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LA THÈSE A ÉTÉ MICROFILMÉE TELLE QUE NOUS L'AVONS RÉCEUE
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
The University of Windsor

Windsor, Ontario, Canada
1977
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 surcharged 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
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.
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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
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." Management 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 installation 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 designer's 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 constraints 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. Grasped 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 evapotranspiration. 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, therefore, 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 determined 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

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

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

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

\[
i_{ave.} = \frac{a}{t_d^b + c} \quad (2.2)
\]

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 aforementioned 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

\[
\frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + g \frac{\partial h}{\partial x} - g (S_o - S_f) = \frac{v Q}{A} \tag{2.3.a}
\]

and the corresponding equation of continuity

\[
\frac{\partial h}{\partial t} + \frac{A}{B} \frac{\partial v}{\partial x} + v \frac{\partial h}{\partial x} = \frac{Q}{B} \tag{2.3.b}
\]

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;
\[ S_o = \text{channel bottom slope}; \]
\[ S_f = \text{friction slope}; \]
\[ q = \text{the distributed lateral inflow (or outflow) as discharge per unit length of the conduit}; \]
\[ A = \text{area of channel cross section}; \]
\[ B = \text{water surface width}. \]

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

\[ I - O = ds/dt \tag{2.4.a} \]

where

\[ I = \text{rate of inflow into the control volume considered}; \]
\[ O = \text{rate of outflow from the control volume considered}; \]
\[ S = \text{storage within the control volume}; \]

or it may be written in finite difference form,
\[ \frac{\Delta t}{2} (I_1 + I_2) = \frac{\Delta t}{2} (O_1 + O_2) + (S_2 - S_1) \]  \hspace{1cm} (2.4.b)

where the subscripts indicate the variables at the beginning and end of the time increment, \( \Delta t \). 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, \( t_f = \frac{L}{V} \), 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 predetermined layout. Some of these models have an additional capability to find the optimum layout (18, 2). The optimization 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:

$$C = a + bD^2 + cX^2$$ (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

\[ C_{\text{pump}} = K \ (P)^m \]  \hspace{1cm} (2.6)

where

- \( C_{\text{pump}} \) = 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

\[ C_{\text{oper.}} = a \ Q + b \ H \ Q \]  \hspace{1cm} (2.7)

where

- \( C_{\text{oper.}} \) = 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-plain areas (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 occurrence 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 $SF = \frac{Q_c}{Q_p}$ 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 neighborhood. 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, \( Q_l - Q_c \), 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

Minimize \[ Z = c_1 x_1 + \ldots + c_n x_n \] (2.8)

Subject to

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

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

\[ a_{m1} x_1 + a_{m2} x_2 + \ldots + a_{mn} x_n = b_m \] (2.9)

and

\[ x_j \geq 0, \quad j = 1, 2, \ldots, n \] (2.10)
is called the standard linear programming form. The function
to be minimized, \( z \), is called the objective function. The
variables \( x_j \) (\( j = 1, 2, \ldots, n \)) are called the design
(decision) variables. The quantities \( c_j \), \( b_i \geq 0 \), and \( a_{ij} \)
(\( i = 1, \ldots, m \) and \( j = 1, \ldots, n \)) 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. (14) 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 hydrograph 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 originality 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 appropriate 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 coefficients \( a, b, \) 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 statistical analysis of many rainfall events in or near the watershed. 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 computational 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
\[ Q_{\text{inf}}(\text{IMH,K}) = (i_k + i_{k-1} + i_{k-2} + \ldots + i_{k-n}) \Delta A \]  

where

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

\[ i_m = \text{average rainfall intensity (in/hr)} \]
during the time increment starting \((m-1) \Delta t\) and ending \((m) \Delta t;\)

\[ \Delta A = \text{increment of impervious area for a time increment } \Delta t, \text{ contributing to junction manhole IMH, acres;} \]
and

\[ n = \text{number of increments of impervious area, contributing to junction manhole IMH;} \]

\[ = \text{impervious area contributing/time increment.} \]

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

\[ S = K_0 + Kx (I - O) \]  

(3.2)

which may be rewritten in a finite difference form,
\[ S_2 - S_1 = K \left[ x(I_2 - I_1) + (1 - x)(O_2 - O_1) \right] \] (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,

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

results in the following equation

\[ O_2 = C_0 I_2 + C_1 I_1 + C_2 O_1 \] (3.5)

where

\[ C_0 = - \frac{Kx - 0.5 \Delta t}{K - Kx + 0.5 \Delta t} \] (3.6)

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

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

\( C_0, C_1, C_2 \) 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.

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

or

$$S = \left(S_o/Q_o\right) Q \quad (3.9.b)$$

But

$$S = KQ$$

hence

$$K = S_o/Q_o \quad (3.10)$$

Substituting for

$$S_o = \frac{\pi}{4} D^2 L,$$  

and

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

then the value of $K$ can be obtained for each sewer using
the following equation,

\[ K_i = \frac{1.696 \, n \, l_i}{D_i^{2/3} \, S_{o_i}^{1/2}} \]  

(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;
- \( S_{o_i} \) = 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,

\[ S_{B_i} = \frac{H_i - H_{i+1}}{l_i} \]  

(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:

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

or

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

Hence,

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

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

$$Q_{av}(i,t) = \sum_{l=1}^{i} Q_{inf}(i,t) - \sum_{l=1}^{i} Q_{st}(i,t) - \sum_{l=1}^{i-1} Q_B(i,t) - Q_B(i,t)/2 \quad (3.15)$$

where

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

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

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

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 variables, 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) Construction of the Energy Line of the System: 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

$$\text{USEL}_i = \text{USEL}_{i+1} + S_e(i,t) \ell_i \quad (3.16)$$
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 \( A, B, \ldots \), 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

\[
d_f(i, t) = \frac{t}{A_{base_i}} \frac{Q_B(i, t) \Delta t(60)}{43560}
\]

where

\[
d_f(i, t) = \text{flooding depth of the basements associated with the } i\text{-th sewer at the end of the } t\text{-th time step, ft};
\]

\[
Q_B(i, t) = \text{average rate of basement flow for } i\text{-th sewer during the } t\text{-th time step, cfs};
\]

\[
\Delta t = \text{time increment, min.};
\]
\( A_{\text{base}_i} \) = 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 \( d_E \). 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, \( Q_{s,\text{house}} \), 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 friction, when combined
together, may be used to derive an equation to compute,
\( Q_{s\text{-house}} \), in the form

\[
Q_{s\text{-house}} = A_{\text{pipe}} \sqrt{\frac{2 \cdot g \cdot h_f}{K_m + \frac{C_f \cdot L_{\text{h\_con.}}}{d_{\text{h\_con.}}}}} \tag{3.18}
\]

where

- \( Q_{s\text{-house}} \): flow in house connection to the basement of
  a single house, cfs;
- \( A_{\text{pipe}} \): cross sectional area of the pipe of the
  house connection, ft\(^2\);
- \( L_{\text{h\_con.}} \): average length of house connections within
  a particular sewer section, ft;
- \( d_{\text{h\_con.}} \): diameter of the pipe of the house connection,
  ft;
- \( h_f \): 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/sec\(^2\).
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, \( N_{\text{house}} \), and the length of the i-th sewer is \( \ell_i \), then, the basement flow per unit length of the sewer, \( Q_x \), see Figure 3.10, can be obtained from

\[
Q_x = \frac{(Q_{s\text{.house}})_i}{(N_{\text{house}})_i} \cdot \frac{1}{\ell_i} \tag{3.19}
\]

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

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

Substitution of Equations 3.18, 3.19 into 3.20 yields

\[
Q_B(i,t)(+ve) = \frac{\sqrt{2g}}{\sqrt{k_m + \frac{C_f}{d_{h\text{.con.}}}}} \frac{A_{\text{pipe}}(N_{\text{house}})_i}{\ell_i} \int_0^F(i,t)\ell_i \sqrt{h_\zeta} dx \tag{3.21}
\]

but, \( h_\zeta \) can be expressed as

\[
h_\zeta = \frac{X}{\text{USHL}} \cdot \frac{F(i,t)\ell_i}{\ell_i} \tag{3.22}
\]
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

\[
Q_B(i,t)(+ve) = \frac{2}{3} F(i,t) (N_{house})^i \left\{ \frac{A_{pipe}}{\sqrt{2g}} \right. \\
\left[ \frac{USHL}{K_m + \frac{C_F L_{h.con.}}{d_{h.con.}}} \right] \right. 
\]

(3.23)

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

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

(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 DSHEL 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 \([1 - F(i,t)] L_i\) in one time step, i.e.
\[ Q_B(i,t)(-ve) \leq \frac{d_f(i,t) [1 - F(i,t)] A_{base} \Delta t}{\Delta t} \quad (3.25) \]

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

\[ Q_B(i,t) = Q_B(i,t)(+ve) - Q_B(i,t)(-ve) \quad (3.26) \]

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 \( Q_B(i,t)(+ve) \),

\[
Q_B(i,t)(+ve) = \frac{2}{3} (N_{house})_i \left\{ A_{pipe} \sqrt{\frac{2g}{K_m + \frac{1/2}{C_f} \frac{t_{h.con.}}{d_{h.con.}}} \left( \frac{(USHL)^{3/2} - (DSHL)^{3/2}}{USHL - DSHEL} \right)} \right\} 
\quad (3.27)
\]

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:

\[
F(i,t) = \frac{DSHL}{|USHL| + DSHL} \quad (3.28)
\]

or

\[
F(i,t) = \frac{USHL}{USHL + |DSHL|} \quad (3.29)
\]

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

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

**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.

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

**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

\[
Q_{st}(i, t) = Q_{in} - Q_{ot}
\]  

(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

$$Q_{in} = \sum_{i=1}^{i} Q_{in}^{(i,t)} - \sum_{i=1}^{i-1} Q_{B}^{(i,t)} - \sum_{i=1}^{i-1} Q_{st}^{(i,t)}$$  \hspace{1cm} (3.33)$$

and computes $Q_{ot}$ from

$$Q_{ot} = Q_{av}^{(i,t)} + Q_{B}^{(i,t)}/2$$  \hspace{1cm} (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 downstream end of the $i$-th sewer, DSEL$_{i}$, computes the corrected energy slope, $S_{e}^{(i,t)}$.

$$S_{e}^{(i,t)} = \frac{H_{i} - DSEL_{i}}{\varepsilon_{i}}$$  \hspace{1cm} (3.35)$$

The resulting $S_{e}^{(i,t)}$ is then used to compute $Q_{av}^{(i,t)}$. 
During the Recession Limb of the Hydrograph: 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 recession 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

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

(3.36)

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

$$V_{st} \propto d_{si}^{3}$$

(3.37)

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

\[ Q_{st} \propto d_{si}^{3/2} \quad (3.38) \]

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

\[ Q_{st}(i,t)(-ve) = C_{Bi} \sqrt{v_{st}(i,t)} \quad (3.39) \]

where \( C_{Bi} \) is a coefficient that depends upon the inlets used and the topography around the inlets. The model proposes a value of 0.01 for \( C_{Bi} \) 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

$$R_{stg} = \frac{V_{act}}{V_{sys}}$$  \hspace{1cm} (3.40)

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

$$V_{sys} = \sum_{i=1}^{n} \left( \frac{\pi}{4} D_i^2 \right) L_i$$  \hspace{1cm} (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 $E_{\text{sum}}$ is not function of the sump water level, instead it is computed from:

$$E_{\text{sum}} = E_{\text{crown}} + \frac{v_n^2}{2g}$$  \hspace{1cm} (3.42)

where $E_{\text{sum}}$ is the energy level immediately upstream of the sump at point A; $E_{\text{crown}}$ is the crown elevation of the downstream end of the outfall sewer $n$, and $v_n$ is the velocity in the $n$-th sewer which can be computed from:

$$v_n = \frac{Q_{\text{out}_n}}{A_n}$$  \hspace{1cm} (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,
Brake Horse Power = \gamma_w Q_R H_R / \eta_R

= \gamma_w Q_p H_p / \eta_p

(3.44)

where \( \eta_p \) is the pump efficiency (overall); \( Q \) is the pumping flowrate; \( H \) is the pumping head; \( \gamma_w \) 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

\[ Q_p = C_p \left(1/H_p\right) \]

(3.45)

where \( C_p \) is a parameter defined as,

\[ C_p = \frac{\eta_p}{\eta_R} H_R Q_R \]

= \eta_{rel} H_R Q_R

(3.46)

The model assigns a default value of 0.95 to \( \eta_{rel} \). 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

\[ \frac{1}{2} \left( Q_s^t + Q_s^{t-1} \right) - \frac{1}{2} \left( Q_p^t + Q_p^{t-1} \right) = \Delta S/\Delta t \]

(3.47)

where \( \Delta S \) is the incremental change in the storage within the sump or from Figure 3.15.
\[ \Delta S = A_s \Delta Y_s \]  
(3.48)

but
\[ \Delta Y_s = -\Delta H_p = -(H_p^t - H_p^{t-1}) \]  
(3.49)

where:

\[ A_s = \text{sump cross sectional area; and} \]

\[ Y_s = \text{sump water depth above minimum sump water level} \]

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

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

where

\[ B = -H_p^{t-1} + \frac{C}{C_p} \left(Q_s^t + Q_s^{t-1}\right) - \frac{C}{H_p^{t-1}} \]  
(3.51)

and

\[ C = \frac{C_p}{2A_s} \Delta t \]  
(3.52)

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

\[ \text{SWL} = E_d - H_p^t \]  
(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

\[ Q_s^t = Q_{out}(n,t) + Q_{inf}(n+1,t) \]  
(3.54)
where \( Q_{\text{inf}} (n+1,t) \) 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 transformation of variables may not only simplify problem formulation, 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 \( X_i \), \( i = 1, 2, \ldots, 5 \).

(i) Variable \( X_1 \): The incremental relative change in the sewers' slope
\[ X_i = \frac{(S_{o_i})_F - (S_{o_i})_I}{(S_{o_i})_I} ; \quad i = 1, 2, 3, \ldots, n \]  

(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 \(X_i\), from

\[ (S_{o_i})_F = (S_{o_i})_I (1 + X_i) \]  

(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 3.48, 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 compatibility, \((Q_R)_F\) is determined from the following equation
\[(Q_R)_P = \frac{(Q_R^H)_I}{(Q_R)_I} (1 + X_2) \]  

(3.58)

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

\[X_3 = \frac{(A_s)_P - (A_s)_I}{(A_s)_I} \]  

(3.59)

or

\[(A_s)_P = (A_s)_I (1 + X_3) \]  

(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

\[X_4 = \frac{(d_{sys})_P - (d_{sys})_I}{(d_{sys})_I} \]  

(3.61)

or

\[(d_{sys})_P = (d_{sys})_I (1 + X_4) \]  

(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;

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

or

$$(D_i)_F = (D_i)_I (1 + X_5)$$  \hspace{1cm} (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 overdesigned components, e.g. pipe and pump sizes. The balancing procedure removes the inconsistencies of over- or underdesigned 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 $P_{\text{max}_i}$, and its diameter is $D_i$, then, the balanced diameter is obtained from

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

(3.65)

If the model switches to surcharged flow, $P_{\text{max}_i}$ is taken equal to the value of $P_i$ just prior to switching. The model has to iterate three times to achieve the same $P_{\text{max}_i}$ 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, \( \Delta Q_P^t \), is obtained from

\[
\Delta Q_P^t = \frac{(SWL_t^t - SWL_{t-1}) A_s}{\Delta t (60)}
\]  

(3.66)

The model selects the maximum value of \( \Delta Q_P^t \) 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

\[
(Q_R)_{adj} = Q_R + \Delta Q_P
\]  

(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
\[ \min Z = (C_0 + d_0) + \sum_{i=1}^{5} c_i X_i + \sum_{i=1}^{5} d_i X_i \] (3.68)

where

\[ c_i = \frac{\partial C}{\partial X_i} \] (3.69)

\[ d_i = \frac{\partial D}{\partial X_i} \] (3.70)

\( C_0 \) and \( C \) are the total annual costs of the initial design and the perturbed system respectively, installation and operational costs, and \( d_0 \) 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

\[ \min Z' = \sum_{i=1}^{5} a_i X_i \] (3.71)

where

\[ Z' = Z - (C_0 + d_0) \] (3.72)

and

\[ a_i = c_i + d_i \] (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

\[
\text{Cost}_i = a + b D_i^2 + c (H_{av})_i \quad (3.74)
\]

where

\[
\begin{align*}
\text{Cost}_i & = \text{unit cost of the } i\text{-th sewer, } \$/\text{ft;} \\
D_i & = \text{diameter of the } i\text{-th sewer, ft;} \\
(H_{av})_i & = \text{average depth of excavation of the } i\text{-th sewer, ft;} \\
a, b, c & = \text{regression coefficients.}
\end{align*}
\]

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

\[ \text{Cost of sewers} = \left( \frac{\sum_{i=1}^{n} \text{Cost}_i \ell_i}{\sum_{i=1}^{n} \ell_i} \right) \text{REI} \quad (3.75) \]

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

\[ \text{CRF} = \frac{I (1 + I)^N}{(1 + I)^N - 1} \quad (3.76) \]

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,

\[ \text{Cost of pumps} = c Q_R H_R + C_{STBY} \quad (3.77) \]

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 $/\text{cfs-ft}$, 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,

\[ \text{Cost of sump} = A_s d_{\text{sump}} c_{\text{exc}} \]  

(3.78)

where

\[ A_s = \text{sump cross-sectional area, ft}^2; \]
\[ d_{\text{sump}} = \text{sump excavation depth, ft}; \]
\[ c_{\text{exc}} = \text{unit cost of excavation, } $/\text{ft}^3 = $/\text{ft}^3, \]

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

$$c_{exc} = c_{exc} \cdot (1 + \frac{d_{sump}}{30})$$  \hspace{1cm} (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:

Cost of building = $C_{land} + C_{structure}$  \hspace{1cm} (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,

$$C_{structure} = (A_s + A_o) \cdot c_st$$  \hspace{1cm} (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 =

$$(A_s e_{\text{sum}} c_{exc} + C_{\text{land}} + C_{\text{structure}}) \cdot (REI)_{\text{sum}} \cdot (CRF)_{\text{sum}}$$  \hspace{1cm} (3.82)
where \((\text{REI})_{\text{sump}}\) is the relative engineering index for the sump and the building; and \((\text{CRF})_{\text{sump}}\) 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

\[
C = C_{\text{sewers, manholes, and inlets}} + C_{\text{pumps}} + C_{\text{sump and building}} + C_{\text{operational}}
\]  

(3.83)

where \(C_{\text{operational}}\) 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 \(c_i\), \(i = 1, 2, ..., 5\), 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, $(\text{Cost})_i$. Then
$c_i$ is obtained from

$$c_i = \frac{(\text{Cost})_i - C_0}{X_i} \quad (3.84)$$

where $C_0$ 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

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

where

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

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

$$ D_{\text{factor}} = \text{normalized damage factor.} $$

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

$$ (B_{\text{value}})_i = (\text{ASS})_i (AF) (1 + FU) R_{B_i} (N_{\text{house}}_i F_{\text{max}}_i) \quad (3.86) $$

where

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

$$ AF = \text{assessment factor to account for the true market value, } = 4 \text{ (default value);} $$

$$ FU = \text{value of basement furniture/basement value} = 0.25 \text{ (default value);} $$

$$ R_{B_i} = \text{portion of property value assigned to basements along the } i\text{-th sewer.} \quad \text{The model assumes a default}$$
value of 0.15;

\[ F_{\text{max}_i} = \text{maximum value of flooding ratio along the} \]
i-th sewer, during the storm being considered. This parameter is obtained from the simulation part of the model.
The damage factor, \( D_{\text{factor}} \), 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 \( D_{\text{factor}} \) is

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

where

\[ \text{HEALTH} = \text{health hazard factor}, \]
\[ = 0.5 \text{ (default value)}; \]
\[ \text{CLEAN} = \text{Basement cleaning (after storm) factor}, \]
\[ = 0.05 \text{ (default value)}. \]

The model computes the value of the severity parameter, \( G \), from

\[ G = 0.5 + 0.166 d_{f_{\text{max}_i}} \quad (3.88) \]

where \( d_{f_{\text{max}_i}} \) 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 \( d_{f_{\text{max}_i}} \) reaches
a value of 3 in., the value of \( G \) becomes unity, i.e. complete basement damage. If \( d_{\text{max}} \) 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, \( D_{\text{prop}} \), 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

\[
D_{\text{prop}} = (\text{CL}) \frac{1}{2} [(\text{ASS})_i (\text{AF}) (N_{\text{house}})_i CT_i + (\text{ASS})_{i-1} (\text{AF}) (N_{\text{house}})_{i-1} CT_{i-1}] (\frac{\nu_{\text{max}}}{A_{\text{surf}}})
\]

(3.89)

where

- \( \text{CL} \) = 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);
\( V_{\text{max}_i} \) = 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, \( \text{ft}^3 \); and

\( A_{\text{surf}} \) = ground surface flooded area, \( \text{ft}^2 \).

