30 CONTINUE                                                           DDM2385
      QINPP(I,J)=SUMINP                                                  DDM2386
   20 CONTINUE                                                           DDM2387
   10 CONTINUE                                                           DDM2388
      CALL MNNS2                                                         DDM2389
      RETURN                                                             DDM2390
      END                                                                DDM2391
C                                                                        DDM2392
C                                                                        DDM2393
C                                                                        DDM2394
      SUBROUTINE MNNS2                                                   DDM2395
C                                                                        DDM2396
C                                                                        DDM2397
C                                                                        DDM2398
      COMMON/  TAP/ NREAD,NPRINT                                         DDM2399
```

```
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,NI
     1        ,NDELTC,INS,IPTS
      COMMON/  NIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1        DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2        NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3        ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4        IPNTP(200),QB(200),TMAX(200),IOPTPP,OPX(200)
      COMMON/ NIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1        QINPP(5,100),NPTMAX,QPLSP(5)
      COMMON/ NIN2/ IOPTSP(5),SPMX(5),SP1(5,20),PP1(5,20),NSP1(5),
     1        QQ1(5,20),QOTPP(5,100),OVFL(5,100),OPTPX(5)
      COMMON/  NJN/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1        NOS,NOSS,IDNJ(200),NSOO(5),NSS(200,5),NSO(200),
     2        QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3        SPEED2(50,10),IS,AS(50,10),QQSS(50,10),NPTS,
     4        IPNTS(200),SDPT(200,100),SPMAX(200),SOMAX(200)
      COMMON/ NJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1        QINP(5,100),NSTMAX
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1        IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2        0),CBMAX(200)
      COMMON/  SUP/ NNSMAX,NINN,NN(200),AN(200),FIMP(200),CSIMP(200),
     1        CNP(200),S(200),WLAT(200),DETIMP(200),DTP(200),
     2        NCON(200,5),NINT(100),NDEPTH(200,5),NFLOW(200),
     3        FMAX(200),FMIN(200),DEC(200),DET(200,5),NONS,
     4        NNS(200,5),SUMQN(200),QR(200,100),IPNTN(200)
      COMMON / TITL/ ITITL(40)
      DIMENSION A(10),B(10),X(3),Y(3)
      REAL GRA(6)
      INTEGER ILINE(50)
      DATA IIG /'I'/
C
C
C
      NDD=NDELTC-1
      DO 10 I = 1 , NNPT
      STPP=0.0
      WRITE(NPRINT,2000) (ITITL(KZ),KZ=1,40)
 2000 FORMAT(1H1,//,5X,20A4,/,5X,20A4,/)
      WRITE(NPRINT,2001)
 2001 FORMAT(///,5X,'RESULTS OF SIMULATION FOR MINOR SYSTEM STORAGE',//)
