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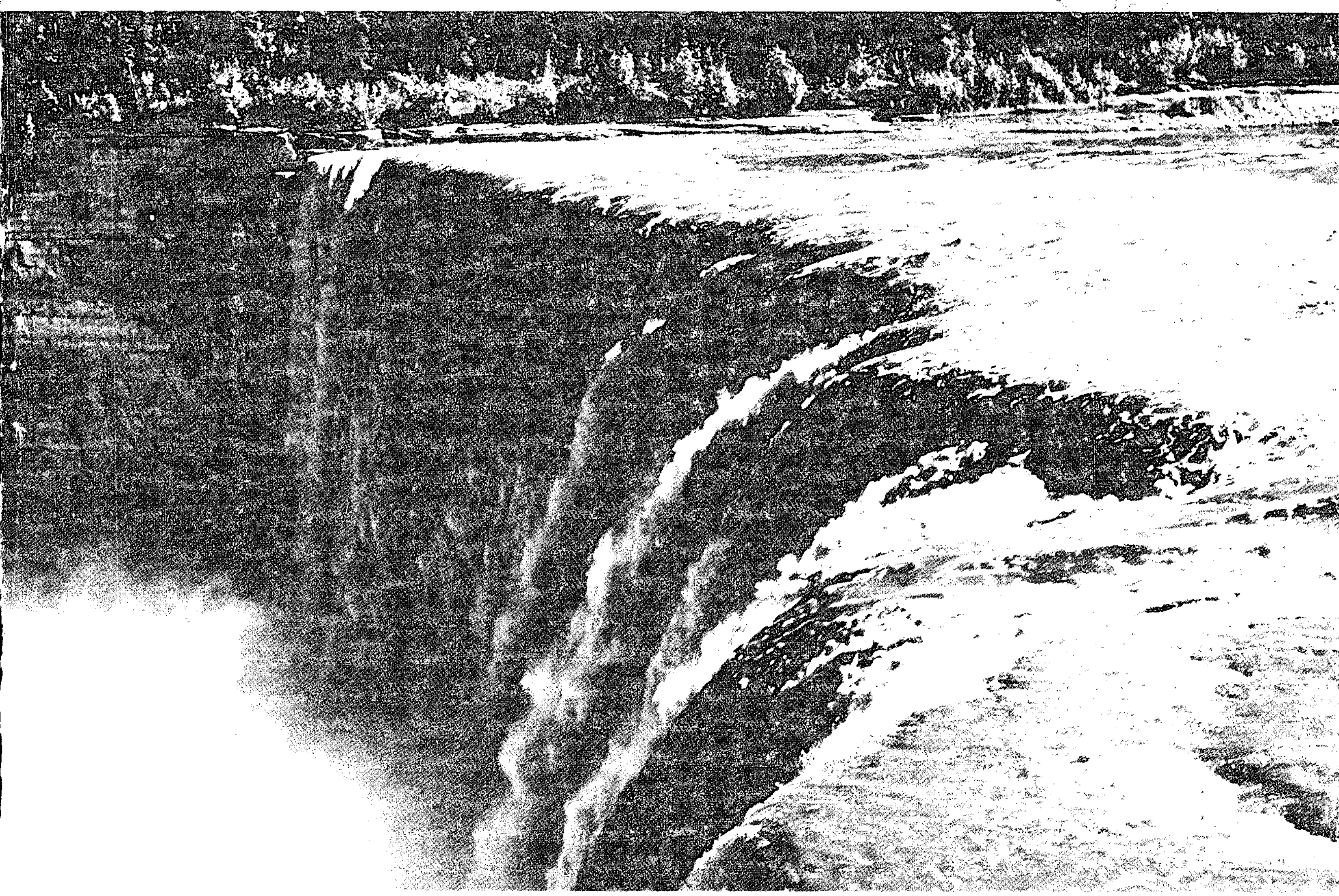

TECHNICAL BULLETIN NO. 120



NATIONAL WATER RESEARCH INSTITUTE
INLAND WATERS DIRECTORATE
CANADA CENTRE FOR INLAND WATERS
BURLINGTON, ONTARIO, 1981

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## PREFACE

This report is based on a draft contract report submitted by Bessette, Crevier, Parent, Tanguay and Associates (BCPTA) to the Department of Supply and Services. The development of the modified ILLUDAS model as well as its testing and sensitivity analysis was done by Mr. G. Patry and Mrs. L. Raymond of BCPTA.

The author provided technical direction for the project as a liaison officer, supplied data for model testing and prepared this summary report.

## MANAGEMENT PERSPECTIVE

Computer models that are to be used for engineering design should be verified correctly in order to establish confidence in the results. This report gives the verification of a new technique and shows clearly that reliable results are obtainable for a modified urban runoff model (ILLUDAS) by using lower cost desk-top computers.

The results can be used wherever runoff rates and quantities must be computed from rainfall events.

T. M. Dick, Chief
Hydraulics Division

## ABSTRACT

The standard version of the ILLUDAS model written for the IBM 360/75 computer was modified and adopted to a Hewlett-Packard 9830 desk-top computer. The modified model was verified on a test catchment and subjected to a sensitivity analysis.

For a small catchment with simple flow routing, the modified model performed equally as well as conventional models requiring large computer systems.

## RÉSUMÉ

On a modifié la version normale du modèle ILLUDAS écrit pour l'ordinateur IBM 360/75 et on l'a adaptée à un ordinateur de pupitre Hewlett-Packard 9830. On a vérifié le modèle modifié sur une prise d'eau d'essai et on l'a soumis à une analyse de la sensibilité.

Pour une petite prise d'eau à cours simple, le modèle modifié a fonctionné aussi bien que les modèles classiques qui nécessitent des systèmes informatiques puissants.

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**Adaptation of the ILLUDAS Model to a Desk-Top Computer**

**J. Marsalek**

## INTRODUCTION

In recent years many computer models for simulation of urban runoff have been developed. As the requirements on sophistication of these models increased, so did the requirements on computers used to run these models. On the one hand, there was some concern that further increases in the use of urban runoff models might be impeded because smaller municipalities and engineering companies would find the use of large commercial computer facilities either too expensive or inconvenient. On the other hand, small desk-top computers are becoming widespread and affordable even for small offices. It was therefore suggested that an increased use of runoff models would be encouraged by adapting one of these models to a desk-top computer. The model selected for this purpose had to be relatively simple and well accepted by the engineering profession. Both these objectives are met by the ILLUDAS (Illinois Urban Drainage Area Simulator) model, which was developed by the Illinois State Water Survey (5).

The development of a desk-top computer version of the ILLUDAS model was contracted by the Department of Supply and Services to the engineering company Bessette, Crevier, Parent, Tanguay and Associates (BCPTA). The terms of reference of this contract may be summarized as follows:

(1) Develop a desk-top computer version of the ILLUDAS model.

(2) Verify this ILLUDAS version on a test catchment.

(3) Conduct a sensitivity analysis of this ILLUDAS version.

The report that follows presents the results of the contractual study conducted by BCPTA.

## DESCRIPTION OF THE MODIFIED VERSION OF ILLUDAS MODEL

The new ILLUDAS model version, which was modified for use on a desk-top computer, not only retains all the features of the original ILLUDAS model (1974 version, ref. 5) but also adds some new features to the original model. Consequently, the description of the modified version starts with a description of the original version followed by a description of newly added features.

**Calculation of Runoff**

For runoff calculations, the catchment under investigation is divided into subcatchments, which represent homogeneous surface elements contributing to a single sewer pipe. On each subcatchment, two types of areas are considered: directly connected paved areas and pervious (grassed) areas. Runoff calculations for each of these two areas differ.

For directly connected paved surfaces, two physical factors need to be evaluated - the area and the time of travel from the farthest point to the inlet. Using this information, a curve of the travel time to inlet versus the contributing area is constructed (see Fig. 1). Such a curve can be approximated by a straight line connecting the point corresponding to the total contributing area with the origin (5).

The rainfall pattern is described as a step function, where the length of the step is a computational time step during which the rainfall intensity is assumed to be constant.

The rainfall pattern is reduced for losses. On paved areas, the losses consist of the initial surface wetting loss and the depression storage loss. Both these losses are typically combined and treated as the initial abstraction loss which is subtracted from the rainfall pattern. The remainder of rainfall will then appear as runoff from the paved area.

The development of the runoff hydrograph is shown in Fig. 1 and may be described as follows:

$$Q_1 = i_1 A_1$$
$$Q_2 = i_1 A_2 + i_2 A_1$$
$$\cdot$$
$$\cdot$$
$$\cdot$$
$$Q_n = i_1 A_n + i_2 A_{n-1} + \dots + i_n A_1 \qquad (1)$$

where Q is the runoff flow rate, i is the supply rate (the rainfall intensity minus the losses), A is the contributing area, and the subscripts correspond to the time steps counted from the start of the storm.

The calculation of runoff from pervious (grassed) areas is very similar to that described above for paved areas. Again, a curve of the travel time to inlet

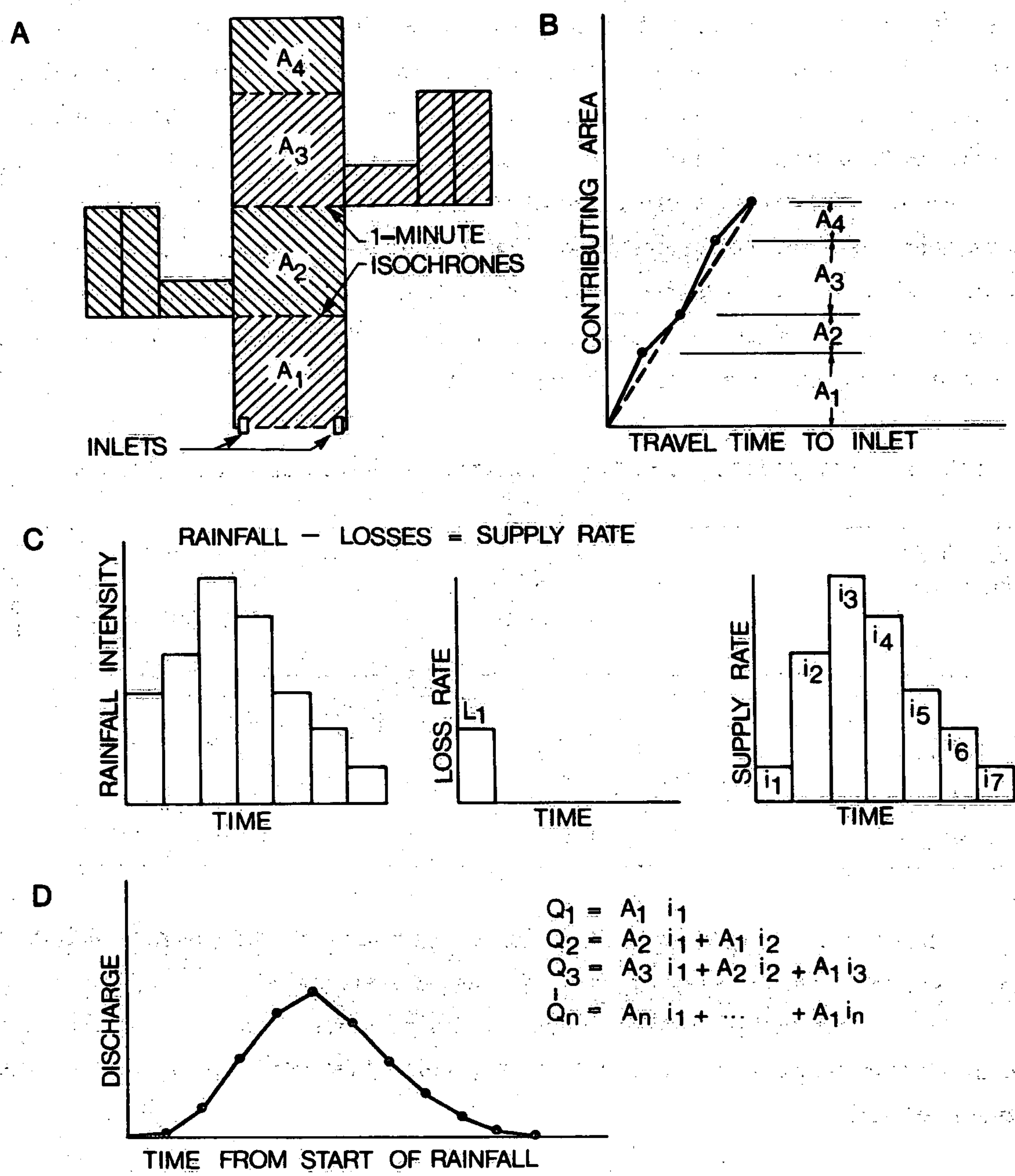



Fig.1. ILLUDAS APPROACH TO CALCULATION OF RUNOFF FROM IMPERVIOUS AREAS

versus the contributing area is constructed. The rainfall pattern, however, requires some modifications. First, the supplemental runoff from impervious areas draining onto pervious areas is added to the rainfall input for pervious areas (see Fig. 2). The rainfall pattern is then reduced for the initial abstraction loss and infiltration losses. The initial abstraction loss must be considered first, before any infiltration takes place. Infiltration curves were developed for the standard hydrologic soil groups A, B, C and D, as classified by the U.S. Soil Conservation Service. In order to use these infiltration curves properly, the antecedent moisture conditions prevailing at the time of a particular storm have to be evaluated and classified, as shown in Table 1. The antecedent moisture condition indices shown in Table 1 are based on the cumulative rainfall that occurred during the five days preceding the storm.