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

\[
A_{\text{surf}} = W_{\text{street}} \frac{1}{2} (l_i + l_{i-1})
\]

(3.90)

where \( W_{\text{street}} \) 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 \( D_{\text{inc}} \) is proposed in the form

\[
D_{\text{inc}} = (N_{\text{cross}})_i (t_p)_i^2/360
\]

(3.91)

where

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

\( (N_{\text{cross}})_i \) = 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 \( V_{\text{max}_i} \) (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 \( t_p \) 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

\[
D = A + B p + C p^2
\]  

(3.92)

where

\[
D = \text{total damage resulting from a rain storm with a probability of occurrence of } p \text{. The probability of occurrence, } p, \text{ is the reciprocal of the return period, } T_r,\]
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, \( p_1 \), and estimates the damage, \( D_1 \), resulting from each storm. Then it substitutes in the following equations to obtain A, B, and C, viz,

\[
C = \frac{(D_2 - D_1)(p_1 - p_3) + (p_2 - p_1)(D_3 - D_1)}{(p_2 - p_3)(p_3 - p_1)(p_1 - p_2)} \tag{3.93}
\]

\[
B = \frac{(D_2 - D_1) - C(p_2^2 - p_1^2)}{p_2 - p_1} \tag{3.94}
\]

and \( A = -B p_1 - C p_1^2 + D_1 \) \tag{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,

\[
D_{av} = A p_o + \frac{1}{2} B p_o^2 + \frac{1}{3} C p_o^3 \tag{3.96}
\]

where \( p_o \) is the probability of occurrence of the most severe
storm that could occur but not cause damage. The model considers \( p_0 \) 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, \( D_{av_0} \) 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

\[
D_{av} = d_o \frac{p_x}{p_x} \quad (3.97)
\]

where

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

\( d_o \) = average annual probable damage for the original balanced system, dollars;

\( p_x \) = 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,

\[
p_x = \frac{-B - \sqrt{B^2 - 4C (A - D_x)}}{2C} \quad (3.98)
\]
where

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

\[ A, B, C = \text{coefficients in damage-probability equation of the original balanced system.} \]

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

\[ d_i = \frac{(D_{av})_i - d_0}{x_i} \quad (3.99) \]

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

\[ X_i \leq (X_{UL})_i \; ; \; i = 1, 2, \ldots, 5 \] (3.100)

\[ X_i \geq (X_{LL})_i \; ; \; i = 1, 2, \ldots, 5 \] (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,
which results for case of having no sewer system, i.e. \( p_0 \) equals unity, see Figure 3.19. An approximate value of \( (D_{av})^\text{ult} \) may be obtained by assuming a linear relationship between damage and probability, in which case, the area under the curve, \( (D_{av})^\text{ult} \) 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,

\[
\text{upper limit } \sum_{i=1}^{t} d_i X_i + d_o \leq \frac{1}{2} A \tag{3.102}
\]

and

\[
d_i X_i + d_0 \leq \frac{1}{2} A ; \ i = 1, 2, \ldots, 5 \tag{3.103}
\]

\[
\text{lower limit } \sum_{i=1}^{5} d_i X_i + d_o \geq 0 \tag{3.104}
\]

and

\[
d_i X_i + d_0 \geq 0 ; \ i = 1, 2, \ldots, 5 \tag{3.105}
\]

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,

\[
\sum_{i=1}^{5} c_i X_i + C_o \geq 0 \tag{3.106}
\]
and \[ c_i X_i + C_o \geq 0; \quad i = 1, 2, \ldots, 5 \] (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,

\[ \Delta C + \Delta D \leq C_o + D_o \] (3.108a)

or

\[ \sum_{i=1}^{5} c_i X_i + \sum_{i=1}^{5} d_i X_i \leq C_o + d_o \] (3.108b)

or

\[ \sum_{i=1}^{5} a_i X_i \leq C_o + d_o \] (3.108c)

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 numerical 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
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 subcatchments 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-ADHYD

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 multiplied by a dampening factor to reduce the fluctuation in its numerical value, viz.,

\[(VAR)_1 = (VAR)_2 \left[ 1 - \gamma (0.9)^{ITR_{ITR}} \right] \quad (4.1)\]

where

\[(VAR)_1 = \text{numerical value of the variable after dampening process; } \]

\[(VAR)_2 = \text{numerical value of the variable as computed} \]
in the iteration being considered;
and \( ITR = \text{iteration index} \).

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.,

\[
(VAR)_{ITR} = (VAR)_{I} + (VAR)_{ITR-1}
\]  \( (4.2) \)

where

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

\( (VAR)_{I} \) = 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 subroutine "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 subroutine.

**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 midpoint 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-DPl**
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 contributing 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 subcatchment.

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 discharge 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 coeffic-
ients 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 $d_0$, $c_i$ and $d_i$.

(i) Number of Houses Along Sewer Sections:
Figure 5.8 shows a typical variation of average annual probable damage, $d_0$, with number of houses along sewer sections. It indicates that $d_0$ 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 subcatchments. 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 surcharging 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 watersheds. 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 main-line 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 backwater 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
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 hydrograph, I

\[ i = I \times p \]

estimate total travel time

\[ T = t_i + \frac{L}{v_{\text{full}}} \]

compute time vs contributing area curve

\[ a_i = \frac{A}{T} \times \Delta t \]

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.

add upstream hydrograph and route hydrograph through pipe segment

\[ \frac{\Delta t}{2} \left( Q_{\text{in}} + Q_2 \text{ in} - Q_1 \text{ out} \right) + S_1 \]
\[ = \frac{\Delta t}{2} Q_2 \text{ out} + S_2 \]

No. All pipes considered?

Yes

Outlet Hydrograph

\[ i = \text{effective rainfall} \]
\[ I = \text{initial rainfall} \]
\[ p = \text{reduction factor} \]
\[ T = \text{total travel time} \]
\[ t_i = \text{inlet time} \]
\[ L = \text{length of pipe segment} \]
\[ v_{\text{full}} = \text{full-flow velocity} \]
\[ a_i = \text{incremental area} \]
\[ A = \text{total impervious area} \]
\[ \Delta t = \text{time increment} \]
\[ Q = \text{runoff hydrograph} \]
\[ Q_{\text{in}} = \text{inflow hydrograph} \]
\[ Q_{\text{out}} = \text{outflow hydrograph} \]
\[ S = \text{pipe storage from constant depth assumption or discharge-storage relationship} \]

FIGURE I.1 ALGORITHM OF THE RRL MODEL
(After Reference 20)
<table>
<thead>
<tr>
<th>Model</th>
<th>Surface Routing</th>
<th>Severe Routing</th>
<th>Quality Routing</th>
<th>Degree of Sophistication of Surface Flow Routing</th>
<th>Degree of Sophistication of Severe Flow Routing</th>
<th>Degree of Sophistication of Quality Routing</th>
<th>Accurate Modeling of Surcharge</th>
<th>Flexibility Modeling of Modeling of In-System Components</th>
<th>Explicit Modelling of Treatment</th>
<th>Modelling</th>
<th>Available Calibration/ Verification</th>
<th>Data Requirements</th>
<th>Simulation Period</th>
<th>Published Documentation</th>
<th>Data Requirements</th>
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<td>EPA-SKMS</td>
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<td>Yes</td>
<td>Yes</td>
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<td>Moderate</td>
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<td>Yes</td>
<td>Yes</td>
<td>Moderate</td>
<td>Individual storms</td>
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<td>Extensive</td>
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<tr>
<td>Cincinnatil (UCER)</td>
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<td>Yes</td>
<td>Yes</td>
<td>High</td>
<td>Low</td>
<td>Low</td>
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<td>No</td>
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<td>Low-Moderate</td>
<td>No</td>
<td>Low</td>
<td>No</td>
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<td>Moderate</td>
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<td>Low-Moderate</td>
<td>Low-Moderate</td>
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<td>NA</td>
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<td>Individual storms</td>
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<td>Moderate</td>
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<td>High</td>
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<td>High</td>
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<td>No</td>
<td>Yes</td>
<td>Moderate</td>
<td>Individual storms or (separate) long term</td>
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<td>Extensive</td>
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<td>NA</td>
<td>NA</td>
<td>NA</td>
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<td>NA</td>
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<td>Rational Method</td>
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<td>Moderate</td>
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<td>No</td>
<td>No</td>
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<td>Individual storms</td>
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<td>Yes</td>
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<td>Yes</td>
<td>High</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
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<td>Individual storms</td>
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<td>Individual storms</td>
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<td>Extensive</td>
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<td>High</td>
<td>No</td>
<td>Low</td>
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<td>NA</td>
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<td>Moderate</td>
<td>Individual storms</td>
<td>Good</td>
<td>Extensive</td>
<td></td>
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* proprietary

**TABLE 1.1 COMPARISON OF URBAN RUNOFF MODELS**
*(After Reference 20)*
<table>
<thead>
<tr>
<th>Model</th>
<th>Routing Technique</th>
<th>Integration Scheme</th>
<th>Does the model include?</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Backwater effects</td>
<td>Surcharge</td>
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<tr>
<td>MWIS</td>
<td>Muskingum combined with the momentum equation with the inertia terms excluded.</td>
<td>Implicit for continuity, explicit for momentum Newton Raphson</td>
<td>No</td>
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<tr>
<td>SWMM of EPA</td>
<td>Quasi-steady dynamic-wave approximation of the St. Venant equation.</td>
<td>Explicit modified Euler method</td>
<td>No</td>
</tr>
<tr>
<td>WRE</td>
<td>( \frac{\partial Q}{\partial t} = -gA_s f + 2V \frac{\partial A}{\partial t} - gA \frac{\partial h}{\partial x} ) [ \frac{\partial h}{\partial t} = \frac{\partial Q t}{A_{st}} ]</td>
<td>Implicit scheme</td>
<td>Yes</td>
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<tr>
<td>IVM</td>
<td>St. Venant equation with lateral flow</td>
<td>Explicit first order characteristics scheme</td>
<td>Yes</td>
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<tr>
<td>SS</td>
<td>St. Venant equations without lateral flow</td>
<td></td>
<td>No</td>
</tr>
<tr>
<td>RL</td>
<td>Hydrologic reservoir routing technique</td>
<td></td>
<td>No</td>
</tr>
<tr>
<td>CUR</td>
<td>Time offset technique</td>
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<td>No</td>
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</table>

**TABLE I.2 COMPARISON OF SELECTED RUNOFF MODELS**  
(After Reference 20)
<table>
<thead>
<tr>
<th>Design of sewers</th>
<th>Conduit shapes handled</th>
<th>Structures considered</th>
<th>Availability</th>
<th>Remarks</th>
</tr>
</thead>
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<tr>
<td>It has an optimization technique</td>
<td>Circular and Trapezoidal</td>
<td>None</td>
<td>Nonproprietary</td>
<td></td>
</tr>
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<td>No</td>
<td>Circular, Semi-elliptical, egg shaped</td>
<td>Pumps, flow dividers, Internal storage units (weirs &amp; orifices)</td>
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<tr>
<td>No</td>
<td>Rectangular, circular horseshoe, basket-handle, eggshape, trapezoidal.</td>
<td>Weirs, pumps, tide gates, orifices</td>
<td>Proprietary</td>
<td>Studied presently by the University of Florida.</td>
</tr>
<tr>
<td>No</td>
<td>Any shape, by inputting the area-depth relationship</td>
<td>Retention rectangular basins, weirs</td>
<td>Proprietary</td>
<td></td>
</tr>
<tr>
<td>Yes</td>
<td>Circular</td>
<td>None</td>
<td>Nonproprietary</td>
<td>Studies are carried on to add a surcharge subroutine</td>
</tr>
<tr>
<td>No</td>
<td>Any shape, by inputting the area-depth relationship</td>
<td>None</td>
<td>Nonproprietary</td>
<td></td>
</tr>
<tr>
<td>No</td>
<td>Circular and rectangular</td>
<td>None</td>
<td>Nonproprietary</td>
<td></td>
</tr>
</tbody>
</table>

**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:

Maximize \[ F = C_1X_1 + C_2X_2 + \ldots + C_NX_N \]  \hspace{1cm} (II.1)

Subject to \[ A_{i1}X_1 + A_{i2}X_2 + \ldots + A_{iN}X_N \leq B_i \]

\[ = B_i \hspace{1cm} i = 1, 2, \ldots, M \]  \hspace{1cm} (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 SIMPLEX.

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.
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. Words 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 S

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 Delimiter 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
* THIS PROGRAM OBTAINS OPTIMUM DESIGN FOR PUMPED SEWER SYSTEMS.

* MAXIMUM NUMBER OF INLET JOINTS (MANHOLES) = 4
* MAXIMUM NUMBER OF SEWERS = 3

COMMON / AREA4A/ A, B, C, R
COMMON / AREA7/ AA, BB, CC, RR
COMMON / AREA71/ PROB
COMMON / AREA72/ NSTM
COMMON / AREA73/ A1, D1, C1
COMMON / AREA74/ CAM
COMMON / AREA75/ AD
COMMON / AREA76/ TCAM
COMMON / AREA82/ AC1
COMMON / AREA83/ NCAM
COMMON / AREA84/ C(5, 3)
COMMON / AREA85/ TANC
COMMON / AREA86/ COST(5)
COMMON / AREA87/ CINITL
DIMENSION ASA(7), BSE(7), SCC(7), RR(7)
DIMENSION FPC(7), CAM(7)

READ INPUT DATA

CALL INPUTD
CALL FLVESAS
CALL EQ I = 1, NSTM
A = AA(I)
B = BB(I)
C = CC(I)
R = RR(I)
CALL RAIFE
CALL INFIND
CALL PRNT2

BALANCE THE DESIGN FOR THE DESIGN STORM

IF I = EQ 1 CALL HYCMR

EVALUATE THE TOTAL DAMAGE FOR THE BALANCED DESIGN FOR VARIOUS STORMS (DIFFERENT RETURN PERIODS).

CALL HYCMOD
CAM(I) = TCAM
CONTINUE
CALL ANCOST
CINITL = TANC

GENERATE THE DAMAGE-PROB. EQUATION FOR THE ORIGINAL BALANCED DESIGN.

