The Assessment of the Impacts of Urban Development on Catchment Response Using The Winnipeg Airport Extension.

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THE ASSESSMENT OF THE IMPACTS OF URBAN DEVELOPMENT ON CATCHMENT RESPONSE USING THE WINNIPEG AIRPORT EXTENSION

BY

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A thesis submitted to the Faculty of Graduate Studies of
the University of Manitoba in partial fulfillment of the requirements
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MASTER OF SCIENCE

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ABSTRACT

Some of the problems encountered in assessing impacts of urbanization in ungauged urban watersheds are explained.

The effects of urbanization on catchment response are examined under various degrees of development using design storms developed from 34 years of data provided by Atmospheric Environment Services. The effects of urbanization are presented in form of growth factors which depict the ratios of urban runoff quantities to those of rural or existing conditions. The derived flood frequencies are also compared with those that were derived using recorded data for rural and urbanized watersheds.

Two approaches were considered. One consisted of using HEC-1, a general flood hydrograph model and the other approach was by utilizing the statistically derived models for assessing peak flow changes.

The results show an average maximum growth of 38 percent for the 2-year flood, 26 percent growth for the 5-year flood, 21 percent for the 10-year flood, 18 percent for the 25-year flood, 16 percent for the 50-year flood, and 14 percent for the 100-year flood above the rural floods.

The results of sensitivity analysis show that for drier basins where the infiltration loss rates are high, urbanization has more pronounced effects than for for wet basins where infiltration loss rates are low.

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Chapter I

INTRODUCTION

1.1 General

In our daily lives throughout the world, people have gathered in centralized communities for various reasons. The growth of these centres known as urban areas has brought about changes in land use from its virgin condition to artificially covered surfaces usually of impervious nature.

Urbanization is the change in land use from natural or agricultural land to various other land uses. It has been well established that changes from natural to urban conditions result in increased runoff peaks as well as increases in runoff volumes. The extent to which these runoff quantities change basically depends upon the type of development, basin physiographic characteristics, soil texture, and soil moisture conditions.

There are many factors which influence runoff characteristics and therefore important in assessing the effects of development on the runoff. In urban drainage basins, the flow pattern is characterised by three major subsystems which include surface subsystem, transport subsystem, and the recieving water subsystem (Kibler 1982).

This section deals with the general theory of rainfall-runoff relationships, a general theory of urban runoff, and the objectives of the research.

The primary issues in the rainfall-runoff relationships are discussed as follows.

1.2 Rainfall-Runoff Relationships.

The processes which link rainfall with runoff are essentially deterministic, that is they are governed by physical laws which are reasonably well known (Overton et al. 1976). Before runoff can take place, a number of physical conditions have to be fulfilled such as satisfaction of the soil moisture so that any additional water runs off or is lost to deep percolation.

The abstractions to precipitation include interception by vegetation, depression storage, and infiltration losses.

1.2.1 Interception.

This is the portion of rainfall which is stored temporarily on leaves of vegetation and eventually gets back to the atmosphere through evaporation. The amount of rainfall stored during a rainfall event is a function of vegetation type and height.

1.2.2 Depression Storage.

In general, smooth surfaces are very rare. Usually a natural basin contains a number of depressions. The quantity of rain which gets trapped in these depressions is termed depression storage. The proportion of rainfall which ends up as depression storage is basically a function of topography, presence of marshes or lakes, land use and prestorm conditions. Leveled ground will have less depression storage than land with terraces for the same soil conditions. Most of the water in depression storage returns to the atmosphere through evaporation.

1.2.3 Infiltration.

Infiltration is the movement of water from the soil air interface into the soil itself. Most of the losses from rain are due to infiltration losses. The losses are basically a function of the soil texture and the antecedent moisture levels. Runoff from rainfall only takes place if the intensity of rainfall is higher than infiltration losses. Even if the soil is dry if the intensity is higher than the soil can take, the excess rainfall will end up as surface runoff after satisfying the interception and depression storage.

1.2.4 Discussion of the Effect of Abstractions of Runoff Peaks and Runoff Volumes

Runoff quantities are functions of the rainfall abstractions described earlier but also of the moisture previous to the storm, and rainfall characteristics.

The volume of runoff depends on antecedent moisture conditions. The higher the antecedent moisture levels, the larger will be the runoff volume and peak flow. The higher runoff values are due to the lower abstractions required to fill or saturate the soil. Peak flows and runoff volumes also depend on the storm characteristics. The storm characteristics which influence these quantities are storm distribution, rainfall intensity, and rainfall duration.

Even or uniform distribution of precipitation has been found to increase basin base flow and to result in maximum peak flows. Precipitation which occurs on the lower portion of the basin would result in higher peak flow than an equal amount of precipitation which occurs on the upper portion of the basin because of channel storage effect (Linsley et al. 1982).