Table 1. Antecedent Moisture Conditions for Pervious Areas (Ref. 4)

| ILLUDAS Number | Description | Total Rainfall During 5 Days Preceding Storm (in.) |
|---|---|---|
| 1 | Bone Dry | 0 |
| 2 | Rather Dry | 0 to 0.5 |
| 3 | Rather Wet | 0.5 to 1.0 |
| 4 | Saturated | over 1.0 |

The rainfall pattern reduced for losses represents the supply rate, which is then used to derive the runoff hydrograph for pervious areas.

The runoff hydrographs from paved and pervious areas are combined for each subcatchment as a single hydrograph, which then becomes an input to the sewer network.

## Flow Routing

A simple storage routing technique is used to transfer the hydrograph from one input point to the next. For this purpose, a storage-discharge curve is developed for each reach of channel or pipe between the input points. First, the Manning equation is used to develop a stage-discharge curve for the reach under

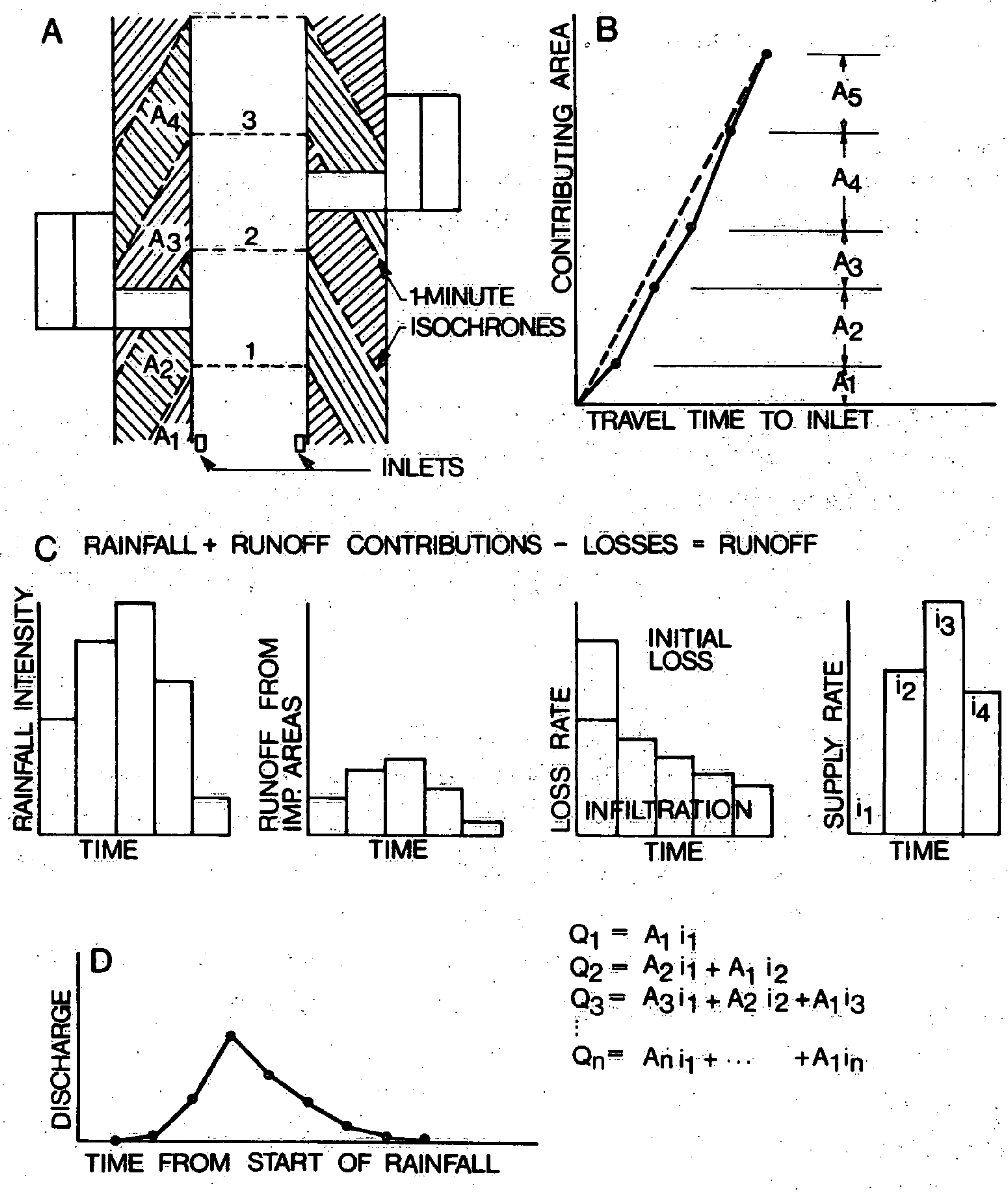



Fig.2. ILLUDAS APPROACH TO CALCULATION OF RUNOFF FROM PERVIOUS AREAS

consideration. From the reach length and cross-sectional dimensions, the storage-discharge curve is then calculated, assuming uniform flow in the reach. Errors caused by this assumption are minimized by keeping the time increment and the reach length as short as practicable (5).

The ILLUDAS routing procedure is shown in Fig. 3. The upper curve, O $Q_{1in}$ $Q_{2in}$, is a section of the inflow hydrograph at the upper end of the reach. The lower curve, O $Q_{1out} Q_{2out}$, is a section of the outflow hydrograph at the lower end of the reach. Using the notation in Fig. 3, one can write

$$\frac{1}{2} Q_{1in} \Delta t = \frac{1}{2} Q_{1out} \Delta t + S_1 \tag{2}$$

As $Q_{1in}$ and $\Delta t$ are known and $S_1$ can be expressed in terms of $Q_{1out}$ using the storage-discharge curve, Eq. 2 can be solved for $Q_{1out}$.

For the next time step,

$$(Q_{1in} + Q_{2in} - Q_{1out}) \Delta t/2 + S_1 = Q_{2out} \Delta t/2 + S_2 \tag{3}$$

The left side of Eq. 3 is known and the right side may be solved for $Q_{2out}$ using the storage-discharge relationship to evaluate $S_2$. Using this step-by-step procedure, all ordinates of the downstream hydrograph can be determined.

The ILLUDAS model also incorporates detention basins into the sewer system. In the analysis of an existing sewer system, the model accumulates the flows greater than the reach capacity, for each reach in the catchment. The maximum volume accumulated is reported in the output and is equivalent to the detention storage required to keep the system operating at capacity.

For a new drainage design, the user may specify the volume of detention storage allowable at any point in the catchment. The model will then incorporate that volume of storage into the design by filling the allowable storage with incoming flows.

## New Features of the Modified ILLUDAS Model

Practical applications of ILLUDAS in many projects undertaken by BCPTA indicated that the model versatility could be significantly enhanced by adding some new features to the original model (1974 version). These features are described in the following sections (3).

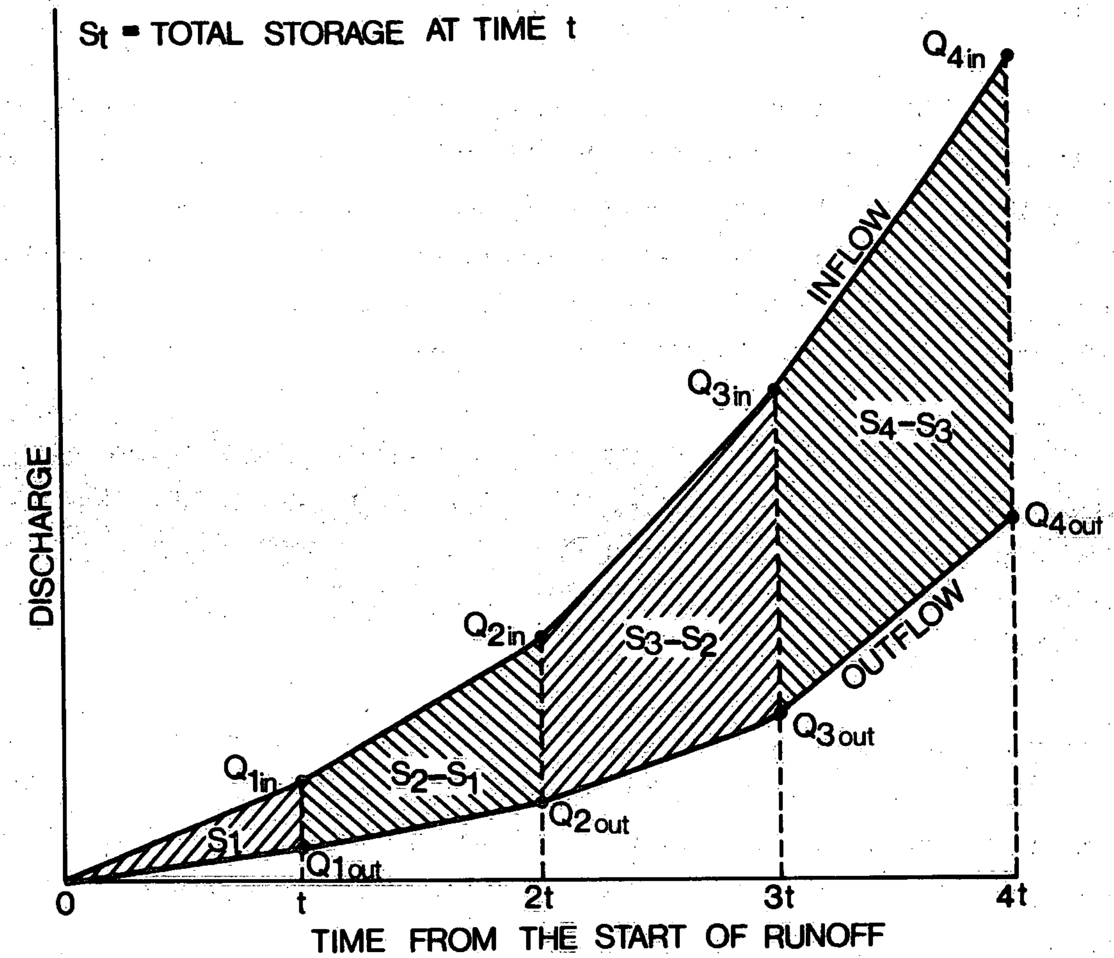



Fig.3. STORAGE ROUTING SCHEME

**Multiple rainfall hyetographs** - The input data structure has been modified to accommodate multiple hyetographs. Every subcatchment (i.e. every reach) may have its own hyetograph. This feature is particularly useful for investigating the effects of spatial rainfall distribution on runoff.

**Input of inlet hydrographs** - Schematization of large catchments may require more than 150 reaches allowed by the ILLUDAS model. To study such large catchments, they may have to first be divided into smaller units. The runoff from the upstream segment is then considered as an input, in the form of inlet hydrographs, to the downstream segments. Thus this feature makes it possible to simulate runoff from very large catchments by sequential simulation runs.

