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Optimal design of pumped storm water systems.

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OPTIMAL DESIGN OF PUMPED STORM WATER SYSTEMS

by

Emad Hamdy Hassan Imam
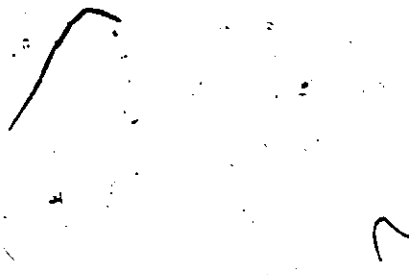
A Thesis
submitted to the Faculty of Graduate Studies
through the Department of
Civil Engineering in Partial Fulfillment
of the requirements for the Degree
of Master of Applied Science at
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Windsor, Ontario, Canada

1977

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To my wife, Nelly

ABSTRACT

OPTIMAL DESIGN OF PUMPED STORM WATER SYSTEMS

by

Emad Hamdy Hassan Imam

A computer-aided design procedure has been developed
to obtain the optimum design of pumped storm sewer systems.
The optimal design is achieved when the combined cost of
both annual operating and capital recovery cost, and average
annual probable damage is minimum. The design variables
include the depths, slopes and diameters of the sewers, as
well as sump area and rated pumping head and discharge for
the pumping station.

The proposed model consists of a simulation sub-model
and a linear programming subroutine. The simulation sub-
model includes a hydraulic simulation algorithm, and
installation cost and potential damage algorithms. The
hydraulic simulation algorithms considers gravity and sur-
charged flows in the sewer system, and flow to and from the
basements as well as the streets, i.e., the ground surface
This hydraulic simulation algorithm has proved to be useful
in obtaining the various parameters that form the bases for
damage evaluation.

The model also presents an effective design tool for

(iv)

dimensioning the sump well and the pumping units, which is an improvement over the available rule of thumb methods. It takes into consideration the mutual effects between the sewer system and the pumping station. The model has the capability of balancing any initial design, i.e., adjusting the pipe and pump sizes so.that all components in the system operate at the same relative capacity.

The flow-damage algorithm, incorporated in the model, integrates high as well as low probability storms, i.e., it considers a wide range of system operating conditions. When this flow-damage algorithm is coupled with the cost algorithm, it presents an effective tool for testing existing systems as well as specific designs.

The optimization model was applied to a hypothetical drainage basin and the resulting design was compared with a design obtained by the Rational Method. The model offered a significant total cost (installation and potential damage) saving over the design by the Rational Method.

A sensitivity analysis was carried out to investigate the effects of selected parameters on the resulting optimum design. This analysis indicates that system performance and damage costs are very sensitive to an increase in sewer diameters. It also points out that increasing sump area is not an economically feasible solution, unless the unit cost of storage is very low.

(v).

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CHAPTER I

INTRODUCTION

1.1 Objective

The purpose of this research is to develop an improved methodology for the design of pumped storm sewer networks. An optimal design is achieved when the combined cost of both annual operating and capital recovery cost, and average annual probable damage is minimum.

1.2 Definition of the Problem

In urban areas, storm water is usually collected in the streets and conveyed through inlets to sewers which carry it to a point where it can be safely disposed off. The collected water is usually discharged into a stream, lake, or ocean. In some instances, the topography is such that pumping of storm water is necessary. Water collects in the wet well until it rises to a predetermined level and starts the pump. The pumping stations may be located at intermediate points or at the outfall. It is a common practice in Windsor, and other municipalities in the Essex County area, to use downstream pumping stations in the storm sewer networks.

In some communities, e.g. Windsor, basements are drained through house connections to separate storm sewers. It is believed that the arrangement of separating storm and sanitary

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wastewater significantly reduces health hazards resulting from backflow in case of storms more severe than the design storm. Though the recent measurements of the quality of storm wastewater indicate that it is almost as polluted as domestic wastewater.

1.3 Motivation

It has been the practice to design storm sewers for gravity flow and to avoid surcharged (pressurized) flow conditions for flows equal to or less than the design flow. In designing storm sewer networks, an appropriate return period is selected reflecting the importance of the district being considered. Sewer slopes and diameters are selected such that, at peak flows, the sewers are just flowing full. Iterative procedures are used to achieve such a goal.

The longer the return period selected for design, the greater will be the capacity of the network and, consequently, the larger will be its annual costs, e.g. investment charges and maintenance. On the other hand, the longer the return period, the less will be the probable average annual damage. This illustrates why it is recommended to use a longer return period in commercial districts than in residential districts. Yet, it is not a traditional practice to make an economic study to determine the optimum return period.

The Rational Method is the simplest of all the models used in the design of storm sewer networks. Its simplicity has led to its wide-spread use. But, it has its own limitations

and deficiencies both as a simulation model and as a design
tool. As a design model, it has the tendency to overdesign,
but the actual safety factors in a sewer network can vary
widely and their values are not fixed. It has been reported
that using the Rational Method can lead to underdesigned as
well as overdesigned sewers. Recognizing the unsuitability
of the Rational Method and considering the investments
required in the near future, the implementation of such an
unpredictable method is no longer acceptable.

Large sums of money are currently spent in the design,
construction, modification, operation, and maintenance of
storm sewer systems. These large investments have motivated
many investigators to develop new rainfall-runoff mathematical
models. These models can be classified into two basic
categories, "management models," and "design models." Manage-
ment models are used primarily for flow simulation of
existing systems. Design models are used to determine the
sewer's diameters and slopes. Various optimization techniques
have been used by these models, e.g., linear programming, non-
linear programming, and dynamic programming.

The majority of the design models are confined to instal-
lation cost minimization only, while very few models attempt to
minimize the combined cost of installation and damage. The
later models obtain a balance between the annual cost of
installing, operating the sewer system, and the potential
damages resulting from adopting a specific design. Obviously,
if any of these models overestimates the probable damage, the

output of the optimization will be an overdesigned network. Consequently, the accuracy of predicting damage cannot be overlooked.

A review of the available models that predict damages shows that they are not based on the true events that take place during flooding storms. In this research, new parameters are introduced as bases for damage evaluation. Basement damages and surface damages constitute most of the damages resulting from an overloaded system. Average assessment value of houses, portion of property value assigned to basements, health hazards, and severity of flooding conditions are used to estimate basement damages. An effective number of vehicle crossings, type of district, property value, and damage severity parameter are used to predict surface damage.

The pumping station used for pumping storm water from the wet well represents one of the major components of the system which drastically affects its performance. But, it is generally designed separately, and the designers neglect the interaction between its pumping capacity, storage capacity, and the sewer network. The present model routes the flow through the entire sewer network including the wet well, such that it can be considered as an integrated system.

1.4 The Approach in General

The model presented in this thesis designs pumped storm sewer networks. To start with, an initial design that satisfies the ASCE conventional constraints is considered.

The model generates a synthetic design hyetograph using a
method developed in Chicago by Chu and Kiefer. The Road
Research Laboratory Method is used to obtain the inflow
hydrographs to the system. A modified form of the Muskingum
Method routes the flow through the sewers. A modified
reservoir routing technique is used to route the outfall
hydrograph through the wet well.

Various design storms, with prescribed return periods,
are generated. The system is balanced for a storm with a
specific return period, e.g. the 1 : 5 year storm. The
balancing involves adjusting the pipe and pump sizes so that
all components in the system operate at the same relative
capacity. The balanced system is tested against all the
generated storms. The model evaluates the damages associated
with each storm, and a damage-probability curve is derived
for any specific design. The average annual probable damage
is determined by integrating the area under this curve. The
average annual cost of the network is estimated. Cost as
well as damage gradients are computed with respect to design
variables.

The data required for linear programming are prepared
using the cost and damage gradients. The transformed con-
straints are determined. The optimal design is obtained using
a linear programming in the Simplex form.

CHAPTER 2

BACKGROUND AND LITERATURE REVIEW

2.1 Preliminary Considerations of Hydrograph Runoff Models

2.1.1 Description of Phenomena and Basic Concepts

An urban watershed consists of two major types of elements: surfaces to be drained and collecting channels. Surfaces normally encountered in urban watersheds can be classified into two basic types, pervious and impervious areas. Grassed areas and gravel roads are considered pervious while roofs (connected to storm sewers) and paved areas are regarded as impervious. Impervious areas may have direct connections to the sewer system, or they may be disconnected and scattered within the pervious areas. Like natural drainage basins, smaller sewer branches convey storm water to larger branches, and so on, until a main or trunk sewer is reached. Gutters, lateral, main and trunk sewers are common names for collecting channels in order of size.

Designing a model to simulate the rainfall-runoff process requires a deep understanding of the events that take place from the time a storm starts until the storm and runoff cease. Storm pattern, infiltration capacity of pervious areas, depression storage, overland flow detention, detention in gutters, and detention in lateral sewer systems are the principal components of the problem.

During the rain storm on a pervious area, water is being abstracted by infiltration, interception and evapo-transpiration. Quantitatively, rainfall interception and evapotranspiration are of minor importance in urban storm drainage and may properly be ignored in design. If the rainfall intensity exceeds the possible rate of infiltration, natural depressions trap some of the excess precipitation creating the depression storage. These depressions are of varying size and depth. They must be filled before runoff (overland flow) commences.

The depth of water detained in the overland sheet flow forms the detention storage on the pervious area which, combined with the depression storage, forms the total surface storage. On the other hand, impervious areas have negligible infiltration losses and much smaller depression and detention storages compared to pervious areas. However, the latter two factors cannot be overlooked if accurate simulation is to be achieved. It is worthy of notice that pervious areas start to contribute surface runoff after approximately one or two hours from the beginning of the rainfall (36). This may vary according to type of soil, surface conditions that exist and antecedent conditions.

Overland flow from both types of areas is collected in gutters or ditches and subsequently enters the sewer system. The problem of predicting the flow, at a certain time, at a specified point in the sewer system can be divided into two parts, namely; surface routing up to the inlets (or inlet

catch basins if any), i.e. basic hydrographs, and routing through the conduits downstream from the inlets. The simulation process is illustrated in Figure 2.1.

Transforming the rainfall to the inlet inflow hydrographs is the first part of the simulation. To achieve this goal, an analysis of the rainfall data should be made. The sewer district, and basic subcatchments should be studied to reduce the data necessary for the rainfall-runoff model. Different models have been used to route the overland supply (Section 2.2.2) with various degrees of sophistication. The generated hydrograph from each of the basic subcatchments is input into the sewer system. The flow entering an inlet does not depend on the depth of water in the inlet nor the discharge into the sewer unless the inlet basin is completely filled and submerged which rarely occurs. This provides means to divide the simulation into sequential subsystems for hydraulic analysis without sacrificing the accuracy and usefulness of the model. These outlet hydrographs of the basic subcatchments are routed by more sophisticated methods (Section 2.2.3).

The degree of schematization (lumping) (20, 26, 28) used in the definition of the basic subcatchments, and the selection of the sewers to be modelled requires some experience. The size chosen for subcatchments should be based upon the sensitivity of the model and the objectives of the simulation. The major sources of errors lie in the simulation of the outlet hydrographs. The cost of collecting input data

describing the catchment characteristics is proportionate to
the degree of detail chosen for defining the basic sub-
catchments.

2.1.2 Model Calibration and Validation

Models used for the simulation of urban rainfall-
runoff have, as an input, a real or synthetic fictitious
design rain storm and, as an output, the runoff (flow) at
one or several locations of the sewer system.

Schaake (20) has discussed a general rationale for
modelling urban runoff in "Treatise of Urban Water Systems."
Imitating the physical system is not the only route to
simulate the runoff process. An exact representation of
nature is not the purpose of the model builder. Proper
selection of equivalent parameters and simplifying assumptions,
may give a tool which yields a useful result with a reasonable
balance between cost of modelling and accuracy.

Natural hydrologic phenomena encountered in the problem
of urban runoff are so complicated that any model must neglect
at least some of their aspects. The modelling problem, there-
fore, is to choose the most important aspects to be represented
and to decide how these could be simulated. Because of this,
models differ from one another. The availability of input
parameters and the cost of data collecting are also important
considerations. Models with moderate data requirements and
reasonable accuracy are preferred by most of the designers.

Any model has parameters which may or may not have a

physical counterpart. Some of the parameters can be deter-
mined from physical data which can be accurately measured or
estimated by the model user without difficulty, such as the
degree of imperviousness or the roughness coefficient of a
sewer. Most models, however, include parameters which are
not readily determined, such as depressions storage and
infiltration capacity characteristics. The non-uniform shape
of the watershed is replaced in some models by equivalent
areas with a uniform slope and, in this case, the equivalent
slope has to be estimated. In this special case, this is
done because theoretical routing methods are available for
a plane surface.

Because of the approximate nature of the previous
factors, models have to calibrate the values of the parameters
by fitting the simulated hydrographs to some of the available
measurements. If an acceptable goodness of fit is achieved,
the model is then validated by comparing other measured
samples with the computed output. The fact that a model
compares favourably with measurements for a limited number of
events may be just the result of the calibration process,
however, and it is not necessarily true that similar results
will be obtained for other input data. Once a model is
calibrated and validated for real rainfall runoff data, it
may be used in design.

2.2 Elements of the Simulation of Urban Runoff

2.2.1 Design Storms

Analysis of rainfall data is the first step in simulating runoff whether for management or design purposes. Before the introduction of the hydrograph models for simulation, the Rational Method was the most frequently used design procedure. One of the important parameters involved in this method is the time of concentration. The basic concept behind the Rational Method is that the storm giving the highest runoff peak for a given catchment would have a duration equal to the time of concentration for the catchment. Consequently, rainfall data are processed to obtain the average rainfall intensity for a given duration (equal to the time of concentration) and a given return period.

In current practice, a design storm hyetograph is an essential part of the rainfall models. A rain storm specified by the municipality or a synthetic hyetograph may be used. To develop a synthetic hyetograph, the following conditions should be met (36, 20):

(i) For any given duration, the maximum average intensity equals that of the intensity-duration curve of the same return period. This condition enables the designer to make use of the reduced data which are available in the form of intensity-duration curves.

(ii) Characteristic parameters of the hyetograph, such as the total duration of the rainfall and the relative advancement of the peak intensity, are obtained by statistical

analysis of many rainfall events in or near the watershed.

The most frequently used method for the derivation of a design hyetograph is that developed in Chicago by Chu and Kiefer (36, 3, 25). The equation for the design storm pattern is

$$\text{(Before the Peak)} \quad i = \frac{a\,[(1-b)\,(t_b/r)^b + c]}{[(t_b/r)^b + c]^2} \qquad (2.1.a)$$

$$\text{(After the Peak)} \quad i = \frac{a\,[(1-b)\,(t_a/1-r)^b + c]}{[(t_a/1-r)^b + c]^2} \qquad (2.1.b)$$

in which t_b and t_a are the time measured from the peak to the left and to the right respectively, and a, b, and c are constants found in the intensity-duration-frequency curve formula such as

$$i_{ave.} = \frac{a}{t_d^b + c} \qquad (2.2)$$

The value of r is a measure of the advancement of the storm pattern and is defined as the elapsed time, in minutes from the beginning of the design storm to its peak, divided by total duration of storm.

A typical graph of a design-storm hyetograph (Figure 2.2.b), and its interrelation with the corresponding intensity-duration curve is shown in Figure 2.2.a. For this example the constants are a = 90, b = 0.9, c = 11, and r = 318.

For hydrograph methods, the same rainfall is considered for all points in the system. · For a small subcatchment, the critical part of the storm occurs during the short period of very high intensities while the initial rainfall preceding this period creates wet antecedent conditions. Antecedent conditions can also be estimated by the statistical analysis of rainfall events.

In some regions, a summer design rainfall with a higher intensity, shorter duration and dry antecedent conditions may be less critical than a spring or autumn storm with lower intensity, but with high soil moisture content or frozen soil conditions. Stoddard and Watt (4) presented a study on the possible relative magnitudes (shifts) of summer and spring frequency curves. It showed that in large watersheds, peak spring flows are higher than peak summer flows for all return periods. On the other hand, it indicated that in small subcatchments which are common in urban areas, peak summer flows are higher than peak spring flows. Consequently, the summer rain storms can be considered the critical type of storm in urban areas, unless the locality requires special study of spring conditions.

2.2.2 Overland Runoff Simulation

The input to this part of the simulation model is rainfall data and characteristics of basic unit subcatchments. The output of the overland runoff model is the basic inflow hydrographs to the sewer system. There are two approaches to

estimate overland runoff due to rainfall and, consequently,
basic hydrographs. The black box (lumped system) approach
produces output from a given input without considering the
true events that take place within the overland surface.
The hydraulic routing (distributed system) approach routes
the rainfall excess through the overland surface to produce
the runoff hydrograph.

Yen (39) subdivided the models using black box approach
into four groups, namely; the rainfall-intensity coefficient
formulas, the frequency formulas, the monograph methods, and
the hydrograph methods. The first three groups give only
the magnitude of the peak rate of storm runoff. The last
group, the hydrograph methods, gives information on the time
distribution of runoff and hence is more useful in solving
urban drainage problems.

Chow (39) presented an excellent summary of the rainfall-
intensity coefficient formulas. All of these formulas were
derived on the basis of regional observed data and have
limitations on basin size, region, and other geographical
conditions of application. All but two of these formulas
are of no use today. They are the Rational and Burkli-Ziegler
formulas which have been applied to different geographic
regions with the coefficients evaluated over a wide range of
conditions. Both formulas can provide reliable estimates of
peak runoff if the coefficients are correctly chosen. Because
of its simplicity, the Rational formula is still used as a
standard method in storm sewer design (ASCE manual). The

deficiencies and limitations of the Rational method are presented in Section 2.5.

The frequency formulas or graphs based on past flood records have been proposed for small as well as large watersheds. However, this method has yet to be successfully adopted to urban environment. The monograph methods were developed primarily for and have been widely used in rural areas. Because the monographs of these methods for urban areas have not been developed, they are seldom used in urban areas.

The hydrograph methods, which give not only the values of the peak flow but also the entire runoff hydrographs are more sophisticated than the previously discussed methods and require more data. The hydrograph methods are based on an approach that establishes a reference hydrograph or hydrographs for a prescribed drainage area which can be used repeatedly for different rainfall storms. The unit hydrograph is a typical example of this approach. In this method, as described by Eagleson (39), measured sewer outflow hydrographs from urban areas of varying types are studied in order to construct synthetic unit hydrographs for areas under design. It is subject to a restriction that no appreciable changes should take place in physical nature of the drainage area, otherwise the reference hydrograph might be altered. None of these methods has been sufficiently tested for urban drainage areas.

The hydraulic routing (distributed system) approach for

the evaluation of overland runoff provides not only the runoff hydrograph but also some information on the flow within the drainage area. Two of these methods, the Izzard (39, 15) and the Horton (39, 15) methods, have long been accepted by hydraulic and sanitary engineers. They studied flow across a sloping plane at an unsteady state. Horton proposed an equation for overland flow, considered suitable for turbulent flow with high discharge on natural surfaces, whereas Izzard developed a dimensionless hydrograph for surface flow which is largely applicable to laminar flow on developed surfaces.

Several sophisticated models have been proposed recently to engineers for practical applications. The Kinematic Wave method, solving the St. Venant equations in one-space dimension and two-space dimensions are possible routing techniques that can be used to evaluate overland runoff. A review of these techniques was reported by Yen (39).

2.2.3 Sewer Flow Simulation

Sewer flow simulation models require basic inflow hydrographs to the sewer system as the input and give as the output, the runoff hydrograph at the point of interest in the drainage system. Several approaches have been proposed to route the inflow hydrographs through the sewer system with various degrees of sophistication. These approaches can be classified into two basic groups, namely; approaches using hydrologic routing techniques, and approaches solving the continuity equation together with various simplifications of

the momentum equation (dynamic routing). All the afore-
mentioned flow routing techniques are applicable to a single
sewer. When these techniques are applied to a network, the
sewers or channels are simply treated individually in sequence
with the flow cascading downstream from one sewer to another
(20, 39). However, in a sewer network, considerations must
be given to such mutual dynamic effects as backwater and
energy losses among sewers and junctions.

The basis for any sewer flow routing technique is the
two basic equations (20) representing the gradually varied
free-surface unsteady flow. These are the momentum equation

$$\frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + g \frac{\partial h}{\partial x} - g (S_o - S_f) = \frac{v \cdot q}{A} \qquad (2.3.a)$$

and the corresponding equation of continuity

$$\frac{\partial h}{\partial t} + \frac{A}{B} \frac{\partial v}{\partial x} + v \frac{\partial h}{\partial x} = \frac{q}{B} \qquad (2.3.b)$$

where

x = longitudinal coordinate along channel bottom
direction;

y = cross sectional average flow velocity along the x
direction;

t = time;

g = gravitational acceleration;

h = depth of flow;

S_o = channel bottom slope;

S_f = friction slope;

q = the distributed lateral inflow (or outflow) as discharge per unit length of the conduit;

A = area of channel cross section;

and B = water surface width.

These quasi-linear hyperbolic first-order partial differential equations are derived from the well-known St. Venant equations by adding the lateral flow term to the right hand side to each equation. These two equations can be solved numerically by using the method of characteristics. Although this method gives the most accurate of all practical methods of flood routing in channels and conduits, it requires a considerable amount of computation time. In addition, difficulties are also encountered in defining the boundary conditions. Hence, various simplifications have been proposed to give simple approximate solutions.

The Hydrologic Routing Techniques:

This approach uses the continuity equation, often rewritten in the form

$$I - O = dS/dt \qquad (2.4.a)$$

where

I = rate of inflow into the control volume considered

O = rate of outflow from the control volume considered

S = storage within the control volume

or it may be written in finite difference form,

$$\frac{\Delta t}{2} (I_1 + I_2) = \frac{\Delta t}{2} (O_1 + O_2) + (S_2 - S_1) \qquad (2.4.b)$$

where the subscripts indicate the variables at the beginning and end of the time increment, Δt. A storage flow relationship has to be derived either by analyzing recorded hydrographs or by making other simplifying assumptions. Obviously, acceleration effects cannot be taken into consideration in this routing technique.

One of the procedures that has been used in association with the hydrologic routing techniques is the method of summing hydrographs (15). The procedure for this method involves the development of an inflow hydrograph from a drainage area at junction manhole A on a sewer, routing of this hydrograph using Equation 2.4.b, through a reach of the conduit to point B, where it is added linearly with all hydrographs from sewers tributary to manhole B. The summed hydrograph is routed to another junction, point C, and so on through the system.

Examining Equation 2.4.b, shows that the basis of the derivation of the storage flow relationship is the most important characteristic of the hydrologic technique. The Road Research Laboratory Model (RRL) uses the idea of a reservoir with a simple overflow weir. There is a fixed relationship between the volume of water above weir level and the rate of discharge over the weir. The relationship is the combination of the relationships between volume stored against head on weir and between head on weir and discharge.

In other words, it is assumed that the storage is a function
of the outflow only, regardless of the inflow.

Another approach that has been proposed for Chicago
(36, 4) to derive the storage flow relationship is the
Muskingum Method. This method (10) involves the concept of
wedge and prism storages. Storage volume can be correctly
related to outflow with a simple function only when inflow
and outflow are equal, that is, when steady flow exists. The
wedge storage which is related to the difference between
the instantaneous values of inflow and outflow, is added to
the prism storage to obtain the total storage. Additional
aspects of this technique are discussed in Section 3.2.3.

An improved but more sophisticated scheme of the steady-
flow routing method (40) is to account for the time shifting
of the hydrographs. The shifting time is equal to the flow
time in the sewer, t_f = L/V, where V is the full-bore
velocity. From the hydraulic point of view, it is a linear
kinematic wave approximation. Tholin and Keifer (36)
improved the aforementioned scheme. In their recommended
scheme, the inflow hydrograph of a sewer is subdivided into
a number of component hydrographs, each shifted by a time
equal to an assumed time of travel. The sum of these
shifted component hydrographs gives the outflow hydrograph
of the sewer.

The Dynamic Routing Techniques

In these routing techniques, the continuity equation
is solved together with various simplifications of the

momentum equation. Table 2.1 illustrates the various
approximate simulation models.

It is obvious that the dynamic routing techniques can
predict more accurately the runoff hydrographs than the
hydrologic routing techniques. Yet, with the present computer
capabilities and existing optimization algorithms, it is not
practical (39) to find the optimum sewer system design with
the hydraulics of the system simulated by using the St. Venant
equations. So, it may be concluded that the St. Venant
equations can be used for simulating the runoff of existing
systems or possibly for finding the sizes of the sewers for
a network with predetermined layout and slopes.

2.3 Classification of Urban Runoff Models

Models are generally used for studies of quantity and
quality problems associated with urban runoff. Three broad
objectives (32) may be identified in these models, namely;
planning, operation and design. The core of all these models
is the same, which is the simulation of the rainfall-runoff
problem. Yet, each objective typically produces models with
somewhat different characteristics, and the different models
overlap to some degree.

Planning or long term models are used for an overall
assessment of the urban runoff problem as well as estimates
of the effectiveness and costs of abatement procedures. They
may be used for evaluating various control options, e.g.
treatment versus storage. They are characterized by relatively

large time steps (hours) and long simulation times (months and years). Data requirements are kept to a minimum and their mathematical complexity is low. An example of such a model is the Storage, Treatment, Overflow, and Runoff Model (STORM) (32).

Operational models (32) are used to produce actual control decisions during a storm event. Rainfall is entered from telemetered stations and the model is used to predict system responses a short time into the future. Various control options then may be employed, e.g. in-system storage, diversions, regulator settings. These models are frequently developed from sophisticated simulating models and applied to an existing system. An example of these models is the operational model for Minneapolis-St. Paul (7).

Design models are used for the detailed simulation of a single storm event. Ultimately, they provide a complete description of flow and pollutant routing from the point of rainfall through the entire urban runoff system and into the receiving waters as well. Such models may be referred to as either quantity or quality simulating models. They are useful for determining least cost abatement procedures for both quantity and quality problems in urban areas.

An example of a model developed specially for simulation of urban quantity and quality processes is the Storm Water Management Model (SWMM). Many other urban runoff models have been described in the literature (20, 39). Yet, almost all of them, except the aforementioned one, lack quality

simulation. Examples of these models range from relatively simple models, e.g. R RL (38, 35) and Chicago (36, 3), to highly complex models that utilize the complete dynamic equations of motion to simulate many aspects of the drainage systems, e.g. SWMM (28, 29, 32), Hydrograph Volume Method (HVM), and Illinois Storm Sewer (ISS) (39, 40). Selected models are described in Appendix I.

The usefulness of the various available design models can be determined by studying the goals of design. There are three approaches (39) to the design of a storm sewer system:.

(i) Select a layout, select the slopes and determine the sizes of the sewers.

(ii) Select a layout, and find the optimum slopes and diameters of the sewers which will ensure the minimum cost or satisfy other objective functions, e.g. minimum combined cost of both installation and potential damages for the selected layout.

(iii) Find the optimum layout and the optimum slopes and diameters of the sewers which will ensure the minimum cost or satisfy some other objective functions.
The majority of the recent sophisticated models, e.g. SWMM, HVM and ISS, are flow simulation models for existing systems but they can be adopted to the first design approach. They are used primarily, from the design viewpoint, in modifying existing systems for better performance. On the other hand, there are only a few design models in existence that can

handle the second and the third design approaches. A review
of this type of optimization model has been presented by Yen,
Mays and Tang (42). The optimization design models are
discussed in Section 2.4.

2.4 Optimization Design Models

2.4.1 General

Optimization design models are generally used to find
the optimum slopes and diameters of the sewers for a pre-
determined layout. Some of these models have an additional
capability to find the optimum layout (18, 2). The optimiza-
tion design models can be classified into two basic groups,
namely; models searching for least cost design and models
minimizing the combined cost of installation and potential
damages. In the first group, several alternative systems,
each meeting the physical and hydraulic requirements, are
analyzed and the least-cost system is selected. The second
group of optimization design models seeks a balance between
the cost of installing, operating the sewer system, and the
potential damages resulting from a specific design. It is
obvious that the second group presents the final solution to
the problem of selecting optimum return period for the system.

2.4.2 Objective Function

In the least-capital cost design models (the first
group), the objective function is to minimize the cost of
the sewer network. The cost of the system consists of the

cost of sewers, manholes, inlets and pumping stations, if any. Other costs, e.g. appurtenance cost, may be considered constant in an optimization problem. In the least-total cost design models (the second group), the objective function is to minimize the sum of the installation cost and the expected potential damage resulting from adopting a specific design.

Cost Functions

Sewers: Estimating the unit costs of construction for a combined or separate sewer line is an essential part in any optimization model. To develop the cost function, the major parameters that affect the cost of construction, purchase, installation, and excavation, must be identified. A review of the literature shows that there is an agreement among the investigators to consider the sewer diameter and the average depth of excavation as the most important variables affecting the unit cost. The cost function can be obtained from current construction data. Two forms have been used in the optimization models to incorporate the cost function. Arrays of unit cost for different excavation conditions and sewer diameters may be used, or alternatively a proposed equation relating the unit cost with the sewer diameter and the average depth of excavation may be adopted.

Walsh and Linfield (37) and Meritt and Bogan (23) used arrays of unit costs for different excavation conditions, pipes, manholes, and pavement replacements. In this way, the actual sewer construction costs could be accurately estimated.

Yet, it has a serious limitation, that it cannot be coupled with many of the optimization algorithms. The aforementioned model builders incorporated it in connection with dynamic programming. Linear and non-linear programming require mathematical relationships for the cost.

Dajani, Gemmel, and Morlok (6) and Dajani and Hasit (7), using linear regression techniques, have shown that a good fit can be obtained for a cost function in which cost is estimated on the basis of sewer diameters and average excavation depth. The general form of their cost function is:

$$C = a + b D^2 + c X^2 \qquad (2.5)$$

where

C = cost per unit length of sewer

a,b,c are regression coefficients

D = sewer diameter and

X = average depth of excavation.

Henry and Ahern (13) used the same form of equation for both separate and combined sewers in their study of the effect of storage on storm and combined sewers. A slightly modified cost function was incorporated in a least-capital cost model by Gupta et al. (11). Several cost equations to cover the whole range of sewer diameters and depths of excavation were proposed by Meredith. These equations by Meredith were adopted by Tang, Mays, and Yen (34), and Mays and Yen (22) in dynamic programming optimization models.

Lemieux, Zech and Delarue (17) studied the geometry of

a typical trench in order to compute the actual volume of excavation. Their proposed form of cost function is relatively complicated and requires additional information about the trench geometry. It appears that this approach does not significantly improve the accuracy of estimating the unit cost and it results in a relatively complicated objective function.

Manholes: The cost of manholes can be estimated by two different approaches. The first approach is merely a black box method which assumes a certain ratio between the cost of manholes and sewers. Baffa 1955, (6) estimated that 85 per cent of the cost of gravity sewer systems is devoted to excavation, pipe supply, and installation. The remaining 15 per cent covers the cost of manholes. Henry and Ahern 1973, (13) estimated component costs in the combined sewer system for the case of no off-channel to be 67.9 per cent for sewer pipes, 20.5 per cent for manholes, and 11.6 per cent for catch basins. The second approach to estimate the cost of manholes is to propose a cost equation and determine its regression coefficients from current construction data. Merrit and Bogan (23) assumed a linear relationship between cost and depth. Mays and Yen (22) considered the cost of a manhole to vary with the square of the depth of the manhole, which was determined by the lowest invert of the sewers joining the manhole.

Pumps: In pumped storm sewer systems, an additional term has to be added to the cost function, which is the cost

of the pumping station. A review of the literature shows
that all the proposed optimization design models for sewer
systems can handle only gravity sewer systems, i.e. no
pumping stations. Deb (8), in his optimization design model
for distribution systems, proposed a pumping cost function.
He expressed the capital cost of the pumping station as a
function of installed power as

$$C_{pump} = K (P)^m \qquad (2.6)$$

where

C_{pump} = capital cost of pumping station

P = power of installed pump

K = regression coefficient, and

m = exponent

Deb also has proposed for pumping station operating
costs, including labour, electricity, and maintenance, an
equation in the form

$$C_{oper.} = a Q + b H Q \qquad (2.7)$$

where

$C_{oper.}$ = operating costs per year

Q = rated discharge

H = rated head, and a and b are regression
coefficients.

It is obvious that the differences between a water pumping
station and a sewerage and storm water pumping station cannot
be overlooked. For example, the sewerage and storm water

pumping stations have wet wells in addition to the dry wells, while water supply pumps may be placed directly in line. It appears that there is a need for an appropriate cost model for pumping stations in storm sewer systems.

Damage Function

Deriving a function that is capable of predicting the damages resulting from adopting a specific design should be one of the major parts in an optimization design model. The first step of developing such a function is to identify the different components constituting the total damage. Burns et al. (5) studied the performance of the combined sewer system in the City of Winnipeg. They reported that a severe flooding problem was found in 50 per cent of the City, as a result of the exceedingly flat terrain, e.g. in the spring of 1974 about 30 per cent of the city experienced basement flooding during heavy rains.

James F. MacLaren Ltd. (20) presented an excellent survey on the problem of ponding in residential areas in Canadian cities. The different aspects of the problem considered in that survey were water ponding in back yards, water ponding at inlets, water entering basements, water just covering street, water rising above curbs, and manhole covers popping off. The above aspects of flood damage can be used as a basis for introducing new appropriate parameters to evaluate damage.

Yen (39) classified the damages into two types. The first type are the damages resulting from temporal flooding of

low land and basements, and interruption of traffic in case
of rain storm being more severe than the design storm. In
other words, it is property damages and inconvenience without
structural failure. The second type involves failure in
structural functioning such as overloading and damaging of
sewage pumps or treatment plants. It appears that the second
type has little contribution to the damages in urban areas.

A review of the approaches used in estimating flood
damages in flood-plains (rural areas) (14) might be useful
to develop a damage predicting model in urban areas. When
reducing the physical damages caused by a given flood event,
an estimate of average damage can replace a property-by-
property damage evaluation. The land market idea is used
for an estimate of the value of flood-plain land. Depth of
flooding is used to estimate the severity of flooding produced
by a given hydrograph. The total severity of a flood event
depends on the areal extent of flooding to each depth. After
estimating flood damages for several flood events with various
frequencies, a damage-frequency curve is plotted. The area
under the curve is the expected annual damage.

In 1975, Tang, Mays and Yen (34) presented an optimal
risk-based design of sewer networks. In their model, the
expected damage cost during the service period of the sewer
was evaluated as the product of the assessed damage value in
the event of a flood exceeding the sewer capacity, Q_c, and
the risk. The risk was defined as the probability of occur-
rence of a flood event exceeding the sewer capacity during

the service period of the sewer. To evaluate the risk, the safety factor, SF, is first computed by SF $= Q_c/Q_p$ where Q_p is the design discharge computed for a design return period equal to or less than the service period of the sewer. With a given value of SF, the corresponding risk can be obtained from the risk-safety factor curve corresponding to the service life of the sewer (41). Yen et al. (42) incorporated the previously discussed approach into their improved model, ILSD-2, for optimal sewer system design.

In the previous work of Tang et al. (34) and Yen et al. (42), the assessed damage cost in the event of insufficient capacity was assumed to be equal to some constant value regardless of the severity of flooding conditions. Moreover, it was assumed that only one flood with the largest inflow will contribute to the expected flood damages over the service period.

In 1976, Tang, Mays and Wenzel (33) developed a model for determining flood damage based on the flood volume-depth relationship for a specific street section and the flood damage-depth relationship for a specific neighbourhood. Hydrographs corresponding to several storm durations were considered. The hydrograph giving the maximum volume of flood water (Figure 2.3) was identified with the corresponding storm duration. Construction of a damage cost-flooding volume relationship is illustrated in Figure 2.4. To determine the depth-total damage cost curve for a given residential or business district, a set of flood-damage cost curves presented.

by Grigg et al. (33) were used. In these curves, flood depth was plotted as a function of the normalized damage (in terms of the fraction of total property value), for various classes of structures. The damage-frequency curves for various combinations of sewer sizes and slopes were plotted. The area under any of these curves is equal to the average annual expected damage if this specific combination of diameter and slope was chosen.

In the previously outlined approach, it was assumed that the excess flow, $Q_1 - Q_c$, Figure 2.3, remains on the ground surface. It is believed that developing a surcharged flow model to replace this assumption will yield a better simulation. Although basement flooding constitutes one of the major components of the total damage, backflow to basements (if any) was not considered by Yen et al. The flood damage was evaluated for each sewer individually neglecting the influence of the adequacy of the other sewers on the sewer considered.

Constraints

A review of sewer design practices shows that the following can be considered as the most commonly used constraints (3, 11, 42, 19):

(i) Free-surface flow exists for the design discharges or hydrographs.

(ii) The commercially available pipe sizes in inches are 8, 10, 12, from 12 to 30 with a 3 in. increment and from 30 to 120 with an increment of 6 in.

(iii) The design diameter is the smallest commercially available pipe that has a flow capacity equal to or greater than the design discharge and satisfies all the appropriate constraints.

(iv) Storm sewers must be placed at a depth that will not be susceptible to frost, will drain basements, and will allow sufficient cushioning to prevent breakage due to ground surface loading.

(v) The sewers are joined at junctions such that the crown elevation of the upstream sewer is no lower than that of the downstream sewer.

(vi) To prevent or reduce permanent deposition in the sewers, a minimum permissible flow velocity at design discharge or at near full-pipe gravity flow is specified. A survey by James F. MacLaren (20) of the Canadian design practices showed that the minimum design velocity ranges from 2 to 3 fps.

(vii) To prevent occurrence of scour and other undesirable effects of high velocity flow, a maximum permissible flow velocity is also specified. The MacLaren survey (20) showed that the maximum design velocity ranges from 10-15 fps.

(viii) At any junction or manhole, the downstream sewer cannot be smaller than any of the sewers of the upstream of that junction.

(ix) To avoid the difficulties and excessive expenses encountered in construction, in case of large excavation depths, a maximum depth of excavation is specified according

to the type of soil, ground water table, and the available methods of construction (11).

Optimization Algorithms

An optimization algorithm requires as an input, an objective function and a set of appropriate constraints, and gives as an output, an optimal design satisfying both the objective function and the technological constraints. Optimization algorithms may be classified into two basic groups, namely; linear and nonlinear programming algorithms, and algorithms using dynamic programming approach or any of its modified forms.

When the objective and constraint functions are linear, the required optimization is said to belong to the linear programming group (31). Specifically, the form

$$\text{Minimize} \quad z = c_1 x_1 + \ldots + c_n x_n \qquad (2.8)$$

Subject to

$$a_{11} x_1 + a_{12} x_2 + \ldots + a_{1n} x_n = b_1$$

$$a_{21} x_1 + a_{22} x_2 + \ldots + a_{2n} x_n = b_2$$

$$\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \qquad (2.9)$$

$$a_{m1} x_1 + a_{m2} x_2 + \ldots = a_{mn} x_n = b_m$$

and

$$x_j \geq 0, \quad j = 1, 2, \ldots, n \qquad (2.10)$$

is called the standard linear programming form. The function
to be minimized, Z, is called the objective function. The
variables x_j (j = 1, 2, ..., n) are called the design

(decision) variables. The quantities c_j, $b_i \geq 0$, and a_{ij}

(i = 1, ..., m and j = 1, ..., n) are assumed to be known

constants, and m, n are positive integers. The constants

b_i are conventionally non-negative, and the c_j and a_{ij} are

unrestricted in sign. Details of linear programming can be

found in standard textbooks (31, 9). A linear programming

problem can be solved by the Simplex Method which has been

programmed and is available in almost any computer's library.

   If the objective function or any of the technological

constraints is non-linear, the problem cannot be directly

solved using linear programming. Sometimes, it is possible

to transform the problem to a linear form to permit the use

of linear programming algorithms. Dajani et al. (6) and

Dajani and Hasit (7) incorporated in their optimization

models a piecewise linearization. This approach can only

be used with convex functions. Another problem, normally

encountered when using linear or non-linear programming is

that they yield impractical fractional pipe diameters.

Dajani and Hasit (7) solved the previous problem by intro-

ducing a new approach called "separable-convex mixed-integer"

programming. The latter approach obtains the optimal solution

in discrete values representing commercially available pipe

sizes. Lemieux et al. (17) developed a methodology to design

storm sewer systems using a nonlinear programming approach.
In their model, optimization was achieved with the Rosen's
projected gradient method. Gupta et al. (11) used Powell's
method of conjugate directions to minimize a nonlinear cost
function subject to a set of nonlinear constraints.

The second group of optimization algorithms uses the
dynamic programming, DP, technique. Dynamic programming is
a technique specially designed for analyzing multiple stage
processes. Some investigators have assumed that sewer
design is a sequential decision process; hence, they
incorporated dynamic programming in their models, e.g. Walsh
and Brown (37), Merritt and Bogan (23), Dajani and Hasit (7),
Tang et al. (34), Yen et al. (42), Tang et al. (33), and Mays
and Yen (22). A sewer system consists of sewers connected
by manholes. The DP computations start at the upstream end
(stage 1) of the sewer system and proceed downstream stage
by stage. For each stage, several combinations of design
variables satisfying the constraints are tested searching
for the optimal combination for that stage.

Dynamic programming has the flexibility to handle any
objective function or constraint equations regardless of
its linearity or non-linearity. Moreover, it gives optimal
solutions with commercial pipe sizes. When DP is applied to
large systems, there are difficulties in obtaining an accurate
optimal solution because of excessive computer time and
storage requirements (12). To overcome these difficulties,
several techniques based upon DP have been developed including

successive approximation, state increment dynamic programming,
and discrete differential dynamic programming, DDDP (22). DDDP
has been found to be efficient in obtaining the optimal
design with reasonable computer time and storage requirements.
Details of DDDP applied to sewer design have been presented
elsewhere (34, 42, 33, 22).

2.5 Assessment of the Rational Method

The Rational Method is the simplest of all models used
in the design of urban drainage systems. Although the urban
hydrograph runoff models present better simulation of the
rainfall-runoff problem, the Rational Method continues to
be used by most of the consultants and municipalities in
Canada and the United States. A survey conducted by James F.
MacLaren Ltd. (20) concerning the methods used for runoff
computation showed that of the 37 cities considered in the
study, 36 used the Rational Method, and one, Sainte-Foy,
used the McMath formula. A comparison of Rational Method
peak flows with measurements has been reported in Reference
20 for different watersheds with varying sizes. The results
indicated that the Rational Method is inconsistent and not
appropriate for the simulation of runoff from a real storm
event.

In comparing the Rational Method with the runoff hydro-
graph methods, it is possible to present the following
comments:

(i) The Rational Method does not take into account the

time distribution of the rainfall.

(ii) The runoff coefficient 'C' lumps all the characteristics of the system, such as the degree of imperviousness (which is exclusively a subcatchment parameter), with infiltration losses, depression, and detention storage, which are also a function of the rainfall characteristics.

(iii) It gives only peak flows, while runoff hydrograph methods give complete hydrographs at any desired point in the system and permit the investigation of storm water management techniques such as storage and ponding.

(iv) While computers have made the development and implementation of hydrograph runoff models possible, the Rational Method is not improved when computerized (16).

Several attempts have been made to improve the Rational Method but apparently none of these methods is widely used in Canada at present (20). For example, Schaake et al. (30) presented empirical equations for computing the values of C and the characteristic "rainfall intensity averaging time" from the physical characteristics of the drainage area. Rogers (19) presented the "Rational 'Rational' Method of storm drainage design" in which he utilized the Rational Formula with a modification to allow for nonuniform runoff. It appears that real progress can only be made by replacing an empirical method by a model that attempts to simulate the physical phenomena.

2.6 Review of New Design Trends

Urban drainage practices in North America are in a transition stage. Although traditional methods continue to be applied on a large scale, there are also many attempts to apply new concepts such as off-channel storage, and runoff detention (20, 13). Rainfall-runoff models are essential for examining these new concepts.

One of the new trends which has proved to be efficient in reducing peak flows from runoff is to provide for controlled ponding. Parking lots may be used to provide surface storage with little inconvenience to the parking lot users. This can be accomplished by sizing the drains to allow runoff from the more frequent minor storms to pass unimpeded to the sewer system. Various degrees of ponding may be allowed in malls, plazas, and on rooftops. Artificial ponds and lakes can be incorporated into proposed parkland and green-belt areas to act as retention basins; moreover, the presence of water in or near a development is very desirable from both aesthetic and recreational viewpoints. Disadvantages of the latter on-site storm water detention facility have been found to be related to maintenance and operational problems. In some cases a constructed facility such as an underground tank may be feasible.

A large part of the stormwater runoff enters sewers by way of catch basins into which street gutters discharge. It may be feasible to incorporate storage at catch basins for temporary detention of street runoff before release to

sewers.

Reduction of urban runoff can also be accomplished by lowering the degree of imperviousness. This is achieved by various means such as requiring roof leaders to drain to lawns, using porous pavements where climate permits, grassed watercourse, gravel-filled channels and seepage pits, and the preservation of natural ravines and green belts. Side effects of new practices require careful consideration. One possible effect might, for example, be a higher ground water table which could increase infiltration in sanitary sewers.

CHAPTER 3

MATHEMATICAL ASPECTS OF THE MODEL

3.1 General

The major task involved in building a model is to organize the available algorithms to achieve the goal of the model. The procedure normally followed to build a model can be divided into a number of steps. The first step is to define the objective of the model. In the present model, the objective is to arrive at the optimal design of pumped storm sewer networks. The optimality is achieved when the total cost of installation and expected damage is minimum. The second step is to draw a flow chart that specifies the various stages along the input-output path, and the required algorithms for each stage. For example, an appropriate algorithm is necessary to transform the rainfall to the inlet inflow hydrographs (see Section 2.1.1).

The third step is to review the literature searching for the available algorithms that will perform the required role. The accuracy and degree of sophistication in the selected methodology should be consistent with the objective of the model. For example, incorporating a highly sophisticated hydraulic routing technique in an optimization design model cannot be justified. Instead, it may be more efficient

to use an appropriate routing technique with less accuracy but with shorter computer time.

It is obvious that each model has to incorporate certain new algorithms, which present better functioning than the existing algorithms. These specially developed algorithms as well as the objectives of the model determine its original-ity and distinction among other models.

The last step before using the model for practical applications is to calibrate and validate the model (see Section 2.1.2). For simulation models, calibration is done by comparing computed and measured hydrographs at the outfall of test areas. For design models, calibration is done by designing the drainage system of a catchment by the model, and comparing the results with the design of another approp-riate model for the same catchment. In some instances, algorithms incorporated in a design model may be tested against other methods which have been known to give relatively accurate solutions.

Figure 3.1 shows a schematic diagram of the model. The mathematical aspects of the model will be illustrated in this chapter in the sequence in which they appear in the diagram.

3.2 Simulation

3.2.1 Design Rainfall Storm

The design rainfall storm incorporated in a design model may be a synthetic hyetograph or a storm specified by the municipality (see Section 2.2.1). The synthetic hyetograph,

generated from the intensity-duration curve for a certain
return period, appears to be more convenient from modelling
viewpoints. This approach requires, as an input, the
coefficients a, b, c used in Equation 2.2 and r which is
the relative advancement of the peak intensity. The co-
efficients a, b, and c of the intensity duration curve can
be easily obtained from rainfall records which are available
for most of the regions in North America. The relative
advancement of the peak intensity can be obtained by statis-
tical analysis of many rainfall events in or near the water-
shed. Based upon the previous discussion, it was decided to
incorporate the design hyetograph developed in Chicago by
Chu and Kiefer (Section 2.2.1).

To simplify the modelling problem, the same rainfall
was considered for all points in the system. Moreover, it
was assumed that the summer rain storms are the critical
type of storm. These two assumptions are reasonable for
small watersheds (Section 2.2.1).

3.2.2   Runoff Model (Basic Inflow Hydrographs)

Various approaches have been proposed to generate the
basic inflow hydrographs (see Section 2.2.2). They vary in
the degree of sophistication, the required input data, and
the computational time. For a design model, the selected
approach should have a minimum data requirement and computa-
tional time and it should be able to develop the inflow
hydrographs with reasonable accuracy. Considering the previous

requirements, it was decided to use a similar approach to that incorporated in the Road Research Laboratory Model (see Appendix I) for generating the inflow hydrographs.

This approach considers only runoff generated on impervious areas directly connected to the sewer system, and neglects the contribution from the pervious areas where infiltration and detention are usually high. The model assumes a runoff coefficient of 1.0 for the impervious areas (Appendix I). To find the impervious area contributing to each manhole, a map of the catchment is prepared showing all impervious areas that are connected to the sewer system.

The Area-Time Diagram: This diagram shows the impervious area contributing runoff to the design point (manhole in the present case, Figure 3.2) at any moment after the start of the storm. The graph starts at the origin of the Area and Time axes and gradually rises to a point corresponding to the total impervious area and the estimated time of concentration of the system (Figure 3.3.a). The time of concentration of the basic subcatchment consists of the inlet time and the travel time in sewer branches to the manhole considered. For simplicity and to avoid excessive input data, the model assumes a linear relationship between contributing area and time.

Generation of Basic Inflow Hydrographs: Figure 3.3 shows the development of the hydrograph from the area-time diagram, and the design storm hyetograph. The ordinates of the hydrograph are calculated from the series

$$Q_{inf}(IMH,K) = (i_k + i_{k-1} + i_{k-2} + \dots + i_{k-n}) \, \Delta A$$

$$(3.1)$$

where

$Q_{inf}(IMH,K)$ = runoff (cfs) at time $k.\Delta t$;

i_m = average rainfall intensity (in/hr)

during the time increment starting $(m-1)$ Δt and ending (m) Δt;

ΔA = increment of impervious area for a time increment Δt, contributing to junction manhole IMH, acres; and n = number of increments of impervious area, contributing to junction manhole IMH;

= impervious area contributing/time increment.

3.2.3 Transport Model

The function of this model is to simulate the flow in the sewer system. For storms less severe than the design storm, the flow in the sewer is gravity flow. On the other hand, if one or more of the sewers are under-designed, or in the case of storms more severe than the design storm, the flow switches to surcharged flow. Also if the pump is under-designed premature surcharge could occur. For a surcharged flow, if the total energy is higher than the basement levels, or the ground surface at the manholes, there will be backflow to basements or to the streets. Based on the previous discussion, the transport model should have the capabilities to simulate the gravity flow, detect switching, and simulate

the surcharged flow considering backflow to basements (if any) and to the ground surface.

3.2.3.1 Gravity Flow Model

Several approaches have been proposed to route the inflow hydrographs through the sewer system. As previously discussed in Section 2.2.3, these approaches can be classified into two basic groups, namely; approaches using hydrologic routing techniques, and approaches solving the continuity equation together with various simplifications of the momentum equation, i.e. dynamic routing. Considering that the main objective of the present model is to optimize the design of a pumped sewer system, and based on the comparison between the two approaches in Section 2.2.3, it was decided to incorporate a hydrologic routing technique in the model. This selection is also supported by the fact that if damage is being considered, the surcharged flow model will be more important than the gravity flow model in predicting the performance of the system.

A modified Muskingum Method (Section 2.2.3) has been adopted as part of the transport model. One merit of the Muskingum Method is that it takes into account the inflow in addition to the outflow in computing the storage, i.e. it includes wedge and prism storage as shown in Figure 3.4. The total storage is

$$S = K\ O + Kx\ (I - O) \tag{3.2}$$

which may be rewritten in a finite difference form,

$$S_2 - S_1 = K \left[x(I_2 - I_1) + (1 - x) (O_2 - O_1) \right] \qquad (3.3)$$

where

    S = storage within the control volume;

    I = rate of inflow into the control volume;

    O = rate of outflow from the control volume;

    K = storage coefficient;

    x = routing parameter.

Subscripts 1, 2 indicate the variables at the beginning and end of the time increment. Combining Equation 3.3 with the continuity equation in the following form,

$$\frac{\Delta t}{2} (I_1 + I_2) = \frac{\Delta t}{2} (O_1 + O_2) + (S_2 - S_1) \qquad (3.4)$$

results in the following equation

$$O_2 = C_o I_2 + C_1 I_1 + C_2 O_1 \qquad (3.5)$$

where

$$C_o = - \frac{Kx - 0.5 \Delta t}{K - Kx + 0.5 \Delta t} \qquad (3.6)$$

$$C_1 = \frac{Kx + 0.5 \Delta t}{K - Kx + 0.5 \Delta t} \qquad (3.7)$$

$$C_2 = \frac{K - Kx - 0.5 \Delta t}{K - Kx + 0.5 \Delta t} \qquad (3.8)$$

C_o, C_1, C_2 are called the Muskingum Coefficients.

    One restriction on the Muskingum Method is that the slope of the sewers has to be significantly greater than zero, otherwise the method is not applicable. The parameter x

used in computing the Muskingum's coefficients depends on the limb of the hydrograph, i.e. rising limb or recession limb, and on the downstream conditions. The model assumes a default value of 0.5 for x .

The coefficient K represents the ratio of storage to weighted discharge in the reach, and has the dimension of time. The techniques used in river routing do not apply for flow in storm sewers because measurements are not available for each reach of sewer. A simplifying assumption was made to find the value of K . Figure 3.5 shows the relationship between the relative storage, S/S_o, and the relative discharge, Q/Q_o, where S_o and Q_o, are the storage within the sewer when flowing full, and the full flow capacity of the sewer. Approximating the graph relating S/S_o and Q/Q_o by a straight line of slope unity, i.e.

$$S/S_o \ = \ Q/Q_o \qquad \qquad (3.9.a)$$

or

$$S \ = \ (S_o/Q_o) \ Q \qquad \qquad (3.9.b)$$

But $\qquad \qquad S \ = \ KQ$

hence $\qquad \qquad K \ = \ S_o/Q_o \qquad \qquad (3.10)$

Substituting for $\qquad S_o \ = \ \frac{\pi}{4} \ D^2 \ell \ ,$

and $\qquad \qquad \qquad Q_o \ = \ \frac{1.49}{n} \ (\frac{\pi}{4} \ D^2) \ (\frac{D}{4})^{2/3} \ S_o^{1/2}$

then the value of K can be obtained for each sewer using

th e following equation,

$$K_i = \frac{1.696 \, n \, \ell_i}{D_i^{2/3} \, S_{o_i}^{1/2}}$$ (3.11)

where

ℓ_i = length of the i-th sewer, ft;

D_i = diameter of the i-th sewer, ft;

n = Manning's Coefficient;

and S_{o_i} = invert slope of the i-th sewer.

3.2.3.2 Surcharged Flow Model

(i) Definitions and Assumptions: Figure 3.6 illustrates a typical sewer section showing i-th sewer i , connecting manhole MNH(i) at the upstream end and manhole MNH(i+1) at the downstream end. It is assumed that ground surface slopes linearly from elevation H_i at MNH(i) to elevation H_{i+1} at MNH(i+1). Also, it is assumed that the basements, if any, are located at an average depth m_i from the ground surface, i.e. the locus of the basement floors (basement grade line) has the same slope, S_{B_i} , as the ground surface slope,

$$S_{B_i} = \frac{H_i - H_{i+1}}{\ell_i}$$ (3.12)

where ℓ_i is the length of the i-th sewer. S_{B_i} is positive when the ground surface slopes downwards while proceeding downstream.

During surcharged flow, the slope of the energy line may no longer·be equal to the sewer slope. In that case, the model evaluates the energy slope, S_e, using Manning's Equation in the following form:

$$S_e(i,t) = \frac{4.66 \ n^2}{D_i^{16/3}} \ [Q_{av}(i,t)]^2 \qquad (3.13.a)$$

or

$$S_e(i,t) = K_{e_i} \ [Q_{av}(i,t)]^2 . \qquad (3.13.b)$$

Hence,

$$K_{e_i} = \frac{4.66 \ n^2}{D_i^{16/3}} \qquad (3.14)$$

where

$S_e(i,t)$ = energy line slope of the i-th sewer, at the t-th time index;

$Q_{av}(i,t)$ = average flowrate in the i-th sewer, during the t-th time interval;

K_{e_i} = coefficient of energy line slope of the i-th sewer;

t = time index representing the interval $t-\Delta t$ to t.

Equation 3.13.b gives the slope of the energy line of the i-th sewer, provided that the average rate of flow is known. The average rate of flow in the i-th sewer can be determined from the mass balance of the inflow to and outflow from the system. Figure 3.7.a shows the different components

constituting the inflow and outflow, namely, basement flows, Q_B, street (ground surface) flows, Q_{st}, and the basic inflow hydrographs, Q_{inf}. The equation used for mass balance is

$$Q_{av}(i,t) = \sum_{1}^{i} Q_{inf}(i,t) - \sum_{1}^{i} Q_{st}(i,t) - \sum_{1}^{i-1} Q_B(i,t)$$

$$- Q_B(i,t)/2 \qquad\qquad (3.15)$$

where

$Q_{inf}(i,t) = $ inflow runoff (cfs) at the upstream end of the i-th sewer, i.e. at MNH(i);

$Q_{st}(i,t) = $ street flow (cfs) at MNH(i);

$Q_B(i,t) = $ basement flow (cfs) along the i-th sewer.