When rainfall intensity is lower than infiltration rate, all rain is lost into soil mass. Runoff volumes and peak flows also depend on surface cover which affects infiltration.

Maximum peak flows also depend on the duration of the storm(Td). For a duration less than basin time of concentration(TC), the resulting peak flows are less than if the duration was equal to time of concentration. The reason for this is that at a Td value less than TC, only a portion of the basin is contributing to the flow at the outlet while at TC equal to or greater than Td, all areas are contributing to outlet flow resulting in overall higher peak flows.

1.3 General Theory of Urban Runoff.

In an urban drainage basin, a predominant characteristic is the man made impervious pathways such as parking lots and streets which guide flows overland. As suggested earlier, a typical urbanised basin consists of three basic runoff subsystems which are: (1) Surface Runoff, (2) Transport Subsystem, and (3) Recieving Water Subsystem. (Kibler, 1982). These factors are discussed as follows.

1.3.1 Surface Runoff Subsystem.

The surface subsystem consists of total area, impervious proportion, and other hydraulic properties. The overland flow process transforms rainfall excess on the surface subsystem to inlet hydrographs. The hydrograph in turn is routed through sewers or drains to receiving subsystems.

Given that rainfall hyetograph is the input to surface subsystem and that time distribution of flow(hydrograph) is the output, the surface subsystem can be represented as shown in Figure 1.1. The resulting peak discharge and total volume of runoff depend on precipitation characteristics as pointed out earlier.

The extent to which overland flow phase of the runoff process predominates depends on the nature of the basin. For hydrologically small basins, overland flow predominates while for hydrologically large basins, channel flow predominates.

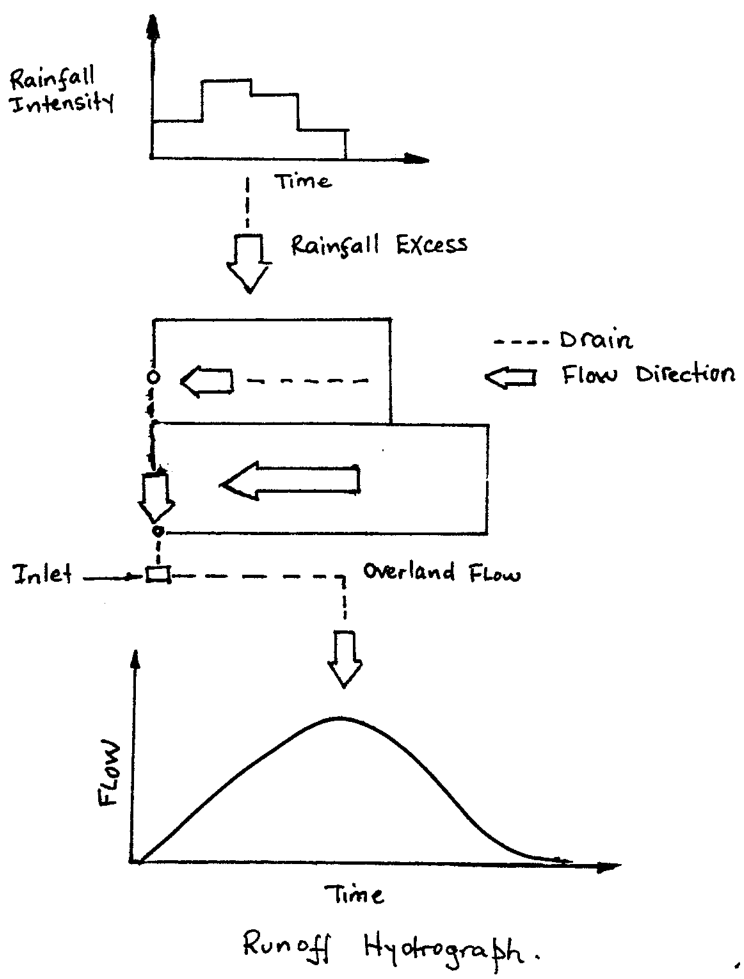

Figure I.I Surface Runoff Subsystem (after Kibler,1982)

1.3.2 Transport Subsystem.

The main function of the transport subsystem is to route the flows through a system of drains and sewers to inlets and eventually to recieving bodies. This subsystem consists of physical works. In the process of routing, the peak flows are generally attenuated by the storage effects in channels.

The transport subsystem is represented diagramatically in Figure 1.2.