**Choice of soil infiltration parameters** - The original model allows the user to choose from four different soil groups to describe infiltration characteristics of a particular soil. The modified version allows the user to describe soil infiltration by Horton's parameters, $f_o$, $f_c$ and k, where $f_o$ is the initial infiltration rate, $f_c$ is the final infiltration rate, and k is the rate of decay.

**Choice of computations of inlet times** - There was some concern expressed that the ILLUDAS computation of inlet times for impervious surfaces might yield unrealistically short times (3). Consequently, an optional computation procedure, the kinematic wave equation, was included in the modified model in the following form:

$$T_i = \frac{0.93\, L^{0.6}\, n^{0.6}}{S^{0.3}} \qquad (4)$$

where $T_i$ is the inlet time (min), L is the length of overland flow (ft), n is the Manning roughness coefficient, and S is the slope (ft/ft) of the overland flow plane.

Compared with the original model computation, the kinematic wave equation yields longer times for impervious areas and shorter times for pervious areas. As discussed later, the use of the kinematic wave equation leads to lower runoff peak flows.

**Dry weather flow (base flow)** - When dealing with hydraulic problems in combined sewers, it is necessary to consider the dry weather flow. A new option was therefore added to the modified model, allowing the user to specify the total dry weather flow generated in the catchment. This total flow is then distributed to individual reaches in direct proportion to the contributing area for each reach.

**Design sewer diameter** - In the original version, the downstream pipe diameter has to be equal to or larger than the upstream diameter. This constraint was removed in the hydraulic design mode of the modified version. From the practical point of view, such a feature may be particularly useful where storage is added to the system.

**Storage on street surface** - In the analysis of an existing sewer system (referred to as the EVAL mode), runoff flows in excess of the pipe capacity are stored on the street surface and returned to the sewer system only when the runoff flow falls below the pipe capacity. The modified version was used in the calculation of the depth of ponding for the typical street cross section shown in Fig. 4.

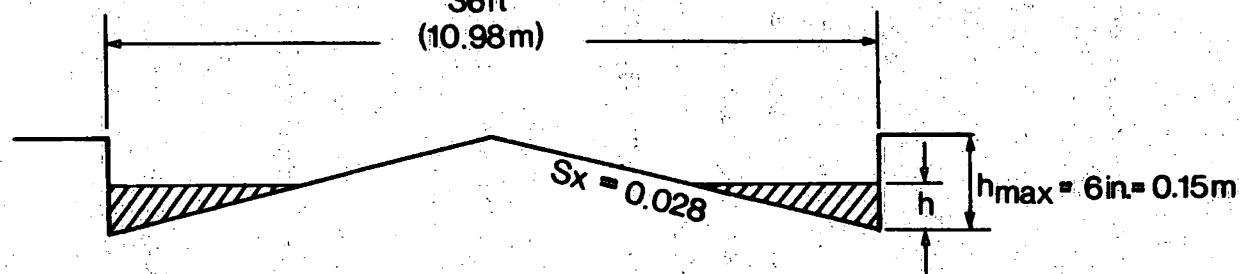
**Pressure flow analysis** - An approximate analysis of pressure sewer flow was added to the modified model version. In this analysis, referred to as the GRAD mode, the sewer system is allowed to surcharge and the corresponding hydraulic grade line is determined. Although the procedure is not very exact, it allows a quick evaluation of hydraulic conditions in the analyzed sewer system.

**Comparison of simulated and observed hydrographs** - A new subroutine serving for the evaluation of the goodness of fit between simulated and observed hydrographs was added to the model. The goodness of fit is evaluated using the following six parameters:

$$Q_{obs}/Q_{sim},\ V_{obs}/V_{sim},\ T_{obs}/T_{sim},\ R,\ R_s,\ \text{and ISE}$$

where Q is the runoff peak flow, V is the runoff volume, T is the time to the runoff peak flow, R is the correlation coefficient, $R_s$ is the special correlation

ASSUMED STREET CROSS SECTION




STORAGE VOLUME : $V = 9 ft^3/ft$ of street

LENGTH OF STORAGE : TAKE LARGER OF THE FOLLOWING VALUES

a) REACH LENGTH (ft)
b) 170 x SUBCATCHMENT AREA (ac)

Fig.4. RUNOFF STORAGE ON STREETS

coefficient, ISE is the integral square error, and subscripts obs and sim refer to observations and simulations, respectively. Definitions of the statistical parameters R, $R_s$ and ISE are given in the Appendix.

**Graphical presentation of results** - The output of simulation results in the original model was completely revised. Both simulated and observed hydrographs can be plotted for a fast visual inspection.

In summary, the modifications outlined above increased the versatility of the ILLUDAS model without greatly affecting the basic computations included in the original model. Possible exceptions to this statement are the optional calculations of inlet times from the kinematic wave equation and the approximate pressure flow analysis.

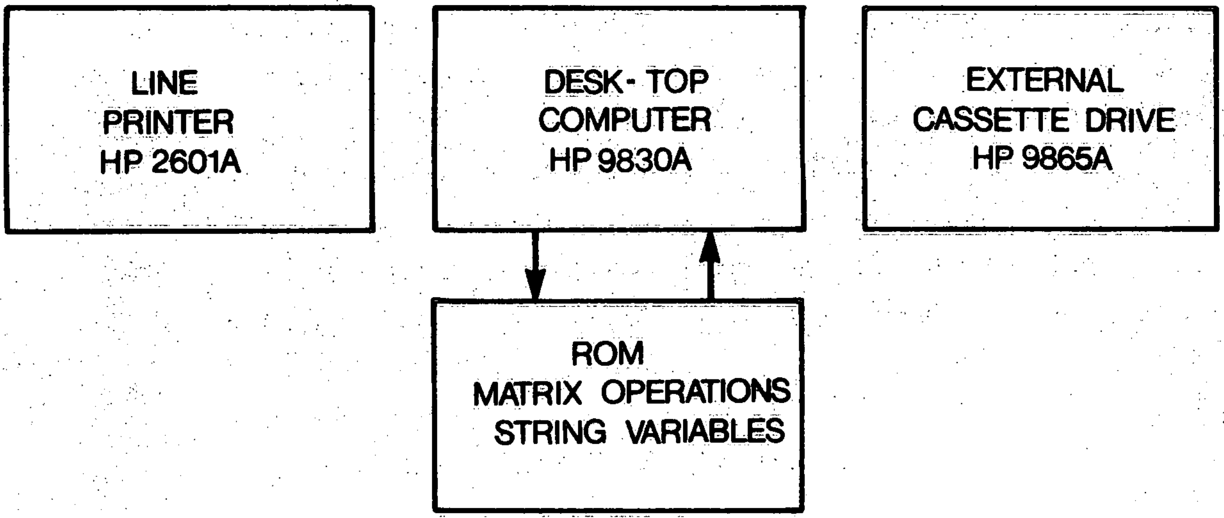
## MODIFIED ILLUDAS PROGRAM

In this section, a general description of the interactive modified ILLUDAS program is given. This particular model version was prepared in the BASIC computer language by BCPTA Consulting Engineers for a particular desk-top computer. For other computer systems, the model may require further modifications. The agency which prepared this report has neither the mandate nor resources to undertake such modifications for various user systems. Such a task could be efficiently handled by computer consultants at relatively low costs.

For brevity, the program listing has been omitted from this report. The program listing for the modified ILLUDAS model and descriptions of variables and sample runs can be obtained, free of charge, by writing to the Hydraulics Division, National Water Research Institute, P. O. Box 5050, Burlington, Ontario, L7R 4A6.

### Computer Hardware Description

The modified ILLUDAS program was prepared by BCPTA for the computer system shown schematically in Fig. 5. The heart of the system is an HP 9830 computer. The program files are read sequentially using an internal cassette drive and loaded into the system memory that has been expanded to 16K bytes. Matrix and character string manipulations are handled by two external ROMs,




SPECIFICATIONS

| | |
|---|---|
| DESK-TOP COMPUTER (PROGRAMMABLE CALCULATOR) | HP 9830 A |
| EXTENDED MEMORY 16K BYTES | HP 11281 A |
| MATRIX OPERATIONS ROM | HP 11270B & OPTION 270 |
| STRING VARIABLES ROM | HP 11274B & OPTION 274 |
| EXTERNAL CASSETTE DRIVE | HP 9865A |
| LINE PRINTER | HP 2607A |

Fig.5. DESK-TOP COMPUTER SYSTEM USED IN THE STUDY

shown in Fig. 5. Once a file is loaded, it is executed in a sequential manner. An external cassette memory is used to load or store data. Simulation results are printed on a 132-character line printer.

### Interactive Program Features

The modified version of ILLUDAS operates in an interactive mode. The program asks for various input data which are entered in a free format. Any syntax errors are brought to the user's attention. The input data can be printed and stored on tape. Once the checking of input data is completed, the user transfers the control to the simulation part of the program. At the end of the simulation, the user regains control of the program. The options available at this point include storage of runoff hydrographs on tape and a statistical analysis of the simulated and observed hydrographs.

### Program Flow Chart

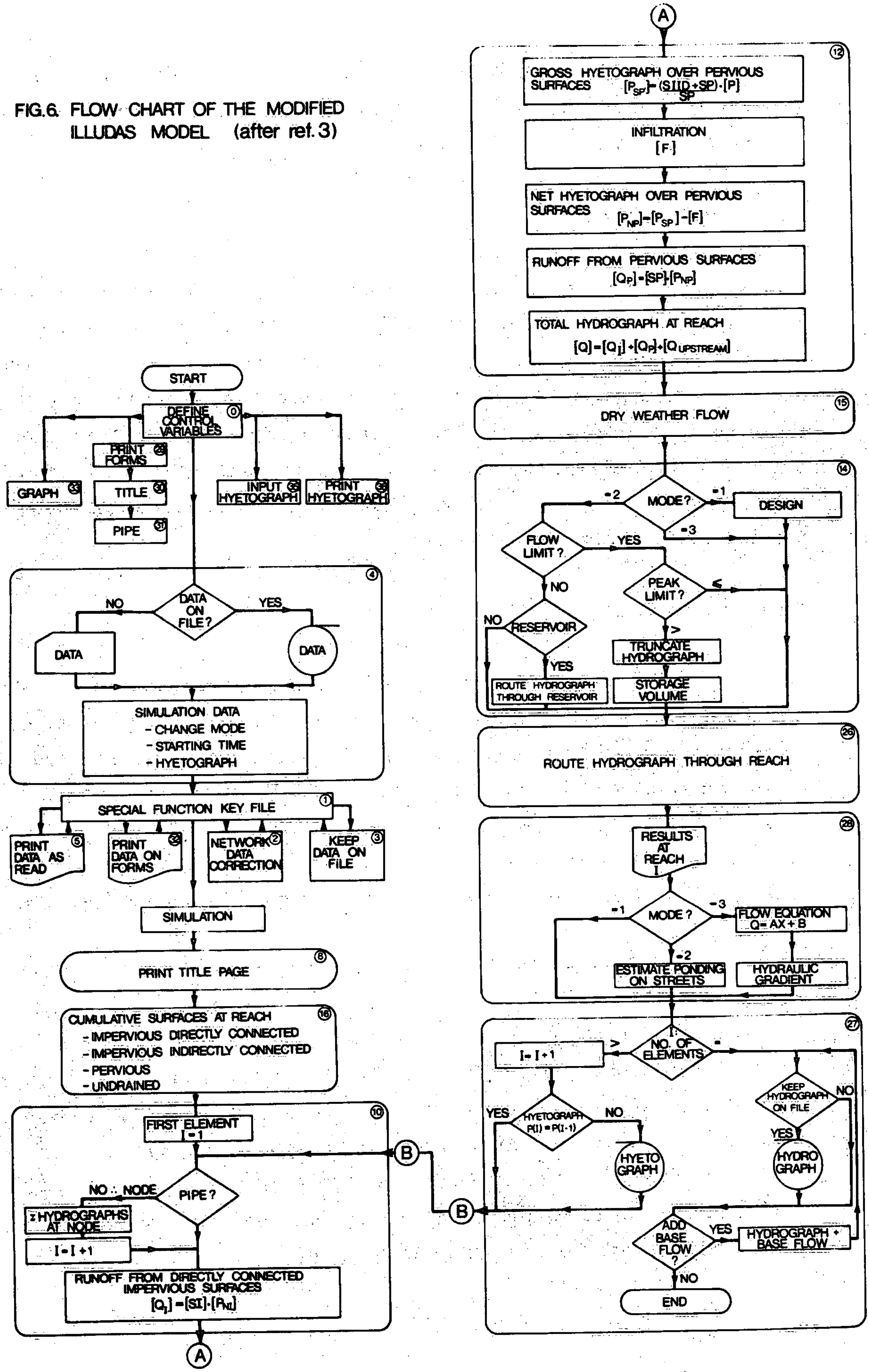
The original ILLUDAS model (1974) contained about 1100 Fortran statements and required 220K bytes of core when run on an IBM 360/75 computer (5). To adapt this model to a desk-top computer, major revisions were needed. The program was completely rewritten in the BASIC language and divided into 23 files that could be loaded and executed sequentially. The final modified version contains over 1500 statements.