      WRITE(NPRINT,2002) IDSTP(I)
 2002 FORMAT(5X,'STORAGE UNIT NO. ',I5,//)
      III=IOPTSP(I)
      GO TO (30,40,50),III
   30 CONTINUE
      DO 29 J = 1 , NDD
      QNET1=QINPP(I,J)-QPLSP(I)
      QNET2=QINPP(I,J+1)-QPLSP(I)
      IF(QNET1.LE.0.0.AND.QNET2.LE.0.0) GO TO 20
      IF(QNET1.LE.0.0.AND.QNET2.GE.0.0) GO TO 24
      IF(QNET1.GE.0.0.AND.QNET2.LE.0.0) GO TO 25
      IF(QNET1.GT.0.0.AND.QNET2.GT.0.) STPP=STPP+.5*(QNET1+QNET2)*DELTCS
      GO TO 29
   24 CONTINUE
      IF(QNET1.EQ.0.0) STPP=STPP+0.5*QNET2*DELTCS
      IF(QNET1.NE.0.0.AND.QNET2.NE.0.0) GO TO 26
      GO TO 29
   26 CONTINUE
      TT2=(DELTCS*QNET2)/(-QNET1+QNET2)
      STPP=STPP+0.5*QNET2*TT2
      GO TO 29
   25 CONTINUE
      IF(QNET2.EQ.0.0) STPP=STPP+0.5*QNET1*DELTCS
      IF(QNET1.NE.0.0.AND.QNET2.NE.0.0) GO TO 28
      GO TO 29
   28 CONTINUE
      TT1=(QNET1*DELTCS)/(QNET1-QNET2)
      STPP=STPP+0.0*QNET1*TT1
   29 CONTINUE
   20 CONTINUE
      WRITE(NPRINT,2003)
 2003 FORMAT(//,5X,'INFLOW HYDROGRAPH INTO STORAGE')
   11 CALL MAXMN(AMAX,AMIN,QINPP,I,NDELTC)
      T1=-DELTC
      INSUB=1
      IVPT=1
      SCALE=(AMAX-AMIN)/50.0
      WRITE(NPRINT,2009)
 2009 FORMAT(T10,'TIME',T20,'FLOW',T70,'FLOW (CFS)',/,
     1       T10,'(MIN)',T20,'(CFS)',/)
      CALL PSCAL(SCALE,GRA,NPRINT,AMAX,AMIN)
      DO 13 K = 1 , NDELTC
      T1=T1+DELTC
      XVAL=QINPP(I,K)
      CALL LINE(ILINE,XVAL,SCALE,IVPT,INSUM,K)
      WRITE(NPRINT,2010) T1,QINPP(I,K),IIG,(ILINE(L),L=1,50),IIG
   13 CONTINUE
 2010 FORMAT(T10,F6.2,T20,F6.2,T51,A1,50A1,A1)
      WRITE(NPRINT,2033)
      WRITE(NPRINT,2032) (GRA(KM),KM=1,6)
      IF(III.EQ.2) GO TO 12
 2032 FORMAT(50X,5(E8.2,2X),1X,E8.2)
 2033 FORMAT(50X,5(1HI,9H---------),2H-I)
      WRITE(NPRINT,2005) QPLSP(I),STPP
 2005 FORMAT(//,5X,'MAXIMUM RELEASE RATE    ',F6.2,//,5X,'REQUIRED STORAG
     1E VOLUME    ',F10.2,'  CU.FT')
      GO TO 10
   40 CONTINUE
      OFVOL=0.0
      OVPP=0.0
      DO 21 J = 1 , NDD
      QNET1=QINPP(I,J)-QPLSP(I)
      QNET2=QINPP(I,J+1)-QPLSP(I)
      IF(QNET1.LE.0.0.AND.QNET2.LE.0.0) GO TO 21
      IF(QNET1.LE.0.0.AND.QNET2.GE.0.0) GO TO 41
      IF(QNET1.GE.0.0.AND.QNET2.LE.0.0) GO TO 42
      IF(QNET1.GT.0.0.AND.QNET2.GT.0.) STPP=STPP+.5*(QNET1+QNET2)*DELTCS
      GO TO 49
   41 CONTINUE
      IF(QNET1.EQ.0.0) STPP=STPP+0.5*QNET2*DELTCS
      IF(QNET1.NE.0.0.AND.QNET2.NE.0.0) GO TO 43
      GO TO 49
   43 CONTINUE
      TT2=(DELTCS*QNET2)/(-QNET1+QNET2)
      STPP=STPP+0.5*QNET2*TT2
      GO TO 49
   42 CONTINUE
      IF(QNET2.EQ.0.0) STPP=STPP+0.5*DELTCS*QNET1
      IF(QNET1.NE.0.0.AND.QNET2.NE.0.0) GO TO 44
```

```
      GO TO 49
   48 CONTINUE
      TT1=(QNET1*DELTCS)/(QNET1-QNET2)
      STPP=STPP+0.5*QNET1*TT1
   49 CONTINUE
      IF(STPP.GE.SPMX(I)) GO TO 22
   21 CONTINUE
      GO TO 27
   22 CONTINUE
      OVPP=0.0
      SDIFF=STPP-SPMX(I)
      OFVOL=SDIFF
      JJ=J+1
      DO 23 K = JJ , NDD
      OVFL1=QINFP(I,K)-QRLSP(I)
      IF(OVFL1.LE.0.0) OVFL1=0.0
      IF(OVFL1.GT.OVPP) OVPP=OVFL1
      OVFL2=QINFP(I,K+1)-QRLSP(I)
      IF(OVFL2.LE.0.0) OVFL2=0.0
      IF(OVFL2.GT.OVPP) OVPP=OVFL2