The summation of $Q_{inf}(i,t)$ for manholes 1 to i is assumed to be the sum of Q_{inf} for each manhole at time t .

Both basement flows and street flows may be either outflow (positive with regard to Equation 3.15) or inflow (negative). At the beginning of the surcharged flow condition, storm water collects into the basements and on the ground, i.e. the basement and street flows are considered as outflow from the system. During the recession limb of the hydrograph, storm water from streets and basements flows back into the sewer system as inflow.

Basement flows and street flows are functions of the respective positions of energy line, basement grade line and ground surface elevation at manholes. Based upon the

previous discussion, it can be seen that the different

varialbes, energy line, Q_B and Q_{st} are interdependent and

none can be expressed explicitly in terms of the others.

The model assumes an initial solution and iterates to obtain

an improved solution.

(ii) <u>Construction of the Energy Line of the System</u>: To

simplify the simulation problem, the model assumes that

energy losses at manholes (junctions) are negligible. Also,

the model approximates the actual energy line by an equivalent

straight line of slope, S_e, computed from Equation 3.13.b

using the average rate of flow along a particular reach of

the sewer. The construction of the energy line of the sewer

system is illustrated in Figure 3.7, and Figure 3.8.

The solution starts from the downstream end of the

outfall sewer energy line, point A, Figure 3.7.b, and

proceeds upstream obtaining points B, C, and so on. Points

A, B, ... etc. are the elevations of the energy line at

manholes n, n-1, n-2, ... etc. at time index t . The model

sets the basement and street flows equal to their values

at the previous time step, t-1. Point A is obtained using

the sump-pump model for time t-1, and is used as an initial

solution. Then, the model computes the average rate of flow

for the i-th sewer using the assumed values of Q_B, Q_{st}.

Equation 3.15.b is then used to find the energy slope, $S_e(i,t)$,

which subsequently will be used in the following equation

$$USEL_i = USEL_{i+1} + S_e(i,t) \, \ell_i \qquad (3.16)$$

where $USEL_i$, $USEL_{i+1}$ are the elevations of the energy line

at the upstream end of sewers i , and i+1 respectively.

After obtaining all the points defining the energy line

A, B, ..., the model computes basement and street flows

using the constructed energy line and incorporates the

refined values in a new iteration. A similar iterative

procedure is necessary for improving the starting point A

by substituting its initial value with another refined value

evaluated by the sump-pump model using the most recent values

of the variables involved.

(iii) Basement Flows

In some urban areas, basements are drained through

house connections to separate storm sewers. Check valves

may be used to protect basements against backflow but the

performance of these valves cannot be relied upon. The model

assumes that these valves are either not existing or are not

working properly. Figure 3.9.a shows a schematic drawing

of a typical house connection. Also, rooftops may be drained

to foundation drains which subsequently are connected to storm

sewers.

Description of Basement-Flooding Process

If the elevation of the energy line along the storm

sewer, point A, Figure 3.9.b, exceeds the basement floor

elevation at a particular house connection, point B, the storm

water is assumed to start flowing from storm sewer, through

the house connection, into the basement (see Figure 3.9.b).

In this case, the basement flow is a function of the difference

in elevation between A and B. The storm water collects in the basement and its depth is called basement "flooding depth," d_f. The water surface in the basement, point C, Figure 3.9.b, represents the total energy level in the basement. In this case, the basement flow is a function of the difference between points A and C; instead of A and B. In the recession limb of the hydrograph, point C exceeds point A in elevation and consequently the basement discharges the collected water into the storm sewer.

Development of Basement Flow Model

To-simplify the modelling problem, the model assumes a uniform flooding depth for all the basements associated with a particular sewer section. This assumption means that the storm water flowing into all the basements will be accumulated and distributed uniformly over the entire area of basements, A_{base}. In other words, after time t , the flooding depth can be obtained from

$$d_f(i,t) = \frac{\sum_{t=k}^{t} Q_B(i,t) \, \Delta t (60)}{A_{base_i} (43560)}$$ (3.17)

where :

$d_f(i,t)$ = flooding depth of the basements associated with the i-th sewer at the end of the t-th time step, ft;

$Q_B(i,t)$ = average rate of basement flow for i-th sewer during the t-th time step, cfs;

Δt = time increment, min.;

A_{base_i} = total basements area associated with the i-th

sewer, acres;

k   = first time step index of the surcharged flow condition.

Figure 3.10 illustrates the possible cases of the respective positions of the sewer energy line, and the locus of the basement total energy levels (basement energy line). Also, it shows that basement energy line has the same slope as the basement grade line, and lies above it by a value equal to d_f. Before proceeding, a new parameter which is

the flooding ratio, F, should be introduced. It is the ratio between the length of the sewer actively being flooded and its total length. For example, in Figure 3.10.b, F is equal to unity because the sewer energy line is higher in elevation than the basement energy line over the whole sewer length. The development of the basement flow model will be divided into three parts, computation of flow through a single house connection, computation of total basement flows along a particular sewer section, and determination of the flooding ratio, F(i,t).

Computation of Flow Through a Single House

Connection: To compute the flow through a single house connection, $Q_{s.house}$, the energy equation is applied between points A and C, Figure 3.9.b. The difference in elevation between points A and C is the head loss, h_ℓ, in the house connection. This head loss is the sum of the frictional head

loss and the secondary losses along the path A-C. The energy equation, the continuity equation, and the Darcy-Weisbach equation for head loss by pipe firction, when combined together, may be used to derive an equation to compute, $Q_{s.house}$, in the form

$$Q_{s.house} = A_{pipe} \sqrt{\frac{2 g h_\ell}{K_m + \frac{C_f \ell_{h.con.}}{d_{h.con.}}}} \qquad (3.18)$$

where

$Q_{s.house}$ = flow in house connection to the basement of a single house, cfs;

A_{pipe} = cross sectional area of the pipe of the house connection, ft²;

$\ell_{h.con.}$ = average length of house connections within a particular sewer section, ft;

$d_{h.con.}$ = diameter of the pipe of the house connection, ft;

h_ℓ = total head loss through a particular house connection, which is equal to the difference in elevation between points A and c, ft;

K_m = minor loss coefficient = 2 (default value);

C_f = coefficient of friction of the pipe of the house connection = 0.03 (default value)

g = gravitational acceleration, ft/sec².

Computation of Total Basement Flows Along a Sewer Section

Case (a) F < 1.00

Figure 3.10.a shows the sewer energy line and the basement energy line for the i-th sewer, during the t-th time step. If the number of houses along the i-th sewer section is, N_{house}, and the length of the i-th sewer is ℓ_i, then, the basement flow per unit length of the sewer, Q_x, see Figure 3.10, can be obtained from

$$Q_x = \frac{(Q_{s.house})_i \; (N_{house})_i}{\ell_i} \tag{3.19}$$

Integrating Q_x over the sewer length $F(i,t)\ell_i$ to obtain the positive basement flow, i.e. storm water flowing from the storm sewer into the basements, gives

$$Q_B(i,t)(+ve) = \int_0^{F(i,t)\ell_i} Q_x \, dx \tag{3.20}$$

Substitution of Equations 3.18, 3.19 into 3.20 yields

$$Q_B(i,t)(+ve) = \sqrt{\frac{2g}{K_m + \frac{C_f \, \ell_{h.con.}}{d_{h.con.}}}} \; \frac{A_{pipe}(N_{house})_i}{\ell_i} \int_0^{F(i,t)\ell_i} \sqrt{h_\ell} \, dx \tag{3.21}$$

but, h_ℓ can be expressed as

$$\frac{h_\ell}{USHL} = \frac{x}{F(i,t)\ell_i} \tag{3.22}$$

where USHL is the total head loss in a house connection at the upstream end of the sewer. Substituting Equation 3.22 into 3.21 gives

$$Q_B(i,t)(+ve) = \frac{2}{3} F(i,t) (N_{house})_i \left\{ A_{pipe} \sqrt{2g} \sqrt{\frac{USHL}{K_m + \frac{C_f \ell_{h.con.}}{d_{h.con.}}}} \right\} \tag{3.23}$$

Similarly, the negative basement flow, $Q_B(i,t)(-ve)$, can be obtained from

$$Q_B(i,t)(-ve) = \frac{2}{3} [1 - F(i,t)] (N_{house})_i \left\{ A_{pipe} \sqrt{2g} \sqrt{\frac{DSHL}{K_m + \frac{C_F \ell_{h.con.}}{d_{h.con.}}}} \right\} \tag{3.24}$$

where $Q_B(i,t)(-ve)$ is the negative basement flow, i.e. storm water flowing back from the basements into storm sewers, and DSHL is the total head loss in a house connection at the downstream end of the sewer. At the start of the surcharged flow conditions, the flooding depth is equal to zero, hence, the model assigns zero to the magnitude of the basement flow. Also, the model restricts the magnitude of the negative basement flow to a flow equivalent to the available volume of water in the basements in the length $[1 - F(i,t)]\ell_i$ in one time step, i.e.

$$Q_B(i,t)(-ve) \leq \frac{d_f(i,t)\ [1 - F(i,t)]\ (A_{base})_i}{\Delta t} \qquad (3.25)$$

After computing $Q_B(i,t)(+ve)$ and $Q_B(i,t)(-ve)$, the model obtains $Q_B(i,t)$ from

$$Q_B(i,t) = Q_B(i,t)(+ve) - Q_B(i,t)(-ve) \qquad (3.26)$$

Case (B) F = 1.00

Figure 3.10.b shows the respective positions of the sewer energy line and the basement energy line in the case of $F(i,t) = 1.00$. Using a similar approach to that used in the case of $F(i,t) < 1.00$, the following equation was derived to compute $Q_B(i,t)$, equal to $Q_B(i,t)(+ve)$,

$$Q_B(i,t)(+ve) = \frac{2}{3}\ (N_{house})_i \cdot \left\{ A_{pipe} \sqrt{\frac{2g}{K_m + \frac{C_f\ \ell_{h.con.}}{d_{h.con.}}}}\right.$$

$$\left. \frac{(USHL)^{3/2} - (DSHL)^{3/2}}{USHL - DSHL} \right\} \qquad (3.27)$$

To simplify the problem, it was decided to consider the length of the sewer actively being flooded, to start from point D instead of the true point E as shown in Figure 3.10.a. This simplifying assumption has resulted in a few minor cases in which neither Equation 3.24 nor Equation 3.27 with their conditions can be directly used. Figures 3.10.c, 3.10.d and 3.10.e illustrate these cases which are normally encountered at the beginning or end of the surcharged flow

conditions.

Case (C) Figure 3.10.c

In this case, the computed $F(i,t)$ is equal to zero, and the basement energy line is higher in elevation than the sewer energy line. The model uses Equation 3.27 to compute $Q_B(i,t)$, which is considered negative in this case.

Case (D) Figure 3.10.d

In this case, the computed $F(i,t)$ is greater than zero and less than unity, and the basement energy line exceeds, in elevation, the sewer energy line. As in case (C), Equation 3.27 is used to compute $Q_B(i,t)$ (-ve).

Case (E) Figure 3.10.e

This case is treated the same as case (A) except that the true $F(i,t)$ is computed using one of the following two equations:

$$F(i,t) = \frac{DSHL}{|USHL| + DSHL} \qquad (3.28)$$

or

$$F(i,t) = \frac{USHL}{USHL + |DSHL|} \qquad (3.29)$$

Determination of Flooding Ratio, $F(i,t)$: Flooding ratio, $F(i,t)$, is defined as the ratio between the length of the sewer actively being flooded and its total length. The flooding ratio is a function of the respective positions of the sewer energy line and the basement grade line. It can be determined if the following four variables are known:

mid-point elevation of basement grade line, B_i, mid-point elevation of sewer energy line, $E(i,t)$, slope of basement grade line, $(S_B)_i$, and slope of sewer energy line, $S_e(i,t)$.

Case (I), Figure 3.11.a

This case is characterized by $S_e(i,t) > (S_B)_i$ and $E(i,t) > B_i$. The value of $F(i,t)$ is obtained from

$$F(i,t) = 0.5 + \frac{E(i,t) - B_i}{\ell_i [S_e(i,t) - (S_B)_i]} \qquad (3.30)$$

Case (II), Figure 3.11.b

In this case, $S_e(i,t) < (S_B)_i$ and $E(i,t) < B_i$. The model uses the following equation to compute $F(i,t)$, viz.

$$F(i,t) = 0.5 - \frac{E(i,t) - B_i}{\ell_i [S_e(i,t) - (S_B)_i]} \qquad (3.31)$$

Case (III), Figure 3.11.c

In this case, $S_e(i,t) > (S_B)_i$ and $E(i,t) < B_i$ and Equation 3.30 is used to compute $F(i,t)$.

Case (IV), Figure 3.11.d

This case is characterized by $S_e(i,t) < (S_B)_i$ and $E(i,t) > B_i$. The value of $F(i,t)$ is obtained by using Equation 3.31.

Case (V), Figure 3.11.e

This case is encountered when $(S_B)_i$ is negative, i.e.

sloping upward when moving downstream. To obtain the value of F(i,t), Equation 3.30 is used.

Case (VI), Figure 3.11.f

In this case, the sewer energy line is higher in elevation than the basement grade line over the whole sewer length, hence, F(i,t) is equal to unity. The model detects this case and if the computed value of F(i,t) exceeds unity, then it resets the value of F(i,t) equal to unity.

(iv) Street Flows

During the Rising Limb of the Hydrograph: After the switching of the flow to a surcharged flow condition, the model checks the energy levels at the manholes at each time step. If the energy level, USEL(i), exceeds in elevation the ground surface, H(i), at a particular manhole, MNH(i), the model assumes that the storm water starts to flow onto the ground surface at this manhole with a rate of flow $Q_{st}(i,t)$. Otherwise, the model proceeds to the next upstream manhole, considering that Q_{st} at MNH(i) equals to zero.

The model computes the storm water flowing upward through the i-th manhole by applying a mass balance equation at the manhole. Figures 3.12 and 3.13 illustrate the procedure of computation. The mass balance equation used is in the form

$$Q_{st}(i,t) = Q_{in} - Q_{ot} \tag{3.32}$$

where Q_{in} is the inflow to MNH(i), and Q_{ot} is the outflow

from MNH(i). The model computes Q_{in} using a similar equation

to Equation 3.15 in the form

$$Q_{in} = \sum_{i=1}^{i} Q_{inf}(i,t) - \sum_{i=1}^{i-1} Q_B(i,t) - \sum_{i=1}^{i-1} Q_{st}(i,t) \qquad (3.33)$$

and computes Q_{ot} from

$$Q_{ot} = Q_{av}(i,t) + Q_B(i,t)/2 \qquad (3.34)$$

To compute $Q_{av}(i,t)$, the model assumes that the energy level

at any manhole where street flow occurs is equal to the

ground surface elevation at this manhole. This assumption

means that the storm water coming out of the manhole is

distributed over the ground surface with shallow depth such

that its energy level can be approximated by the elevation

of the ground surface at the manhole. Hence, the model sets

the energy level at the manhole, USEL(i), equal to the ground

surface elevation, H_i, and using the energy level at the down-

stream end of the i-th sewer, $DSEL_i$, computes the corrected

energy slope, $S_e(i,t)$.

$$S_e(i,t) = \frac{H_i - DSEL_i}{\ell_i} \qquad (3.35)$$

The resulting $S_e(i,t)$ is then used to compute $Q_{av}(i,t)$.

<u>During the Recession Limb of the Hydrograph</u>: If
the energy level at a particular manhole drops down below
the ground surface elevation, the storm water collected on
the ground surface, V_{st}, starts to flow back through the
manhole into the sewer system. In this case, the street
flow is considered inflow to the system instead of being
outflow, i.e. has a negative sign. The exact simulation of
the street flow in this case is difficult and necessitates
routing of the flow over the irregular ground surface.
Considering this difficulty, and realizing that the reces-
sion limb of the hydrograph is not critical in damage
evaluation, it was decided to follow a simplified approach.

The model assumes no contribution from $V_{st}(i-1,t)$
or $V_{st}(i+1,t)$ to MNH(i), where $V_{st}(i,t)$ is the collected
storm water on the ground surface at the i-th manhole,
computed from

$$V_{st}(i,t) = \sum_{t=k}^{t} Q_{st}(i,t)\ \Delta t \qquad (3.36)$$

The model considers a characteristic flooding depth d_{s_i} at
the inlets contributing to a particular manhole and it is
assumed that

$$V_{st} \quad \alpha \quad d_{s_i}^3 \qquad (3.37)$$

The flow of storm water into the inlets can be simulated
using a simple weir relationship between head and discharge,

viz,

$$Q_{st} \quad \alpha \quad d_{s_i}^{3/2} \qquad (3.38)$$

Based upon the previous two relationships, the negative
street flow can be computed from

$$Q_{st}(i,t)(-ve) \quad = \quad C_{B_i} \sqrt{\Psi_{st}(i,t)} \qquad (3.39)$$

where C_{B_i} is a coefficient that depends upon the inlets used

and the topography around the inlets. The model proposes a

value of 0.01 for C_{B_i} for a sloping intersection, and a

value of 0.02 for an intersection in a depression.

3.2.3.3 Switching Criteria

Gravity flow and surcharged flow models have been
developed in the previous two sections. It has been shown
that the model routes the flow through the sewer system time
step by time step. The model has to detect for each time
step the condition of flow, gravity or surcharged. The
conventional procedure to differentiate between gravity and
surcharged flow is to compare the actual flow in the sewer
with its flow capacity. If the actual flow exceeds the sewer
capacity then the flow is surcharged, otherwise it is
gravity flow. In the present model, this procedure cannot
be used because both gravity flow and surcharged flow models
were developed to handle only one type of flow and it is
not necessary that all the sewers switch from gravity flow

to surcharged flow in the same time step.

To arrive at an appropriate procedure that can be used to detect the flow condition for each time step, it was decided to consider the actual storage in the sewers instead of the actual flow. Figure 3.14 shows a simplified flow chart for the switching procedure. For each time step, in case of gravity flow, the model computes the actual storage in each sewer using Equation 3.2 and the total actual storage, V_{act}, in the system. If the total maximum available storage, neglecting manholes storage, is V_{sys}, then a parameter, R_{stg}, defined as

$$R_{stg} = V_{act}/V_{sys} \qquad (3.40)$$

may be used to govern switching. The total maximum available storage, V_{sys}, is computed from

$$V_{sys} = \sum_{i=1}^{n} (\frac{\pi}{4} D_i^2) \ell_i \qquad (3.41)$$

where n is the number of sewers. If R_{stg} exceeds a certain value, theoretically 1.00, then, the sewers are flowing full and the flow is surcharged. The program compares R_{stg} with a default value of 0.98.

If there is inadequate pumping capacity for the outflow from the outfall sewer, premature surcharge could occur. In this case, the water level in the sump rises and submerges the downstream end of the outfall sewer possibly creating a

surge that would move upstream. The model assumes that
gravity flow predominates, ignoring the effect of any
surge, so long as the relative storage at the time step
under consideration is less than the critical limit.

3.2.3.4 Sump-Pump Model

The main function of this model is to route the outfall
hydrograph through the sump-pump system. Also, it provides
the information about the performance of the pumping system
that is required to improve its design. In the surcharged
flow model it was shown that the solution starts from the
energy level immediately upstream of the sump, point A,
Figure 3.7.a. Point A can be determined only if the water
level in the sump is known. If the water level in the sump
is below the crown elevation of the outfall sewer, the
elevation of point A, E_{sump}, is not function of the sump
water level, instead it is computed from ;

$$E_{sump} = E_{crown} + V_n^2/2g \qquad (3.42)$$

where E_{sump} is the energy level immediately upstream of the
sump at point A; E_{crown} is the crown elevation of the down-
stream end of the outfall sewer n, and V_n is the velocity in
the n-th sewer which can be computed from

$$V_n = Q_{out_n} / A_n \qquad (3.43)$$

where Q_{out_n} is the flowrate at the downstream end of the

n-th sewer and A_n is the cross-sectional area of the n-th

sewer. On the other hand, if the sump water level, SWL,

exceeds in elevation E_{crown}, Equation 3.42 is used to

compute E_{sump} after replacing E_{crown} by SWL.

Figure 3.15 shows a typical arrangement of the wet

well (sump) and the pumping units. When the storm water

flows into the sump, its water surface rises. If the sump

water level exceeds the starting elevation(s) of the pump(s),

it actuates an automatic control that starts the pump(s) (24).

At the beginning of the rising limb of the hydrograph, the

pumping flowrate Q_p is greater than the inflow to the sump,

Q_s. In this case, the sump water level draws down until it

falls below the stopping level, and it stops. In practice,

several pumping units are used, and consequently, there are

more than one set of starting and stopping levels. To

simplify the modelling problem and to reduce the input data

required to specify the design and operation policies, it

was decided to consider one pump (equivalent), and one

control level (minimum sump water level). This control

level is assumed to be the average of the starting and stopping

levels.

The model assumes constant-speed pumps and constant

mechanical power supplied by the motor, hence,

$$\text{Brake Horse Power} = \gamma_w Q_R H_R / \eta_R$$

$$= \gamma_w Q_p H_p / \eta_p \qquad (3.44)$$

where η_p is the pump efficiency (overall); Q is the pumping flowrate; H is the pumping head; γ_w is the specific weight of the storm water and subscripts R and p denote the variables at the rated condition at maximum efficiency and operating conditions respectively. Equation 3.44 may be rewritten in the form

$$Q_p = C_p (1/H_p) \qquad (3.45)$$

where C_p is a parameter defined as,

$$C_p = \frac{\eta_p}{\eta_R} \cdot H_R Q_R$$

$$= \eta_{rel} H_R Q_R \qquad (3.46)$$

The model assigns a default value of 0.95 to η_{rel}. A reservoir routing approach is used to determine the sump water level at any time index t. The continuity equation applied for the sump may be written in a finite difference form as follows

$$\frac{1}{2}(Q_s^t + Q_s^{t-1}) - \frac{1}{2}(Q_p^t + Q_p^{t-1}) = \Delta S / \Delta t \qquad (3.47)$$

where ΔS is the incremental change in the storage within the sump or from Figure 3.15 -.

$$\Delta S = A_s \, \Delta Y_s \qquad (3.48)$$

but

$$\Delta Y_s = -\Delta H_p = -(H_p^t - H_p^{t-1}) \qquad (3.49)$$

where

A_s = sump cross sectional area; and

Y_s = sump water depth above minimum sump water level

From Equations 3.45, 3.46, 3.47, and 3.48, the following equation can be obtained

$$H_p^t = -\frac{1}{2} B + \frac{1}{2} \sqrt{B^2 + 4 C} \qquad (3.50)$$

where

$$B = -H_p^{t-1} + \frac{C}{C_p} \, (Q_s^t + Q_s^{t-1}) - \frac{C}{H_p^{t-1}} \qquad (3.51)$$

and

$$C = \frac{C_p}{2 A_s} \, \Delta t \qquad (3.52)$$

Knowing the pumping head, H_p^t, permits the computation of sump water level, SWL, by

$$SWL = E_d - H_p^t \qquad (3.53)$$

where E_d is the energy level on the delivery side of the pump which may be the elevation of the water surface of the receiving body.

Also, Q_s in Equation 3.51 is computed from

$$Q_s^t = Q_{out}(n,t) + Q_{inf}(n+1,t) \qquad (3.54)$$

where Q_{inf} (n+1,t) is the inflow from sub-catchment (n+1),

contributed directly into the sump well.

3.3 Optimization

3.3.1 General

The purpose of this research is to develop an improved
methodology for the optimal design of pumped storm sewer
networks. Generally, in any optimization problem, the
procedure followed to arrive at the optimal solution can be
divided into the following steps. First, the several elements
of the system to be designed are identified. For example, in
the present case, it is required to design the sewer elements
(depth, diameter and slope), sump (cross sectional area),
and pumps (rated head and discharge). In many cases, it is
useful to manipulate the obvious design variables in order to
arrive at more appropriate design variables. Such trans-
formation of variables may not only simplify problem formu-
lation, but can also permit the use of available optimization
algorithms.

The second step in the design procedure is to set up
the objective function and the set of constraints in terms of
the chosen design variables. The form of both the objective
function and the constraints determine the optimization
algorithm to be used, e.g. for a problem with a linear
objective function and constraints, a linear programming
technique is used to obtain the optimal design.

3.3.2 Design Variables

A highly sophisticated damage evaluation model was incorporated in the simulation to improve the objective function (see Section 3.3.4). This damage model evaluates the average annual probable damage resulting from adopting a specific design. The computer time required to perform this role is relatively high and it is impractical to apply the model indiscriminately for all the possible decision variables that could be considered in the objective function. This problem led the previous investigators to incorporate relatively simple damage models.

The design variables for the present problem are the diameter and slope of all the sewers, 2n, sump cross sectional area, and pump rated head and discharge. If the number of sewers, n, in the network being designed is large, the number of design variables will be accordingly large (2n + 3). To avoid the problem of having a large number of design variables, and to permit the use of the proposed damage model, it was decided to consider only five decision variables in the optimization model. These variables are simply the relative incremental changes in the corresponding physical quantities. These relative incremental changes are applied for a previously balanced design as outlined in Section 3.3.3. The design variables involved in the present design problem are X_i, i = 1, 2, ..., 5.

(i) <u>Variable X_1</u>: The incremental relative change in the sewers' slope

$$X_1 = \frac{(S_{o_i})_F - (S_{o_i})_I}{(S_{o_i})_I} \quad ; \quad i = 1, 2, 3, \ldots, n$$

(3.55)

where subscripts I and F represent the variable at the initial and final state respectively. In other words, $(S_{o_i})_F$ can be obtained knowing $(S_{o_i})_I$, and X_1, from

$$(S_{o_i})_F = (S_{o_i})_I \cdot (1 + X_1)$$

(3.56)

where $(S_{o_i})_I$ is the slope of the i-th sewer according to the initial design. The initial design is a preliminary design and is essential to start the solution.

(ii) Variable X_2: This variable combines two variables, the rated head, H_R, and the rated discharge, Q_R, into one variable which represents the pump power $(Q_R H_R)$. This is done because they appear together in the routing equation of the sump-pump model, Equation 3.46, and pump cost and operation is proportional to power. Hence, X_2 is defined as

$$X_2 = \frac{(Q_R H_R)_F - (Q_R H_R)_I}{(Q_R H_R)_I}$$

(3.57)

After obtaining X_2 from the optimization, and $(H_R)_F$ from the geometry of the final design, maintaining its compatability, $(Q_R)_F$ is determined from the following equation

$$(Q_R)_F = \frac{(Q_R H_R)_I}{(H_R)_F} (1 + X_2) \qquad (3.58)$$

(iii) Variable X_3: X_3 is defined as the incremental relative change in the sump cross sectional area, A_s,

$$X_3 = \frac{(A_s)_F - (A_s)_I}{(A_s)_I} \qquad (3.59)$$

or

$$(A_s)_F = (A_s)_I (1 + X_3) \qquad (3.60)$$

(iv) Variable X_4: X_4 takes into consideration the overall system depth, i.e. the lowering or raising of the whole sewer system by a certain amount. It is defined as

$$X_4 = \frac{(d_{sys})_F - (d_{sys})_I}{(d_{sys})_I} \qquad (3.61)$$

or

$$(d_{sys})_F = (d_{sys})_I (1 + X_4) \qquad (3.62)$$

where d_{sys} is a characteristic depth of the sewer system. The model considers d_{sys} equal to the depth from the ground surface to the crown elevation of the sewer at the most upstream end of the system. This variable reflects the effect of the respective positions of both basement grade line and sewer energy line on the system design.

Obviously, a system with a relatively large depth, d_{sys}, will have a low street and basement flooding damage. On the other hand, the unit cost of the sewers in the system will

increase resulting in an increase to the initial cost of the system.

(v) Variable X_5: X_5 is the incremental relative change in the diameter of the sewers;

$$X_5 = \frac{(D_i)_F - (D_i)_I}{(D_i)_I} ; \quad i = 1, 2, \ldots, n \quad (3.63)$$

or

$$(D_i)_F = (D_i)_I (1 + X_5) \quad (3.64)$$

A review of the previously discussed design variables shows that the role of optimization is to find the optimum change from an initial design to arrive at a better design. The outcome of this procedure is a local optimum design rather than a global optimum design. If a global optimum design is desired, the local optimum design should be adopted as an initial solution and the optimization process repeated. To achieve a similar improvement without repeated optimization, this model improves the initial design by a balancing procedure, see Section 3.3.3. This balancing procedure yields a consistently designed sewer system for a storm with a specified return period which permits a single optimization to yield a practical global optimum design.

3.3.3 Balancing Procedure

The main idea behind balancing the initial design is to improve it by allowing a more consistent performance for all the sewers and the pumping system. The unbalanced initial

design may have some underdesigned components and some over-
designed components, e.g. pipe and pump sizes. The balancing
procedure removes the inconsistencies of over- or under-
designed components. By combining a balanced initial design
with an optimization procedure, a more efficient sewer system
is obtained without using excessive computation time.

Balancing, as defined in this model, is a procedure
that corrects sewer diameters to maintain same maximum
relative storage in all sections for a preselected storm of
a known return period. To be consistent with the switching
criteria, the model considers the maximum relative storage
instead of the relative flow capacity. If the maximum
relative storage of the i-th sewer during a specific storm
is P_{max_i}, and its diameter is D_i, then, the balanced diameter
is obtained from

$$(D_i)_{bal} = D_i (P_{max})_i^{1/4} ; \quad i = 1, 2, \ldots, n \quad (3.65)$$

If the model switches to surcharged flow, P_{max_i} is taken equal
to the value of P_i just prior to switching. The model has to
iterate three times to achieve the same P_{max_i} for all the
sewers.

The capacity of the pumping station, Q_R, is a major
factor in determining the performance of the sewer system.
The model adjusts the pumping capacity so that it just prevents

surcharging of the outfall during the specified storm. The procedure to perform this role is illustrated in Figure 3.16. The adjustment procedure assumes that the pumping capacity is initially underdesigned and then computes the increase in pumping capacity required to lower the sump water level to the crown elevation of the downstream end of the outfall sewer, ENDSEW. The required increase in pumping capacity during each time step, ΔQ_p^t, is obtained from

$$\Delta Q_p^t = \frac{(SWL^t - SWL^{t-1}) A_s}{\Delta t \ (60)} \tag{3.66}$$

The model selects the maximum value of ΔQ_p^t during all the time steps considered up to the switching time or to the end of the storm whichever comes first. The new pumping capacity is then computed from

$$(Q_R)_{adj} = Q_R + \Delta Q_p \tag{3.67}$$

where $(Q_R)_{adj}$ is the adjusted pumping capacity. It was found that two iterations are sufficient to maintain the sump water level just below ENDSEW during the specified storm.

3.3.4 Objective Function

The objection function to be minimized in this model is the total annual cost of installation and the expected potential annual damage cost. The proposed form of the objective function is

$$\text{min. } Z = (C_o + d_o) + \sum_{i=1}^{5} c_i X_i + \sum_{i=1}^{5} d_i X_i \qquad (3.68)$$

where

$$c_i = \frac{\partial C}{\partial X_i} \qquad (3.69)$$

$$d_i = \frac{\partial D}{\partial X_i} \qquad (3.70)$$

C_o and C are the total annual costs of the initial design and the perturbed system respectively, installation and operational costs, and d_o and D are the average annual probable damages resulting from adopting the initial design and the perturbed system respectively; c_i is the installation cost gradient with respect to the i-th decision variable; d_i is the damage gradient with respect to the i-th decision variable. The procedures followed to obtain the numerical values of c_i and d_i are illustrated in the next section. Rearranging Equation 3.68 into the following standard form, gives

$$\text{min. } Z' = \sum_{i=1}^{5} a_i X_i \qquad (3.71)$$

where

$$Z' = Z - (c_o + d_o) \qquad (3.72)$$

and $\quad a_i = c_i + d_i \qquad (3.73)$

Since X_i is unrestricted in sign, i.e. positive as well as

negative changes are both expected as an output of the optimization process, each X_i variable is replaced by another variable which is the difference between two positive variables, $(X_i - XX_i)$, where X_i, $XX_i \geq 0$, and $i = 1, 2, \ldots, 5$.

3.3.4.1 Cost Model.

The main function of this model is to compute the annual cost of installation and operation of the sewer system. The annual cost of installation of the sewer system consists of the annual cost of installation of the various components, namely, sewers, manholes, inlets and pumping station (pumps, sump and building).

(i) Cost of Sewers, Manholes and Inlets: The model computes the unit cost of any sewer, using an equation in the form

$$Cost_i = a + b\,D_i^2 + c\,(H_{av})_i^2 \qquad (3.74)$$

where

$Cost_i$ = unit cost of the i-th sewer, \$/ft;

D_i = diameter of the i-th sewer, ft;

$(H_{av})_i$ = average depth of excavation of the i-th sewer, ft;

a,b,c = regression coefficients.

The model assigns default values of 2.8, 2, and 0.045 for a, b, and c respectively (13). These values were based on 1973 prices, hence, the cost of sewers is accordingly adjusted using an appropriate value of the relative engineering index for sewer prices. The relative engineering index, REI, as defined in the present model, is the ratio between the engineering index for construction year and the engineering index for the base year. REI was taken as 1.33 for 1976 as the construction year. After computing the unit cost of

sewers, their cost is computed from

$$\text{Cost of sewers} = (\sum_{i=1}^{n} \text{Cost}_i \, \ell_i) \, \text{REI} \qquad (3.75)$$

The costs of manholes and inlets are computed by assigning certain ratios between their cost and the cost of sewers. The model assumes that the cost of sewers represents 65 per cent (default value) of the total cost of sewers, manholes and inlets. After computing the total cost of sewers, manholes, and inlets, the model converts the resulting cost figure into an equivalent annual cost by multiplying the present cost by the capital recovery factor. The capital recovery factor, CRF, is computed from the expression

$$\text{CRF} = \frac{I\,(1+I)^N}{(i+I)^N - 1} \qquad (3.76)$$

where

I = interest rate per annum

= 0.12, default value assumed by the model;

N = years of estimated life.

The model assumes that the estimated life of sewers, manholes, and inlets is equal to 20 years default value. Also, it is assumed that all salvage values are negligible.

(ii) Cost of Pumps: The cost of pumps is assumed to be proportional to the pump power, $Q_R H_R$, viz,

$$\text{Cost of pumps} = c\,Q_R\,H_R + C_{STBY} \qquad (3.77)$$

where c is a regression coefficient, and C_{STBY} is the

installation cost of the stand-by pumping unit(s). The default value proposed by the model for c is 26 $/cfs·ft, and the model considers a minimum pump with the stand-by capacity of 25 ft rated pumping head, 7 cfs rated discharge and a diesel engine at 1976 price of $6000. It is worthy of notice, that these pump-cost data are typical, and they can be replaced easily by the actual cost data obtained from a special cost analysis for the locality being considered. As in the case of the sewer costs, the pump installation costs are converted into an equivalent annual cost by using an appropriate capital recovery factor. The model uses a default value of 20 years (19), for the estimated life of pumps and motors. Also, the relative engineering index for pumps and motors is used to correct the prices from the base year to the construction year.

(iii) Cost of Sump and Building: The cost of construction of the sump is a function of its volume and its unit cost of excavation, viz,

$$\text{Cost of sump} = A_s \, d_{sump} \, c_{exc} \qquad (3.78)$$

where

A_s = sump cross-sectional area, ft^2;

d_{sump} = sump excavation depth, ft;

c_{exc} = unit cost of excavation, $/ft^3 = $1/ft^3$,

default value.

The model sets the bottom elevation of the sump, 3 ft (default value) below the minimum sump water level. Also,

it takes into account the variation of c_{exc} with the depth of excavation by correcting its value as

$$c'_{exc} = c_{exc} (1 + d_{sump}/30) \qquad (3.79)$$

Equation 3.79 signifies that if the sump depth of excavation is 30 ft, the unit cost is corrected by a factor of 2.

The cost of the building of the pumping station is estimated by the following proposed equation:

$$\text{Cost of building} = C_{land} + C_{structure} \qquad (3.80)$$

The model assumes that the cost of land, C_{land}, is equal to $20,000, a default value for the price of 1/2 acre. Also the model relates the cost of the structure, $C_{structure}$, to its plan area, viz,

$$C_{structure} = (A_s + A_o) c_{st} \qquad (3.81)$$

where

A_o = minimum control area to operate the pumping station excluding the sump area, ft^2 = 2000 ft^2, default value; and

c_{st} = unit cost of structure per unit area of its plan. The model uses a 1976 default value of $ 30/ft^2. The model computes the annual cost of sump and building from

Annual cost of sump and building =

$$(A_s e_{sump} c'_{exc} + C_{land} + C_{structure}) (REI)_{sump} (CRF)_{sump}$$

$$(3.82)$$

where $(REI)_{sump}$ is the relative engineering index for the sump and the building; and $(CRF)_{sump}$ is the sump capital recovery factor. The model assumes a default value of 50 years for the estimated life of the sump and the building. The total annual cost of the sewer system is then computed from

$$C = C_{sewers,\ manholes,\ and\ inlets} + C_{pumps} +$$
$$C_{sump\ and\ building} + C_{operational} \qquad (3.83)$$

where $C_{operational}$ is the annual operational and maintenance costs. The model assigns a default value of $15,000 for the annual operational and maintenance costs.

Cost Gradients, c_i: It has been shown in the previous sections that the proposed system of equations to compute the annual capital recovery (and operational) costs cannot be directly incorporated in the linear form of the objective function. Moreover, the chosen decision variables do not permit the development of such explicit form of the objective function, i.e. expressing the cost gradients in terms of the decision variables. Instead, the model computes the numerical values of the cost gradients. The procedure followed by the model to compute c_i, $i = 1, 2, \ldots, 5$, is illustrated in Figure 3.17.

The model computes the forward cost gradients, which will be considered as the cost gradients, Figure 3.18, by

perturbing the system with preselected values of X_i and

determining the cost of the perturbed system, $(Cost)_i$. Then

c_i is obtained from

$$c_i = \frac{(Cost)_i - C_o}{X_i} \qquad (3.84)$$

where C_o is the cost of the original balanced sewer system.

It is worthy of notice that the model makes necessary adjust-

ments accompanied by each perturbation of the system, X_i, in

order to maintain the compatibility of the design. It

adjusts the rated pumping head and the minimum sump water

level such that they are consistent with the geometry of the

perturbed system.

3.3.4.2 Damage Model

(i) Basement Damages

Basement damages represent a major component in the

damages encountered when flooding takes place. It has been

shown in Section 3.2.3, that under certain conditions, storm

water may flow from storm sewers through house connections,

into basements resulting in damages to basement furnishings.

Also, it causes health hazards and inconvenience to the

property owners since their basements should be cleaned after

each flooding. The damages to furnishings as well as the

health hazards and inconvenience have to be estimated in

dollar terms in order to be included in the optimization

process. The model uses an equation to estimate the basement damages in the form of

$$(D_{Base})_i = (B_{value})_i (D_{factor}) \qquad (3.85)$$

where

$(D_{Base})_i$ = damages due to basements flooding from the Storm being considered along the i-th sewer, dollars;

$(B_{value})_i$ = assessed value of basements subjected to damage along the i-th sewer, dollars; and

D_{factor} = normalized damage factor.

The assessed value of basements subjected to damage is a function of the assessed value of houses. The model estimates $(B_{value})_i$ from

$$(B_{value})_i = (ASS)_i (AF)(1 + FU) RB_i (N_{house_i} F_{max_i}) \qquad (3.86)$$

where

$(ASS)_i$ = average assessment value of houses along the i-th sewer, dollars. The model assumes a default value of $8,000;

AF = assessment factor to account for the true market value, = 4 (default value);

FU = value of basement furniture/basement value = 0.25 (default value);

RB_i = portion of property value assigned to basements along the i-th sewer. The model assumes a default

value of 0.15;

F_{max_i} = maximum value of flooding ratio along the

i-th sewer, during the storm being considered. This para-meter is obtained from the simulation part of the model.
The damage factor, D_{factor}, is a parameter that represents
the severity of the storm event, and may be considered as a
function of the basement "flooding depth," d_{f_i}. The proposed

formula to compute D_{factor} is

$$D_{factor} = [1 + HEALTH + CLEAN (1 + HEALTH)] G \qquad (3.87)$$

where

HEALTH = health hazard factor,

= 0.5 (default value);

CLEAN = Basement cleaning (after storm) factor,

= 0.05 (default value).

The model computes the value of the severity parameter, G,
from

$$G = 0.5 + 0.166 \, d_{f_{max_i}} \qquad (3.88)$$

where $d_{f_{max_i}}$ is the maximum value of basement flooding depth,

d_{f_i}, during the storm, along the i-th sewer (inches).

Equation 3.88 signifies that the minimum value of G is
0.5 which corresponds to a storm just above to flood the
basements. Equation 3.88 also shows that if $d_{f_{max_i}}$ reaches

a value of 3 in., the value of G becomes unity, i.e. complete basement damage. If d_{max_i} exceeds 3 in., the model sets the value of G equal to unity.

(ii) Ground Surface Damages

As previously discussed in Section 3.2.3, storm water may flow through the inlets onto the streets. Ground surface damages include property damages due to surface flooding and inconvenience due to traffic interruption. To evaluate property damages, D_{prop}, the model classifies the districts into two classes, commercial districts and residential districts. The proposed equation to compute this damage component is in the form

$$D_{prop} = (CL) \frac{1}{2} [(ASS)_i (AF) (N_{house})_i CT_i + (ASS)_{i-1}$$

$$(AF) (N_{house})_{i-1} CT_{i-1}] (\forall_{max_i}/A_{surf}) \qquad (3.89)$$

where

CL = a calibration factor, fraction of the maximum possible damage. Default value = 1, i.e. 1 ft depth of water over the flooded area corresponds to 100 percent property damage;

CT_i = a factor that takes into account type of district along the i-th sewer,

= default values are 2.0 (commercial) and 1.0 (residential);

V_{max_i} = maximum value of the volume of storm water

collected on the ground surface at the i-th manhole junction during the storm being considered. This parameter is obtained from the simulation part of the model, ft³; and

A_{surf} = ground surface flooded area, ft².

The model approximates the flooded area, A_{surf}, by the area of the street along which the sewer is laid, viz;

$$A_{surf} = W_{street} \frac{1}{2} (\ell_i + \ell_{i-1}) \qquad (3.90)$$

where W_{street} is the width of the street, and it is assigned a default value of 60 ft.

The model evaluates the inconvenience resulting from the interruption of traffic by estimating the probable number of persons that may be affected by the storm and their delay time. An equation to estimate D_{inc} is proposed in the form

$$D_{inc} = (N_{cross})_i (t_p)_i^2 / 360 \qquad (3.91)$$

where

D_{inc} = inconvenience cost encountered at the i-th manhole junction (intersection) dollars;

$(N_{cross})_i$ = effective number of vehicle crossings per hour on the average basis at the i-th manhole junction (intersection);

$(t_p)_i$ = the time elapsed from the start of intersection flooding to the time of the peak V_{max_i} (min.).

To arrive at Equation 3.91, several simplifying assumptions were made. The model assumes two persons per vehicle and it estimates the duration of intersection flooding to be 5 times the time to peak. The time to peak $(t_p)_i$ is obtained from the simulation part of the model. The model considers that the average inconvenience cost is 5 \$/hr/person. The numerical values used in the previous set of equations for damage prediction are typical and the format of the computer model is flexible to permit the replacement of any value(s) with a more appropriate one.

(iii) Computation of Average Annual Probable Damage

The average annual probable damage resulting from adopting a specific design can be computed using the following procedure, see Section 2.4.2. The sewer system is tested against several rain storms with various frequencies and the corresponding total basement and ground surface damages are estimated as previously discussed. A damage-probability curve is plotted, and the area under it is the average annual expected damage, Figure 3.19. A good fit of the damage-probability relationship can be obtained by an equation of the form

$$D = A + B p + C p^2 \qquad (3.92)$$

where

D.   = total damage resulting from a rain storm with a probability of occurrence of p . The probability of occurrence, p, is the reciprocal of the return period, T_r,

which is assumed to be obtained by the annual series.

A, B, C = coefficients in the damage-probability
equation. To compute the values of A, B, and C, the model
obtains 3 points on the damage-probability curve. It
applies 3 preselected storms with various probabilities of
occurrence, p_i, and estimates the damage, D_i, resulting
from each storm. Then it substitutes in the following
equations to obtain A, B, and C, viz,

$$C = \frac{(D_2 - D_1)(p_1 - p_3) + (p_2 - p_1)(D_3 - D_1)}{(p_2 - p_3)(p_3 - p_1)(p_1 - p_2)} \quad (3.93)$$

$$B = \frac{(D_2 - D_1) - C(p_2^2 - p_1^2)}{p_2 - p_1} \quad (3.94)$$

and $A = -B p_1 - C p_1^2 + D_1$ (3.95)

where subscript 1 denotes the variables corresponding to the
rainstorm used in obtaining the initial design and in the
balancing procedure. Subscripts 2 and 3 denote the variables
corresponding to two other specified storms with higher
return periods.

The average annual probable damage, D_{av}, is the area
under the damage-probability curve. Hence, it may be obtained
by integrating Equation 3.93,

$$D_{av} = A p_0 + \frac{1}{2} B p_0^2 + \frac{1}{3} C p_0^3 \quad (3.96)$$

where p_0 is the probability of occurrence of the most severe

storm that could occur but not cause damage. The model considers P_O equal to P_1 since the system is balanced for a storm with a probability of occurrence of P_1.

The previous procedure is only followed in computing the average annual probable damage, D_{av_O} or d_O, for the original balanced system. In case of perturbed systems, the model assumes that the damage-probability curves maintain the same shape, Figure 3.20, and hence D_{av} is computed from the following proportionality equation

$$D_{av} = d_O \ (p_k/p_x) \tag{3.97}$$

where

D_{av} = average annual probable damage for the perturbed system, dollars;

d_O = average annual probable damage for the original balanced system, dollars;

P_k = probability of occurrence of a preselected representative storm, and

P_x = probability of occurrence of a storm resulting in the same damage when applied on the original balanced system, see Figure 3.20.

The value of p_x is computed by solving the damage-probability equation,

$$p_x = \frac{- B - \sqrt{B^2 - 4 \ C \ (A - D_k)}}{2C} \tag{3.98}$$

where

D_k = total damage resulting from applying a pre-selected representative storm with a probability of occurrence of p_k on the perturbed system; and

A,B,C = coefficients in damage-probability equation of the original balanced system.

Damage Gradients, d_i: The procedure followed by the model to compute the numerical values of the damage gradients is illustrated in Figure 3.21. The model computes the forward damage gradients, d_i, $i = 1, 2, \ldots, 5$, by perturbing the system with preselected values of X_i, $i = 1, 2, \ldots, 5$, and estimating the annual damages resulting from the perturbed systems $(D_{av})_i$. Then, d_i is obtained from

$$d_i = \frac{(D_{av})_i - d_o}{X_i} \qquad (3.99)$$

As in the case of cost gradients, the model makes the necessary adjustments accompanied by each perturbation of the system, X_i, in order to maintain the compatibility of the design.

3.3.5 Transformed Constraints

The conventional technological constraints incorporated in the design of sewer systems were discussed in Section 2.4.2. The format of these constraints does not permit its direct use in the present model because of the form of the chosen

decision variables. Hence, another set of constraints will be established to suit the decision variables involved in the present model.

The first group of the constraints that will be considered in this study replaces the conventional constraints of minimum depth, minimum velocity, maximum velocity and maximum depth of excavation. This group of constraints sets two limits on the decision variables, i.e. upper limit, positive relative changes, and lower limit, negative relative changes. The 10 constraints belonging to this group are

$$X_i \leq (X_{UL})_i \quad ; \quad i = 1, 2, \ldots, 5 \qquad (3.100)$$

and

$$X_i \geq (X_{LL})_i \quad ; \quad i = 1, 2, \ldots, 5 \qquad (3.101)$$

where $(X_{UL})_i$ and $(X_{LL})_i$ are the upper and lower limits respectively. These limits are specified before proceeding in the optimization process. After determining the original balanced design, $(X_{UL})_i$ and $(X_{LL})_i$ are assigned numerical values such that the design variables at both extremes, upper and lower limits, satisfy the previously discussed constraints of minimum depth, minimum velocity, maximum velocity and maximum excavation depth.

The second group of constraints sets two limits on average annual probable damage resulting from adopting the optimum system, see Figure 3.22. D_{av} should not exceed, in magnitude, the ultimate average annual probable damage,

$(D_{av})_{ult}$, which results for case of having no sewer system, i.e. p_o equals unity, see Figure 3.19. An approximate value of $(D_{av})_{ult}$ may be obtained by assuming a linear relationship between damage and probability, in which case, the area under the curve, $(D_{av})_{ult}$, equals to A/2. The lower limit of D_{av} is obviously zero, i.e. D_{av} cannot be a negative value. The second group of constraints consists of 12 constraints, viz,

$$\text{upper limit} \quad \sum_{i=1}^{t} d_i X_i + d_o \leq \tfrac{1}{2} A \qquad (3.102)$$

and

$$d_i X_i + d_o \leq \tfrac{1}{2} A ; \quad i = 1, 2, \ldots, 5 \qquad (3.103)$$

$$\text{lower limit} \quad \sum_{i=1}^{5} d_i X_i + d_o \geq 0 \qquad (3.104)$$

and

$$d_i X_i + d_o \geq o ; \quad i = 1, 2, \ldots, 5 \qquad (3.105)$$

where d_o is the average annual probable damage for the original balanced system and d_i is the damage gradient with respect to the i-th decision variable.

The third group of constraints sets a lower limit on the capital recovery cost of the sewer system, C, see Figure 3.18. The lower limit of C is zero, i.e. capital recovery cost of the sewer system cannot be negative. This group consists of 6 constraints, viz,

$$\sum_{i=1}^{5} c_i X_i + C_o \geq 0 \qquad (3.106)$$

and $\quad c_i X_i + C_o \geq 0 \; ; \qquad i = 1, 2, \ldots, 5$ (3.107)

where C_o is the capital recovery cost of the original balanced system; and c_i is the cost gradient with respect to the i-th decision variable.

The fourth group of constraints consists of only one constraint that restricts the combined change in C and D_{av} not to exceed $C_o + d_o$, viz,

$$\Delta C + \Delta D \not> C_o + D_o \tag{3.108.a}$$

or $\quad \displaystyle\sum_{i=1}^{5} c_i X_i + \sum_{i=1}^{5} d_i X_i \leq C_o + d_o$ (3.108.b)

or $\quad \displaystyle\sum_{i=1}^{5} a_i X_i \leq C_o + d_o$ (3.108.c)

3.3.6 Optimization Algorithm

It has been shown in Sections 3.3.4 and 3.3.5 that both the objective function and the set of constraints are linear in terms of the decision variables, X_i. Hence, the model obtains the optimum design using a linear programming technique in the "Simplex" form. The Simplex Method has been programmed and is available in the computer library in the form of a subroutine named "SIMPLX," see Appendix II, for the user's guide to this program.

(i) Input Data

The input data to the optimization algorithm are the coefficients of the objective function and the constraint equations. Appendix II presents the user's manual of the "Simplex Algorithm Program." It illustrates the procedure that should be followed in preparing the data cards. The first step to prepare the input data is to establish the objective function and constraints matrix (tableau), see Table 3.1. Table 3.1 shows the coefficients of the decision variables in both the objective function and the set of constraints after rewriting them in the standard form. These coefficients are given in terms of cost gradients, c_i, damage gradients, d_i, combined gradients, a_i, $(X_{UL})_i$, $(X_{LL})_i$, d_o, C_o and A, which are obtained from the simulation part of the model, the first program.

(ii) Output

The subroutine prints out the numercial values of the basis variables and the non-basic variables. The optimum value of any decision variable is the difference, $X_i - XX_i$. The original balanced design and the optimum values of the decision variables are then used to obtain the optimum design of the sewer system with the aid of Equations 3.58, 3.60, 3.62, 3.64 and 3.66. The resulting sewer diameters are corrected to conform with the commercially available pipe sizes by choosing the smallest commercially available pipe size with diameter larger than the one obtained by optimization.

3.4 Master Flow Chart of the Model

This model consists of two computer programs. The function of the first program (the main program) is to obtain the balanced design of the sewer system, compute its capital recovery cost, C_o, and its average annual probable damage, d_o. Moreover, it computes the cost gradients, c_i, and damage gradients, d_i. Figure 3.23 is a schematic flow chart of the model. The initial design required to start the program is a preliminary design for a storm with a specified return period which is used in the balancing procedure. This design has to satisfy the technological constraints and it is assumed to be overdesigned.

The output from the first program is used to prepare the input data for the second program. The second program is the optimization program and its output is the optimum values of the decision variables. The last step to arrive at the final design is to correct the sewer diameters to conform with the commercially available pipe sizes.

CHAPTER 4

DESCRIPTION OF COMPUTER PROGRAMS
AND DOCUMENTATION

4.1 General

The model presented in this research consists of two computer programs written in the WATFIV version of the FORTRAN language. The first program is the major program and it requires approximately 90,000 bytes of storage. The average execution time for the first program to perform its role is approximately 3.5 minutes (CPU), class B, slow core. The second program is the optimization program, and it consists of a main program and a linear programming subroutine "SIMPLX." Both programs were run on an IBM 360/65 computer at the University of Windsor Computer Centre. A listing of the first program is given in Appendix III.

The drainage basin may be subdivided into subcatchment areas. These, in turn, may drain into gutters or sewers which finally connect to the inlet points for the sewer system being considered by the model. The model obtains the optimum design of pumped sewer systems. For convenience, the program is limited to 3 sewers and 4 inlet points (junction manholes), but it can be easily extended. The scheme suggested for using the model is to design all the sewers in the system except the most downstream sewers near the outfall, by the conventional design procedures such as

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the Rational Method. Then, the model designs the last few sewers of the trunk line and the pumping station.

4.2 Main Program and Subroutines

Main Program

Figure 4.1 is a flow chart for the main program. The different variables that appear in the flow chart are defined in the subroutines. The subroutines called by the main program will be discussed in the subsequent sections.

Subroutine-INPUTD

This subroutine reads in the input data. Section 4.3 illustrates the different groups of input data cards.

Subroutine-PRINT1

This subroutine is used to print out the input data, if desired. Also, it prints out the dimensions of the balanced design. Moreover, it may be used to print and plot the rainfall hyetograph. Flag statements are used to activate the printing process, with respect to each group of parameters to be printed. Figure 4.2 shows a flow chart of the subroutine.

Subroutine-PRINT2

This subroutine prints out basic inflow hydrographs at manhole junctions.

Subroutine-RAINF

This subroutine is used to generate the rainfall intensity array, assuming a storm length of 180 minutes. Figure 4.3 shows a flow chart for subroutine "RAINF."

Subroutine-INFHYD

Basic inflow hydrographs contributing to manhole junctions are generated provided that rainfall intensities, impervious area and time of concentration of the basic sub-catchments are given. Figure 4.4 is a flow chart for subroutine "INFHYD."

Subroutine-MUSKRT

The major function of this subroutine is to route, using a Muskingum Routing Technique, the inflow hydrograph through a particular sewer to obtain the outflow hydrograph from the sewer.

Subroutine-ADDHYD

The ordinates of the hydrographs contributing to the manhole junction being considered are added to obtain the inflow to the downstream sewer.

Subroutine-HYDMOD

This subroutine is used for hydraulic simulation of flow in the sewer system. It routes the hydrographs under gravity flow conditions, otherwise it calls the surcharged flow model subroutine "PRESML." If gravity flow predominates during the whole storm, it sets the total damage resulting from this storm equal to zero. Figure 4.5 is a flow chart for this subroutine.

Subroutine-SUMP1

This subroutine computes the sump water level and the total energy level at the downstream end of the outfall sewer. A reservoir routing technique is used to perform

this role. A flow chart for subroutine "SUMP1" is shown in
Figure 4.6.

Subroutine-STRMOD

This subroutine computes the storm water flowing upward
through the i-th manhole by applying a mass balance equation.
Figure 4.7 shows a flow chart for this subroutine.

Subroutine-PRESML

The function of this subroutine is to simulate the
surcharged flow in the sewer system, taking into consideration
the flow to and from basements as well as streets (ground
surface). As previously discussed in Section 3.2.3, the
model has to iterate 6 times to construct the energy line
along the sewer system, and to obtain basement and street
flows. Also, the model has to iterate 3 times to improve
the starting point A, Figure 3.7.b. .To reduce the number of
iterations and to stabilize the solution, the model uses
one or both of the following approaches if necessary. .In
the first approach, the variable being considered is multip-
lied by a dampening factor to reduce the fluctuation in its
numerical value, viz.,

$$(VAR)_1 = (VAR)_2 [1 - (0.9)^{ITR^{ITR}}] \qquad (4.1)$$

where

$(VAR)_1$ = numerical value of the variable after
dampening process;

$(VAR)_2$ = numerical value of the variable as computed

in the iteration being considered;

and   ITR = iteration index.