CALL CAMP4C

GENERATE THE AVERAGE ANNUAL PROBABLE DAMAGE ARRAY.
DC CO I = 1.5
C(1,1) = AD
C(1,3) = 0.5* A1
60 CONTINUE
C STORE THE ORIGINAL BALANCED DESIGN
C CALL STORE
DC 100 NCHNG=1.5
C PERTURB THE BALANCED SYSTEM WITH RESPECT TO THE I-TH
C DECISION VARIABLE.
C CALL CHANGE
C APPLY THE REPRESENTATIVE STORM TO THE PERTURBED SYSTEM
A = AA(2)
B = BB(2)
C = CC(2)
R = RP(2)
CALL RAINF
CALL INFHYC
CALL HYMOD
C AM(2) = TQAM
C COMPUTE THE AVERAGE ANNUAL PROBABLE DAMAGE FOR THE
C PERTURBED SYSTEM.
C CALL CPI
D(NCHNG,2) = AC1
C CALL ANCOST
C GSTP(NCHNG) = TANC
C RESET THE MODEL
C CALL ORIGIN
100 CONTINUE
C PRINT OUT THE DATA NECESSARY FOR THE OPTIMIZATION
C ALGORITHM. CO, CO, A
C PRINT 1C00, CINITL, AC, A1
C CALL GRAD1
C CALL COAD1E
C CALL PRINT1
1000 F FORMAT('///10X,"ANNUAL COST OF INSTALLATION",/" LUX,
C AVERAGE ANNUAL PROBABLE DAMAGE FOR THE BALANCED SEWER SYSTEM=CO=F12.1/,///
C AVERAGE ANNUAL PROBABLE DAMAGE",/10X,
C RESULTING FROM THE BALANCED SEWER SYSTEM=CO=5*,F12.1/,///
C ULTIMATE POSSIBLE DAMAGE RESULTING",/10X,
C FROM THE BALANCED SEWER SYSTEM=A=5*,F12.1/,///
C STOP
C ENDC

SUBROUTINES INPUT
********************************************************************
* THIS SUBROUTINE READS IN ALL THE INPUT DATA
* ********************************************************************
* INPUT DATA CARDS ARE AS FOLLOWS : 
* 1ST CARD (( NUMBER OF MANHOLES, I3 )) 
* AMH...CARDS(FOR EACH MANHOLE: IMPERVIOUS AREA (ACRES), 
* COEF. OF INLET CATCH BASIN (CB), EFFECTIVE 
* NO. OF VEHICLE CROSSINGS AT MANHOLE INTER 
* SECTION PER HOUR ON THE AVERAGE BASIN) 
* FORMAT USED 1F10.0 
* AMH...CARDS((FOR EACH MANHOLE: IMPERVIOUS AREA (ACRES) 
* TIME OF CONCENTRATION OF SUBCATCHMENT AREA 
* (MINUTES), FORMAT USED 2F6.3) 
* CNE CARC ((TIME INCREMENT (MINUTES), FA.1 )) 
* NPIPE...CARDS((FOR EACH PIPE: MANNING'S N, LENGTH, 
* DIAMETER, SLOPE, FORMAT USED 4F10.4)) 
* NPIPE...CARDS((FOR EACH PIPE: AREA OF BASEMENTS (ACRES) 
* PRINT OUT THE PIPES CONTRIBUTING TO THE PIPE AVERAGE 
* DEPTH OF BASEMENTS BELOW G.S. (FT), 
* DISTRICT INDEX, FORMAT USED 2F10.0, I1) 
* CNE CARC ( NUMBER OF STORMS CONSIDERED, FORMAT 1 3) 
* NSTORM CARDS((FOR EACH STORM: A, B, C, R, PROB, FORMAT 5F10.0 
* CNE CAR C (EASY FLOW DISCHARGE, CFS, FORMAT 5F10.0, SIZE)
DIMENSION AA(7), BB(7), CC(7), RR(7), PR0H(7),
      NN(7), CCN(3), ALPIPE(3),
      P(4), ABASE(3), DBSMT(3),
      AN(3), ALNTM(3), DIAM(3), SC(3),
      ARIHM(4), ET(4),
      CINLET(220),
      VEHH(4),
      READ 1980, NMH
      DC 16 I = 1, NMH
16 READ 1985, *1(I), CB(I), VEHH(I)
      NP1PE = NMH - 1
      DC 18 I = 1, NP1PE
18 VEH(I) = VEHH(I)
      DO 14 I = 1, NMH
14 READ 1990, *AFIMP(I), ET(I)
      READ 2000, T
      CC 12 I = 1, NP1PE
      READ 1900, AN(I), ALNTM(I), DIAM(I), SO(I)
      CONTINUE
      DO 11 I = 1, NP1PE
11 READ 2010, ABASE(I), CBSMT(I), ICLASS(I)
      CONTINUE
      READ 1980, NSTCRM
      DC 500 I = 1, NSTCRM
      READ 1800, AA(I), BY(I), CC(I), RF(I), FOCKS(I)
      CONTINUE
      READ 2100, QBAFL
      DO 10 I = 1, 220
10 GINLET(I) = QBAFL
      DC 100 I = 1, NP1PE
100 READ 2300, ALPIPE(I), CHOCN(I), NHCLS(I)
      READ 2400, RILETA, HMC, EA
      READ 2510, ASLMP, ECANL
      READ 2500, USELEV
      ENDSEW = USELEV
      DO 50 I = 1, NP1PE
50 ENDSEW = ENDSEW - SO(I) * ALNTM(I)
      THE SUMP LOWEST W.L. WILL BE SET 3.0 FT BELOW THE W/S
      INVERT OF THE LAST PIPE (DEFAULT VALUE)
      SECTCM = ENDSEW - DIAM(NP1PE) - 3.0
      READ 2500, (CHGSCS(I), I = 1, 5)
      READ 2500, (CHGSCG(I), I = 1, 5)
      CONTINUE
      FORMAT STATEMENTS FOR INPUT DATA
      1800 FORMAT(SF10.0)
      1900 FORMAT(4F10.0)
      1580 FORMAT(I3)
      1585 FORMAT(3F10.0)
      1990 FORMAT(2F6.0)
      2000 FORMAT(F20.0)
      2010 FORMAT(2F10.0, I1)
      2100 FORMAT(F10.0)
      2300 FORMAT(2F10.0, I2)
      2400 FORMAT(3F10.0)
      2410 FORMAT(2F10.0)
      2500 FORMAT(SF10.0)
      RETURN
      END
SUBROUTINE PRINT1
*****************************************************************************
* THIS SUBROUTINE IS USED TO PRINT OUT THE INPUT DATA*           
* (IF DESIRED). ALSO IT PRINTS AND PLOTS THE RAINFALL:*           
* HYETOGRAPH.                                                     
*****************************************************************************
COMMON/AREA/ T
COMMON/AREA0/ A, B, C, R
COMMON/AREA0E/ RAIN
COMMON/AREAOC/ SPAN
COMMON/AREA;/ PRINT, PRINTN
COMMON/AREA;/ SO
COMMON/AREA;/ AN
COMMON/AREA;/ CIAM
COMMON/AREA;/ NP1PE
COMMON/AREA;/ ALNTF
COMMON/AREA;/ ABASE
COMMON/AREA;/ P
COMMON/AREA;/ EBSMT
COMMON/AREA;/ HCLCN, ALPID
COMMON/AREA;/ NHCUS
COMMON/AREA;/ ASUMP
COMMON/AREA;/ ECHANL
COMMON/AREA;/ SBOTCM
COMMON/AREA;/ RELETA, MR, QR
COMMON/AREA;/ ENUSQ
COMMON/AREA;/ EC(4)
DIMENSION RAIN(220), PRINT(220)
DIMENSION AN(3), ALNTH(3), DIAM(3), SO(3)
DIMENSION FRIMP(4), ET(4)
DIMENSION P(4), CESMT(3), ABASE(3)
DIMENSION UHCC(3), ALPID(3), NHOLS(3)
INTEGER SPAN, SPAN1
FLAG = 0.0
FLAG1 = 1.0
IF (FLAG1.EQ.0.0) GO TO 100
PRINT 3000
PRINT 1000
PRINT 8000
DC 10 I = 1
10 PRINT 8010, I, R(I), ANIMP(I), IT(I), CI(I)
PRINT 1400
PRINT 8020
DC 20 I = 1, NPIPE
20 PRINT 8030, I, AN(I), ALNTH(I), DIAM(I), SU(I)
PRINT 1000
PRINT 8040
DC 30 I = 1, NPIPE
30 PRINT 8050, I, NHCUS(I), ABASE(I), DBSMT(I)
PRINT 1030
PRINT 9030
DC 40 I = 1, NPIPE
40 PRINT 8070, I, ALPID(I), UHCC(I)
PRINT 1000
PRINT 8090, RELETA, QR
PRINT 1000
PRINT 5600, SURF, SBOTCM
PRINT 1000
PRINT 3010, T
PRINT 1000
PRINT 4010
PRINT 4020, T, E, C
PRINT 4030, R
PRINT 1000
C CONTINUE
IF (FLAG2 = 0)
IF (IFLAG2 = 0) GO TO 200
SPAN1 = SPAN + 1
GO TO 1100, I = 1, 220
PRINT(I) = 0.0
PRINT(I) = 0.0
DO 45 I = 1, SPAN
IK = I*T - T
PRINT(I) = IK
PRINT(I) = RAIN(I)
45 PRINT(3020, IK, RAIN(I))
CALL PLOT 3 (PRINT, PRINTO, 100)
CONTINUE

FORMAL STATMENTS FOR PRINTING

1000 FORMAT (10X, 'A
3010 FORMAT (10X, 'TIME INSPECTED THROUGHOUT THE PROGRAM', 
A
3020 FORMAT (10X, 'PRINTOUT OR INPUT DATA', 
A
4020 FORMAT (10X, 'RELATIVE ADVANCEMENT OF THE PEAK OF THE SYNTHETIC', 
A
5020 FORMAT (10X, 'STOP', 
A
6020 FORMAT (10X, 'TIME =', 
A
7020 FORMAT (10X, 'RAIN INTENSITY =', 
A
8020 FORMAT (10X, 'MH NO.', 
A
9020 FORMAT (10X, 'PIPE NO.', 
A
100 FORMAT (10X, 'PIPE No. N. USERS ARE SEE (Ft)', 
A
110 FORMAT (10X, 'DIAM. (FT) SLOPE', 
A
120 FORMAT (10X, 'DIAM. H. CON. (FT)', 
A
130 FORMAT (10X, 'DIAM. (FT)', 
A
140 FORMAT (10X, 'DIAM. (FT)', 
A
150 FORMAT (10X, 'DIAM. (FT)', 
A
160 FORMAT (10X, 'DIAM. H. CON. (FT)', 
A
170 FORMAT (10X, 'DIAM. (FT)', 
A
180 FORMAT (10X, 'DIAM. H. CON. (FT)', 
A
SUBROUTINE PRINT2

** THIS SUBROUTINE PRINTS OUT BASIC INFLOW HYDROGRAPHS
** AT INLET POINTS (MANHOLE JUNCTIONS).

CCMCN / AREA/ T
CCMCN / AREA07 MPH, ARIMP, ET
CCMCN / AREA2A / INFHD
DINSEICK INFHD(4, 220)
DIMENSION ARIMP(4), ET(4)
PEAL INFHD
IFLAGS = 0
IFLAGS = 1
IF( IFLAGS <= 0 ) GO TO 210
DO 190 IMP = 1, MH
PRINT 3040, IMP
190 I = 1.220
IK = I*T
74 PRINT 3050, IK, INFHD(I, I)
180 CONTINUE
185 CONTINUE
190 CONTINUE
210 CONTINUE
3040 FORMAT (10X, 'MANHOLE NUMBER =', *I3)
3050 FORMAT (10X, 'TIPL =', *I3, ' MINUTES', 
X10X, 'INFLOW DISCHARGE =', *F10.5, ' CFS')
RETURN
END
SUBROUTINE RAINF

* THIS SUBROUTINE IS USED TO GENERATE THE RAINFALL.
* INTENSITY ARRAY, ASSUMING STORM LENGTH OF 180 MINUTES
* SPAN = NUMBER OF TIME INTERVALS IN THE STORM.
* AREAS, ET, T HAVE BEEN DEFINED IN SUBROUTINE INPUT.

COMMON/AREA/T
COMMON/AREA0/A,B,C,R
COMMON/AREA0X/RAIN
COMMON/AREA0C/SPAN
DIMENSION RAIN(220)
INTEGER SPAN
INTEGER SPAN1
DO 15 I = 1, 220
   RAIN(I) = 0
   SPAN = 180.0/T
   BEFPK = T
   K = BEFPK/T
   C = RAINFALL INTENSITIES BEFORE PEAK.
   DO 25 I = 1, K
      RAIN(I) = A*((1.0-B)*(BEFPK/R)**C)+C/((((BEFPK/R)**2)*C)**2)
      BEFPK = BEFPK - T
   CONTINUE
   J = K + 1
   TA = 0.0
   SPAN1 = SPAN + 1
   C = RAINFALL INTENSITIES AFTER PEAK.
   DO 35 J = J, SPAN1
      RAIN(J) = A*((1.0-B)*(TA/(1.0-R)**C)*C/(((TA/(1.0-R)**C)**2))
      TA = TA + T
   CONTINUE
RETURN
END

SUBROUTINE INFHYD

* THIS SUBROUTINE COMPUTES THE ORDINATES OF THE INFLOW
* HYDROGRAPH, FLOWING THROUGH THE MANHOLES TO THE
* U/S END OF THE CONSIDERED PIPE.

INFHD(I, IT) = INFLOW DISCHARGE MANH NO. I AT TIME IT.
ET(I) = TIME OF CONCENTRATION OF THE I-TH BASIC
* SUBTREMEMENT, MINUTES.
AFIMP(I) = IMPERVIOUS AREA CONTRIBUTING TO MNH (I).

COMMON/AREA/T
COMMON/AREA0/AMPH, AFIMP, ET
COMMON/ARAIN/RAIN
COMMON/AREA0C/SPAN
COMMON/AREA2/INFHD
DIMENSION INFHD(4, 220), ET(4), AFIMP(4)
DIMENSION RAIN(220)
INTEGER SPAN
REAL INFHD
DO 55 J = 1, NMH
   DO 55 I = 1, 220
      INFHD(J, I) = 0.0
   DC 75 IMP = I*NMH
   NARING = ET(INMH)/T
   AFIMP = AFIMP(IMP)/NARING
   KS = 1.5*SPAN
   DO 75 I = 1, KS
55 CONTINUE
RETURN
END
SUM = 0.0
CC = 65 J = 1 + ARINC
KK = I - (J - 1)
KK1 = KK + 1
IP (KK) = 60, 60, 70
60 CALCA = 0.0
GO TO 64
RAIN(I) = THE RAINFALL INTENSITY AT THE BEGINNING OF
INTERVAL (I). AVERAGE RAINFALL INTENSITY DURING
INTERVAL (I) WILL BE (RAIN(I) + RAIN(I+1))/2.0
70 CALCA = (RAIN(KK) + RAIN(KK1))* ARINC /2.0
64 CONTINUE
55 SUM = SUM + CALCA
INFIC (1M + I) = SUM
75 CONTINUE
RETURN
END

SUBROUTINE NUSKAT
************************************************************
* THIS SUBROUTINE COMPUTES THE DOWNSPOKET DISCHARGE *
* FOR A CIRCULAR PIPE GIVEN THE INFLOW HYDROGRAPH *
* AND PIPE CHARACTERISTICS. *
************************************************************
* DIAM = DIAMETER IN FEET *
* ALNTH = PIPE LENGTH IN FEET *
* SO = PIPE SLOPE IN FT/FT *
* AN = MANNING'S N *
* T = TIME INCREMENT BETWEEN SUCCESSIVE ELEMENTS *
* KMUS = STORAGE CONSTANT IN KMUS KINGW METOD *
* CO, C1, C2 = MUSKINUM COEFFICIENTS *
* ( C0 = CO*I(I) + C1 * I(1) + C2 * 0(I) ) *
* QOUT = OUTFLOW DISCHARGE AT U/S END OF SEWER *
* QINF = INFLOW DISCHARGE THROUGH THE U/S END THE SEWER *
******************************************************
COMMCMN / AREA1/ T
COMMCMN / AREA2/ SO
COMMCMN / AREA3/ CINLET
COMMCMN / AREA4/ ALNTH
COMMCMN / AREA5/ DIAM
COMMCMN / AREA6/ AN
COMMCMN / AREA7/ IP
COMMCMN / AREA8/ QINF
COMMCMN / AREA9/ QOUT
COMMCMN / AREA10/ KMUS
DIMENSION AN(3), ALNTH(3), DIAM(3), SO(3)
DIMENSION QINF(3,220), QOUT(3,220)
DIMENSION KMUS(3)
REAL KMUS
X = 0.5
KMUS(IP) = AN(IP)*ALNTH(IP)*1.696/(DIAM(IP)**0.637)/
(SC(IP)**0.8)
T WILL BE IN FT TO THE SAME UNITS AS KMUS (SEC IN(0.3)
CC = KMUS(IP) - KMUS(IP)*X + 30.0*T
CC0 = (30.0* X - KMUS(IP) )/ 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 30
QOUT(IP, I) = QINF(IP, I)* C0 + C1 = QINLET(I) + C2*QINLET(I)
GO TO 110
50 CONTINUE
IT1 = IT - 1
QOUT(IP, IT) = QINF(IP, IT) * C0 + QINF(IP, IT1) * C1
+ QOUT(IP, IT1) * C2
110 CONTINUE
RETURN
END
SUBROUTINE HYDROC
******************************************************************************
* THIS SUBROUTINE IS USED FOR HYDRAULIC SIMULATION *
* CF FLOW IN THE SEWER SYSTEM *
******************************************************************************
COMMON /AREA14/ QBSMT
COMMON /AREA44/ IP
COMMON /AREA45/ QINF,COUT
COMMON /AREA8/ NPIPE
COMMON /AREA9A/ KMUS
COMMON /AREA9C/ M
DIMENSION QBSMT(3,220)
DIMENSION QINF(3,220),COUT(3,220)
DIMENSION KMUS(3)
REAL KMUS(3)
N = 0
DO 50 I = 1,NPIPE
50 FMX(I) = 0.0
CALL KENREG
CALL VCLSYS
DO 100 ITT = 1,180
ITT = ITT
IF( N .LT. C ) GO TO 105
DO 90 IPK = 1,NPIPE
IP = IPK
CALL ACDFHC
CALL MUSKMT
90 CONTINUE
CALL SUMP1
CALL SWITCH
100 CONTINUE
105 CONTINUE
DO 110 I = 1,NPIPE
DO 110 J = 1,220
OPSMT(I,J) = C
110 CONTINUE
IF( M .EQ. 0 ) GO TO 120
CALL PFFSL
120 CONTINUE
**SUBROUTINE SUMF1**

* THIS SUBROUTINE COMPUTES THE SUMP WATER LEVEL AND *
* THE ELEVATION OF THE C/S END OF THE C/L. OF THE NTH *
* PIPE. ( CUTOFF ENERGY LEVEL ) *
* FELETA = OPERATING EFFICIENCY/RATED EFFICIENCY *
* MP = RATEC HEAD (FT) *
* CR = RATEC DISCHARGE (CFM) *
* OSUMP1 = DISCHARGE FLOWING TO SUMP AT TIME (T - 1) *
* OSUMP = DISCHARGE FLOWING TO SUMP AT TIME (T) *
* CH1 = PUMPING HEAD AT TIME (T - 1) *
* CH2 = PUMPING HEAD AT TIME (T) *
* ECHANL = ELEVATION OF THE ENERGY LINE C/S OF THE *
* PUMP. ( DELIVERY ELEVATION ) COULD BE CHANNEL *
* WATER LEVEL *
* SBOTCM = ELEVATION OF MINIMUM SUMP WATER LEVEL START *
* ING AND STOPPING LEVEL *
* SWL = SUMP WATER LEVEL *
* ESUMP( ) = ELEVATION OF THE C/S END OF THE ENERGY *
* LINE OF THE NTH PIPE *

CCMCN / AREA / T
CCMCR / AREA2A / INFH
CCMCN / AREA2E / GINLET
CCMCN / AREA4E / QINFH
CCMCN / AREA6E / CIAM
CCMCN / AREA6E / NPIPE
CCMCN / AREA8E / IT
CCMCN / AREA9C / M
CCMCN / AREA20 / OSUMP
CCMCN / AREA28 / ASUMP
CCMCN / AREA29 / ECHANL
CCMCN / AREA30 / SBOTCM
CCMCN / AREA31 / RELETA,HR,OR
CCMCN / AREA32 / ENDSW
CCMCN / AREA40 / SWL-OSUMP
DIMENSION CIAM(3)
DIMENSION GINLET(220)
DIMENSION GCUT(3,220),GINFH(3,220)
DIMENSION INFH(4,220)
DIMENSION ESUMP(220)