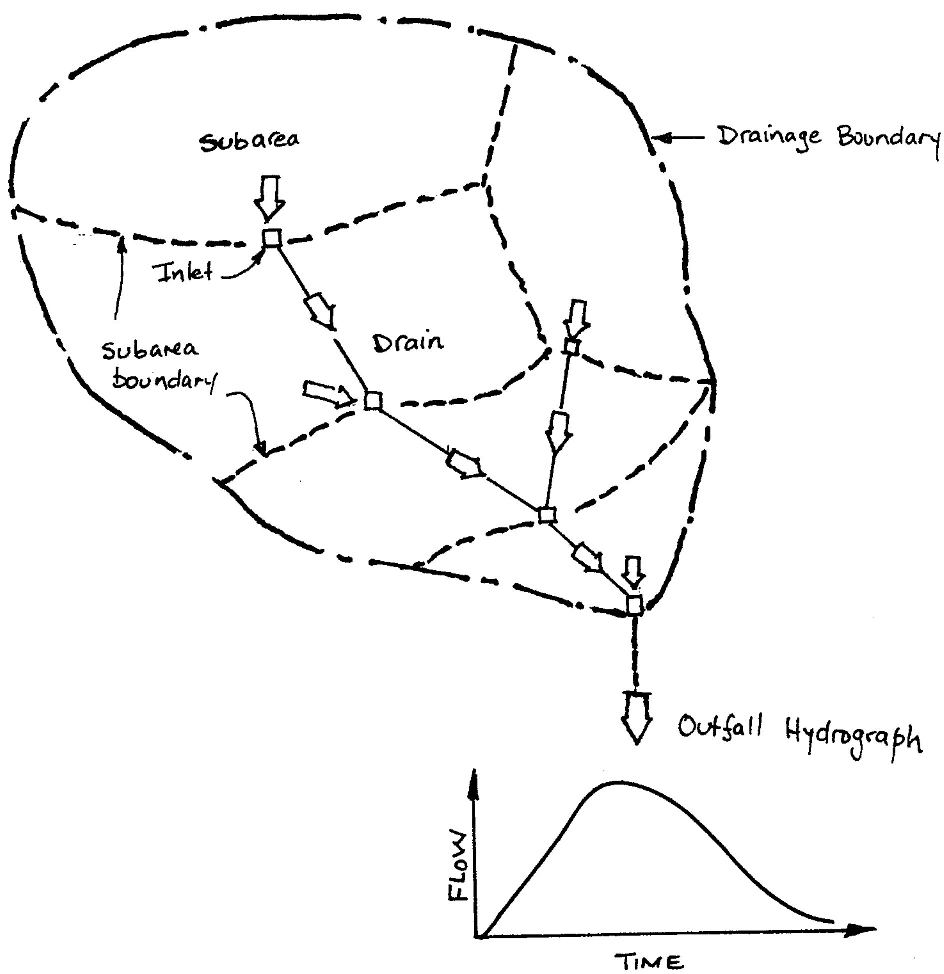

Figure I.2 Transport Subsystem (after Kibler, 1982).

1.3.3 The Receiving Subsystem.

Usually all urban drainage basins route their flows to either estuaries, lakes or rivers. Examination of the nature of the receiving body is beyond the scope of this study and is consequently not addressed any further in this thesis.

1.4 Description of the Watershed Characteristics.

This study was based on the Truro Creek watershed on the west side of the Winnipeg International Airport as shown in Figure 1.3. The basin considered is the watershed above the gauging station at Truro Creek near Assiniboine golf course as shown in Figure 1.3. This section gives a description of the basin physiographic features as well as existing and proposed land use patterns.

1.4.1 Basin Physiographic Features.

Truro Creek is one of the two creeks that drain the Winnipeg airport. The total drainage area is 6.29 square miles and the total stream length is 3.5 miles with a mean channel slope of 0.14 percent.

The watershed is divied into two subbasins as shown in Figure 1.4. Subbasin one has a total area of 4.44 square miles while subbasin two has an area of 1.85 square miles. Each of these subbasins is equiped with a seasonal gauging station as shown in Figure 1.4.

Truro Creek itself is an ephemeral creek which means that there is no flow during periods of no precipitation. Quantifiable flows are measured only after precipitation.

The overlying soil consists of black earth fine textured soils of depth ranging from 6 inches to 12 inches (Ehrlich et al. 1953). The underlying soil is predominantly heavy plastic clay while the vegetal cover mainly consists of gloves of willow and aspen. Table 1.1 shows a summary of the basin physiographic features.

This basin receives an average precipitation total of 21 inches of which snowfall constitutes an average of 5 inches and rainfall 16 inches. The normal highest precipitation occurs in June-July months well after snowmelt. Table 1.4 shows the average basin precipitation.