The second approach is to average the numerical value of the variable obtained from Equation 4.1 and its numerical value resulted from the previous iteration, viz.,

$$(VAR)_{ITR} = (VAR)_{1_{ITR}} + (VAR)_{ITR-1} \qquad (4.2)$$

where

$(VAR)_{ITR}$ = most recent numerical value of the variable being obtained at the end of iteration ITR;

$(VAR)_{1_{ITR}}$ = numerical value of the variable after dampening process, in iteration ITR;

and $(VAR)_{ITR-1}$ = numerical value of the variable resulted from iteration ITR-1. Figure 4.8 is a flow chart for sub-routine "PRESML" and it shows the application of these two approaches in the iteration procedures.

Subroutine-QHOUSS

This subroutine computes the average rate of basement flow for the t-th time step, see Figure 4.9.

Subroutine-KENERG

This subroutine is used to compute the coefficient of the energy slope equation.

Subroutine-FRATIO

It computes the ratio between the length of the sewer actively being flooded and its total length, i.e. basement

, flooding ratio.   Figure 4.10 is a flow chart for this sub-routine.

Subroutine-SWITCH

This subroutine is used to determine the condition of flow in the sewer system, gravity flow or surcharged flow.

Subroutine-VOLSYS

This subroutine computes the volume of the pipes in the system, which represents the maximum pipe storage.

Subroutine-ELVBAS

This subroutine is used to compute the slope and mid-point elevation of the basement grade line associated with the i-th sewer.

Subroutine-CPIPES

This subroutine computes the total installation costs of all the sewers in the system.

Subroutine-STDAMG

The function of this subroutine is to estimate ground surface damages including inconvenience cost and property damage.

Subroutine-BDAMAG

This subroutine estimates basement damages associated with the i-th sewer.   It includes damages to furnishings as well as health hazards and inconvenience to property owners.

Subroutine-ANCOST

This subroutine computes capital recovery costs of the sewer system including sewers, manholes, inlets (and catch basins, if any), pump, sump and structure of the pumping

station.   Operational and maintenance costs are also included.

Subroutine-ANDAM

Basement and ground surface damages are added for each sewer section and then they are summed up for all the sewers.

Subroutine-BALD

This subroutine corrects sewer diameters to maintain same maximum relative storage in all sections.

Subroutine-HYDMB

This subroutine is used to balance the preliminary design. It attempts to maintain the same maximum relative storage for each sewer section for a preselected storm of a known return period. Also, it adjusts the pumping capacity so that it just prevents surcharging of the outfall during the specified storm, see Figure 4.11.

Subroutine-DAMPRO

This subroutine computes the coefficients in the damage-probability equation. It is also used to compute average annual probable damage.

Subroutine-DP1

This subroutine is used to compute average annual probable damage for a perturbed system. A, B, C and average annual probable damage of the original system are known. Damage resulting from a preselected prescribed storm is used to determine the new average annual probable damage.

Subroutine-CHANGE

This subroutine perturbs the original balanced design with respect to the i-th decision variable. Also, it makes

the necessary adjustments accompanied by each perturbation of the system in order to maintain the compatibility of the design.

Subroutine-ORIGNL

This subroutine is used to reset the perturbed system to its original balanced dimensions.

Subroutine-STORE

This subroutine stores the original balanced design of the sewer system.

Subroutine-GRAD1

This subroutine is used to obtain first derivatives of both cost and damage with respect to decision variables. Forward gradients are computed using positive relative changes.

Subroutine-GRAD1B

In this subroutine, backward cost gradients are computed using negative relative changes, less cost and higher damage. This subroutine is used to compare the forward and backward cost gradients but is not generally called during the design procedure.

4.3   Input Data Card Formats

4.3.1   Method of Discretization

Discretization begins with the identification of drainage boundaries, i.e. basic subcatchments. To find the impervious area contributing to each inlet point (junction manhole), a map of the drainage basin is prepared showing

all impervious areas that are connected to the sewer system,
for an example see Figure 3.2. The impervious area contribu-
ting to the first upstream inlet point is the summation of
all the impervious areas contributing to the sewers upstream
of this inlet point. The time of concentration of the basic
subcatchments is estimated as the summation of both inlet
time and the time of travel in sewers within the basic sub-
catchment.

4.3.2   Estimate of Parameters and Coefficients

(i)   Rainfall Data

The rainfall data necessary as an input to the program
are the coefficients a, b and c of the intensity-duration
curve. Also, the user specifies the relative advancement of
the peak intensity, r, as outlined in Section 3.2.1. The
values of a, b, c and r have to be specified, for three
storms with different specified probabilities of occurrence,
p.

The first storm is the one used in obtaining the initial
design, and the balancing procedure, e.g. 1 in 5 years storm.
The second storm is more severe than the first storm and it
represents the category of storms that results in moderate
damages, e.g. 1 in 10 years storm. The second storm is also
used as a representative storm to find the average annual
probable damage for the perturbed system. The third storm
represents the category of storms resulting in very severe
damages, such· as 1 in 50 years storm.

(ii)   Subcatchment areas

The subcatchment areas contributing to the inlet points are identified by their impervious areas directly connected to the storm sewers, and their time of concentration.

The determination of such parameters is outlined in Section 4.3.1.

(iii)   Inlet Points (Manhole Junctions)

The data required to identify inlet points as well as subcatchment areas are supplied by the user starting from the upstream and proceeding downstream. The parameters necessary to characterize the physical properties of the inlet points are ground surface elevation, coefficient of inlet catchbasin and effective number of vehicle crossings per hour on an average basis. The coefficient of inlet catch basin has been discussed in Section 3.2.3.2. The effective number of vehicle crossings can be obtained from a traffic survey in the drainage basin if the system is already existing or in a similar drainage basin.

(iv)   Sewers

To start the solution, the program requires an initial design that satisfies the ASCE technical constraints. Manning's coefficient, length, diameter, slope and depth of sewers must be specified. The depth is identified by the crown elevation of the upstream end of the first upstream sewer. The model assumes no drop manholes and aligns the sewers by assuming the same crown elevation for upstream and downstream trunk sewers connected at each manhole.

(v)   Pump

As in the case of the initial design of the sewers,
the user specifies an initial design rated head, rated dis-
charge and average relative operating efficiency of the pump.

(vi)   Sump Well (wet well)

The data regarding the sump are its cross-sectional
area and the total energy level on the delivery side of the
pumps which may be the elevation of the water surface of the
receiving body.   The sump cross-sectional area is obtained
from the initial design of the sewer system.

(vii)   Basements

The model requires specific data concerning basements
that will be used in both the hydraulic simulation and the
damage evaluation.   The user has to specify, for each sewer
section, basements area, average depth of basements below
ground surface, and class of district in which the sewer
section is laid, commercial or residential.   Also, the
model requires as input data, average length and diameter of
the pipes for house connections as well as the number of
houses along each sewer section.

(viii)   Increments Used to Obtain Cost and
Damage Gradients

Section 3.3.4 illustrates the procedures followed to
obtain both cost and damage gradients.   The positive as
well as negative relative incremental changes used to perturb
the balanced system are required as input to compute the cost

and damage gradients with respect to the decision variables. The values assigned to these changes should be in approximately the same range as expected optimum changes so that the computed gradients will be representative.

(ix) Time Increment

The user has to specify the time increment used by the computer program. The range recommended for this time increment is from 1 to 3 minutes depending on the degree of accuracy required.

(x) Base Flow

A non-zero base flow has to be supplied.

4.3.3 Preparation of Data Cards for the First Computer Program

The data cards should be prepared according to Table 4.1. This table shows how the data cards are to be punched and lists the default values (if any) to be used in accordance with the variables considered.

4.3.4 Preparation of Data Cards for the Second Computer Program

Figure 4.12 shows typical input data cards for the optimization program. The matrix coefficients are also given in terms of cost gradients, c_i, damage gradients, d_i, combined gradients, a_i, upper limits of the decision variables, $(X_{UL})_i$, lower limits of the decision variables

$(X_{LL})_i$, capital recovery cost of the balanced system, c_o, average annual probable damage for the balanced system, d_o, and ultimate damage resulting from the balanced system, A. The variables c_o, d_o, c_i, d_i and A are obtained from the first program. The determination of $(X_{UL})_i$ and $(X_{LL})_i$ was discussed in Section 3.3.5. The values of the matrix coefficients that appear in Figure 4.12 were divided by 1000 to suit the subroutine printing formats.

4.4. Output

The output of the first program is a printout of the input data and the balanced design.

The capital recovery cost of the balanced system, c_o, the average annual probable damage, d_o, and the ultimate damage resulting from adopting the balanced design, A, are also printed. If desired, the rainfall hyetograph used in testing the perturbed systems can be printed and plotted along with the corresponding basic inflow hydrographs.

A typical output of the optimization program (second program) is shown in Figure 4.13. The printout includes the values of X_i, XX_i; i = 1, 2, ..., 5 at the optimum state as well as the corresponding value of the objective function. The optimum value of a decision variable X_i is computed as the difference $X_i - XX_i$.

CHAPTER 5

EVALUATION, APPLICATION OF THE MODEL

5.1 Hydraulic Performance of the Model

The hydraulic performance of the model is best illustrated by examining typical outputs of the various components of the model. Figure 5.1 shows a typical design storm hyetograph for a 1 in 10 years storm. The relative advancement of the peak intensity used to generate this hyetograph is 0.375. Using the rainfall-runoff part of the model, the rainfall hyetograph is transformed to the inlet inflow hydrographs. Figure 5.2 presents a typical basic inflow hydrograph generated from a basic subcatchment with an impervious area of 13 acres, and a time of concentration of 25 minutes. The model routes the basic inflow hydrographs, contributing to the inlet points, through the sewer system and obtains the outfall hydrograph. A typical outfall hydrograph to the sump well is shown in Figure 5.3, for a drainage basin with an impervious area of 22 acres and resulting from a 1 in 50 years storm.

Figure 5.4 presents a typical plot of sump water level versus elapsed time. This figure illustrates the effectiveness of the pumping system (pumping units and sump). It also shows the crown elevation of the outfall sewer at the sump, which is used to determine the submergence of the outfall.

Figure 5.5 shows a typical variation of basement flow along a typical sewer section with time elapsed from the beginning of the storm. It is worthy of notice that the program terminates when all the basements start to discharge the collected storm water back to the sewers. A typical plot of basement flooding depth versus time is presented in Figure 5.6. Figure 5.7 shows the variation of the excess volume of storm water contributing to street flooding with time at a typical manhole.

5.2 Sensitivity Analysis

An awareness of the sensitivity of the model to its various parameters is a valuable aid towards a successful application of the model. A sensitivity analysis is presented in the subsequent sections to show the effects of various parameters on the average annual probable damage, d_o, cost gradients, c_i, and damage gradients, d_i, and consequently, on the resulting optimum design. The sensitivity analysis may be divided into two parts. The function of the first part is to show the effects of the selected parameters on d_o, c_i and d_i. In the second part, the results of changing d_o, c_i, and d_i on the resulting optimum design are investigated. The parameters chosen to test the model sensitivity represent two groups. The first group consists of the parameters that are involved in the hydraulic simulation such as sewers roughness coefficient, Manning's n and basement area

(number of houses). On the other hand, the second group is characterized by the parameters that are involved in the damage evaluation, e.g. average assessment value of houses, portion of property value assigned to basements and number of houses. The number of houses along the sewer sections is common to the two groups because basements area affects the basement flow, while the corresponding number of houses determines the property value subject to damage.

5.2.1 Effects of Selected Parameters on d_o, c_i and d_i.

(i) Number of Houses Along Sewer Sections: Figure 5.8 shows a typical variation of average annual probable damage, d_o, with number of houses along sewer sections. It indicates that d_o is proportionate to the number of houses along sewer sections. Typical variations of cost and damage gradients with number of houses are shown in Figure 5.9. Examining Figure 5.9 gives an indication of the relative importance of the various damage gradients and consequently, their impact on the optimum design. It is noticed that damage gradient with respect to the sewer diameter has a much greater magnitude than other damage gradients, hence, it is expected to influence the optimization process significantly. Also, it may be easily seen that c_i is invariant with the number of houses along sewer sections.

(ii) Average Assessment Value of Houses, ASS: From Figure 5.10 it is shown that the average annual probable damage varies linearly with ASS. The effect of ASS on cost

and damage was investigated, but not presented and it was found to be similar to case (i).

(iii) Portion of Property Value Assigned to Basements, RB: Figure 5.10 shows a typical variation of average annual probable damage with RB. It has similar effects as cases (i), (ii), on d_o, cost and damage gradients.

(iv) Roughness Coefficient, Manning's n: Two values of n, 0.013 and 0.002, were used to investigate its effects on the average annual probable damage as well as cost and damage gradients. The results are presented in Table 5.1. Table 5.1 also shows the effects of the value of n on the balanced design (dimensions and capital recovery cost).

(v) Time Step, T: A hypothetical drainage area was designed by the model twice, using two values for the time step, i.e. 2 and 3 minutes. The effects of the magnitude of the time step on the accuracy of determining the balanced design and the gradients are shown in Table 5.1.

5.2.2 Effects of d_o, c_i and d_i on the Optimum Design

The magnitudes of the various variables, d_o, c_i and d_i, significantly affect the results of the optimization process. The decision variable having the greatest magnitude in the combined gradient, a_i, is expected to dominate.

A typical case was investigated in which two sets of d_o, c_i and d_i, varying in magnitude, were used to obtain the optimum design. Table 5.2 presents the results of the

optimization process for these cases. The two cases previously
considered were obtained by designing a sewer system for two
drainage basins which are identical except for the number of
houses along each section, 10 and 30 houses respectively, i.e.
basements area along each section are 1/3 or 1 acre.

They had the same balanced design and cost gradients
but different damage gradients, as shown in Table 5.2.
Examining the optimum design in both cases indicates that a
completely different design is obtained when different
gradients are encountered. Similar results are expected in
other cases when other parameters such as average assessment
value of houses or portion of property value assigned to
basements are changed.

5.2.3 Effect of the Initial Design on the Balanced Design

To investigate the effect of the initial design on the
resulting balanced design, a typical drainage basin was
designed 3 times with different initial designs. Table 5.3
presents both the initial and balanced designs for each case.
It indicates that, for a wide range of initial designs,
approximately the same balanced design is obtained, i.e. the
model is insensitive to the initial design.

5.3 An Example

The model was first applied to a hypothetical network
in order to test its usefulness as a design tool. The
optimal design obtained by the model was later compared to a
conventional Rational Method design.

A schematic drawing for the hypothetical drainage basin for the example is shown in Figure 5.11. It includes the topography, flow directions, and boundaries of basic sub-catchments. The drainage basin is a residential district of 55 acres in area with 22 acres as impervious area, directly connected to storm sewers. The collected storm water is discharged to a lake with a mean water level of 98 ft. above a reference level. The water surface in the lake may rise due to wind setup effects to an elevation of 102 ft. The later elevation, being more critical, was considered in the design. Due to insufficient natural ground surface gradient, it was decided to incorporate a pumping station at the outfall to facilitate the disposal of the storm water to the lake.

The example drainage basin represents a typical area in which basement and street flooding could be a problem. The characteristics of the drainage basin are presented in Tables 5.4.a and 5.4.b, while the rainfall data are presented in Table 5.5. Due to the present format of the model, it was decided to design only the last 3 sewers at the outfall, sewer sections 1, 2 and 3, and to lump the upstream portion of the drainage basin as one basic subcatchment of 32.5 acres, with 13 acres as an impervious area directly connected to storm sewers. The estimated time of concentration for this basic subcatchment was 25 minutes. The crown elevation of the trunk sewer at manhole 1 should not exceed 99.5 ft, to permit a drainage outfall for the first basic subcatchment.

The sewer sections under consideration were designed by

the Rational Method for a design return period of 5 years.
The design as well as the intermediate computations are
presented in Table 5.6. The runoff coefficient was assumed
to be equal to the ratio between the impervious area of the
basic subcatchment and its total area. The pumping capacity
was equated to the outfall discharge of 61 cfs computed by
the Rational Method and the rated pumping head was chosen
as 15 ft. The sump cross-sectional area was determined by
rule of thumb as the area which would yield a storage
sufficient to detain the design flow (corresponding to the
1 in 5 years storm) for 1.5 minutes; it was found to be
900 ft^2.

The model was then applied to the drainage basin under
consideration, with the previously obtained Rational design
as an initial solution (see Table 5.7). The procedure used
to apply the model and the format of the input data have
been illustrated in Chapter 3. The optimal design obtained
by the model is presented in Table 5.8.

The lower limit imposed on the sump cross-sectional
area was set equal to 10 percent of the initial design
because of the restriction that this area must accommodate
the pumping units. Also, the lower limit on the change in
pumping power was selected as 10 percent, because the forward
damage gradient with respect to pump, is not representative
of large negative incremental changes. Due to minimum cover
constraint, it was decided not to allow negative changes for
both slope and depth decision variables.

Table 5.8 presents the results of both the Rational design and the optimal design obtained by the model. It shows the fractional as well as the commercially available sewer sizes. To evaluate and compare the two designs with fractional and commercially available sewer sizes, the model was used for each case to compute the capital recovery cost, c_o, and the average annual probable damage, d_o. Examining the values of the net benefit, combined cost, $c_o + d_o$, indicates that the optimal design saves \$37,000 in case of fractional sewer sizes while it saves \$2,600 in case of commercial sewer size. In other words, the model offers an optimal design with an average saving of one-half (\$37,000 + \$2,600), i.e. \$20,000. This saving represents 25 percent of the initial combined cost. The actual cost saving should be significantly greater for larger networks.

For further comparison between the two designs, previously outlined, it may be useful to examine the degree of utilization (consistency of the design) of the various sewer sections in the network. Table 5.9 presents the values of the maximum relative storage for each sewer section during a 1 in 5 years storm. In case of fractional sewer sizes, the design obtained by the model is fairly consistent, while the Rational design may be considered inconsistent as sewer (1) is overdesigned, whereas sewers (2) and (3) are underdesigned.

5.4 Suggested Further Developments of the Model

The present model gives rise to some interesting topics that warrant further attention. The following are a few potential future extensions:

(i) The present model consists of two computer programs with a manual link between them. For the user's convenience, it is recommended that the two programs be integrated into one computer program. This can be accomplished by either changing the format of the SIMPLX-SUBROUTINE to replace the reading process by a set of COMMON statements, or by changing the output format of the first program to punched cards that match the required input format the SIMPLX program.

(ii) The model, in its present format, handles only a network with a single main line (trunk) with a few sewers. Therefore, it should be modified to treat branched sewer systems with larger number of sewers. Handling large number of sewer sections can be achieved by modifying certain subroutines to store only the variables for two successive time steps. This procedure significantly reduces the required storage and allows treatment of more sewer sections.

(iii) An investigation should be made into the accuracy of the adopted linear programming technique for a wider range of negative as well as positive incremental changes. It may prove useful to examine a non-linear programming technique to permit more accurate treatment of the constraints, and take into consideration the mixed derivatives.

(iv) The model should have an option to handle off-

channel storage at any location in the sewer network.

(v) Further research should be carried out on the switching procedure to detect the possible surges.

(vi) Further research into damage evaluation, based on actual damage data, should be made to improve the built-in default values.

(vii) An investigation into the possibility of improving the gravity flow model, Muskingum Method, would be beneficial. This may be achieved by relating the Muskingum's constant, x, to the hydrograph limb, i.e. rising or falling and to the type of surface profile occurring in the pipe.

(viii) For user's convenience, the model may be extended to incorporate additional subroutines for design by the Rational Method and for sewer branches other than those optimized by the model.

CHAPTER 6

CONCLUSIONS

The computer-aided design procedure developed in this research presents an optimal design for pumped storm sewer networks. The model obtains practical global optimum values of the depths, slopes and diameters, sump area and rated pumping head and discharge for a storm sewer network. The model provides a logical alternative to the arbitrarily selected return period used in other design procedures, such as the Rational Method. The objective function which is to be minimized is the sum of the operational and capital recovery costs of the system, as well as the average annual probable damage of the proposed system.

In the first part of the model, the hydraulic simulation sub-model considers surcharged flow in the sewer system and flow to and from the basements as well as the streets, i.e., the ground surface. These hydraulic subroutines have proved to be useful in obtaining the various parameters that form the bases for damage evaluation.

The model also presents an effective design tool for dimensioning the sump well and the pumping units, which is an improvement over the available rule of thumb methods. It takes into consideration the mutual effects between the sewer system and the pumping station.

The damage evaluation procedure integrates high as well as low probability storms, i.e. it considers a wide range of system operating conditions.

The model has the capability of evaluating modifications to an existing system or any specific design. This can be achieved by using the gravity flow model coupled with the balancing procedure. The computer time for this design option is very short.

The optimization model was applied to a hypothetical drainage basin and the resulting design was compared with a design obtained by the Rational Method. The model offered a cost saving of about 25 percent of the total cost of the initial design, $C_o + d_o$, for the network considered in the example. The format of the model is well suited for the evaluation and analysis of existing systems as well as specific designs.

Based on the findings of the sensitivity analysis, it may be concluded that increasing sewer diameters is very effective in improving system performance, and reducing damage cost. Also, the model indicates that increasing sump area is not an economically feasible solution unless the unit cost of storage is very low.

APPENDIX I

BRIEF PRESENTATION OF SELECTED HYDROGRAPH MODELS

APPENDIX I

BRIEF PRESENTATION OF SELECTED HYDROGRAPH MODELS

At present, there is a large number of runoff hydrograph models which indicates the general interest in urban drainage problems. Some of these models are proprietary (20), in which case, even if the basic approach is known, the program is not generally available. Other models have only fair documentation making their application difficult. Table I.1 shows an excellent comparison of various runoff models.

(i) The Road Research Laboratory Model (RRL)

The RRL model has been developed and extensively used in Great Britain. Some experiences with this model in North America were described by Terstriep and Stall (35), and Marsalek et al. (21). The original version of the model was described by Watkins (35, 21). The RRL model considers runoff generated on impervious areas only and neglects the contributions from pervious areas where infiltration and detention are usually high. The depth of rainfall is reduced by a factor to account for surface storage. Marsalek et al. (21) used a factor of 0.9 in their study "Comparative Evaluation of Three Urban Runoff Models." Terstriep and Stall (35) assumed no abstraction in their application of the RRL

Method on three urban watersheds, i.e. runoff coefficient was considered to be 1.0 for the impervious areas. The resulting effective rainfall data are then applied to the impervious areas directly connected to the sewer system.

The impervious area is characterized by a curve of contributing area versus the time of travel. The curve is usually linearized (an assumption made for simplicity and to avoid excessive input data), by connecting the point representing the total contributing area and the origin with a straight line (Figure I.1). The time of travel generally consists of the inlet concentration time and the travel time in the sewer. The inlet concentration time is usually obtained as for the Rational Method by an empirical estimate or by using the linear kinematic-wave solution for the overland flow, as suggested by Terstriep and Stall (35). The time of travel inside the sewers is calculated from the full-bore velocity.

The inlet hydrograph is derived by combining the effective rainfall hyetograph with the contributing area versus time diagram. After adding the time lagged upstream hydrographs, the resulting hydrograph is routed through the main sewers using storage routing based on a linear reservoir concept. The design version of the model deals with sur-charging by automatically increasing the would-be surcharged sewer diameter. Figure I.1 shows a flow chart of the algorithm of the modified RRL.

(ii)   The Illinois Storm Sewer Simulation Model (ISS Model)

The St. Venant equations for continuity and momentum
(40) are used in this model. The lateral flow term is not
included and it is assumed that inflow of storm water into
the sewer system occurs only at discrete model points. This
is of course justified for larger areas and equivalent water-
sheds. The equations are solved numerically by an implicit
first order characteristic scheme. Backwater effects are
considered and reservoir type junctions are modelled where
manhole storage is taken into account. Energy and continuity
equations are formed for each manhole and the entire system
of equation is solved simultaneously at each time step (21).
The model handles circular sewers only and it assumes that
there are no more than three sewers joined together at a
manhole. The ISS model can be used for flow prediction as
well as for design of sewer sizes for a given system layout
and specified slopes (39).

(iii)   Hydrograph Volume Method by Dorsch (HVM)

The St. Venant momentum and continuity equations are
applied in each sewer reach (20, 16). The backwater effects
are considered, which means that the sewer system is simulated
as an interdependent network and the effect of a certain
network.element on the remaining elements is taken into
account. The manholes, simulated as node points, are
treated by means of energy and continuity equations. Using
an implicit scheme, the numerical solution of the entire

system of equations is carried out using an iterative technique. The model simulates lateral flow, but it does not consider manhole storage. It handles retention basins by applying a reservoir routing technique.

(IV) The Storm Water Management Model (SWMM)

The SWMM is a comprehensive computer model used to simulate real or design storm events on an urban catchment (20, 26, 28, 32). The program is comprised of five major computational blocks, each simulating a different part of the Rainfall/Runoff cycle. The EXECUTIVE block is the mainline routine which controls the execution of the other computational blocks. The RUNOFF block computes storm water quantity and quality characteristics from each surface sub-catchment and then stores the hydrographs and pollutographs at designated inlet points to the main sewer system. The TRANSPORT block combines and routes the inlet hydrographs and pollutographs through the sewer system to the outlet. The STORAGE-TREATMENT block models the effects of off-line storage units and/or storm water treatment operations. The RECEIVING WATER block simulates the impact of storm water discharges on estuaries, lakes, and rivers.

In the RUNOFF block, the urban drainage basin consists of a series of rectangular sub-basins with a varying degree of imperviousness. The precipitation input onto these sub-catchments is taken as the precipitation reduced for the infiltration rates computed from the Horton Equation. The

rainfall excess over the detention depth is routed over the rectangular sub-basins using a linear kinematic wave approximation. The overland flow does not commence until the surface depression storage is filled. The gutter flow is calculated using the Manning's Equation and storage routing. The flows reaching the point of interest at any particular time are added to produce the inlet hydrograph. These inlet hydrographs are then routed through the major sewer pipes. The routing procedure is based on the continuity and normalized flow-area relationship calculated from Manning's Equation for uniform flow. Marsalek et al. (21) found that, after calibration, the RUNOFF block alone was adequate for their study of watersheds.

SWMM RUNOFF generates surface runoff hydrographs and routes them through the local drainage system. SWMM TRANSPORT then routes the hydrographs through the main drainage network (trunk sewer system). The flow routing is based on the quasi-steady dynamic wave approximation, this being a form of the St. Venant equations with the time rate of change of velocity term neglected. An explicit scheme has been adopted for the momentum equation, while an implicit scheme is used to solve the continuity equation. SWMM routes hydrographs independently of downstream conditions, so potential back-water effects are not considered. Moreover, it does not accurately simulate surcharge conditions. Excess inflows to a conduit are stored at the upstream manhole until sewer capacity becomes available. A hydraulic design option is

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included to revise pipes to accommodate free surface flows.

Table I.2 presents a comparison of several models, namely, high speed model developed by the Colorado State University (MWIS), SWMM of EPA, improved SWMM developed by Water Resources Engineers, Inc., (WRE), HVM, ISS, RRL, and Cincinnati (UCUR). The aspects of comparison are routing technique, integration scheme, backwater effects, surcharge, manhole storage conduit shapes treated, structures considered and availability.

rainfall hyetograph, I	

| rainfall reduction for surface storage $i = I \times p$ | i = effective rainfall
I = initial rainfall
p = % reduction factor |

| estimate total travel time $T = t_i + \dfrac{L}{V_{full}}$ | T = total travel time
t_i = inlet time |

| compute time vs contributing area curve $a_i = \dfrac{A}{T} \times \Delta t$ | L = length of pipe segment
V_{full} = full-flow velocity |

| compute runoff hydrograph
$Q_1 = a_1 i_1$
$Q_2 = a_2 i_1 + a_1 i_2$
$Q_3 = a_3 i_1 + a_2 i_2 + a_1 i_3$
etc. | a_i = incremental area
A = total impervious area tributary to pipe segment
Δt = time increment
Q = runoff hydrograph |

| add upstream hydrograph and route hydrograph through pipe segment

$\dfrac{\Delta t}{2}(Q_{1\ in} + Q_{2\ in} - Q_{1\ out}) + S_1$

$= \dfrac{\Delta t}{2} Q_{2\ out} + S_2$ | Q_{in} = inflow hydrograph
Q_{out} = outflow hydrograph
S = pipe storage from constant depth assumption or discharge – storage relationship |

All pipes considered?

No.

Yes

Outlet Hydrograph

FIGURE I.1 ALGORITHM OF THE RRL MODEL
(After Reference 20)

Model	Surface Routing	Sewer Routing	Quality Routing	Degree of Sophistication of Surface Flow Routing	Degree of Sophistication of Sewer Flow Routing	Accurate Modeling of Surcharging	Flexibility of Modeling of Sewer Components	Explicit Modeling of In-System Storage	Treatment Modelling	Receiving Water Body Model Available	Degree of Calibration/Verification Required	Simulation Period	Published Documentation	Data Requirements
EPA-SWMM	Yes	Yes	Yes	High	Moderate	No	High	Yes	Yes	Yes	Moderate	Individual storms	Good	Extensive
Cincinnati (UCUR)	Yes	Yes	Yes	High	Low	No	Low	No	No	No	Moderate	Individual storms	Fair	Extensive
RRL	Yes	Yes	No	Moderate	Low-Moderate	No	Low	No	NA	No	Moderate	Individual storms	Good	Moderate
Unit Pulse	Yes	Yes	No	Low-Moderate	Low-Moderate	No	Low	No	NA	No	Moderate	Individual storms	Fair	Moderate
Porsch (HVM)*	Yes	Yes	No	High	High	Yes	High	?	NA	Yes	Moderate	Individual storms or (separate) long term	Poor	Extensive
MIT*	Yes	No	No	High	NA	NA	NA	NA	NA	No	Moderate	Individual storms	Fair	Moderate
Rational Method	Yes Peak Flows Only	Peak Flows Only	No	Low	Low	No	Low	No	NA	No	Usually not verified	Individual storms	Good	Low
Chicago	Yes	No	No	Moderate	NA	NA	NA	NA	NA	No	Moderate	Individual storms	Fair	Moderate
Unit Hydrograph	Yes	In combination with surface	No	Low	Low	No	Low	No	NA	No	High	Individual storms	Fair	Moderate
STORM	Yes	In combination with surface	Yes	Low	Low	No	Low	No	Yes	No	Low	Long term	Good	Low
Battelle*	Yes	Yes	Yes	Low	Moderate	No	Moderate	No	No	No	Moderate	Individual storms	Poor	Moderate
WRE-SWMM*	Yes	Yes	Yes	High	High	Yes	High	Yes	Yes	Yes	Moderate	Individual storms	Poor	Extensive
SOGREAH*	Yes	Yes	?	High	High	Yes	High	?	?	Yes	Moderate	Individual storms	Poor	Extensive
Hydrocomp*	Yes	Yes	Yes	Moderate	Moderate	No	Low	No	No	Yes	High	Individual storms or long term	Fair	Extensive
Illinois (ISS)	Yes	Yes	No	Moderate	High	No	Low	No	NA	No	Moderate	Individual storms	Good	Extensive

* proprietary

Models Considered in This Study

Other Models

TABLE I.1   COMPARISON OF URBAN RUNOFF MODELS
(After Reference 20)

Model	Routing Technique	Integration Scheme	Does the model include?		
			Backwater effects	Surcharge	Manhole storage
MWIS	Muskingum combined with the momentum equation with the inertia terms excluded		No	No	No
SWMM of EPA	Quasi-steady dynamic-wave approximation of the St. Venant equation.	Implicit for continuity explicit for momentum Newton Raphson	No	No	No
WRE	$\frac{\partial Q}{\partial t} = -GAS_f + 2V\frac{\partial A}{\partial t} - gA\frac{\partial h}{\partial x}$ $\frac{\partial h}{\partial t} = \frac{\Sigma Qt}{A_{st}}$	Explicit modified Euler method	No	Yes	Yes
IVM	St. Venant equation with lateral flow	Implicit scheme	Yes	Yes	No
SS	St. Venant equations without lateral flow	Explicit first order characteristics scheme	Yes	No	Yes
RL	Hydrologic reservoir routing technique		No	No	No
CUR	Time offset technique		No	No	No

TABLE I.2   COMPARISON OF SELECTED RUNOFF MODELS
(After Reference 20)

Design of sewers — It has an optimization technique	Conduit shapes handled	Structures considered	Availability	Remarks
No	Circular and Trapezoidal	None	Nonproprietory	
No	Circular, Semi-elliptical, egg shaped	Pumps, flow dividers, internal storage units (weirs & orifices)	Proprietory	
No	Rectangular, circular horseshoe, basket-handle, eggshape, trapezoidal.	Weirs, pumps, tide gates, orifices	Proprietory	Studied presently by the University of Florida.
No	Any shape, by inputting the area-depth relationship	Retention rectangular basins, weirs	Proprietory	
Yes	Circular	None	Nonproprietory	Studies are carried on to add a surcharge subroutine
No	Any shape, by inputting the area-depth relationship	None	Nonproprietory	
No	Circular and rectangular	None	Nonproprietory	

TABLE I.2 (continued)

APPENDIX II

USER'S GUIDE TO SIMPLEX ALGORITHM PROGRAM

APPENDIX II

USER'S GUIDE TO SIMPLEX ALGORITHM PROGRAM

This procedure finds the maximum of a multivariable linear function subject to linear constraints:

$$\text{Maximize} \quad F = C_1 X_1 + C_2 X_2 + \ldots + C_N X_N \quad (II.1)$$

$$\text{Subject to} \quad A_{i1} X_1 + A_{i2} X_2 + \ldots + A_{iN} X_N \leq ,$$

$$= , \geq B_i \quad i = 1, 2, \ldots, M \quad (II.2)$$

$$X_1, X_2, \ldots, X_N \geq 0$$

where the A_{ij}, B_i, and C_j are given constants and the X_j are the decision variables.

The program is available in batch (Fortran or Watfiv) or on the terminal (Watfiv only) in the form of a Subroutine, with one parameter. The Subroutine is called SIMPLX.

The program requires that the Objective Linear Function be presented as Maximization problem. The function, the variables and the constraints must be given names of up to 5 characters long. The slack variables are generated with the names of the corresponding constraints. The program also requires that all the Right Hand Side (RHS) values of all the constraints be positive.

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The program can handle up to 75 rows (including the Objective Function) and 75 columns (structural variable and also the negative slack variables generated for GE, (>=) type of constraints). Multiple problems can be solved in one run of the program by stacking the sets of data for one problem followed by the next problem. From one problem to the next, if the only difference in data is the RHS values, then, the program can save the rest of the data to be used with the new RHS values. The program automatically prints the data for small problems (up to 8 columns) in a table form. Optionally, the program can also print the data as it is read in. This option is active for each call to the Subroutine and hence to all the problems being run by one call. You can call the Subroutine more than once in your program and use different options for different calls.

The program first tries to see if a solution is feasible. If the solution is not feasible, it prints a message to that effect, prints the values at that stage and completes normally. If the solution is feasible, it continues with the calculations. If a variable turns out to be unbounded, it will print a message and terminate. Otherwise, it will complete the optimization and then reach a normal termination.

Following is the format and order of one set of data for one problem. Word(s) written in capital letters for given columns must be punched exactly as they are.

1. Title Card: 1 card per problem
    col. 1 - 80: Title information

2. Row Id Card: 1 card per problem
   - col. 1 - 6: ROW ID
   - col. 8 - 11: CONT, if same data, except for RHS values, is to be used for this as well as the following problem. Otherwise, blank.

3. Row Identification Cards: As many as there are rows, including the Objective Function, which must be the first card

   col. 12 : Constraint Symbol; = for =
   Only > is used to indicate >= and only < is used to indicate <=
   col. 14 - 18: Row Name

4. Matrix Header Card: 1 card per problem
   col. 1 - 6: MATRIX

5. Matrix Element Cards: As many as non-zero elements, placed in column order and for each column, rows in the same order as they appear in the Row Identification Cards
   col. 8 - 12: Column Name
   col. 14 - 18: Row Name
   col. 19 - 20: Element Value including decimal point, and sign if needed

6. RHS Header Card: 1 card per problem
   col. 1 - 7: FIRST B

7. RHS Value Cards: As many as there are constraints
   col. 14 - 18: Row Name
   col. 19 - 30: RHS Value including decimal point (must be positive value)

8. Problem Delimeter Card: 1 card per problem
   col. 1 - 3: EOF
   col. 14 - 17: CONT, if in the following problem data, only RHS values are being provided

The parameter in the CALL to the Subroutine can be either 0 or 1. If it is 1, then, the program prints the input data as it is read. Otherwise, it will not print the data.