      OFVOL=OFVOL+0.50*(OVFL1+OVFL2)*DELTCS
   23 CONTINUE
   27 CONTINUE
      GO TO 11
   12 WRITE(NPRINT,2006) SPMX(I),QRLSP(I),OVPP,OFVOL
 2006 FORMAT(//,5X,'STORAGE VOLUME     ',F12.2,'  CU.FT.',//,
     1          5X,'MAXIMUM RELEASE    ',F12.2,'  CFS',//,
     2          5X,'PEAK OVERFLOW      ',F12.2,'  CFS',//,
     3          5X,'OVERFLOW VOLUME    ',F12.2,'  CU.FT')
      GO TO 10
   50 CONTINUE
      ID1=0
C
C
C     ESTABLISH Q VS. Q+2S/DT RELATIONSHIP
C
C
      NSP11=NSP1(I)
      DO 60 K = 1 , NSP11
      QQ1(I,K)=PP1(I,K)+2.0*SP1(I,K)/DELTCS
   60 CONTINUE
C
C
C     ROUTING
C
C
      TS1=0.0
      DO 70 N = 1 , NDELTC
      IF(TS1.GE.SPMX(I).AND.ID1.EQ.1) GO TO 71
      GO TO 72
   71 CONTINUE
      QOTFP(I,N)=QRIMX(I)
      OVFL(I,N)=QINFP(I,N)-QOTFP(I,N)
      IF(OVFL(I,N).LE.0.0) GO TO 100
      GO TO 70
   72 CONTINUE
      IF(N.EQ.1) GO TO 75
      QPP1=QINFP(I,N-1)+QINFP(I,N)-QPPR1
      GO TO 76
   75 CONTINUE
      QPP1=QINFP(I,N)
   76 CONTINUE
      IF(QPP1.LE.0.001) GO TO 92
      DO 90 K = 1 , NSP11
      ANS=QQ1(I,K)
      IF(QPP1.LT.ANS) GO TO 91
   90 CONTINUE
   91 CONTINUE
      IF(K.EQ.1) K=K+1
      IF(K.EQ.NSP11) K=K-1
      X(1)=QQ1(I,K-1)
      X(2)=QQ1(I,K)
      X(3)=QQ1(I,K+1)
      Y(1)=PP1(I,K-1)
      Y(2)=PP1(I,K)
      Y(3)=PP1(I,K+1)
      CALL LESQ(A,B,X,Y,2,3)
      QOTFP(I,N)=B(1)+B(2)*QPP1+B(3)*QPP1**2.0
      TSDDT=QPP1-QOTFP(I,N)
      QPPR1=QOTFP(I,N)-TSDDT
      TS1=TSDDT*DELTCS/2.0
      IF(TS1.GE.SPMX(I)) GO TO 501
      GO TO 502
  501 CONTINUE
      TS1L=TS1
      TS1=SPMX(I)
      QOTFP(I,N)=PP1(I,NSP11)
      QPIMX(I)=QOTFP(I,N)
      ID1=1
      TSDDT=2.0*TS1/DELTCS
      QPPR1=QOTFP(I,N)-TSDDT
  502 CONTINUE
      GO TO 70
   92 CONTINUE
      QOTFP(I,N)=0.0
      QPPR1=0.0
      GO TO 70
  100 CONTINUE
      OVFL(I,N)=0.0
      ID1=0
   70 CONTINUE
      T=-DELTC
      CALL MAXMN(AMAX,AMIN,QINFP,I,NDELTC)
      AMAX2=AMAX
      AMIN2=AMIN
      CALL MAXMN(AMAX,AMIN,QOTFP,I,NDELTC)
      AMAX3=AMAX
      AMIN3=AMIN
      CALL MAXMN(AMAX,AMIN,OVFL,I,NDELTC)
      AMAX=AMAX1(AMAX2,AMAX3,AMAX)
      AMIN=AMIN1(AMIN2,AMIN3,AMIN)
      IVPT=3
      SCALE=(AMAX-AMIN)/50.0
      WRITE(NPRINT,2040)
 2040 FORMAT(T10,'TIME',T20,'INFLOW',T30,'OUTFLOW',T40,'OVERFLOW',
     1       T70,' * INFLOW (CFS)',/,
     2       T10,'(MIN)',T20,'(CFS)',T30,'(CFS)',T40,'(CFS)',
     3       T70,' + OUTFLOW (CFS)',/,T70,' X OVERFLOW (CFS)',/)
      CALL PSCAL(SCALE,GRA,NPRINT,AMAX,AMIN)
      DO 101 N = 1 , NDELTC
```

```
      T=T+DELTC
      INSUB=1
      IVAL=QINFP(I,N)
      CALL LINE(ILINE,IVAL,SCALE,IVPT,INSUB,N)
      INSUB=2
      IVAL=QOTFP(I,N)
      CALL LINE(ILINE,IVAL,SCALE,IVPT,INSUB,N)
      INSUB=3
      IVAL=OVFL(I,N)
      CALL LINE(ILINE,IVAL,SCALE,IVPT,INSUB,N)
      WRITE(NPRINT,2011)T,QINFP(I,N),QOTFP(I,N),OVFL(I,N)
     1                 ,IIG,(ILINE(L),L=1,50),IIG
 2011 FORMAT(T10,F6.2,T20,F6.2,T30,F6.2,T40,F6.2,T51,A1,50A1,A1)
  101 CONTINUE
      WRITE(NPRINT,2033)
      WRITE(NPRINT,2032)  (GPA(KN),KN=1,6)
C
C
C     COMPUTE OVERFLOW VOLUME
C
C
      OFVOL=TSIL-SPMX(I)