Table 1.1 Basin Physiographic Features

Physiographic Feature	Subbasin1	Subbasin2
Hydraulic length	13200 feet	13200 feet
Length to centroid	1.24 miless	1.40 miless
Slope of channel	0.14 percent	0.14 percent
Soil cover No.	83	89
Drainage Area	4.44 sq. mi.	1.43 sq. mi.

Where:
Length to centroid refers to that length from subarea inlet to centroid of the area.
Soil cover No. is as for Soil Conservation Service Service (1978)

Table 1.2 Watershed Monthly and Annual Normal Precipitation

Month	Rain (in)	Snow (in)
January	0.01	0.98
February	0.03	0.78
March	0.24	0.83
April	1.00	0.45
May	2.15	0.10
June	3.16	Trace
July	3.16	0.00
August	2.90	0.00
September	2.07	0.01
October	1.15	0.22
November	0.28	0.84
December	0.03	0.94
Average Total	16.00	5.00

1.4.2 Existing and Proposed Land Use Patterns.

Currently subbasin 1 is still in its natural condition except for road developments. Subbasin two, however, has been partially developed into a number of facilities as described in Tables 1.3 and 1.4.

The proposed development consists of extending the existing facilities and construction of new runways as shown in Figures 1.5 and 1.6.

Figure 1.3 Study Area Location Map

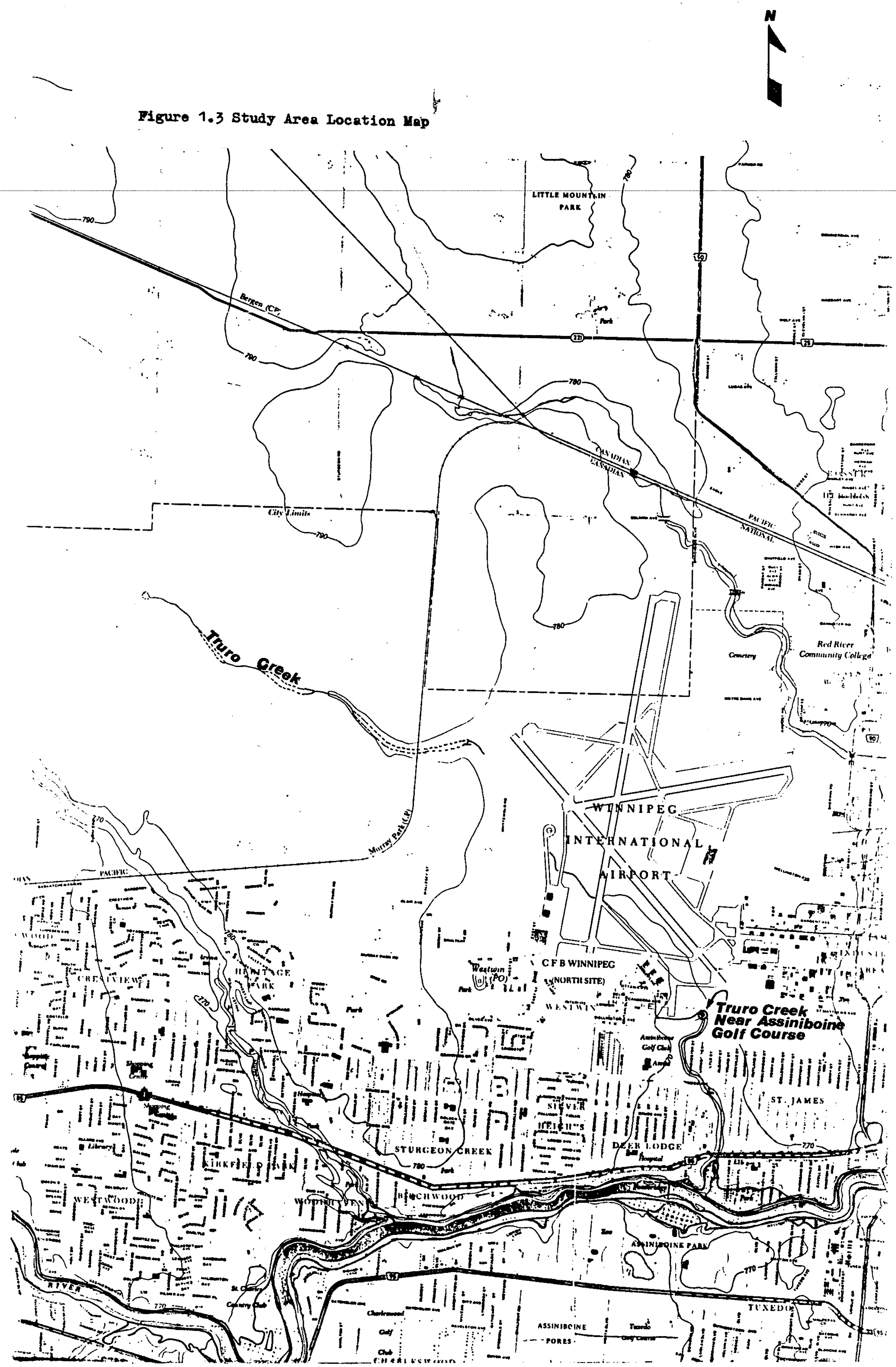

Figure 1.4 **EXISTING LAND USE**

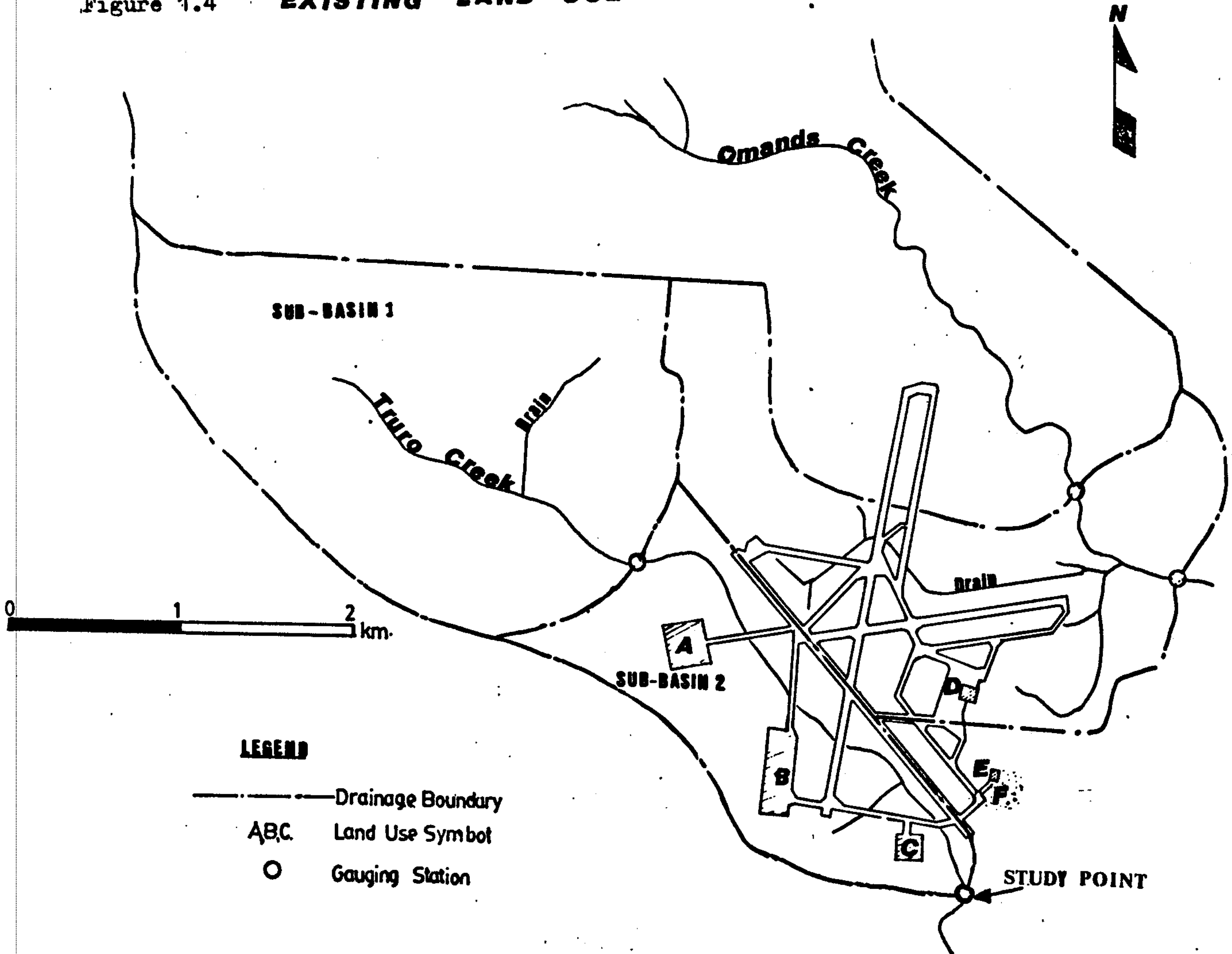

Table 1.3 Existing Land Use Pattern

Symbol	Land Use description
A	Airline Hangar
B	Department of National Defence
C	General Aviation
D	Airport Terminal Building
E	Aviation Support