The flow chart of the modified ILLUDAS program is shown in Fig. 6.

## VERIFICATION OF THE MODIFIED ILLUDAS MODEL

One of the study objectives was to verify the modified desk-top computer version of the ILLUDAS model on a Canadian urban test catchment. The catchment selected for this purpose was the Malvern catchment, which had been monitored for a number of years. Furthermore, simulation results obtained with the Storm Water Management Model (SWMM) were available for the Malvern catchment, and these results could be used as a yardstick for evaluating the results obtained with the modified ILLUDAS model.

FIG.6. FLOW CHART OF THE MODIFIED ILLUDAS MODEL (after ref. 3)



### Test Catchment Description

The Malvern urban test catchment is a modern residential development located in Burlington, Ontario. Runoff from the catchment has been monitored for a number of years. Monitoring results as well as detailed catchment characteristics have been reported elsewhere (1, 2). A brief description of the catchment is given below.

The Malvern catchment has an area of 23.3 ha (57.6 acres) of which 7.88 ha (19.5 acres) is impervious. The catchment is gently sloping (s=0.01) in the northeast-southwest direction; local slopes, however, depend on the grading of lots. The soil in the catchment can be characterized as a well-drained sandy loam. A summary of catchment surface characteristics is given in Table 2; estimates of pertinent hydrologic parameters used in earlier studies are given in Table 3.

Table 2. Malvern Catchment Surface Characteristics

| Surface | Area | | | | Percent of Catchment Area |
|---|---|---|---|---|---|
| | Impervious | | Pervious | | |
| | (ac) | (ha) | (ac) | (ha) | |
| Backyards | - | - | 30.10 | 12.18 | 52.2 |
| Front yards | - | - | 8.00 | 3.24 | 13.9 |
| Driveways | 3.10 | 1.25 | - | - | 5.4 |
| Roofs | 8.10 | 3.28 | - | - | 14.1 |
| Sidewalks | 1.62 | 0.66 | - | - | 2.8 |
| Streets | 6.68 | 2.70 | - | - | 11.6 |
| Total | 19.50 | 7.89 | 38.10 | 15.42 | 100.0 |

The catchment is served by a tree-type converging network of storm sewers. Table 4 lists basic characteristics of this sewer network. The sewers are made of concrete pipes; their roughness is characterized by the Manning roughness coefficient n=0.013.

**Table 3. Malvern Catchment - Estimates of Parameters Used in Previous Studies**

| Parameter | Pervious Area | Impervious Area |
|---|---|---|
| Ground Slope (ft/ft) | 0.03 | 0.03 |
| Overland Flow Length (ft) | 143.3 | 143.3 |
| Manning n for Overland Flow | 0.25 | 0.013 |
| Surface Depression Storage (in.) | 0.184 | 0.020 |
| Horton's Infiltration Parameters | | |
| $f_o$ (in./h) | 3.00 | - |
| $f_c$ (in./h) | 0.52 | - |
| k ($s^{-1}$) | 0.00115 | - |

**Verification Rainfall/Runoff Events**

Twelve events were selected for the verification of the modified ILLUDAS model. Characteristics of these events are given in Table 5.

It should be stressed that all the verification events have a fairly high frequency of occurrence; the most severe event produced a runoff peak with a return period of about one year.

On the average, the verification storms produced a rainfall depth of about 16 mm (0.63 in.) and their duration was slightly over four hours. The average five-day antecedent rainfall was about 16 mm (0.63 in.).

**Runoff Simulations with the Modified ILLUDAS**

The selected rainfall/runoff events were reproduced, for the Malvern catchment, by the modified version of ILLUDAS which was run on a Hewlett-Packard programmable calculator HP9830 (16K bytes). Details of these simulations follow.

The Malvern catchment was subdivided into 40 subcatchments which were drained by 40 sewer pipes. The characteristics of these subcatchments are shown in Table 6.

## Table 4. Malvern Catchment - Storm Sewers

| Pipe | Drains into Pipe Number | Pipe Diameter (in.) | Pipe Length (ft) | Invert Slope (%) |
|---|---|---|---|---|
| 1 | 2 | 12 | 295 | 0.80 |
| 2 | 3 | 15 | 220 | 0.70 |
| 3 | 4 | 18 | 225 | 0.50 |
| 4 | 8 | 18 | 300 | 0.50 |
| 5 | 6 | 12 | 149 | 0.50 |
| 6 | 7 | 12 | 210 | 0.80 |
| 7 | 8 | 12 | 213 | 1.30 |
| 8 | 9 | 18 | 151 | 1.00 |
| 9 | 12 | 18 | 148 | 1.32 |
| 10 | 11 | 12 | 266 | 0.80 |
| 11 | 12 | 15 | 260 | 0.80 |
| 12 | 17 | 21 | 187 | 1.20 |
| 13 | 14 | 12 | 132 | 0.50 |
| 14 | 15 | 15 | 291 | 0.50 |
| 15 | 16 | 15 | 292 | 0.50 |
| 16 | 17 | 18 | 298 | 0.50 |
| 17 | 21 | 24 | 242 | 1.00 |
| 18 | 19 | 12 | 229 | 0.50 |
| 19 | 20 | 12 | 156 | 1.50 |
| 20 | 21 | 21 | 304 | 2.00 |
| 21 | 22 | 27 | 192 | 1.20 |
| 22 | 24 | 27 | 192 | 1.20 |
| 23 | 24 | 10 | 140 | 1.50 |
| 24 | 25 | 27 | 161 | 0.90 |
| 25 | 40 | 30 | 396 | 0.50 |
| 26 | 27 | 12 | 268 | 0.90 |
| 27 | 28 | 15 | 300 | 1.00 |
| 28 | 30 | 18 | 301 | 0.68 |
| 29 | 30 | 10 | 160 | 1.20 |
| 30 | 31 | 18 | 224 | 1.20 |
| 31 | 33 | 18 | 296 | 1.56 |
| 32 | 33 | 10 | 88 | 0.60 |
| 33 | 34 | 27 | 273 | 0.24 |
| 34 | 35 | 27 | 273 | 0.24 |
| 35 | 39 | 27 | 194 | 0.20 |
| 36 | 37 | 12 | 247 | 0.70 |
| 37 | 38 | 12 | 172 | 2.00 |
| 38 | 39 | 12 | 238 | 2.36 |
| 39 | 40 | 27 | 280 | 0.42 |
| 40 | Outfall | 33 | 176 | 0.86 |

**Table 5. Characteristics of Verification Events**

| Event Number | Date Day/Mo/Yr | Total Rainfall | | Storm Duration | 5-Day Antecedent Rainfall | | Antecedent Moisture Index | Maximum Temporary Intensity | |
|---|---|---|---|---|---|---|---|---|---|
| | | (in.) | (mm) | (h) | (in.) | (mm) | | (in./h) | (mm/h) |
| 1 | 22/09/73 | 0.71 | 18 | 2.30 | 0.75 | 19 | 3 | 2.10 | 53 |
| 2 | 23/09/73 | 0.36 | 9 | 2.17 | 1.02 | 26 | 4 | 1.65 | 42 |
| 3 | 13/10/73 | 0.31 | 8 | 2.63 | 0.00 | 0 | 1 | 0.60 | 15 |
| 4 | 28/10/73 | 1.16 | 29 | 11.08 | 0.13 | 3 | 2 | 0.30 | 8 |
| 5 | 29/10/73 | 1.43 | 38 | 11.83 | 1.66 | 42 | 4 | 0.48 | 12 |
| 6 | 14/11/73 | 0.60 | 15 | 9.83 | 0.00 | 0 | 1 | 0.54 | 14 |
| 7 | 15/11/73 | 0.71 | 18 | 4.33 | 0.64 | 16 | 3 | 0.36 | 9 |
| 8 | 28/11/73 | 0.47 | 12 | 4.08 | 0.48 | 12 | 2 | 0.54 | 14 |
| 9 | 31/05/74 | 0.63 | 16 | 0.68 | 1.40 | 36 | 4 | 3.48 | 88 |
| 10 | 04/07/74 | 0.24 | 6 | 0.27 | 0.89 | 23 | 3 | 2.76 | 70 |
| 11 | 28/09/74 | 0.63 | 16 | 1.52 | 0.08 | 2 | 2 | 2.03 | 52 |
| 12 | 20/11/74 | 0.30 | 8 | 2.37 | 0.59 | 15 | 3 | 0.60 | 15 |
| Mean | | 0.63 | 16 | 4.43 | 0.65 | 16 | 2.7 | 1.29 | 32.7 |

Table 6. **Malvern Catchment - Discretization for ILLUDAS Simulations (1973 Data, Ref. 3)**

| Subcatchment Number | Total Area (acres) | Impervious Area Directly Connected (acres) | Contributing Pervious Area* (acres) | Maximum Length of Travel on Impervious Areas (ft) |
|---|---|---|---|---|
| 1 | 1.47 | 0.50 | 0.20 | 248 |
| 2 | 1.82 | 0.62 | 0.25 | 308 |
| 3 | 1.56 | 0.53 | 0.22 | 272 |
| 4 | 1.56 | 0.54 | 0.21 | 312 |
| 5 | 0.63 | 0.22 | 0.09 | 175 |
| 6 | 0.92 | 0.33 | 0.12 | 230 |
| 7 | 1.08 | 0.39 | 0.15 | 262 |
| 8 | 1.69 | 0.60 | 0.23 | 276 |
| 9 | 0.76 | 0.27 | 0.10 | 200 |
| 10 | 1.11 | 0.47 | 0.13 | 283 |
| 11 | 1.25 | 0.53 | 0.15 | 313 |
| 12 | 1.44 | 0.59 | 0.18 | 274 |
| 13 | 1.20 | 0.56 | 0.13 | 286 |
| 14 | 1.07 | 0.50 | 0.12 | 262 |
| 15 | 1.48 | 0.69 | 0.17 | 342 |
| 16 | 1.50 | 0.70 | 0.17 | 345 |
| 17 | 1.93 | 0.77 | 0.24 | 317 |
| 18 | 1.27 | 0.39 | 0.18 | 265 |
| 19 | 1.14 | 0.35 | 0.17 | 243 |
| 20 | 1.37 | 0.42 | 0.20 | 280 |
| 21 | 2.23 | 0.72 | 0.32 | 298 |
| 22 | 1.29 | 0.46 | 0.17 | 242 |
| 23 | 0.45 | 0.16 | 0.06 | 120 |
| 24 | 1.37 | 0.54 | 0.17 | 227 |
| 25 | 1.07 | 0.54 | 0.11 | 329 |
| 26 | 1.64 | 0.47 | 0.25 | 284 |
| 27 | 1.99 | 0.57 | 0.30 | 334 |
| 28 | 2.10 | 0.60 | 0.30 | 351 |
| 29 | 0.56 | 0.16 | 0.08 | 130 |
| 30 | 2.40 | 0.69 | 0.36 | 313 |
| 31 | 1.67 | 0.51 | 0.24 | 310 |
| 32 | 0.69 | 0.22 | 0.10 | 164 |
| 33 | 1.98 | 0.63 | 0.28 | 335 |
| 34 | 1.65 | 0.53 | 0.24 | 323 |
| 35 | 1.41 | 0.45 | 0.20 | 284 |
| 36 | 1.88 | 0.43 | 0.30 | 324 |
| 37 | 1.44 | 0.33 | 0.23 | 260 |
| 38 | 1.41 | 0.33 | 0.23 | 255 |
| 39 | 2.45 | 0.57 | 0.40 | 309 |
| 40 | 1.72 | 0.61 | 0.23 | 248 |

*Front yards

Inlet times for both impervious and pervious areas were determined using the procedures in the original ILLUDAS model. For impervious areas, the inlet times varied from 2.5 minutes to 3.3 minutes. For pervious areas, the calculation was limited to the front yards only, since backyards were unlikely to produce any runoff for the storms studied. The mean inlet time for front yards was 16.8 minutes.