EXAMPLE:

```
CALL SIMPLX(  0  )

CALL SIMPLX(  1  )
```

APPENDIX III

LISTING OF THE FIRST PROGRAM

```
$JCB    WATFIV  X>>XXXXXXXX NAME-IMAM
```

```
      ************************************************************
      * THIS PROGRAM OBTAINS  OPTIMUM DESIGN FOR PUMPED          *
      * SEWER SYSTEMS.                                           *
      ************************************************************
      * MAXIMUM NUMBER OF INLET POINTS (MANHOLES)  =  4          *
      * MAXIMUM NUMBER OF SEWERS  =  3                           *
      ************************************************************
      COMMON /AREA0A/ A,B,C,R
      COMMON /AREA7C/AA,BB,CC,RR
      COMMON /AREA71/ PROB
      COMMON /AREA72/ NSTORM
      COMMON /AREA73/ A1,B1,C1
      COMMON /AREA74/ DAM
      COMMON /AREA75/ AD
      COMMON /AREA81/ TDAM
      COMMON /AREA82/ AD1
      COMMON /AREA83/ NCHNG
      COMMON /AREA84/ D(5,3)
      COMMON /AREA91/ TANC
      COMMON /AREA92/ COSTP(5)
      COMMON /AREA94/ CINITL
      DIMENSION   AA(7),BB(7),CC(7),RR(7)
      DIMENSION   PROB(7),DAM(7)
C     -----------------------------------
C     READ   INPUT   DATA
C     -----------------------------------
      CALL   INPUTD
      CALL   ELVEAS
      DO  50   I = 1,NSTORM
      A = AA(I)
      B = BB(I)
      C = CC(I)
      R = RR(I)
      CALL   RAINF
      CALL   INFHYD
      CALL   PRINT2
C     -----------------------------------------------------
C     BALANCE THE DESIGN FOR THE DESIGN STORM
C     -----------------------------------------------------
      IF( I .EQ.  1)  CALL  HYDMB
C     -----------------------------------------------------
C     EVALUATE THE TOTAL DAMAGE FOR THE BALANCED DESIGN FOR
C     VARIOUS STORMS(DIFFERENT RETURN PERIOD).
C     -----------------------------------------------------
      CALL   HYDMOD
      DAM(I) = TDAM
   50 CONTINUE
      CALL   ANCOST
      CINITL = TANC
C     -----------------------------------------------------
C     GENERATE THE DAMAGE-PROB. EQUATION FOR THE   ORIGINAL
C     BALANCED DESIGN.
C     -----------------------------------------------------
      CALL   DAMPRO
C     -----------------------------------------------------
C     GENERATE THE AVERAGE ANNUAL PROBABLE DAMAGE ARRAY.
C     -----------------------------------------------------
```

```
      DC  60  I = 1,5
      D(I,1) = AO
      D(I,3) = 0.5* A1
   60 CCNTINUE
C     STCRE THE ORICINAL BALANCEO DESIGN
      CALL  STCRE
      DC  100  NCHNG=1,5
C     PERTURE THE EALANCED SYSTEM WITH RESPECT TO THE I-TH
C     DECISICN VARIAELE.
      CALL  CHANGE
C     APFLY THE REPRESENTATIVE STORM TO THE PERTURBED SYSTEM
      A = AA(2)
      B = BB(2)
      C = CC(2)
      R = RF(2)
      CALL  RAINF
      CALL  INFHYC
      CALL  HYCMOD
      CAM(2) = TDAM
C     CCMPUTE THE AVERAGE ANNUAL PROBABLE CAMAGE FOR THE
C     PERTUREEC SYSTEM.
      CALL  CPI
      D(NCHNG,2) = ACI
      CALL  ANCOST
      CUSTP(NCHNG) = TANC
C     RESET THE MCDEL
      CALL  ORIGNL
  100 CCNTINUE
C     PRINT OUT THE CATA NECESSARY FOR THE CPTIMIZATIJN
C     ALGORITHN,  CO,  CO,  A
      FRINT  1000, CINITL, AC, A1
      CALL  GRAD1
      CALL  GRADIE
      CALL  PRINT1
 1000 FCRMAT(////,10X,'ANNUAL CUST OF INSTALLATICN',/, 10X,
     @'AND OPERATICN CF THE BALANCED SEWER SYSTEM=CO=$',F12.1,////,
     @10X,'AVEPAGE ANNUAL PRCBABLE CAMAGE',/, 10X,
     @'RESULTING FROM THE BALANCED SEWER SYSTEM=CO=$',F12.1,////,
     @10X,'ULTIMATE POSSIBLE CAMAGE RESULTING',/, 10X,
     @'FRCM THE EALANCED SEWER SYSTEM=A=$',F12.1,///)
      STCP
      ENC



      SUBROUTINE  INPUTD
C     *****************************************************
C     * THIS SUBRCUTINE READS IN ALL THE INPUT CATA       *
C     *****************************************************
C     * INPUT  CATA  CARDS ARE AS FOLLOWS :               *
C     * 1 ST CARD (( NUMBER  OF MANHOLES, I3 ))           *
C     * NMH..CARCS(FCR EACH MNH,GROUND SURFACE ELEV.(FT), *
C     *            COEF. OF INLET CATCH BASIN(CB), EFFECTIVE
C     *            NO. OF VEHICLE CROSSINGS AT MANHOLE(INTER
C     *            SECTION PER HOUR. ON THE AVERAGE BASIS. *
C     *            FORMAT USED 3F10.0                      *
C     * NMH...CARCS((FOR EACH MANHOLE:IMPERVICUS APEA(ACRES)
C     *            TIME OF CCNCENTRATION OF SUBCATCHMENT AREA
C     *            (MINUTES),FORMAT USED 2F6.3             *
C     * CNE  CARC ((TIME INCREMENT(MINUTES), F4.1 ))      *
C     * NPIPE...CARCS ((FOR EACH PIPE ;MANNING'S N,LENGTH, *
C     *            DIAMETER,SLOPE,FCRMAT USED  4F10.4))    *
C     * NPIPE...CARCS(FOR EACH PIPE,AREA OF BASEMENTS (ACRES
C     *            ) CONTRIBUTING TO THE PIPE,AVERAGE      *
C     *            CEPTH OF EASEMENTS BELOW G.S.(FT),      *
C     *            CISTRICT INDEX,FORMAT USED 2F10.0,I1.   *
C     * CNE  CARD ( NUMBER OF STORMS CONSICERED,FORMAT I 3)*
C     * NSTORM  CARCS(FOR EACH STORM,A,B,C,R,PROB;FORMAT5F10.0
C     * CNE CARC (EASE FLOW DISCHARGE,CFS,FGRMAT USED,F10.0*
```

```
*   NPIPE...CARDS(FOR EACH PIPE,AVERAGE LENGTH OF HOUSES
*               CONNECTION(FT),F10.0,DIAMETER OF        *
*               HOUSE CONNECTION(FT),F10.0,NUMBER OF    *
*               HOUSES CONNECTED TO THE SEWER, I3. )    *
*   CNE   CARD (PUMP CHARACTERISTICS ,RELATIVE EFFICIENCY*
*               ,RATED HEAD (FT), RATED DISCHARGE(CFS)  ,*
*               FORMAT USED 3F10.0)                     *
*   CNE   CARD (SUMP CHARACTERISTICS,AREA OF THE SUMP(FT**
*               FT),CHANNEL WATER LEVEL (FT),FORMAT USED *
*               2 F10.0.                                *
*   CNE   CARD ( CROWN ELEV. OF THE U/S END OF THE 1 ST *
*               PIPE IN THE SYSTEM.(FT),FORMAT USED  F10.0
*   CNE   CARD (POSITIVE RELATIVE CHANGES IN DESIGN VARIABLES
*               WITH RESPECT TO ORIGINAL DESIGN (POS =  *
*               LOWERING DAMAGE). FORMAT USED  5F10.0   *
*               1. SLOPES   2. QR*HR   3. ASUMP   4. OVERALL*
*               DEPTH OF THE SYSTEM   5. DIAMTERS       *
*   CNE   CARD (NEGATIVE RELATIVE CHANGES IN DESIGN     *
*               VARIABLES WITH RESPECT TO ORIGINAL DESIGN*
*               (NEG = INCREASING DAMAGE).FORMAT USED 5F10.0
*************************************************************
*   NMH = NUMBER OF MANHOLES                            *
*   ARIMP(I) = IMPERVIOUS AREA OF PIPE (I)              *
*   T   = TIME INCREMENT BETWEEN SUCCESSIVE ELEMENTS    *
*         , MEASURED IN MINUTES                         *
*   AN(I) = MANNING'S   N OF PIPE (I)                   *
*   ALNTH(I) = LENGTH OF PIPE (I). FT                   *
*   DIAM(I) = DIAMETER  OF PIPE(I) .FT.                 *
*   SO(I) = INVERT SLOPE OF PIPE (I)                    *
*   A,B,C = COEFFICIENTS IN THE RAINFALL INTENSITY      *
*         / TIME FORMULA , I = A/((T**B + C)            *
*   R   = RELATIVE ADVANCEMENT OF PEAK OF SYNTHETIC     *
*         STORM                                         *
*   PROB(X) = PROBABILITY OF OCCURRENCE OF THIS STORM   *
*   QINLET = LONG TERM DISCHARGE,CFS,(BASE FLOW), OR    *
*            INLET HYDROGRAPH FROM PREVIOUS SYSTEM.     *
*   H(I) = GROUND SURFACE ELEVATION OF THE I TH MNH(FT)*
*   ABASE(K) = BASEMENTS AREA DISCHARGING TO THE K TH   *
*              PIPE (ACRES).                            *
*   DBSMT(K) = AVERAGE DEPTH OF BASEMENTS BELOW THE G.S.*
*              (FEET), OF THE  K TH PIPE                *
*   USELEV = CROWN ELEV. OF THE U/S END OF THE 1 ST PIPE*
*              IN THE SYSTEM.                           *
*************************************************************
      COMMON/AREA/   T
      COMMON /AREA0/ NMH,ARIMP,ET
      COMMON /AREA0A/ A,B,C,R
      COMMON /AREA1A/        SO
      COMMON /AREA2B/ QINLET
      COMMON /AREA5A /AN
      COMMON/AREA5B / DIAM
      COMMON/AREA6B/NPIPE
      COMMON /AREA7B /ALNTH
      COMMON /AREA8H/  QBAFL
      COMMON /AREA22/  H
      COMMON /AREA13/ ABASE
      COMMON /AREA23 / DBSMT
      COMMON /AREA25/DHCON,ALPIPE
      COMMON /AREA26/ NHOUS
      COMMON/AREA28/ ASUMP
      COMMON/AREA29/ECHANL
      COMMON/AREA30/SBCTCM
      COMMON/AREA31/ RELETA,HR,QR
      COMMON/AREA32/ ENDSEW
      COMMON /AREA38/   CB(4)
      COMMON /AREA49/ ICLASS(3), VEH(3)
      COMMON /AREA55/   USELEV
      COMMON /AREA70/ AA,BB,CC,RR
      COMMON /AREA71/ PROB
      COMMON /AREA72/ NSTORM
      COMMON /AREA80/ CHGPOS(5),CHGNEG(5)
```

```
      DIMENSION   AA(7),BB(7),CC(7),RR(7),PROB(7)
      DIMENSION   NHOUS(3)
      DIMENSION   CHCCN(3),ALPIPE(3)
      DIMENSION   H(4),ABASE(3),OBSMT(3)
      DIMENSION   AN(3), ALNTH(3) ,DIAM(3), SO(3)
      DIMENSION   ARIMP(4), ET(4)
      DIMENSION   QINLET(220)
      DIMENSION   VEHH(4)
      READ   1980 ,NMH
      DO   16   I = 1,NMH
   16 READ   1985 ,H(I),CB(I), VEHH(I)
      NPIPE = NMH - 1
      DO   18   I = 1, NPIPE
   18 VEH(I) = VEHH(I)
      DO   14   I = 1,NMH
   14 READ   1990 ,ARIMP(I),ET(I)
      READ   2000, T
      DO   12   I = 1,NPIPE
      READ   1900,AN(I),ALNTH(I),DIAM(I),SO(I)
   12 CONTINUE
      DO   11   I = 1,NPIPE
   11 READ 2010, ABASE(I), OBSMT(I), ICLASS(I)
C     --------------------------------
C       RAINFALL STORMS  DATA
C     --------------------------------
      READ   1980,NSTORM
      DO   500   I = 1,NSTORM
      READ      1800,AA(I),BB(I),CC(I),RR(I),PROB(I)
  500 CONTINUE
      READ   2100,  QBAFL
      DO   10   I = 1,220
   10 QINLET(I) =   QBAFL
      DO   100   I = 1,NPIPE
  100 READ   2300 ,ALPIPE(I),CHCCN(I)   ,NHOLS (I)
      READ   2400,RELETA,HR,CR
      READ   2410 , ASLMP, ECHANL
      READ   2100, USELEV
      ENDSEW = USELEV
      DO   50   I = 1, NPIPE
   50 ENDSEW = ENDSEW - SO(I) * ALNTH(I)
C     THE SUMP LOWEST W.L. WILL BE SET 3.0 FT BELOW THE D/S
C     INVERT OF THE LAST PIPE.(DEFAULT VALUE)
      SECTOM = ENDSEW - DIAM(NPIPE) - 3.0
      READ   2500,(CHGPCS(I),I = 1,5)
      READ   2500,(CHGNEG(I),I = 1,5)
C     --------------------------------
C       FORMAT  STATMENTS   FOR   INPUT   DATA
C     --------------------------------
 1800 FORMAT(5F10.0)
 1900 FORMAT    (4F10.0)
 1980 FORMAT( I3)
 1985 FORMAT(3F10.0)
 1990 FORMAT( 2F6.0)
 2000 FORMAT(F4.0)
 2010 FORMAT(2F10.0,I1)
 2100 FORMAT( F10.0)
 2300 FORMAT(2F10.0,I3)
 2400 FORMAT(3F10.0)
 2410 FORMAT(2F10.0)
 2500 FORMAT(5F10.0)
      RETURN
      END
```

```
      SUBROUTINE   PRINT1
C     **************************************************
C     * THIS SUBROUTINE IS USED TO PRINT OUT THE INPUT DATA*
C     *(IF DESIRED). ALSO IT PRINTS AND PLOTS THE RAINFALL *
C     * HYETOGRAPH.
C     **************************************************
      COMMON/AREA/ T
      COMMON /AREA0/ NMH,ARIMP, ET
      COMMON /AREA0A/ A,B,C,R
      COMMON /AREA0B/ RAIN
      COMMON /AREA0C/ SPAN
      COMMON /AREA0P/ PRINTT,PRINTO
      COMMON /AREA1A/ SO
      COMMON /AREA5A/ AN
      COMMON /AREA5B / DIAM
      COMMON /AREA6B/ NPIPE
      COMMON /AREA7E/ ALNTH
      COMMON /AREA13/ ABASE
      COMMON /AREA22/ H
      COMMON /AREA23/ DBSMT
      COMMON /AREA25/ CHCON,ALPIPE
      COMMON /AREA26/ NHOUSE
      COMMON /AREA28/ ASUMP
      COMMON /AREA29/ ECHANL
      COMMON /AREA30/ SBOTOM
      COMMON /AREA31/ RELETA,HR,QR
      COMMON /AREA32/ ENDSEW
      COMMON /AREA38/ CE(4)
      DIMENSION   RAIN(220)
      DIMENSION   PRINTT(220), PRINTO(220)
      DIMENSION   AN(3),ALNTH(3),DIAM(3),SO(3)
      DIMENSION   ARIMP(4), ET(4)
      DIMENSION   H(4),DBSMT(3),ABASE(3)
      DIMENSION   CHCON(3),ALPIPE(3),NHOUSE(3)
      INTEGER  SPAN , SPAN1
      FLAG1 =0.0
      FLAG1 = 1.0
      IF ( FLAG1.EQ. 0.0) GO TO  100
      PRINT  3000
      PRINT  1000
      PRINT  8000
      DO  10  I = 1,NMH
10    PRINT  8010,I,H(I),ARIMP(I),ET(I),CE(I)
      PRINT  1000
      PRINT  8020
      DO  20   I = 1,NPIPE
20    PRINT  8030,I,AN(I),ALNTH(I),DIAM(I),SO(I)
      PRINT  1000
      PRINT  8040
      DO  30   I = 1,NPIPE
30    PRINT  8050,I,NHOUSE(I),ABASE(I),DBSMT(I)
      PRINT  1000
      PRINT  8060
      DO  40   I = 1,NPIPE
40    PRINT  8070,I,ALPIPE(I),CHCON(I)
      PRINT  1000
      PRINT  8080,RELETA,HR,QR
      PRINT  1000
      PRINT  8090,ASUMP,SBOTOM
      PRINT  1000
      PRINT  3010,T
      PRINT  1000
      PRINT  4010
      PRINT  4020 ,A,B,C
      PRINT  4030 ,  R
      PRINT  1000
100   CONTINUE
      IFLAG2 = 0
      IFLAG2 = 1
      IF(IFLAG2.EQ.  0 )  GO TO  200
      SPAN1 = SPAN+ 1
      DO  110 I = 1,220
```

```
      PRINTT(I) = 0.0
  110 PRINTO(I) = 0.0
      DO  45 I = 1 ,SPAN1
      IK = I*T  - T
      PRINTT(I) = IK
      PRINTO(I) = RAIN(I)
   45 PRINT 3030,IK.,RAIN(I)
      CALL PLOT  3 (PRINTT,PRINTO, 100)
  200 CONTINUE
C     ------------------------------
C              FORMAT  STATMENTS FOR   PRINTING
C     ------------------------------
 1000 FORMAT(10X,'----------------------------------
     @-----------------------------------------------')
 3010 FORMAT(10X,'TIME INSREMENT THROUGHOUT THE PROGRAM'
     @,F4.1,5X,'MINUTES',////)
 3000 FORMAT (1H1,30X,'PRINTOUT OR INPUT DATA ',//////)
 4010 FORMAT(10X,'COEFFICIENTS IN THE RAINFALL INTENSITY/TIME FORMULA')
 4020 FORMAT(10X,'A =',F5.1,/,
     @        10X,'B = ',F5.1,/,
     @        10X,'C = ',F5.1,///)
 4030 FORMAT(10X,' RELATIVE ADVANCEMENT OF THE PEAK OF THE SYNTHETIC
     @        STORM  = ',F5.3,//)
 3030 FORMAT(10X,'TIME = ',I3,3X,'MINUTES',
     @'RAINFALL INTENSITY = ',F6.2,3X,'INCHES / HR')
 8000 FORMAT(10X,'MNH NO.',5X,'ELEV.(FT)',5X,'IMP. AREA(ACRE)',
     #5X,'INLET TIME(MIN.)',8X,'CB',////)
 8010 FORMAT(15X,I2,10X,F6.1,12X,F5.2,15X,F4.1,14X,F5.3)
 8020 FORMAT(10X,'PIPE NO.  MANNING   LENGTH(FT) DIAM.(FT)    SLOPE'
     @,///)
 8030 FORMAT(13X,I2,7X,F5.3,6X,F5.1,5X,F4.2,6X,F5.3)
 8040 FORMAT(10X,'PIPE NO.  N.HOUSES  ARASE(ACRE)   BSMT DEPTH(FT)',
     @///)
 8050 FORMAT(13X,I2,8X,I2,8X,F4.2,11X,F4.1)
 8060 FORMAT(10X,'PIPE NO.  L.  HOUSE CONNECTION(FT)  DIAM.H.CON.(FT)')
 8070 FORMAT(13X,I2,13X,F5.1,17X,F4.2)
 8080 FORMAT(10X,'RELATIVE EFFICIENCY = ',F4.2,/,
     @        10X,'RATED HEAD = ',F4.1,'  FT',/,
     @        10X,'RATED DISCHARGE = ',F4.1,'  CFS',/)
 8090 FORMAT(10X,'SUMP AREA = ',F6.1,'  FT*FT',/,
     @        10X,'SUMP LOWEST W.L. = ',F7.2,'  FT',/)
      RETURN
      END




      SUBROUTINE  PRINT2
C     *********************************************************
C     * THIS SUBROUTINE PRINTS OUT BASIC INFLOW HYDROGRAPHS*
C     * AT INLET POINTS (MANHOLE JUNCTIONS).              *
C     *********************************************************
      COMMON /AREA/ T
      COMMON /AREA0/ NMH,ARIMP,ET
      COMMON /AREA2A / INFHD
      DIMENSION  INFHD(4,220)
      DIMENSION  ARIMP(4), ET(4)
      REAL  INFHD
      IFLAGS = 0
      IFLAGS = 1
      IF( IFLAGS .EQ.0 )  GO  TO  210
      DO  190  IMH = 1,NMH
      PRINT  3040,IMH
      DO 180 I = 1,220
      IK1 =I*T
   74 PRINT 3050, IK1, INFHD(IMH,I)
      IF(INFHD(IMH,I).LT. 0.0001) GO TO  185
  180 CONTINUE
  185 CONTINUE
  190 CONTINUE
  210 CONTINUE
 3040 FORMAT(10X,'MANHOLE NUMBER =',I3)
 3050 FORMAT(10X,'TIME =',I3,'   MINUTES',
     X10X,'INFLOW DISCHARGE =',F10.5,'   CFS')
      RETURN
      END
```

```
      SUBROUTINE  RAINF
C     ***********************************************************
C     * THIS SUBROUTINE IS USED TO GENERATE THE RAINFALL       *
C     * INTENSITY ARRAY, ASSUMING STORM LENGTH OF 180 MINUTES
C     ***********************************************************
C     * RAIN(I) = RAINFALL INTENSITY AT TIME (I)               *
C     * SPAN = NUMBER OF TIME INCREMENTS IN THE STORM          *
C     * A,B,C,R,T   HAVE BEEN DEFINED IN SUBROUTINE INPUTD     *
C     ***********************************************************
      COMMON /AREA/ T
      COMMON /AREA0A/ A,B,C,R
      COMMON /AREA0B/ RAIN
      COMMON /AREA0C/ SPAN
      DIMENSION  RAIN(220)
      INTEGER  SPAN
      INTEGER  SPAN1
      DO  15  I =1,220
   15 RAIN(I)=0.
      SPAN =180.0/T
      TB =180.0*R
      BEFPK =TB
      K = BEFPK/T
C     RAINFALL INTENSITIES BEFORE PEAK.
      DO  25  I =1,K
      RAIN(I) =   A*((1.0-B)*((BEFPK/R)**B)+C)/((((BEFPK/R)
     X**B)+C)**2)
      BEFPK =BEFPK -T
   25 CONTINUE
      J =K +1
      TA = 0.0
      SPAN1 = SPAN  + 1
C     RAINFALL INTENSITIES AFTER PEAK
      DO  35  (I = J, SPAN1
      RAIN(I)=A*((1.0-B)*((TA/(1.0-R))**B)+C)/
     X(((((TA/(1.0-R))**B)+C)**2)
      TA =TA +T
   35 CONTINUE
      RETURN
      END




      SUBROUTINE   INFHYD
C     ***********************************************************
C     * THIS SUBROUTINE COMPUTES THE ORDINATES OF THE INFLOW   *
C     * HYDROGRAPH, FLOWING THROUGH THE MANHOLES TO THE        *
C     * U/S END OF THE CONSIDERED PIPE                         *
C     ***********************************************************
C     * INFHD(I,IT) = INFLOW DISCHARGE,MNH NO.I ,AT TIME IT*
C     * ET(I) = TIME OF CONCENTRATION OF THE I-TH BASIC        *
C     *           SUBATCHMENT,MINUTES.                         *
C     AFIMP(I) = IMPERVIOUS AREA CONTRIBUTING TO MNH (I)       *
C     ***********************************************************
      COMMON /AREA/   T
      COMMON /AREA0 /NMH,AFIMP,ET
      COMMON /AREA0B/ RAIN
      COMMON /AREA0C /SPAN
      COMMON /AREA2A/ INFHD
      DIMENSION  INFHD(4,220),ET(4),ARIMP(4)
      DIMENSION  RAIN(220)
      INTEGER  SPAN
      REAL  INFHD
      DO  55  J=1,NMH
      DO  55  I = 1,220
   55 INFHD(J,I) =0.0
      DO  75  IMH =1,NMH
      NARINC =ET(IMH)/T
      ARINC =ARIMP(IMH)/NARINC
      KS =1.5*SPAN
      DO 75 I = 1,KS
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      SUM =0.0
      DO  65 J=1,NARINC
      KK = I-(J-1)
      KK1 = KK + 1
      IF (KK).60,60,70
   60 CALCA = 0.0
      GC TO  64
C     RAIN(I) IS THE RAINFALL INTENSITY AT THE BEGINNING OF
C     INTERVAL (I). AVERAGE RAINFALL INTENSITY DURING
C     INTERVAL (I) WILL BE = (RAIN(I) + RAIN(I+1))/2.0
   70 CALCA = (RAIN(KK) + RAIN(KK1))* ARINC /2.0
   64 CONTINUE
   65 SUM = SUM + CALCA
      INFHC(IMH,I) =SUM
   75 CONTINUE
      RETURN
      END



      SUBROUTINE  MUSKRT
C     **********************************************************
C     *    THIS SUBROUTINE COMPUTES THE DOWNSTREAM DISCHARGE   *
C     *    FOR A CIRCULAR PIPE GIVEN THE INFLOW HYDROGRAPH     *
C     *    AND PIPE CHARACTERISTICS.                           *
C     **********************************************************
C     *    DIAM = DIAMETER IN FEET                             *
C     *    ALNTH = PIPE LENGTH IN FEET                         *
C     *    SO = PIPE SLOPE IN FT/FT                            *
C     *    AN = MANNING'S  N                                   *
C     *    T  = TIME INCREMENT BETWEEN SUCCESSSIVE ELEMENTS    *
C     *    (MINUTES),ROUTING PERIOD(BASED UPON "K" UNITS       *
C     * KMUS = STORAGE CONSTANT IN MUS KINGUM METHOU           *
C     * CO,C1,C2 = MUSKINUM COEFFICIENTS                       *
C     *       ( Q(2) =CO*I(2) +C1 *I(1) +C2 *O(1))             *
C     * COUT = OUTFLOW DISCHARGE AT D/S END OF SEWER           *
C     *  QINF = INFLOW DISCHARGE THROUGH THE U/S END THE SEWER *
C     **********************************************************
      COMMON /AREA/ T
      COMMON /AREA1A/  SO
      COMMON /AREA2E/  QINLET
      COMMON /AREA7E/   ALNTH
      COMMON /AREA5E/  DIAM
      COMMON /AREA5A/  AN
      COMMON /AREA4A / IP
      COMMON /AREA4E/ QINF, QOUT
      COMMON /AREA8/ IT
      COMMON /AREA9A/ KMUS
      DIMENSION   AN(3),ALNTH(3),DIAM(3), SO(3)
      DIMENSION  QINF(3,220),QOUT(3,220)
      DIMENSION  QINLET(220)
      DIMENSION  KMUS(3)
      REAL KMUS
      X =0.5
      KMUS(IP) = AN(IP)*ALNTH(IP)*1.696/(DIAM(IP)**0.667)/
     *     (SO(IP)**0.5)
C     T WILL ADJUSTED TU THE SAME UNITS AS KMUS (SECONDS)
      CC = KMUS(IP) - KMUS(IP) * X + 30.0 * T
      CO = (30.0 * T - KMUS(IP) * X)/ CC
      C1 = ( KMUS(IP) * X + 30.0 * T) / CC
      C2 = (KMUS(IP ) - KMUS(IP) * X - 30.0 * T) / CC
      IF(IT. NE.1 )  GO  TO  50
      QOUT(IP,1) = QINF(IP,1)*CO+C1*QINLET(1)+C2*QINLET(1)
      GC  TO  110
   50 CONTINUE
      IT1 = IT - 1
      QOUT(IP,IT) = QINF(IP,IT) * CO + QINF(IP,IT1) * C1
     @              + QOUT(IP,IT1) * C2
  110 CONTINUE
      RETURN
      END
```

```
      SUEROUTINE  ACCHYD
C     *****************************************************
C     * THE ORDINATES OF THE HYDROGRAPHS CONTRIBUTING TO  *
C     * THE MANHOLE JUNCTION BEING CONSIDERED ARE ADDED, TO*
C     * OBTAIN THE INFLOW TO THE DOWNSTREAM SEWER.        *
C     *****************************************************
C     * QINLET = BASE FLOW                                *
C     * QINF = INFLOW TO THE UPSTREAM END OF THE SEWER.   *
C     *****************************************************
      COMMON/AREA2A/ INFHD
      COMMON /AREA2B/  QINLET
      COMMON /AREA4A/ IP
      COMMON /AREA4B/  QINF,QOUT
      COMMON /AREA8/   IT
      DIMENSION QINLET(220),INFHD(4,220)
      DIMENSION  QOUT(3,220),QINF(3,220)
      REAL   INFHD
      IF(IP .GT.1) GO TO 100
      QINF(IP,IT)    = QINLET(IT) + INFHD(IP,IT)
      GO TO 210
  100 CONTINUE
      IPP = IP- 1
      QINF(IP,IT) =QOUT(IPP,IT) +INFHD(IP,IT)
  210 CONTINUE
      RETURN
      END




      SUBROUTINE  HYDMOD
C     *****************************************************
C     * THIS SUBROUTINE IS USED FOR HYDRAULIC SIMULATION  *
C     * OF FLOW IN THE SEWER SYSTEM.                      *
C     *****************************************************
      COMMON /AREA1A/  QBSMT
      COMMON /AREA4A/ IP
      COMMON /AREA4B/  QINF,QOUT
      COMMON/AREA6E/ NPIPE
      COMMON/AREA8/   IT
      COMMON /AREA9A/ KMUS
      COMMON /AREA9B/ FMAX(3)
      COMMON /AREA8I/ TDAM
      COMMON /AREA9C/  M
      DIMENSION   QBSMT(3,220)
      DIMENSION   QINF(3,220),QOUT(3,220)
      DIMENSION  KMUS(3)
      REAL  KMUS
      M = 0
      DO  50  I = 1,NPIPE
   50 FMAX(I) = 0.0
      CALL   KENERG
      CALL   VOLSYS
      DO  100   ITT= 1,180
      IT = ITT
      IF( M  .NE.  0 ) GO  TO  105
      DO  90   IPK = 1,NPIPE
      IP = IPK
      CALL   ACCHYD
      CALL   MUSKRT
   90 CONTINUE
      CALL  SUMPI
      CALL   SWITCH
  100 CONTINUE
  105 CONTINUE
      DO  110  I = 1, NPIPE
      DO  110  J = 1,220
      QBSMT(I,J) = 0.0
  110 CONTINUE
      IF( M .EQ. 0)  GO  TO  120
      CALL   PRFSML
  120 CONTINUE
```

```
      IF( N .EQ.  1) GO  TO  130
      TDAM  = 0.0
130 CONTINUE
      RETURN
      END



      SUBROUTINE  SUMP1
C     *****************************************************
C     * THIS SUBROUTINE COMPUTES   THE SUMP WATER LEVEL AND*
C     * THE ELEVATION OF THE D/S END OF THE E.L. OF THE N TH *
C     * PIPE.( OUTFALL ENERGY LEVEL)                       *
C     *****************************************************
C     * RELETA = OPERATING EFFICIENCY/RATED EFFICIENCY    *
C     * HR     = RATED HEAD (FT)                          *
C     * QR     = RATED DISCHARGE (CFS)                    *
C     * QSUMP1 = DISCHARGE FLOWING TO SUMP AT TIME (T - 1) *
C     * QSUMP  = DISCHARGE FLOWING TO SUMP AT TIME (T)     *
C     * HP1    = PUMPING HEAD AT TIME (T - 1)             *
C     * HP     = PUMPING HEAD AT TIME (T)                 *
C     * ECHANL = ELEVATION OF THE ENERGY LINE D/S OF THE   *
C     *          PUMP( DELIVERY ELEVATION),COULD BE CHANNEL*
C     *          WATER LEVEL.                             *
C     * SBOTOM = ELEVATION OF MINIMUM SUMP WATER LEVEL,START*
C     * ING AND STOPPING LEVEL.                           *
C     * SWL    = SUMP WATER LEVEL                          *
C     * ESUMP( )= ELEVATION OF THE D/S END OF THE ENERGY   *
C     *          LINE OF THE N TH PIPE.                   *
C     *****************************************************
      COMMON /AREA/   T
      COMMON /AREA2A/  INFHD
      COMMON /AREA2E/  QINLET
      COMMON /AREA4E/  QINF,QOUT
      COMMON /AREA5E/  DIAM
      COMMON /AREA6E/  NPIPE
      COMMON /AREA8/   IT
      COMMON /AREA8D/  ITS
      COMMON /AREA9C/  M
      COMMON /AREA20 /ESUMP
      COMMON /AREA28/ ASUMP
      COMMON/AREA29/ECHANL
      COMMON/AREA30/SBOTOM
      COMMON/AREA31/ RELETA,HR,QR
      COMMON/AREA32/ ENDSEW
      COMMON /AREA40/ SWL,QSUMP
      DIMENSION   DIAM(3)
      DIMENSION   QINLET(220)
      DIMENSION   QOUT(3,220),QINF(3,220)
      DIMENSION   INFHD(4,220)
      DIMENSION   ESUMP(220)
      REAL   INFHD
      IF( M .EQ.   0 )  GO  TO  30
      K = ITS
      GO  TO  35
30 K = IT
35 CONTINUE
      ANPIPE = 0.7854* DIAM(NPIPE)**2
      CPUMP = RELETA * HR * QR
      C = T * CPUMP * 30./ ASUMP
      IF( K .NE. 1 )  GO  TO  40
      QSUMP1 = QINLET(1)
      QSUMP = QOUT(NPIPE,1) + INFHD(NPIPE+1,1)
      HP1 = ECHANL - SBOTOM
      GO  TO  50
40 QSUMP1 = QOUT(NPIPE,K-1) + INFHD(NPIPE+1,K-1)
      QSUMP = QOUT(NPIPE,K) + INFHD(NPIPE+1,K)
      HP1 = ECHANL - SWL
50 CONTINUE
      QSUMPA= (QSUMP+ QSUMP1)/ 2.0
      B =(-1.0)*HP1 + 2.0*C*QSUMPA/CPUMP - C/ HP1
      HP =(-0.5)*B + 0.5* SQRT(B*B + 4.0*C)
      HPMAX = ECHANL - SBOTOM
      IF(HP .GT. HPMAX) HP = HPMAX
```

```
      SWL = ECHANL - HP
      VNPIPE = QOUT(NPIPE,K)/ ANPIPE
      YHEAD = VNPIPE*VNPIPE/64.4
      IF(SWL.LT.ENDSEW)  GC  TJ   80
      ESUMF(K) = SWL + VHEAD
      GO  TO  100
   80 ESUMF(K) = ENDSEW + VHEAD
  100 CCNTINUE
      RETURN
      END



      SUBROUTINE  STRNCD
C     ********************************************************
C     * THIS SUBROUTINE COMPUTES THE STORM WATER FLOWING    *
C     * UPWARD THROUGH THE I-TH MANHOLE BY APPLYING A MASS  *
C     * BALANCE EQUATION.                                   *
C     ********************************************************
C     * SEWHL     = HEAD LOSS THROUGH PIPE.                 *
C     * CIN       = INFLOW DISCHARGE TO MANHOLE.            *
C     * QOT       = OUTFLOW DISCHARGE FROM MANHOLE.         *
C     * DVOLST(N)= INCREMENT OF VOLUME OF WATER FLOWING TO  *
C     *             STREET THROUGH THE N TH MANHOLE.        *
C     ********************************************************
      COMMON /AREA / T
      COMMON /AREA2A/ INFHD
      COMMON /AREA2B/ CINLET
      COMMON /AREA6E/ NPIPE
      COMMON /AREA6C/ KES
      COMMON /AREA7B/ ALNTH
      COMMON /AREA7C/ KM
      COMMON /AREA8C/ ITS
      COMMON /AREA10/ USEL
      COMMON /AREA14/ QBSMT
      COMMON /AREA18/ SE,EMP,F
      COMMON /AREA20/ ESUMF
      COMMON /AREA22/ H
      COMMON /AREA35/ QAV
      COMMON /AREA36/ QSTR
      DIMENSION  USEL(3),H(4),EMP(3,220)
      DIMENSION  ESUMF(220),ALNTH(3),QAV(3,220)
      DIMENSION  SE(3,220),KES(3),INFHD(4,220)
      DIMENSION  CINLET(220),QBSMT(3,220),QSTR(3)
      DIMENSION  DVOLST(3),F(3,220)
      REAL  KES,INFHD
      USEL(KM) = H(KM)
      IF(KM.EQ. NPIPE)  GO  TO  100
      KM2 = KM + 1
      SEWHL = USEL(KM) - USEL(KM2)
      EMP(KM,ITS) = (USEL(KM) + USEL(KM2))/2 .
      GO  TO  200
  100 SEWHL = USEL(KM) - ESUMP(ITS)
      EMP(KM,ITS) = (ESUMP(ITS) + USEL(KM))/2.0
  200 CONTINUE
      SE(KM,ITS) = SEWHL / ALNTH(KM)
      QAV(KM,ITS) = SQRT(ABS(SE(KM,ITS)/KES(KM))) *SE(KM,ITS)/ABS
     @(SE(KM,ITS))
      KM1 = KM - 1
      SUMQST = 0.0
      SUMIHD = CINLET(ITS)
      DO  300  IM1 = 1,KM
  300 SUMIHD = SUMIHD + INFHD(IM1,ITS)
      IF(KM1.EQ. 0)  GO  TO  500
      SUMQB = C.0
      DC  400  IM2 = 1,KM1
      SUMQST = SUMQST + QSTR(IM2)
  400 SUMQB = SUMQB + QBSMT(IM2,ITS)
      GC  TO  600
  500 SUMQB = C.0
  600 CONTINUE
      CIN = SUMIHD - SUMQB - SUMQST
      QOT = QAV(KM,ITS) + QBSMT(KM,ITS)/2.0
      QSTR(KM) = CIN - QOT
      RETURN
      END
```

```
      SUBROUTINE   PRESML
C     ***********************************************************
C     *  THE FUNCTION OF THIS SUBROUTINE IS TO SIMULATE         *
C     *  THE SURCHARGED FLOW IN THE SEWER SYSTEM. TAKING INTO
C     *  CONSIDERATION FLOW TO AND FROM BASEMENTS AS WELL AS*
C     *  STREETS (GROUND SURFACE).                              *
C     ***********************************************************
C     *  OBSMT(I,T)= AVERAGE RATE OF BASEMENT FLOW FOR THE  *
C     *              I-TH SEWER DURING THE T-TH TIME STEP,CFS
C     *  SE(I,T) = SLOPE OF ENERGY LINE OF THE I-TH SEWER AT*
C     *  T-TH TIME INDEX.                                       *
C     *  KES( ) = COEFFICIENT TO COMPUTE ENERGY SLOPE,IN THE*
C     *M          FORMULA,  S = K* U**2                        *
C     *  QAV( . ) = AVERAGE DISCHARGE FLOWING THROUGH   THE  *
C     *             PIPE                                        *
C     *  EMP( , ) = ELEVATION OF THE MID-POINT OF THE ENERGY*
C     *             LINE                                        *
C     ***********************************************************
      COMMON /AREA/ T
      COMMON /AREA1A/    SO
      COMMON /AREA2A/    INFHD
      COMMON /AREA2E/    QINLET
      COMMON /AREA4E/    QINF,QOUT
      COMMON /AREA7E/    ALNTH
      COMMON /AREA7C/    KM
      COMMON /AREA6E/    NPIPE
      COMMON /AREA6C/    KES
      COMMON /AREA8/ IT
      COMMON /AREA6C/    ITS
      COMMON/AREA10/    USEL
      COMMON/  AREA14/   OBSMT
      COMMON /AREA18/ SE,EMP,F
      COMMON /AREA19/ BMP
      COMMON /AREA20/ ESUMP
      COMMON /AREA22/ F
      COMMON /AREA24/ OHCUS
      COMMON /AREA35/ QAV
      COMMON /AREA36/ QSTR
      COMMON /AREA38/   CB(4)
      COMMON /AREA40/ SWL,QSUMP
      COMMON /AREA 41/ DFLCOD
      COMMON /AREA43/   DMAX
      COMMON /AREA46/   FMAX
      COMMON /AREA48/ VMAX,TPEAK
      DIMENSION   TPEAK(3)
      DIMENSION   F(4)
      DIMENSION   QSTR(3),VOLST(3),DVOLST(3)
      DIMENSION   SO(3)
      DIMENSION   INFHD(4,220),QINLET(220)
      DIMENSION   KES(3)
      DIMENSION   QAV(3,220),OBSMT(3,220)
      DIMENSION   USEL(3), EMP(3,220)
      DIMENSION   BMP(3)
      DIMENSION   ESUMP(220)
      DIMENSION   SE(3,220),ALNTH(3)
      DIMENSION   XOFLOD(3),DMAX(3),FMAX(3),VMAX(3)
      DIMENSION   F(3,220)
      DIMENSION   QINF(3,220), QOUT(3,220)
      REAL  INFHD ,KES
      XO = 0.0
      DO  30   I = 1,NPIPE
      F(I,IT-1) = 0.0
      SE(I,IT-1) = SO(I)
      EMP(I,IT-1) = BMP(I)
      QSTR(I) = 0.0
      VOLST(I) = 0.0
      DVOLST(I) =0.0
      DMAX(I) = 0.0
      FMAX(I) = 0.0
      VMAX(I) = 0.0
      TPEAK(I) = 0.0
   30 CONTINUE
```

```
      DO  600  ITS = IT.220
      ITS1 = ITS - 1
      DC  50  IMO = 1.NPIPE
      QBSMT(IMO.ITS) = QBSMT(IMO.ITS1)
   50 CONTINUE
      ESUMP(ITS) = ESUMP(ITS-1)
      DC  450  ITR1 = 1.3
      DO  410  ITR = 1.6

      DO  400  IM= 1.NPIPE
      KM = NPIPE- IM + 1
      CC = QSTR(KM)
      SUMIHD = QINLET(ITS)
      SUMQST = 0.0
      DC  150  IM1 = 1.KM
      SUMQST = SUMQST + QSTR(IM1)
  150 SUMIHD = SUMIHD + INFHD(IM1.ITS)
      KM1 = KM - 1
      IF ( KM1 .EQ. 0 ) GC  TO  250
      SUMQB = C.0
      DC  200  IM2 =1. KM1
  200 SUMQB = SUMQB + QBSMT(IM2.ITS)
      GC  TO  300
  250 SUMQB = 0.0
  300 CONTINUE
      QAV(KM.ITS)=SUMIHD-SUMQB-QBSMT(KM.ITS)/2.0-SUMQST
      SE(KM.ITS) = KES(KM)*QAV(KM.ITS)*AES(QAV(KM.ITS))
      IF(IM. GT . 1 ) GO TC 350
      USEL(KM) = ESUMP(ITS) + SE(KM.ITS)*ALNTH(KM)
      EMP(KM.ITS) = (ESUMP(ITS) + USEL(KM))/2.0
      GO TO  375
  350 KM2 = KM + 1
      USEL(KM) = USEL(KM2) + SE(KM.ITS)*ALNTH(KM)
      EMP(KM.ITS) = (USEL(KM2) + USEL(KM))/2.0
  375 CONTINUE
      IF(USEL(KM).LE.H(KM))  GO  TO  380
      CALL  STRMOD
      GO  TO  395
  380 IF(VCLST(KM).GT.0.01) GO  TO  385
      QSTR(KM) = 0.0
      DVCLST(KM) = C.0
      GO  TO  395
  385 QSTR(KM) = (-1.0)*CE(KM) * SQRT(ABS(VCLST(KM)))
  395 CONTINUE
      CALL  FRATIC
      CALL  QHCUSS
      XQFLOD(KM) = QFLOOD * 12.
      X = QBSMT(KM.ITS)
      QBSMT(KM.ITS) = QHCUS*(1.0-(0.9**ITR) **ITR)
      QBSMT(KM.ITS) =(QBSMT(KM.ITS)+ X )/2.0
      QSTR(KM) =QSTR(KM)*(1.0-(0.9**ITR)**ITR)
      QSTR(KM) = (QSTR(KM) + QQ)/2.0
      DVCLST(KM) = QSTR(KM) * T * 60.
  400 CONTINUE
  410 CONTINUE
      QOUT(NPIPE.ITS) = QAV(NPIPE.ITS) - QBSMT(NPIPE.ITS)/2.0
      Y = ESUMP(ITS)
      CALL  SUMP1
      ESUMP(ITS) = (ESUMP(ITS) + Y ) / 2.0
  450 CONTINUF
      DO  460  KI =1.NPIPE
      IF( DVCLST(KI) .LE.  0.0 ) GO  TO  451
      TPEAK(KI) = TPEAK(KI) + T
  451 CONTINUE
      VCLST(KI) = VCLST(KI) + DVCLST(KI)
  460 CONTINUE
      DC  490  I = 1.NPIPE
      IF(DMAX(I).GE.XQFLOD(I)) GO  TO  470
      DMAX(I) = XQFLOD(I)
  470 CONTINUE
      IF(FMAX(I).GE.F(I.ITS)) GO  TO  480
      FMAX(I)=  F(I.ITS)
  480 CONTINUE
```

```
      IF (VMAX(I) .GE. VOLST(I)) GO TO 490
      VMAX(I) = VOLST(I)
490   CONTINUE
C
C     IF RECESION LIMB IS ENCOUNTERED,RETURN TO SUBROUTINE
C     HYCMOD
      NFLAG = 0
      DO 500 I = 1,NPIPE
      IF(QBSMT(I,ITS).LT.XQ)NFLAG=NFLAG+1
500   CONTINUE
      IF (NFLAG .EQ. NPIPE) GO TO 550
      IF(ITS.EQ.(IT+4)) XQ=0.01
600   CONTINUE
550   CONTINUE
      CALL ANDAM
      RETURN
      END




      SUBROUTINE QHCUSS
C     **************************************************************
C     * THIS SUBROUTINE COMPUTES THE AVERAGE RATE OF BASEMENT
C     * FLOW, FOR I-TH SEWER DURING THE T-TH TIME STEP,CFS *
C     **************************************************************
C     * DFLOOD = FLOODING DEPTH OF THE BASEMENTS ASSOCIATED*
C     *          WITH THE I-TH SEWER,AT THE END OF THE T-TH*
C     *          TIME STEP,FT
C     * APIPE = CROSS SECTIONAL AREA OF THE PIPE OF THEC
C     *         HOUSE CONNECTION,SQ,FT                    *
C     * DHCON(I) = DIAMETER OF THE PIPE OF THE HOUSE      *
C     *            CONNECTION,FT                          *
C     * ALPIPE(I) = AVERAGE LENGTH OF HOUSE CONNECTIONS   *
C     *             WITHIN A PARTICULAR SEWER SECTION,FT   *
C     **************************************************************
      COMMON /AREA7C/ I
      COMMON /AREA8/ IT
      COMMON /AREA8C/ ITS
      COMMON /AREA18/ SE,EMP,F
      COMMON /AREA/ T
      COMMON /AREA7E/ ALNTH
      COMMON /AREA1C/ USEL
      COMMON /AREA11/ SBASE
      COMMON /AREA13/ ABASE
      COMMON /AREA14/ QBSMT
      COMMON /AREA19/ BMP
      COMMON /AREA24/ QHCUS
      COMMON /AREA25/ DHCON,ALPIPE
      COMMON /AREA26/ NHOUS
      COMMON /AREA 41/ DFLOOD
      DIMENSION SBASE(3),ALNTH(3)
      DIMENSION ABASE(3)
      DIMENSION QBSMT(3,220)
      DIMENSION USEL(3)
      DIMENSION EMP(3,220),SE(3,220),F(3,220)
      DIMENSION ALPIPE(3), DHCON(3)
      DIMENSION BMP(3)
      DIMENSION NHOUS(3)
      APIPE = DHCON(I)**2.0*0.7854
      A = 8.02*APIPE*NHOUS(I)/SQRT(2.0+0.03*ALPIPE(I)/DHCON(I))
      SQBSMT = 0.0
      DO 50 M = IT,ITS
50    SQBSMT = SQBSMT + QBSMT(I,M)
      FLVOL = SQBSMT* T * 60.
      DFLOOD = FLVOL /ABASE(I)/ 43560.
      IF(DFLOOD)60,70,70
60    DFLOOD = 0.0
70    CONTINUE
      ENBAS = BMP(I) + DFLOOD
      USEBL = ENBAS + SBASE(I)*ALNTH(I)/2.0
      DSEBL = ENBAS - SBASE(I)* ALNTH(I)/2.0
      USEL1 = USEL(I)
```

```
      USHL = USEL1 - USEBL
      DSEL = USEL1 - SE(I,ITS)*ALNTH(I)
      DSHL = DSEL - DSEBL
      IF(F(I,ITS))100,100,200
  100 IF(DFLOOD .GT. 0.0)  GO  TO  110
      QHOUS = 0.0
      GO  TO  400
  110 B=((ABS(USHL)**1.5)-(ABS(DSHL)**1.5))/(ABS(USHL)-ABS(DSHL))
      QHCUS=(-1.0)*0.667* A * B
      IF(FLVOL-ABS(QHOUS)*T*60.)  120,130,130
  120 QHOUS = FLVOL / T / 60. *(-1.0)
  130 GO  TO  400
  200 IF(F(I,ITS).LT.1.0)  GO  TO  300
      IF( USHL  .LT.  0.0)  GO  TO  160
      IF(DSHL  .LT.  0.0)  GO  TO  150
      GO  TO  170
  150 FT = USHL/(USHL + ABS(DSHL))
      GO  TO  180
  160 IF(DSHL  .LT.  0.0)  GO  TO  170
      FT = DSHL/(DSHL + ABS(USHL))
      GO  TO  180
  170 B=((ABS(USHL)**1.5)-(ABS(DSHL)**1.5))/(ABS(USHL)-ABS(DSHL))
      QHOUS = 0.667 * A * B * USHL/ ABS(USHL)
      GO  TO  400
  180 F(I,ITS) = FT
  300 IF(USHL)210,220,220
  210 IF(DSHL  .LT.  0.0) GO  TO  110
      QHOUSP = 0.667*F(I,ITS)*SQRT(DSHL)*A
      QHOUSN = 0.667*(1.0-F(I,ITS))*SQRT(ABS(USHL))*A
      GO  TO  230
  220 QHOUSP = 0.667*F(I,ITS)*SQRT(ABS(USHL))*A
      QHOUSN = 0.667*(1.0-F(I,ITS))*SQRT(ABS(DSHL))*A
  230 CONTINUE
      VOLAV = DFLOOD*(1.0-F(I,ITS))*ARASE(I)*43560.
      IF(VOLAV-QHOUSN*T*60.) 270,280,280
  270 QHOUSN = VOLAV/T/60.
  280 CONTINUE
      QHOUS = QHOUSP -QHOUSN
  400 CONTINUE
      RETURN
      END



      SUBROUTINE  KENERG
C     ********************************************************
C     * THIS SUBROUTINE COMPUTES THE COEFFICIENT OF THE     *
C     * ENERGY SLOPE EQUATION,BASED ON MANNING'S EQUATION   *
C     ********************************************************
      COMMON /AREA5A/  AN
      COMMON /AREA5E/  DIAM
      COMMON /AREA6E/  NPIPE
      COMMON /AREA6C/  KES
      DIMENSION KES(3),DIAM(3),AN(3)
      REAL KES
      DO 100 I = 1,NPIPE
      KES(I) = 4.6369 * AN(I) **2/(DIAM(I)**(16./3.))
  100 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE   FRATIO
C     ***********************************************************
C     *  IT COMPUTES THE RATIO BETWEEN THE LENGTH OF THE SEWER
C     *  ACTIVELY BEING FLOODED AND ITS TOTAL LENGTH,I.E.,    *
C     *  BASEMENT FLOODING RATIO F( . )                        *
C     ***********************************************************
C     *  BMP( ) = MID-POINT ELEVATION OF BASEMENTS GRADE LINE
C     *  SBASE( . ) = SLOPE OF THE BASEMENTS LINE ASSOCIATED  *
C     *               WITH PIPE (I).                          *
C     *  F( I,T)=FRACTION OF THE I-TH SEWER CONTRIBUTING TO   *
C     *          BASEMENTS STORAGE,DURING THE T-TH TIME STEP  *
C     *  USEL( . ) = UPSTREAM ELEV. OF ENERGY LINE            *
C     *  DSEL = DOWNSTREAM ELEV. OF ENERGY LINE               *
C     *  USBL = UPSTREAM ELEV. OF BASEMENTS GRADE LINE.       *
C     ***********************************************************
      COMMON /AREA7E/  ALNTH
      COMMON /AREA7C/   IP
      COMMON /AREA8C/    IT
      COMMON /AREA18/ SE,EMP,F
      COMMON / AREA11/  SBASE
      COMMON /AREA19/  BMP
      COMMON / AREA 10 / USEL
      DIMENSION   SE(3,220),USEL(3)
      DIMENSION   EMP(3,220),BMP(3)
      DIMENSION   SBASE(3), ALNTH(3)
      DIMENSION   F(3,220)
      DSEL = USEL(IP) - SE(IP,IT) *ALNTH(IP)
      DSBL = EMP(IP) - SBASE(IP) *ALNTH(IP)/2.0
      USBL = EMP(IP) + SBASE(IP) * ALNTH(IP) / 2.0
      IF( USEL(IP) .GT. USBL) GO TO 50
      IF(DSEL .GT. DSBL ) GO TO 50
      F(IP,IT) = 0.0
      GO TO 300
   50 CONTINUE
      FLOAT = SE(IP,IT)- SBASE(IP)
      IF(FLOAT .GT. C.000001) GO TO 60
      F(IP,IT) = 1.0
      GO TO 300
   60 CONTINUE
      C = (EMP(IP,IT) - EMP(IP))/ (SE(IP,IT) -SBASE(IP))/ALNTH(IP)
      IF(SE(IP,IT).GE.SBASE(IP)) GO TO 100
      F(IP,IT) = 0.5 -C
      GO TO 200
  100 F(IP,IT) = 0.5 + C
  200 CONTINUE
      IF( F(IP,IT) .GT. 1.0 ) F(IP,IT) = 1.0
  300 CONTINUE
      RETURN
      END



      SUBROUTINE  SWITCH
C     ***********************************************************
C     *  THIS SUBROUTINE DETERMINES WHETHER THE FLOW IS        *
C     *  GRAVITY FLOW(OPEN CHANNEL FLOW),OR SURCHARGED FLOW   *
C     *  (PRESSURIZED FLOW ).                                  *
C     ***********************************************************
C     *  STGP = VOL. OF WATER INTO PIPE STORAGE.              *
C     *  STGS = VOL. OF WATER INTO SYSTEM STORAGE(PIPES ONLY) *
C     *  RELSTG = RELATIVE STORAGE,                           *
C     *         = VOL. OF WATER INTO STORAGE/ VOL. OF PIPES   *
C     *  M = FLAG,                                            *
C     *    = 0   FOR GRAVITY  FLOW                            *
C     *    = 1   FOR   PRESSURIZED FLOW.                      *
C     ***********************************************************
      COMMON/AREA6E/  NPIPE
      COMMON /AREA4C/  QINF,QOUT
      COMMON /AREA7C/   IP
      COMMON /AREA8/ IT
      COMMON /AREA9C/  M
      COMMON /AREA9A/ KMUS
      COMMON /AREA1E/ VOL
      COMMON /AREA51/ VPIPE(3)
```

```
      CCMMON /AREA32/ PMAX(3)
      DIMENSICN  CINF(3,220),QOUT(3,220)
      DIMENSICN  KNUS(3)
      DIMENSION  STGP(3),PRELS(3)
      REAL  KNUS
      STGS = 0.0
      DO  50   IPI = 1,NPIPE
      STGP(IPI)=KNUS(IPI)*(CINF(IPI,IT)+QOUT(IPI,IT))*0.5
      PRELS(IPI) = STGP(IPI) / VPIPE(IPI)
C     PMAX IS A PARAMETER TO BE USED IN THE BALANCING PROCEDURE
      IF(PRELS(IPI).GT.PMAX(IPI))PMAX(IPI) = PRELS(IPI)
      STGS = STGS + STGP(IPI)
   50 CCNTINUE
      RELSTG = STGS/VOL
      IF( RELSTG .LT.0.98 ) GO  TO  100
      M = 1
      GC  TO  200
  100 M = 0
  200 CCNTINUE
      RETURN
      END
```

```
      SUBROUTINE  VOLSYS
C     *****************************************************************
C     * THIS SUBROUTINE COMPUTES THE VOLUME OF THE PIPES    *
C     * IN THE SYSTEM, REPRESENTING THE MAXIMUM PIPE STORAGE.
C     *****************************************************************
C     * VOL  = VOLUME OFPIPES IN THE SYSTEM, CUBIC FEET     *
C     *****************************************************************
      CCMMON /AREA56 / DIAM
      CCMMON /AREA6E / NPIPE
      CCMMON /AREA7E / ALNTH
      CCMMON /AREA16 / VOL
      CCMMON /AREA51/ VPIPE(3)
      DIMENSICN  ALNTH(3), DIAM(3)
      VOL = 0.0
      DO  100   I = 1, NPIPE
      VPIPE(I) = 0.785*(DIAM(I)**2)*ALNTH(I)
      VOL = VOL + VPIPE(I)
  100 CCNTINUE
      RETURN
      END
```

```
      SUBROUTINE  ELVEAS
C     *****************************************************************
C     * THIS SUBROUTINE COMPUTES THE ELEVATION OF THE MID- *
C     * POINT OF THE EASEMENTS GRADE LINE FOR EACH SEWER. IT
C     * ALSO COMPUTES SLOPE OF THE EASEMENTS GRADE LINE FOR*
C     * EACH SEWER SECTION.
C     *****************************************************************
C     * SEASE(I) = SLOPE OF THE EASEMENTS GRADE LINE OF THE*
C     *            I-TH SEWER;                              *
C     * DBST(I) = AVERAGE DEPTH OF BASEMENTS BELOW THE GROUND
C     *            SURFACE,ASSOCIATED WITH THE I-TH SEWER   *
C     *****************************************************************
      CCMMON /AREA11/ SBASE
      CCMMON /AREA19/ BMP
      CCMMON /AREA23/ DBSMT
      CCMMON /AREA7B/ ALNTH
      CCMMON /AREA22 /  H
      CCMMON /AREA6E/  NPIPE
      DIMENSICN  H(4),SBASE(3), BMP(3)
      DIMENSICN  ALNTH(3), DBSMT(3)
      DO  100   I = 1,NPIPE
      SBASE(I) = (H(I)    - H(I+1))/ ALNTH(I)
      BMP(I) = (H(I) + H(I+1))/2.0   - DBSMT(I)
  100 CCNTINUE
      RETURN
      END
```

```
      SUBROUTINE CPIPES
C     ***********************************************************
C     * THIS SUBROUTINE COMPUTES THE TOTAL INSTALLATION         *
C     *(SUPPLY AND EXCAVATION) COST OF ALL THE SEWERS IN THE    *
C     * SYSTEM.                                                 *
C     ***********************************************************
C     * USPL(I) = CROWN ELEV. OF THE U/S END OF THE I TH PIPE
C     * DSPL(I) = CROWN ELEV. OF THE D/S END OF THE I TH PIPE
C     * H1      = U/S DEPTH OF EXCAVATION.                      *
C     * H2      = D/S DEPTH OF EXCAVATION.                      *
C     * HAV     = AVERAGE DEPTH OF EXCAVATION.                  *
C     * COSTP(I)= COST PER UNIT LENGTH OF THE I TH PIPE         *
C     * A, B, C = COEFFICIENTS IN THE PIPE-COST EQUATION        *
C     *           COST = A + B* D**2 + C * H**2                 *
C     * COSTPS  = TOTAL COST OF PIPES IN SYSTEM.                *
C     ***********************************************************
      COMMON /AREA1A/  SO
      COMMON /AREA5C/  DIAM
      COMMON /AREA6E/  NPIPE
      COMMON /AREA7C/  ALNTH
      COMMON /AREA22/  H
      COMMON /AREA55/  USELEV
      COMMON /AREA56/  COSTPS
      DIMENSION H(4),ALNTH(3),DIAM(3),SO(3)
      DIMENSION USPL(3), DSPL(3),COSTP(3)
C     THE FOLLOWING DEFAULT VALUES ARE BASED ON
C     1973 PRICES ( BASE  YEAR)
      A = 2.8
      B = 2.0
      C = 0.045
      USPL(1) = USELEV - DIAM(1)
      DO 100   K = 1, NPIPE
      IF( K. NE. 1 ) USPL(K)=DSPL(K-1)+DIAM(K-1)-DIAM(K)
      USPL(1) = USELEV - DIAM(1)
      H1 = H(K) - USPL(K)
      DSPL(K) = USPL(K)-SO(K)*ALNTH(K)
      H2 = H(K+1) - DSPL(K)
      HAV = (H1 + H2)/ 2.0
      COSTP(K) = A+B*DIAM(K)**2+C*HAV*HAV
  100 CONTINUE
      COSTPS = 0.0
      DO  200   K = 1,NPIPE
      COSTPS = COSTPS  + COSTP(K)* ALNTH(K)
  200 CONTINUE
      RETURN
      END



      SUBROUTINE STDAMG
C     ***********************************************************
C     * THIS SUBROUTINE ESTIMATES GROUND SURFACE DAMAGES,       *
C     * INCLUDING INCONVENIENCE AND PROPERTY DAMAGES.           *
C     ***********************************************************
C     * VEH(I) = EFFECTIVE NUMBER OF VEHICLE CROSSINGS AT       *
C     *           THE I-TH MANHOLE(INTERSECTION) PER HOUR,ON*
C     *           THE AVERAGE BASIS.                            *
C     * ICLASS(I) = DISTRICT INDEX                              *
C     *           = 0    (RESIDENTIAL )                         *
C     *           = 1    (COMMERCIAL )                          *
C     * CT      = FACTOR TAKES INTO ACCOUNT TYPE OF DISTRICT*
C     *           DEFAULT VALUES ARE  2.0 ( COMMERCIAL ) &  *
C     *           1.0  ( RESIDENTIAL )                         *
C     * CL      = CALIBRATION FACTOR , FRACTION OF THE MAX. *
C     *           DAMAGE. (DEFAULT VALUE = 1.0 ). 1 FT DEPTH*
C     *           OF WATER OVER  FLOODED AREA CORRESPONDS TO*
C     *           100  %  DAMAGE.                             *
C     ***********************************************************
      COMMON /AREA7C/ ALNTH
      COMMON /AREA26/  NHOLS
      COMMON /AREA46/ VMAX,TPEAK
```

```
      COMMON /AREA49/ ICLASS(3), VEH(3)
      COMMON /AREA47/ ASS(3), AF
      COMMON /AREA44/ K
      COMMON /AREA50/ STDAM
      DIMENSION  ALNTH(3),NHOUS(3)
      DIMENSION  VMAX(3)
      DIMENSION  TPEAK(3)
      CL = 0.5
      STDAMI = ( 1./ 360.) * VEH(K) * TPEAK(K)**2.0
      IF(ICLASS(K).EQ. 0 )  GO TO  100
      CT = 2.0
      GO TO  200
  100 CT = 1.0
  200 CONTINUE
      IF( K .NE. 1). GO TO  300
      ASURF = ALNTH(1) * 60.0
C     WIDTH OF STREET = 60.0  FT  ( DEFAULT VALUE)
      STDAMP = CL * ASS(1) * AF * NHOUS(1) * CT * VMAX(1) / ASURF
      GO TO  600
  300 CONTINUE
      IF( ICLASS(K-1) .EQ. 0 )  GO TO  400
      CT1 = 2.0
      GO TO  500
  400 CT1 = 1.0
  500 CONTINUE
      ASURF = 60.0 * ( ALNTH(K-1) + ALNTH(K)) / 2.0
      STDAMP = CL * ( ASS(K) * AF * NHOUS(K) * CT + ASS(K-1)
     @         * AF * NHOUS(K-1) * CT1)/2.0 * (VMAX(K) /ASURF  )
  600 CONTINUE
      STDAM = STDAMI + STDAMP
      RETURN
      END




      SUBROUTINE  BDAMAG
C     ***********************************************************
C     * THIS SUBROUTINE COMPUTES DAMAGE TO BASEMENTS ASSO-      *
C     * CIATED WITH EACH PIPE.                                  *
C     ***********************************************************
C     * AAS(I) = AVERAGE ASSESMENT VALUE OF HOUSES OF I TH      *
C     *          MNH.(DEFAULT VALUE = 8000)                     *
C     * AF     = ASSESMENT FACTOR.(DEFAULT VALUE = 4.0)         *
C     * FU     = FURNITURE VALUE(RATIO TO PROPERTY VALUE),      *
C     *          (DEFAULT VALUE = 0.25)                         *
C     * RE(I)  = PORTION OF PROPERTY VALUE ASSIGNED TO BASE-    *
C     *          -MENTS OF I-TH SEWER,DEFAULT VALUE=0.15        *
C     * HEALTH = HEALTH HAZARD FACTOR.(DEFAULT VALUE = 0.5)     *
C     * CLEAN  = BASEMENT CLEANING(AFTER STORM) FACTOR.         *
C     *          (DEFAULT VALUE = 0.05)                         *
C     * G      = DAMAGE RATIO TO TOTAL BASEMENT VALUE,          *
C     *          DEPENDENT ON DEPTH OF FLOODING.                *
C     *        = 1.00 FOR  D .GE. 3.00 INCHES                   *
C     ***********************************************************
      COMMON /AREA6E/ NPIPE
      COMMON /AREA43/ DMAX
      COMMON /AREA26/ NHOUS
      COMMON /AREA44/ K
      COMMON /AREA45/ BSDAM
      COMMON /AREA46/ FMAX
      COMMON /AREA47/ ASS(3), AF
      DIMENSION  RE(3),FMAX(3),NHOUS(3),DMAX(3)
      DO  100  I = 1, NPIPE
      ASS(I) = 8000
      RE(I) = 0.15
  100 CONTINUE
      AF = 4.0
      FU = 0.25
      HEALTH = 0.5
      CLEAN = 0.05
      IF( DMAX(K)) 200,200,300
```

```
  200 BSCAM = 0.0
      GC TO  600
  300 IF( DMAX(K). GE.  3.00) GO  TO  400
      G = 0.5  + 0.166* DMAX(K)
      GC TO  500
  400 G = 1.00
  500 CCNTINUE
      CAMFCT =(1.0 + HEALTH + CLEAN *(1.C + HEALTH))* G
      BSMTV = ASS(K)* AF*(1. + FU) *RB(K)* FMAX(K)*NHOJS(K)
      BSCAM = BSMTV * CAMFCT
  600 CCNTINUE
      RETUFN
      END



      SUBROUTINE  ANCCST
C     *********************************************************
C     * THIS SUBROUTINE CCMPUTES CAPITAL RECOVERY COSTS OF  *
C     * THE SEWER SYSTEM, INCLUDING SEWERS,MANHOLES,INLETS  *
C     *(AND CATCH BASINS,IF ANY),PUMP,SUMP AND STRUCTURE OF*
C     * THE PUMPING STATION                                 *
C     *********************************************************
C     * AI      = INTEREST RATE PER ANNUM(EXPRESSED AS A DEC-
C     *           IMAL FRACTICN).                           *
C     * NYRP    = YEARS OF ESTIMATED LIFE OF SEWERS         *
C     *         = 20  YEARS ( DEFAULT VALUE FOR CONCRETE PIPES
C     * RP      = CCST OF PIPES/ ( COST OF PIPES + MNH'S +INLETS)
C     *         = 0.65  (DEFAULT VALUE)
C     * CRFP,CRFPMP,CRFSMP  ARE CAPITAL RECOVERY FACTORS FOR
C     *           PIPES,PUMP AND  SUMP RESPECTIVELY.        *
C     * REI     = ENGINEERING INDEX FOR CONSTRUCTION YEAR/  *
C     *           ENGINEERING INDEX FOR BASE YEAR.          *
C     * CPIPE1  = CCST OF PIPES ,MANHOLES AND  INLETS       *
C     *           (BASE YEAR , DEFAULT VALUE  1973)          *
C     * CPIPE2  = CCST OF PIPES, MANHOLES AND INLETS        *
C     *           ( CONSTRUCTION  YEAR )                    *
C     * CPIPE3  = ANNUAL CCST OF PIPES,MNH'S AND INLETS     *
C     * C1,CSTBY= CCNSTANTS IN PUMP-COST EQUATION           *
C     *           CCST  = C1 * QR * HF  +  CSTBY            *
C     *         = 26.0  ,4500 RESPECTIVELY(DEFAULT VALUES   *
C     *           FOR MINIMUM PUMP SIZE FOR STAND BY;HR=25FT*
C     *           QR = 7 CFS AND DIESEL MCTOR.PRICE = $ 6000
C     *           (1976  PRICE)                             *
C     * NYRPMP  = YEARS OF ESTIMATED LIFE OF PUMP AND MOTOR *
C     *         = 20  YEARS ( DEFAULT VALUE )               *
C     * CPUMP1, CPUMP2, CPUMP3 = PUMP COSTS( BASE YEAR,     *
C     *           CCNSTRUCTION YEAR, ANNUAL COST)           *
C     * DSUMP   = DEPTH OF EXCAVATION FOR SUMP  (FT)        *
C     * CUILD   = CCST OF BUILDING                          *
C     * NYRSMP  = YEARS OF ESTIMATED LIFE OF SUMP AND BUILD-*
C     *           ING ;  50  YEARS (DEFAULT VALUE )         *
C     * CSUMP1,CSUMP2,CSUMP3 = SUMP COSTS( BASE YEAR,       *
C     *           CCNSTRUCTION YEAR, ANNUAL COST)           *
C     * COPER   = CPERATIONAL AND MAINTENANCE COST          *
C     * TANC    = TOTAL CAPITAL RECOVERY COST OF THE SEWER  *
C     *           SYSTEM                                    *
C     *********************************************************
      COMMON /AREA56/   COSTPS
      COMMON /AREA22/   F
      COMMON /AREA31/   RELETA,  HR, QR
      COMMON /AREA66/   NPIPE
      COMMON /AREA30 /  SBOTCM
      COMMON /AREA26/   ASUMP
      COMMON /AREA91/ TANC
      DIMENSION  F(4)
C     -------------------------------------------------
C         PART (I)  PIPES+MANHOLES+INLETS
C     -------------------------------------------------
      CALL  CPIPES
      REI = 1.33
```

```
      AI = 0.12
      NYRP = 20
      RP = 0.65
      CRFP=AI*(1.+AI)**NYRP/((1.+AI)**NYRP-1.)
      CPIPE1 = CCSTPS/ RP
      CPIPE2 = CPIPE1  *  REI
      CPIPE3 = CPIPE2  *  CRFP
C     -----------------------
C         PART (II)  P U M P
C     -----------------------
      REIPMP = 1.33
      C1 = 26.
      CSTBY = 4500.
      NYRPMP= 20
      CRFPMP=AI*(1.+AI)**NYRPMP/((1.+AI)**NYRPMP-1.)
      CPUMP1 = C1* QR * HR +  CSTBY
      CPUMP2 =  REIPMP  *  CPUMP1
      CPUMP3 =  CRFPMP * CPUMP2
C     ------------------------------
C,        PART  (III)  SUMP + BUILDING
C     ------------------------------
      DSUMP = H(NPIPE+1) - SBOTOM + 3.0
C SUMP BED WILL BE SET 3.0 FT BELOW SUMP LOWEST W.L.(DEFAULT VALUE)
      CBUILD = (2000. + ASUMP) *  30.
C A DEFAULT VALUE OF 2000  SQ.FT WILL BE ASSIGNED FOR MIN. CONTROL



C  BUILDING  AREA.:UNIT COST OF BUILDING = 30. $/ SQ.FT(DEFAULT VALUE)
      CLAND = 20000.
C COST OF LAND = $ 20.000   (DEFAULT VALUE.ASSUMING 1/2 ACRE)
      CEXC = 1.
C CEXC = COST OF EXCAVATION( $/CU.FT)= 1.0 $/CU.FT(DEFAULT VALUE)
      CEXC1 = CEXC * ( 1. + DSUMP/ 30. )
C CEXC1 = COST OF EXCAVATION TAKING INTO ACCOUNT DEPTH OF EXCAVATION
      REISMP= 1.33
      CSUMP1 = CBUILD + CLAND + CEXC1 * ASUMP * DSUMP
      CSUMP2 = REISMP * CSUMP1
      NYRSMP = 50
      CRFSMP =AI*(1.+AI)**NYRSMP/((1.+AI)**NYRSMP-1.0)
      CSUMP3 = CRFSMP  *  CSUMP2
C     -----------------------------------------------
C         PART (IV)  CAPITAL RECOVERY COST OF SEWER SYSTEM
C     -----------------------------------------------
      COPER = 15000.
C OPERATIONAL AND MAINTENANCE COSTS =$15.000 (DEFAULT VALUE)
      TANC = CPIPE3 + CPUMP3 + CSUMP3 + COPER
      RETURN
      END




      SUBROUTINE  ANDAM
C     *****************************************************
C     * EASEMENT AND GROUND SURFACE DAMAGES ARE ADDED FOR  *
C     * EACH SEWER SECTION,THEN,THEY ARE SUMMED UP FOR THE  *
C     * SEWERS.                                             *
C     *****************************************************
      COMMON /AREA6E/  NPIPE
      COMMON /AREA44/  K
      COMMON /AREA45/  BSDAM
      COMMON /AREA50/  STDAM
      COMMON /AREA81/  TDAM
      SUM = 0.0
      DO  100  K = 1, NPIPE
         CALL   BDAMAG
         CALL   STDAMG
      SUM = SUM + BSDAM + STDAM
100   CONTINUE
      TDAM = SUM
      RETURN
      END
```

```fortran
      SUBROUTINE  BALD
C     *****************************************************
C     * THIS SUBROUTINE BALANCES THE INITIAL DESIGN,BY    *
C     * CORRECTING SEWER DIAMETERS TO MAINTAIN SAME MAXIMUM*
C     * RELATIVE STORAGE IN ALL SECTIONS,FOR A PRESELECTED *
C     * STORM OF A KNOWN RETURN PERIOD.                    *
C     * NEW DIAM. = OLD DIAM. * (REL. STG.)**0.25          *
C     *****************************************************
      COMMON /AREA6E/  NPIPE
      COMMON /AREA58/ DIAM
      COMMON /AREA52/ PMAX(3)
      DIMENSION  DIAM(3)
      DO  100  I = 1,NPIPE
      DIAM(I) = DIAM(I) * PMAX(I) **0.25
  100 CONTINUE
      NPIPE1 = NPIPE -  1
      DO  200  I = 1,NPIPE1
      IF(DIAM(I+1) .LT. DIAM(I))  DIAM(I+1) = DIAM(I)
  200 CONTINUE
      CALL  VOLSYS
      RETURN
      END



      SUBROUTINE  HYDMB
C     *****************************************************
C     * THIS SUBROUTINE IS USED TO BALANCE THE PRELIMINARY *
C     * DESIGN. IT ATTEMPTS TO MAINTAIN THE SAME RELATIVE  *
C     * STORAGE FOR EACH SEWER SECTION, FOR A PRESELECTED STORM
C     * OF A KNOWN RETURN PERIOD.                          *
C     * ALSO. IT ADJUSTS THE PUMPING CAPACITY SO THAT IT   *
C     * JUST PREVENTS SURCHARGING OF THE OUTFALL DURING THE *
C     * SPECIFIED STORM.                                   *
C     *****************************************************
      COMMON /AREA/  T
      COMMON /AREA28/  ASUMF
      COMMON /AREA31/ RELETA,HK,QR
      COMMON /AREA32/ ENDSEW
      COMMON /AREA40/ SWL, QSUMP
      COMMON /AREA30/ SBOTCM
      COMMON /AREA4A/  IP
      COMMON /AREA6E/  NPIPE
      COMMON /AREA8/ IT
      COMMON /AREA9C/ M
      COMMON /AREA52/  PMAX(3)
      COMMON /AREA58/ DIAM
      COMMON /AREA8D/ ITS
      COMMON /AREA1A/  SO
      COMMON /AREA55/ USELEV
      COMMON /AREA7E/  ALNTH
      DIMENSION  ALNTH(3),SO(3)
      DIMENSION  DIAM(3)
      REAL  KMUS
      CCP = 0.0
      SWL = SBOTCM
      M = 0
      DO  50  I = 1,NPIPE
   50 PMAX(I)=0.0
      CALL  VOLSYS
C     3 ITERATIONS ARE USED TO BALANCE SEWER SIZES.
      DO  900  NNN= 1,3
      DO  700  ITT= 1, 180
      IT  =  ITT
      IF( M.NE. 0)  GO  TO  800
      DO  300  IPK = 1,NPIPE
      IP = IPK
      CALL  ADDHYD
      CALL  MUSKET
  300 CONTINUE
      CALL  SWITCH
      IF( NNN .NE. 3)  GO  TO  700
```

```
         ITS = IT
         SWL1 = SWL
         CALL    SUMP1
         DELSWL = SWL - SWL1
         DQP1 = DQP
         IF(SWL.GT.ENDSEW) DQP = DELSWL*ASUMP/T/60.
         IF(DQP.LT. DQP1) DQP=DQP1
  700 CONTINUE
  800 CONTINUE
         CALL    SALD
         DO    2000    I = 1.NPIPE
 2000 PMAX(I) = 0.0
         M = 0
  900 CONTINUE
         DQPMAX = DQP
         QR = QR + DQPMAX
C        THIS IS THE 2 -ND ITER. TO ADJUST THE PUMPING CAPACITY.
         M = 0
         DQP= 0.0
         SWL = SBCTOM
         DO    4000   ITT=  1,  180
         IT = ITT
         IF(M.NE.0)   GO   TO   5000
         DO 4010 IPK=1.NPIPE
         IP = IPK
         CALL    ADUHYD
         CALL    MUSKRT
 4010 CONTINUE
         CALL    SWITCH
         SWL1 = SWL
         CALL    SUMP1
         DELSWL=SWL-SWL1
         DQP1 = DQP




         IF(SWL.GT.ENDSEW)DQP=DELSWL*ASUMP/T/60.
         IF(DQP.LT.DQP1) DQP= DQP1
 4000 CONTINUE
 5000 CONTINUE
         DQPMAX = DQP
         QR = QR + DQPMAX
         ENDSEW=USELEV
         DO    6000   I=1.NPIPE
 6000 ENDSEW=ENDSEW-SO(I)*ALNTH (I)
         SBCTOM=ENDSEW-DIAM(NPIPE)-3.0
         RETURN
         END




         SUBROUTINE   DAMPRO
C     *********************************************************
C     * THIS SUBROUTINE COMPUTES THE COEFFICIENTS IN THE      *
C     * PROBABILITY-DAMAGE   EQUATION:                        *
C     *        DAMAGE = A + B*P + C*(P**2)                    *
C     * IT IS ALSO USED TO COMPUTE AVERAGE PROBABLE ANNUAL    *
C     * DAMAGE.                                               *
C     *********************************************************
C     * P(I) = PROBABILITY OF OCCURRENCE OF I TH STORM        *
C     * DAM(I) = TOTAL DAMAGE RESULTING FROM SPECIFIC STORM*
C     * NSTORM = NUMBER OF CONSIDERED STORMS                  *
C     * AD    = AVERAGE PROBABLE ANNUAL DAMAGE                *
C     *********************************************************
         COMMON /AREA71/ P
         COMMON /AREA72/ NSTORM
```

```
      COMMON /AREA73/ A,B,C
      COMMON /AREA74/ DAM
      COMMON /AREA75/ AD
      DIMENSION  P(7),DAM(7)
      P1 = P(1)
      P2 = P(2)
      P3 = P(3)
      D1 = DAM(1)
      D2 = DAM(2)
      D3 = DAM(3)
      C1=(D2-D1)*(P1-P3)+(P2-P1)*(D3-D1)
      C = C1/(P2-P3)/( P1-P2)
      B = ((D2-D1)-C*(P2*P2-P1*P1))/(P2-P1)
      A = D1- B*P1- C*P1*P1
C     AVERAGE PROBABLE ANNUAL DAMAGE = AREA UNDER PROB-DAMAGE CURVE
      AD = A*P1 + B*P1*P1/2.  +C*P1**3/3.
      RETURN
      END


      SUBROUTINE  DF1
C     *********************************************************
C     * THIS SUBROUTINE IS USED TO COMPUTE AVERAGE ANNUAL     *
C     * PROBABLE DAMAGE FOR PERTURBED SYSTEMS. A,B,CAND       *
C     * AVERAGE ANNUAL PROB. DAMAGE OF THE ORIGINAL SYSTEM    *
C     * ARE KNOWN.DAMAGE RESULTING FROM A PRESELECTED SPECIFIC *
C     * STORM IS USED TO DETERMINE THE NEW AVG. ANN. PROJ.DAM. *
C     *********************************************************
C     * KSTORM = INDEX OF THE STORM TO BE CONSIDERED IN       *
C     *           COMPUTING ANNUAL AVERAGE PROBABLE DAMAGE.   *
C     *********************************************************
      COMMON /AREA71/ P
      COMMON /AREA72/ NSTORM
      COMMON /AREA73/ A,B,C
      COMMON /AREA74/ DAM
      COMMON /AREA75/ AD
      COMMON /AREA82/ AD1
      DIMENSION  P(7),DAM(7)
      KSTORM = 2
      DX = DAM(KSTORM)
      PX = ( -B- SQRT(B*B- 4.* C * (A-DX)))/2./C
      AD1 = AD *P(KSTORM)/PX
      RETURN
      END


      SUBROUTINE  CHANGE
C     *********************************************************
C     * THIS SUBROUTINE PERTURBS THE ORIGINAL BALANCED DESIGN  *
C     * WITH RESPECT TO THE I-TH DECISION VARIABLE. IT MAKES  *
C     * THE NECESSARY ADJUSTMENTS ACCOMPANIED BY EACH PERT*   *
C     * URBATION OF THE SYSTEM, IN ORDER TO MAINTAIN THE      *
C     * THE COMPATIBILITY OF THE DESIGN.                      *
C     *********************************************************
      COMMON /AREA1A/ SO
      COMMON /AREA5E/ DIAM
      COMMON/AREA6E/ NPIPE
      COMMON /AREA7E/ALNTH
      COMMON /AREA22/ H
      COMMON /AREA28/ ASUMP
      COMMON /AREA30/ SBOTCM
      COMMON /AREA31/ RELETA,HR,QR
      COMMON /AREA32/ENDSEW
      COMMON /AREA55/USELEV
      COMMON /AREA80/ CHGPOS(5),CHGNEG(5)
      COMMON /AREA83/ NCHNG
      COMMON /AREA85/DDIAM(3),SSO(3),QRR,HRR,AASUMP,USLV. ,SSBOT
      DIMENSION  SO(3),DIAM(3),H(4),ALNTH(3)
```

```
C -    NCHNG = 1  ( CHANGE  IN  SLOPE )
       IF(NCHNG .NE. 1 ) GO TO 100
       DO 50  I = 1,NPIPE
       SO(I) = SO(I)*(1. + CHGPOS(NCHNG))
   50 CONTINUE
       ENDSEW = USELEV
       DO 60  I = 1,NPIPE
   60 ENDSEW = ENDSEW - SO(I)* ALNTH(I)
       SBOTCM = ENDSEW-DIAM(NPIPE)-3.0
       DHR = SSECT - SBOTCM
       HR = HR + DHR
  100 CONTINUE
C      NCHNG = 2 ( CHANGE  IN  PUMP CHARACTERISTICS )
       IF( NCHNG .NE. 2) GO TO 200
       QR = QR * (1. + CHGPOS(NCHNG))
  200 CONTINUE
C      NCHNG = 3  ( CHANGE IN SUMP  AREA )
       IF(NCHNG .NE. 3) GO TO 300
       ASUMP = ASUMP * ( 1. + CHGPOS(NCHNG))
  300 CONTINUE
C      NCHNG = 4  ( CHANGE IN SYSTEM OVERALL DEPTH  )
       IF( NCHNG .NE. 4) GO TO 400
       DEPTH = H(1) - USELEV
       DEPTH = DEPTH*(1. + CHGPOS(NCHNG))
       USELEV = H(1) - DEPTH
       ENDSEW = USELEV
       DO 350  I = 1,NPIPE
  350 ENDSEW = ENDSEW-SO(I)*ALNTH(I)
       SBOTCM = ENDSEW- DIAM(NPIPE) - 3.0
       DHR = SSEOT - SBOTCM
       HR = HR + DHR
  400 CONTINUE
C      NCHNG = 5  (CHANGE IN DIAMETERS )
       IF(NCHNG .NE. 5) GO TO 500
       DO 450  I = 1,NPIPE
  450 DIAM(I) = DIAM(I)*(1. + CHGPOS(NCHNG))
       ENDSEW = USELEV
       DO 470  I = 1,NPIPE
  470 ENDSEW= ENDSEW - SO(I)* ALNTH(I)
       SBOTCM = ENDSEW - DIAM(NPIPE) - 3.0
       DHR =SSECT      -SBOTCM
       HR= HR + DHR
  500 CONTINUE
       RETURN
       END


       SUBROUTINE  ORIGNL
C      ***************************************************
C      * THIS SUBROUTINE IS USED TO RESET THE PERTURBED SYSTEM
C      * TO ITS ORIGINAL BALANCED DIMENSIONS.               *
C      ***************************************************
       COMMON /AREA1A/ SO
       COMMON /AREA5E/  DIAM
       COMMON/AREA6E/ NPIPE
       COMMON /AREA30/ SBOTCM
       COMMON /AREA29/  ASUMP
       COMMON /AREA31/ RELETA,HR,QR
       COMMON /AREA32/ENDSEW
       COMMON /AREA55/ USELEV
       COMMON /AREA8/ NCHNG
       COMMON /AREA85/DDIAM(3),SSO(3),QRR,HRR,AASUMP,USLV ,SSBOT
       COMMON /AREA90/ ENDS
       DIMENSION  DIAM(3), SO(3)
       IF(NCHNG.NE.1) GO TO 100
       DO 50  I = 1,NPIPE
   50 SO(I) = SSO(I)
       SBOTCM = SSEOT
       HR =HRR
       ENDSEW = ENDS
```

```
100  CONTINUE
     IF(NCHNG.NE.2)GO  TO  200
     QP = QRP
200  CONTINUE
     IF(NCHNG .NE. 3) GO TC  300
     ASUMP = AASUMP
300  CONTINUE
     IF(NCHNG .NE. 4)  GO  TO  400
     USELEV = USLV
     SBOTOM = SSBOT
     HR = HRR
     ENDSEW = ENCS
     HR = HRR
400  CONTINUE
     IF(NCHNG .NE.  5)  GC  TO  500
     DO  450  I = 1,NPIPE
450  DIAM(I) = OCIAM(I)
     SBOTOM = SSBOT
     HR = HRR
500  CONTINUE
     RETURN
     END




     SUBROUTINE  STORE
C    *****************************************************
C    * THIS SUBROUTINE IS USED TO STORE THE ORIGINAL BALANCED
C    * DESIGN OF THE SEWER SYSTEM.                          *
C    *****************************************************
     COMMON/AREA6E/ NPIPE
     COMMON /AREA5E/  CIAM
     COMMON /AREA1A/ SO
     COMMON /AREA31/ RELETA,HR,QR
     COMMON /AREA28/  ASUMP
     COMMON /AREA30/ SBOTOM
     COMMON /AREA32/ENDSEW
     COMMON /AREA55/ USELEV
     COMMON /AREA5/CCIAM(3),SSO(3),QRR,HRR,AASUMP,USLV ,SSBOT
     COMMON /AREA90/ ENDS
     DIMENSION  CIAM(3),SO(3)
     DO  100  I = 1,NPIPE
     CCIAM(I) = CIAM(I)
     SSO(I) = SO(I)
100  CONTINUE
     QRR = QR
     AASUMP = ASUMP
     USLV= USELEV
     SSBOT = SBOTOM
     HRR =HR
     ENCS = ENDSEW
     RETURN
     END




     SUBROUTINE  GRAD1
C    *****************************************************
C    * THIS SUBROUTINE COMPUTES FIRST DEIVATIVES OF BOTH   *
C    * COST AND DAMAGE WITH RESPECT TO DESIGN VARIABLES.   *
C    *(FORWARD GRADIENTS).                                 *
C    *****************************************************
C    * CG1(K) = 1 ST DERIVATIVE OF THE SYSTEM ANNUAL COST  *
C    *          WITH RESPECT TO THE K TH VARIABLE          *
C    * DG1(K) =  1ST DERIVATIVE OF THE SYSTEM ANNUAL PROB- *
C    *          ABLE DAMAGE WITH RESPECT TO THE K TH VAR.  *
C    *****************************************************
     COMMON /AREA90/CHGPOS(5),CHGNEG(5)
     COMMON /AREA4/ C(5,3)
     COMMON /AREA92/ COSTP(5)
     COMMON /AREA93/ CG1(5),DG1(5)
```

```
      COMMON /AREA94/ CINITL
      DO  100  K = 1,5
      CG1(K) =(CCSTF(K)- CINITL)/CHGPOS(K)
      DG1(K) =(D(K,2)-D(K,1))/CHGPOS(K)
  100 CONTINUE
      DO  200  I = 1,5
      PRINT  1000,I,CG1(I),CG1(I)
  200 CONTINUE
 1000 FORMAT(10X,'DESIGN VARIABLE NO.=  ',I3,/,
     @  10X,'COST GRADIENT(1ST) = $ ',F12.1,'  /UNIT CHANGE',/,
     @  10X,'DAMAGE GRAD. (1ST) = $',F12.1,'  /UNIT CHANGE',//)
      RETURN
      END




      SUBROUTINE  GRAD1B
C     *************************************************************
C     * THIS SUBROUTINE COMPUTES FIRST DERIVATIVES OF THE       *
C     * ANNUAL COST OF THE SYSTEM WITH RESPECT TO  DESIGN       *
C     * VARIABLES.  BACKWARD DERIVATIVES ARE COMPUTED USING*
C     * NEGATIVE RELATIVE CHANGES( LESS COST&HIGHER DAMAGE)*
C     *************************************************************
C     * CG1B(I)= 1ST DERIVATIVE OF THE SYSTEM ANNUAL COST       *
C     *          WITH RESPECT TO THE I TH VAR.(BACKWARD)        *
C     *************************************************************
      COMMON /AREA80/CHGPOS(5),CHGNEG(5)
      COMMON /AREA83/ NCHNG
      COMMON /AREA91/ TANC
      COMMON /AREA94/ CINITL
      COMMON /AREA95/ CG1B(5)
      DIMENSION  CHGP(5),CCSTN(5)
      DO  100  K = 1,5
      CHGP(K) = CHGPOS(K)
  100 CONTINUE
      DO  200  NCHNG=1,5
      CHGPOS(NCHNG) = CHGNEG(NCHNG)*(-1.0)
      CALL  CHANGE
      CALL  ANCOST
      CCSTN(NCHNG) = TANC
      CALL  ORIGNL
      CHGPOS(NCHNG) = CHGP(NCHNG)
  200 CONTINUE
      DO  300  I = 1,5
      CG1B(I)=(CINITL- CCSTN(I))/ CHGNEG(I)
  300 CONTINUE
      DO  400  N=1,5
      PRINT  1000,N,CG1B(N)
  400 CONTINUE
 1000 FORMAT(10X,'DESIGN VAR. NO. = ',I3,/,
     @    10X,'BACKWARD COST GRAD.(1ST) = $ ',F12.1,'  /UNIT CHANGE
     @ ',//)
      RETURN
      END

$ENTRY
```