Figure 1.5 ***PROPOSED AIRPORT LAND USE***

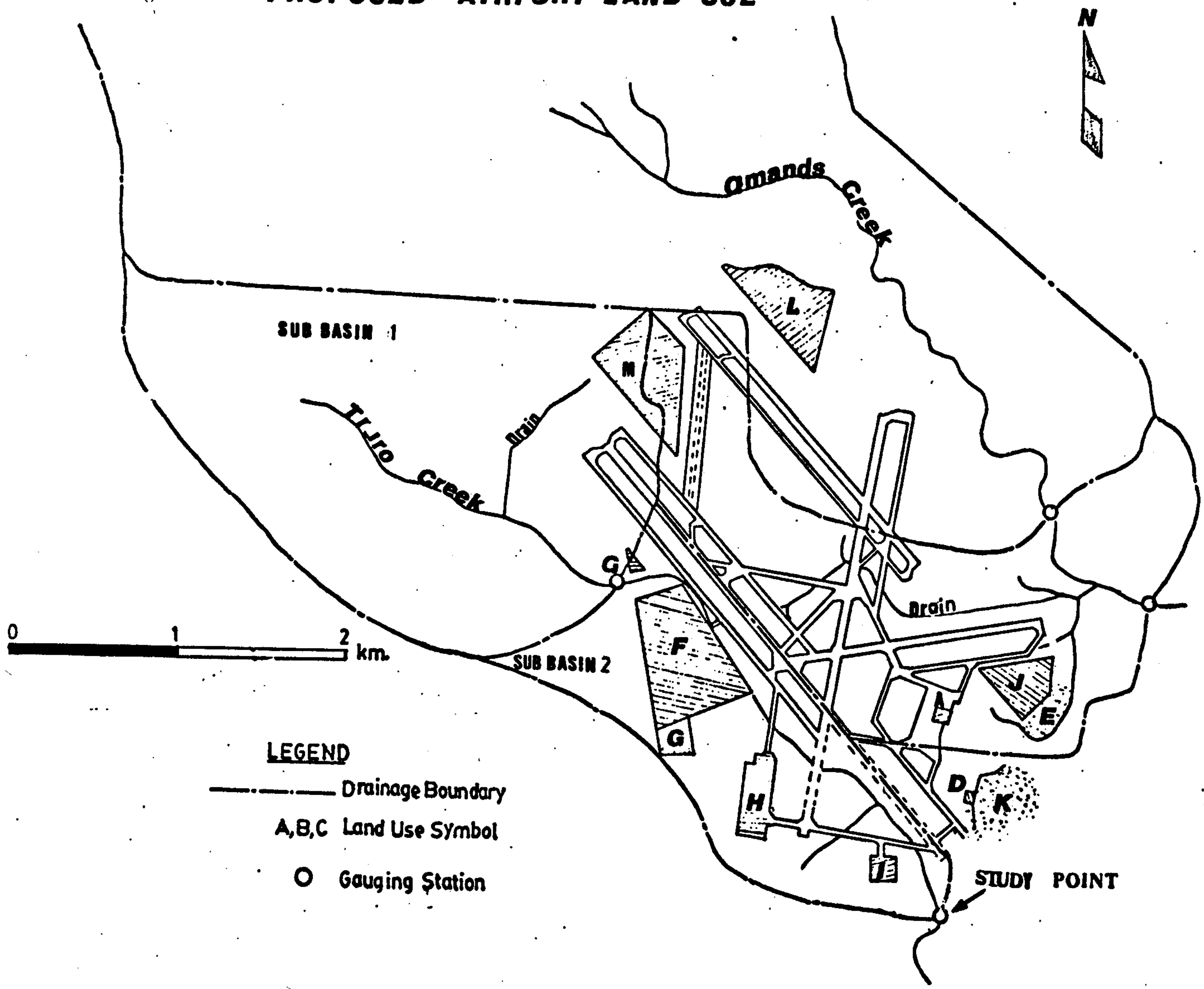

Table 1.4 Proposed Airport Land Use

Symbol	Land Use Description
A	Airport Terminal Building
B	Future Airport Terminal Building
C	Future Airline Hangar
D	Aviation Support
E	Aviation Support
F	Airline Hangar
G	Aviation Support
H	Department Of National Defence
I	General Aviation
J	Airline Support

1.5 Objectives of Research.

Most researchers have found that urbanization results in an increase in both peak flows as well as runoff volumes and a decrease in the time taken when effective rainfall begins and the peak flows occur thenceforth termed the time to peak. In predicting these changes, most of the researchers have used various rainfall-runoff model case studies and statistical approaches such as Beard (1979), and Keelway(1979). In most of these studies, emphasis has been on the effects of impervious proportion on cathment response. Bearing in mind that urban areas often lack enough data for detailed frequency analysis, various predictive models have been used without emphasis on the soil moisture conditions of the basin. The moisture levels in a basin keeps on changing with time and season making the whole basis of prediction difficult.