IF ( T .EQ. 0 ) GO TO 30
K = IT
GO TO 35
30 K = IT
35 CCNTINUE
ANPIPE = 0.78544 DIAM(NPIPE)**2
CPUMP = FELETA * HR * OR
C = T * CPUMP * 30. / ASUMP
IF ( K .NE. 1 ) GO TO 40
OSUMP1 = GINLET(IN)
OSUMP = CUT(NPIPE,1) + INFH(NPIPE+1,1)
CH1 = ECHANL - SBOTCM
GO TO 50
40 OSUMP = CUT(NPIPE,K-1) + INFH(NPIPE+1,K-1)
OSUMP = CUT(NPIPE,K) + INFH(NPIPE+1,K)
CH1 = ECHANL - SWL
50 CCNTINUE
OSUMP = (OSUMP + USUMP1) / 20.
B = (-0.5)*HP + 2.0*C*OSUMP A/CPUMP - C/HP1
HP = (-0.5)*B + 0.5*SQRT(B**2 + 4.0*C)
HPMAX = ECHANL - SBOTCM
IF (HP .GT. HPMAX) HP = HPMAX
SWL = ECHNL - HP
VNPipeZ = QOUT(NPIPE,K) / ANPIPE
VHEAD = VNPIPE + VNPIPE/63.4.
IF(SWL.LT.ENCSEW) GC TO 10
ESUMP(K) = SWL + VHEAD
GO TO 100

100 CONTINUE
RETURN
END

SUBROUTINE STORM
******************************************************************************
* THIS SUBROUTINE COMPUTES THE STORM WATER FLOWING:
* UPWARD THROUGH THE I-TH MANHOLE BY APPLYING A MAJS:
* EQUATION:
******************************************************************************
* FWHL = FEAC LCSS THROUGH PIPE:
* CIN = INFLOW DISCHARGE TO MANHOLE:
* OOT = CUTFLOW DISCHARGE FROM MANHOLE:
* DVCLST(I) = INCREMENT OF VOLUME OF WATER FLOWING TO:
* STREET THROUGH THE I-TH MANHOLE:
******************************************************************************
COMMON AREA2/ T
COMMON AREA2A/ INFPC
COMMON AREA2B/ QINLET
COMMON AREA6E/ NPIPE
COMMON AREA8C/ KES
COMMON AREA8F/ ALNTH
COMMON AREA87/ KM
COMMON AREA88/ ITS
COMMON AREAl0/ USEL
COMMON AREAl4/ QSMT
COMMON AREAl8/ SE, BMAX, F,
COMMON AREAl8/ USUMF
COMMON AREAl8/ T
COMMON AREAl8/ GAV
COMMON AREAl8/ QSTR
DIMENSION USEL(3), H(4), IMP(3, 220)
DIMENSION ESUMP(220), ALNTH(3), JAY(3, 220)
DIMENSION SC(3, 220), KEJ(3), INFD(4, 220)
DIMENSION QINLET(220), QSMT(3, 220), QSTR(3)
DIMENSION DVCLST(3), F(3, 220)
REAL KES, INFPC
USEL(KM) = H(KM)
IF(KM.EQ.0, NPIPE) GG TO 100
KM2 = KM + 1
SEHWL = USEL(KM) - USEL(KM2)
EMF(KM, ITS) = (USEL(KM) + USEL(KM2))/2
GO TO 200

100 SEHWL = USEL(KM) - ESUMP(ITS)
EMF(KM, ITS) = (ESUMP(ITS) + USEL(KM))/2.0

200 CONTINUE
SE(I, ITS) = SEHWL / ALNTH(KM)
QAV(KM, ITS) = SQRT(ABS(SE(I, ITS) / KES(KM))) * SQRT(KM, ITS) / ABS(ABS(KM, ITS) - 1)
KM1 = KM - 1
SUMOST = 0.0
SUMIHD = QINLET(ITS)
DO 300 IM1 = 1, KM
300 SUMIHD = SUMIHD + INFPC (IM1, ITS)
IF(IM1.EQ.0) GO TO 300
SUM = 0.0
DO 400 IM2 = 1, KM
400 SUM = SUM + QSTP(IM2, ITS)
GO TO 600

500 SUMB = SUMB8 + QSMT(KM, ITS)
GO TO 600

600 CONTINUE
CIN = SUMIHD - SUMB8 - SUMCST
COT = QAV(KM, ITS) + QSTP(KM, ITS)/2.0
QSTR(KM) = CIN - COT
RETURN
END
SUBROUTINE PRESML

* THE FUNCTION OF THIS SUBROUTINE IS TO SIMULATE ...
* CONSIDER FLUID FLOW IN THE SEWER SYSTEM, TAKING INTO
* CONSIDERATION FLOW TO AND FROM DASHER INTS AS WELL AS*
* STREAMS (GROUND SURFACE).*
* OBSMT(I,T) = AVERAGE RATE OF DASHER INT FLOW FOR THE *
* I-TH SEWER DURING THE T-TH TIME STEP.CFS*
* SE(I,T) = SLOPE OF ENERGY LINE OF THE I-TH SEWER AT *
* T-TH TIME INDEX.*
* KES( ) = COEFFICIENT TO COMPUTE ENERGY SLOPE, IN THE*
* FORMULA, s = k* pi**2*
* QAV( ) = AVERAGE DISCHARGE FLOWING THROUGH THE *
* PIPE.*
* EMP( ) = ELEVATION OF THE MID-POINT OF THE ENERGY*
* LINE.*

COMMON / AREA1/ T
COMMON / AREA3/ SO
COMMON / AREA2/ INFHC
COMMON / AREA22/ GINLET
COMMON / AREA6/ QINF0, QOUT
COMMON / AREA7/ ALNTP
COMMON / AREA8/ KM
COMMON / AREA6/ NPIPE
COMMON / AREA6/ KES
COMMON / AREA8/ IT
COMMON / AREA6/ ITS
COMMON / AREA10/ USEL
COMMON / AREA14/ OBSMT
COMMON / AREA16/ SE, EMP,F
COMMON / AREA19/ BMP
COMMON / AREA20/ ESUMP
COMMON / AREA22/ F
COMMON / AREA24/ OPCSUS
COMMON / AREA25/ QAV
COMMON / AREA26/ QSTR
COMMON / AREA52/ C3(4)
COMMON / AREA40/ SWL, OSUMP
COMMON / AREA 41/ DFLCCD
COMMON / AREA43/ DMAX
COMMON / AREA46/ FMAX
COMMON / AREA8/ VMAX, TPEAK
DIMENSION TPEAK(3)
DIMENSION P(4)
DIMENSION QSTR(3), VC LST(3), DVOLST(3)
DIMENSION SO(3)
DIMENSION INFHC(4, 220), GINLET(220)
DIMENSION KES(3)
DIMENSION GAV(3, 220), OBSMT(I, 220)
DIMENSION USEL(3), EMP(I, 220)
DIMENSION BMP(3)
DIMENSION ESUMP(220)
DIMENSION SE(3, 220), ALNTH(3)
DIMENSION ADFLOC(3), DMAX(3), FMAX(3), VMAX(3)
DIMENSION F(3, 220)
DIMENSION GINF(I, 220), GOUT(3, 220)
REAL INFHC, 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)
QSTP(I) = 0.0
VOLST(I) = 0.0
DVCLST(I) = 0.0
DMAX(I) = 0.0
FMAX(I) = 0.0
VMAX(I) = 0.0
TPEAK(I) = 0.0
30 CONTINUE
DO 400 ITS = IT $ 220
  ITS1 = ITS - 1
  DC S0 IM = 1, NPIPE
  QBSMT(I$0, ITS) = QBSMT(IMC, ITS1)
  CONTINUE
  FSUMP(I$) = ESLMF(I$-1)
  DC 450 ITR1 = 1.3
  DC 410 ITR = 1.6
  DO 400 IM = 1, NPIPE
    KM = NPIPE- IM + 1
    GC = QSTR(KM)
    SUMHC = QINL & ET(I$)
    SUMPST = 0.0
    DC 160 IM1 = 1, KM
    SUMPST = SUMPST + QSTR(I$1)
    150 SUMHC = SUMHC + INFHC(I$1, ITS)
    KM1 = KM - 1
    IF ( KM1 .GE. 0 ) GC TO 250
    SUMPQ = 0.0
    DC 270 IM2 = 1, KM1
  200 SUMPQ = SUMPQ + QBSMT(I$2, ITS)
  GC TO 300.
  250 SUMPQ = 0.0
  300 CONTINUE
  QAV(KM, ITS) = SUMHC/2.0 - SUMQST
  SE(KM, ITS) = KES(KM)*QAV(KM, ITS)*AES(QAV(KM, ITS))
  IF ( I$ .GT. 1 ) GC TO 350
  LSEL(KM) = ESUMP(I$) + SE(KM, ITS)*ALNTK(M)
  EMP(KM, ITS) = (ESUMP(I$) + LSEL(KM))/2.0
  GC TO 375
  350 KM2 = KM + 1
  USEL(KM) = LSEL(KM2) + 3*(<1, ITS)*ALNTK(M)
  EMP(KM, ITS) = (USEL(KM2) + USEL(K4))/2.0
  CONTINUE
  IF (USEL(KM) .GE. LST(KM)) GC TO 330
  CALL STMOC
  GC TO 395
  380 IF (VCLST(KM) .GT. 1) GC TO 385
  QSTR(KM) = 0.0
  DVC(LST(KM) = 0.0
  GC TO 395
  385 QSTR(KM) = (-1.0)*CE(KM) * SCRT(ABS(VCLST(KM)))
  395 CONTINUE
  CALL FRTIC
  CALL GCHLSS
  XDFLOD(KM) = XFLCCD * 12.
  X = QSTR(KM, ITS)
  QBSMT(KM, ITS) = QHCUS((1.0-X)*9**ITR) ** ITW
  QBSMT(KM, ITS) = QBSMT(KM, ITS) + X )/2.0
  QSTR(KM) = QSTR(KM) * 1.0 * (D**ITR)**ITW
  QSTR(KM) = (QSTR(KM) + GQ)/2.0
  DVC(LST(KM) = QSTR(KM) * T * GQ.
  400 CONTINUE
  410 CONTINUE
  QOUT(NPIPE, ITS) = QAV(NPIPE, ITS) - QBSMT(NPIPE, ITS)/2.0
  Y = FSUMP(ITS)
  CALL SUMP1
  ESUMP(ITS) = (ESUMP(ITS) + Y) / 2.0
  450 CONTINUE
  DO 460 KI = 1, NPIPE
    IF ( VCLST(KI) .LE. 0.0 ) GC TO 451
    TPEAK(KI) = TPEAK(KI) + T
  451 CONTINUE
  VCLST(KI) = VCLST(KI) + DVC(LST(KI))
  460 CONTINUE
  DC 490 I = 1, NPIPE
  IF (DMAX(I, CE*XDFLOD(I)) GO TO 470
  DMAX(I) = XDFLOD(I)
  470 CONTINUE
  IF (FMAX(I) .GE. F(I, ITS) ) GO TO 480
  FMAX(I) = F(I, ITS)
  480 CONTINUE
IF (VMAX(I) .GE. VCLST(I)) GO TO 450
VMAX(I) = VCLST(I)
490 CONTINUE
IF (RECEISON LIMB IS ENCOUNTERED) RETURN TO SUBROUTINE
400 HYCMOD
410 MOD
NFLAG = 0
420 DO 500 I = 1, NPIPE
430 IF (GSESMT(I, ITS) .LT. XGHNF = NFLAG + 1
440 CONTINUE
450 IF (NFLAG .EQ. NPIPE) GO TO 550
460 IF (ITS .EQ. (IT + 4)) XG = 0.01
470 CONTINUE
480 CALL AHCAM
490 RETURN
550 CONTINUE
SUBROUTINE OFCLSS
******************************************************************************
* THIS SUBROUTINE COMPUTES THE AVERAGE RATE OF BASEMENT * 
* FLOW FOR I-TH SEWER DURING THE T-TH TIME STEP, CFS * 
******************************************************************************
* OFLOOD = FLOODING DEPTH OF THE BASEMENTS ASSOCIATED * 
* WITH THE I-TH SEWER, AT THE END OF THE T-TH * 
* TIME STEP, FT * 
* APIE = CROSS SECTIONAL AREA OF THE PIPE OF THE * 
* HOUSE CONNECTION, SQ.FT * 
* DHCON(I) = DIAMETER OF THE PIPE OF THE HOUSE * 
* CONNECTION, FT * 
* ALPIE(I) = AVERAGE LENGTH OF HOUSE CONNECTIONS * 
* WITHIN A PARTICULAR SEWER SECTION, FT * 
******************************************************************************
CCMNCN / AREA7/ I
CCMNCN / AREA8/ IT
CCMNCN / AREA9/ ITS
CCMNCN / AREA10/ SE*EMP,F
CCMNCN / AREA11/ T
CCMNCN / AREA12/ ALNTF
CCMNCN / AREA13/ USSEL
CCMNCN / AREA14/ SBASE
CCMNCN / AREA15/ ABASE
CCMNCN / AREA16/ QSMSMT
CCMNCN / AREA17/ BMP
CCMNCN / AREA18/ CHOS
CCMNCN / AREA19/ DCON, ALPIE
CCMNCN / AREA20/ NPCUS
CCMNCN / AREA21/ DFLOOD
DIMENSION SEASE(I), ALNTH(I)
DIMENSION AFASE(I)
DIMENSION CGBMT(I, 3, 220)
DIMENSION USSEL(I)
DIMENSION EMF(I, 3, 220), SE(I, 220), F(I, 3, 220)
DIMENSION ALPIE(I), DHCON(I)
DIMENSION EMF(I)
DIMENSION NPCUS(I)
APIE = D+CCCA(I) ** 2.0 * 0.7554
A = 9.02 * APIE * NPCUS(I) / SQRT(T2.0 + 0.03 * ALPIE(I) / DHCON(I))
SQSMSMT = 0.0
CC = 50.0 * IT, ITS
50 SQSMSMT = SQSMSMT + SQSMSMT(I, M)
FLVCL = SQSMSMT * T * 60.
DFLOOD = FLVCL / ABASE(I) / 43560.
IF (DFLOOD) 60, 70
60 DFLCDD = 0.0
70 CONTINUE
ENBAS = EMF(I) + DFLCDD
USEBL = ENBAS + SBASE(I) * ALNTH(I) / 2.0
DSEBL = ENBAS - SBASE(I) = ALNTH(I) / 2.0
USELI = USELI(I)
USHL = INSL1 - USRL
DSRL = INSL2 - USRL
SHL = USRL - DSRL
IFF(I,ITS) 1CC, 1CC*2CO
100 IF(CDFLOCC GT 0.0) GU TO 110
 
CHUS = 0.0
GC TO 460
110 R = (ABS(DSRL)*1.5 - (ABS(DSRL)**1.5))/1(ABS(USSH) - ABS(DSRL))
CHUSF = (1.0 - 0.567*A*A)*B
IFF(FLVLCC - ABS(CHCLSC)**T = 60.) 120, 130 TT 30
120 CHUS = FLVLCC/T/60. *(-1.0)
130 GC TO 460
200 IF(F(I,ITS) LT 1.0) GO TO 300
 
CHUS = USRL/(USRL + ABS(DSRL))
GO TO 180
160 IF(DSRL LT 0.0) GC TO 170
 
CHUS = USRL/ABS(USRL)
GO TO 180
170 R = (ABS(USRL)*1.5 - (ABS(USRL)**1.5))/1(ABS(USRL) - ABS(DSRL))
CHUS = 0.667*A*B*USRL/ABS(USRL)
GO TO 460
180 F(I,ITS) = FT
300 IF(USSH) 120, 220, 220
210 IF(DSRL LT 0.0) GO TO 110
 
CHUSP = 0.667*F(I,ITS)*SORT(DSRL)*A
CHUSN = 0.667*(1.0-F(I,ITS))*SORT(ABS(USSH))*A
GO TO 230
220 CHUSP = 0.667*F(I,ITS)*SORT(ABS(USSH))*A
CHUSN = 0.667*(1.0-F(I,ITS))*SORT(ABS(DSRL))*A
230 CONTINUE
 
VGLAV = DFLGCC*(1.0-F(I,ITS))*ABS(SE(I))**43560.
IFF(VGLAV = CHUSN**T = 60.) 270, 290, 290
270 CHUS = VGLAV/T/60.
280 CONTINUE
CHUS = CHCLSC - QFOLSN
400 CONTINUE
RETURN
END

SUBROUTINE KENEFG
******************************************************************************
* THIS SUBROUTINE COMPUTES THE COEFFICIENT OF THE
* ENERGY SLOPE EQUATION, BASED ON MANNING'S EQUATION
******************************************************************************
COMMON / AREA5A/ AN
COMMON / AREASE/ DIAM
COMMON / AREA6E/ NPIPE
COMMON / AREA6C/ KES
DIMENSION KES(3), DIAM(3), AN(3)
REAL KES
GO TO 100
KES(I) = 1.0*NPIPE
KES(I) = 4.6369 * AN(I) ** 2/(DIAM(I) ** (16./3.))
GO TO 100
CONTINUE
RETURN
END
SUBROUTINE FHATIC
**************************************************************************************************
* IT COMPUTES THE RATIO BETWEEN THE LENGTH OF THE SEWER *
* ACTIVELY BEING FLOODED AND ITS TOTAL LENGTH. i.e., *
* BASEMENT FLOODING RATIO ( ) *
**************************************************************************************************
* BMP( ) = MIC-POINT ELEVATION OF BASEMENTS GRADE LINE *
* SBASE( , ) = SLOPE OF THE BASEMENTS LINE ASSOCIATED WITH PIPE (I) *
* F( I, IT)= FRACTION OF THE I-TH SEWER CONTRIBUTING TO *
* BASEMENTS STORAGE DURING THE T-TH TIME STEP *
* USEL( ) = UPSTREAM ELEV. OF ENERGY LINE *
* DSEL = DOWNSTREAM ELEV. OF ENERGY LINE *
* USBL = UPSTREAM ELEV. OF BASEMENTS GRADE LINE *
**************************************************************************************************
COMMON / AREA7E / ALNTH
COMMON / AREA7C / IP
COMMON / AREA8C / IT
COMMON / AREA19 / 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)
CSB = EMP(IP) - SBASE(IP) * ALNTH(IP) / 2.0
USBL = EMP(IP) + SEASE(IP) * ALNTH(IP) / 2.0
IF( USEL(IP), GT, USBL) GO TO 50
IF(IP(SEL + GT, CSBL)) GO TO 50
F(IP, IT) = 0 + C
GO TO 300
50 CONTINUE
FLOAT = SE(IP, IT) - SBASE(IP)
IF(FLOAT, GT, 0.0) GO TO 60
F(IP, IT) + 1.0
GO TO 200
60 CONTINUE
C = (EMP(IP, IT) - EMP(IP)) / (SE(IP, IT) - SBASE(IP) * ALNTH(IP))
IF(SE(IP, IT), GT, SBASE(IP)) GO TO 100
F(IP, IT) + C - C
GO TO 200
100 F(IP, IT) = 0 + C
200 CONTINUE
IF(F(IP, IT, GT, 1.0)) F(IP, IT) = 1.0
300 CONTINUE
RETURN
END