      DO 102 N = 1 , NDD
      OFVOL=OFVOL+0.50*(OVFL(I,N)+OVFL(I,N+1))*DELTCS
  102 CONTINUE
      WRITE(NPRINT,2008) OFVOL
 2008 FORMAT(///,5X,'OVERFLOW VOLUME      ',F12.2,'  CU.FT.')
   10 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE DPRINT
C
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ INVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTN,NI
     1              ,NDELTC,INS,INTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDNN(200),NPOC(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QCOTP(200,100),ODP(200),QMAX(200),QDOP(200),
     4              IPRTP(200),QB(200),TMAX(200),IOPTNP,OZI(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1              QINPP(5,100),NPTMAX,QRLSP(5)
      COMMON/ MIN2/ IOPTSP(5),SPMX(5),SP1(5,20),RP1(5,20),NSP1(5),
     1              QQ1(5,20),QOTFP(5,100),OVFL(5,100),OPTMX(5)
      COMMON/  MJN/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1              NOS,NOSS,IDNJ(200),NSOO(5),NSS(200,5),NSO(200),
     2              QINS(200,100),QOUTS(200,100),IS(50,10),FLOW(50,10),
     3              SPFED2(50,10),IS,AS(50,10),QQSS(50,10),NPTS,
     4              IPPTS(200),SDPT(200,100),SPEAK(200),SONAX(200)
      COMMON/ MJN1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1              QINP(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),OCAP(10,50),NURCB(200),
     1              IINC(200),QLIN(200),NIN(200),NIP(200,5),QCAPT(200,10
     2              0),CBMAX(200)
      COMMON/  SUP/ NUSMAX,NINW,NW(200),AW(200),PIMP(200),CNIMP(200),
     1              CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2              WCON(200,5),PINT(100),WDEPTH(200,5),WFLOW(200),
     3              FMAX(200),FMIN(200),DEC(200),DPT(200,5),NOWS,
     4              NWS(200,5),SUMQW(200),QB(200,100),IPPTW(200)
      COMMON / TITL/ ITITL(40)
      DATA IIG,IPO /'I','+'/
      REAL GPA(6)
      INTEGER ILINE(50)
C
C
C
C
C     THIS SUBROUTINE PRINTS DETAILED SIMULATION RESULTS AT SELECTED
C     LOCATIONS
C
C
C
C
C     SURFACE RUNOFF
C
C
      DO 100 I = 1 , NOWS
      IF(IPPTW(I).GT.0) GO TO 110
      GO TO 100
  110 CONTINUE
      CALL MAXMIN(AMAX,AMIN,QB,I,NDELTC)
      INSUB=1
      SCALE=(AMAX-AMIN)/50.0
      IVPT=1
      WRITE(NPRINT,2000)  (ITITL(KZ),KZ=1,40)
      WRITE(NPRINT,2001)
      WRITE(NPRINT,2034) NW(I)
 2034 FORMAT(T11,'SUBAREA NO.',I5,//)
      WRITE(NPRINT,2005)
 2005 FORMAT(T10,'TIME',T23,'FLOW',T70,'FLOW(CFS)',/,
     1       T10,'(MIN)',T23,'(CFS)',/)
      DELTC1=-DELTC
      CALL PSCAL(SCALE,GPA,NPRINT,AMAX,AMIN)
      DO 200 K = 1 , NDELTC
      DELTC1=DELTC1+DELTC
      IVAL=QB(I,K)
      CALL LINE(ILINE,IVAL,SCALE,IVPT,INSUB,K)
      WRITE(NPRINT,2003) DELTC1,QB(I,K),IIG,(ILINE(L),L=1,50),IIG
  200 CONTINUE
 2003 FORMAT(8X,F6.2,6X,F6.2,T51,A1,50A1,A1)
      WRITE(NPRINT,2033)
      WRITE(NPRINT,2032)  (GPA(KN),KN=1,6)
 2000 FORMAT(1H1,//,5X,20A4,/,5X,20A4,/)
 2001 FORMAT(//,T40,'SURFACE RUNOFF - DETAILED SIMULATION RESULTS',/)
 2002 FORMAT(//,2X,'SUBAREA NO. ',I5,20X,'FLOWS IN CFS',//)
  100 CONTINUE
C
C
C     TIME HISTORY OF FLOWS AND DEPTHS ON STREETS
C
C
 2030 FORMAT(//,T40,'MAJOR SYSTEM DETAILED SIMULATION RESULTS',//)
      DO 120 I = 1 , NOS
      T1=-DELTC
```

```
      IF(IPNIS(I).GT.0) GO TO 130
      GO TO 120
  130 CONTINUE
      WRITE(NPRINT,2000)  (ITITL(KN),KN=1,80)
      WRITE(NPRINT,2030)
      WRITE(NPRINT,2006)  NS(I)
      WRITE(NPRINT,2031)
 2031 FORMAT(T10,'TIME',T20,'INFLOW',T30,'DEPTH',T40,'OUTFLOW',
     1       T70,' * INFLOW  (CFS)',/,
     1       T10,'(MIN)',T20,'(CFS)',T31,'(IN)',T41,'(CFS)',
     1       T70,' + DEPTH  (IN)',/)
      CALL MAXMIN(AMAX,AMIN,QINS,I,NDELTC)
      AMAX2=AMAX
      AMIN2=AMIN
      CALL MAXMIN(AMAX,AMIN,SDPT,I,NDELTC)
      AMAX=AMAX1(AMAX,AMAX2)
      AMIN=AMIN1(AMIN,AMIN2)
      INSUB=1
      SCALE=(AMAX-AMIN)/50.0
      IVPT=2
      CALL PSCAL(SCALE,GRA,NPRINT,AMAX,AMIN)
      DO 140 J = 1 , NDELTC
      T1=T1+DELTC
      XVAL=QINS(I,J)
      INSUB=1
      CALL LINE(ILINE,XVAL,SCALE,IVPT,INSUB,J)
      INSUB=2
      XVAL=SDPT(I,J)
      CALL LINE(ILINE,XVAL,SCALE,IVPT,INSUB,J)
      WRITE(NPRINT,2004)  T1,QINS(I,J),SDPT(I,J),QOUTS(I,J),