APPENDIX IV

FIGURES
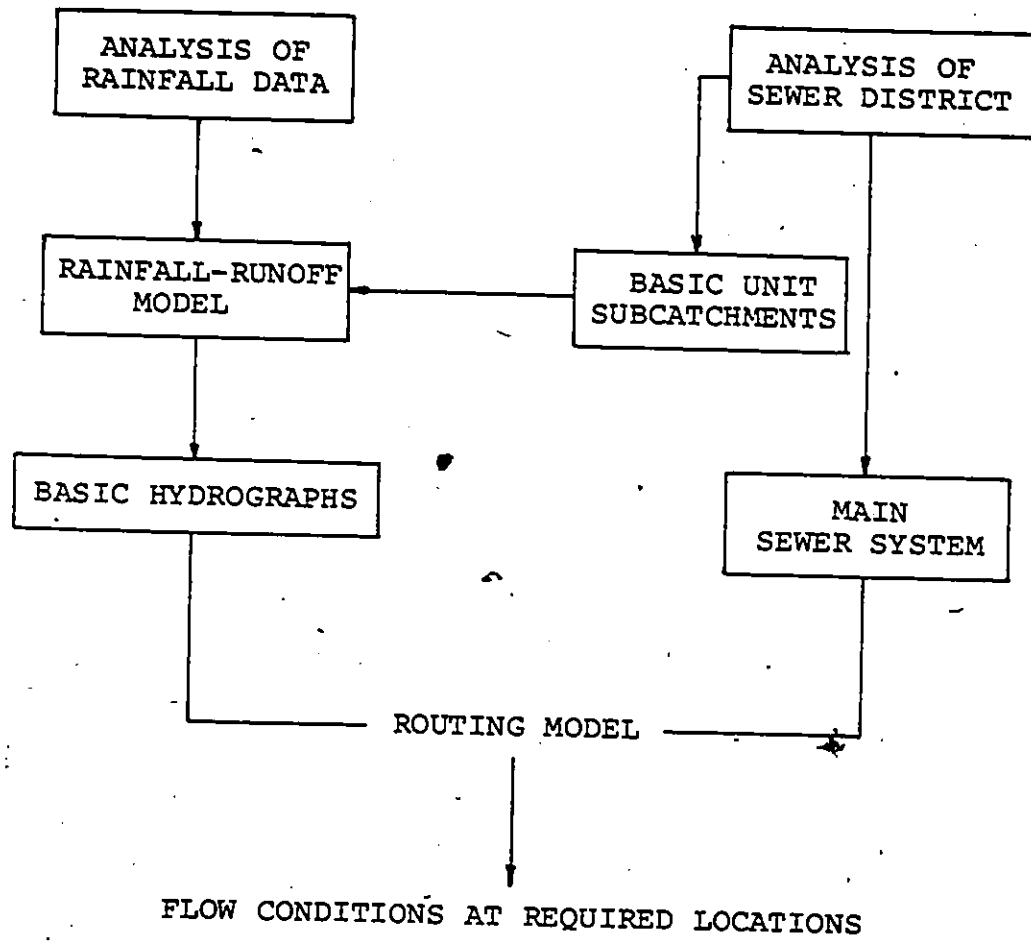
FIGURE 2.1   ELEMENTS OF THE SIMULATION OF URBAN RUNOFF
                (After Reference 20)
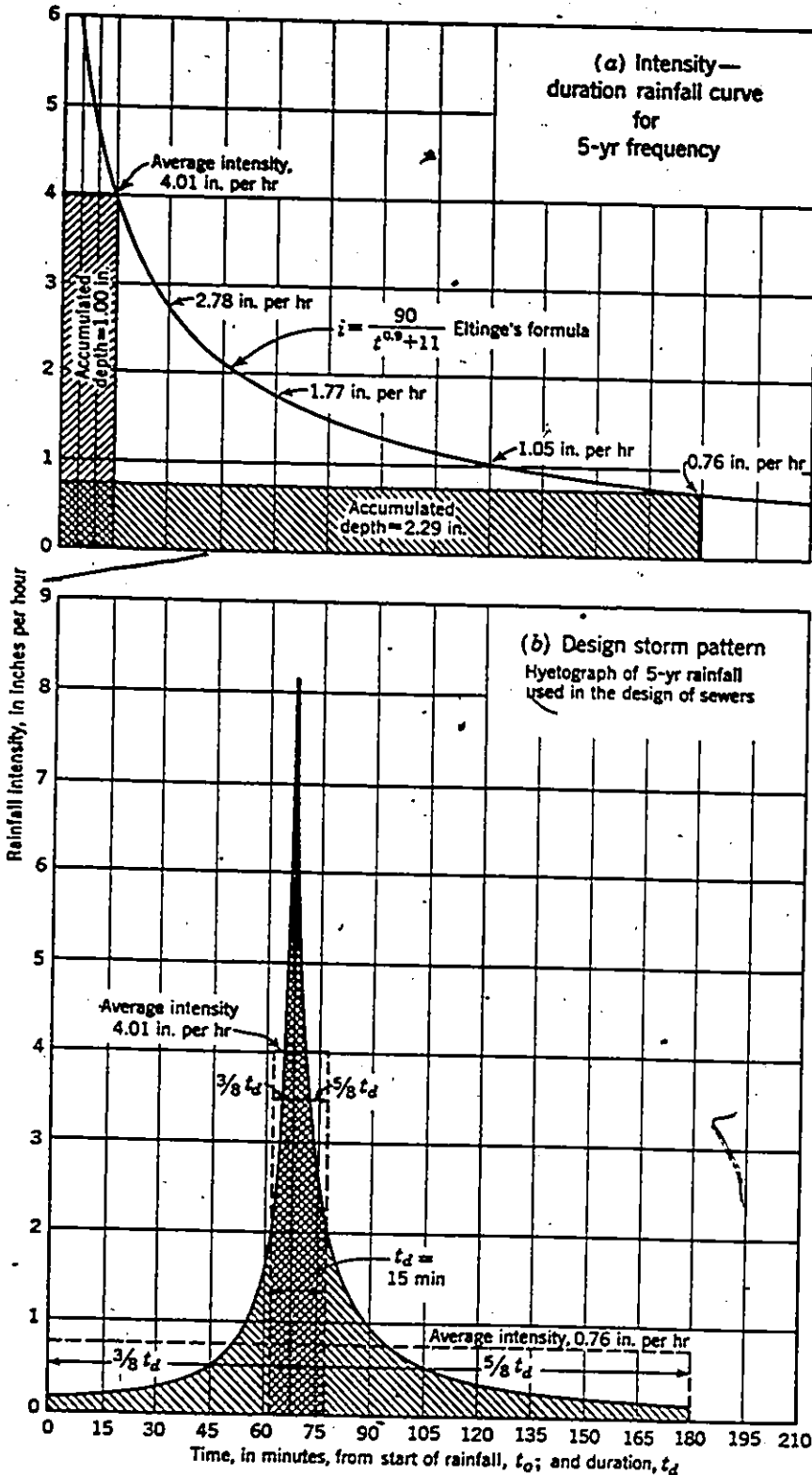
FIGURE 2.2   A DESIGN-STORM HYETOGRAPH AND
ITS INTERRELATION WITH THE
CORRESPONDING INTENSITY-DURATION
CURVE
(After Reference 3)
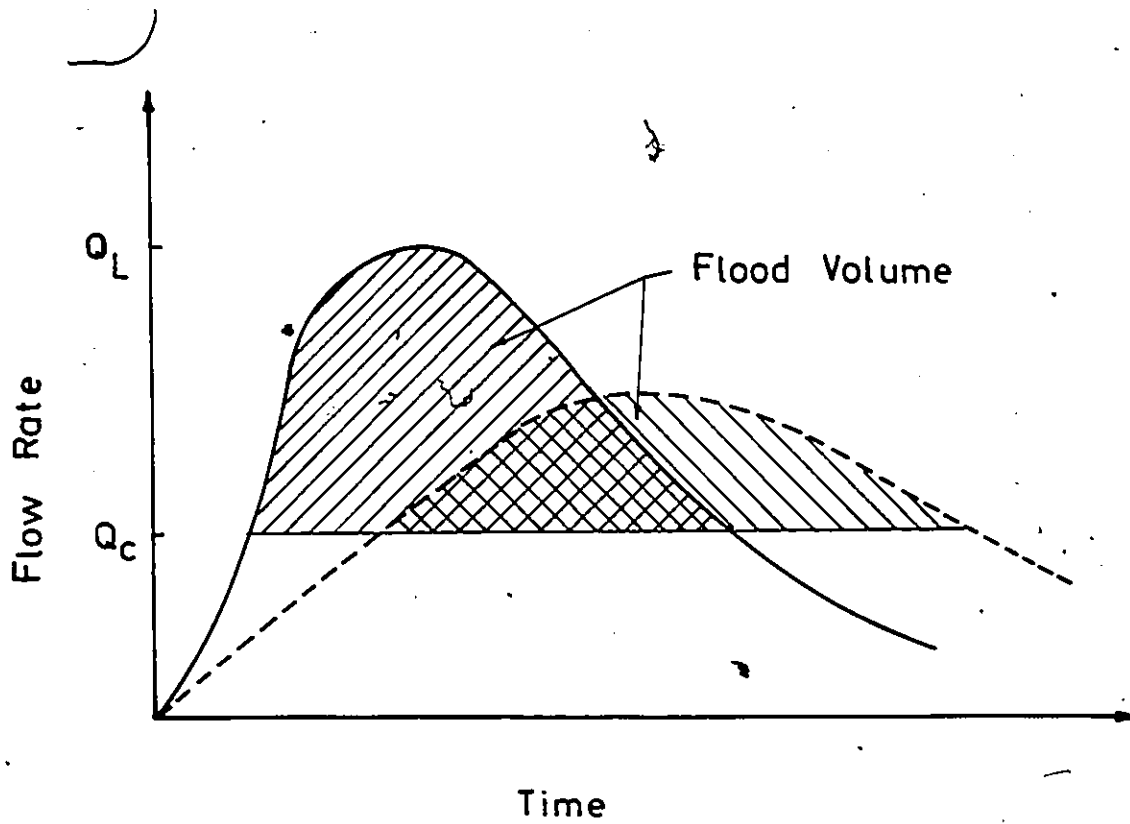
FIGURE 2.3   DETERMINATION OF FLOOD VOLUME
FROM INFLOW HYDROGRAPHS
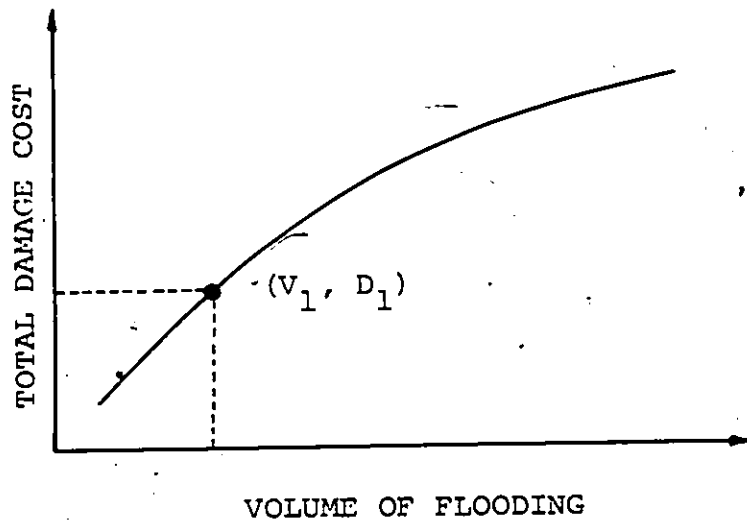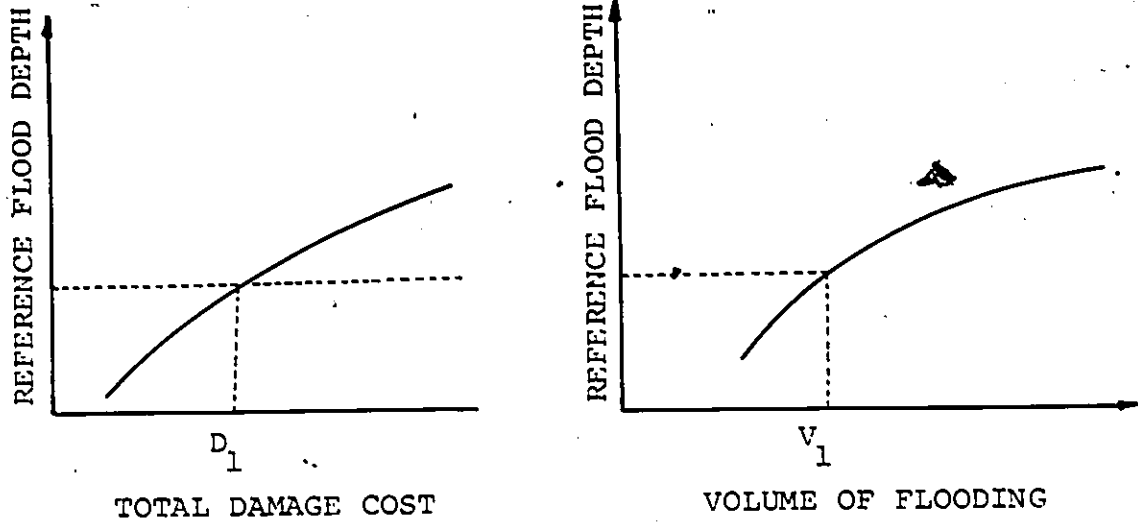
FIGURE 2.4 CONSTRUCTION OF DAMAGE COST-FLOOD VOLUME RELATIONSHIP

```
┌──────────────┐
│  I N P U T   │
└──────────────┘
        │
        │
        ▼
┌──────────────────────┐
│ S I M U L A T I O N  │
└──────────────────────┘
        │
        │
        ▼
┌──────────────────────────────┐
│ O P T I M I Z A T I O N      │
└──────────────────────────────┘
        │
        │
        ▼
┌──────────────────┐
│  O U T P U T     │
└──────────────────┘
```
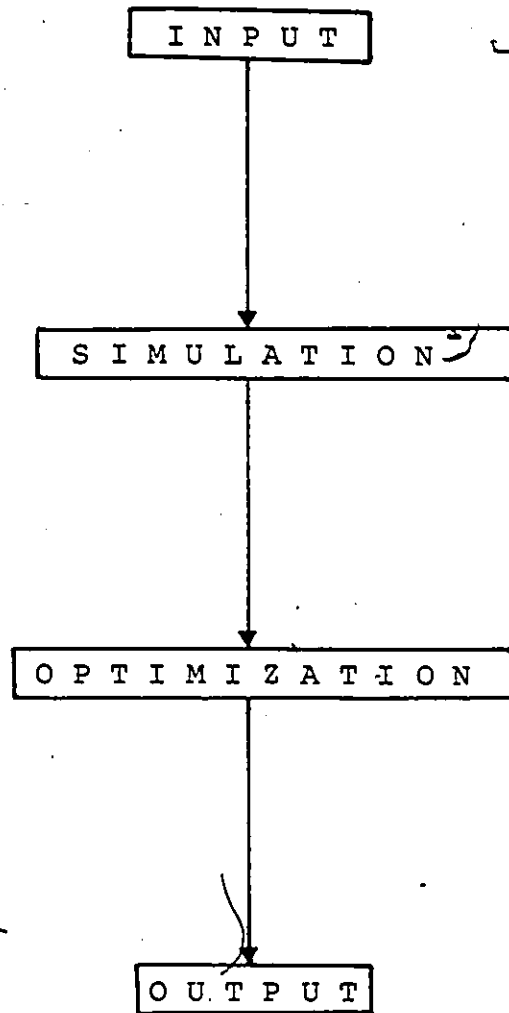
FIGURE 3.1 A SCHEMATIC DIAGRAM OF THE MODEL
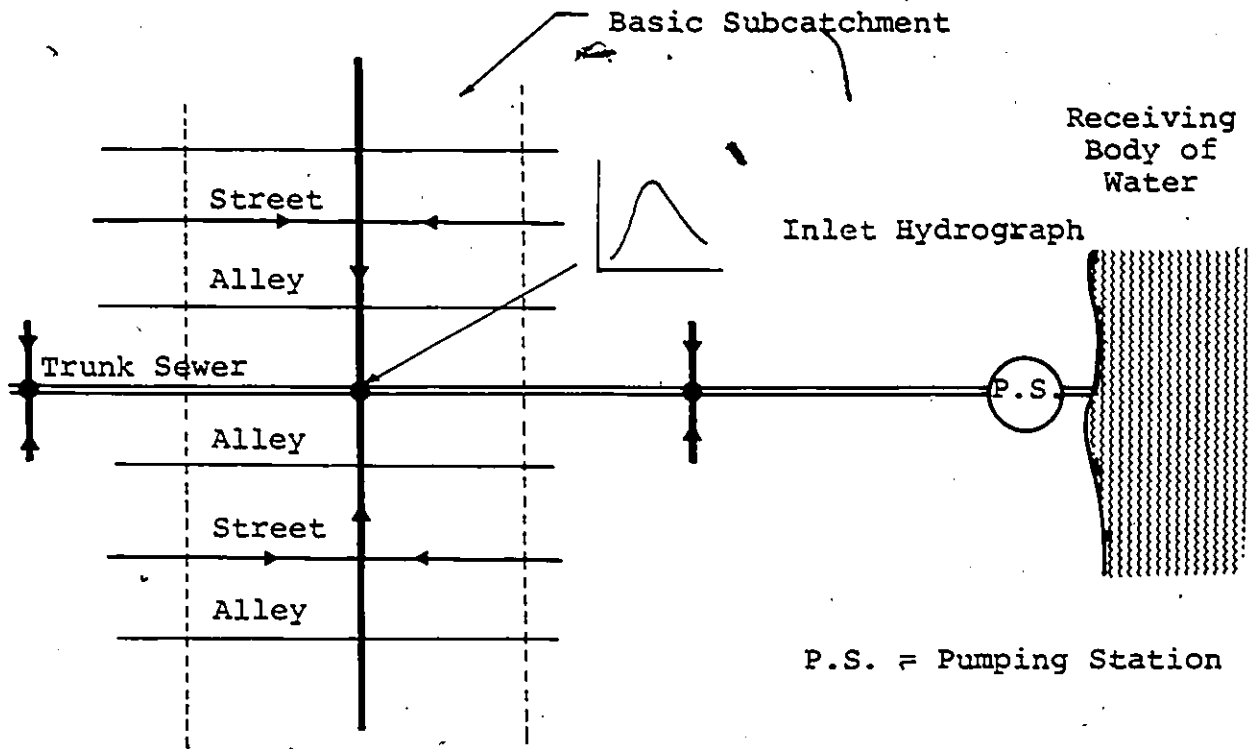
FIGURE 3.2   EXAMPLE OF A SEWERED DISTRICT AND
A BASIC SUBCATCHMENT

$$Q_{inf}(IMH,1) = i_1 \Delta A$$

$$Q_{inf}(IMH,2) = (i_1 + i_2) \Delta A$$

$$Q_{inf}(IMH,3) = (i_1 + i_2 + i_3) \Delta A$$
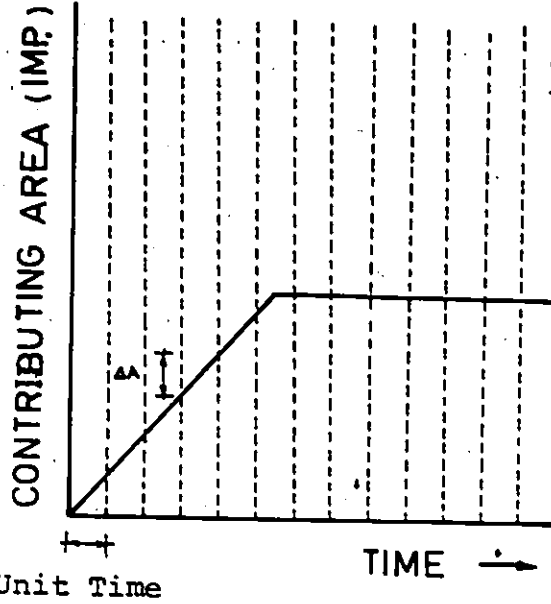
(a) AREA/TIME DIAGRAM
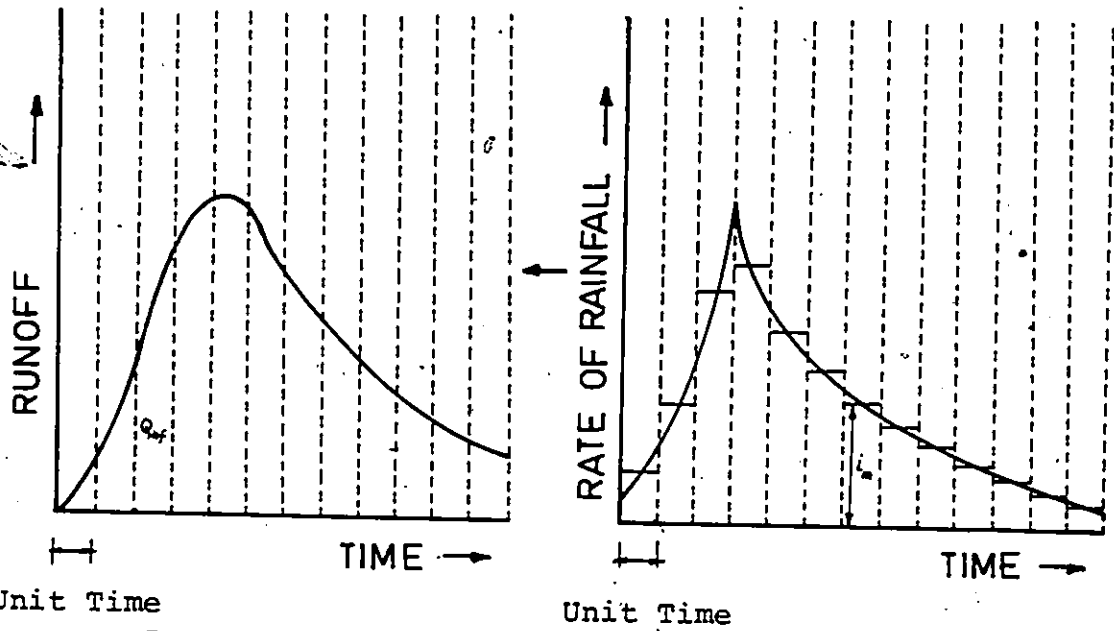
(c) RUNOFF HYDROGRAPH (b) RAINFALL/TIME CURVE

FIGURE 3.3 BASIC INFLOW HYDROGRAPH DEVELOPMENT

Wedge Storage=Kx(I-0)

Prism Storage=KO

I

0
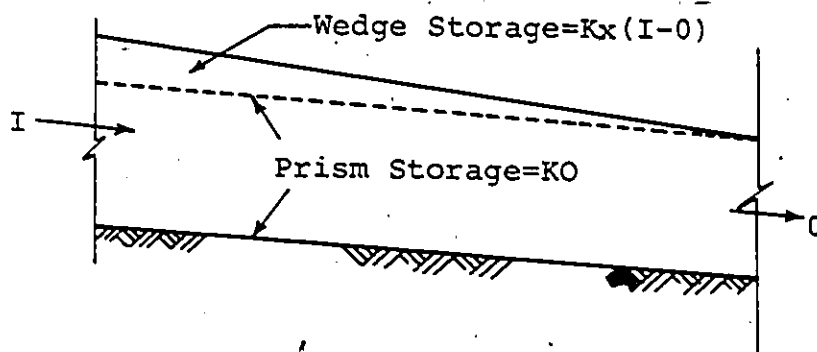
FIGURE 3.4   PRISM AND WEDGE STORAGE
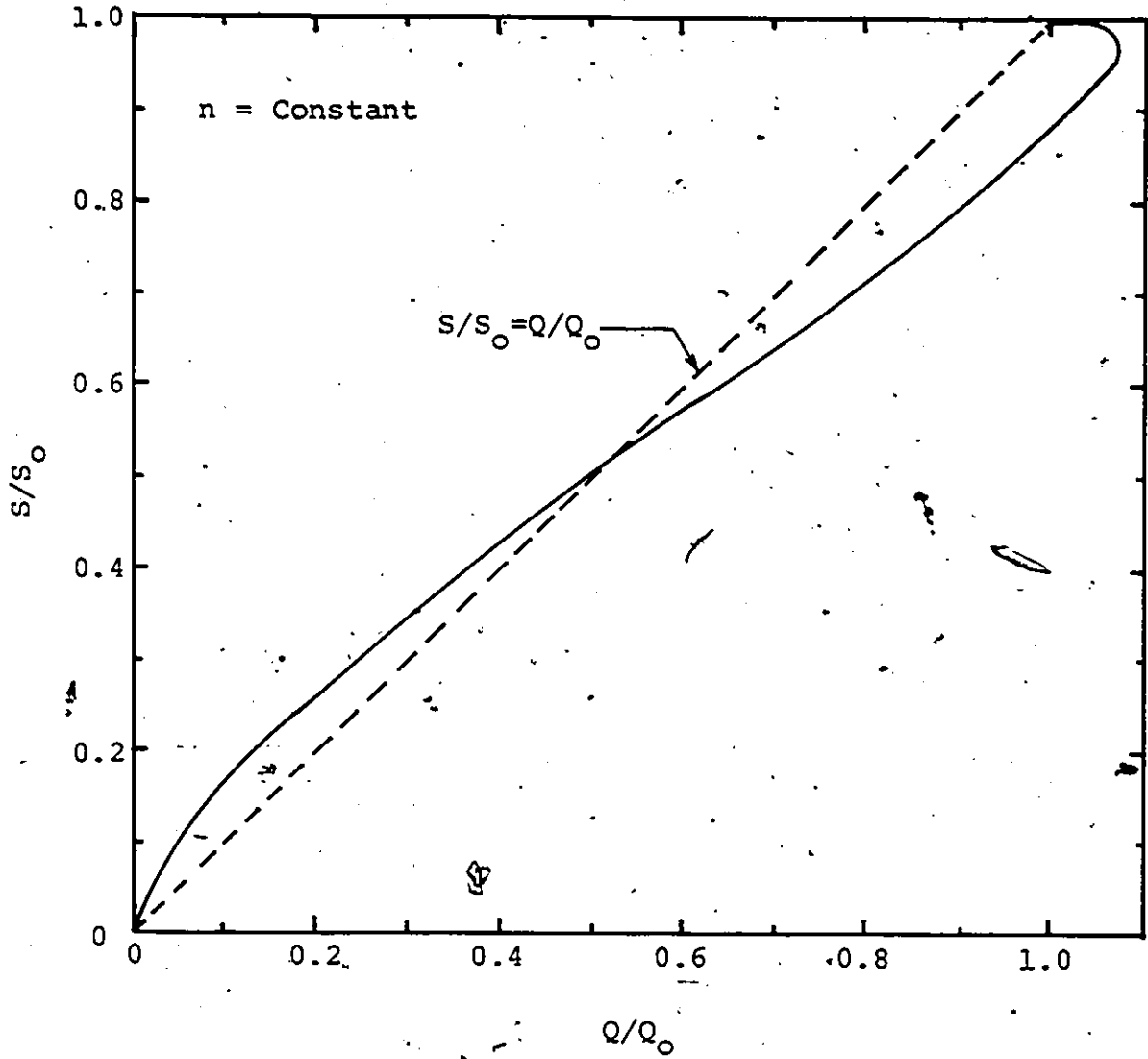
FIGURE 3.5   RELATIVE STORAGE VERSUS RELATIVE DISCHARGE
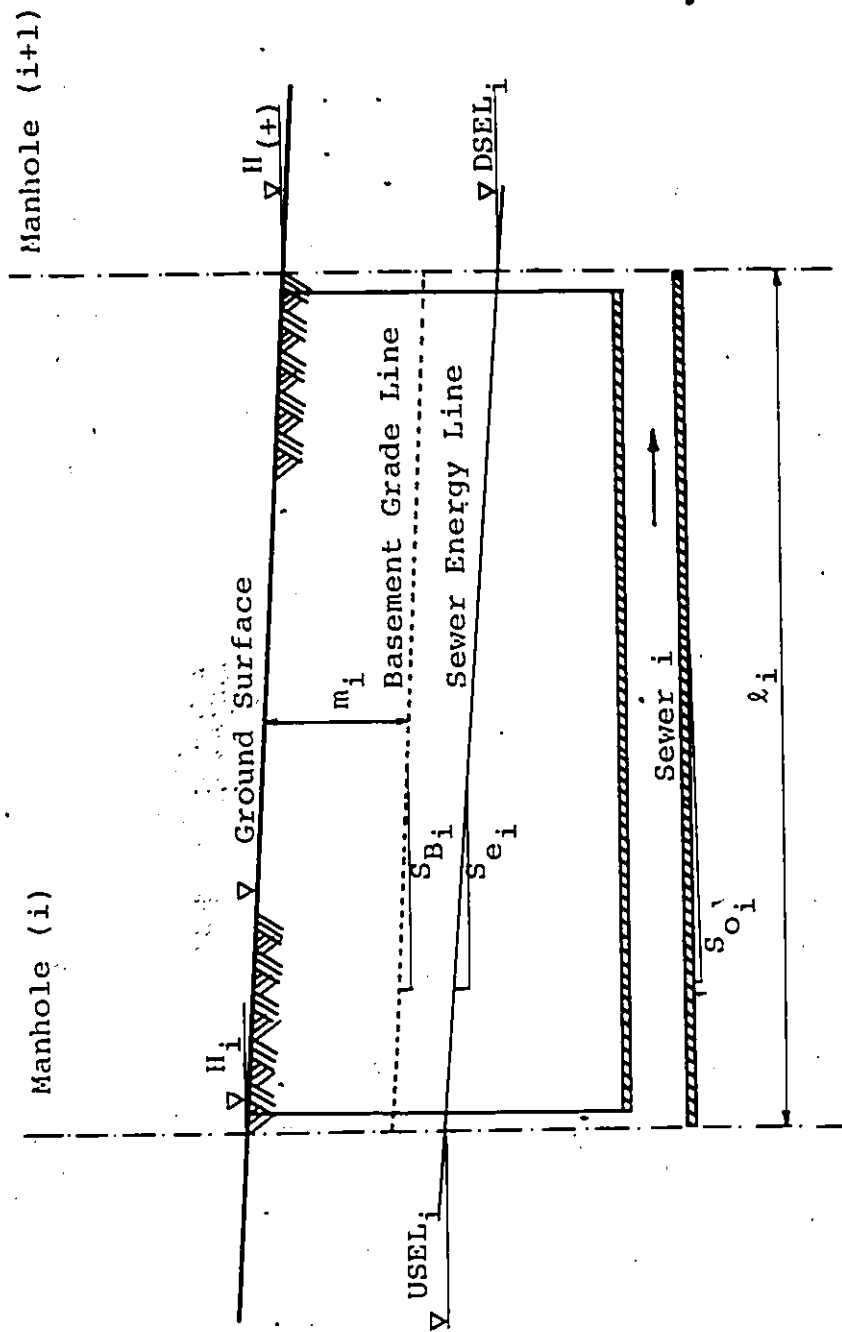
FIGURE 3.6 A TYPICAL SEWER SECTION
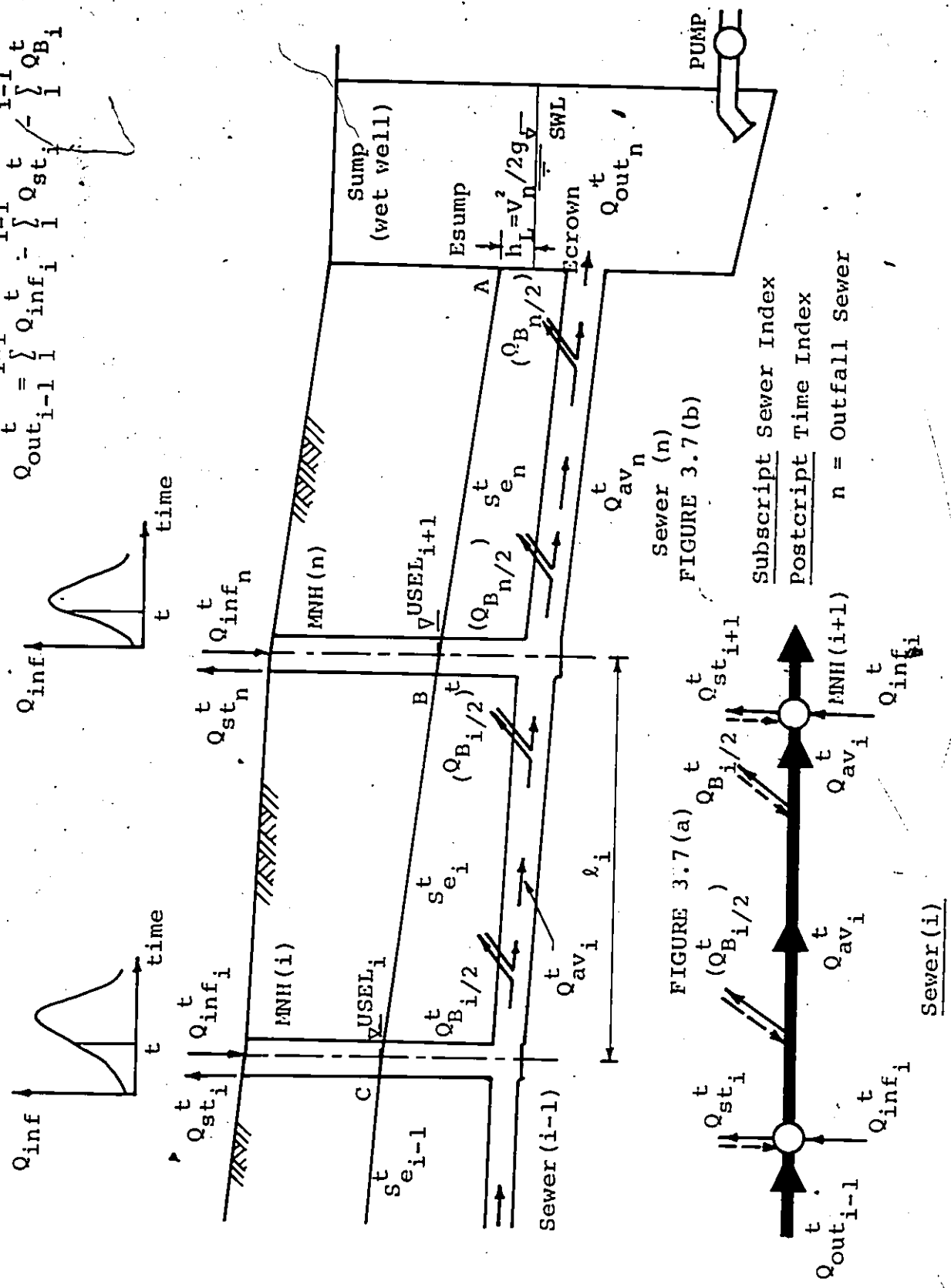
FIGURE 3.7 DEFINING SKETCH, COMPONENTS CONSTITUTING INFLOW TO AND OUTFLOW FROM SEWER SYSTEM

$$\boxed{\text{SET } Q_{st}(i,t), \ Q_B(i,t) \text{ EQUAL TO THEIR VALUES AT THE PREVIOUS TIME STEP, } t-1}$$

$$\boxed{\begin{array}{l} \text{SUM ALL UPSTREAM } Q_{inf}(i,t) \\ \text{SUM ALL UPSTREAM } Q_{st}(i,t) \\ \text{SUM ALL UPSTREAM } Q_B(i,t) \end{array}}$$

$$Q_{av}(i,t) = \sum_i^i Q_{inf}(i,t) - \sum_i^i Q_{st}(i,t) - \sum_i^{i-t} Q_B(i,t) - Q_B(i,t)/2$$

$$S_e(i,t) = K_{e_i} [Q_{av}(i,t)]^2$$

$$(\text{U/S ENERGY LEVEL})_i = (\text{U/S ENERGY LEVEL})_{i+1} + S_e(i,t)*\ell_i$$

$$\boxed{\text{COMPUTE } Q_{st}(i,t), \ Q_B(i,t)}$$

ITERATE 3 TIMES FOR SUMP ENERGY LEVEL, POINT A

ITERATE 6 TIMES FOR $Q_{st}(i,t)$, $Q_B(i,t)$

ARE ALL PIPES CONSIDERED (STARTING FROM THE OUTFALL SEWER TOWARDS THE UPSTREAM)?