In some studies, experimental results based on initially dry soils have been used as values for basin infiltration losses. However, this infiltration loss is not only a function of soil type but also soil moisture conditions.

The objective of this research was to predict the impacts of extending Winnipeg airport on Truro Creek and to look at problems in predicting the impacts of urbanization in ungauged watersheds. The other objective was to predict changes in flows due to progressive development and to develop corresponding frequencies for degrees of development. The third objective was to explore the effects of soil moisture conditions on catchment response due to urbanization.

1.6 Methodology of Research.

In predicting the effects of changes in runoff due to changes in land use, the following procedure was adopted.

(i) Derivation of design storms for 2-year to 100-year storms.

(ii) Review of available models, model choice, calibration, verification and validation.

(iii) Assessment of the impacts of urbanization.

(iv) Conducting a sensitivity analysis.

Chapter II

REVIEW OF LITERATURE

Urbanization of a catchment area has generally been found to increase peak flows, volume of runoff, and to reduce the time to peak. There are two major factors which cause the above changes in runoff characteristics in urbanized basins. The first factor is covering of the parts of the catchment area with impervious surfaces such as streets and parking lots or buildings. This factor reduces the infiltration losses close to zero in the covered areas resulting in increased runoff volumes as well as increased peak flows.

The second factor is the increased conveyance efficiency of the basin caused by lining or straightening channels and installing sewers (Kibler,1982). This increase in conveyance efficiency results in an earlier occurrence of peak flows, hence a decrease in time to peak. It also results in reduced times of concentration thus allowing short duration high intensity storms to cause the whole basin to contribute simultaneously to outlet flows.

Both the effficiency of conveyance and imperviousness are therefore the causes of increased peaks and runoff volumes. Figure 2 shows the qualitative modification of runoff hydrograph from natural to urbanised basin for the same area and moisture conditions.

The extent of the increase in runoff quantities depends on the nature of development. In the case of development on hilly areas, generally slopes are reduced during construction which may increase time to peak (Bras, 1975). On the other hand, construction of buildings with parapet walls may also affect the runoff hydrograph by delaying the peak caused by storage effect of these roofs.