SUBROUTINE SWITCH
*************************************************************************
* THIS SUBROUTINE DETERMINES WHETHER THE FLOW IS *
* GRAVITY FLOW (OPEN CHANNEL FLOW), OR SUPERCHARGED FLOW *
* (PRESSURIZED FLOW ). *
*************************************************************************
* STOP = VUL. OF WATER INTO PIPE STORAGE *
* STGS = VUL. OF WATER INTO SYSTEM STORAGE (PIPES ONLY) *
* FELESTG = RELATIVE STORAGE: *
* U = VUL. OF WATER INTO STORAGE/ VUL. OF PIPES *
* N = FLAG: *
* = 0 FOR GRAVITY FLOW *
* = 1 FOR PRESSURIZED FLOW *
*************************************************************************
COMMON / AREA6E / NPIPE
COMMON / AREA4C / QINF,2JUT
COMMON / AREA7C / IP
COMMON / AREA8C / IT
COMMON / AREA9C / M
COMMON / AREA16 / KMUS
COMMON / AREA16 / VOLS
COMMON / AREA51 / VPIPE(3)
SUBROUTINE VOLSYS

* THIS SUBROUTINE COMPUTES THE VOLUME OF THE PIPES IN THE SYSTEM, REPRESENTING THE MAXIMUM PIPE STORAGE *

* VOL = VOLUME OF PIPES IN THE SYSTEM, CUBIC FEET *

COMMON /AREAS/ DIAM
COMMON /AREAG/ NPIPE
COMMON /AREAS/ ALNTF
COMMON /AREAG/ VOL
COMMON /AREAV/ VPIPE(3)
DIMENSION ALNTF(3), DIAM(3)
VOL = 0.0
DC 100 I = 1, NPIPE
VPIPE(I) = 0.785 * (DIAM(I)**2) * ALNTF(I)
VOL = VOL + VPIPE(I)
100 CONTINUE
RETURN
END

SUBROUTINE ELEVEAS

* THIS SUBROUTINE COMPUTES THE ELEVATION OF THE MID-POINT OF THE BASEMENTS GRADE LINE FOR EACH SEWER. IT ALSO COMPUTES SLOPE OF THE BASEMENTS GRADE LINE FOR EACH SEWER SECTION. *

* SBASE(I) = SLOPE OF THE BASEMENTS GRADE LINE OF THE I-TH SEWER; *
* DBST(I) = AVERAGE DEPTH OF BASEMENTS BELOW THE GROUND SURFACE ASSOCIATED WITH THE I-TH SEWER *

COMMON /AREA1/ SBASE
COMMON /AREA1/ BMP
COMMON /AREA2/ CBSMT
COMMON /AREA3/ ALNTF
COMMON /AREA2/ H
COMMON /AREA3/ NPIPE
DIMENSION F(4), SBASE(3), BMP(3)
DIMENSION ALNTF(3), CBSMT(3)
DO 100 I = 1, NPIPE
SBASE(I) = (H(I) - H(I+1)) / ALNTF(I)
BMP(I) = (H(I) + H(I+1))/2.0 - CBSMT(I)
100 CONTINUE
RETURN
END

COMMON /AREAS/ PMAX(3)
DIMENSION CINF(2,20), NOUT(?), 220)
DIMENSION KUS(2)
DIMENSION STOP(3), PRELS(3)
REAL KMLS
STGS = 0.0
DO 50 IPI = 1, NPIPE
STOP(IPI) = KMLS(IP) * (CINF(IP),IQ) + GOUT(IP),IT) * 0.0
PRELS(IP) = STOP(IP) / VPipe(IP)
PMAX IS A PARAMETER TO BE USED IN THE BALANCING PROCEDURE IF(PRELS(IP) + GT + PMAX(IP)) * PMAX(IP) = PRELS(IP)
STGS = STGS + STOP(IP)
50 CONTINUE
PRESTG = STGS/VCL
IF( PRESTG .LT .0.0 ) go to 200
M = 1
go to 200
200 CONTINUE
RETURN
END
SUBROUTINE CPIPES

* THIS SUBROUTINE COMPUTES THE TOTAL INSTALLATION COST OF ALL THE SEWERS IN THE SYSTEM.

* LSPL(1) = CROWN ELEV. CF THE U/S END OF THE I TH PIPE
* DSPL(1) = CROWN ELEV. OF THE D/S END OF THE I TH PIPE
* H1 = U/S DEPTH OF EXCAVATION
* H2 = D/S DEPTH OF EXCAVATION
* HAV = AVERAGE DEPTH OF EXCAVATION
* COSTP(I) = COST PER UNIT LENGTH OF THE I TH PIPE
* A, B, C = COEFFICIENTS IN THE PIPE-COST EQUATION
* COST = A + B*DIAM**2 + C*H*H
* COSTPS = TOTAL COST OF PIPES IN SYSTEM

COMMON /AREA/ SO
COMMON /AREA2/ CIAM
COMMON /AREAGE/ NPIPE
COMMON /AREA7/ ALNTF
COMMON /AREA22/ H
COMMON /AREAGE/ USELEV
COMMON /AREA5/ COSTPS
DIMENSION LSPL(3), DSPL(3), COSTP(3)

THE FOLLOWING DEFAULT VALUES ARE BASED ON 1973 PRICES (EASE YEAR)
A = 2.8
B = 2.0
C = 0.045
LSPL(1) = USELEV - CIAM(1)
DO 100 K = 1, NPIPE
IF (K NE. 1) LSPL(K) = DSPL(K-1) + CIAM(K-1) - CIAM(K)
USPL(1) = USELEV - CIAM(1)
H1 = H(K) - LSPL(K)
H2 = H(KP) - DSPL(K)
HAV = (H1 + H2)/2.0
COSTP(K) = A*H1 + B*CIAM(K)**2 + C*HAV*HAV
100 CONTINUE
COSTPS = 0.0
DO 200 K = 1, NPIPE
COSTPS = COSTPS + COSTP(K)*ALNTF(K)
200 CONTINUE
RETURN
END

SUBROUTINE STCDAMG

* THIS SUBROUTINE ESTIMATES GROUND SURFACE DAMAGES, INCLUDING INCREDIBLE AND PROPERTY DAMAGES.

* VEHI(I) = EFFECTIVE NUMBER OF VEHICLE CROSSINGS AT THE I-TH MANHOLE INTERSECTION PER HOUR, UN.
* THE AVERAGE BASIS.

* ICLASS(I) = DISTRICT INDEX
* = 0 (RESIDENTIAL)
* = 1 (COMMERCIAL)
* CT = FACTOR TAKES INTO ACCOUNT TYPE OF DISTRICT
* DEFAULT VALUES ARE 2.0 (COMMERCIAL) & 1.0 (RESIDENTIAL)
* CL = CALIBRATION FACTOR, FRACTION OF THE MAX.
* DAMAGE (DEFAULT VALUE = 1.0). 1 FT DEPTH
* CF WATER COVER FLOODED AREA CORRESPOND TO
* 10G X DAMAGE.

COMMON /AREA/ ALNTF
COMMON /ARC26/ NHOLES
COMMON /AREAGE/ VMAX, TP1K
COMMON /AREA49/ ICLASS(1), VEH(3)
COMMON /AREA47/ ASS(1), AF
COMMON /AREA44/ K
COMMON /AREA50/ STEAM
DIMENSION ALNTH(3), NHOUS(3)
DIMENSION VMAX(3)
DIMENSION TPEAK(3)
CL = 0.5
STEAMI = (1./360.) * VEH(K) * TPEAK(K) * 2.0
IF (ICLASS(K) .eq. 0) GO TO 100
CT = 2.0
GC TO 200
100 CT = 1.0
200 CONTINUE
IF (K .ne. 1) GC TO 300
ASURF = ALNTH(1) * 60.0
WIDTH_CFFSTREET = 60.0 FT (DEFAULT VALUE)
STEAMP = CL * ASS(1) * AF * NHOUS(1) * CT * VMAX(1) / ASURF
GC TO 600
300 CONTINUE
IF (ICLASS(K-1) .eq. 0) GC TO 400
CT1 = 2.0
GC TO 500
400 CT1 = 1.0
500 CONTINUE
ASURF = 60.0 * (ALNTH(K-1) + ALNTH(K)) / 2.0
STEAMP = CL * ASS(K) / (AF * NHOUS(K) * CT + ASS(K-1) + ASSESS(K-1))
GC TO 600
600 CONTINUE
STDAM = STEAMI + STDAMP
RETURN
END

SUBROUTINE SCAMAG
******************************************************************************
* THIS SUBROUTINE COMPUTES DAMAGE TO EASEMENTS ASSUMED WITH EACH PIPE. *
******************************************************************************
* ASS(1) = AVERAGE ASSESSMENT VALUE IF HOUSES OF 1 TH  
ULD, (DEFAULT VALUE = 8,000) *
* AF = ASSESSMENT FACTOR, (DEFAULT VALUE = 4.0) *
* FU = FURNITURE VALUE (DEFAULT VALUE = 0.25) *
* (DEFAULT VALUE = 0.25) *
* RE(I) = PORTION OF PROPERTY VALUE ASSIGNED TO BASEMENT *
* (DEFAULT VALUE = 0.15) *
* HEALTH = HEALTH HAZARD FACTOR (DEFAULT VALUE = 0.5) *
* CLEAN = BASEMENT CLEANING (AFTER STORM) FACTOR. *
* (DEFAULT VALUE = 0.05) *
* G = DAMAGE RATIO TO TOTAL BASEMENT VALUE, *
* (DEFAULT VALUE = 0.1) *
* = DEPENDENT ON DEPTH OF FLOOING. *
* = 1.00 FOR C * GE. 3.00 INCHES *
******************************************************************************
COMMON /AREA6/ NPIPE
COMMON /AREA3/ DMAX
COMMON /AREA2/ NHOUS
COMMON /AREA4/ K
COMMON /AREA5/ BSDAM
COMMON /AREA6/ BMAX
COMMON /AREA7/ ASS(2), AF
DIMENSION RE(3), FMNX(3), NHOU(3), DMNX(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)) E600, 200, 300
**SUBROUTINE ANCST**

* THIS SUBROUTINE COMPUTES CAPITAL RECOVERY COSTS.  JF*
* THE SEWER SYSTEM INCLUDING SEWERS, MANHOLES, INLETS*
* (AND CATCH BASINS, IF ANY), PUMP, SUMP AND STRUCTURE DF*
* THE PUMPING STATION*

* AI = INTEREST RATE PER ANNUM (EXPRESSED AS A DEC-
  INAL FRACTION)*
* NYRP = YEARS OF ESTIMATED LIFE OF SEWERS*
* RP = 20 YEARS ( DEFAULT VALUE FOR CONCRETE PIPES)*
* CRFP, CRFFMP, CRFSMF ARE CAPITAL RECOVERY FACTORS FOR*
* PIPES, PUMP AND SUMP RESPECTIVELY.*
* FEI = ENGINEERING INDEX FOR CONSTRUCTION YEAR*
* CPIPES = COST OF PIPES, MANHOLES AND INLETS*
* (BASE YEAR, DEFAULT VALUE 1973)*
* CPIPE2 = COST OF PIPES, MANHOLES AND INLETS*
* (CONSTRUCTION YEAR)*
* CPIPE3 = ANNUAL COST OF PIPES, MANHOLES AND INLETS*
* C1,CSTY = CONSTANTS IN PUMP-COST EQUATION*

**COST = C1 * CR * HP * CSTY**

-= 26.0 .4500 RESPECTIVELY(DEFAULT VALUES)
FOR MINIMUM PUMP SIZE FOR STAND BY: HR=2.7FT*
GR = 7 CFS AND DIESEL MOTOR, PRICE = $ 6000
(1976 PRICE)*
NYFMP = YEARS OF ESTIMATED LIFE OF PUMP AND MOTHER*
CPUMP1 = COST OF PUMP, SUMP COSTS (BASE YEAR, *
CONSTRUCTION YEAR, ANNUAL COST)*
DSUMP = DEPTH OF EXCAVATION FOR SUMP (FT)*
BILD = COST OF BUILDING*
NYFSMF = YEARS OF ESTIMATED LIFE OF SUMP AND BUILD-
*ING (): 50 YEARS (DEFAULT VALUE)*
CSUMP1, CSUMP2, CSUMP3 = SUMP COSTS (BASE YEAR, *
CONSTRUCTION YEAR, ANNUAL COST)*
COPER = OPERATIONAL AND MAINTENANCE COST*
TANC = TOTAL CAPITAL RECOVERY COST OF THE SEWER*
* SYSTEM*

**COMMEN / AREAS6/ COSTSPE**
**COMMEN / AREA22/ P**
**COMMEN / AREA21/ R**
**COMMEN / AREA6/ NR**
**COMMEN / AREA30 / SBOTCM**
**COMMEN / AREA32/ ASUMP**
**COMMEN / AREA51/ TANC**

**DIMENSION P(4)**

**PART (I) PIPES + MANHOLES + INLETS**

**CALL CHIPES**
**REI = 1.33**
AI = 0.12
NYFP = 20
RP = 0.65
CFPP = A1*(1.0+AI)**NYFP/((1.0+AI)**NYFP-1.0)
CPIPE1 = CCSTBY/ RP
CPIPE2 = CPIPE1 * REI
CPIPE3 = CPIPE2 * CRFP

PART (II) PUMP

FEIPMP = 1.33
CQ = 26.
CSTBY = 450.
NYFPMP = 20
CRFMP = A1*(1.0+AI)**NYFPMP/((1.0+AI)**NYFPMP-1.0)
CPUMP1 = C1* QR * HR + CSTBY
CPUMP2 = CRFMP * CPUMP1
CPUMP3 = CRFMP * CPUMP2

PART (III) SUMP + BUILDING

DSUMP = (NFIP+1) - SHOTAY + 3.0
SUMP REC WILL BE SET 3.0 FT BELOW SUMP LOWST W.L. (DEFAULT VALUE)
A DEFAULT VALUE OF 2000 SQ. FT WILL BE ASSIGNED FOR MIN. CONTROL

BUILDING AREA: UNIT COST OF BUILDING = 30. #$ SQ. FT (DEFAULT VALUE)
CLAND = 20000.
COST OF LAND = $ 20,000 (DEFAULT VALUE) (ASSUMING 1/2 ACRE)
CEXC = 1.
C EXC = COST OF EXCAVATION ( $/CU. FT) = 1.0 $/CU. FT(DEFAULT VALUE)
CEXC1 = CEXC * (1.0 + DSUMP/30.)
C EXC1 = COST OF EXCAVATION TAKING INTO ACCOUNT DEPTH OF EXCAVATION
REISM = 1.03
CSUMP1 = CSBUILD + CLAND + CEXC1 * ASUMP * DSUMP
CSUMP2 = REISM * CSUMP1
NYRSM = 30
CFRSM = A1*(1.0+AI)**NYRSM/((1.0+AI)**NYRSM-1.0)
CSUMP3 = CFRSM * CSUMP2

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

SLURGUAGE ANCAM

******************************************************************************
*BASEMENT AND GROUND SURFACE DAMAGES ARE ADDED FOR *
*EACH SEWER SECTION THEN THEY ARE SUMMED UP FOR THE *
*SEwers *
******************************************************************************
CC**MON / AREA6/ NPIPE
CC**MON / AREA44/ K
CC**MON / AREA45/ BSCAM
CC**MCN / AREA50/ TDAM
CC**MCN / AREA81/ TDAM
) SUM = 0.0
CO 100 K = 1, NPIPE
CALL BDAMAG
CALL STC AM
SUM = SUM + BSCAM + STC AM
100 CONTINUE
TDCAM = SUM
RETURN
FKE
SUBROUTINE BANO
 *******************************************
 * THIS SUBROUTINE BALANCES THE INITIAL DESIGN, BY  
 * CORRECTING SEWER DIAMETERS TO MAINTAIN SAME MAXIMUM  
 * RELATIVE STORAGE IN ALL SECTIONS, FOR A PRESELECTED  
 * STORM OF A KNOWN RETURN PERIOD.  
 * NEW CIAM = OLD CIAM * (REL. STG.)**0.25  
 *******************************************
COMMON / AREA6/ NPIPE
COMMON / AREA8/ CIAM
COMMON / AREA2/ FMAX(I)
DIMENSION CIAM(3)
DC 100 I = 1, NPIPE
DIAM(I) = DIAM(I) * FMAX(I) ** 0.25
100 CONTINUE
NPIPE = NPIPE - 1
DC 200 I = 1, NPIPE
IF(DIAM(I+1) LT DIAM(I)) CIAM(I+1) = CIAM(I)
200 CONTINUE
CALL VOLSYS
RETURN
END