The runoff simulation results are listed in Table 7 and plotted in Fig. 8. A discussion of verification results follows.

**Table 7. Verification Results Obtained with the Modified ILLUDAS Model (Basic Data after Ref. 3)**

| Event Number | Peak Flows | | | Runoff Volumes | | | Times to Peak | | |
|---|---|---|---|---|---|---|---|---|---|
| | $Q_{obs}$ (cfs) | $Q_{sim}$ (cfs) | $\frac{Q_{obs}}{Q_{sim}}$ | $V_{obs}$ ($ft^3$) | $V_{sim}$ ($ft^3$) | $\frac{V_{obs}}{V_{sim}}$ | $T_{obs}$ (min) | $T_{sim}$ (min) | $T_{obs}-T_{sim}$ (min) |
| 1 | 32.40 | 33.5 | 0.97 | 54 600 | 48 343 | 1.13 | 42 | 42 | 0 |
| 2 | 25.26 | 21.5 | 1.17 | 25 400 | 23 770 | 1.07 | 117 | 122 | - 5 |
| 3 | 8.45 | 7.9 | 1.07 | 19 900 | 20 288 | 0.98 | 112 | 110 | + 2 |
| 4 | 8.52 | 5.4 | 1.58 | 86 100 | 79 969 | 1.08 | 316 | 310 | + 6 |
| 5 | 10.86 | 8.5 | 1.28 | 110 300 | 102 411 | 1.08 | 437 | 425 | +12 |
| 6 | 10.47 | 9.3 | 1.13 | 44 200 | 40 667 | 1.09 | 344 | 355 | -11 |
| 7 | 6.47 | 6.9 | 0.94 | 46 100 | 48 398 | 0.95 | 142 | 145 | - 3 |
| 8 | 9.54 | 8.1 | 1.18 | 32 800 | 33 732 | 0.97 | 27 | 35 | - 8 |
| 9 | 31.82 | 37.4 | 0.85 | 44 717 | 39 011 | 1.15 | 34 | 30 | + 4 |
| 10 | 27.21 | 23.4 | 1.16 | 15 925 | 14 023 | 1.14 | 13 | 9 | + 4 |
| 11 | 15.11 | 18.5 | 0.82 | 36 183 | 39 316 | 0.92 | 13 | 9 | + 4 |
| 12 | 8.81 | 7.1 | 1.24 | 20 283 | 17 955 | 1.13 | 54 | 52 | + 2 |
| Mean | 16.24 | 15.63 | 1.12 | 44 709 | 42 324 | 1.06 | 137.6 | 137.0 | 0.6 |
| Standard Deviation | 9.93 | 11.11 | 0.21 | 28 193 | 25 975 | 0.08 | 146.3 | 145.3 | 6.4 |
| Coefficient of Variation (%) | 61.15 | 71.08 | 18.75 | 63.06 | 61.37 | 7.55 | 106.32 | 106.06 | - |

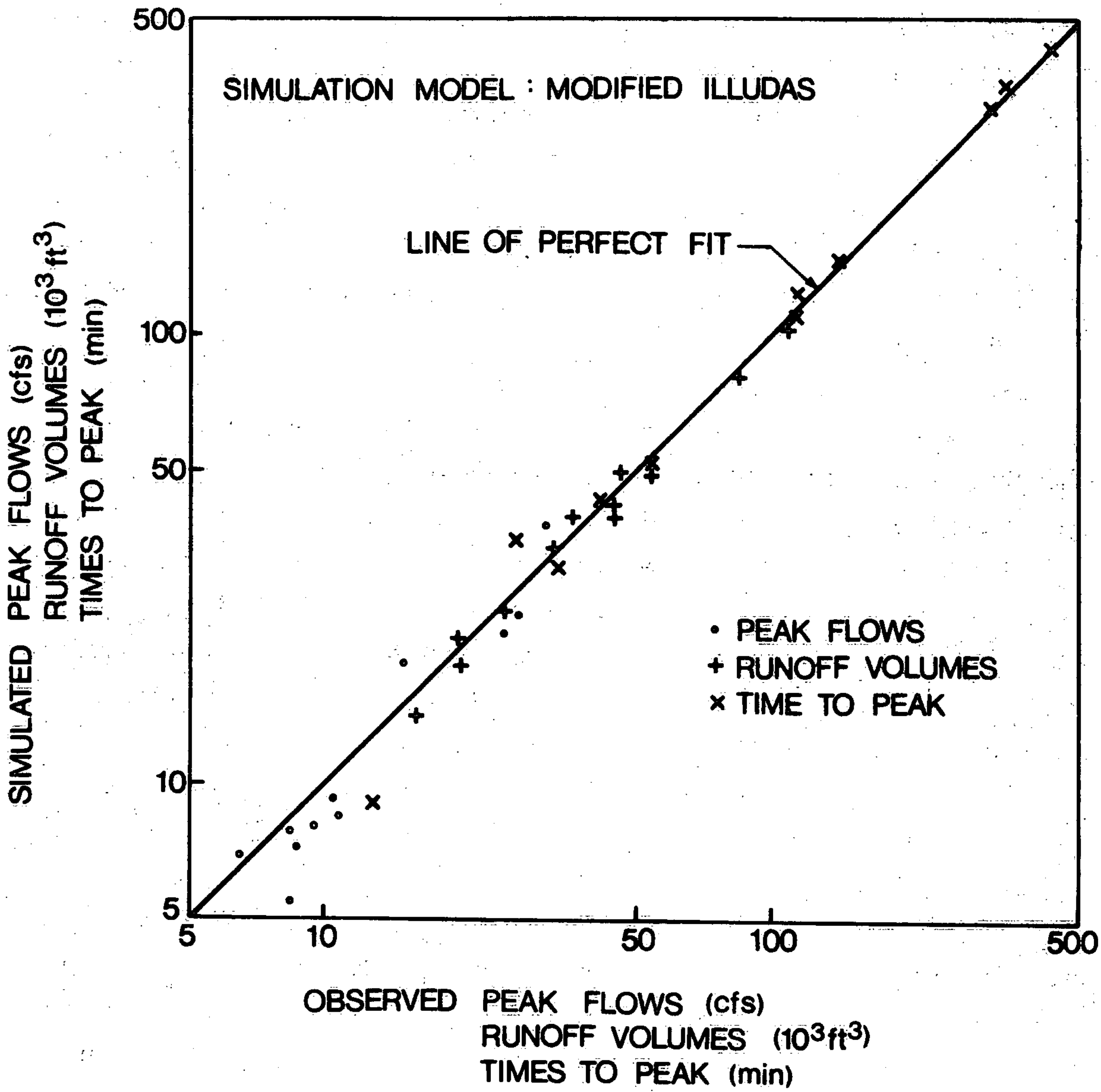



Fig. 7. PARAMETERS OF OBSERVED AND SIMULATED RUNOFF HYDROGRAPHS

The simulated runoff peak flows were on the average about 11 percent smaller than the observed ones, with the coefficient of variation of 19 percent. Such a goodness of fit is about the same as reported earlier for other runoff models (1, 2). The deviation between observed and simulated results was largely caused by poor results for two events of low rainfall intensity. Without these two events, the mean error in the simulated peaks was reduced to 5 percent. It should be recognized that deviations between simulated and observed results are caused not only by modelling bias but also by errors in the observed rainfall and runoff. Such errors may have contributed to poor results obtained for the two events discussed here.

Note also that while the observed peak flows represent instantaneous peaks, the simulated peak flows are averaged over the computational time step. Thus there is an inherent tendency in the simulated peak flows to underestimate the observed peaks.

Simulated runoff volumes were about six percent smaller than the observed ones. The coefficient of variation, about the mean, of the ratio $V_{obs}/V_{sim}$ was 7.5 percent. This underestimation may have been affected by an overestimation of losses on impervious areas. Note that a possible undercatch of the catchment rain gauge would also contribute to low simulated runoff volumes.

Times to runoff peak were simulated fairly accurately. On the average, the difference between simulated and observed times was less than one minute and the standard deviation was about six minutes.

The statistical parameters recommended by Sarma, Delleur and Rao (4) for evaluation of the goodness of fit of simulated and observed hydrographs were also studied. For this purpose, the timing of the simulated hydrographs was first adjusted to minimize the integral square error. The resulting changes in timing were characterized by a mean time shift of 0.83 minutes and a standard deviation of six minutes. After this adjustment, the goodness of fit of all of the simulated and observed hydrographs was rated as good to very good.

Attempts to improve simulation results by accounting for the antecedent moisture conditions failed, as such considerations affect only runoff from pervious areas which did not contribute significantly to the catchment runoff.

Finally, the verification results presented are affected by two limitations - a relatively small number of events and their fairly high frequency of occurrence. In none of the selected events did the pervious areas contribute significantly to the total catchment runoff.

## COMPARISON BETWEEN ILLUDAS AND SWMM SIMULATIONS FOR MALVERN CATCHMENT

The simulation results obtained for the Malvern catchment with the modified ILLUDAS model can be further evaluated by comparing them with those obtained earlier with the SWMM model (Table 8). Such a comparison is of particular interest because the SWMM model is perhaps the most widely accepted and applied urban runoff model. The significance of this comparison should not be overstated, because the SWMM model, in its entirety, has a much wider scope than the ILLUDAS model. There are, however, practical applications in which the desk-top computer version of ILLUDAS may be successfully used to replace a much more complex model.

**Table 8. Comparison of Verification Results Obtained with the ILLUDAS and SWMM Models (Basic Data after Ref. 3)**

| Event Number | Peak Flows | | | Runoff Volumes | | | Times to Peak | | |
|---|---|---|---|---|---|---|---|---|---|
| | $Q_{I}$* (cfs) | $Q_{S}$† (cfs) | $Q_S/Q_I$ | $V_I$ ($ft^3$) | $V_S$ ($ft^3$) | $V_S/V_I$ | T (min) | $T_S$ (min) | $T_S - T_I$ (min) |
| 1 | 33.5 | 34.50 | 1.0299 | 48 343 | 49 500 | 1.0239 | 42 | 40 | - 2 |
| 2 | 21.5 | 22.40 | 1.0419 | 23 770 | 24 400 | 1.0265 | 122 | 122 | 0 |
| 3 | 7.9 | 8.30 | 1.0506 | 20 288 | 20 900 | 1.0302 | 110 | 110 | 0 |
| 4 | 5.4 | 5.40 | 1.0000 | 79 969 | 80 200 | 1.0029 | 310 | 321 | +11 |
| 5 | 8.5 | 8.80 | 1.0353 | 102 411 | 103 700 | 1.0126 | 425 | 412 | -13 |
| 6 | 9.3 | 9.40 | 1.0108 | 40 667 | 41 400 | 1.0180 | 355 | 354 | - 1 |
| 7 | 6.9 | 6.90 | 1.0000 | 48 398 | 49 200 | 1.0166 | 145 | 148 | + 3 |
| 8 | 8.1 | 9.60 | 1.1852 | 33 732 | 32 800 | 0.9724 | 35 | 44 | + 9 |
| 9 | 37.4 | 38.66 | 1.0337 | 39 011 | 39 629 | 1.0158 | 30 | 28 | - 2 |
| 10 | 23.4 | 23.51 | 1.0047 | 14 023 | 14 418 | 1.0282 | 9 | 9 | 0 |
| 11 | 18.5 | 18.55 | 1.0027 | 39 316 | 39 901 | 1.0149 | 9 | 9 | 0 |
| 12 | 7.1 | 7.47 | 1.0521 | 17 955 | 18 470 | 1.0287 | 52 | 54 | + 2 |
| Mean | 15.63 | 16.12 | 1.04 | 42 324 | 42 877 | 1.02 | 137.00 | 137.58 | 0.58 |
| Standard Deviation | 11.11 | 11.37 | 0.05 | 25 975 | 26 163 | 0.02 | 145.32 | 143.73 | 5.95 |
| Coefficient of Variation (%) | 71.08 | 70.53 | 4.87 | 61.37 | 61.02 | 1.56 | 106.07 | 104.47 | - |

*Subscript I refers to the ILLUDAS model
†Subscript S refers to the SWMM model (Runoff Block)

On the average, the runoff peaks simulated by ILLUDAS were about 3.7 percent smaller than those simulated by SWMM. Similarly, the runoff volumes produced by ILLUDAS were about 1.6 percent smaller. In a more detailed examination, these differences were found statistically insignificant at a 95 percent confidence level.