FIGURE 3.8   SIMPLIFIED FLOW CHART FOR CONSTRUCTION OF THE SEWER ENERGY LINES

Roof Drains

Basement
▽ Floor Level

Foundation drains

Check Valve (if any)

Storm Sewer
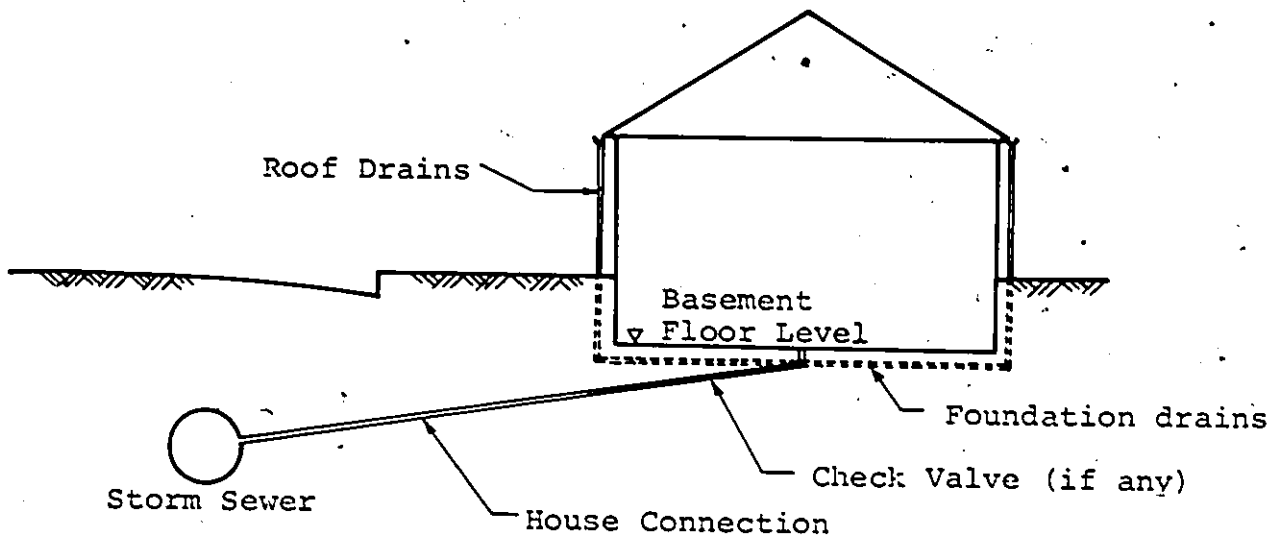
House Connection

**FIGURE 3.9.a A TYPICAL HOUSE CONNECTION DRAINING
A BASEMENT TO A STORM SEWER**

h_ℓ

Flooding
Depth

▽ Water Level C
▽ Basement Floow
Level B

Basement
Grade Line

A

S.E.L

Storm
Sewer

Sewer Energy
Line

Q_{av}
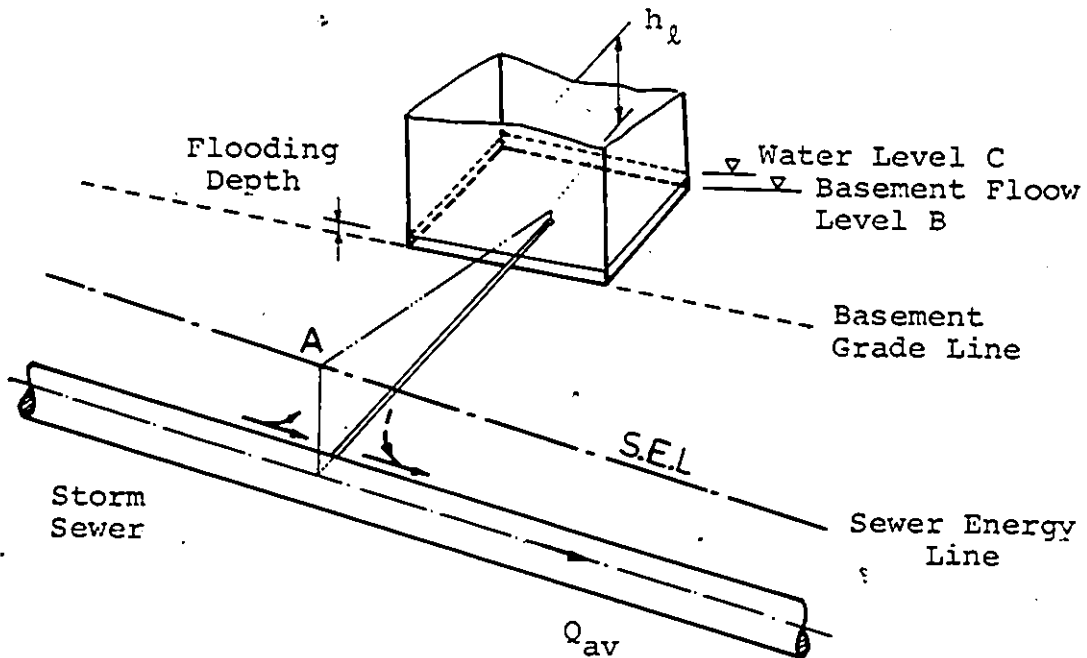
**FIGURE 3.9.b   ENERGY LINES FOR BOTH THE STORM SEWER
AND THE HOUSE CONNECTION**
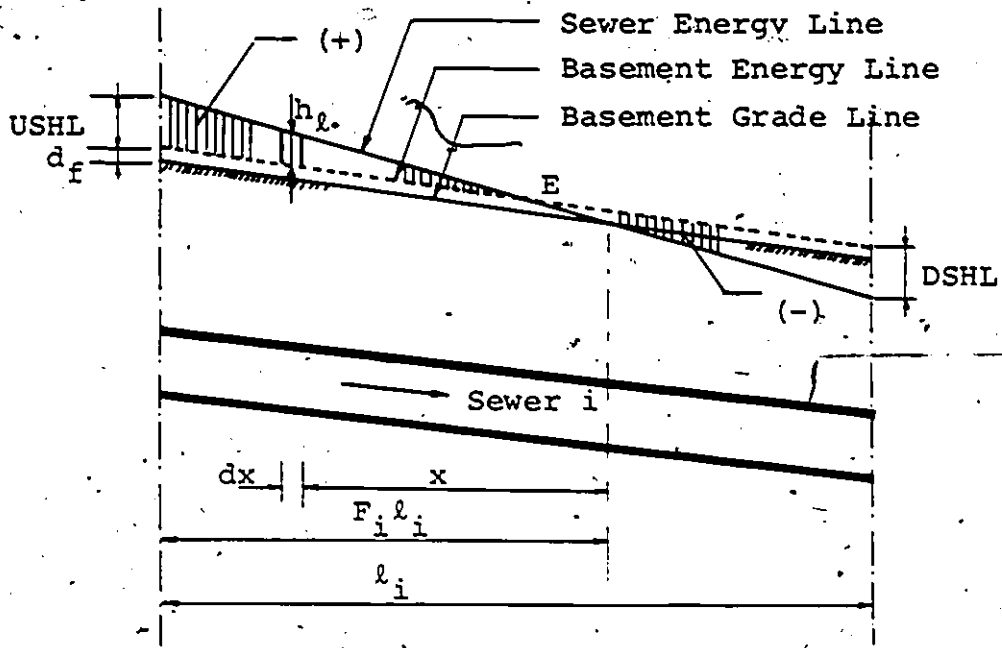
FIGURE 3.10.a Case (A) F<1.00
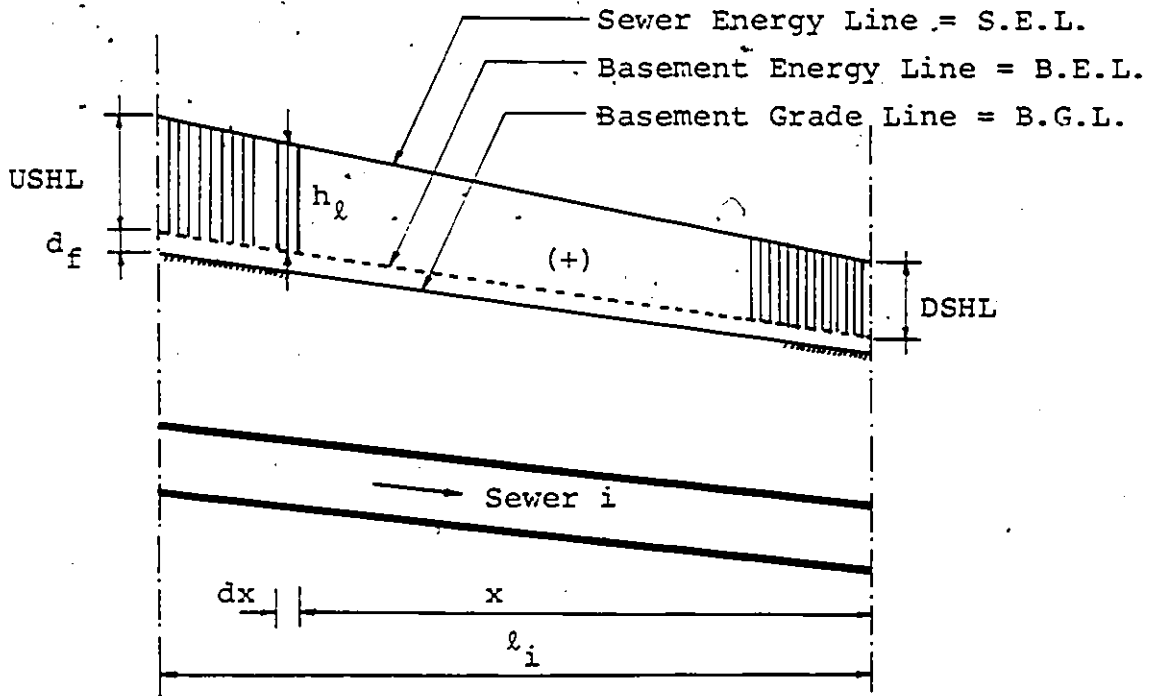
FIGURE 3.10.b Case (B) f = 1.00

FIGURE 3.10 COMPUTATION OF TOTAL BASEMENT FLOWS
ALONG A SEWER SECTION
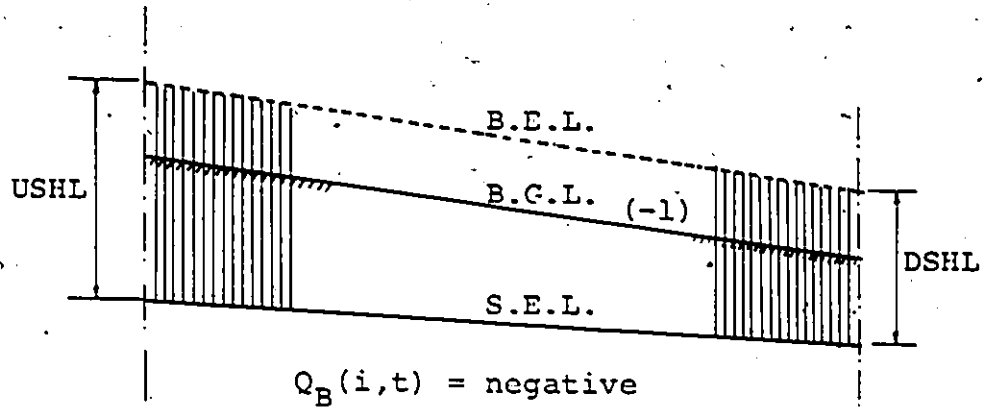
FIGURE 3.10.c   Case(C)   F=0.00
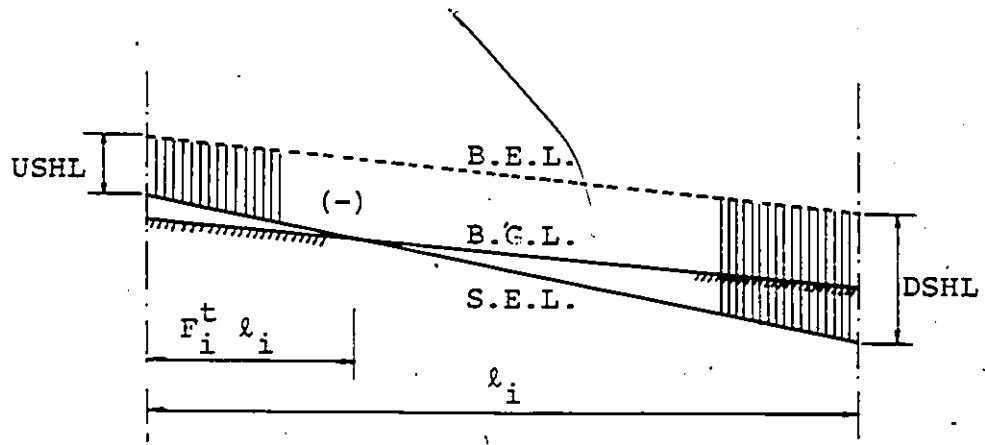
FIGURE 3.10.d   Case(D)
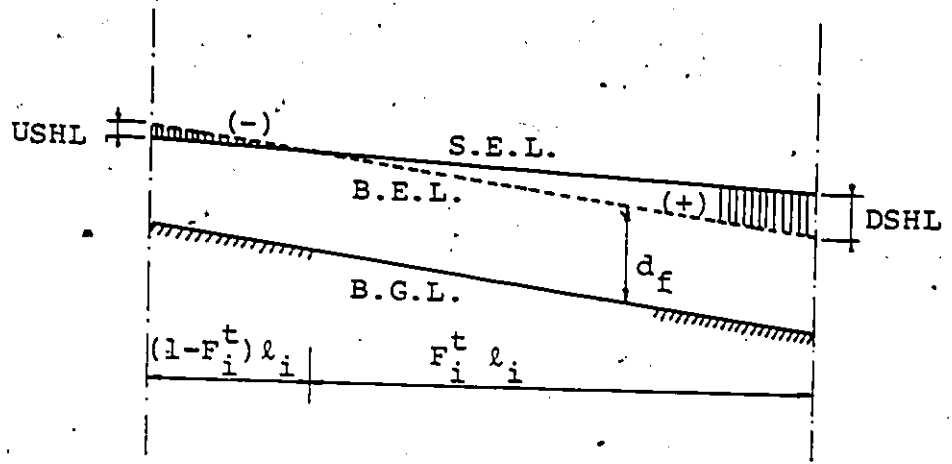
$$F_i^t = \frac{DSHL}{|USHL| + DSHL}$$
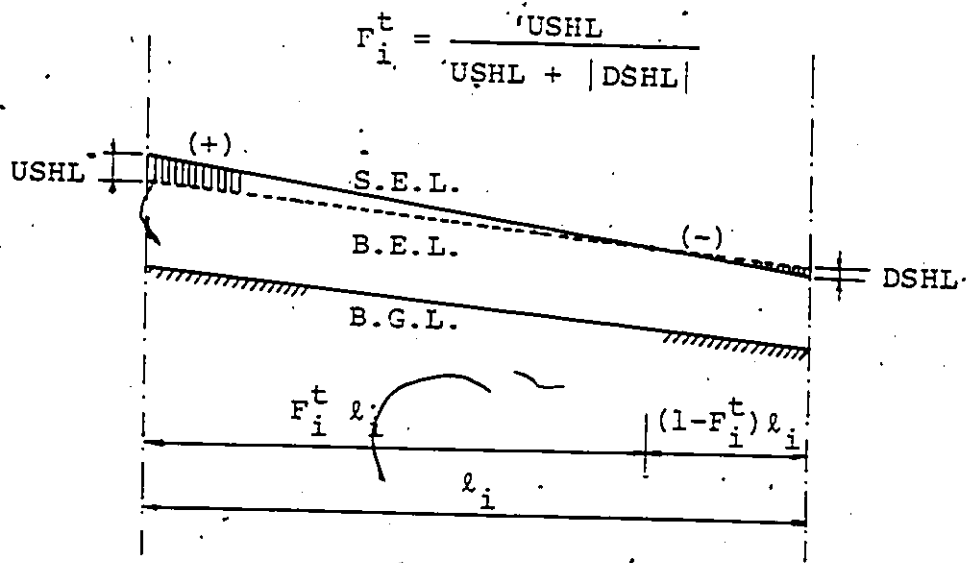
$$F_i^t = \frac{USHL}{USHL + |DSHL|}$$

FIGURE 3.10.e   Case(E)

S.E.L.=Sewer Energy Line
B.G.L.=Basement Grade Line
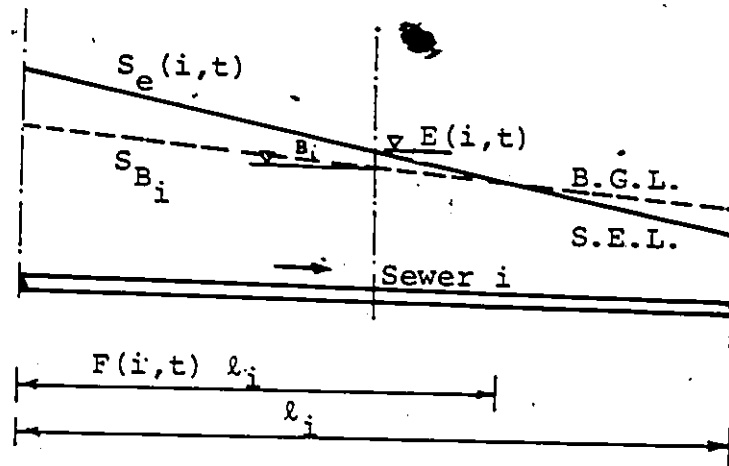
FIGURE 3.11.a   DETERMINATION OF THE FLOODING RATIO
                Case (I)
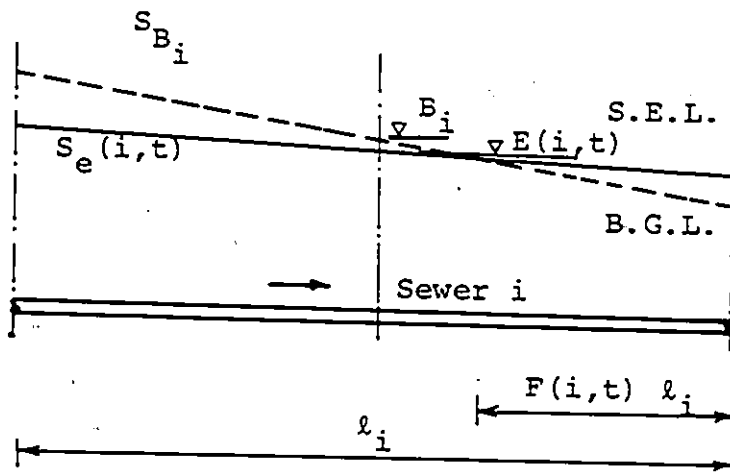
FIGURE 3.11.b   DETERMINATION OF THE FLOODING RATIO
                Case (II)
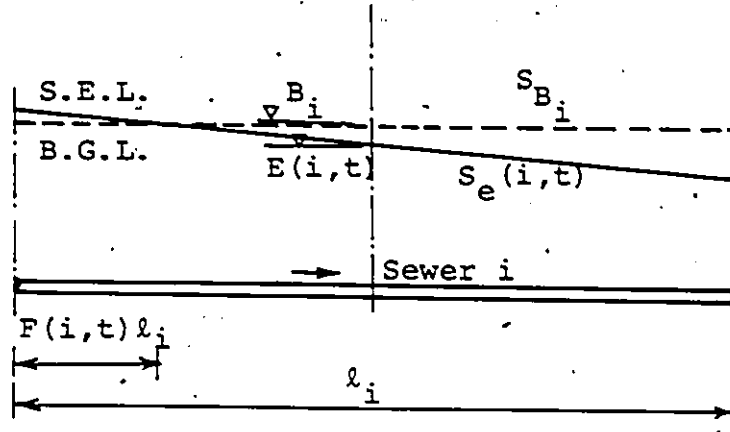
FIGURE 3.11.c DETERMINATION OF THE FLOODING RATIO
Case (III)
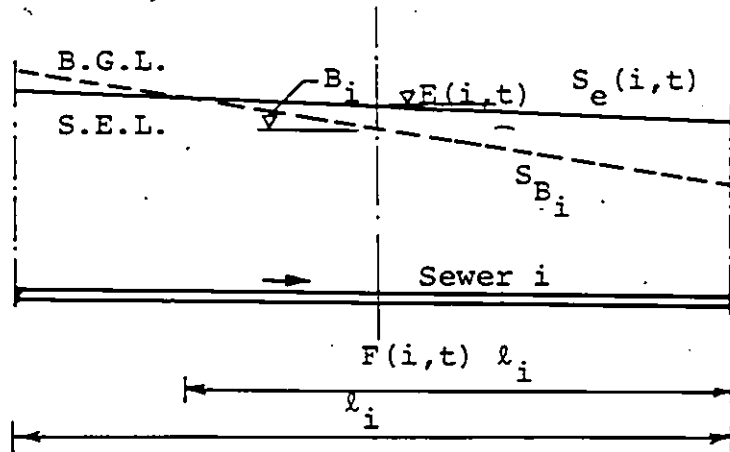
FIGURE 3.11.d DETERMINATION OF THE FLOODING RATIO
Case (IV)
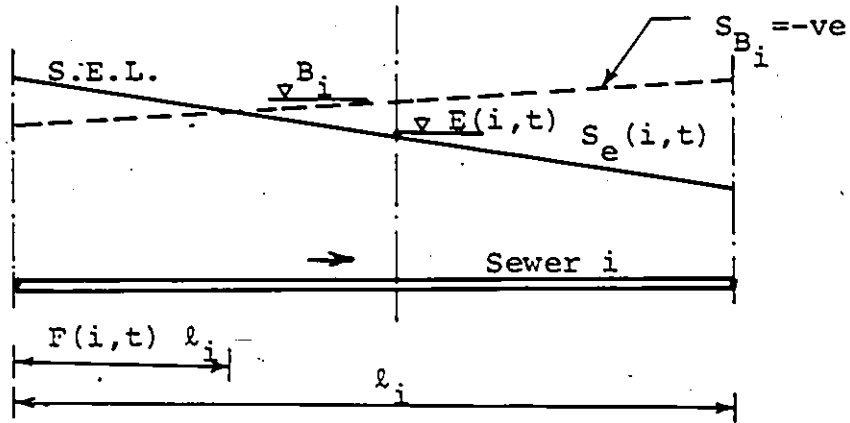
FIGURE 3.11.e   DETERMINATION OF THE FLOODING RATIO
Case (V)
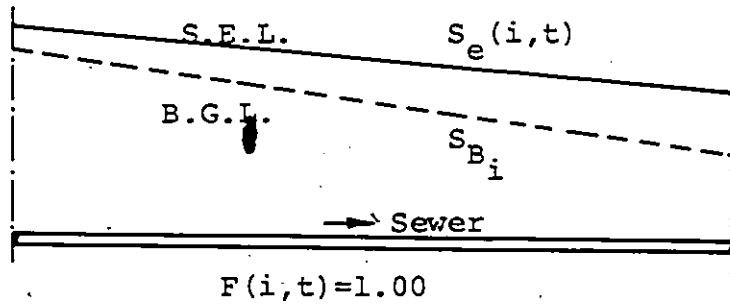
FIGURE 3.11.f   DETERMINATION OF THE FLOODING RATIO
Case(VI)

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$$\text{'IF'}\ USEL_i > H_i$$

No → $Q_{st}(i,t) = 0.0$

Yes

$$USEL_i = H_i$$

$$S_e(i,t) = \frac{USEL_i - DSEL_i}{\ell_i}$$

$$Q_{av}(i,t) = \frac{S_e(i,t)}{K_{e_i}}$$

$$Q_{in} = \sum_{i=1}^{i} Q_{inf}(i,t) - \sum_{i=1}^{i-1} Q_B(i,t) - \sum_{i=1}^{i-1} Q_{st}(i,t)$$

$$Q_{ot} = Q_{av}(i,t) + Q_B(i,t)/2$$

$$Q_{st}(i,t) = Q_{in} - Q_{ot}$$
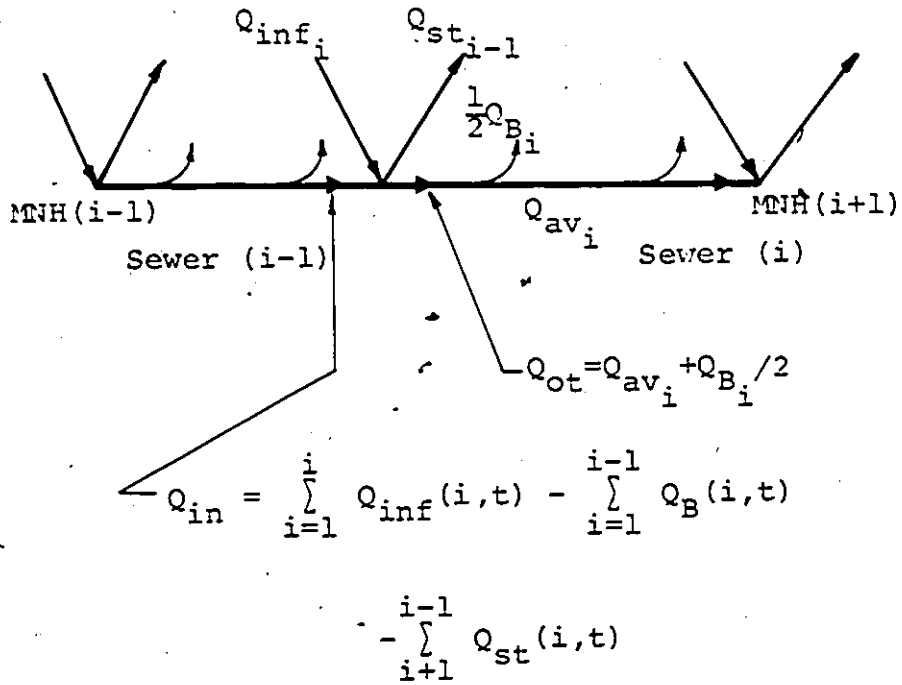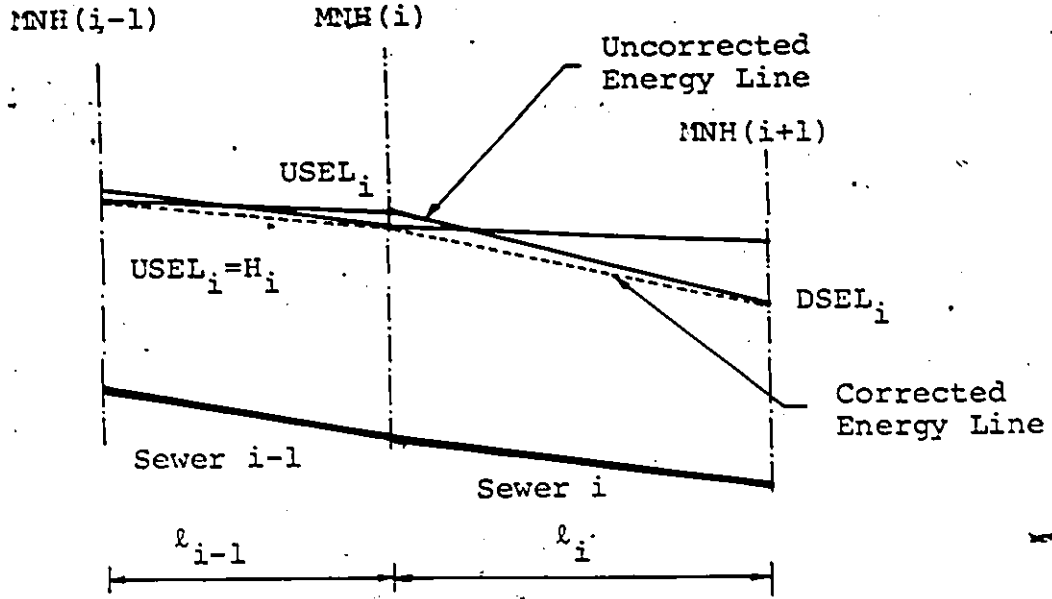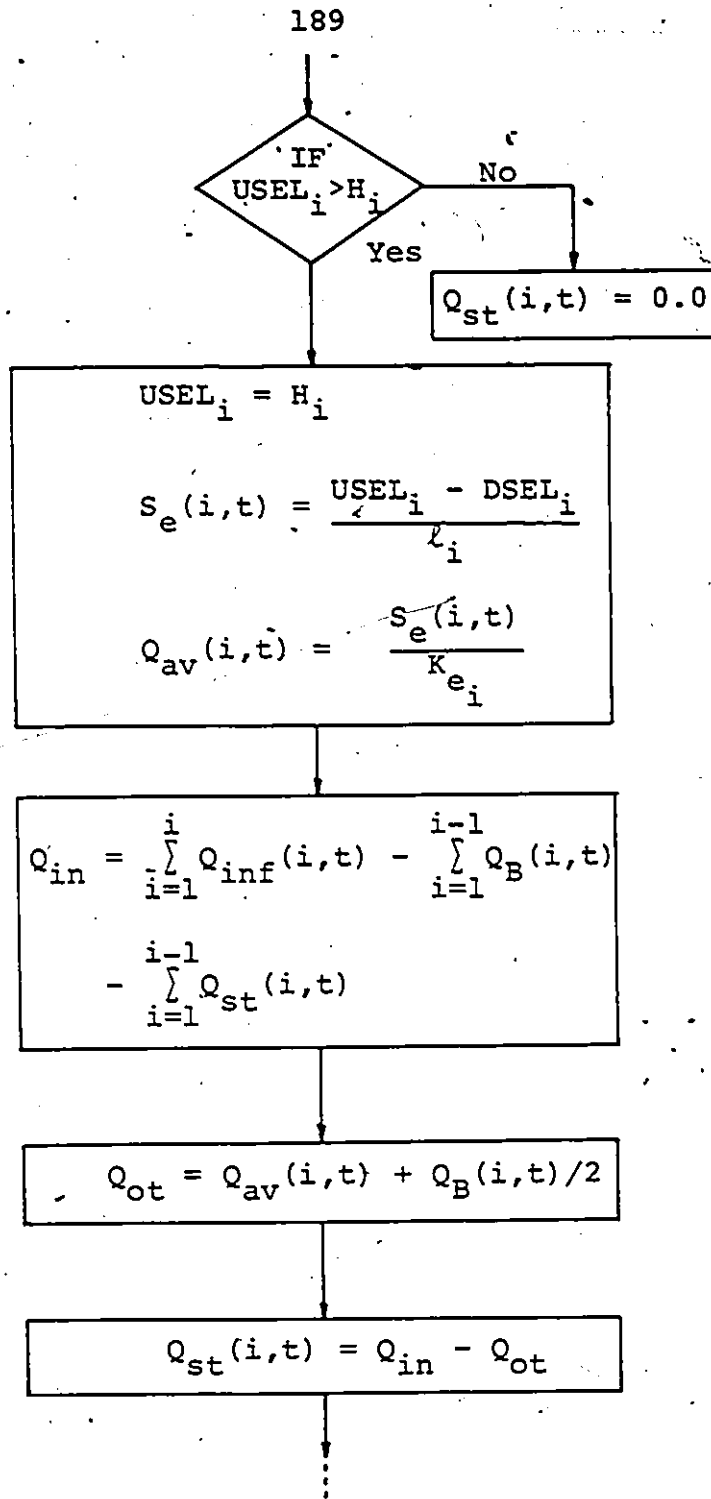
FIGURE 3.13   SIMPLIFIED FLOW CHART FOR COMPUTATION OF STREET FLOWS

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TIME INDEX = t

ARE ALL PIPES CONSIDERED?

$$(STORAGE)_i = K_{MUSKINGUM}[xI + (1-x)0]$$

$$(ACTUAL\ STORAGE)_{SYSTEM} = \sum_{i=1}^{n} (STORAGE)_i$$

$$RELATIVE\ STORAGE = \frac{(ACTUAL\ STORAGE)_{SYSTEM}}{(MAX.AVAILABLE\ STORAGE)_{SYSTEM}}$$

IF
REL.STORAGE $\geq$ 0.98

GRAVITY FLOW MODEL

SURCHARGED FLOW MODEL
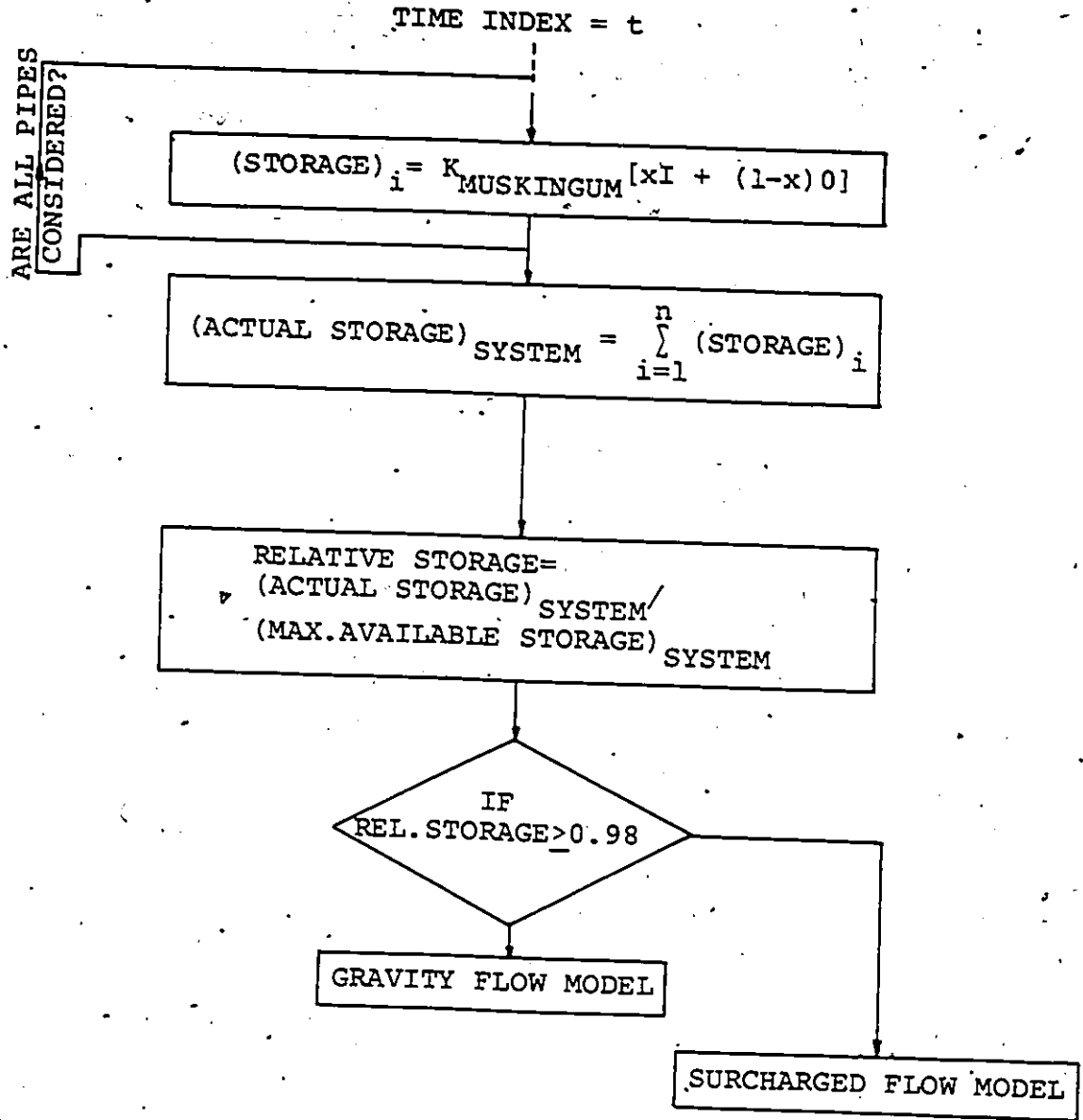
FIGURE 3.14   SIMPLIFIED FLOW CHART FOR THE SWITCHING
PROCEDURE
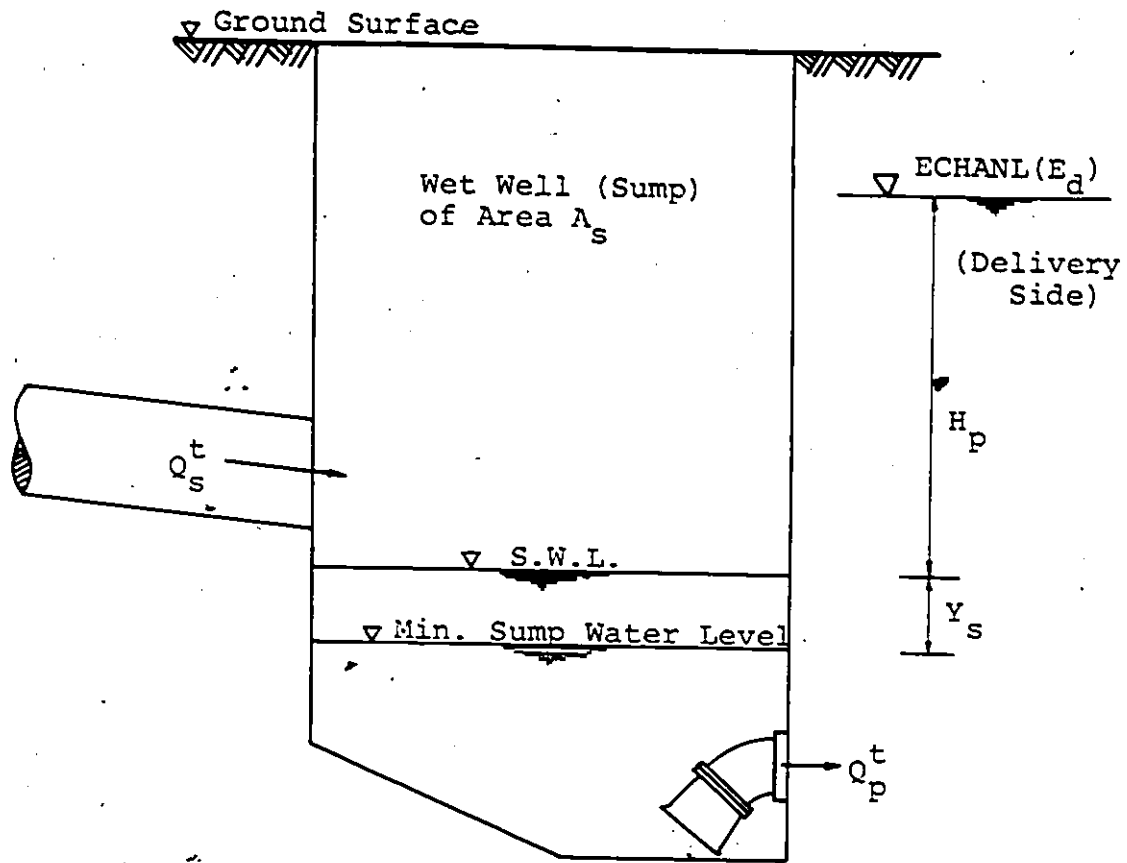
FIGURE 3.15 DEFINING SKETCH, SUMP-PUMP MODEL
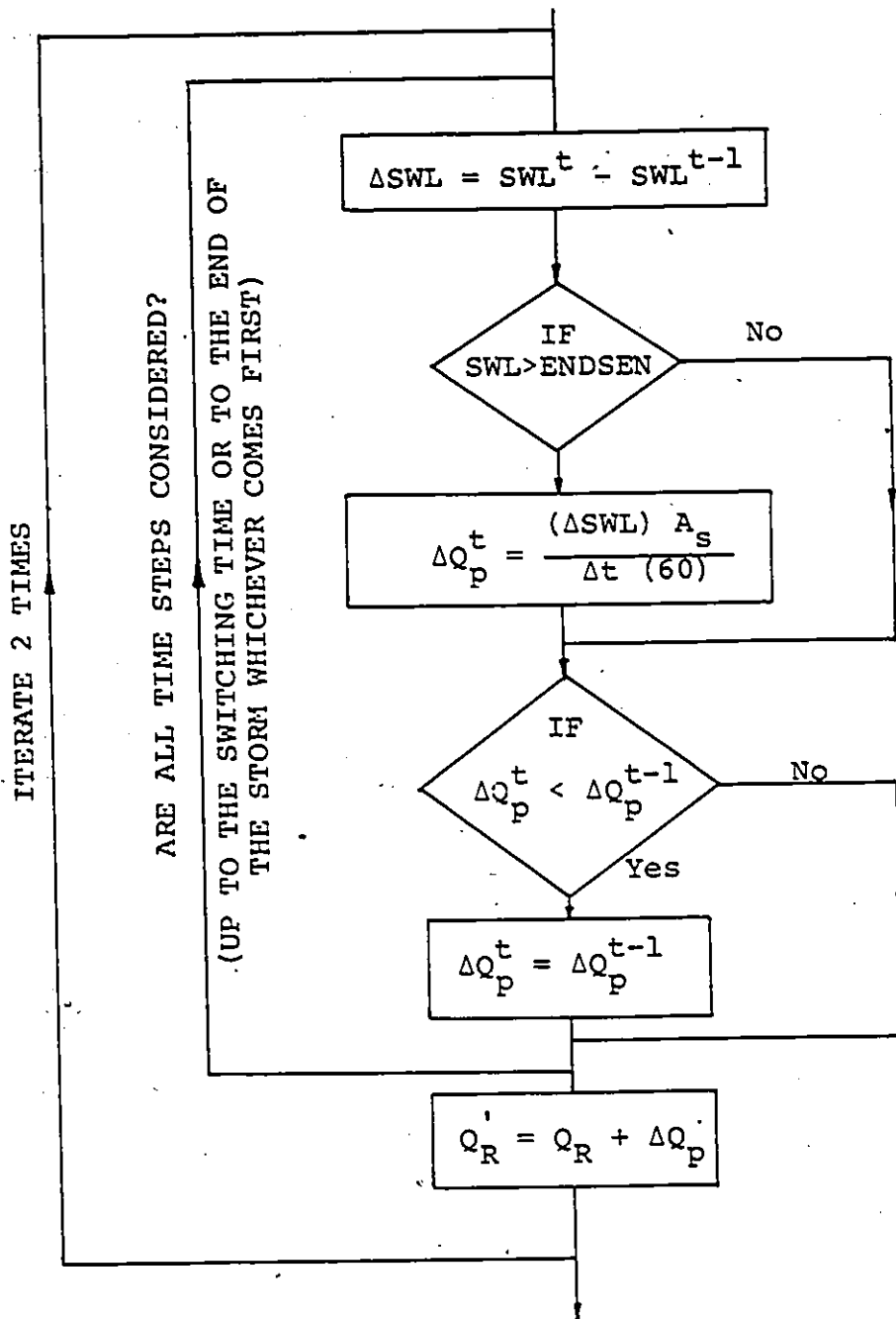
FIGURE 3.16   SIMPLIFIED FLOW CHART FOR ADJUSTING
                PUMPING CAPACITY
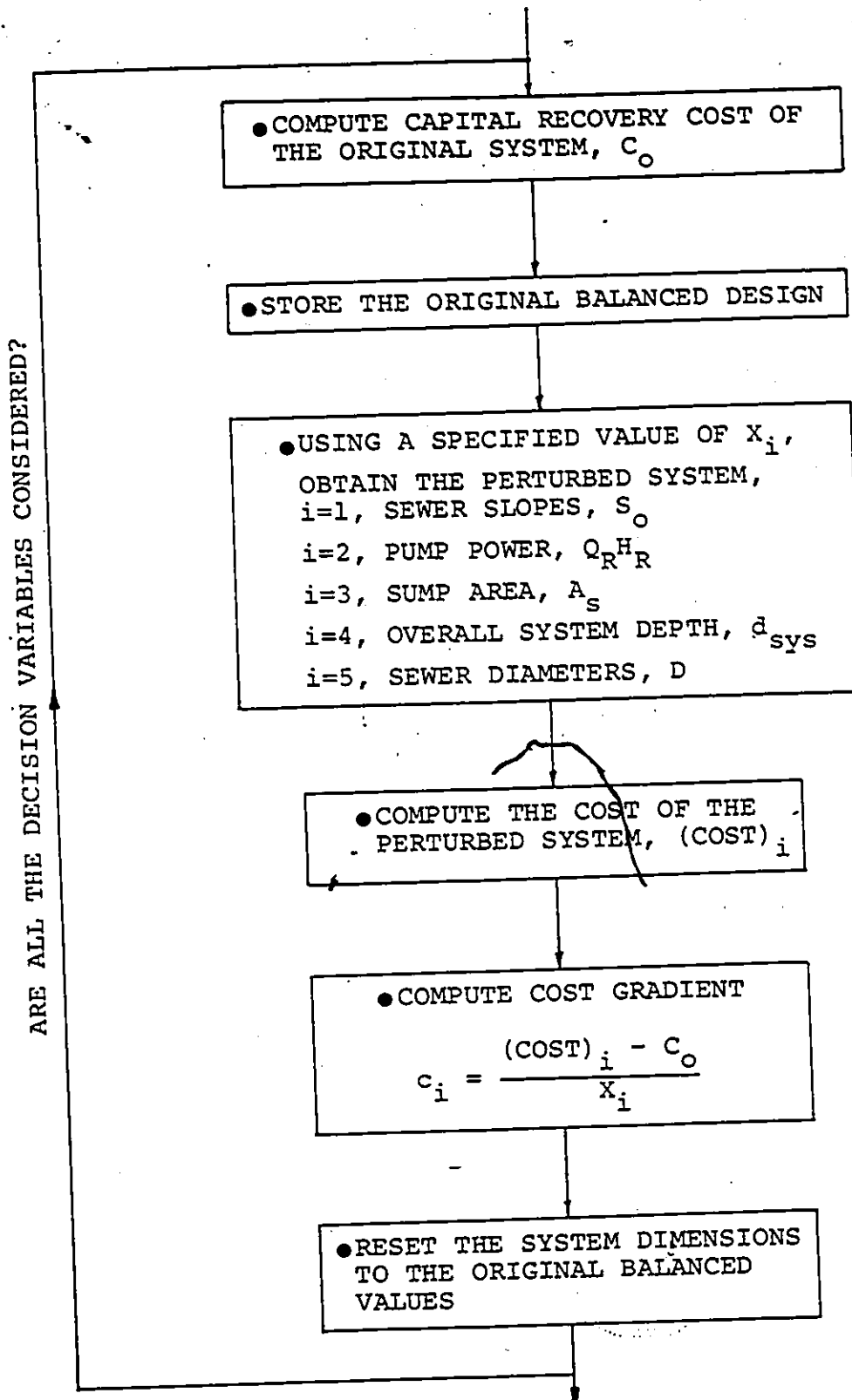
FIGURE 3.17 SIMPLIFIED FLOW CHART FOR COMPUTATION OF COST GRADIENTS

$$c_i = \frac{C_1 - C_0}{X_i}$$
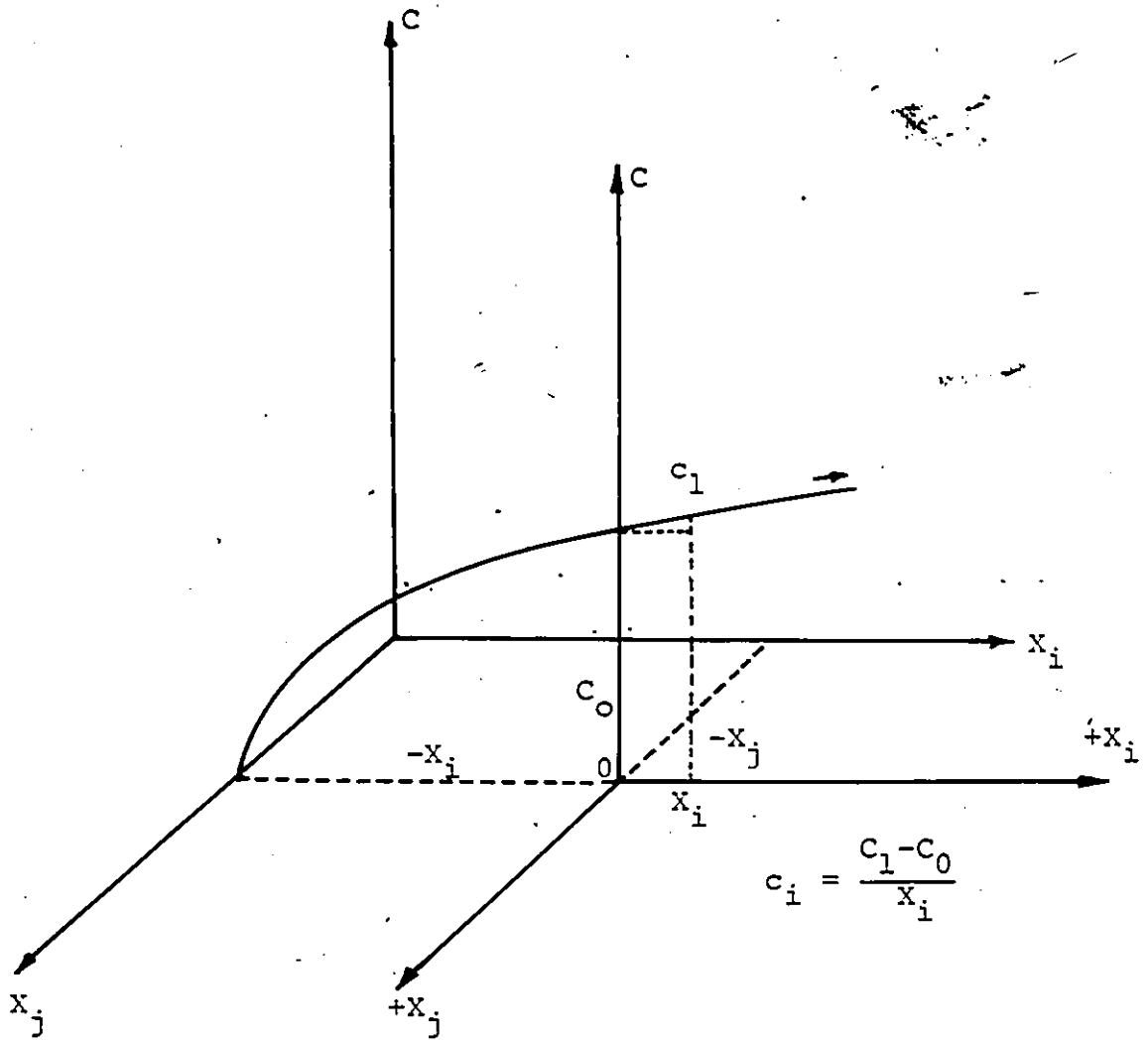
FIGURE 3.18   DEFINING SKETCH, FORWARD COST GRADIENTS
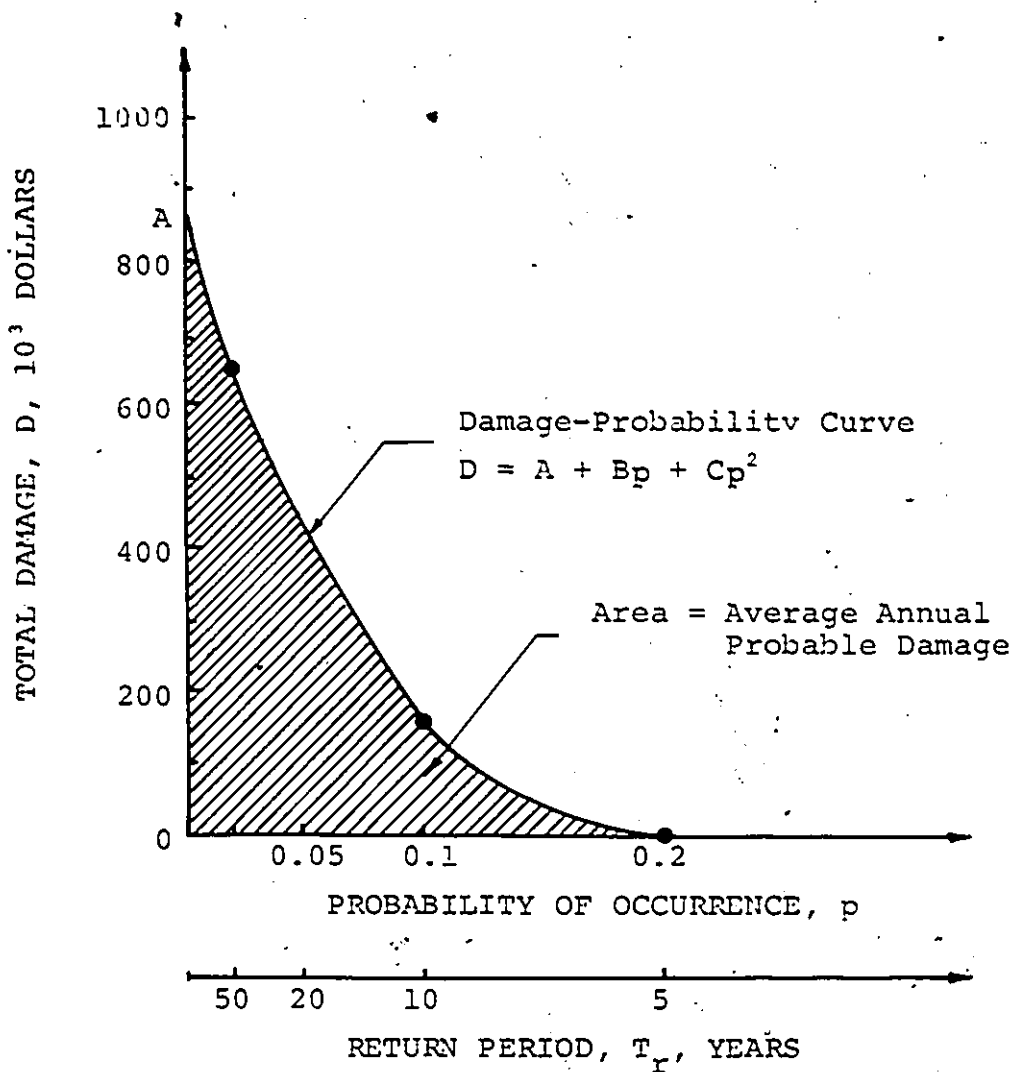
FIGURE 3.19 A TYPICAL DAMAGE-PROBABILITY CURVE
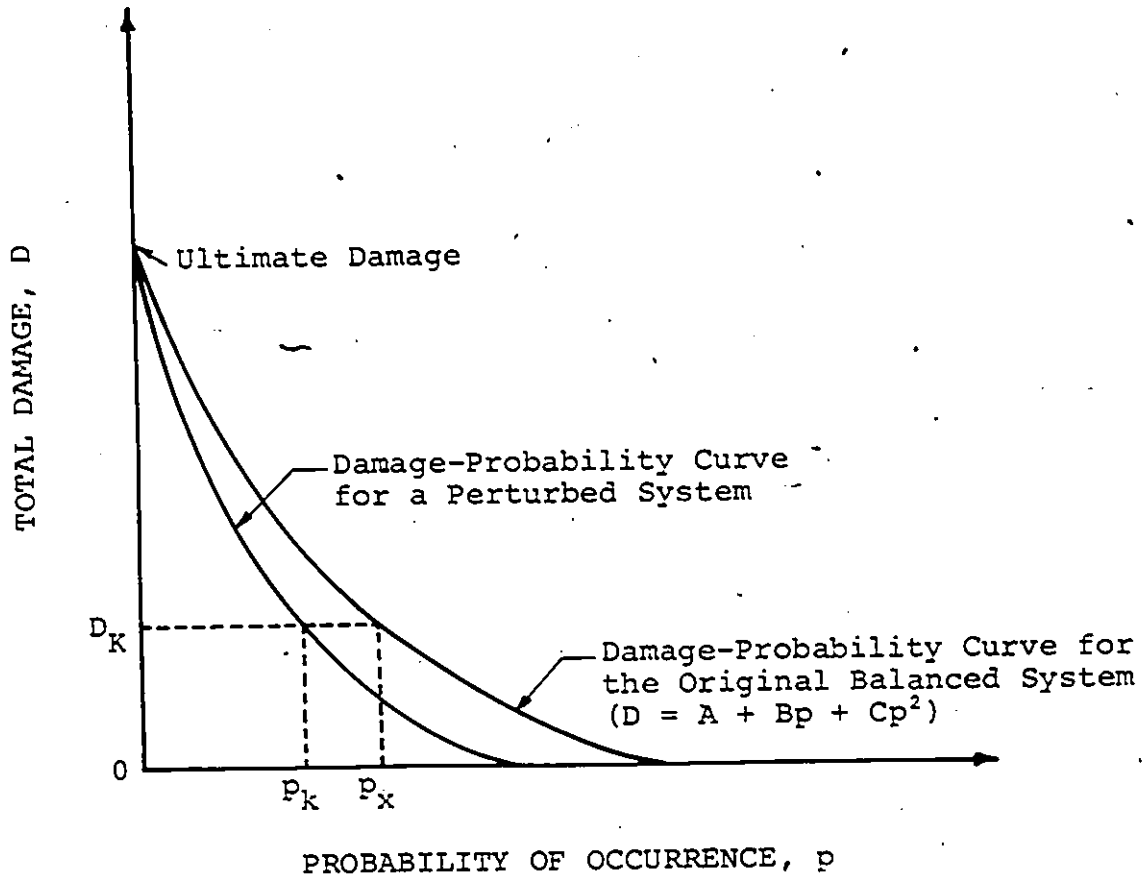
FIGURE 3.20   COMPUTATION OF AVERAGE ANNUAL PROBABLE DAMAGE
FOR A PERTURBED SYSTEM

.GENERATE A DAMAGE-PROBABILITY CURVE
FOR THE ORIGINAL BALANCED SYSTEM
(FIND A, B, AND C)
.COMPUTE THE AVERAGE ANNUAL PROBABLE
DAMAGE FOR THE ORIGINAL BALANCED,
d_o, FROM

$$d_o = Ap_1 + \frac{1}{2} Bp_1^2 + Cp_1^3$$

.USING A SPECIFIED VALUE OF X_i (THE
SAME USED IN OBTAINING THE COST
GRADIENTS), OBTAIN THE PERTURBED
SYSTEM.

.APPLY THE PRESELECTED REPRESENTATIVE
STORM, WITH A PROBABILITY OF OCCUR-
RENCE, p_k, ON THE PERTURBED SYSTEM
.USING THE DAMAGE MODEL, COMPUTE THE
TOTAL DAMAGE, D_K, RESULTING FROM THIS
PARTICULAR STORM.

.COMPUTE p_x, USING THE VALUES A, B AND
C OBTAINED FROM THE ORIGINAL BALANCED
SYSTEM FROM

$$p_k = \frac{-B - \sqrt{B^2 - 4C(A - D_K)}}{2C}$$

.COMPUTE THE AVERAGE ANNUAL PROBABLE
DAMAGE FOR THE PERTURBED SYSTEM, FROM

$$(D_{av})_i = d_o \left(\frac{p_k}{p_x}\right)$$

.DAMAGE GRADIENT,

$$d_i = \frac{(D_{av})_i - d_o}{X_i}$$

.RESET THE SYSTEM DIMENSIONS TO THE
ORIGINAL BALANCED VALUES.

ARE ALL THE DECISION VARIABLES CONSIDERED?

FIGURE 3.21   COMPUTATION OF DAMAGE GRADIENTS

FIGURE 3.22   UPPER AND LOWER LIMITS ON THE AVERAGE
              ANNUAL PROBABLE DAMAGE OF THE OPTIMUM SYSTEM

```
┌─────────────────────┐
│  READ INPUT DATA    │
└─────────────────────┘
```

┌──┐
│ .FOR A PRESELECTED REPRESENTATIVE STORM, BALANCE   │
│ THE INITIAL DESIGN OF THE SEWER SYSTEM BY THE   │
│ FOLLOWING PROCEDURE,   │
│   (i)   GENERATE RAINFALL INTENSITY ARRAY,   │
│   (ii)   GENERATE BASIC INFLOW HYDROGRAPHS,   │
│   (iii)   ROUTE THE HYDROGRAPHS THROUGH THE SEWER   │
│   SYSTEM, USING THE GRAVITY FLOW MODEL,   │
│   (iv)   BALANCE THE INITIAL DESIGN BY ITERATING   │
│   3 TIMES FOR SEWER DIAMETERS AND 2 TIMES   │
│   FOR THE PUMPING SYSTEM.   │
└──┘

┌──┐
│ .COMPUTE THE CAPITAL RECOVERY COST, C_o, OF THE   │
│ ORIGINAL BALANCED DESIGN.   │
└──┘

┌──┐
│ .APPLY TWO OTHER PRESELECTED REPRESENTATIVE   │
│ STORMS WITH HIGHER RETURN PERIODS   │
│   (i)   GENERATE RAINFALL INTENSITY ARRAY,   │
│   (ii)   GENERATE BASIC INFLOW HYDROGRAPHS,   │
│   (iii)   ROUTE THE HYDROGRAPHS THROUGH THE   │
│   SEWER SYSTEM, USING THE GRAVITY FLOW   │
│   MODEL, AND CHECK FOR SURCHARGED FLOW   │
│   CONDITION. IF NO SURCHARGED FLOW IS   │
│   ENCOUNTERED, SET THE DAMAGE RESULTING   │
│   FROM THIS STORM EQUAL TO ZERO. OTHERWISE,   │
│   USE THE SURCHARGED FLOW-DAMAGE MODEL TO   │
│   EVALUATE THE RESULTING DAMAGE.   │
└──┘

┌──┐
│ .WITH THE PREVIOUSLY DETERMINED 3 POINTS ON THE   │
│ DAMAGE-PROBABILITY CURVE OF THE ORIGINAL   │
│ BALANCED SYSTEM, COMPUTE THE COEFFICIENTS A, B   │
│ AND C OF EQUATION 3.92 AND THE AVERAGE ANNUAL   │
│ PROBABLE DAMAGE, d_o.   │
└──┘

(a)

FIGURE 3.23   FLOW CHART OF THE MODEL

FIGURE 3.23 (continued)

$$\text{(a)}$$

.COMPUTE COST GRADIENTS, C_i, AND DAMAGE GRADIENTS, d_i, BY PERTURBING THE ORIGINAL BALANCED SEWER SYSTEM, WITH RESPECT TO EACH DECISION VARIABLE, AND COMPUTING THE CAPITAL RECOVERY COST, c_i, AND THE AVERAGE ANNUAL PROBABLE DAMAGE, $(D_{av})_i$, IN EACH CASE. THEN C_i AND $(D_{av})_i$ ARE USED TO COMPUTE c_i AND d_i.