SUBROUTINE HYDMB
 *******************************************
 * THIS SUBROUTINE IS USED TO BALANCE THE PRELIMINARY *  
 * DESIGN. IT ATTEMPTS TO MAINTAIN THE SAME RELATIVE *  
 * STORAGE FOR EACH SEWER SECTION, FOR A PRESELECTED STORM  
 * CF & A KNOWN RETURN PERIOD. *  
 * ALSO, IT ADJUSTS THE PUMPING CAPACITY SO THAT IT *  
 * JUST PREVENTS SURCHARGING OF THE OUTFALL DURING THE *  
 * SPECIFIED STORM. *  
 *******************************************
COMMON / AREA/ T
COMMON / AREA2/ ASW
COMMON / AREA1/ RELATA
COMMON / AREA2/ ENCSEW
COMMON / AREA4/ SWLQSUW
COMMON / AREA3/ SHETCM
COMMON / AREA4/ IP
COMMON / AREA6/ NPIPE
COMMON / AREA8/ IT
COMMON / AREA9/ M
COMMON / AREA2/ FMAX(I)
COMMON / AREA8/ CIAM
COMMON / AREA1/ ITS
COMMON / AREA1/ SO
COMMON / AREA5/ LSELEV
COMMON / AREA7/ ALNTH
DIMENSION ALNTH(3), SO(3)
DIMENSION CIAM(3)
REAL KMUS
DCP = 0.0
SWL = SHETCM
M = 0
DC 50 I = 1, NPIPE
300 CONTINUE
50 FMAX(I) = 0.0
CALL VOLSYS
3 ITERATIONS ARE USED TO BALANCE SEWER SIZES.
DO 900 NNN = 1, 3
DC 700 IIT = 1, 180
IT = IIT
IF( M .NE. 0) GO TO 800
DC 200 IPK = 1, NPIPE
IP = IPK
CALL ACCHYC
CALL MUSKFT
300 CONTINUE
CALL SWITCH
IF( NN .NE. 3) GC TO 700
ITS = IT
S-L1 = S-L
CALL SUMP1
DELSWL = SWL - S-L1
DCF1 = DCF
IF(SWL GT ENDCW) DCF = DELSWL * AUMP / T/60.
IF(DCF LT DGPI) DCF = DGPI
700 CONTINUE
800 CONTINUE
CALL $ALD
DC 2000 I = 1. NPIPE
2000 PMAX(I) = 0. C
N = 0
900 CONTINUE
DC PMAX = DCF
OR = OR + DCFMAX
C THIS IS THE 2-ND ITER. TO ADJUST THE PUMPING CAPACITY.
N = 0
DCF = 0.0
SWL = SBCUM
DC 4000 /ITT= 1, 180
IT = ITT
IF(M.NE.0) GC TO 5000
DC 4010 IFK=1. NPIPE
IP = IFK
CALL ADHYD
CALL NUSKRT
4010 CONTINUE
CALL SWITCH
SWL = SWL
CALL SUMP1
DELSWL = SWL - S-L1
DQF1 = DCF
IF(SWL GT ENDCW) DCF = DELSWL * AUMP / T/60.
4000 CONTINUE
PC MAX = DCF
OR = OR + DCFMAX
ENDCW = LSELEV
DC 6000 I = 1. NPIPE
6000 ENDCW = ENDCW - E0(I) # ALNTH(I)
SECTION = ENDCW - LA*N(NPIPE) = 1.0
RETURN
END

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

************************************************************************************
* THIS SUBROUTINE IS USED TO COMPUTE AVERAGE ANNUAL *                        *
* PROBABLE DAMAGE FOR Perturbed SYSTEMS. * A * CAND *                          *
* AVERAGE ANNUAL PROB. DAMAGE OF THE ORIGINAL SYSTEM *                       *
* ARE Known. DAMAGE RESULTING FROM A PRESELECTED SPECIFIC *                   *
* STORM IS USED TO DETERMINE THE NEW AVERAGE PROB. DAM.                      *
************************************************************************************
* KSTORM = INDEX OF THE STORM TO BE CONSIDERED IN *                          *
* COMPUTING ANNUAL AVERAGE PROBABLE DAMAGE. *                                *
************************************************************************************

COMMON / AREA72/ A,B,C
COMMON / AREA74/ CAM
COMMON / AREA75/ AD
DIMENSION P(7),CAM(7)
P1 = P(1)
P2 = P(2)
P3 = P(3)
D1 = CAM(1)
D2 = CAM(2)
D3 = CAM(3)
C1 = (C2-D1)*(P1-P2)+(P2-P1)*(D3-D1)
C = C1/(P2-P3)/(P3-P1)/(P1-P2)
B = (C2-D1)*C*(P2-P1-P1)/P2-P1
A = C1- E=P1- C*P1
AVERAGE PROBABLE ANNUAL DAMAGE = AREA UNDER PROB-DAMAGE CURVE
= A*P1 + E*P1*P1/2. + C*P1**3/3.
RETURN
END

SUBROUTINE CHANCE

************************************************************************************
* THIS SUBROUTINE PERTURBS THE ORIGINAL BALANCED DESIGN *                    *
* WITH RESPECT TO THE I-TH DECISION VARIABLE. IT MAKES *                     *
* THE NECESSARY ADJUSTMENTS ACCOMPANIED BY EACH PERT*                       *
*urbation of the system, in order to maintain the *                         *
* THE COMPATIBILITY OF THE DESIGN. *                                          *
************************************************************************************

COMMON / AREA1/ S0
COMMON / AREA5/ CIAM
COMMON / AREA6/ NPIPE
COMMON / AREA7/ ALNTH
COMMON / AREA22/ T
COMMON / AREA28/ ASUMP
COMMON / AREA30/ SDTCM
COMMON / AREA31/ RELETA,HR,QR
COMMON / AREA32/ ENCSW
COMMON / AREA55/ PSELEV
COMMON / AREA80/ CFGPOS(5),CHGNEME(5)
COMMON / AREA83/ NCNG
COMMON / AREA85/ DIA(2),S0(2),SS0(2),OHR,HRR,AASLMP,USLV,SS80
DIMENSION S0(2),CIAM(2),M(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. + C*GPFS(NCHNG))
  CONTINUE

C  ENDSEW = USELEV
  DC 60 I = 1, NPIPE
  60 ENDSEW = ENDSEW - SO(I)*ALNTH(I)
  SECTION = ENDSEW - CIAM(NPIPE) - 3.0
  DMR = SSEQT - SECTION
  HR = H - DMR

C  NCHNG = 2 (CHANGE IN PUMP CHARACTERISTICS)
  IF(NCHNG .NE. 2) GO TO 200
  OR = OR + (1. + C*GPFS(NCHNG))

C  NCHNG = 3 (CHANGE IN SUMP AREA)
  IF(NCHNG .NE. 3) GO TO 300
  ASUMP = ASUMP + (1. + C*GPFS(NCHNG))

C  NCHNG = 4 (CHANGE IN SYSTEM OVERALL DEPTH)
  IF(NCHNG .NE. 4) GO TO 400
  DEPTH = H(I) - USELEV
       USELEV = M(I) - DEPTH
  ENDSEW = USELEV
  DO 350 I = 1, NPIPE
       350 ENDSEW = ENDSEW - SO(I)*ALNTH(I)
  SECTION = ENDSEW - CIAM(NPIPE) - 3.0
  DMR = SSEQT - SECTION
  HR = H - DMR

C  NCHNG = 5 (CHANGE IN DIAMETERS)
  IF(NCHNG .NE. 5) GO TO 500
  DO 450 I = 1, NPIPE
       450 CIAM(I) = CIAM(I) + (1. + C*GPFS(NCHNG))
  ENDSEW = USELEV
  DO 470 I = 1, NPIPE
       470 ENDSEW = ENDSEW - SO(I)*ALNTH(I)
  SECTION = ENDSEW - CIAM(NPIPE) - 3.0
  DMR = SSEQT - SSEQT
  HR = H - DMR

C  CONTINUE
  RETURN
END

SUBROUTINE ORIGNL

**********************************************************************
* THIS SUBROUTINE IS USED TO RESET THE PERTURBED SYSTEM TO ITS ORIG************
*INAL BALANCED DIMENSIONS.
**********************************************************************

COMMON /AREA1/ SO
COMMON /AREA5/ CIAM
COMMON /AREA6/ NPIPE
COMMON /AREA30/ SSEQT
COMMON /AREA47/ ASUMP
COMMON /AREA21/ RELETA, HR, QR
COMMON /AREA23/ ENCS2H
COMMON /AREA55/ USELEV
COMMON /AREA46/ NCHNG
COMMON /AREA66/ CIAM(3), SO(3), HRR, HRR, ASUMP, USLV, SSEQT
COMMON /AREA49/ ENDS
DIMENSION CIAM(2), SO(3)
IF(NCHNG .NE. 1) GO TO 100
DO 50 I = 1, NPIPE
       50 SO(I) = SO(I)
       SSEQT = SSEQT
       HR = H
       ENDSEW = ENDS
100 CONTINUE
   IF(NCHKG NE 2) GO TO 200
200 CONTINUE
   IF(NCHKG NE 3) GO TO 300
300 CONTINUE
   IF(NCHKG NE 4) GO TO 400
   USELEV = USLV
   SRCTOM = SS80T
   HR = HRR
   ENDSW = ENDS
   RETURN
400 CONTINUE
   IF(NCHKG NE 5) GO TO 500
   DO 450 I = 1, NPIPE
   DIAM(I) = OC1AM(I)
   SRCTCM = SS80T
   HR = HRR
   RETURN
500 CONTINUE
   RETURN
END

SUBRUTINE STORE
******************************************************************************
* THIS SUBRUTINE IS USED TO STORE THE ORIGINAL BALANCED
* DESIGN OF THE SYSTEM.
******************************************************************************
COMMON / AREA35W, NPIPE-
COMMON / AREA35W, CIAM
COMMON / AREA35W, QRR
COMMON / AREA35W, HR, QR
COMMON / AREA35W, ASUMP
COMMON / AREA35W, SRCTOM
COMMON / AREA35W, USELEV
COMMON / AREA35W, CIAM(I), SS0(I), QRR, HR, ASUMP, USLV, SS80T
COMMON / AREA35W, ENDS
DIMENSION CIAM(I), SS0(I)
DO 100 I = 1, NPIPE
   DIAM(I) = CIAM(I)
100 CONTINUE
   QRR = QR
   ASUMP = ASUMP
   USLV = USELEV
   SS80T = SS80T
   HR = HR
   ENDS = ENDSW
   RETURN
END

SUBRUTINE CFAC1
******************************************************************************
* THIS SUBRUTINE COMPUTES FIRST DERIVATIVES OF BOTH
* COST AND DAMAGE WITH RESPECT TO DESIGN VARIABLES.
* (FORWARD GRADIENTS).
******************************************************************************
* CG1(K) = 1ST DERIVATIVE OF THE SYSTEM ANNUAL COST
* WITH RESPECT TO THE KTH VARIABLE
* DG1(K) = 1ST DERIVATIVE OF THE SYSTEM ANNUAL PROJ
* ABLE DAMAGE WITH RESPECT TO THE KTH VAR.
******************************************************************************
COMMON / AREA35W, CG1(K), DM1(K)
COMMON / AREA35W, CG1(K), DM1(K)
COMMON / AREA35W, CG1(K), DM1(K)
COMMON / AREA35W, CG1(K), DM1(K)
COMMON /AREA94/ CINITL
DO 100 K = 1,S
CGL(K) = (CCSF(K) - CINITL) / CMPS(K)
DG(K) = (C(K,2) - C(K,1)) / CMFS(K)
100 CONTINUE
DO 200 I = 1,S
PRINT 1000,1,CG1(I),DG1(I)
200 CONTINUE
1000 FORMAT(10X,'DESIGN VARIABLE NO. = ',I3,/, /
2 10X,'COST GRADIENT (1ST) = $ ',F12.1,' $ /UNIT CHANGE',/)
3 10X,'DAMAGE GRAD. (1ST) = $ ',F12.1,' $ /UNIT CHANGE',/)
RETURN
END

SUBROUTINE GRAD1B
*******************************************************************************/
* THIS SUBROUTINE COMPUTES FIRST DERIVATIVES OF THE *
* ANNUAL COST OF THE SYSTEM WITH RESPECT TO DESIGN. *
* VARIABLES. BACKWARD DERIVATIVES ARE COMPUTED USING: *
* NEGATIVE RELATIVE CHANGES (LESS COST = HIGHER DAMAGE) *
*******************************************************************************/
* CG1B(I)= 1ST DERIVATIVE OF THE SYSTEM ANNUAL COST *
* WITH RESPECT TO THE I TH VAR. (BACKWARD) *
*******************************************************************************/
COMMON /AREA94/ CMPS(S), CMNEG(S)
COMMON /AREA95/ NCHNG
COMMON /AREA96/ TANC
COMMON /AREA97/ CINITL
COMMON /AREA98/ CGL(S)
DIMENSION CCHP(S), CESTN(S)
DO 100 K = 1,S
CCHP(K) = CMPS(K)
100 CONTINUE
DO 200 NCHNG = 1,S
CMPS(NCHNG) = CMNEG(NCHNG)*(-1.0)
CALL CHANGE
CALL ANCOST
CESTN(NCHNG) = TANC
CALL CRIGNL
CMPOS(NCHNG) = CCHP(NCHNG)
200 CONTINUE
DO 300 I = 1,S
CGLB(I) = (CINITL - CESTN(I)) / CMNEG(I)
300 CONTINUE
DC 400 N = 1,S
PRINT 1600, N, CGLB(N)
400 CONTINUE
1000 FORMAT(10X,'DESIGN VAR. NO. = ',I3,/, /
2 10X,'BACKWARD COST GRAD. (1ST) = $ ',F12.1,' $ /UNIT CHANGE',/)
3 10X,'BACKWARD DAMAGE GRAD. (1ST) = $ ',F12.1,' $ /UNIT CHANGE',/)
RETURN
END
APPENDIX IV

FIGURES
FIGURE 2.1 ELEMENTS OF THE SIMULATION OF URBAN RUNOFF
(After Reference 20)
FIGURE 2.2 A DESIGN-STORM HYSTOGRAPH 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
FIGURE 3.1 A SCHEMATIC DIAGRAM OF THE MODEL
FIGURE 3.2 EXAMPLE OF A SEWERED DISTRICT AND A BASIC SUBCATCHMENT
\[ Q_{\text{inf}}(\text{IMH,1}) = i_1 \Delta A \]
\[ Q_{\text{inf}}(\text{IMH,2}) = (i_1 + i_2) \Delta A \]
\[ Q_{\text{inf}}(\text{IMH,3}) = (i_1 + i_2 + i_3) \Delta A \]

(a) AREA/TIME DIAGRAM

(b) RAINFALL/TIME CURVE

(c) RUNOFF HYDROGRAPH

FIGURE 3.3 BASIC INFLOW HYDROGRAPH DEVELOPMENT
FIGURE 3.4 PRISM AND WEDGE STORAGE

Wedge Storage = Kx(I - O)

Prism Storage = KO
Figure 3.5: Relative Storage Versus Relative Discharge
**FIGURE 3.7** DEFINING SKETCH, COMPONENTS CONSTITUTING INFLOW TO AND OUTFLOW FROM SEWER SYSTEM
Figure 3.8  Simplified Flow Chart for Construction of the Sewer Energy Lines
FIGURE 3.9.a A TYPICAL HOUSE CONNECTION DRAINING A BASEMENT TO A STORM SEWER

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
$Q_B(i,t) = \text{negative}$

**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)
\[ Q_{in} = \sum_{i=1}^{i-1} Q_{inf}^{(i,t)} - \sum_{i=1}^{i-1} Q_{B}^{(i,t)} - \sum_{i+1}^{i-1} Q_{st}^{(i,t)} \]

FIGURE 3.12 DEFINING SKETCHES, STREET FLOWS
IF USEL <i> > H <i>

Yes

$Q_{st}(i,t) = 0.0$

USEL <i> = H <i>

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

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

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

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

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

FIGURE 3.13 SIMPLIFIED FLOW CHART FOR COMPUTATION OF STREET FLOWS
TIME INDEX = \( t \)

\[
(\text{STORAGE})_i = K_{\text{MUSKINGUM}} [x_1 + (1-x)0]
\]

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

RELATIVE STORAGE = 
\[
\frac{(\text{ACTUAL STORAGE})_{\text{SYSTEM}}}{(\text{MAX AVAILABLE STORAGE})_{\text{SYSTEM}}}
\]

IF 
REL. STORAGE > 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
\[ \Delta SWL = SWL_t - SWL_{t-1} \]

IF

\[ SWL > ENDSEN \]

No

\[ \Delta Q_p^t = \frac{(\Delta SWL) A_s}{\Delta t (60)} \]

IF

\[ \Delta Q_p^t < \Delta Q_p^{t-1} \]

No

Yes

\[ \Delta Q_p^t = \Delta Q_p^{t-1} \]

\[ Q_R' = Q_R + \Delta Q_p' \]

FIGURE 3.16 SIMPLIFIED FLOW CHART FOR ADJUSTING PUMPING CAPACITY
- Compute capital recovery cost of the original system, $C_o$

- Store the original balanced design

- Using a specified value of $x_i$,
  obtain the perturbed system,
  $i=1$, sewer slopes, $S_o$
  $i=2$, pump power, $Q_{R_{PR}}$
  $i=3$, sump area, $A_s$
  $i=4$, overall system depth, $d_{SYS}$
  $i=5$, sewer diameters, $D$

- Compute the cost of the perturbed system, $(COST)_i$

- Compute cost gradient
  $c_i = \frac{(COST)_i - C_o}{x_i}$

- Reset the system dimensions to the original balanced values

**Figure 3.17** Simplified flow chart for computation of cost gradients
Figure 3.18 Defining Sketch, Forward Cost Gradients

$$c_i = \frac{c_1 - c_0}{x_i}$$
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
\[ d_o = Ap_1 + \frac{1}{2} Bp_2^2 + Cp_3^3 \]

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 OCCURRENCE, \(p_k\), 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.

FIGURE 3.21 COMPUTATION OF DAMAGE GRADIENTS
FIGURE 3.22 UPPER AND LOWER LIMITS ON THE AVERAGE ANNUAL PROBABLE DAMAGE OF THE OPTIMUM SYSTEM
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, $C_0$, 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, $a_0$.

FIGURE 3.23 FLOW CHART OF THE MODEL
FIGURE 3.23 (continued)

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

PRINT/BALANCED DESIGN, COST GRADIENTS DAMAGE GRADIENTS, $c_o$, $d_o$ AND A.