     1                    IIG,(ILINE(L),L=1,50),IIG
  140 CONTINUE
      WRITE(NPRINT,2033)
      WRITE(NPRINT,2032)  (GRA(KN),KN=1,6)
 2033 FORMAT(50X,5(1HI,9H---------),2H-I)
 2032 FORMAT(50X,5(E8.2,2X),1X,E8.2)
 2004 FORMAT(T10,F6.2,T20,F6.2,T30,F6.2,T40,F6.2,T51,A1,50A1,A1)
  120 CONTINUE
C
C
C     TIME HISTORY OF PIPE FLOW-ONLY IF IEVAL =0,1,OR 2
C
C
      IF(IEVAL.EQ.3) GO TO 170
 2040 FORMAT(T40,'MINOR SYSTEM - DETAILED SIMULATION RESULTS',/)
      DO 150 I = 1 , NOP
      IF(IPNTP(I).GT.0) GO TO 160
      GO TO 150
  160 CONTINUE
      WRITE(NPRINT,2000)  (ITITL(KN),KN=1,80)
      WRITE(NPRINT,2040)
      WRITE(NPRINT,2009)  NP(I)
      WRITE(NPRINT,2041)
 2041 FORMAT(T10,'TIME',T20,'INFLOW',T30,'OUTFLOW',T70,
     1       'INFLOW  (CFS)',/,T10,'(MIN)',T20,'(CFS)',T30,'(CFS)',/)
      CALL MAXMIN(AMAX,AMIN,QINP,I,NDELTC)
      INSUB=1
      IVPT=1
      SCALE=(AMAX-AMIN)/50.0
      CALL PSCAL(SCALE,GRA,NPRINT,AMAX,AMIN)
      T1=-DELTC
      DO 180 K = 1 , NDELTC
      XVAL=QINP(I,K)
      T1=T1+DELTC
      CALL LINE(ILINE,XVAL,SCALE,IVPT,INSUB,K)
      WRITE(NPRINT,2012)  T1,QINP(I,K),QOUTP(I,K),IIG,(ILINE(L),L=1,50),
     1                    IIG
  180 CONTINUE
 2012 FORMAT(T10,F6.2,T20,F6.2,T30,F6.2,T51,A1,50A1,A1)
      WRITE(NPRINT,2033)
      WRITE(NPRINT,2032)  (GRA(KN),KN=1,6)
  150 CONTINUE
  170 CONTINUE
 2006 FORMAT(//,2X,'MAJOR SYSTEM SEGMENT NO. ',I5,//)
 2007 FORMAT(5X,8(5X,F6.2))
 2009 FORMAT(//,5X,'PIPE NO.  ',I5,//)
 2010 FORMAT(5X,10(2X,F6.2))
      RETURN
      END
C
C
C
      SUBROUTINE SUMPRT
C
C
C
C
C
C     THIS SUBROUTINE PRINTS SUMMARY OF SIMULATION RESULTS FOR
C     MAJOR AND MINOR SYSTEMS
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,RI
     1             ,NDELTC,INS,INTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1             OP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2             NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3             ,QOUTP(200,100),DDP(200),QMAX(200),QDQP(200),
     4             IPNTP(200),QB(200),TMAX(200),ICPTRP,QFI(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NNSTP(5,5),NNPT,
     1             QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/  MJR/ NSMAX,NISS,NS(200),NSD(200),IS(200),IDSS(200),
     1             NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2             QINS(200,100),QOUTS(200,100),TS(50,10),FLOW(50,10),
     3             SPEED2(50,10),IS,AS(50,10),QQSS(50,10),NPTS,
     4             IPNTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NNSTS(5,5),NNST,
     1             QINP(5,100),NSTMAX,SS(10),CELAR(50,10)
      COMMON/  INL/ NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1             IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2             0),CBMAX(200)
      COMMON/  SUP/ NUSMAX,NINW,NW(200),AW(200),PIMP(200),CNIMP(200),
     1             CNP(200),S(200),WLAT(200),DETIMP(200),DETP(200),
     2             WCON(200,5),RINT(100),WDEPTH(200,5),WFLOW(200),
     3             FMAX(200),FMIN(200),DEC(200),DET(200,5),NOWS,
     4             NWS(200,5),SUROW(200),QR(200,100),IPPTW(200)
      COMMON / TITL/ ITITL(80)
      INTEGER ITN(3)
      DATA ITN /'YES','NO ',' - '/
```

```
C
C
C
C     MAJOR SYSTEM
C
C
      WRITE(NPRINT,2000) (ITITL(KX),KX=1,40)
      WRITE(NPRINT,2010)
 2010 FORMAT(T40,'MAJOR SYSTEM',//,T30,'SUMMARY OF SIMULATION RESULTS',/
     1)
      WRITE(NPRINT,2011)
 2011 FORMAT(T10,'SEGMENT',T20,'MAX.',T30,'MAX.',T40,'MAX.',T50,'INLET',
     1       /,T12,'NO',T19,'FLOW',T29,'DEPTH',T39,'CAPTURE',T49,
     2       'RESTRICTION',/,
     3       T20,'(CFS)',T30,'(IN)',T40,'(CFS)',/)
      DO 100 I = 1 , NOS
      IREST=ITN(2)
      QLLMT=QLIM(I)*NUMCB(I)
      IF(CBMAX(I).GE.QLLMT) IREST=ITN(1)
      IF(NUMCB(I).EQ.0) IREST=ITN(3)
      WRITE(NPRINT,2003) I,NS(I),SPMAX(I),SDMAX(I),CBMAX(I),IREST
  100 CONTINUE
C
C
C .   MINOR SYSTEM
C
C
      IF(IEVAL.EQ.3) GO TO 120
      WRITE(NPRINT,2000) (ITITL(KX),KX=1,40)
      WRITE(NPRINT,2001)
 2001 FORMAT(/,T40,'MINOR SYSTEM',//,T30,'SUMMARY OF SIMULATION RESULTS'
     1       ,//)
      IF(IEVAL.EQ.0) WRITE(NPRINT,2006).