— COMPUTER

--- MANUAL

.PRINT/
BALANCED DESIGN, COST GRADIENTS
DAMAGE GRADIENTS, C_o, d_o AND A.

.PREPARE INPUT DATA TO SUBROUTINE "SIMPLX"

.OPTIMIZATION

.FINAL DESIGN
(COMMERCIAL PIPE SIZES)

```
                    ┌─────────────┐
                    │    START    │
                    └─────────────┘
                           │
                           ▼
              ╱─────────────────────────╲
             ╱  COMMON, DIMENSION         ╲
             ╲───────────────────────────╱
                           │
                           ▼
                   ┌───────────────┐
                   │  CALL INPUTD  │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  CALL ELVBAS  │
                   └───────────────┘
                           │
                           ▼
                         ( ◯ )        I=1,NSTORM
                           │
                           ▼
         ┌──────────────────────────────────────────┐
         │  A=AA(I), B=BB(I), C=CC(I), R=RR(I)       │
         └──────────────────────────────────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  CALL RAINF   │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  CALL INFHYD  │
                   └───────────────┘
                           │
                           ▼
                        ╱────╲         Yes    ┌───────────────┐
                       ╱ I=1  ╲───────────────│  CALL HYDMB   │
                       ╲      ╱               └───────────────┘
                        ╲────╱   No
                           │◄──────────────────────┘
                           ▼
                   ┌───────────────┐
                   │  CALL HYDMOD  │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  DAM(I)=RDAM  │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  CALL ANCOST  │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  PRINT TANC   │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  CINITL=TANC  │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │  CALL DAMPRO  │
                   └───────────────┘
                           │
                           ▼
                   ┌───────────────┐
                   │   PRINT AD    │
                   └───────────────┘
                           │
                           ▼
                          (a)
```

FIGURE 4.1 FLOW CHART FOR THE MAIN PROGRAM

FIGURE 4.1 (continued)

```
                                    ( a )
                                      │
                                      ▼
                            ┌──────────( )────── I=1,5
                            │           │
                            │           ▼
                            │   ┌──────────────────┐
                            │   │ D(I,1)=AD        │
                            │   │ D(I,3)=0.5A1     │
                            │   └──────────────────┘
                            │           │
                            └───────────┤
                                        ▼
                                ┌──────────────────┐
                                │ CALL STORE       │
                                └──────────────────┘
                                        │
                    ┌───────────────────( )────── NCHNG=1,5
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ CALL CHANGE      │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │   ┌──────────────────────────────────────┐
                    │   │ A=AA(2), B=BB(2), C=CC(2), R=RR(2)    │
                    │   └──────────────────────────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ CALL RAINF       │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ CALL INFHYD      │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ CALL HYDMOD      │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ DAM(2)=TDAM      │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ CALL DP1         │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ D(NCHNG,2)=AD1   │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ CALL ANCOST      │
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    │            │ COSTP(NCHNG)=TANC│
                    │            └──────────────────┘
                    │                    │
                    │                    ▼
                    │            ┌──────────────────┐
                    └────────────│ CALL ORIGNL      │
                                 └──────────────────┘
                                        │
                                        ▼
                                ┌──────────────────┐
                                │ CALL GRAD1       │
                                └──────────────────┘
                                        │
                                        ▼
                                      ( b )
```

FIGURE 4.1 (continued)

```
    (b)
     |
     v
+-----------+
| CALL GRADB|
+-----------+
     |
     v
  ( STOP )
```

FIGURE 4.2   FLOW CHART FOR SUBROUTINE-PRINT1

FIGURE 4.2 (continued)

```
                    ┌─────────────┐
                    │    ENTER    │
                    └──────┬──────┘
                           │
                   ╱───────┴────────────╲
                  ╱  COMMON,DIMENSION     ╲
                 ╱─────────┬───────────────╲
                           │
                          ┌┴┐
                          ( ) ── I=1,220
                          └┬┘
             ┌─────────────┤
             │      ┌──────┴──────┐
             │      │  RAIN(I)=0.0 │
             │      └──────┬──────┘
             └─────────────┤
                           │
                    ┌──────┴──────┐
                    │ SPAN=180/T  │
                    │ TB=180*R    │
                    │ K=BEFPK/TB  │
                    └──────┬──────┘
                          ┌┴┐
                          ( ) ── I=1,K
                          └┬┘
      ┌────────────────────┤
      │   ┌─────────────────┴──────────────────────────┐
      │   │ RAIN(I)=A*((1.0-B)*((BEFPK/R)**B)+C)/       │
      │   │        (((BEFPK/R)**B)+C)**2)               │
      │   │ BEFPK=BEFPK-T                               │
      │   └─────────────────┬──────────────────────────┘
      └────────────────────┤
                           │
                    ┌──────┴──────┐
                    │ J=K+1       │
                    │ TA=0.0      │
                    │ SPAN1=SPAN+1│
                    └──────┬──────┘
                          ┌┴┐
                          ( ) ── I=J,SPAN1
                          └┬┘
      ┌────────────────────┤
      │   ┌─────────────────┴──────────────────────────┐
      │   │ RAIN(I)=A*((1-B)*((TA/(1-R))**B)+C)/        │
      │   │        ((((TA/(1-R))**B)+C**2)              │
      │   │ TA=TA+T                                     │
      │   └─────────────────┬──────────────────────────┘
      └────────────────────┤
                    ┌───────┴──────┐
                    │    RETURN     │
                    └──────────────┘
```

FIGURE 4.3 FLOW CHART FOR SUBROUTINE-RAINF

FIGURE 4.4   FLOW CHART FOR SUBROUTINE-INFHYD

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```
        ┌──────────┐
        │  ENTER   │
        └────┬─────┘
             │
     ╱───────▼────────╲
    ╱ COMMON,DIMENSION ╲
    ───────────┬────────
               │
    ┌──────────▼─────────────────┐
    │ M=0                        │
    │ (I=1,NPIPE),PMAX(I)=0.0    │
    └──────────┬─────────────────┘
               │
    ┌──────────▼─────────┐
    │   CALL KENERG      │
    └──────────┬─────────┘
               │
    ┌──────────▼─────────┐
    │   CALL VOLSYS      │
    └──────────┬─────────┘
               │
             (  )  ITT=1,160
               │
    ┌──────────▼─────────┐
    │   IT=ITT           │
    └──────────┬─────────┘
               │
          ╱────▼────╲   Yes
          ╲  M≠o    ╱──────►
           ╲───┬───╱
               │ No
             (  )  IPK=1,NPIPE
               │
    ┌──────────▼─────────┐
    │   IP=IPK           │
    └──────────┬─────────┘
               │
    ┌──────────▼─────────┐
    │   CALL ADDHYD      │
    └──────────┬─────────┘
               │
    ┌──────────▼─────────┐
    │   CALL MUSKRT      │
    └──────────┬─────────┘
               │
    ┌──────────▼─────────┐
    │   CALL SUMP1       │
    └──────────┬─────────┘
               │
    ┌──────────▼─────────┐
    │   CALL SWITCH      │
    └──────────┬─────────┘
               │
┌──────────────▼──────────────────────────┐
│ (I=1,NPIPE),(J=1,220)QBSMT(I,J)=0.0      │
└──────────────┬──────────────────────────┘
               │
          ╱────▼────╲   Yes
          ╲  M=o    ╱──────►
           ╲───┬───╱
               │ No
    ┌──────────▼─────────┐
    │   CALL PRESML      │
    └──────────┬─────────┘
               │
             (  a  )
```

FIGURE 4.5   <u>FLOW CHART FOR SUBROUTINE-HYDMOD</u>

Figure 4.5 (continued)

FIGURE 4.6   FLOW CHART FOR SUBROUTINE-SUMP1

```
                    ┌─────────────┐
                   (    ENTER      )
                    └──────┬──────┘
                           │
                  ╱────────────────────╱
                 ╱ COMMON,DIMENSION   ╱
                 ────────────────────╱
                           │
                  ┌──────────────────┐
                  │ USEL(KM)=H(KM)    │
                  └─────────┬────────┘
                            │
                       ╱───────────╲          Yes
                      ╱ KM=NPIPE     ╲──────────────────┐
                      ╲              ╱                   │
                       ╲───────────╱                    │
                            │                            │
```

┌─────────────────────────────┐   ┌─────────────────────────────────┐
│ KM2=KM+1   │   │ SEWHL=USEL(KM)-ESUMP(ITS)   │
│ SEWHL=USEL(KM)-USEL(KM2)   │   │ EMP(KM,ITS)=(ESUMP(ITS)+   │
│ EMP(KM,ITS)=(USEL(KM)+   │   │   USEL(KM))/2.0   │
│   USEL(KM2))/2   │   └─────────────────────────────────┘
└─────────────────────────────┘

┌──┐
│ SE(KM,ITS)=SEWHL/ALNTH(KM)   │
│ QAV(KM,ITS)=SQRT(ABS(SE(KM,ITS)/   │
│   KES(KM)))*SE(KM,ITS)   │
│   /ABS(SE(KM,ITS))   │
│   │
│ KM1=KM-1   │
│ SUMQST=0.0   │
│ SUMIHD=QINLET(ITS)   │
└──┘

```
                    (○)  IM1=1,KM
                     │
       ┌─────────────────────────────────────┐
       │ SUMIHD=SUMIHD+INFHD(IM1,ITS)         │
       └─────────────────────────────────────┘

                  ╱───────────╲     Yes
                 ╱ KM1=0        ╲───────────────┐
                 ╲              ╱                │
                  ╲───────────╱                 │
                       │ No                      │
            ┌──────────────────┐      ┌──────────────────┐
            │ SUMQB=0.0        │      │ SUMQB=0.0        │
            └──────────────────┘      └──────────────────┘
                    (○)  IM2=1,KM1
       ┌─────────────────────────────────────┐
       │ SUMQST=SUMQST+QSTR(IM2)              │
       │ SUMQB=SUMQB+QBSMT(IM2,ITS)           │
       └─────────────────────────────────────┘
```

┌──┐
│ QIN=SUMIHD-SUMQB-SUMQST   │
│ QOT=QAV(KM,ITS)+QBSMT(KM,ITS)/2.0   │
│ QSTR(KM)=QIN-QOT   │
└──┘

FIGURE 4.7   FLOW CHART FOR SUBROUTINE-STRMOD

FIGURE 4.8   FLOW CHART FOR SUBROUTINE-PRESML

FIGURE 4.8 (continued)

SUMQST=SUMQST+QSTR(IM1)
SUMIHD=SUMIHD+INFHD(IM1,ITS)

KM1=KM-1

KM1=0   Yes / No

SUMQB=0.0

SUMQB=0.0

IM2=1,KM1

SUMQB=SUMQB+QBSMT(IM2,ITS)

QAV(KM,ITS)=SUMIHD-SUMQB-QBSMT(KM,ITS)/2.0-SUMQST
SE(KM,ITS)=KES(KM)*QAV(KM,ITS)*ABS(QAV(KM,ITS))

IM>1   Yes / No

USEL(KM)=ESUMP(ITS)+SE(KM,ITS)*ALNTH(KM)
EMP(KM,ITS)=(ESUMP(ITS)+USEL(KM))/2.0

KM2=KM+1
USEL(KM)=USEL(KM2)+SE(KM,ITS)
         *ALNTH(KM)
EMP(KM,ITS)=(USELCKM2)+
            USEL(KM))/2.0

USEL(KM)<H(KM)   Yes / No

CALL STRMOD

VOLST(KM)> .01   Yes

QSTR(KM)=0.0
DVOLST(KM)=0.0

QSTR(KM)=(-1.0)*CB(KM)*
         SQRT(ABS(VOLST(KM)))

FIGURE 4.8 (continued)

k j i h   g

CALL FRATIO

CALL QHOUSS

```
XDFLOD(KM)=DFLOOD*12.
X=QBSMT(KM,ITS)
QBSMT(KM,ITS)=QHOUS*(1.-(.9**ITR)**ITR)
QBSMT(KM,ITS)=(QBSMT(KM,ITS)+X)/2.0
QSTR(KM)=QSTR(KM)*(1.-(.9**ITR)**ITR)
QSTR(KM)=(QSTR(KM)+QQ)/2.
DVOLST(KM)=QSTR(KM)*T*60.
```

```
QOUT(NPIPE,ITS)=QAV(NPIPE,ITS)-QBSMT(NPIPE,ITS)/2.0
Y=ESUMP(ITS)
```

CALL SUMP1

ESUMP(ITS)=(ESUMP(ITS)+Y)/2.0

KI=1,NPIPE

DVOLST(KI) ≤ 0.0   Yes

No

TPEAK(KI)=TPEAK(KI)+T

VOLST(KI)=VOLST(KI)+DVOLST(KI)

I=1,NPIPE

DMAX(I)≥XDFLOD(I)   Yes

No

DMAX(I)=XDFLOD(I)

n   m   l

FIGURE 4.8 (continued)

```
                    ╭─────────────╮
                    │    ENTER     │
                    ╰──────┬──────╯
                           │
                   ╱───────────────────╲
                  ╱ COMMON,DIMENSION     ╲
                  ╲───────────────────────╱
                           │
       ┌───────────────────────────────────────────┐
       │ APIPE=DHCON(I)**2.0*0.7854                 │
       │ A=8.02*APIPE*NHOUS(I)/SQRT(2.0+            │
       │    0.03*ALPIPE(I)/DHCON(I))                │
       │ SQBSMT=0.0                                 │
       └───────────────────────┬───────────────────┘
                               │
                              ╭───╮
                              │   │   M=IT,ITS
                              ╰─┬─╯
                               │
              ┌──────────────────────────────────┐
              │ SQBSMT=SQBSMT+QBSMT(I,M)          │
              └──────────────┬───────────────────┘
                             │
              ┌──────────────────────────────────┐
              │ FLVOL=SQBSMT*T*60                 │
              │ DFLOOD=FLVOL/ABASE(I)/43560.      │
              └──────────────┬───────────────────┘
                             │
              -ve       ╱─────────╲       +ve
             ┌─────────╱  DFLOOD    ╲──────────┐
             │         ╲───────────╱           │
             │              │                   │
       ┌───────────┐        │                   │
       │ DFLOOD=0.0│        o                   │
       └─────┬─────┘        │                   │
             │              │                   │
             └──────────────┴───────────────────┘
                             │
       ┌───────────────────────────────────────────────┐
       │ ENBAS=BMP(I)+DFLOOD                            │
       │ USEBL=ENBAS+SBASE(I)*ALNTH(I)/2.0             │
       │ DSEBL=ENBAS-SBASE(I)*ALNTH(I)/2.0             │
       │ USEL1=USEL(I)                                 │
       │ USHL=USEL1-USEBL                              │
       │ DSEL=USEL1-SE(I,ITS)*ALNTH                    │
       │ DSHL=DSEL-DSEBL                               │
       └───────────────────────┬───────────────────────┘
                               │
                              ╭───╮
                              │ a │
                              ╰───╯
```

FIGURE 4.9   FLOW CHART FOR SUBROUTINE-QHOUSS

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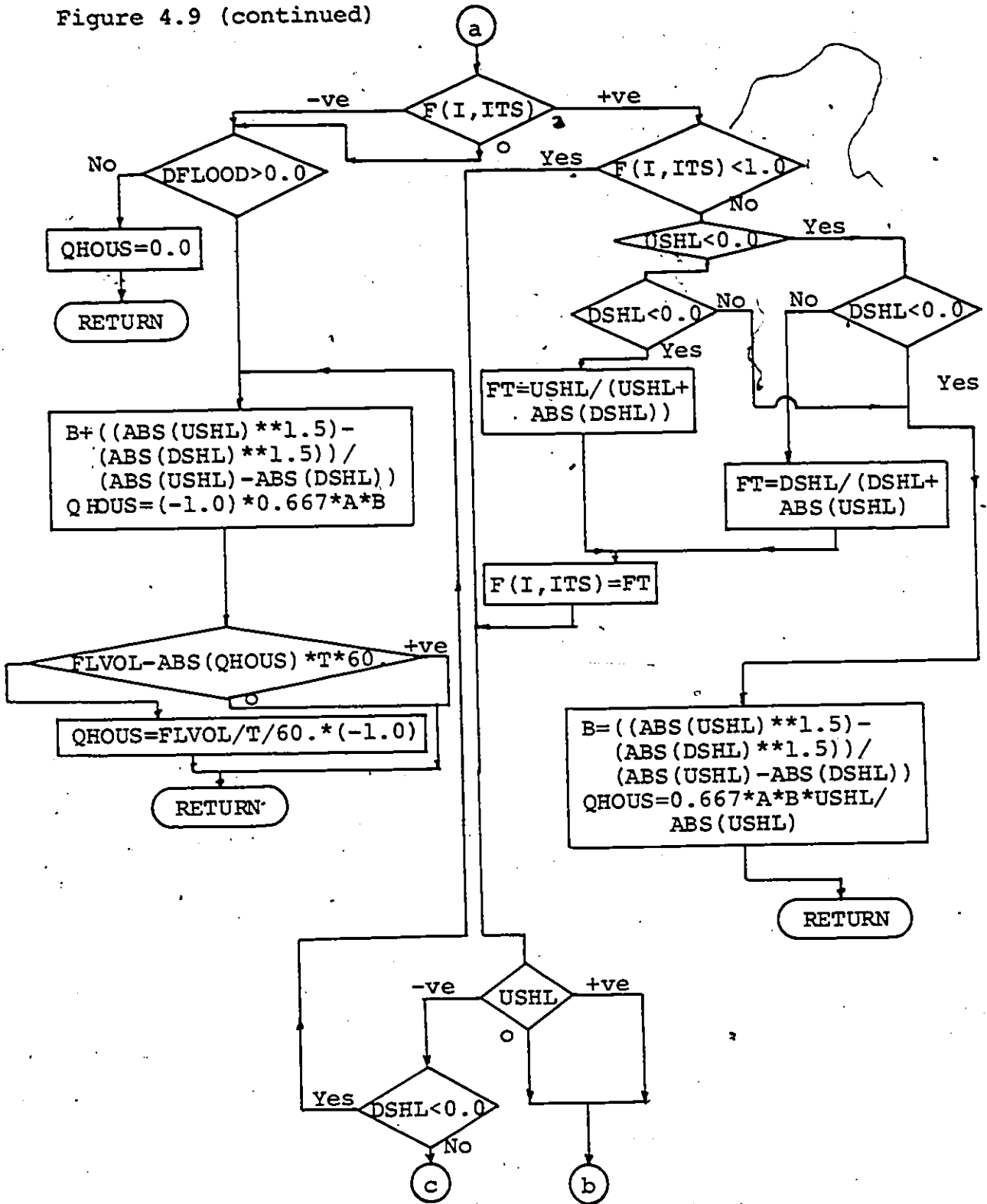
Figure 4.9 (continued)

FIGURE 4.9 (continued)

```
                    ( c )                                ( b )

                                        ┌────────────────────────────────┐
                                        │QHOUSP=0.667*F(I,ITS)*SQRT       │
                                        │        (ABS(USHL))*A            │
                                        │QHOUSN=0.667*(1-F(I,ITS))*       │
                                        │        SQRT(ABS(DSHL))*A        │
                                        └────────────────────────────────┘

┌────────────────────────────┐
│ QHOUSP=0.667*F(I,ITS)*      │
│         SQRT(DSHL)*A        │
│ QHOUSN=0.667*(1.0-F(I,ITS)* │
│         SQRT(ABS(USHL))*A   │
└────────────────────────────┘

        ┌──────────────────────────────────────────────────┐
        │ VOLAV=DFLOOD*(1.0-F(I,ITS))*ABASE(I)*43560.        │
        └──────────────────────────────────────────────────┘

    -ve              VOLAV-QHOUSN*T*60              +ve

┌──────────────────────┐
│ QHOUSN=VOLAV/T/60.    │                  o
└──────────────────────┘

                ┌──────────────────────────┐
                │ QHOUS=QHOUSP-QHOUSN        │
                └──────────────────────────┘

                        (  RETURN  )
```

```
                    ┌──────────┐
                    │  ENTER   │
                    └────┬─────┘
                         │
                ╱────────┴────────────╲
                │ COMMON, DIMENSION    ╱
                ╲────────┬────────────╱
                         │
        ┌────────────────┴──────────────────────┐
        │ DSEL=USEL(IP)-SE(IP,IT)*ALNTH(IP)      │
        │ DSBL=BMP(IP)-SBASE(IP)*ALNTH(IP)/2.0   │
        │ USBL=BMP(IP)+SBASE(IP)*ALNTH(IP)/2.0   │
        └────────────────┬──────────────────────┘
                         │
                   ◇─────┴─────◇    Yes
                  < USEL(IP)>USBL >─────────┐
                   ◇─────┬─────◇            │
                         │                  │
                   ◇─────┴─────◇    Yes     │
                  <  DSEL>DSBL   >──────────┤
                   ◇─────┬─────◇            │
                         │                  │
        ┌────────────────┴──┐    ┌──────────┴──────────────────┐
        │  F(IP,IT)=0.0      │    │ FLOAT=SE(IP,IT)-SBASE(IP)   │
        └─────────┬──────────┘    └──────────┬──────────────────┘
                  │                          │
            ┌─────┴─────┐            ◇───────┴────────◇    No
            │  RETURN   │           <  FLOAT>10^(-6)   >──────┐
            └───────────┘            ◇───────┬────────◇      │
                                        Yes  │              │
                                             │     ┌────────┴──────┐
                                             │     │ F(IP,IT)=1.0  │
                                             │     └────────┬──────┘
                                             │              │
                                             │        ┌─────┴─────┐
                                             │        │  RETURN   │
                                             │        └───────────┘
        ┌────────────────────────────────────┴────┐
        │ C=(EMP(IP,IT)-BMP(IP))/(SE(IP,IT)        │
        │   -SBASE(IP))/ALNTH(IP)                  │
        └────────────────┬─────────────────────────┘
                         │
                   ◇─────┴──────────────◇   Yes
                  < SE(IP,IT)≥SBASE(IP)  >──────────┐
                   ◇─────┬──────────────◇           │
                      No │                          │
        ┌────────────────┴──┐         ┌─────────────┴─────┐
        │ F(IP,IT)=0.5-C     │         │ F(IP,IT)=0.5+C    │
        └────────────────┬──┘         └─────────────┬─────┘
                         └───────────┬──────────────┘
                                     │
                             ◇───────┴───────◇   Yes    ┌──────────────┐
                            < F(IP,IT)>1.0    >────────│ F(IP,IT)=1.0 │
                             ◇───────┬───────◇          └──────┬───────┘
                                  No │                         │
                                     │◄────────────────────────┘
                               ┌─────┴─────┐
                               │  RETURN   │
                               └───────────┘
```

FIGURE 4.10 <u>FLOW CHART FOR SUBROUTINE-FRATIO</u>

FIGURE 4.11   FLOW CHART FOR SUBROUTINE-HYDMB
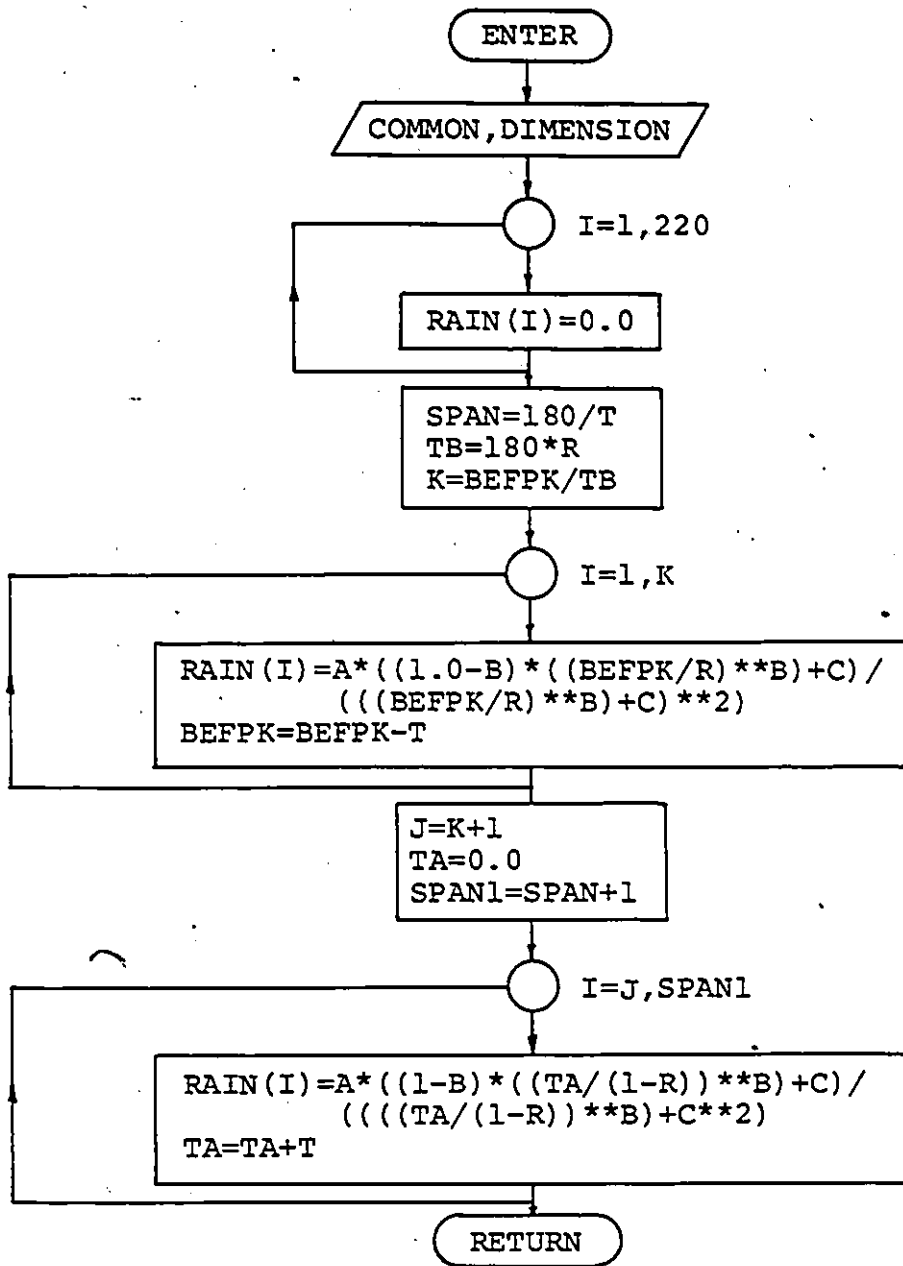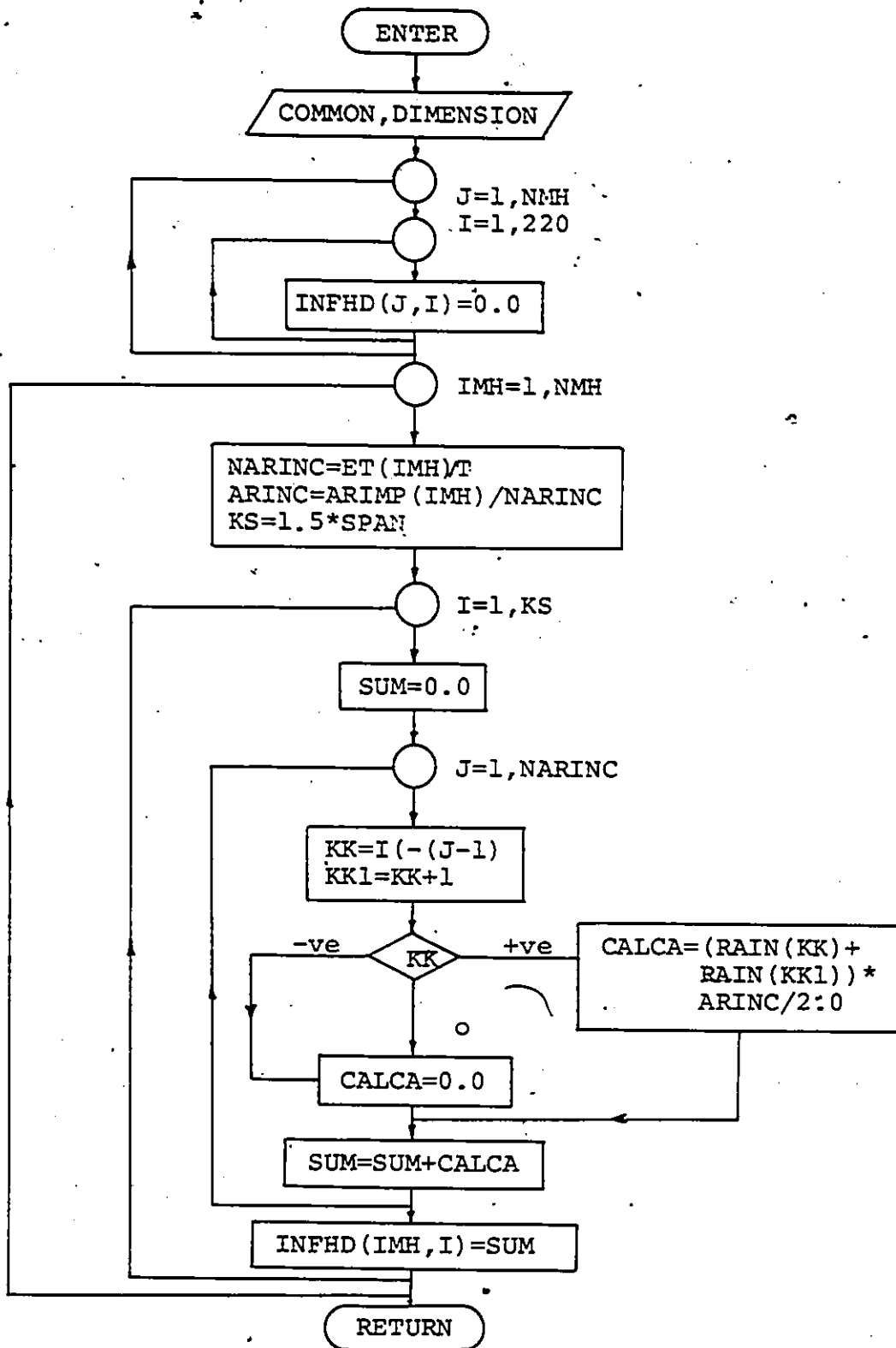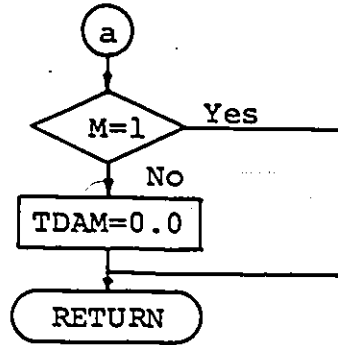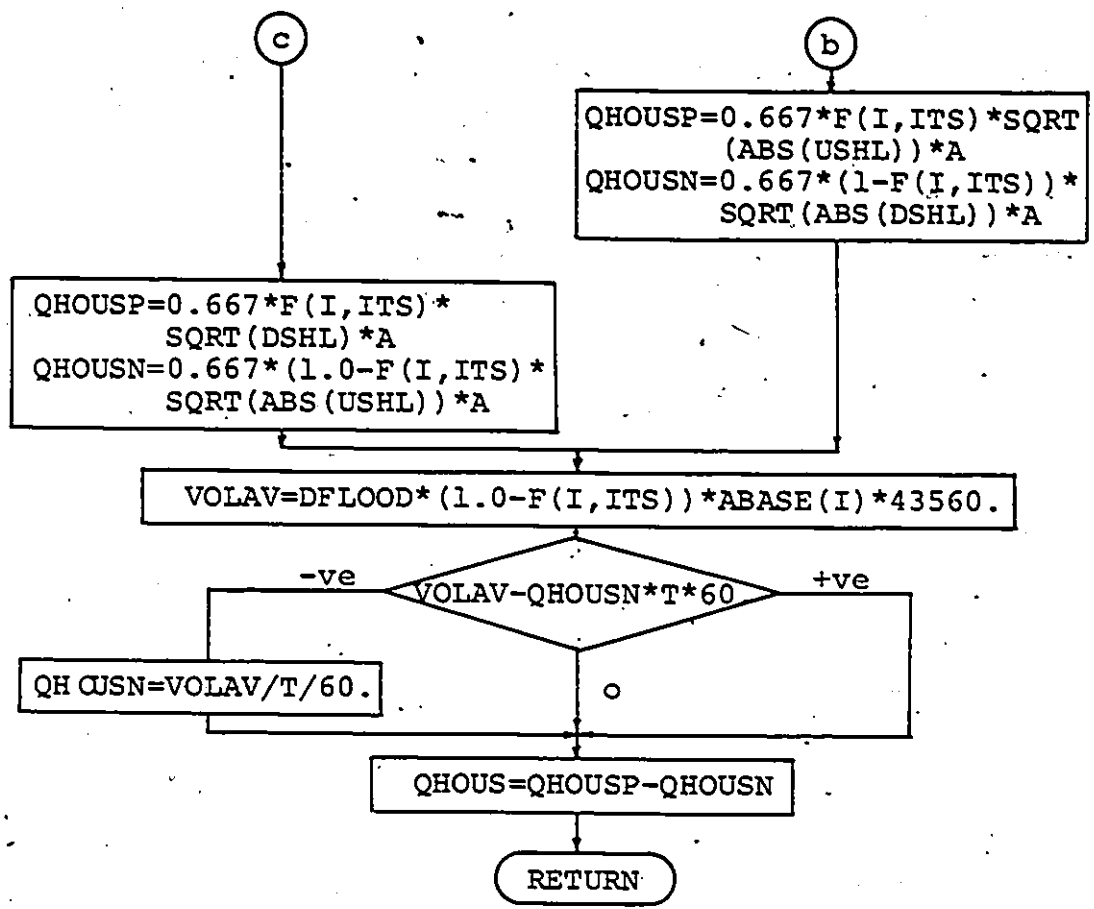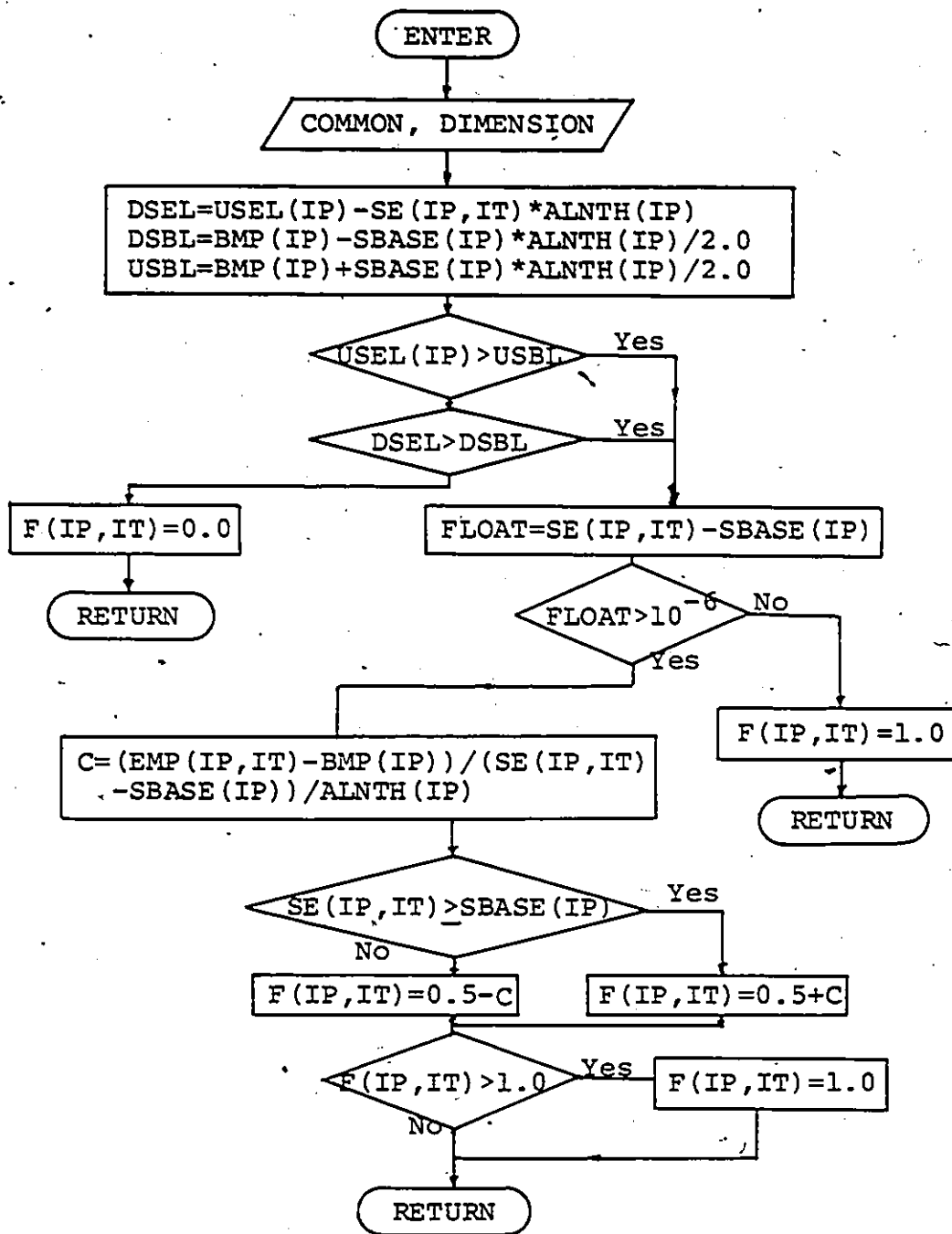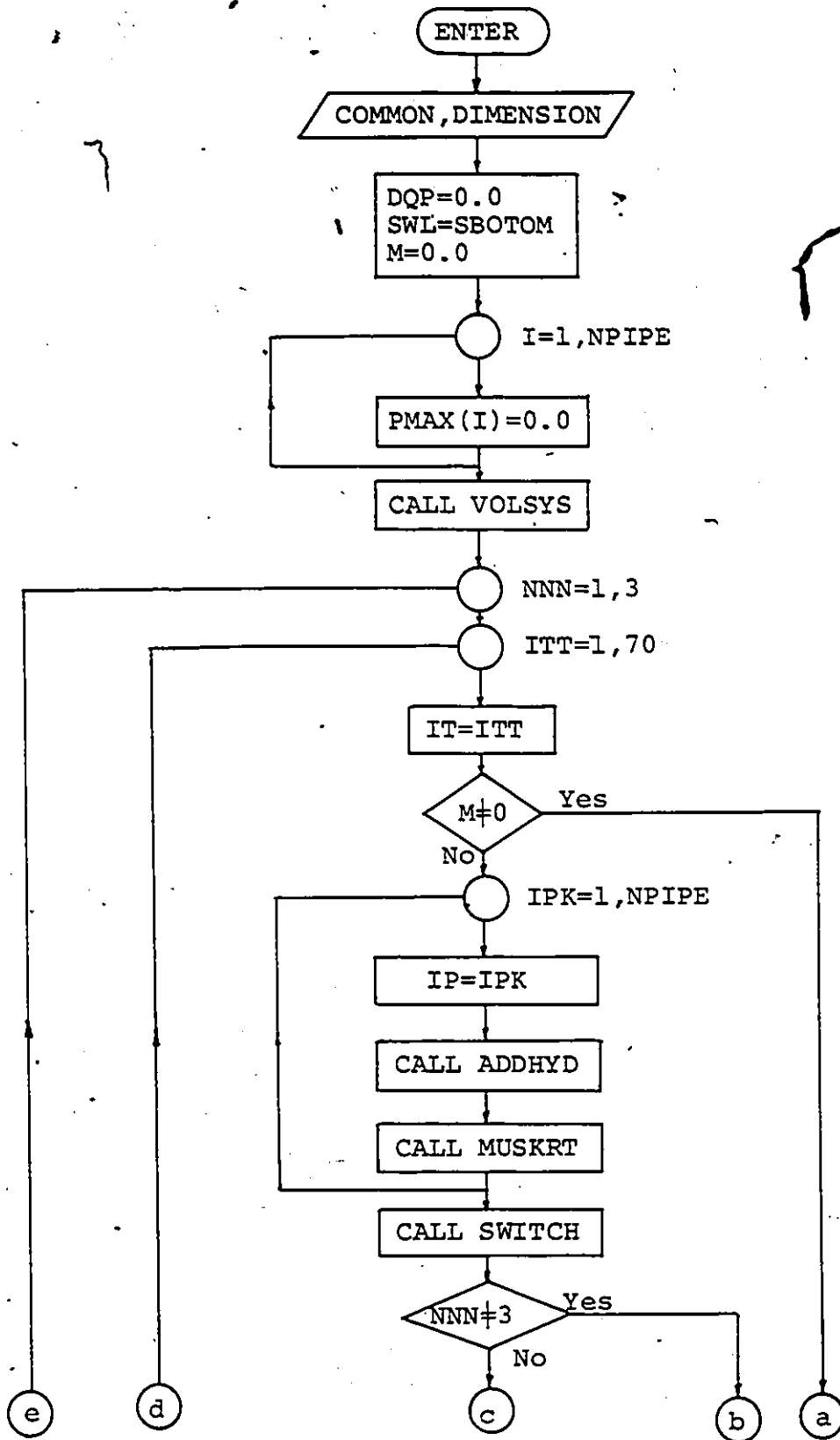
FIGURE 4.11 (continued)

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FIGURE 4.11 (continued)
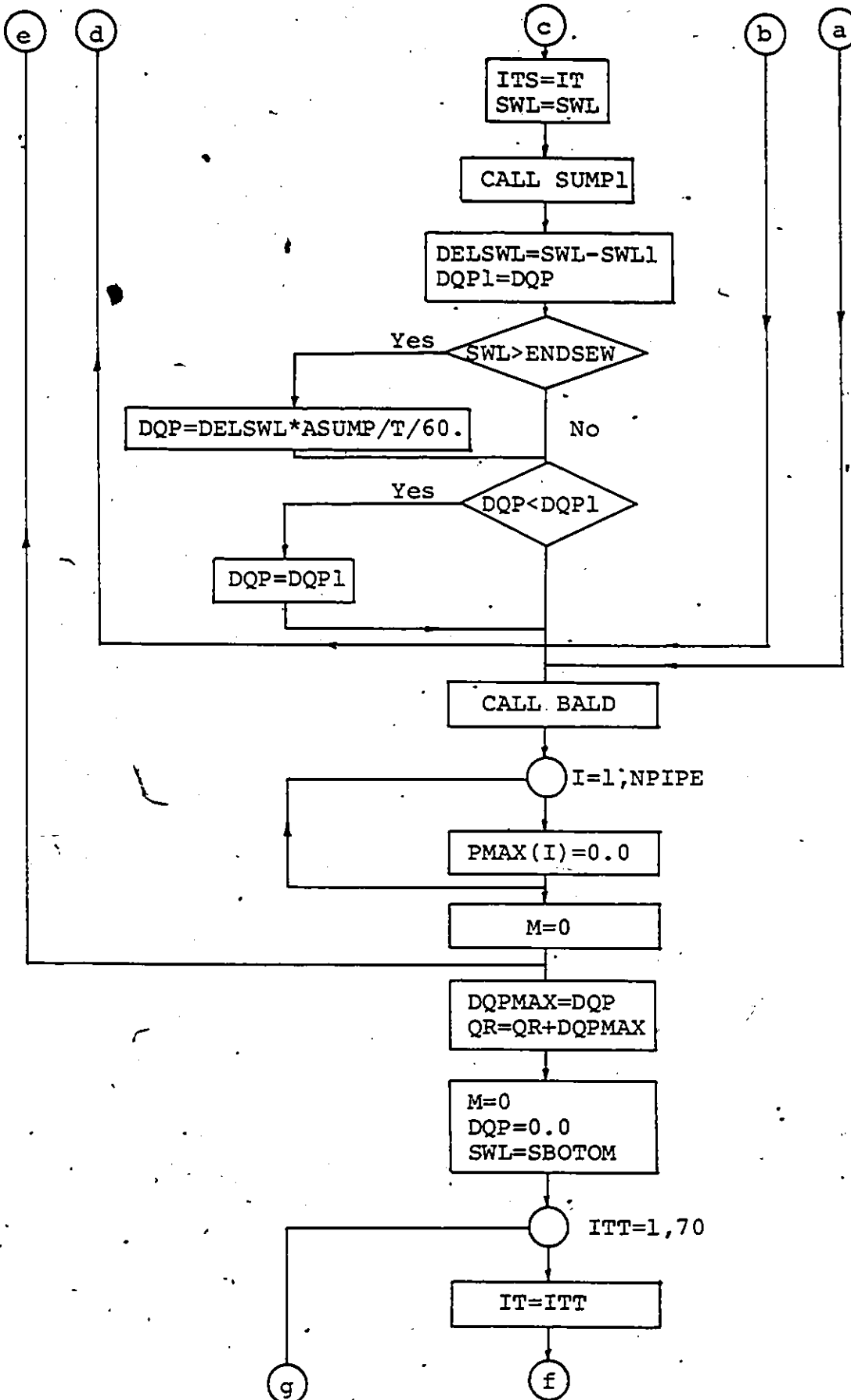
Figure 4.11 (continued)

```
        ( h )
          |
          v
+-----------------------------------+
| SBOTOM=ENDSEW-DIAM(NPIPE)-3.0     |
+-----------------------------------+
          |
          v
     ( RETURN )
```
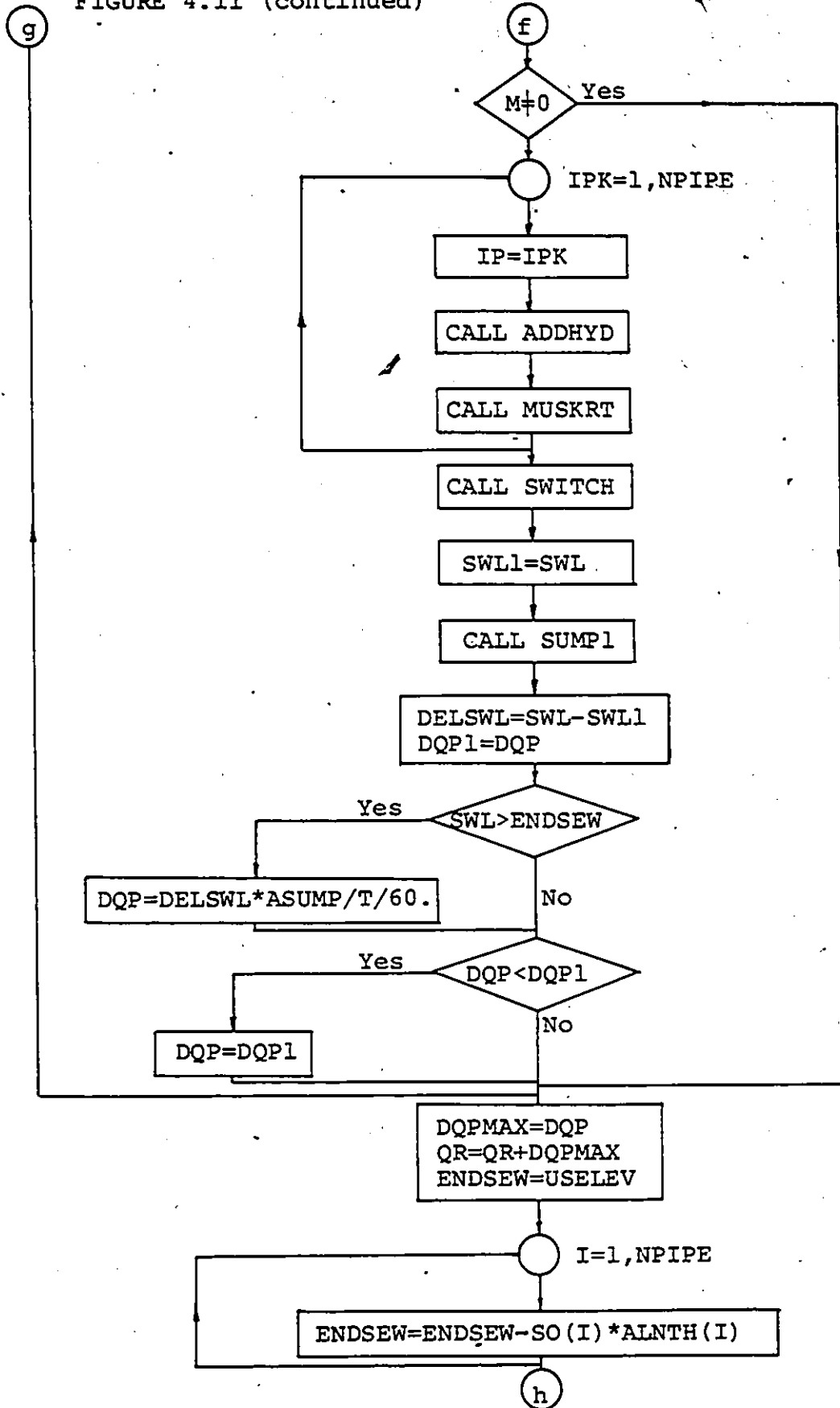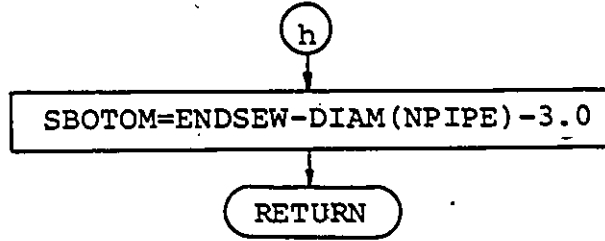
```
$JOB    WATFIV   XXXXXXXXX NAME-IMAM
1       CALL    SIMPLX(1)
2       STOP
3       END