Times to runoff peak simulated by ILLUDAS and SWMM were practically identical.

Thus one may conclude that for a simple simulation of runoff from impervious areas and an open-channel flow routing in a converging sewer network, the modified ILLUDAS model produced results almost identical with those obtained with the Runoff Block of the SWMM model.

## SENSITIVITY ANALYSIS OF THE MODIFIED ILLUDAS MODEL

An experimental sensitivity analysis of the modified ILLUDAS model was conducted for the Malvern catchment. In this analysis, the selected input parameters were varied over a wide range of values and the resulting effects on the model output were studied. This type of information is useful for model users, because it indicates which input parameters strongly affect the modelling results and therefore should be specified quite accurately. Other parameters may be just roughly estimated.

In particular, the following factors affecting the ILLUDAS simulations were studied:

Design rainfall input -
- Return period
- Storm duration
- Time distribution of rainfall intensities
- Time step

Hydrologic and hydraulic parameters -
- Initial loss
- Antecedent moisture and infiltration
- Inlet time
- Pipe roughness

Simulation techniques -
- Discretization of the catchment
- Simulation mode

Numerical values of input parameters and a description of various simulation techniques used in the sensitivity analysis are given in Table 9. The best estimates of input parameters that were used in a reference simulation are also listed in Table 9. The results of the sensitivity analysis follow.

**Table 9. Sensitivity Analysis - Variations in Input Parameters**

| Parameter | Reference Simulation | Variations | | | |
|---|---|---|---|---|---|
| <u>Design Rainfall Input</u> | | | | | |
| Return Period (years) | 5 | 2 | 10 | | |
| Duration (h) | 1 | 0.5 | 3 | | |
| Intensity Distribution $t_p/T$* | 0.52 | 0.03 | 0.26 | 0.77 | |
| Time Step (min) | 2 | 1 | 5 | 10 | 30 |
| <u>Hydrologic and Hydraulic Parameters</u> | | | | | |
| Initial Abstraction (in.) - Impervious | | 0.02 | 0 | 0.1 | 0.25 |
| - Pervious | 0.184 | 0 | 0.2 | 0.50 | |
| Soil Infiltration Curve (According to the Soil Group) | SWMM † | A | B | C | D |
| Antecedent Moisture Conditions | 1 | 2 | 3 | 4 | |
| Inlet Time | $T_i$ | 0.1 $T_i$ | 0.3 $T_i$ | 3 $T_i$ | $T_{kwe}$ |
| Sewer Pipe Roughness (Manning n) | 0.013 | 0.010 | 0.015 | | |
| <u>Simulation Techniques</u> | | | | | |
| Discretization Level (No. of Elements) | 15 | 1 | 5 | 40 | |
| Simulation Mode | 1 (Design) | 2 (Analysis) | 3 (Press. Flow) | | |

* Overall Distribution after Mitci (3)

† Soil Infiltration Described by Data in Table 3

**Design Rainfall Input**

The selection of a design rainfall input (design storm) seems to be a subject of controversy. Much of the criticism of the design storm approach centres on the underlying assumption that the return periods of a storm event and of the resulting runoff event are identical. Additional criticism stems from somewhat arbitrary definitions of the parameters of design storms. The purpose of the discussion presented here is not to examine the fundamentals of design storms, but simply to demonstrate the effects of variations in design storm parameters on simulation results.

The design storm used in this study was that developed by Mitci (3) for Montreal.

**Return period** - As the residential drainage is typically designed for events with return periods ranging from 2 to 10 years, the same range was used in the sensitivity analysis. The 5-year return period was taken as the reference value.

Simulation results obtained for various return periods are shown in Table 10. Both runoff peak flows and volumes increased by 40 percent to 50 percent with a return period increasing from 2 to 10 years.

**Table 10. Sensitivity of Runoff Peaks and Volumes to Rainfall Return Period**

| Rainfall Return Period | Runoff Peak Flow | | Runoff Volume | |
|---|---|---|---|---|
| (years) | (cfs) | (%) | ($ft^3$) | (%) |
| 2 | 66.6 | 79 | 67 200 | 77 |
| 5 | 84.5 | 100 | 87 600 | 100 |
| 10 | 100.5 | 119 | 111 900 | 128 |

**Storm duration** - The selection of the design storm duration is fairly arbitrary. In this study, the storm duration was varied from 0.5 to about three hours. For the design storm employed here, the storm duration does not affect the peak rainfall intensity, but it does affect the total rainfall depth.

The simulation results obtained for a 5-year storm with durations varying from 0.5 to 3 hours are summarized in Table 11. It is of interest to note that while the simulated peaks were not affected by the storm duration, the simulated runoff volumes increased with an increasing storm duration. The volumetric runoff coefficient, however, remained constant.

Table 11. **Sensitivity of Runoff Hydrograph Parameters to Rainfall Duration**

| Rainfall Duration (h) | Runoff Peak Flow (cfs) | Runoff Peak Flow (%) | Runoff Volume ($ft^3$) | Runoff Volume (%) | Volumetric Runoff Coefficient |
|---|---|---|---|---|---|
| 0.50 | 83.9 | 99 | 74 600 | 85 | 0.35 |
| 1.03 | 84.5 | 100 | 87 600 | 100 | 0.35 |
| 3.03 | 85.5 | 101 | 98 200 | 112 | 0.35 |

**Time distribution of rainfall intensities** - The distribution of intensities during a design storm is typically described by two parameters - a distribution function often derived from the rainfall intensity-duration-frequency (IDF) curves and the relative timing of the peak intensity. The intensity distribution used here was that developed by Mitci (3) and could be described for the reference storm as follows:

$$i = \frac{86}{t+12} \qquad (5)$$

where i is the rainfall intensity (in./h) and t is the time (min) measured both before and after the intensity peak. Thus to derive an intensity distribution for a design storm of a particular return period and duration, the designer first selects the timing of the intensity peak and then calculates intensities for various times before and after the peak.

For the purpose of this study, four different timings of the intensity peak were considered. These timings are described by a ratio of $t_p/T$, where $t_p$ is the time to peak and T is the storm duration. The four distributions used can be described as follows:

Fully advanced distribution ($t_p/T = 0.03$)
Advanced distribution ($t_p/T = 0.26$)
Centred distribution ($t_p/T = 0.52$) - reference
Delayed distribution ($t_p/T = 0.77$)

Runoff peaks and volumes simulated for various intensity distributions are listed in Table 12. The lowest peaks and volumes were found for the fully advanced distribution when the peak intensity coincided with maximum losses due to high infiltration and the filling of surface storage. The peak flows increased with increasing values of $t_p/T$. The rate of increase in peak flows diminished for $t_p/T > 0.5$. The total difference between the peak flows for the fully advanced and delayed distributions was only 20 percent.

**Table 12. Sensitivity of Runoff Peaks and Volumes to Timing of Peak Intensity**

| Relative Time of Peak Intensity $t_p/T$ | Runoff Peak Flow | | Runoff Volume | | Volume of Runoff from Pervious Areas | |
|---|---|---|---|---|---|---|
| | (cfs) | (%) | ($ft^3$) | (%) | ($ft^3$) | (%) |
| 0.03 | 69.5 | 82 | 83 800 | 96 | 7 900 | 68 |
| 0.26 | 80.3 | 95 | 85 800 | 98 | 9 800 | 85 |
| 0.52 | 84.5 | 100 | 87 600 | 100 | 11 600 | 100 |
| 0.77 | 85.8 | 102 | 89 000 | 102 | 13 100 | 122 |

Simulated total runoff volumes proved to be barely sensitive to the distribution of intensities. The difference in runoff volumes simulated for the fully advanced and delayed distributions was only 6 percent. Markedly different results were noticed for volumes of runoff from the pervious parts of the catchment. The volume simulated for the fully advanced distribution amounted to only about one half of that corresponding to the delayed distribution.

**Time step** - The rainfall input is discretized into short time intervals, which in the case of the ILLUDAS model are identical with the computational time step used in simulation. The ILLUDAS manual (5) offers some guidance in the selection of the time step - it should be as short as the quality of the rainfall data will allow and ideally it should be 1/2 to 1/3 of the average inlet time for paved areas.

In the sensitivity analysis, the time step was varied from 1 to 30 minutes. The results of all simulations are shown in Table 13.

The simulated peak flows were fairly sensitive to the length of the time step. The two shortest time steps, 1 and 2 minutes, met the criteria for the time step selection and produced virtually identical results. Further increases in the time step reduced the simulated peak flows considerably and produced unrealistic results.

**Table 13. Sensitivity of Runoff Hydrograph Parameters to Simulation Time Step**

| Time Step (min) | Runoff Peak Flow (cfs) | Runoff Peak Flow (%) | Runoff Volume ($ft^3$) | Runoff Volume (%) | Time Lag* (min) |
|---|---|---|---|---|---|
| 1 | 85.5 | 101 | 87 400 | 100 | 4 |
| 2 | 84.5 | 100 | 87 600 | 100 | 4 |
| 5 | 78.1 | 92 | 88 300 | 101 | 5 |
| 10 | 60.3 | 71 | 89 600 | 102 | 0 |
| 30 | 35.6 | 42 | 95 700 | 109 | 0 |

*Time Lag = Time to Peak - Time to Peak Intensity

Simulated runoff volumes were barely affected by the length of the time step (see Table 13). The effects of the time step on the simulated times to peak were also fairly small.

### Hydrologic and Hydraulic Parameters

The following four parameters are considered in this section: the initial abstraction loss, soil infiltration for various antecedent moisture conditions, the inlet time, and the sewer pipe roughness. The first two parameters are pertinent to the calculation of losses in the catchment; the last two affect flow routing on the surface as well as in sewers.

**Initial abstraction loss** - The initial abstraction loss varies depending on the catchment surface. For impervious areas, the loss varied from 0 to 6 mm (0 to 0.25 in.). On pervious areas, the loss varied from 0 to 12 mm (0 to 0.5 in.). The results of simulations for various initial losses are given in Table 14 for both peak flows and volumes. It can be inferred from Table 14 that the simulated peak flows were barely affected, even by large variations in the initial loss. One should bear in mind, however, that these results were obtained for the centred rainfall distribution and that different results could be obtained, e.g., for the fully advanced distribution.

Table 14. **Sensitivity of Runoff Peaks and Volumes to Initial Abstraction Loss**

| Initial Abstraction | | | Runoff Peak Flow | | Runoff Volume | |
|---|---|---|---|---|---|---|
| Impervious Area (in.) | Pervious Area (in.) | Weighted Mean (in.) | (cfs) | (%) | ($ft^3$) | (%) |
| 0.00 | 0.00 | 0.00 | 89.0 | 105 | 94 200 | 108 |
| 0.02 | 0.184 | 0.07 | 84.5 | 100 | 87 600 | 100 |
| 0.10 | 0.20 | 0.13 | 84.2 | 100 | 82 000 | 94 |
| 0.25 | 0.50 | 0.32 | 78.0 | 92 | 63 800 | 73 |

Simulated runoff volumes were more sensitive to the initial abstraction (see Table 14). The range of variation in simulated runoff volumes amounted to about 27 percent. Such a variation occurs because the rainfall excess is reduced in direct proportion to the initial loss.

**Infiltration and antecedent moisture conditions** - Infiltration and antecedent moisture conditions were considered together. Altogether, 20 possible combinations of soil groups and antecedent moisture conditions were considered. Such a set of 20 simulations was repeated for three different cases - a 5-year storm (the reference storm), a 10-year storm and, finally, backyards directly connected to streets. The results of all simulations are given in Tables 15-17. The discussion of results starts with peak flows followed by runoff volumes.