 2006 FORMAT(T10,'SEWER',T20,'SLOPE',T30,'MANNING',T40,'MAX.',T50,'SEWER
     1',T60,'SEWER',T70,'Q/QFULL',/,T11,'NO',T19,'(FT/FT)',T32,'N',
     2       T39,'FLOW',T50,'DIAM.',T59,'CAPACITY',T70,'(RATIO)',/,
     3       T39,'(CFS)',T51,'(IN)',T61,'(CFS)',//)
      IF(IEVAL.EQ.1) WRITE(NPRINT,2007)
 2007 FORMAT(T10,'SEWER',T20,'SLOPE',T30,'MANNING',T40,'MAX.',T50,
     1       'ORIGINAL',T60,'AVAILABLE',T72,'Q/QFULL',T90,'REQUIRED',
     2       T100,'CAPACITY',T110,'Q/QFULL',/,
     3       T11,'NO',T20,'(FT/FT)',T32,'N',T40,'FLOW',T51,'SIZE',
     4       T60,'CAPACITY',T72,'ORIGINAL',T91,'SIZE',T101,'(CFS)',
     5       T109,'REQUIRED',/,
     6       T40,'(CFS)',T51,'(IN)',T61,'(CFS)',T73,'SIZE',T91,
     7       '(IN)',T111,'SIZE',/)
      IF(IEVAL.EQ.2) WRITE(NPRINT,2008)
 2008 FORMAT(T13,'SEWER',T23,'SLOPE',T30,'MANNING',T43,'MAX.',T53,
     1       'SEWER',T63,'SEWER',T73,'Q/QFULL',T92,'EXCESS',/,
     2       T11,'NO',T22,'(FT/FT)',T32,'N',T43,'FLOW',T53,'SIZE',
     3       T62,'CAPACITY',T72,'AVAILABLE',T92,'FLOW',/,
     4       T42,'(CFS)',T53,'(IN)',T63,'(CFS)',T92,'(CFS)',/)
      DO 110 I = 1 , NOP
      IF(IEVAL.EQ.0) QFLL=QMAX(I)/QDOP(I)
      IF(IEVAL.EQ.0) WRITE(NPRINT,2004) I,NP(I),SP(I),CP(I),QMAX(I),
     1                               DDP(I),QFLL,ODOP(I)
      IF(IEVAL.EQ.1) GO TO 10
      GO TO 20
   10 CONTINUE
      AOR=(22./7.)*DP(I)**2.0/576.0
      QFLL1=AOR*1.486/CP(I)*(DP(I)/48.)**(2./3.)*SP(I)**0.5
      ANEW=(22./7.)*DDP(I)**2.0/576.0
      QFLL2=ANEW*1.486/CP(I)*(DDP(I)/48.)**(2./3.)*SP(I)**0.5
      QTQF=QMAX(I)/QFLL1
   20 CONTINUE
      IF(IEVAL.EQ.1) WRITE(NPRINT,2005) I,NP(I),SP(I),CP(I),QMAX(I),
     1                               DP(I),QFLL1,QTQF,DDP(I),QFLL2,
     2                               ODOP(I)
      IF(IEVAL.EQ.2) GO TO 30
      GO TO 40
   30 CONTINUE
      AOP=(22./7.)*DP(I)**2.0/576.0
      QFLL=AOP*1.486/CP(I)*(DP(I)/48.)**(2./3.)*SP(I)**0.5
      QTQF=QMAX(I)/QFLL
      WRITE(NPRINT,2009) I,NP(I),SP(I),CP(I),QMAX(I),
     1                DP(I),QFLL,QTQF,QEX(I)
   40 CONTINUE
  110 CONTINUE
  120 CONTINUE
 2000 FORMAT(1H1,//,5X,20A4,/,5X,20A4,/)
 2003 FORMAT(T2,I5,T8,I5,T18,F6.2,T28,F6.2,T38,F6.2,T51,A3)
 2004 FORMAT(T2,I5,T11,I5,T21,F7.5,T31,F6.4,T40,F7.2,T50,F7.2,T61,F7.2,
     1       T71,F7.2)
 2005 FORMAT(T2,I5,T9,I5,T19,F7.5,T29,F7.5,T39,F7.2,T50,F7.2,
     1       T61,F7.2,T73,F4.2,T90,F7.2,T100,F7.2,T112,F4.2)
 2009 FORMAT(T2,I5,T11,I5,T21,F7.5,T31,F6.4,T40,F7.2,T50,F7.2,T61,F7.2,
     1       T71,F7.2,T90,F7.3)
      RETURN
      END
C
C
C
      SUBROUTINE SURCH
C
C
C
C
C
C     THIS SUBROUTINE STORES FLOWS CAPTURED BY INLETS ON A DISK FILE-
C     TO BE USED AS AN INPUT TO IMPSWM-EXTRAN
C
C
      COMMON/  TAP/ NREAD,NPRINT
      COMMON/  CON/ IEVAL,TIMEC,DELTC,DELTCS,DELTC2,TTOTC,DELTR,RI