$ENTRY
```

SIMPLEX METHOD FOR SOLVING LP PROBLEMS

FOLLCWING IS THE LISTING OF DATA AS IT IS READ IN, EITHER FRUM CARDS OR
PARTLY FROM PREVIOUS RUN AND PARTLY FROM CARDS.

STORM SEWER DESIGN (WSSC),SENSTIVITY ANALYSIS
ROW ID
```
        = CBJF
        < R1
        < R2
        < R3
        < R4
        < R5
        < R6
        < R7
        < R8
        < R9
        < R10
        < R11
        < R12
        < R13
        < R14
        < R15
        < R16
        < R17
        < R18
        < R19
        < R20
        < R21
        < R22
        < R23
        < R24
        < R25
        < R26
        < R27
        < R28
        < R29
```

FIGURE 4.12   A TYPICAL INPUT DATA FOR
              THE OPTIMIZATION PROGRAM

FIGURE 4.12 (continued)

MATRIX			Explanation
X1	OBJF	3.314000	$-a_1$
X1	R1	0.001000	1.00
X1	R6	-0.001000	-1.00
X1	R11	-13.487000	d_1
X1	R12	-13.487000	d_1
X1	R17	13.487000	$-d_1$
X1	R18	13.487000	$-d_1$
X1	R23	-10.173000	$-c_1$
X1	R24	-10.173000	$-c_1$
X1	R29	3.314000	$-a_1$
XX1	OBJF	-3.314000	a_1
XX1	R1	-0.001000	-1.00
XX1	R6	0.001000	1.00
XX1	R11	13.487000	$-d_1$
XX1	R12	13.487000	$-d_1$
XX1	R17	-13.487000	d_1
XX1	R18	-13.487000	d_1
XX1	R23	10.173000	c_1
XX1	R24	10.173000	c_1
XX1	R29	-3.314000	a_1
X2	OBJF	0.497000	$-a_2$
X2	R2	0.001000	1.00
X2	R7	-0.001000	-1.00
X2	R11	-5.142000	d_2
X2	R13	-5.142000	d_2
X2	R17	5.142000	$-d_2$
X2	R19	5.142000	$-d_2$
X2	R23	-4.645000	$-c_2$
X2	R25	-4.645000	$-c_2$
X2	R29	0.497000	$-a_2$
XX2	OBJF	-0.497000	a_2
XX2	R2	-0.001000	-1.00
XX2	R7	0.001000	1.00
XX2	R11	5.142000	$-d_2$
XX2	R13	5.142000	$-d_2$
XX2	R17	-5.142000	d_2
XX2	R19	-5.142000	d_2
XX2	R23	4.645000	c_2
XX2	R25	4.645000	c_2
XX2	R29	-0.497000	a_2
X3	OBJF	-7.095000	$-a_3$
X3	R3	0.001000	1.00
X3	R8	-0.001000	-1.00
X3	R11	-1.217000	d_3
X3	R14	-1.217000	d_3
X3	R17	1.217000	$-d_3$
X3	R20	1.217000	$-d_3$
X3	R23	-8.312000	$-c_3$
X3	R26	-8.312000	$-c_3$
X3	R29	-7.095000	$-a_3$
XX3	OBJF	7.095000	a_3
XX3	R3	-0.001000	-1.00
XX3	R8	0.001000	1.00
XX3	R11	1.217000	$-d_3$
XX3	R14	1.217000	$-d_3$
XX3	R17	-1.217000	d_3
XX3	R20	-1.217000	d_3
XX3	R23	8.312000	c_3
XX3	R26	8.312000	c_3
XX3	R29	7.095000	a_3
X4	OBJF	6.653000	$-a_4$
X4	R4	0.001000	1.00
X4	R9	-0.001000	-1.00
X4	R11	-22.315000	d_4

FIGURE 4.12 (continued)

			<u>Explanation</u>
X4	R15	-22.315000	d_4
X4	R17	22.315000	$-d_4$
X4	R21	22.315000	$-d_4$
X4	R23	-15.661990	$-c_4$
X4	R27	-15.661990	$-c_4$
X4	R29	6.653000	$-a_4$
XX4	OBJF	-6.653000	a_4
XX4	Q4	-0.001000	-1.00
XX4	R9	0.001000	1.00
XX4	R11	22.315000	$-d_4$
XX4	R15	22.315000	$-d_4$
XX4	R17	-22.315000	d_4
XX4	R21	-22.315000	d_4
XX4	Q23	15.661990	c_4
XX4	R27	15.661990	c_4
XX4	R29	-6.653000	a_4
X5	OBJF	45.458950	$-a_5$
X5	R5	0.001000	1.00
X5	R10	-0.001000	-1.00
X5	R11	-67.436000	d_5
X5	R16	-67.436000	d_5
X5	R17	67.436000	$-d_5$
X5	C22	67.436000	$-d_5$
X5	R23	-21.977000	$-c_5$
X5	R28	-21.977000	$-c_5$
X5	R29	45.458950	$-a_5$
XX5	OBJF	-45.458950	a_5
XX5	R5	-0.001000	-1.00
XX5	R10	0.001000	1.00
XX5	R11	67.436000	$-d_5$
XX5	R16	67.436000	$-d_5$
XX5	R17	-67.436000	d_5
XX5	R22	-67.436000	d_5
XX5	R23	21.977000	c_5
XX5	R28	21.977000	c_5
XX5	R29	-45.458950	a_5

FIRST e

R1	0.000500	$(X_{UL})_1$
R2	0.001000	$(X_{UL})_2$
R3	0.001000	$(X_{UL})_3$
R4	0.001000	$(X_{UL})_4$
R5	0.000100	$(X_{UL})_5$
R6	0.000100	$(X_{LL})_1$
R7	0.000500	$(X_{LL})_2$
R8	0.000500	$(X_{LL})_3$
R9	0.000000	$(X_{LL})_4$
R10	0.000100	$(X_{LL})_5$
R11	153.391000	$A/2 - d_o$
R12	153.391000	$A/2 - d_o$
R13	153.391000	$A/2 - d_o$
R14	153.391000	$A/2 - d_o$
R15	153.391000	$A/2 - d_o$
R16	153.391000	$A/2 - d_o$
R17	19.498000	d_o
R18	19.498000	d_o
R19	19.498000	d_o
R20	19.498000	d_o
R21	19.498000	d_o
R22	19.498000	d_o
R23	55.361990	c_o
R24	55.361990	c_o
R25	55.361990	c_o
R26	55.361990	c_o
R27	55.361990	c_o
R28	55.361990	c_o
R29	74.859000	$c_o + d_o$

EOF

STORM SEWER DESIGN (WSSD),SENSTIVITY ANALYSIS

NO. OF ROWS -   29   NO. OF COLUMNS -   10

NO. OF LE(<=) ROWS -   29   NO. OF GE(>=) ROWS -   0   NO. OF E(=) ROWS -   0

NO. OF NON-ZERO RHS'S -   29

NO. OF NON-ZERO MATRIX ELEMENTS -   100
(THE MATRIX INCLUDES THE OBJECTIVE FNCT. VALUES AS A ROW)

THE SLACK VARIABLES TAKE THE NAMES OF CORRESPONDING CONSTRAINT EQUATIONS.

ITERATION   VAR IN   VAR OUT   OBJ FN

SOLUTION FEASIBLE.

1	X5	R5	4.546
2	XX3	R8	8.091
3	X4	R17	12.077
4	XX2	R7	12.635
5	XX1	R6	12.656

FIGURE 4.13   A TYPICAL OUTPUT FROM THE OPTIMIZATION PROGRAM

FIGURE 4.13 (continued)

OPTIMUM VALUE REACHED AT THE END OF ITERATION —   5

OPTIMUM VALUE OF THE OBJECTIVE FUNCTION —   12.666

BASIS VAR	AMOUNT	UNIT PROFIT
R1	0.000600	0.000000
R2	0.001500	0.000000
R3	0.001500	0.000000
R4	0.000226	0.000000
X5	0.100000	45.453950
XX1	0.100000	-3.314000
XX2	0.500000	-0.497000
XX3	0.500000	7.035000
R9	0.000774	0.000000
P10	0.000200	0.000000
R11	172.688900	0.000000
R12	152.042200	0.000000
R13	150.815900	0.000000
R14	152.782500	0.000000
R15	170.673500	0.000000
R16	160.134500	0.000000
X4	0.774483	6.653000
R18	20.846590	0.000000
R19	22.069000	0.000000
R20	20.106490	0.000000
R21	2.219402	0.000000
R22	12.754400	0.000000
R23	62.193330	0.000000
R24	54.344690	0.000000
R25	53.039480	0.000000
R26	51.205990	0.000000
R27	67.491940	0.000000
R28	57.55969C	0.000000
R29	62.193840	0.000000

NON-BASIC VAR.	REDUCED COST
X1	-0.00000
R6	707.01980
X2	-0.00000
R7	1036.03500
X3	-0.00000
R8	7497.63500
R17	0.29814
XX4	-0.00000
R5	25353.61000
XX5	-0.00000

IF ANY OF THE ABOVE REDUCED COSTS IS ZERO,
IT INDICATES THAT MULTIPLE OPTIMA EXIST.

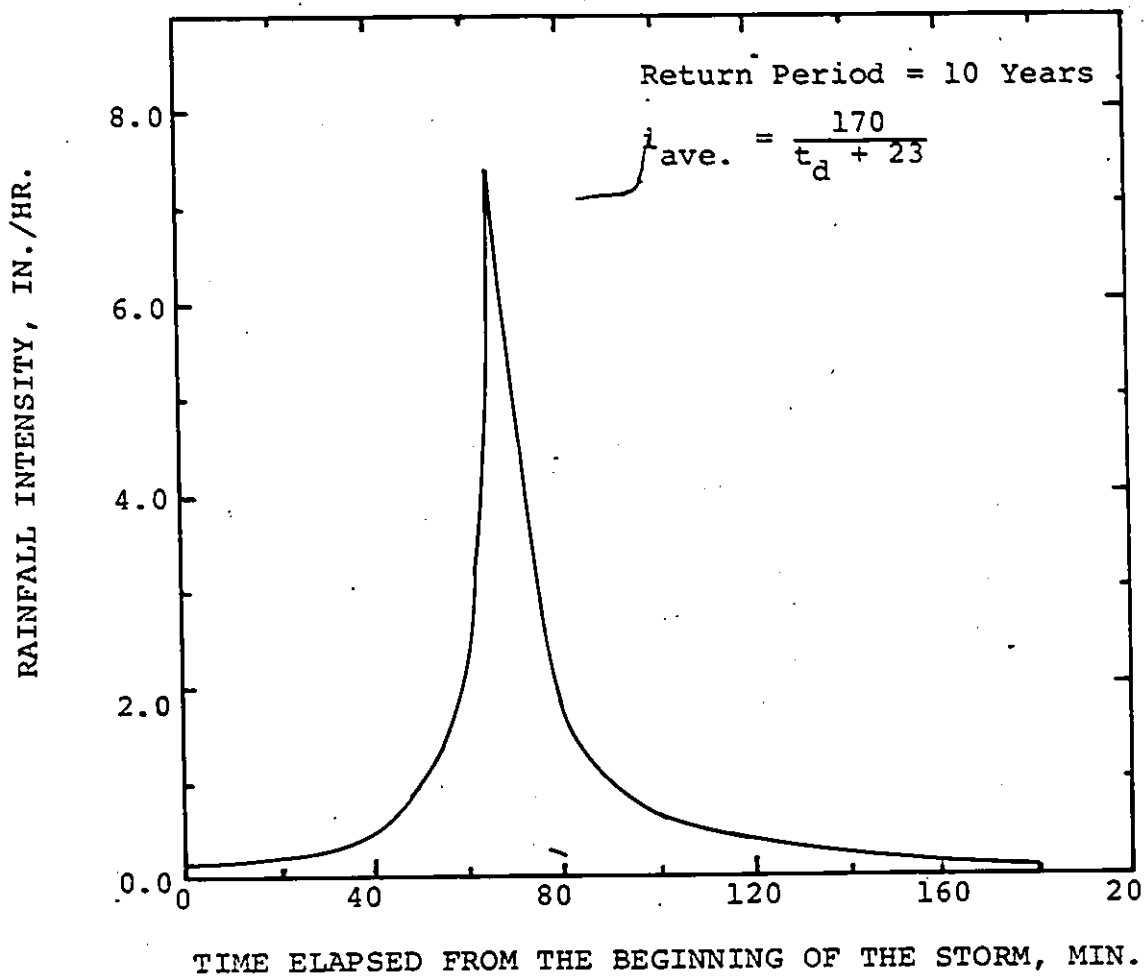FIGURE 5.1   A TYPICAL DESIGN STORM HYETOGRAPH

Subcatchment Impervious
Area = 13 Acres

Time of Concentration
= 25 Minutes

INFLOW RUNOFF, CFS

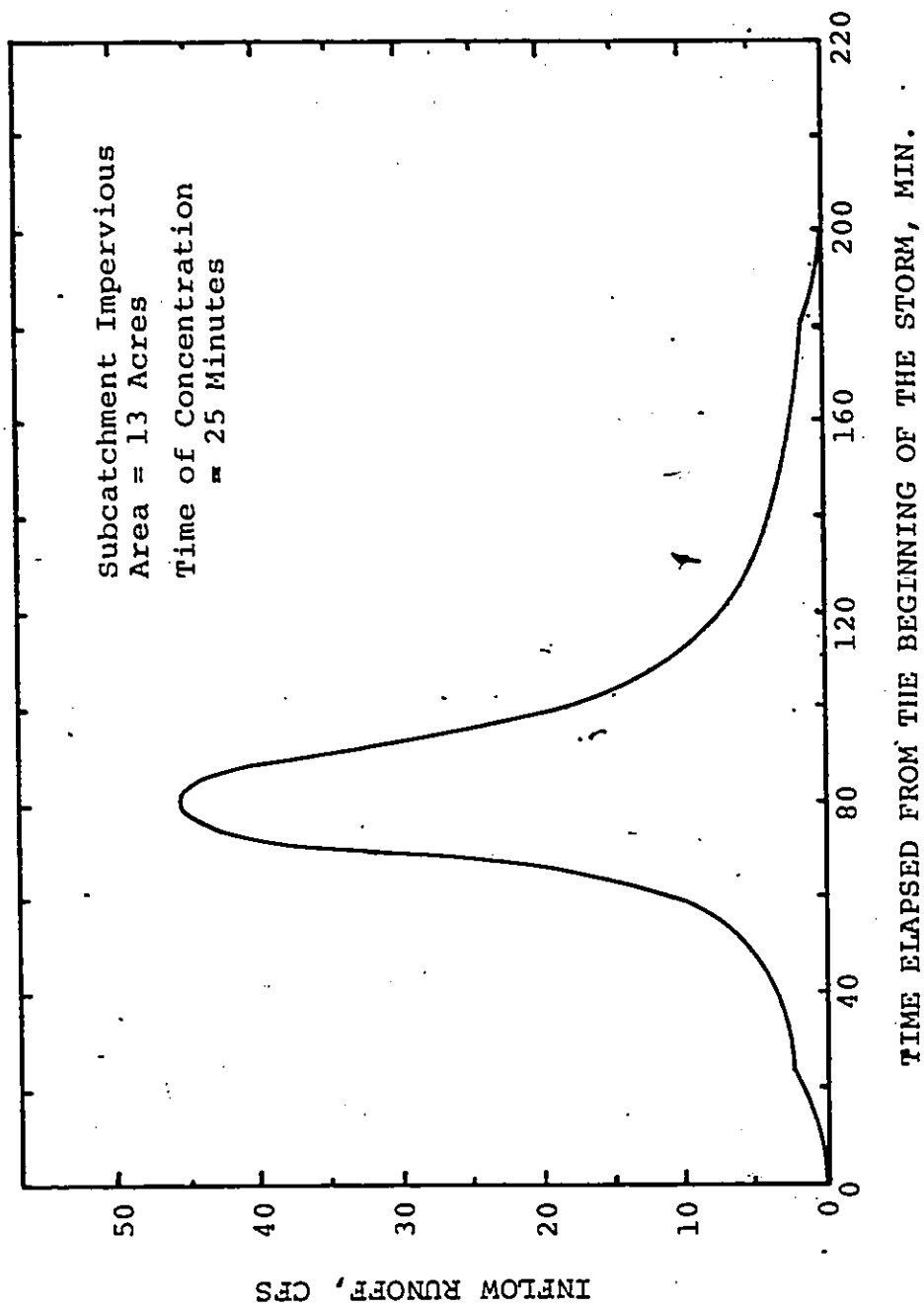TIME ELAPSED FROM THE BEGINNING OF THE STORM, MIN.

FIGURE 5.2   A TYPICAL BASIC INFLOW HYDROGRAPH
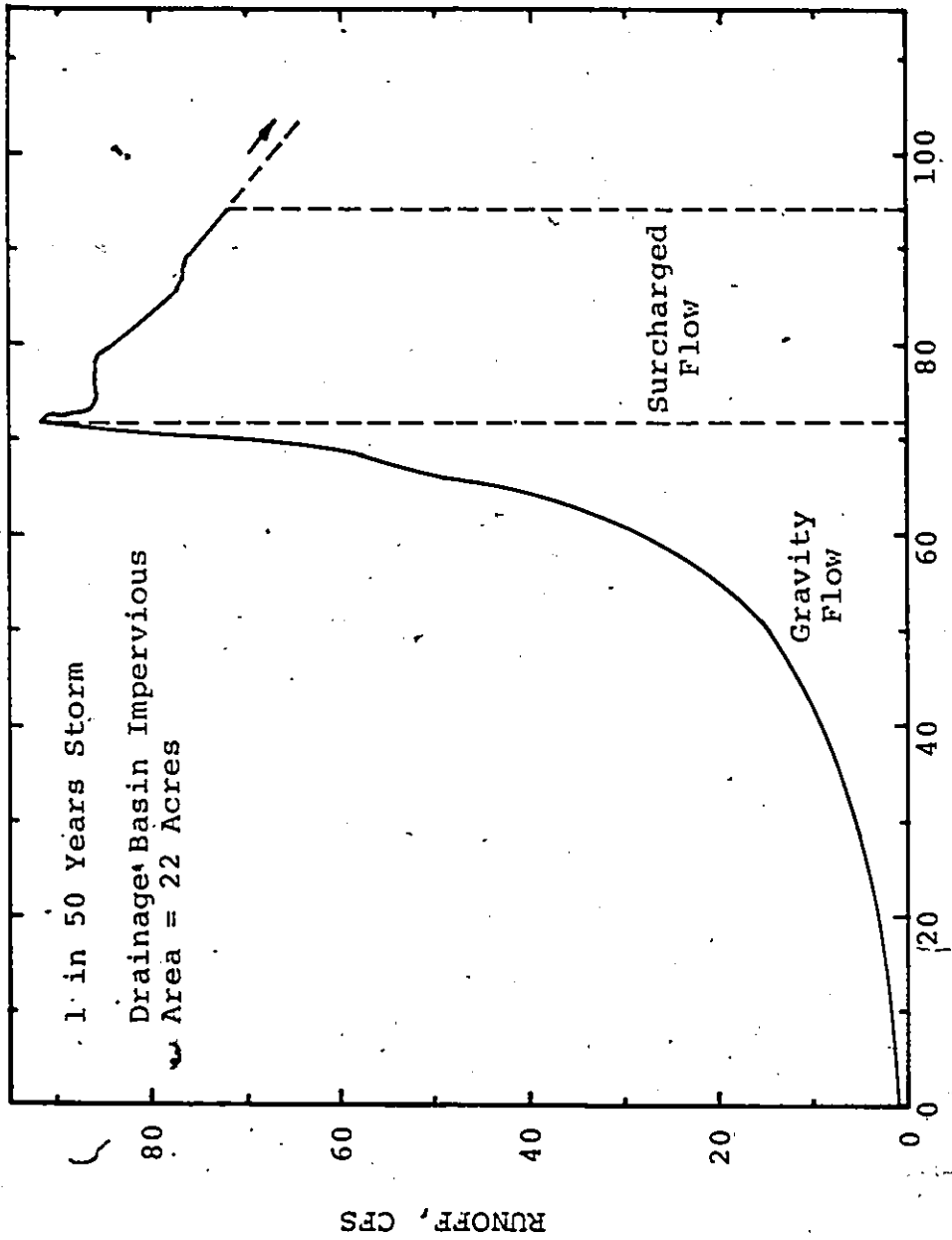
FIGURE 5.3   A TYPICAL OUTFALL HYDROGRAPH TO THE SUMP WELL

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FIGURE 5.4   A TYPICAL PLOT OF SUMP WATER LEVEL VERSUS ELAPSED TIME

FIGURE 5.5 BASEMENT FLOW ALONG A TYPICAL SEWER SECTION VERSUS TIME

FIGURE 5.6   A TYPICAL PLOT OF BASEMENT FLOODING DEPTH VERSUS TIME.
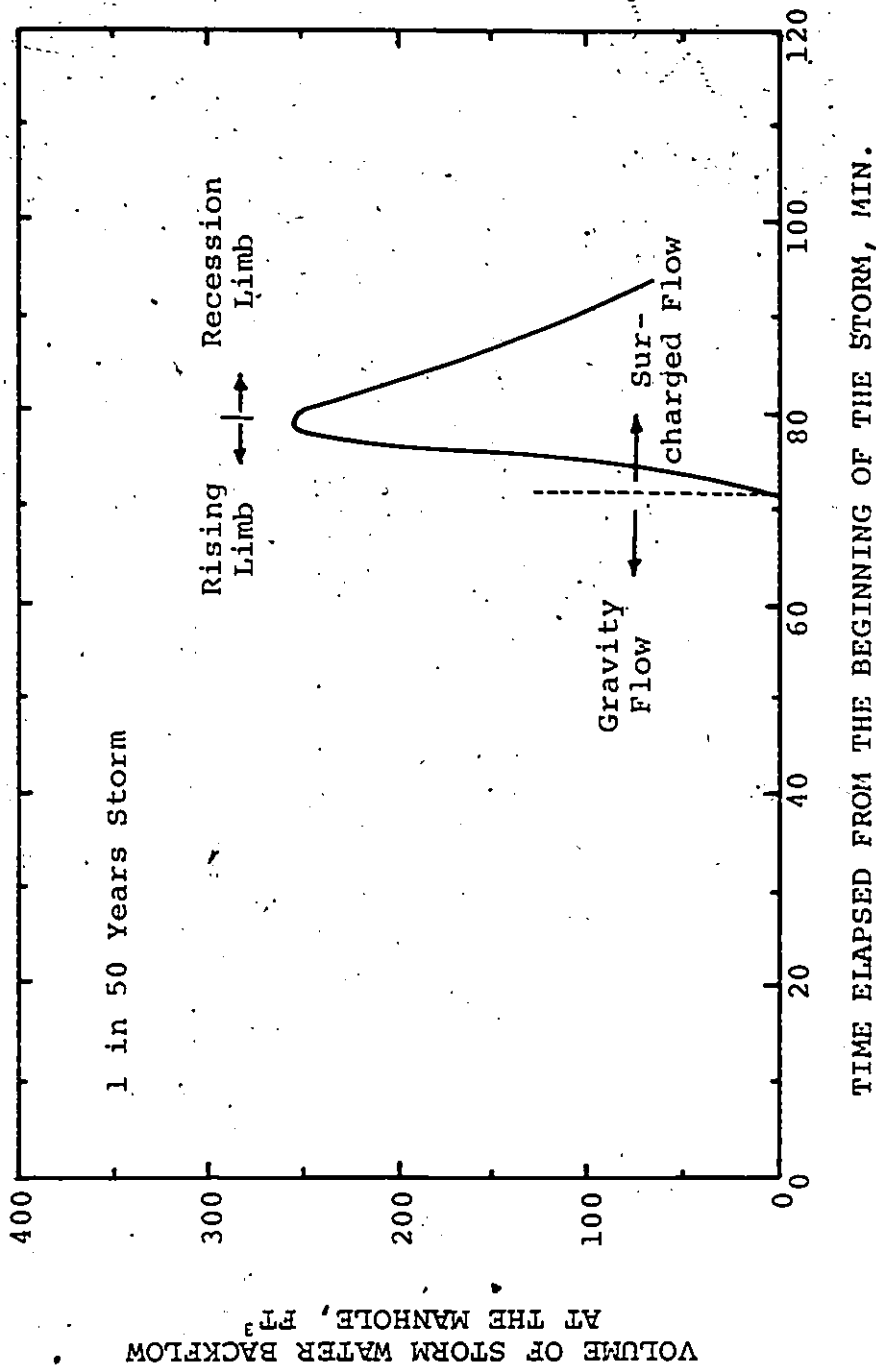
FIGURE 5.7 EXCESS VOLUME OF STORM WATER CONTRIBUTING TO STREET FLOODING VERSUS TIME, AT A TYPICAL MANHOLE
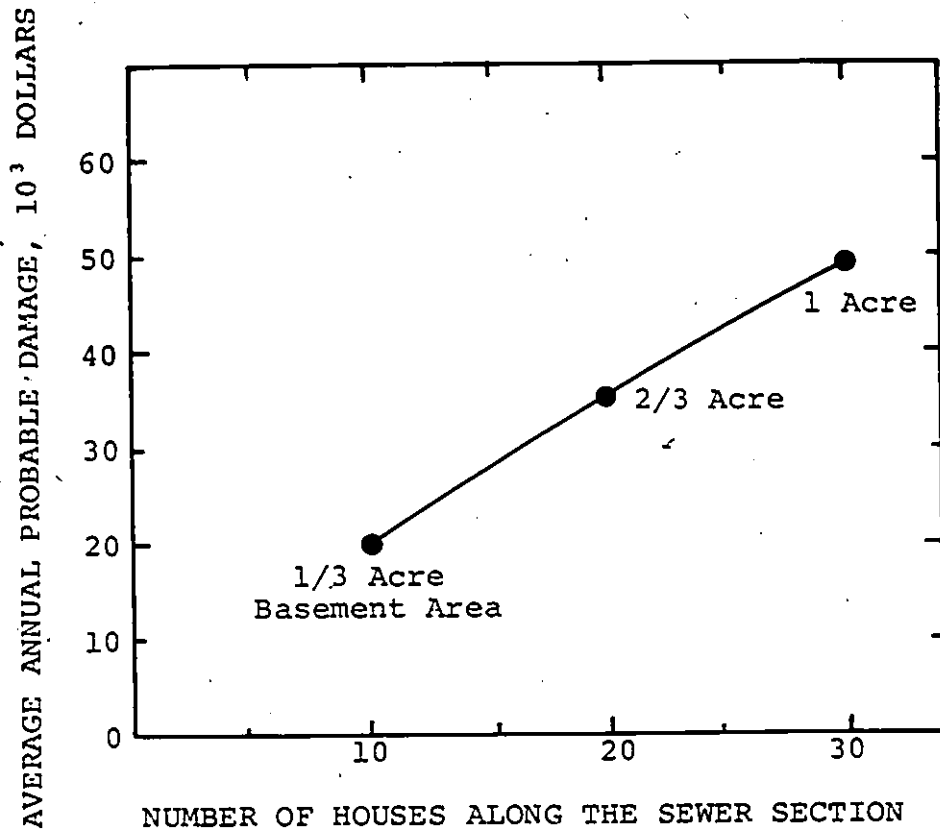
FIGURE 5.8   A TYPICAL VARIATION OF AVERAGE ANNUAL PROBABLE
             DAMAGE WITH NUMBER OF HOUSES ALONG THE SEWER
             SECTION
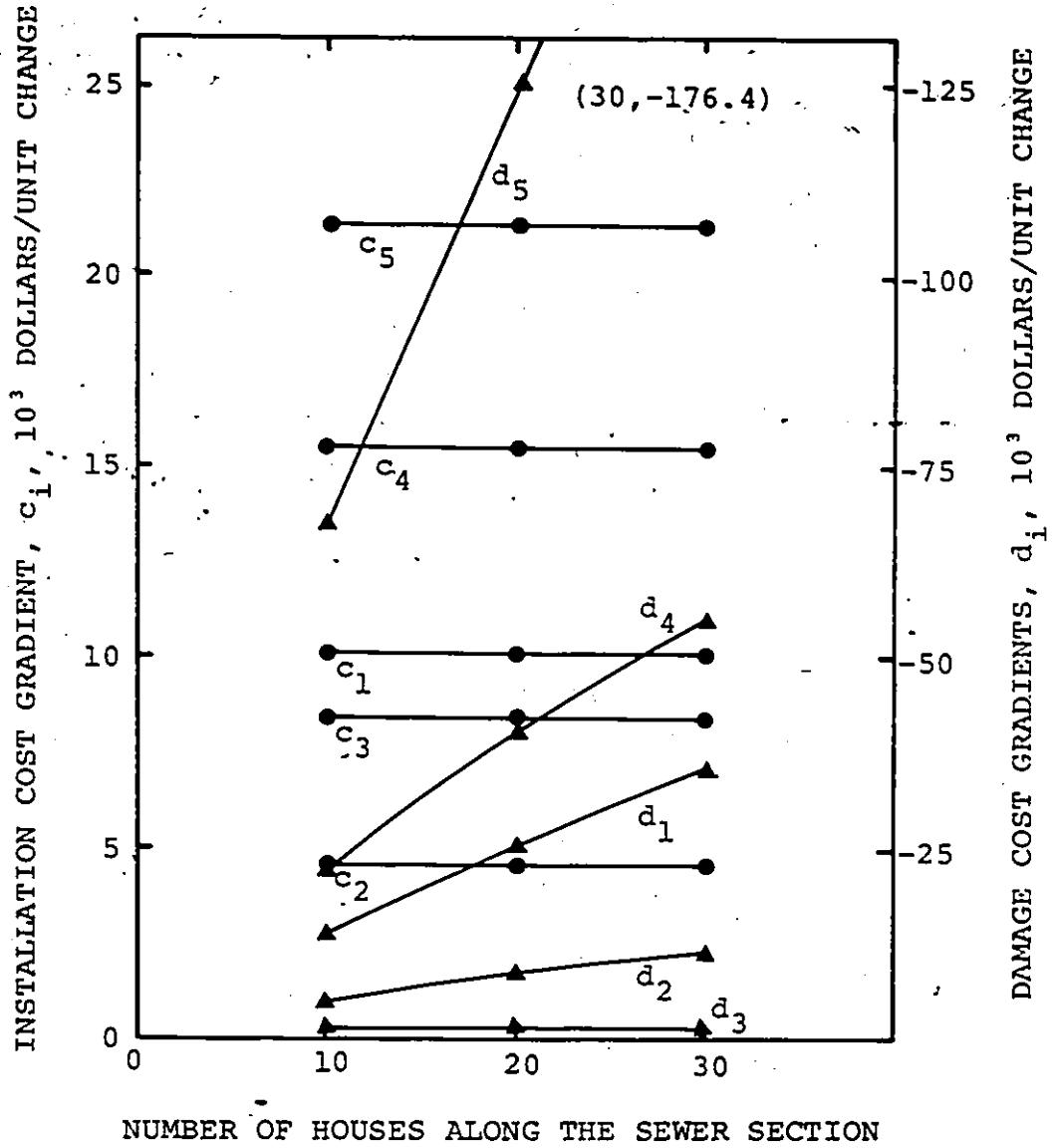
FIGURE 5.9   TYPICAL VARIATIONS OF COST AND DAMAGE GRADIENTS
             WITH NUMBER OF HOUSES ALONG THE SEWER SYSTEM
             (i=1, SEWER SLOPES, i=2, PUMP POWER, i=3, SYSTEM
             DEPTH, i=5, SEWER DIAMETERS)
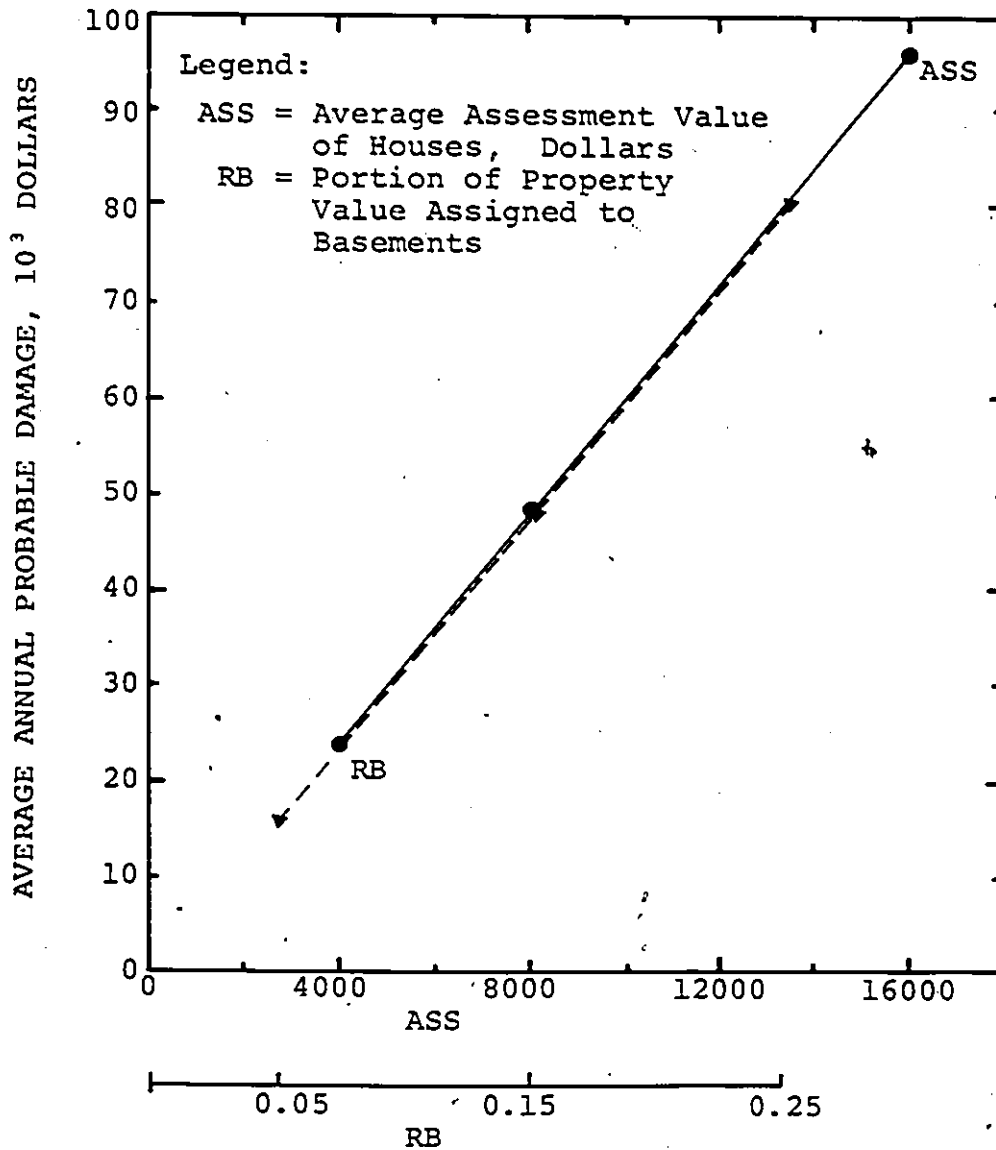
FIGURE 5.10   TYPICAL VARIATIONS OF AVERAGE ANNUAL PROBABLE
DAMAGE WITH ASS AND RB

FIGURE 5.11 SCHEMATIC DRAWING FOR THE HYPOTHETICAL
DRAINAGE BASIN FOR THE EXAMPLE

APPENDIX V

TABLES

TABLE 2.1 HYDRAULIC ROUTING MODELS (After Reference 39)

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \frac{\partial h}{\partial x} = g\,(S_o - S_f)$$

KINEMATIC-WAVE APPROXIMATION

DIFFUSION-WAVE APPROXIMATION

QUASI-STEADY DYNAMIC-WAVE APPROXIMATION

COMPLETE DYNAMIC-WAVE MODEL
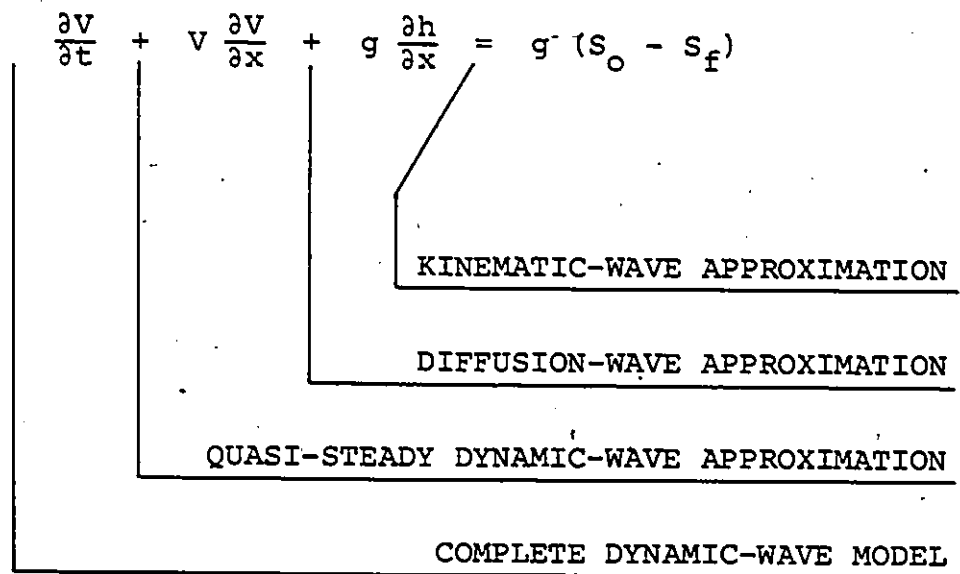
TABLE 3.1   OBJECTIVE FUNCTION AND CONSTRAINTS MATRIX

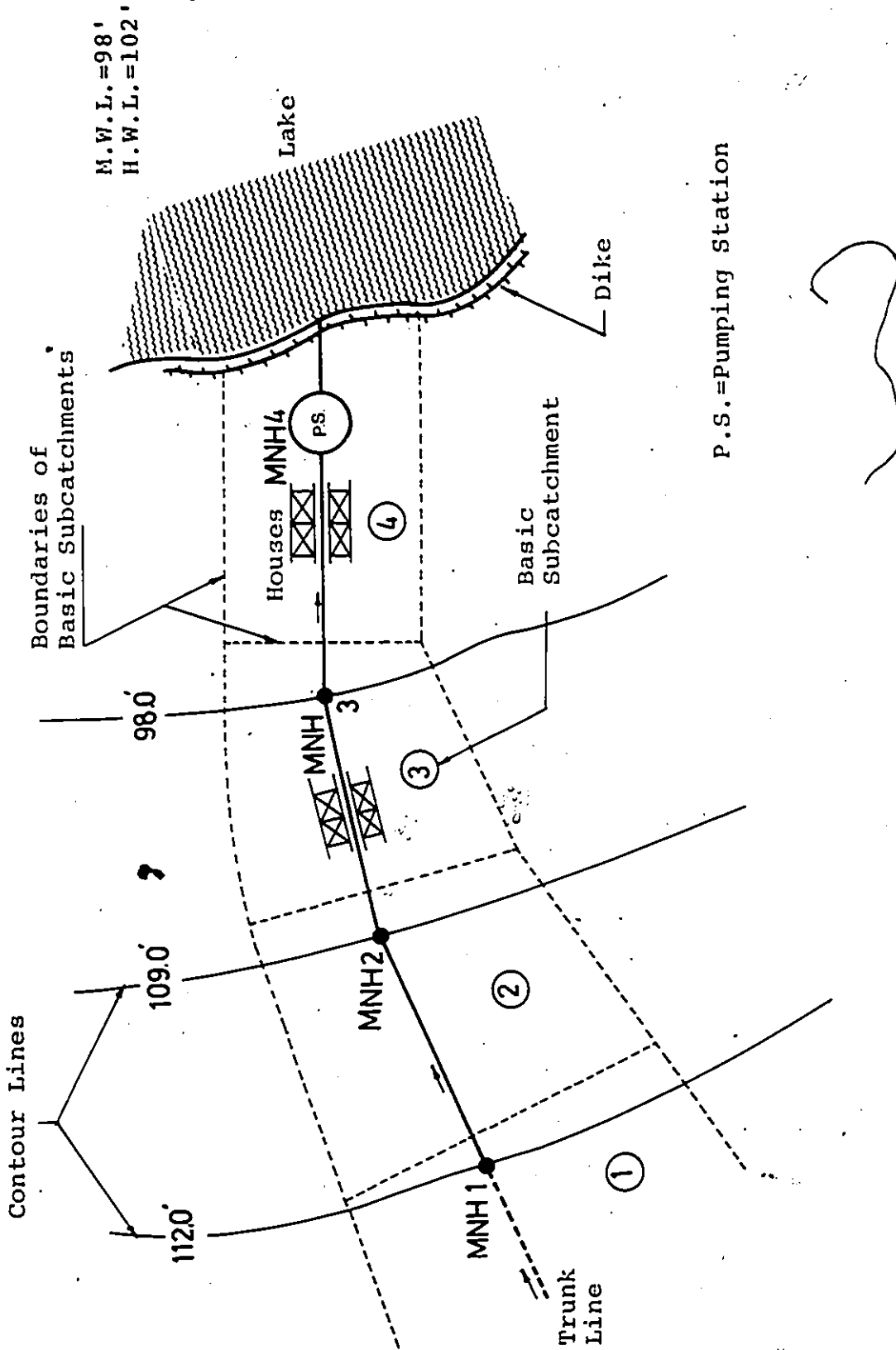| Row Name / Col. Name	Type of Const.	x_1	XX_1	x_2	XX_2	x_3	XX_3	x_4	XX_4	x_5	XX_5	b_i
OBJF	$=$	$-a_1$	$-a_1$	$-a_2$	a_2	$-a_3$	a_3	$-a_4$	a_4	$-a_5$	a_5	
R_1	$\le$	1.0	-1.0									$(x_{UL})_1$
R_2	$\le$			1.0	-1.0							$(x_{UL})_2$
R_3	$\le$					1.0	-1.0					$(x_{UL})_3$
R_4	$\le$							1.0	-1.0			$(x_{UL})_4$
R_5	$\le$									1.0	-1.0	$(x_{UL})_5$
R_6	$\le$	-1.0	1.0									$(x_{LL})_1$
R_7	$\le$			-1.0	1.0							$(x_{LL})_2$
R_8	$\le$					-1.0	1.0					$(x_{LL})_3$
R_9	$\le$							-1.0	1.0			$(x_{LL})_4$
R_{10}	$\le$									-1.0	1.0	$(x_{LL})_5$
R_{11}	$\le$	d_1	$-d_1$	d_2	$-d_2$	d_3	$-d_3$	d_4	$-d_4$	d_5	$-d_5$	$\frac{A}{2} - d_o$
R_{12}	$\le$	d_1	$-d_1$			d_3'	$-d_3'$					$\frac{A}{2} - d_o$
R_{13}	$\le$			d_2'	$-d_2$							$\frac{A}{2} - d_o$
R_{14}	$\le$					d_3	$-d_3$					$\frac{A}{2} - d_o$

TABLE 3.1 (continued)

Row Name	Type of Const.	X_1	XX_1	X_2	XX_2	X_3	XX_3	X_4	XX_4	X_5	XX_5	b_i
R_{15}	$\leq$							d_4	$-d_4$			$\frac{A}{2} - d_0$
R_{16}	$\leq$									d_5	$-d_5$	$\frac{A}{2} - d_0$
R_{17}	$\leq$	$-d_1$	d_1	$-d_2$	d_2	$-d_3$	d_3	$-d_4$	d_4	$-d_5$	d_5	d_0
R_{18}	$\leq$	$-d_1$	d_1									d_0
R_{19}	$\leq$			$-d_2$	d_2							d_0
R_{20}	$\leq$					$-d_3$	d_3					d_0
R_{21}	$\leq$							$-d_4$	d_4			d_0
R_{22}	$\leq$									$-d_5$	d_5	d_0
R_{23}	$\leq$	$-c_1$	c_1	$-c_2$	c_2	$-c_3$	c_3	$-c_4$	c_4	$-c_5$	c_5	c_0
R_{24}	$\leq$	$-c_1$	c_1									c_0
R_{25}	$\leq$			$-c_2$	c_2							c_0
R_{26}	$\leq$					$-c_3$	c_3					c_0
R_{27}	$\leq$							$-c_4$	c_4			c_0
R_{28}	$\leq$									$-c_5$	c_5	c_0
R_{29}	$\leq$	$-a_1$	a_1	$-a_2$	a_2	$-a_3$	a_3	$-a_4$	a_4	$-a_5$	a_5	$c_0 + d_0$

TABLE 4.1 CARD DATA FOR THE FIRST PROGRAM

Card Group	Format	Card Columns	Description	Variable Name	Default Value
1	I3	1-3	Number of manholes, inlet points, equal to the number of trunk sewers + 1	NMH	None
2	3F10.0		REPEAT CARD GROUP2 FOR EACH MANHOLE PROCEEDING DOWNSTREAM		
		1-10	Ground surface elevation, ft.	H(I)	None
		11-20	Coefficient of inlet catch basin, - sloping intersection, - an intersection in depression.	CB(I)	0.01 0.02
		21-30	Number of vehicle crossings at manhole (intersection) per hour, on an average basis.	VEHH(I)	None
3	2F6.0		REPEAT CARD GROUP 3 FOR EACH MANHOLE PROCEEDING DOWNSTREAM		
		1-6	Basic subcatchment impervious area, acres.	ARIMP(I)	None
		7-12	Time of concentration of basic subcatchment, minutes.	ET(I)	None
4	F4.0	1-4	Time step, minutes	T	3.0
5	4I0.0		REPEAT CARD GROUP5 FOR EACH SEWER PROCEEDING DOWNSTREAM		
		1-10	Manning's Coefficient, n.	AN(I)	0.013
		11-20	Sewer length, ft.	ALNTH(I)	None
		21-30	Sewer diameter (initial design), ft.	DIAM(I)	None
		31-40	Sewer slope (initial design).	SO(I)	None

TABLE 4.1 (continued)

Card Group	Format	Card Columns	Description	Variable Name	Default Value
6			REPEAT CARD GROUP 6 FOR EACH SEWER PROCEEDING DOWNSTREAM		
	2F10	1-10	Basements area, acres	ABASE(I)	None
		11-20	Average depth of basements below ground surface, ft.	DBSMT(I)	None
	I1	21	District index	ICLASS(I)	
			– residential		1
			– commercial		2
7	I3	1-3	Number of storms considered	NSTORM	3
8			REPEAT CARD GROUP 8 FOR EACH STORM CONSIDERED		
	510.0	1-10	a (Coefficients in the rainfall intensity-	AA(I)	None
			b (time formula,	BB(I)	None
			$i = \dfrac{a}{t_d^{\,b} + c}$		
			c (	CC(I)	None
		31-40	Relative advancement of peak intensity of the storm	RR(J)	0.375
		41-50	Probability of occurrence of this storm	PROB(I)	
			– 1st storm (1/5 years storm).		0.2
			– 2nd storm (1/10 years storm)		0.1
			– 3rd storm (1/50 years storm)		.02
9	F10.0	1-10	Base flow, cfs.	QBAFL	1.0
10			REPEAT CARD GROUP 10 FOR EACH SEWER PROCEEDING DOWNSTREAM		
	2F10.0	1-10	Average length of the pipe of house connections, ft.	ALPIPE(I)	None
		11-20	Diameter of the pipe of house connections, ft.	DHCON(I)	None
	I3	21-23	Number of houses connected to the sewer.	NHOUS(I)	None

TABLE 4.1 (continued)

Card Group	Format	Card Columns	Description	Variable Name	Default Value
11	3F10.0	1-10	Pump relative operating efficiency	RELETA	0.95
		11-20	Pumping rated head (initial design), ft	HR	None
		21-30	Pumping rated discharge (initial design), ft.	QR	None
12	2F10.0	1-10	Sump cross sectional area, ft². (from initial design)	ASUMP	None
		11-20	Total energy level of the storm water at the delivery side of the pump, may be water surface elevation of receiving body, ft. To be obtained from basin characteristics.	ECHANL	None
13	F10.0	1-10	Crown elevation of the upstream end of the first sewer in the system, ft.	USELEV	None
14	5F10.0		Positive relative changes in decision variables with respect to	POSCHG(I)	
		1-10	i=1, sewer slopes		None
		11-20	i=2, pump power		0.5
		21-30	i=3, sump area		0.5
		31-40	i=4, system overall depth		None
		41-50	i=5, sewer diameters		None
15	5F10.0		Negative relative changes in decision variables with respect to		
		1-10	i=1, sewer slopes		None
		11-20	i=2, pump power		0.5
		21-30	i=3, sump area		0.5
		31-40	i=4, system overall depth		None
		41-50	i=5, sewer diameters		None

Note: 1 ft = 0.3048 m; 1 acre = 0.405 ha; 1 cfs = 0.028 m³/s

TABLE 5.1 EFFECTS OF MANNING'S COEFFICIENT AND TIME STEP ON THE DESIGN

Time Step (min.)	Manning's Coefficient (n)	Balanced Design							
		DIAM(i) (ft)			Q_R (cfs)	H_R (ft)	C_o $	d_o $
		(1)	(2)	(3)				
3	.013	2.75	2.97	3.18	66.3	15	55400	48200
	.020	3.26	3.52	3.77	66.4	15	59300	43600
2	.013	2.73	2.95	3.18	66.1	15	55100	50600

TABLE 5.1 (continued).

Time Step (min.)	Manning's Coefficient (n)	Cost Gradients 10^3 dollars/unit change					Damage Gradients 10^3 dollars/unit change				
		c_1	c_2	c_3	c_4	c_5	d_1	d_2	d_3	d_4	d_5
3	.013	10.17	4.65	8.31	15.66	21.38	-35.29	-11.67	-1.54	-55.32	-176.4
	.020	10.30	4.54	8.50	15.85	30.11	-28.82	-14.64	-2.88	-59.94	-144.1
2	.013	10.23	4.59	8.31	15.60	21.64	-39.21	-13.63	-1.22	-53.48	-196.1

TABLE 5.2 EFFECT OF d_o, c_i AND d_i ON THE OPTIMUM DESIGN

	Case (I) Number of Houses =30 House/section	(Case II) Number of Houses =10 House/section
c_o, Dollars	55,000	55,000
d_o, Dollars	50,400	19,500
Damage Gradients, Dollars/Unit Change		
d_1 (Slope)	-39,750	-13,500
d_2 (Pump)	-15,050	- 5,140
d_3 (Sump)	- 1,150	- 1,200
d_4 (Depth)	-54,300	-22,300
d_5 (Diameter)	-198,800	-67,400
Cost Gradients, Dollars/Unit Change		
c_1 (Slope)	10,000	10,000
c_2 (Pump)	4,650	4,650
c_3 (Sump)	8,300	8,300
c_4 (Depth)	15,700	15,700
c_5 (Diameter)	22,000	22,000
Optimum Design (1) Diameters, ft.		
Sewer (1)	3.0'	3.0'
Sewer (2)	3.3'	3.3'
Sewer (3)	3.5'	3.5'
(2) Slopes, %		
Sewer (1)	0.9	0.54
Sewer (2)	0.9	0.54
Sewer (3)	0.9	0.54
(3) Sump Area, ft²	450	450
(4) Pump		
Rated Head, ft	23.7	25.1
Rated Discharge, cfs	74	36
(5) Characteristic Depth, ft	12.5	20.4

TABLE 5.3   EFFECT OF THE INITIAL DESIGN ON THE BALANCED DESIGN

	Case (I)		Case (II)		Case (III)	
	Initial	Balanced	Initial	Balanced	Initial	Balanced
Diameters, ft						
Sewer (1)	6.00	2.79	4.00	2.75	3.00	2.72
Sewer (2)	6.00	3.02	4.00	2.97	3.00	2.94
Sewer (3)	6.00	3.23	4.00	3.18	3.50	3.17
Pumping Capacity						
Q_R, cfs	30	63.2	30	66.3	60	67.7

TABLE 5.4.a   CHARACTERISTICS OF DRAINAGE BASIN FOR THE EXAMPLE

Manhole Index	Ground Surface Elevation (ft)	Impervious Area (acre)	Time of Concentration (min.)	Coefficient of Inlet Catch Basin	Number of Effective Vehicle Crossing, per Hour
1	112.0	13.0	25.0	0.02	100
2	109.0	3.0	15.0	0.01	100
3	98.0	3.0	15.0	0.02	100
4	97.0	3.0	15.0	0.02	100

TABLE 5.4.b   CHARACTERISTICS OF DRAINAGE BASIN
FOR THE EXAMPLE

	Sewer Section (1)	Sewer Section (2)	Sewer Section (3)
Number of Houses Along Sewer Section	30	30	30
Basements Area Along Sewer Section = Acres	1.00	1.00	1.00
Average Depth of Basements Below Ground Surface, ft	5.0	5.0	5.0
Average Length of the Pipe of the House Connection, ft	100.0	100.0	100.0
Average Diameter of the Pipe of the House Connection, ft	0.33	0.33	0.33
Portion of Property Value Assigned to Basements	0.15	0.15	0.15
Average Assessment Value of Houses, dollars	8,000	8,000	8,000
Assessment Factor	4.0	4.0	4.0
Class Index	1	1	1

TABLE 5.5   RAINFALL DATA

Storm Index	Coefficients in the Rainfall Intensity/Time formula $[i = a/(t_a^b + c)]$			Probability of Occurrence, (Annual Series)
	a	b	c	
1	131.0	1.0	19.0	0.20
2	170.0	1.0	23.0	0.10
3	250.0	1.0	27.0	0.02

. Relative Advancement of the Peak Intensity = 0.375

TABLE 5.6   INITIAL DESIGN BY RATIONAL METHOD

| (1) Line Number | (2) Location | Manhole Number | | (5) Length, ft | Tributary Area (Acres) | | Time of Flow (Min.) | | (10) Impervious Area % of Total = C | (11) Runoff cfs/acre Q=ci | (12) Total Runoff (cfs) |
		(3) From	(4) To		(6) Increment	(7) Total	(8) To Upper End	(9) In Section			
I	U/S	1	2	600	32.5	32.5	25	1.4	0.4	1.21	39.3
II		2	3	600	7.5	40	26.4	1.4	0.4	1.17	46.87
III		3	4	600	7.5	47.5	27.8	1.23	0.4	1.14	54.0
IV		Sump Well	Stream	50	7.5	55	29	--	0.4	1.11	60.9

$i = 133/(t_d + 19)$, 1 in 5 years storm

TABLE 5.6 (continued)

Line Number	Slope of Sewer % (13)	Diameter, in. (14)	Diameter, in. Commercial Sizes (15)	Capacity Full (cfs) (16)	Velocity Full ft/sec. (17)	Sewer Crown Elevation (ft) Upper End (18)	Lower End (19)	Ground Surface Elevation (ft) Upper End (20)	Lower End (21)	Remarks
I	0.6	32.5"	36"	51.9	7.3	99.5	95.9	112.0	109.0	N=.013
II	0.6	34.7"	36"	51.9	7.3	95.9	92.30	109.0	98.0	
III	0.6	36.6"	42"	78.25	8.13	92.30	88.7	98.0	97.0	

TABLE 5.7   INITIAL DESIGN FOR THE EXAMPLE

Sewer No.		(1)	(2)	(3)
Sewers	Invert Slope = % Diameter, ft Manning's n Length = ft	0.6 3.0 0.013 600	0.6 3.0 0.013 600	0.6 3.5 0.013 600
Pump	Rated Head = ft Rated Discharge, cfs	15 61		
Sump Well	Cross Sectional Area, ft^2	900		

TABLE 5.8   COMPARISON BETWEEN THE MODEL'S OPTIMUM DESIGN
AND THE RATIONAL METHOD

	Rational Method Design		Optimum Design by the Model	
	Fractional Sewer Sizes	Commercial Sewer Sizes	Fractional Sewer Sizes	Commercial Sewer Sizes
(I) Sewers				
Diameters, ft;				
Sewer (1)	2.70	3.00	3.00	3.00
Sewer (2)	2.90	3.00	3.23	3.50
Sewer (3)	3.05	3.50	3.50	3.50
Slopes, %				
Sewer (1)	0.6	0.6	0.6	0.6
Sewer (2)	0.6	0.6	0.6	0.6
Sewer (3)	0.6	0.6	0.6	0.6
(II) Pump				
Rated Head, ft	15	15	26	26
Rated Discharge, cfs	61	61	35	35
(III) Sump Well				
Area, ft²	900	900	810	810
(IV) Depth				
Characteristic Depth, ft	12.5	12.5	20.2	20.2
(V) Capital Recovery Cost, dollars/ year	54,400	56,400	64,200	64,900
(VI) Average Annual Probable Damage dollars/year	69,300	27,200	22,000	16,100
(VII) Net Annual Cost	123,700	83,600	86,200	81,000

TABLE 5.9 CONSISTENCY OF DESIGN OBTAINED BY RATIONAL METHOD AND MODEL

Method of Design	Rational Design		Model (Optimum)	
	Fractional Sewer Sizes	Commercial Sewer Sizes	Fractional Sewer Sizes	Commercial Sewer Sizes
Maximum Relative Storage, during a 1 in 5 years storm, P_{max_i}, of				
Sewer (1)	0.981	0.764	0.764	0.764
Sewer (2)	1.029	0.948	0.778	0.628
Sewer (3)	1.088	0.758	0.758	0.758

APPENDIX VI

NOMENCLATURE

NOMENCLATURE

A	Area of channel cross section.
A, B, C	Coefficients in the damage-probability equation
A_{base_i}	Total basements area
AF	Assessment factor to account for the true market value
A_o	Minimum control area to operate the pumping station
A_n	Cross-sectional area of the n-th sewer
A_{pipe}	Cross-sectional area of the pipe of house connection
A_s	Sump cross-sectional area
$(ASS)_i$	Average assessment value of houses
A_{surf}	Ground surface flooded area
ΔA	Increment of impervious area
a	Constant
B	Water surface width
B_i	Mid-point elevation of basement grade line
$(B_{value})_i$	Assessed value of basements subjected to damage
b	Constant
C	Cost
c	Cost per unit length of sewer
$\tilde{c}$	Total annual costs of a perturbed system

C_{B_i}	Coefficient of inlet catch basin
C_f	Coefficient of friction of the pipe of the house connection
CL	Calibration factor
$CLEAN$	Basement cleaning factor
C_{land}	Cost of land
C_o	Capital recovery cost of the balanced system
C_{oper}	Annual operating cost
C_o, C_1, C_2	Muskingum Coefficients
$Cost_i$	Cost per unit length of sewer
CRF	Capital recovery factor
C_{STBY}	Installation cost of the stand-by pumping unit(s)
CT_i	A land use factor
c	Constant
c_{exc}	Unit cost of excavation
c_i	Installation cost gradient
c_{st}	Unit cost of structure per unit area of its plan
D, D_i	Sewer diameter
D	Total damage resulting from a rain storm
D_{av}	Average annual probable damage for a perturbed system
$(D_{av})_o$, d_o	Average annual probable damage for the balanced system
$(D_{av})_{ult.}$	Ultimate average annual probable damage

$(D_{Base})_i$	Damages due to basements flooding
D_{factor}	Normalized damage factor
D_{Inc}	Inconvenience cost
D_k	Total damage resulting from applying a preselected storm on the perturbed system
D_{prop}	Property damages
DSHL	Total head loss in a house connection at the downstream end of the sewer
$d_f(i,t)$	Basements flooding depth
$d_{f_{max_i}}$	Maximum value of the basement flooding depth
$d_{h.con.}$	Diameter of the pipe of the house connection
d_i	Damage cost gradient for variable i
d_{sump}	Sump excavation depth
d_{sys}	Characteristic depth of sewer system
$E(i,t)$	Mid-point elevation of sewer energy line
E_{crown}	Crown elevation of the downstream end of the outfall sewer
E_d	Energy level on the delivery side of the pump
E_{sump}	Energy level immediately upstream of the sump
F	Subscript represents variables at the final state
$F(i,t)$	Flooding ratio
F_{max_i}	Maximum value of flooding ratio
FU	Value of basement furniture/basement value

G	—	Severity parameter
g		Gravitational acceleration
H		Head
H_R		Rated pumping head
$(H_{av})_i$		Average depth of excavation of the sewer
HEALTH		Health hazards factor
H_i		Ground surface elevation at manhole i
H_p^t		Operating pumping head at time t
h		Depth of flow
h_ℓ		Total head loss through a particular house connection
I		Rate of inflow
I		Subscript represents variables at the initial state
I		Interest rate per annum
i		Index
i, i_m		Rainfall intensity
i_{av}		Average rainfall intensity for a duration t_d
j		Index
K		Storage Coefficient (Muskingum Method)
K		Regression coefficient
K		First time step index of the surcharged flow condition
K_{e_i}		Coefficient of energy line slope
K_m		Minor losses coefficient

$\ell_{h.con.}$	Average length of the pipes of house connections with a particular sewer section
ℓ_i	Sewer length
$MNH(i)$	Manhole i
m	Exponent
N	Years of estimated life
$(N_{cross})_i$	Effective number of vehicle crossings per hour on the average basis
N_{house}	Number of houses along a particular sewer section
n	Manning's Coefficient
n	Number of increments of impervious area
O	Rate of outflow from the control volume considered
P	Power of installed pump
P_{max_i}	Maximum relative storage of the i-th sewer, during a specific storm
p	Probability of occurrence of the rain storm
p_x	Probability of occurrence of a storm resulting in the same damage when applied on the original balanced system
p_k	Probability of occurrence of a preselected representative storm
Q	Discharge
Q_R	Rated pumping discharge
$Q_{av}(i,t)$	Average flowrate in the i-th sewer
$Q_B(i,t)$	Basement flow
Q_{in}	Inflow to manhole
$Q_{inf}(IMH,K)$	Runoff contributing to manhole IMH at time $K.\Delta t$

Q_o — Flow capacity of the sewer when flowing full

Q_{ot} — Outflow from manhole

Q_{out_n} — Flow rate at the downstream end of the n-th sewer

Q_p — Operating pumping flowrate

$(Q_R)_{adj}$ — The adjusted pumping capacity

Q_S — Inflow to the sump

$Q_{S.house}$ — Flow in house connection to the basement of a single house

$Q_{st}(i,t)$ — Street flow

Q_x — Basement flow per unit length of the sewer

q — Distributed lateral inflow (or outflow) as discharge per unit length of the conduit

RB_i — Portion of property value assigned to basements

REI — Relative engineering index

R_{stg} — Relative storage

r — Relative advancement of the rainfall peak intensity

s — Storage within the control volume

$(S_B)_i$ — Slope of basement grade line

$S_e(i,t)$ — Slope of sewer energy line

S_f — Friction slope

S_o — Channel bottom slope

S_{o_i} — Invert slope of the i-th sewer

SWL	Sump water level
t	Time index representing the interval $t-\Delta t$ to t
t_a, t_b	Time measured from the peak to the left and the right respectively
t_d	Duration of rain storm
t_f	Time of flow in the conduit
$(t_p)_i$	Time elapsed from the start of intersection flooding to the time the peak V_{max_i}
$USEL_i$	Elevation of the energy line at the upstream end of the i-th sewer
USHL	Total head loss in a house connection at the upstream end of the sewer
V	Full-bore velocity
V	Cross-sectional average flow velocity along the x-direction
V_{act}	Total actual storage
V_{max_i}	Maximum value of the volume of storm water collected on the ground surface at manhole i
V_n	Velocity in the outfall sewer
V_{st}	Volume of storm water collected on the ground surface
V_{sys}	Total maximum available storage of the sewer system
W_{street}	Width of street
X	Average depth of excavation
X_i, X_j	Decision (design) variable
XX_i	Slack variable
X_{LL}	Lower limit of the decision variable

X_{UL}	Upper limit of the decision variable
x	Routing parameter
x	Longitudinal coordinate along channel bottom direction
Y_s	Sump water depth above minimum sump water level
Z	Objective function
γ_w	Specific weight of water
Δ	Small change
η	Pump efficiency

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