The results of runoff peak simulations are summarized in Table 15 for the 5-year storm and the existing catchment drainage. It is interesting to note that even for a large variation in the soil type and antecedent moisture conditions, the runoff peak flows did not vary much. The smallest peak represented 92 percent of the reference value; the largest peak represented 117 percent of the reference value. For any particular soil, the range of peak flow variations due to the variations in the antecedent moisture conditions did not exceed 24 percent. Similarly, for any antecedent moisture condition, the range of peak flow variations for various soils did not exceed 19 percent. When all 20 peak flows in this set were grouped together, they could be characterized by a mean of 1.02 (of the reference peak) and the standard deviation of 0.08.

Table 15. Runoff Peak Flows and Volumes for Various Soils and Antecedent Moisture for Pervious Areas Drained (Front Yards) - 5-Year Storm

| Antecedent Moisture Index | 1 | | 2 | | 3 | | 4 | |
|---|---|---|---|---|---|---|---|---|
| Soil Infiltration Curve | Peak Flow (cfs) | Peak Flow (%) | Peak Flow (cfs) | Peak Flow (%) | Peak Flow (cfs) | Peak Flow (%) | Peak Flow (cfs) | Peak Flow (%) |
| SWMM | 84.5 | 100 | 89.8 | 106 | 93.5 | 111 | 93.5 | 111 |
| A | 77.6 | 92 | 77.6 | 92 | 81.3 | 96 | 86.7 | 103 |
| B | 77.6 | 92 | 77.7 | 92 | 83.8 | 99 | 88.5 | 105 |
| C | 77.9 | 92 | 82.3 | 97 | 88.4 | 105 | 96.3 | 114 |
| D | 83.2 | 98 | 87.7 | 104 | 97.1 | 115 | 98.5 | 117 |
| | R. Volume ($ft^3$) | R. Volume (%) | R. Volume ($ft^3$) | R. Volume (%) | R. Volume ($ft^3$) | R. Volume (%) | R. Volume ($ft^3$) | R. Volume (%) |
| SWMM | 87 600 | 100 | 97 100 | 100 | 98 900 | 113 | 98 900 | 113 |
| A | 76 000 | 87 | 76 000 | 87 | 80 700 | 92 | 90 400 | 103 |
| B | 76 000 | 87 | 76 200 | 87 | 84 900 | 97 | 94 000 | 107 |
| C | 76 500 | 87 | 83 100 | 95 | 94 300 | 108 | 104 500 | 119 |
| D | 84 100 | 96 | 92 900 | 106 | 107 100 | 122 | 109 300 | 125 |

The same analysis was repeated for a 10-year storm (Table 16) with similar results. The variation in peak flows simulated for various soil groups and antecedent moisture conditions increased very little. For all 20 peak flows, the mean was 1.01 of the reference peak flow and the standard deviation was 0.10.

The lack of sensitivity of simulated runoff peaks to infiltration and antecedent moisture was somewhat surprising. A closer examination of the catchment drainage pattern indicated that only the front yards contributed effectively to the total runoff. The runoff from backyards is much too delayed to contribute effectively to the catchment peak runoff. Consequently, the effective catchment area contributing to the peak flow is only 11.4 ha (= impervious area + front yards) and the imperviousness of this area is 71 percent. Changes in infiltration therefore affect runoff from only 29 percent of the effective area and have a limited effect on the total catchment runoff.

Table 16. Runoff Peak Flows and Volumes for Various Soils and Antecedent Moisture for Pervious Areas Drained (Front Yards) - 10-Year Storm

| Antecedent Moisture Index | 1 | | 2 | | 3 | | 4 | |
|---|---|---|---|---|---|---|---|---|
| Soil Infiltration Curve | Peak Flow | | Peak Flow | | Peak Flow | | Peak Flow | |
| | (cfs) | (%) | (cfs) | (%) | (cfs) | (%) | (cfs) | (%) |
| SWMM | 92.2 | 100 | 100.7 | 109 | 102.2 | 111 | 102.2 | 111 |
| A | 79.5 | 86 | 79.6 | 86 | 86.5 | 94 | 95.9 | 104 |
| B | 79.5 | 86 | 81.3 | 88 | 90.5 | 98 | 98.6 | 107 |
| C | 82.0 | 89 | 88.6 | 96 | 98.7 | 107 | 105.1 | 114 |
| D | 89.4 | 97 | 97.5 | 106 | 105.8 | 115 | 106.9 | 116 |
| | R. Volume | | R. Volume | | R. Volume | | R. Volume | |
| | ($ft^3$) | (%) | ($ft^3$) | (%) | ($ft^3$) | (%) | ($ft^3$) | (%) |
| SWMM | 111 900 | 100 | 122 000 | 109 | 123 900 | 111 | 124 000 | 111 |
| A | 93 000 | 83 | 93 100 | 83 | 102 000 | 91 | 113 700 | 102 |
| B | 93 000 | 83 | 95 000 | 85 | 107 800 | 96 | 118 400 | 106 |
| C | 96 100 | 86 | 105 500 | 94 | 119 300 | 107 | 130 500 | 117 |
| D | 107 000 | 96 | 117 900 | 105 | 132 900 | 119 | 135 200 | 121 |

One would expect that runoff peaks from catchments with larger contributing pervious areas would be more sensitive to changes in soil infiltration. To pursue this point further, a hypothetical catchment was investigated in the last series of simulations. This hypothetical catchment was identical with the Malvern catchment in all aspects except for the drainage of backyards connected directly to the streets. Thus the entire pervious area (15.45 ha = 66 percent of the total catchment area) was effectively contributing to the catchment runoff. The results of simulations for the hypothetical catchment are given in Table 17 and indicate high sensitivity of peak flows to both soil characteristics and the antecedent moisture conditions. The range of peak flow variations for a particular soil group and various antecedent moisture conditions increased to 60 percent. Similarly, the range of peak flow variations for particular antecedent moisture conditions and various soils increased to 55 percent.

**Table 17. Runoff Peak Flows and Volumes for Various Soils and Antecedent Moisture for Pervious Areas Drained (Front Yards) - 5-Year Storm**

| Antecedent Moisture Index | 1 | | 2 | | 3 | | 4 | |
|---|---|---|---|---|---|---|---|---|
| Soil Infiltration Curve | Peak Flow (cfs) | Peak Flow (%) | Peak Flow (cfs) | Peak Flow (%) | Peak Flow (cfs) | Peak Flow (%) | Peak Flow (cfs) | Peak Flow (%) |
| SWMM | 93.2 | 100 | 109.1 | 117 | 115.0 | 123 | 115.0 | 123 |
| A | 77.6 | 83 | 77.6 | 83 | 81.4 | 87 | 100.9 | 108 |
| B | 77.6 | 83 | 77.6 | 83 | 88.4 | 95 | 105.6 | 113 |
| C | 77.6 | 83 | 84.7 | 91 | 105.2 | 113 | 123.8 | 133 |
| D | 86.3 | 93 | 102.1 | 110 | 126.1 | 135 | 133.6 | 143 |
| | R. Volume ($ft^3$) | R. Volume (%) | R. Volume ($ft^3$) | R. Volume (%) | R. Volume ($ft^3$) | R. Volume (%) | R. Volume ($ft^3$) | R. Volume (%) |
| SWMM | 110 600 | 100 | 153 400 | 139 | 161 900 | 146 | 162 000 | 146 |
| A | 76 000 | 69 | 76 000 | 69 | 83 800 | 76 | 125 100 | 113 |
| B | 76 000 | 69 | 76 000 | 69 | 101 000 | 91 | 139 700 | 126 |
| C | 76 000 | 69 | 92 600 | 84 | 140 500 | 127 | 186 300 | 168 |
| D | 96 800 | 87 | 133 600 | 121 | 198 100 | 179 | 208 600 | 189 |

Runoff volumes were found to be only slightly more sensitive to soil infiltration than peak flows. For the 5-year storm and the existing catchment drainage, the range of variations in runoff volumes due to various soil groups and antecedent moisture conditions was 32 percent. The results obtained for the 10-year storm were practically identical. As discussed for peak flows, the portion of the catchment effectively contributing to the total runoff is highly impervious, and anywhere from 83 percent to 100 percent of the total runoff is contributed by the impervious areas. Consequently, the variations in runoff from the pervious area have a limited effect on the total runoff.

Finally, the hypothetical case with backyards draining directly onto the streets was studied (see Table 17). As expected, much larger variations in runoff volumes were found. In fact, the runoff volumes varied by a factor of 2.7.

**Inlet time** - The inlet time is a fairly important parameter which controls the speed of runoff in subcatchments. In the sensitivity analysis, two approaches to calculating inlet times were considered - the expressions built into the original model and the kinematic wave equation (Eq. 4).

In the original ILLUDAS model, the inlet time $T_i$ is calculated from the following expressions:

Impervious Surface
$$T_i = \frac{Ln}{(1.486 \times 0.2^{2/3} \times S^{1/2}) \times 60} + 2 \qquad (6)$$

Pervious Surface
$$T_i = 1.0214 \frac{L^{0.4}}{S^{0.333}} \qquad (7)$$

where $T_i$ is the inlet time (min), L is the length of overland flow (ft), S is the slope of the travelled path (ft/ft), and n is the Manning roughness coefficient.

The following five sets of values of inlet times were used in simulations:

$$0.1\ T_i; \quad 0.3\ T_i; \quad T_i; \quad 3\ T_i; \quad T_{i\ kwe}$$

where the first four times were calculated from Eqs. 6 and 7 (i.e. the original ILLUDAS approach) and the last time, $T_{i\ kwe}$, corresponds to the kinematic wave equation (Eq. 4). It is of interest to note that for impervious areas, the mean inlet time $T_{i\ kwe}$ was about twice as long as the mean time calculated from Eq. 6.

Results of runoff simulations for various inlet times are listed in Table 18. The runoff peaks varied considerably with varying inlet times. By increasing the inlet time $T_i$ from 0.1 $T_i$ to 3 $T_i$, the runoff peaks were reduced by a factor of two. The kinematic wave equation produced a runoff peak about 20 percent smaller than that corresponding to the original model computations.

Variations in inlet times did not affect runoff volumes at all (see Table 18).

**Pipe roughness** - The pipe roughness affects the flow routing in sewers. In the sensitivity analysis, the roughness was varied in three steps: n=0.010, 0.013, and 0.015. The results of simulations for various values of pipe roughness are shown in Table 19.

Table 18. **Sensitivity of Runoff Peaks and Volumes to Inlet Time Calculation Procedure**

| Inlet Time Calculation | Runoff Peak Flow | | Runoff Volume | |
|---|---|---|---|---|
| | (cfs) | (%) | ($ft^3$) | (%) |
| 0.1 $T_i$ | 115.9 | 137 | 87 600 | 100 |
| 0.3 $T_i$ | 94.2 | 111 | 87 612 | 100 |
| $T_i$* | 84.5 | 100 | 87 612 | 100 |
| $T_{i\,kwe}$† | 69.5 | 82 | 87 616 | 100 |
| 3 $T_i$ | 57.0 | 67 | 87 619 | 100 |

*$T_i$ = Inlet time as calculated by the original ILLUDAS model

†$T_{i\,kwe}$ = Inlet time calculated from the kinematic wave equation

Table 19. **Sensitivity of Runoff Peaks to Sewer Pipe Roughness**

| Sewer Pipe Roughness | 5-Year Storm | | | 10-Year Storm | | |
|---|---|---|---|---|---|---|
| | Peak Flow | | Number of Changes in Commercial Diameters* | Peak Flow | | Number of Changes in Commercial Diameters* |
| (Manning n) | (cfs) | (%) | | (cfs | (%) | |
| 0.010 | 87.1 | 103.1 | -10 | 91.8 | 99.0 | -12 |
| 0.013 | 84.5 | 100.0 | 0 | 92.8 | 100.0 | 0 |
| 0.015 | 84.1 | 99.5 | + 4 | 92.0 | 99.1 | + 2 |

*(+) sign means increases in diameters (by one increment)
(-) sign means reductions

Effects of the pipe roughness on simulated peak flows were rather small. In fact, by increasing the pipe roughness from 0.010 to 0.015, the total peak flow was reduced by only 3.6 percent. Although the total peak flow did not change much, more significant changes could be found for individual subcatchments and sewer pipes. Consequently, the model was run in the design mode, and changes in commercial pipe sizes resulting from changes in the pipe roughness were examined. By reducing the roughness from 0.013 to 0.010 and by using the commercial pipe sizes, 10 out of 15 reaches were designed with smaller diameters. An increase in n from 0.013 to 0.015 resulted in an increase of four pipe sizes. It would appear that although the changes in the pipe roughness do not greatly affect the catchment peak flow, they may have some economical significance because of a number of changes in the individual pipe diameters.