---MANUAL

PREPARE INPUT DATA TO SUBROUTINE "SIMPLEX"

OPTIMIZATION

FINAL DESIGN (COMMERCIAL PIPE SIZES)
FIGURE 4.1 FLOW CHART FOR THE MAIN PROGRAM
FIGURE 4.1 (continued)

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 DPL

D(NCHNG,2)=AD1

CALL ANCOST

-COSTP(NCHNG)=TANC

CALL ORIGINL

CALL GRAD1
FIGURE 4.2 FLOW CHART FOR SUBROUTINE—PRINT1
FIGURE 4.2 (continued)

\[ I = 1, \text{SPAN1} \]

\[ I_k = I^* T - T \]
\[ \text{PRINTT}(I) = I_k \]
\[ \text{PRINTQ}(I) = RAIN(I) \]

\[ \text{PRINT/} \]
\[ I_k, RAIN(I) \]

CALL PLOT 3(\text{PRINTT, PRINTQ, 100})

RETURN
206
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 4.5 FLOW CHART FOR SUBROUTINE-HYDMOD
Figure 4.5 (continued)

```
(a)

M=1

Yes

No

TDAM=0.0

RETURN
```
210

ENTER

COMMON,DIMENSION

\[ M=0 \]

\[ K=ITS \]

\[ K=IT \]

\[ ANPIPE=0.7854 \times DIAM(NPIPE)^2 \]
\[ CPUMP=RELETA \times HR \times QR \]
\[ C = T \times CPUMP \times 30 \times ASUMP \]

\[ K \neq 1 \]

\[ QSUMP1=QINLET(1) \]
\[ QSUMP=QOUT(NPIPE,1) + INFHD(NPIPE+1,1) \]
\[ HP1=ECHANL-SBOTOM \]

\[ QSUMP=QOUT(NPIPE,K-1) + INFHD(NPIPE+1,K-1) \]
\[ QSUMP=QOUT(NPIPE,K) + INFHD(NPIPE+1,K) \]
\[ HP1=ECHANL-SWL \]

\[ QSUMPA=(QSUMP+QSUMP1)/2.0 \]
\[ B=(-1.0) \times HP1+2.0 \times C \times QSUMPA/CPUMP \]
\[ -C/HP1 \]
\[ HP=(-0.5) \times B+.5 \times SQRT(B*B+4.0*C) \]
\[ HPMAX=ECHANL-SBOTOM \]

\[ HP>HPMAX \]

\[ SWL=ECHANL-HP \]
\[ VNPIPE=QOUT(NPIPE,K)/ANPIPE \]
\[ VHEAD=VNPIPE*VNPIPE/64.4 \]

\[ SWL<ENDSEW \]

\[ ESUMP(K)=SWL+VHEAD \]

\[ ESUMP(K)=ENDSEW+VHEAD \]

RETURN
Figure 4.7 Flow Chart for Subroutine-STRMOD
212

ENTER

COMMON, DIMENSION

XQ = 0.0

I = 1, NPIPE

F(I, IT-1) = 0.0, SE(I, IT-1) = S0(I)
EMP(I, IT-1) = BMP(I), QSTR(I) = 0.0
VOLST(I) = 0.0, DVOLST(I) = 0.0
BMAX(I) = 0.0, FMAX(I) = 0.0
VMAX(I) = 0.0, TPEAK(I) = 0.0

ITS = IT, 220

ITS1 = ITS - 1

IMO = 1, NPIPE

QBSMT(IMO, ITS) = QBSMT(IMO, ITS1)

ESUMP(ITS) = ESUMP(ITS - 1)

ITR1 = 1, 3
ITR = 1, 6
IM = 1, NPIPE

KM = NPIPE - IM + 1
QQ = QSTR(KM)
SUMIHD = QINLET(ITS)
SUMQST = 0.0

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

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) > 0.01

Yes

QSTR(KM) = 0.0
DVOLST(KM) = 0.0

QSTR(KM) = (-1.0) * CB(KM) * SQRT(ABS(VOLST(KM)))
FIGURE 4.8 (continued)

CALL FRATIO
CALL QHOUSS

\[\begin{align*}
XDFLOD(KM) &= DFLOOD \times 12. \\
X &= QBSMT(KM, ITS) \\
QBSMT(KM, ITS) &= QHOUS \times (1. - (0.9 \times ITR) \times ITR) \\
QBSMT(KM, ITS) &= (QBSMT(KM, ITS) + X) / 2.0 \\
QSTR(KM) &= QSTR(KM) \times (1. - (0.9 \times ITR) \times ITR) \\
QSTR(KM) &= (QSTR(KM) + QQ) / 2.0 \\
DVOLST(KM) &= QSTR(KM) \times T \times 60.
\end{align*}\]

\[\begin{align*}
QOUT(NPIPE, ITS) &= QAV(NPIPE, ITS) - QBSMT(NPIPE, ITS) / 2.0 \\
Y &= ESUMP(ITS)
\end{align*}\]

CALL SUMPL

\[\begin{align*}
ESUMP(ITS) &= (ESUMP(ITS) + Y) / 2.0
\end{align*}\]

\[\begin{align*}
KI &= 1, NPIPE \\
DVOLST(KI) &= 0.1 \\
TPEAK(KI) &= TPEAK(KI) + T \\
VOLST(KI) &= VOLST(KI) + DVOLST(KI)
\end{align*}\]

\[\begin{align*}
I &= 1, NPIPE \\
DMAX(I) &= XDFLOD(I) \\
DMAX(I) &= XDFLOD(I)
\end{align*}\]
FIGURE 4.8 (continued)

1.

FMAX(I) > F(I, ITS)  Yes

No

FMAX(I) = F(I, ITS)

VMAX(I) > VOLST(I)

VMAX(I) = VOLST(I)

NFLAG = 0

I = 1, NPIPE

OBSMT(I, ITS) < XQ  Yes

No

NFLAG = NFLAG + 1

NFLAG = NPIPE  Yes

No

ITS = IT + 4  Yes

No

XQ = 0.01

CALL ANDAM

RETURN
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

DFLOOD=0.0

+ve

ENBAS=BMP(I)+DFLOOD
USEBL=ENBAS+SBASE(I)*ALNTH(I)/2.0
DSEBL=ENBAS-SBASE(I)*ALNTH(I)/2.0
USELL=USEL(I)
USHL=USELL-USEBL
DSEL=USEL-SE(I,ITS)*ALNTH
DSHL=DSEL-DSEBL

FIGURE 4.9 FLOW CHART FOR SUBROUTINE-QHOUS
Figure 4.9 (continued)

217

\[ a \]

-ve

\[ P(I,ITS) \]

+ve

-ve

DFLOOD \( \geq 0.0 \)

No

QHOUSS = 0.0

RETURN

Yes

\[ P(I,ITS) < 1.0 \]

No

USHL \( < 0.0 \)

Yes

DSHL \( < 0.0 \)

No

No

DSHL \( < 0.0 \)

Yes

\[ FT = \frac{USHL}{(USHL + ABS(DSHEL))} \]

\[ FT = DSHEL / (DSHEL + ABS(USHL)) \]

\[ B = ((ABS(USHL) \times 1.5) - (ABS(DSHEL) \times 1.5)) / (ABS(USHL) - ABS(DSHEL)) \]

QHOUSS = \(-1.0 \times 0.667 \times A \times B\)

\[ QHOUSS = (-1.0) \times 0.667 \times A \times B \]

FLVOL - ABS(QHOUSS) \( \times T \times 60 \)

+ve

-ve

\[ USHL \]

Yes

DSHL \( < 0.0 \)

\[ FT = P(I,ITS) \]

RETURN

No

c

b

RETURN
FIGURE 4.9 (continued)

\[ \text{VOLAV} = \text{DFLOOD} \times (1.0 - F(I, ITS)) \times \text{ABASE}(I) \times 43560. \]

- if \( \text{VOLAV} - \text{QHOUSSN} \times T \times 60 \) is negative, then \( \text{PHOUS} = \text{QHOUSSP} - \text{QHOUSSN} \)
- if \( \text{VOLAV} - \text{QHOUSSN} \times T \times 60 \) is positive, then \( \text{QHOUTN} = \text{VOLAV} / T \times 60 \)

\[ \begin{align*}
\text{QHOUSSP} &= 0.667 \times F(I, ITS) \times \sqrt{\text{ABS}(\text{USHL})} \times A \\
\text{QHOUSSN} &= 0.667 \times (1.0 - F(I, ITS)) \times \sqrt{\text{ABS}(\text{USHL})} \times A \\
\end{align*} \]
FIGURE 4.10 FLOW CHART FOR SUBROUTINE-FRATIO
FIGURE 4.11  FLOW CHART FOR SUBROUTINE-HYDME
FIGURE 4.11 (continued)

If M=0

IFK=1, NPIPE

IP=IPK
CALL ADDHYD
CALL MUKRT
CALL SWITCH
SWL1=SWL
CALL SUMP1
DELSWL=SWL-SWL1
DQPI=DQP

If SWL>ENDSEW

DQP=DELSWL*ASUMP*T/60.

If DQP<DQPI

DQP=DQPI

DQPMAX=DQP
QR=QR+DQPMAX
ENDSEW=USELEV

I=1, NPIPE

ENDSEW=ENDSEW-SSO(I)*ALNTH(I)
Figure 4.11 (continued)

\[ h \]

\[ SBOTTOM=ENDSEW-DIAM(NPIPE)-3.0 \]

RETURN
SIMPLEX METHOD FOR SOLVING LP PROBLEMS

THE FOLLOWING IS THE LISTING OF DATA AS IT IS READ IN, EITHER FROM CARDS OR PARTLY FROM PREVIOUS RUN AND PARTLY FROM CARDS.

STORM SEWER DESIGN (WSSC). SENSITIVITY 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
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<th>OBJF</th>
<th>3.314000</th>
<th>Explanation</th>
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FIGURE 4.12 (continued)

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FIRST B

| R1 | 0.000500  | (x_{12}) |
| R2 | 0.000500  | (x_{13}) |
| R3 | 0.000500  | (x_{14}) |
| R4 | 0.000500  | (x_{15}) |
| R5 | 0.000500  | (x_{16}) |
| R6 | 0.000500  | (x_{17}) |
| R7 | 0.000500  | (x_{18}) |
| R8 | 0.000500  | (x_{19}) |
| R9 | 0.000500  | (x_{10}) |
| R10| 0.000500  | (x_{11}) |
| R11| 153.391000| N_{1} - d_{1}|
| R12| 153.391000| N_{1} - d_{1}|
| R13| 153.391000| N_{1} - d_{1}|
| R14| 153.391000| N_{1} - d_{1}|
| R15| 153.391000| N_{1} - d_{1}|
| R16| 153.391000| N_{1} - d_{1}|
| R17| 19.498000 | d_{2}      |
| R18| 19.498000 | d_{2}      |
| R19| 19.498000 | d_{2}      |
| R20| 19.498000 | d_{2}      |
| R21| 19.498000 | d_{2}      |
| R22| 19.498000 | d_{2}      |
| R23| 55.361990 | C_{3}      |
| R24| 55.361990 | C_{3}      |
| R25| 55.361990 | C_{3}      |
| R26| 55.361990 | C_{3}      |
| R27| 55.361990 | C_{3}      |
| R28| 55.361990 | C_{3}      |
| R29| 74.850000 | C_{4} + d_{6} |
STORM SEWER DESIGN (WSSD), SENSITIVITY 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.

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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

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NON-BASIC VAR.  REDUCED COST

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If any of the above reduced costs is zero, it indicates that multiple optima exist.
FIGURE 5.1 A TYPICAL DESIGN STORM HYETOGRAPH

Return Period = 10 Years

\[
I_{\text{ave.}} = \frac{170}{t_d + 23}
\]
FIGURE 5.2 A TYPICAL BASIC INFLOW HYDROGRAPH
1 in 50 Years Storm
Drainage Basin Impervious
Area = 22 Acres

TIME ELAPSED FROM THE BEGINNING OF THE STORM, MIN.

RUNOFF, CPS

Gravity Flow

Surcharged Flow

FIGURE 5.3 A TYPICAL OUTFALL HYDROGRAPH TO THE SUMP WELL
A typical plot of sump water level versus elapsed time.
Gravity Flow

Surcharged Flow

Storm Water Flows
From Sewer to Basements

(From Basements to Storm Sewer)

Time from the beginning of the storm, min.

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)
**Legend:**

ASS = Average Assessment Value of Houses, Dollars

RB = Portion of Property Value Assigned to Basements

**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**
<p>| Row Type of | Const. | $x_1$ | $x_{x1}$ | $x_2$ | $x_{x2}$ | $x_3$ | $x_{x3}$ | $x_4$ | $x_{x4}$ | $x_5$ | $x_{x5}$ | $b_1$ |
| Name Name of | | | | | | | | | | | | |
| OBJF | = | $-a_1$ | $-a_1$ | $-a_2$ | $a_2$ | $-a_3$ | $a_3$ | $-a_4$ | $a_4$ | $-a_5$ | $a_5$ | |
| R1 | $\leq$ | 1.0 | -1.0 | | | | | | | | | $(x_{UL})<em>1$ |
| R2 | $\leq$ | | 1.0 | -1.0 | | | | | | | | $(x</em>{UL})<em>2$ |
| R3 | $\leq$ | | | 1.0 | -1.0 | | | | | | | $(x</em>{UL})<em>3$ |
| R4 | $\leq$ | | | | 1.0 | -1.0 | | | | | | $(x</em>{UL})<em>4$ |
| R5 | $\leq$ | | | | | 1.0 | -1.0 | | | | | $(x</em>{UL})_5$ |
| R6 | $\leq$ | | | | | | 1.0 | -1.0 | | | | |
| R7 | $\leq$ | | | | | | | 1.0 | -1.0 | | | |
| R8 | $\leq$ | | | | | | | | 1.0 | -1.0 | | |
| R9 | $\leq$ | | | | | | | | | 1.0 | -1.0 | |
| R10 | $\leq$ | | | | | | | | | | 1.0 | -1.0 |
| R11 | $\leq$ | | | | | | | | | | | 1.0 |
| R12 | $\leq$ | | | | | | | | | | | | |
| R13 | $\leq$ | | | | | | | | | | | | |
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<tr>
<td>1</td>
<td>I3</td>
<td>1-3</td>
<td>Number of manholes, inlet points, equal to the number of trunk sewers + 1</td>
<td>NMH</td>
<td>None</td>
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<td>REPEAT CARD GROUP 2 FOR EACH MANHOLE PROCEEDING DOWNSTREAM</td>
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<td>3</td>
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<td>Ground surface elevation, ft.</td>
<td>H(I)</td>
<td>None</td>
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<td>Coefficient of inlet catch basin, - sloping intersection, - an intersection in depression.</td>
<td>CB(I)</td>
<td>0.01</td>
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<td></td>
<td></td>
<td>Number of vehicle crossings at manhole (intersection) per hour, on an average basis.</td>
<td>VEHH(I)</td>
<td>None</td>
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<td>REPEAT CARD GROUP 3 FOR EACH MANHOLE PROCEEDING DOWNSTREAM</td>
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<td>7</td>
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<td>Basic subcatchment impervious area, acres.</td>
<td>ARIMP(I)</td>
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<td>Time of concentration of basic subcatchment, minutes.</td>
<td>ET(I)</td>
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<td>Time step, minutes</td>
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<td>REPEAT CARD GROUP 5 FOR EACH SEWER PROCEEDING DOWNSTREAM</td>
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<td>Manning's Coefficient, m.</td>
<td>AN(I)</td>
<td>0.013</td>
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<td>Sewer length, ft.</td>
<td>ALNTN(I)</td>
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<td>13</td>
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<td>Sewer diameter (initial design), ft.</td>
<td>DIAM(I)</td>
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<td>Sewer slope (initial design).</td>
<td>SO(I)</td>
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<td>Basements area, acres</td>
<td>DBSMT(I)</td>
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<td>Average depth of basements below ground surface, ft.</td>
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<td>Number of storms considered</td>
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<td>REPEAT CARD GROUP 8 FOR EACH STORM CONSIDERED</td>
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<td>a (Coefficients in the rainfall intensity-time formula, i = \frac{a}{t + c}}</td>
<td>BB(I)</td>
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<td>c = \frac{a}{t + c}}</td>
<td>CC(I)</td>
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<td>31-40</td>
<td></td>
<td>Relative advancement of peak intensity of the storm</td>
<td>RR(J)</td>
<td>0.375</td>
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<td>41-50</td>
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<td>Probability of occurrence of this storm</td>
<td>PROB(I)</td>
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<td>- 1st storm (1/5 years storm)</td>
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<td>- 2nd storm (1/10 years storm)</td>
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<td>Base flow, cfs.</td>
<td>QBAFL</td>
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<td>REPEAT CARD GROUP 10 FOR EACH SEWER PROCEEDING DOWNSTREAM</td>
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<td>11-20</td>
<td>Average length of the pipe of house connections, ft.</td>
<td>DHCON(I)</td>
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<td>21-23</td>
<td>Diameter of the pipe of house connections, ft.</td>
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<td>11</td>
<td>3F10.0</td>
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<td>Pump relative operating efficiency</td>
<td>RELETA</td>
<td>0.95</td>
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<td>11-20</td>
<td>Pumping rated head (initial design), ft</td>
<td>HR</td>
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<td>21-30</td>
<td>Pumping rated discharge (initial design), ft.</td>
<td>QR</td>
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<td>1-10</td>
<td>Sump cross sectional area, ft². (from initial design)</td>
<td>ASUMP</td>
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<td>11-20</td>
<td>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.</td>
<td>ECHANL</td>
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<td>Crown elevation of the upstream end of the first sewer in the system, ft.</td>
<td>USELELV</td>
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<td>Positive relative changes in decision variables with respect to i=i, sewer slopes</td>
<td>POSCHG(I)</td>
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<td>1-10</td>
<td>i=1, sewer slopes</td>
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<td>11-20</td>
<td>i=2, pump power</td>
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<td>21-30</td>
<td>i=3, sump area</td>
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<td>31-40</td>
<td>i=4, system overall depth</td>
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<td>41-50</td>
<td>i=5, sewer diameters</td>
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<td>i=1, sewer slopes</td>
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<td>11-20</td>
<td>i=2, pump power</td>
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<td>21-30</td>
<td>i=3, sump area</td>
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<td>31-40</td>
<td>i=4, system overall depth</td>
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<td>i=5, sewer diameters</td>
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Note: 1 ft = 0.3048 m; 1 acre = 0.405 ha; 1 cfs = 0.028 m³/s
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<th>Manning's Coefficient (n)</th>
<th>Balanced Design</th>
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<td>DIAM(i) (ft)</td>
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<td>3</td>
<td>0.013</td>
<td>2.75</td>
</tr>
<tr>
<td></td>
<td>0.020</td>
<td>3.26</td>
</tr>
<tr>
<td>2</td>
<td>0.013</td>
<td>2.73</td>
</tr>
<tr>
<td>Time Step (min.)</td>
<td>Manning's Coefficient (n)</td>
<td>Cost Gradients $\times 10^3$ dollars/unit change</td>
</tr>
<tr>
<td>----------------</td>
<td>---------------------------</td>
<td>-----------------------------------------------</td>
</tr>
<tr>
<td></td>
<td>c₁</td>
<td>c₂</td>
</tr>
<tr>
<td>3</td>
<td>.013</td>
<td>10.17</td>
</tr>
<tr>
<td></td>
<td>.020</td>
<td>10.30</td>
</tr>
</tbody>
</table>
TABLE 5.2 EFFECT OF \( d_o \), \( c_i \) AND \( d_i \) ON THE OPTIMUM DESIGN