     1              ,NDELTC,INS,IPTS
      COMMON/  MIN/ NPMAX,NP(200),NPD(200),IP(200),SP(200),CP(200),
     1              DP(200),NOP,NOPP,IDMN(200),NPOO(5),NPO(200),
     2              NPP(200,5),NINP,SUMQIN(200),QINP(200,100)
     3              ,QOUTP(200,100),DDP(200),QMAX(200),ODOP(200),
     4              IPTYP(200),QB(200),TMAX(200),IOPTNP,QEX(200)
      COMMON/ MIN1/ JPP,NPPS(200),IDSTP(5),IPTOS(200),NPSTP(5,5),NNPT,
     1              QINPP(5,100),NPTMAX,QBLSP(5)
      COMMON/  MJR/ NSMAX,NINS,NS(200),NSD(200),IS(200),IDSS(200),
     1              NOS,NOSS,IDMJ(200),NSOO(5),NSS(200,5),NSO(200),
     2              QINS(200,100),QOUTS(200,100),YS(50,10),FLOW(50,10),
     3              SPEED2(50,10),IS,AS(50,10),QOSS(50,10),NPTS,
     4              IPSTS(200),SDPT(200,100),SPMAX(200),SDMAX(200)
      COMMON/ MJR1/ JSS,NSTS(200),IDSTS(5),ISTOS(200),NPSTS(5,5),NNST,
```

```
     1               QINF(5,100),NSTMAX,SS(10),CELAP(50,10)
      COMMON/  INL/  NINI,NPCAP(10),QAPP(10,50),QCAP(10,50),NUMCB(200),
     1               IINC(200),QLIM(200),NIN(200),NIP(200,5),QCAPT(200,10
     2               0),CBMAX(200)
      COMMON/  SUP/  NUSMAX,NINV,NU(200),AN(200),PIMP(200),CHIMP(200),
     1               CMP(200),S(200),NLAT(200),DETIMP(200),DPTP(200),
     2               NCON(200,5),NINT(100),NDEPTH(200,5),NPLCN(200),
     3               PMAX(200),PMIN(200),DEC(200),DEY(200,5),NOWS,
     4               NUS(200,5),SUMQN(200),QN(200,100),IPPTN(200)
C
C
C
      WRITE(NTAPE) NOS,(NIN(I),I=1,NOS)
      DO 100 I = 1 , NDELTC
      T1=DELTC*(I-1)/60.0
      WRITE(NTAPE) T1,(QCAPT(J,I),J=1,NOS)
  100 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE LESQ(A,B,X,Y,M,N)
C
C
C
      DIMENSION A(10),B(3),X(3),Y(3)
C
C
C
      M1=M+1
      L=M+M1
      K=(M-1)*M/2
      DO 10 I = 1 , L
      J=I+K
      A(J)=0.0
   10 IF(I.LE.M1) B(I)=0.0
      DO 20 IN = 1 , N
      T=1.0
      DO 20 I = 1 , L
      J = I + K
      A(J)=A(J)+T
      IF(I.LE.M1) B(I)=B(I)+T*Y(IN)
   20 T = T*X(IN)
      K1=0
      K=K-1
      DO 30 I = 1 , M1
      DO 30 J = 1 , I
      K1=K1+1
      L=I+J+K
   30 A(K1)=A(L)
      CALL PDMSOL(A,M1,X,B)
      RETURN
      END
C
C
C
      SUBROUTINE PDMSOL(A,N,IERR,X)
C
C
C
      DIMENSION A(10),X(3)
      IERR=N
      DO 10 I = 1 , N
      II=I*(I-1)/2
      IN=I-1
      IP=II+I
      IF(I.EQ.1) GO TO 14
      DO 13 K = 1 , IN
      IK=II+K
   13 A(IP)=A(IP)-A(IK)**2
   14 IF(A(IP).LE.0.0) RETURN
      A(IP)=SQRT(A(IP))
      IF(I.EQ.N) GO TO 19
      IM=I+1
      DO 11 J = IM , N
      JJ=J*(J-1)/2
      IJ=JJ+I
      IF(I.EQ.1) GO TO 11
      DO 15 K = 1 , IN
      IK=II+K
      JK=JJ+K
   15 A(IJ)=A(IJ)-A(IK)*A(JK)
   11 A(IJ)=A(IJ)/A(IP)
   10 CONTINUE
   19 DO 20 I = 1 , N
      II=I*(I-1)/2
      IM=I+1
      IP=II+I
      A(IP)=1./A(IP)