### Simulation Techniques

In this category, two simulation aspects were considered - the level of catchment discretization and the simulation mode. The former aspect depends to a large extent on the judgement of the model user, and the latter aspect then follows from requirements of a particular model application.

**Discretization level** - The discretization is defined here as the subdivision of the catchment into a number of subcatchments, each of which has a corresponding sewer pipe for drainage. In the sensitivity analysis, four different levels of discretization were used:

1, 5, 15 and 40 subcatchments (pipes).

Using these levels of discretization, runoff simulations were done for the 5-year storm, 10-year storm, and the 12 actual events used in the verification study. The results of these simulations are summarized in Tables 20 and 21.

**Table 20. Sensitivity of Runoff Peaks and Volumes to Detail of Catchment Discretization - 5-Year Storm**

| Number of Subcatchments | Peak Flow | | Runoff Volume | |
|---|---|---|---|---|
| (pipes) | (cfs) | (%) | ($ft^3$) | (%) |
| 1 | 72.8 | 84 | 87 624 | 100 |
| 5 | 77.5 | 89 | 87 622 | 100 |
| 15 | 84.5 | 97.5 | 87 612 | 100 |
| 40 | 86.7 | 100 | 87 527 | 100 |

Table 21. **Sensitivity of Runoff Peaks and Volumes to Detail of Catchment Discretization - Verification Events**

| Event No. | Peak Flows (cfs) | | | Runoff Volumes ($ft^3$) | | |
|---|---|---|---|---|---|---|
| | $Q_{40}$* | $Q_1$ | $Q_1/Q_{40}$ | $V_{40}$ | $V_1$ | $V_1/V_{40}$ |
| 1 | 33.5 | 28.5 | 0.85 | 48 300 | 48 500 | 1.00 |
| 2 | 21.5 | 18.4 | 0.86 | 23 800 | 23 900 | 1.00 |
| 3 | 7.9 | 7.9 | 1.00 | 20 800 | 20 300 | 0.98 |
| 4 | 5.4 | 5.7 | 1.06 | 80 000 | 80 000 | 1.00 |
| 5 | 8.5 | 8.9 | 1.05 | 102 400 | 102 000 | 1.00 |
| 6 | 9.3 | 9.7 | 1.04 | 40 700 | 40 700 | 1.00 |
| 7 | 6.9 | 7.0 | 1.01 | 48 400 | 48 400 | 1.00 |
| 8 | 8.1 | 9.6 | 1.18 | 33 700 | 33 700 | 1.00 |
| 9 | 37.4 | 30.1 | 0.81 | 39 000 | 39 100 | 1.00 |
| 10 | 23.4 | 18.8 | 0.80 | 14 000 | 14 100 | 1.01 |
| 11 | 18.5 | 17.0 | 0.92 | 339 300 | 39 400 | 1.00 |
| 12 | 7.1 | 7.4 | 1.04 | 18 000 | 18 000 | 1.00 |
| Mean | 15.6 | 14.1 | 0.97 | 42 400 | 42 400 | 1.00 |
| Standard Deviation | 11.1 | 8.4 | 0.12 | 25 900 | 26 000 | 0.008 |
| Coefficient of Variation | 71.1 | 59.6 | 12.4 | 61.1 | 61.3 | 0.8 |

*The subscripts refer to the number of subcatchments

As expected, the peak flows became smaller with a decreasing number of subcatchments. A reduction in the number of subcatchments from 40 to 1 resulted in the reduction of peak flows by 10 percent and 16 percent for the 10-year and 5-year storms, respectively. In the case of verification storms, the peak flows were reduced by 3 percent. For verification storms of low intensity which produced runoff peaks less than 0.015 $m^3/s/ha$, the differences between runoff peaks simulated for 1 and 40 subcatchments were negligible.

**Simulation Mode**

As discussed earlier. the modified ILLUDAS model can be run in three modes - the design, analysis and surcharge modes. In the design mode, the model selects a pipe diameter necessary to convey the incoming flow. In the analysis mode, the flows above the pipe capacity are stored outside the system and reenter when the flows fall below the pipe capacity. The newly added surcharge mode attempts to approximate the pressurized flow by calculating the elevations of the hydraulic grade line required to convey the flows exceeding the full-pipe capacity.

The model was run in all three modes for the 5-year and 10-year design storms. The results are given in Table 22.

**Table 22. Peak Flows and Times to Peak for Various Simulation Modes**

| Simulation Mode | 5-Year Storm | | | 10-Year Storm | | |
|---|---|---|---|---|---|---|
| | Peak Flow | | Time to Peak | Peak Flow | | Time to Peak |
| | (cfs) | (%) | (min) | (cfs) | (%) | (min) |
| 1 - Design | 84.5 | 100 | 36 | 92.2 | 100 | 40 |
| 2 - Analysis | 49.0 | 58 | 34 - 36 | 49.0 | 53 | 35 - 55 |
| 3 - Pressure Flow | 89.3 | 106 | 34 | 107.0 | 116 | 35 |

The results for the analysis mode are of little interest, because the peak flow is controlled by the capacity of the outfall pipe ($Q=49$ cfs$=1.392$ $m^3/s$). The other two modes yielded more interesting results. The approximate flow routing under surcharge speeded up runoff and produced peaks which were 10 percent to 16 percent higher than those obtained in the design mode (i.e. an open-channel flow).

## SUMMARY AND CONCLUSIONS

A standard version of the ILLUDAS model was modified for operation in an interactive mode on a desk-top computer HP 9830 (16K-byte memory) with peripheral devices. The modified model version not only retained all the features of the original model (1974 version) but was further expanded for a number of new features. These new features include interactive program operation, multiple rainfall hyetographs, input of hydrographs from upper reaches, optional calculation of inlet times from a kinematic wave equation, storage of flows on street surface, approximate analysis of pressurized flow in sewers, dry weather flow, and statistical analysis of simulated and observed hydrographs.

The modified ILLUDAS model was verified on the Malvern test catchment with good results. Most of the verification events represented medium storms with a fairly high frequency of occurrence. On the average, the simulated runoff peaks and volumes were about 10 percent and 5 percent smaller than the observed ones, respectively. The simulated times to peak flow corresponded fairly closely to the observed ones.

The verification results obtained with the modified ILLUDAS model were compared with those obtained earlier with the Runoff Block of the SWMM model. Although the SWMM model reproduced the Malvern data slightly better than the ILLUDAS model, this difference was statistically insignificant. It can be concluded that on a small urban catchment with runoff controlled by the impervious area and an open-channel flow routing in sewers, the modified ILLUDAS model performed as well as the Runoff Block of the SWMM model.

A sensitivity analysis of the modified ILLUDAS model was undertaken for the catchment studied. The analysis dealt with the effects of the design storm characteristics, hydrologic and hydraulic parameters, and simulation techniques on simulated hydrographs.

Both runoff peaks and volumes increased significantly with an increasing return period of the design storm. Runoff peaks were practically unaffected by the storm duration, by time steps shorter than the mean inlet time for paved areas and by intensity distributions with the peak occurring later than in the first quarter of the storm duration. Time steps larger than the mean inlet time for impervious areas and intensity distributions with peaks in the first quarter resulted in reduced peak flows. Runoff volumes increased significantly with an increasing storm duration and delay in the intensity peak, but were unaffected by the time step.

The hydrologic and hydraulic parameters included the initial abstraction loss, infiltration and antecedent moisture, inlet times and pipe roughness.

The initial loss hardly affected the peak flows, but had a more pronounced effect on runoff volumes. For the catchment studied, the effects of the soil infiltration and antecedent moisture conditions on runoff peaks and volumes were not pronounced. For a full range of antecedent moisture conditions and the soil groups studied, the dispersion of simulated peak flows and volumes about the mean could be characterized by variation coefficients of 8 percent and 12 percent, respectively. The mean values were within 2 percent of the values obtained for the reference conditions (i.e. the best estimates of parameters). It should be stressed that the catchment configuration is such that runoff from backyards is rather delayed and hardly contributes to the catchment runoff. The remaining contributing part of the catchment is highly impervious and therefore runoff from this part is barely affected by variations in soil infiltration. For a hypothetical case, runoff from backyards was directly conveyed to the streets. The mean peak flow for all soils and antecedent moisture conditions exceeded the reference peak by 5 percent and the coefficient of variation increased to 20 percent. The runoff volumes were affected even more. The mean volume represented 1.12 of the reference volume and the coefficient of variation was 39 percent.

Variations in inlet times affected runoff peaks, but not runoff volumes. The optional calculation of inlet times from the kinematic wave equation resulted in peak flows about 20 percent smaller than those calculated from the original procedure.

Variations in the sewer pipe roughness did not greatly affect the catchment runoff peak, but resulted in a number of pipe size changes in individual reaches. It would appear that the choice of the sewer pipe roughness may have some impact on the drainage costs.

Among the simulation techniques, the effects of catchment discretization and simulation mode on the simulated runoff hydrographs were studied. Runoff peaks slightly decreased with a decreasing number of subcatchments. Runoff volumes remained the same.

Among the simulation modes, the highest peak flows were obtained for the pressure flow mode, followed by the design mode. The analysis mode limited the peak flows to the outfall pipe capacity.

A description of the computer hardware used in this study and the program flow chart are given in Figures 5 and 6, respectively. A complete program listing is available on request from the Hydraulics Division of the National Water Research Institute.

In summary, runoff simulations for small urban catchments and open-channel flow routing can be accomplished on a small desk-top computer with results fully comparable with those obtained with more complex models requiring large computers.

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# APPENDIX

## STATISTICAL MEASURES (after Ref. 4)

Assuming a linear relationship between two variables, the observed variable, O, and the computed variable, C, the linear correlation coefficient R is defined as:

$$R = \frac{N\left(\sum_{i=1}^{N} O_i C_i\right) - \left(\sum_{i=1}^{N} O_i\right)\left(\sum_{i=1}^{N} C_i\right)}{\left\{\left[N\left(\sum_{i=1}^{N} O_i^2\right) - \left(\sum_{i=1}^{N} O_i\right)^2\right]\left[N\left(\sum_{i=1}^{N} C_i^2\right) - \left(\sum_{i=1}^{N} C_i\right)^2\right]\right\}^{1/2}}$$

where N is the number of observations of O and C. The closer the value of R is to either +1 or -1, the better the agreement between the two variables.

The special correlation coefficient, $R_s$, is defined as:

$$R_s = \frac{2\sum_{i=1}^{N} O_i C_i - \sum_{i=1}^{N} C_i^2}{\sum_{i=1}^{N} O_i^2}$$

The closer the value of $R_s$ is to +1, the better the agreement between the observed and computed variables.

Finally, the integral square error (ISE) is defined as:

$$ISE = \frac{\left[\sum_{i=1}^{N} (O_i - C_i)^2\right]^{1/2} .100}{\sum_{i=1}^{N} O_i}$$

The smaller the value of ISE, the better the agreement between the observed and calculated variables.

Numerical values of the statistical measures are qualitatively evaluated as follows:

| R | $R_s$ | ISE | |
|---|---|---|---|
| $0.99 \leq R < 1.0$ | $0.99 \leq R_s \leq 1.0$ | $0\% < ISE \leq 3.0\%$ | Excellent |
| $0.95 \leq R < 0.99$ | $0.95 \leq R_s \leq 0.99$ | $3.0\% < ISE \leq 6.0\%$ | Very good |
| $0.90 \leq R < 0.95$ | $0.90 \leq R_s < 0.95$ | $6.0\% < ISE \leq 10.0\%$ | Good |
| $0.85 \leq R < 0.90$ | $0.85 \leq R_s < 0.90$ | $10.0\% < ISE \leq 25.0\%$ | Fair |
| $0.00 \leq R < 0.85$ | $0.00 \leq R_s < 0.85$ | $25.0\% < ISE$ | Poor |