<table>
<thead>
<tr>
<th></th>
<th>Case (I) Number of Houses = 30 House/section</th>
<th>(Case II) Number of Houses = 10 House/section</th>
</tr>
</thead>
<tbody>
<tr>
<td>( c_0 ), Dollars</td>
<td>55,000</td>
<td>55,000</td>
</tr>
<tr>
<td>( d_o ), Dollars</td>
<td>50,400</td>
<td>19,500</td>
</tr>
<tr>
<td>Damage Gradients,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dollars/Unit Change</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( d_1 ) (Slope)</td>
<td>-39,750</td>
<td>-13,500</td>
</tr>
<tr>
<td>( d_2 ) (Pump)</td>
<td>-15,050</td>
<td>-5,140</td>
</tr>
<tr>
<td>( d_3 ) (Sump)</td>
<td>-1,150</td>
<td>-1,200</td>
</tr>
<tr>
<td>( d_4 ) (Depth)</td>
<td>-54,300</td>
<td>-22,300</td>
</tr>
<tr>
<td>( d_5 ) (Diameter)</td>
<td>-198,800</td>
<td>-67,400</td>
</tr>
<tr>
<td>Cost Gradients,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dollars/Unit Change</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( c_1 ) (Slope)</td>
<td>10,000</td>
<td>10,000</td>
</tr>
<tr>
<td>( c_2 ) (Pump)</td>
<td>4,650</td>
<td>4,650</td>
</tr>
<tr>
<td>( c_3 ) (Sump)</td>
<td>8,300</td>
<td>8,300</td>
</tr>
<tr>
<td>( c_4 ) (Depth)</td>
<td>15,700</td>
<td>15,700</td>
</tr>
<tr>
<td>( c_5 ) (Diameter)</td>
<td>22,000</td>
<td>22,000</td>
</tr>
<tr>
<td>Optimum Design</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1) Diameters, ft.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sewer (1)</td>
<td>3.0'</td>
<td>3.0'</td>
</tr>
<tr>
<td>Sewer (2)</td>
<td>3.3'</td>
<td>3.3'</td>
</tr>
<tr>
<td>Sewer (3)</td>
<td>3.5'</td>
<td>3.5'</td>
</tr>
<tr>
<td>(2) Slopes, %</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sewer (1)</td>
<td>0.9</td>
<td>0.54</td>
</tr>
<tr>
<td>Sewer (2)</td>
<td>0.9</td>
<td>0.54</td>
</tr>
<tr>
<td>Sewer (3)</td>
<td>0.9</td>
<td>0.54</td>
</tr>
<tr>
<td>(3) Sump Area, ft²</td>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td>(4) Pump</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rated Head, ft</td>
<td>23.7</td>
<td>25.1</td>
</tr>
<tr>
<td>Rated Discharge, cfs</td>
<td>74</td>
<td>36</td>
</tr>
<tr>
<td>(5) Characteristic</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth, ft</td>
<td>12.5</td>
<td>20.4</td>
</tr>
<tr>
<td></td>
<td>Case (I)</td>
<td></td>
</tr>
<tr>
<td>-------------------------------</td>
<td>----------</td>
<td>----------</td>
</tr>
<tr>
<td></td>
<td>Initial</td>
<td>Balanced</td>
</tr>
<tr>
<td>Diameters, ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sewer (1)</td>
<td>6.00</td>
<td>2.79</td>
</tr>
<tr>
<td>Sewer (2)</td>
<td>6.00</td>
<td>3.02</td>
</tr>
<tr>
<td>Sewer (3)</td>
<td>6.00</td>
<td>3.23</td>
</tr>
<tr>
<td>Pumping Capacity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_R$, cfs</td>
<td>30</td>
<td>63.2</td>
</tr>
<tr>
<td>Manhole Index</td>
<td>Ground Surface Elevation (ft)</td>
<td>Impervious Area (acre)</td>
</tr>
<tr>
<td>---------------</td>
<td>------------------------------</td>
<td>------------------------</td>
</tr>
<tr>
<td>1</td>
<td>112.0</td>
<td>13.0</td>
</tr>
<tr>
<td>2</td>
<td>109.0</td>
<td>3.0</td>
</tr>
<tr>
<td>3</td>
<td>98.0</td>
<td>3.0</td>
</tr>
<tr>
<td>4</td>
<td>97.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>
### TABLE 5.4.b CHARACTERISTICS OF DRAINAGE BASIN FOR THE EXAMPLE

<table>
<thead>
<tr>
<th></th>
<th>Sewer Section (1)</th>
<th>Sewer Section (2)</th>
<th>Sewer Section (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Houses Along Sewer Section</td>
<td>30</td>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>Basements Area Along Sewer Section = Acres</td>
<td>1.00</td>
<td>-1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Average Depth of Basements Below Ground Surface, ft</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Average Length of the Pipe of the House Connection, ft</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>Average Diameter of the Pipe of the House Connection, ft</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
</tr>
<tr>
<td>Portion of Property Value Assigned to Basements</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>Average Assessment Value of Houses, dollars</td>
<td>8,000</td>
<td>8,000</td>
<td>8,000</td>
</tr>
<tr>
<td>Assessment Factor</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Class Index</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>
TABLE 5.5 RAINFALL DATA

<table>
<thead>
<tr>
<th>Storm Index</th>
<th>Coefficients in the Rainfall Intensity/Time formula</th>
<th>Probability of Occurrence, (Annual Series)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$[i = \frac{a}{(t_a^b + c)}]$</td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>b</td>
<td>c</td>
</tr>
<tr>
<td>1</td>
<td>131.0</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>170.0</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>250.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Relative Advancement of the Peak Intensity = 0.375
TABLE 5.6 INITIAL DESIGN BY RATIONAL METHOD

<table>
<thead>
<tr>
<th>Line Number</th>
<th>Location</th>
<th>From</th>
<th>To</th>
<th>Length, ft</th>
<th>Increment (Acres)</th>
<th>Total (Acres)</th>
<th>Time of Flow (Min.)</th>
<th>Impervious Area % of Total = C</th>
<th>Runoff cfs/acre = Q=i</th>
<th>Total Runoff (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>U/S</td>
<td>1</td>
<td>2</td>
<td>600</td>
<td>32.5</td>
<td>32.5</td>
<td>25</td>
<td>1.4</td>
<td>0.4</td>
<td>1.21</td>
</tr>
<tr>
<td>(2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II</td>
<td></td>
<td>2</td>
<td>3</td>
<td>600</td>
<td>7.5</td>
<td>40</td>
<td>26.4</td>
<td>1.4</td>
<td>0.4</td>
<td>1.17</td>
</tr>
<tr>
<td>III</td>
<td></td>
<td>3</td>
<td>4</td>
<td>600</td>
<td>7.5</td>
<td>47.5</td>
<td>27.8</td>
<td>1.23</td>
<td>0.4</td>
<td>1.14</td>
</tr>
<tr>
<td>IV</td>
<td>Sump Well</td>
<td></td>
<td>Stream</td>
<td>50</td>
<td>7.5</td>
<td>55.0</td>
<td>29</td>
<td>--</td>
<td>0.4</td>
<td>1.11</td>
</tr>
</tbody>
</table>

i = 133/(t_d + 19), 1 in 5 years storm
<table>
<thead>
<tr>
<th>Line Number</th>
<th>Slope of Sewer %</th>
<th>Diameter, in.</th>
<th>Diameter, in. Commercial Sizes</th>
<th>Capacity Full (cfs)</th>
<th>Velocity Full ft/sec.</th>
<th>Sewer Crown Elevation (ft)</th>
<th>Ground Surface Elevation (ft)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.6</td>
<td>32.5&quot;</td>
<td>36&quot;</td>
<td>51.9</td>
<td>7.3</td>
<td>99.5</td>
<td>95.9</td>
<td>112.0</td>
</tr>
<tr>
<td>II</td>
<td>0.6</td>
<td>34.7&quot;</td>
<td>36&quot;</td>
<td>51.9</td>
<td>7.3</td>
<td>95.9</td>
<td>92.30</td>
<td>109.0</td>
</tr>
<tr>
<td>III</td>
<td>0.6</td>
<td>36.6&quot;</td>
<td>42&quot;</td>
<td>78.25</td>
<td>8.13</td>
<td>92.30</td>
<td>88.7</td>
<td>98.0</td>
</tr>
</tbody>
</table>


<table>
<thead>
<tr>
<th>Sewer No.</th>
<th>(1)</th>
<th>(2)</th>
<th>(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sewers</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Invert Slope = %</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Diameter, ft</td>
<td>3.0</td>
<td>3.0</td>
<td>3.5</td>
</tr>
<tr>
<td>Manning's n</td>
<td>0.013</td>
<td>0.013</td>
<td>0.013</td>
</tr>
<tr>
<td>Length = ft</td>
<td>600</td>
<td>600</td>
<td>600</td>
</tr>
<tr>
<td><strong>Pump</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rated Head = ft</td>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rated Discharge, cfs</td>
<td>61</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Sump Well</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cross Sectional Area, ft²</td>
<td>900</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
TABLE 5.8 COMPARISON BETWEEN THE MODEL’S OPTIMUM DESIGN AND THE RATIONAL METHOD

<table>
<thead>
<tr>
<th></th>
<th>Rational Method Design</th>
<th>Optimum by the Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fractional Sewer Sizes</td>
<td>Commercial Sewer Sizes</td>
</tr>
<tr>
<td><strong>(I) Sewers</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameters, ft;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sewer (1)</td>
<td>2.70</td>
<td>3.00</td>
</tr>
<tr>
<td>Sewer (2)</td>
<td>2.90</td>
<td>3.00</td>
</tr>
<tr>
<td>Sewer (3)</td>
<td>3.05</td>
<td>3.50</td>
</tr>
<tr>
<td><strong>Slopes, %</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sewer (1)</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Sewer (2)</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Sewer (3)</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td><strong>(II) Pump</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rated Head, ft</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Rated Discharge, cfs</td>
<td>61</td>
<td>61</td>
</tr>
<tr>
<td><strong>(III) Sump Well</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Area, ft²</td>
<td>900</td>
<td>900</td>
</tr>
<tr>
<td><strong>(IV) Depth</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Characteristic Depth, ft</td>
<td>12.5</td>
<td>12.5</td>
</tr>
<tr>
<td><strong>(V) Capital Recovery Cost, dollars/year</strong></td>
<td>54,400</td>
<td>56,400</td>
</tr>
<tr>
<td><strong>(VI) Average Annual Probable Damage dollars/year</strong></td>
<td>69,300</td>
<td>27,200</td>
</tr>
<tr>
<td><strong>(VII) Net Annual Cost</strong></td>
<td>123,700</td>
<td>83,600</td>
</tr>
</tbody>
</table>
**TABLE 5.9 CONSISTENCY OF DESIGN OBTAINED BY RATIONAL METHOD AND MODEL**

<table>
<thead>
<tr>
<th>Method of Design</th>
<th>Rational Design</th>
<th>Model (Optimum)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fractional Sewer Sizes</td>
<td>Commercial Sewer Sizes</td>
</tr>
<tr>
<td>Maximum Relative Storage, during a 1 in 5 years storm, $P_{\text{max}}$, of</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sewer (1)</td>
<td>0.981</td>
<td>0.764</td>
</tr>
<tr>
<td>Sewer (2)</td>
<td>1.029</td>
<td>0.948</td>
</tr>
<tr>
<td>Sewer (3)</td>
<td>1.088</td>
<td>0.758</td>
</tr>
</tbody>
</table>
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_{unit}  Cost per unit length of sewer.
C  Total annual costs of a perturbed system.
$C_{Bi}$  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

$C_{o1}, C_{o2}$  Muskingum Coefficients

Cost$_i$  Cost per unit length of sewer

CRF  Capital recovery factor

$\text{STBY}$  Installation cost of the stand-by pumping unit(s)

$\text{CT}_i$  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_{\text{Base}})_i \quad \text{Damages due to basements flooding}

D_{\text{factor}} \quad \text{Normalized damage factor}

D_{\text{Inc}} \quad \text{Inconvenience cost}

D_k \quad \text{Total damage resulting from applying a preselected storm on the perturbed system}

D_{\text{prop}} \quad \text{Property damages}

D_{\text{SHL}} \quad \text{Total head loss in a house connection at the downstream end of the sewer}

d_{f(i,t)} \quad \text{Basements flooding depth}

d_{\text{fmax}}^i \quad \text{Maximum value of the basement flooding depth}

d_{\text{h.con.}} \quad \text{Diameter of the pipe of the house connection}

D_i \quad \text{Damage cost gradient for variable } i

\tilde{d}_{\text{sump}} \quad \text{Sump excavation depth}

\tilde{d}_{\text{sys}} \quad \text{Characteristic depth of sewer system}

E(i,t) \quad \text{Mid-point elevation of sewer energy line}

E_{\text{crown}} \quad \text{Crown elevation of the downstream end of the outfall sewer}

E_d \quad \text{Energy level on the delivery side of the pump}

E_{\text{sump}} \quad \text{Energy level immediately upstream of the sump}

F \quad \text{Subscript represents variables at the final state}

F(i,t) \quad \text{Flooding ratio}

F_{\text{fmax}}^i \quad \text{Maximum value of flooding ratio}

FU \quad \text{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
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
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$l_{h\text{-con.}}$</td>
<td>Average length of the pipes of house connections with a particular sewer section</td>
</tr>
<tr>
<td>$l_i$</td>
<td>Sewer length</td>
</tr>
<tr>
<td>MNH($i$)</td>
<td>Manhole $i$</td>
</tr>
<tr>
<td>$m$</td>
<td>Exponent</td>
</tr>
<tr>
<td>$N$</td>
<td>Years of estimated life</td>
</tr>
<tr>
<td>($N_{cross^i}$)</td>
<td>Effective number of vehicle crossings per hour on the average basis</td>
</tr>
<tr>
<td>$N_{house}$</td>
<td>Number of houses along a particular sewer section</td>
</tr>
<tr>
<td>$n$</td>
<td>Manning's Coefficient</td>
</tr>
<tr>
<td>$n$</td>
<td>Number of increments of impervious area</td>
</tr>
<tr>
<td>$O$</td>
<td>Rate of outflow from the control volume considered</td>
</tr>
<tr>
<td>$P$</td>
<td>Power of installed pump</td>
</tr>
<tr>
<td>$P_{max_i}$</td>
<td>Maximum relative storage of the $i$-th sewer, during a specific storm</td>
</tr>
<tr>
<td>$P$</td>
<td>Probability of occurrence of the rain storm</td>
</tr>
<tr>
<td>$P_x$</td>
<td>Probability of occurrence of a storm resulting in the same damage when applied on the original balanced system</td>
</tr>
<tr>
<td>$P_K$</td>
<td>Probability of occurrence of a preselected representative storm</td>
</tr>
<tr>
<td>$Q$</td>
<td>Discharge</td>
</tr>
<tr>
<td>$Q_R$</td>
<td>Rated pumping discharge</td>
</tr>
<tr>
<td>$Q_{av(i,t)}$</td>
<td>Average flowrate in the $i$-th sewer</td>
</tr>
<tr>
<td>$Q_B(i,t)$</td>
<td>Basement flow</td>
</tr>
<tr>
<td>$Q_{in}$</td>
<td>Inflow to manhole</td>
</tr>
<tr>
<td>$Q_{inf(IMH,K)}$</td>
<td>Runoff contributing to manhole IMH at time $K\cdot\Delta t$</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$Q_o$</td>
<td>Flow capacity of the sewer when flowing full</td>
</tr>
<tr>
<td>$Q_{ot}$</td>
<td>Outflow from manhole</td>
</tr>
<tr>
<td>$Q_{outn}$</td>
<td>Flow rate at the downstream end of the n-th sewer</td>
</tr>
<tr>
<td>$Q_p$</td>
<td>Operating pumping flowrate</td>
</tr>
<tr>
<td>$(Q_R)^{adj}$</td>
<td>The adjusted pumping capacity</td>
</tr>
<tr>
<td>$Q_s$</td>
<td>Inflow to the sump</td>
</tr>
<tr>
<td>$Q_{s,house}$</td>
<td>Flow in house connection to the basement of a single house</td>
</tr>
<tr>
<td>$Q_{st}(i,t)$</td>
<td>Street flow</td>
</tr>
<tr>
<td>$Q_x$</td>
<td>Basement flow per unit length of the sewer</td>
</tr>
<tr>
<td>$q$</td>
<td>Distributed lateral inflow (or outflow) as discharge per unit length of the conduit</td>
</tr>
<tr>
<td>$RB_i$</td>
<td>Portion of property value assigned to basements</td>
</tr>
<tr>
<td>$REI$</td>
<td>Relative engineering index</td>
</tr>
<tr>
<td>$R_{stg}$</td>
<td>Relative storage</td>
</tr>
<tr>
<td>$r$</td>
<td>Relative advancement of the rainfall peak intensity</td>
</tr>
<tr>
<td>$S$</td>
<td>Storage within the control volume</td>
</tr>
<tr>
<td>$(S_B)^i$</td>
<td>Slope of basement grade line</td>
</tr>
<tr>
<td>$S_{e}(i,t)$</td>
<td>Slope of sewer energy line</td>
</tr>
<tr>
<td>$S_f$</td>
<td>Friction slope</td>
</tr>
<tr>
<td>$S_o$</td>
<td>Channel bottom slope</td>
</tr>
<tr>
<td>$S_{oi}$</td>
<td>Invert slope of the i-th sewer</td>
</tr>
</tbody>
</table>
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 \( V_{\text{max}_i} \)

\( \text{USEL}_i \): Elevation of the energy line at the upstream end of the \( i \)-th sewer

\( \text{USHL} \): 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

\( V_{\text{act}} \): Total actual storage

\( V_{\text{max}_i} \): 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

\( V_{\text{sys}} \): Total maximum available storage of the sewer system

\( W_{\text{street}} \): Width of street

\( X \): Average depth of excavation

\( X_i, X_j \): Decision (design) variable

\( XX_i \): Slack variable

\( X_{\text{LL}} \): 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

$\gamma_w$
Specific weight of water

$\Delta$
Small change

$n$
Pump efficiency
REFERENCES
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1950 Born on the 16th of October in Cairo, Egypt.

1973 Graduated with a B.Sc. in Civil Engineering from Cairo University, Egypt.

1973 Appointed as an instructor of Civil Engineering at Cairo University, Egypt.

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