      IF(I.EQ.N) GO TO 29
      DO 21 J = IM , N
      J1=J-1
      JJ=J*J1/2
      AA=0.0
      DO 22 K = I , J1
      KJ=JJ+K
      KK=K*(K-1)/2
      IK=KK+I
   22 AA=AA+A(IK)*A(KJ)
      IJ=JJ+I
      JP=J+JJ
   21 A(IJ)=-AA/A(JP)
   20 CONTINUE
   29 IERR=0.
      I=N
      DO 45 II = 1 , N
      AA=0.
      IJ=(I*(I+1))/2
      J=I
      DO 46 JJ = 1 , I
      AA=AA+A(IJ)*X(J)
      J=J-1
   46 IJ=IJ-1
      X(I)=AA
   45 I=I-1
      DO 40 I = 1 , N
      AA=0.0
      IJ=(I*(I+1))/2
      DO 41 J = I , N
```

```
      AA=AA+A(IJ)*X(J)
   41 IJ=IJ+J
   40 X(I)=AA
      RETURN
      END
C
C
C
      SUBROUTINE LINE(ILINE,XVAL,SCALE,IVPT,INSUB,XX)
C
C
C
      INTEGER ILINE(50)
      DATA ISTAR,IPLUS,IA,IBLAN /'*','+','X',' '/
      DATA IS /'S'/
C
C
C
      IP=IFIX((XVAL/SCALE)+1.0)
      IF(IP.GT.50) IP=50
      IF(IP.LE.0) IP=1
      IF(IVPT.EQ.1) GO TO 10
      IF(INSUB.EQ.1) GO TO 10
    5 GO TO (6,7,8), INSUB
    6 ILINE(IP)=ISTAR
      RETURN
    7 IF(ILINE(IP).NE.ISTAR) ILINE(IP)=IPLUS
      IF(ILINE(IP).EQ.ISTAR) ILINE(IP)=IS
      RETURN
    8 IF(ILINE(IP).EQ.IBLAN) GO TO 40
      IF(ILINE(IP).EQ.ISTAR) GO TO 41
      IF(ILINE(IP).EQ.IPLUS) GO TO 41
      IF(ILINE(IP).EQ.IA) RETURN
      IF(ILINE(IP).EQ.IS) RETURN
   40 ILINE(IP)=IA
      RETURN
   41 ILINE(IP)=IS
      RETURN
   10 DO 20 K = 1 , 50
      ILINE(K)=IBLAN
   20 CONTINUE
      IF(IVPT.GT.1) GO TO 5
      DO 30 K = 1 , IP
      ILINE(K)=ISTAR
   30 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE MAXMIN(AMAX,AMIN,VAL,I,ITIMES)
C
C
C
      REAL VAL(200,100)
C
C
C
      AMAX=10.0E-50
      AMIN=10.0E+50
      DO 10 K = 1 , ITIMES
      AMAX=AMAX1(VAL(I,K),AMAX)
      AMIN=AMIN1(VAL(I,K),AMIN)
   10 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE MAXMN(AMAX,AMIN,VAL,I,ITIMES)
C
C
C
      REAL VAL(5,100)
C
C
C
      AMAX=10.0E-50
      AMIN=10.0E+50
      DO 10 K = 1 , ITIMES
      AMAX=AMAX1(VAL(I,K),AMAX)
      AMIN=AMIN1(VAL(I,K),AMIN)
   10 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE MAXMN1(AMAX,AMIN,VAL,I,ITIMES)
C
C
C
      REAL VAL(100)
C
C
C
      AMAX=10.0E-50
      AMIN=10.0E+50
      DO 10 K = 1 , ITIMES
      AMAX=AMAX1(VAL(K),AMAX)
      AMIN=AMIN1(VAL(K),AMIN)
   10 CONTINUE
      RETURN
      END
C
C
C
      SUBROUTINE PSCAL(SCALE,GRA,NPRINT,AMAX,AMIN)
C
C
C
      REAL GRA(6)
C
C
C
      GRA(1)=AMIN
      GRA(6)=AMAX
      DO 100 KN = 2 , 5
      GRA(KN)=SCALE*FLOAT(KN-1)*10.0
  100 CONTINUE
```

```
      WRITE(NPRINT,2020) (GRL(KB),KB=1,6)                    DDM3240
      WRITE(NPRINT,2021)                                     DDM3241
2020  FORMAT(50X,5(F8.2,2X),1X,F8.2)                         DDM3242
2021  FORMAT(50X,5(1HI,9H---------),2H-I)                    DDM3243
      RETURN                                                 DDM3244
      END                                                    DDM3245
```