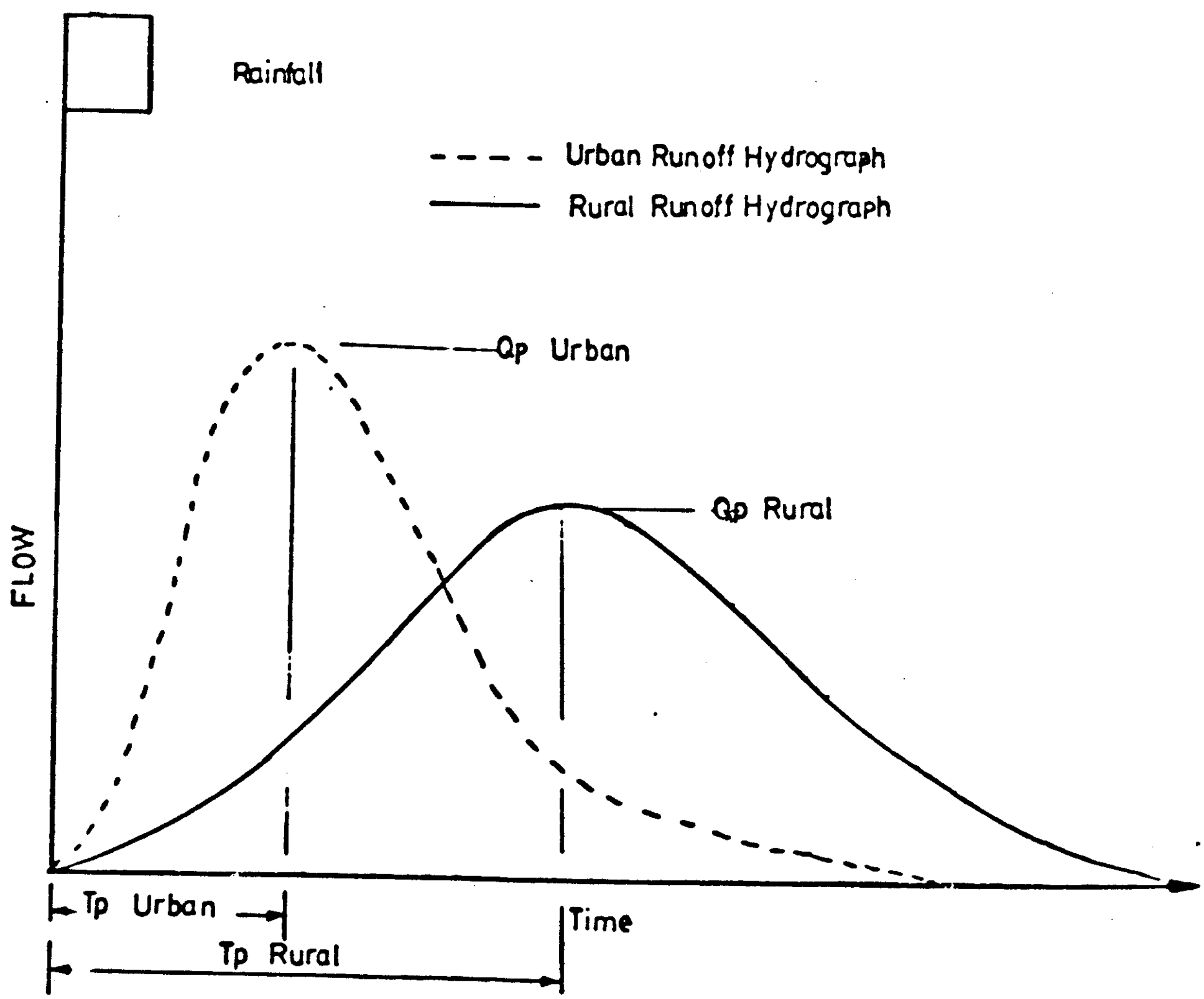

Figure 2 Comparison of Rural and Urban Hydrographs (after Kibler, 1982)

2.1 Historical Development of the Effects of Urbanization on Runoff Conditions

Work on the assessment of urbanization effects on catchment response started in the mid fifties. The effects have been measured in terms of peak flows(Qp), time to peak(Tp), and runoff volume(Vol). The results of most studies have shown that no universal formula that can be used to assess or predict these effects exists.

Sarma et al. (1969) have given detailed analysis of the previous studies. In the last decade, a number of additional studies have been done on this subject.

Dempster(1974) in his study found that with 40 percent of the watershed being impervious, the 2-year flood increased by 35 percent while the 50-year flood increased by 16 percent for the same conditions.

This and other evidence have shown that urbanization has very little effects on rare floods. Durbin(1974) has reported increases in peak flows of three to six times resulting from transition from rural to urban environment for the more frequent floods in his study in Santa valley, California. The 100-year flood however, showed no significant increases in peak flows.

Installing sewers in a watershed has also been found to cause higher peak flows because of increased conveyance efficiency. Bras(1975) has reported increases of 30 to 100 percent for a basin changing from natural to 100 percent sewer service with 50 percent of the basin under impervious cover for the 10-year flood. For the same conditions, the 50-year flood showed increases of 10 to 20 percent. Urbanization has also been found to decrease basin time to peak (Bras, 1975).

Urbanization has been found to have larger effects on smaller areas than larger ones because in small areas overland flow predominates while for larger ones, channel flow predominates and the effect of channel storage affects the peak flows. This is evident by studies done by McCuen et al.(1975).

Besides the size of the areas, the kind of development and the type of soils in the basin can have differing results for the same extent of development as reported by Bras (1975) and McCuen et al.(1975).

2.2 Engineering Aspects of Changes in Runoff Conditions.

Evaluation of the expected peak flows as well as volumes of runoff has been the goal of many engineers involved with the design of water related structures in municipalities. In municipal waterworks, the objective is to design the sewers and detention resevoirs for the predicted loading.

In order to evaluate these runoff quantities, several mathematical models have been used. In this case, a mathematical model is defined as a mathematical description of either physical, chemical, biological or any combination of these phenomena.

Mathematical models for the rainfall-runoff process can be grouped into three categories

(i) Conceptual (lumped parameter) models

(ii) Physically-based models and

(iii) Continuous simulation models

This categorization reflects the different data requirements and philosophical approaches in modeling the rainfall-runoff process.

Conceptual models are single transform functions that convert rainfall events into watershed response. The watershed is viewed as a black box. These models as typified by the Rational Method were used before the advent of the high speed computers.

Physically based models, however, do not ignore the interdependent mechanisms of stormwater flow but they rather attemt to approximate the physical processes occuring in a watershed. These models have been developed with the advent of the high speed computer.

Continuous simulation models are similar to the physically based models but these latter models keep account of all the water in the watershed continuously.

A number of these models have been documented in literature. A few models are briefly reviewed. These models include "the Stanford Watershed Model" (Crawford and Linsley, 1966),"The Stormwater Management Model" (Metcalf and Eddy, 1971),"Hydrologic Engineering Centre-1" (U.S. Army, Corps of Engineers, 1973), and "Illinois Urban Drainage Area Simulator (Terstreip and Stall, 1974).

The Stanford Watershed Model(SWM)

This is the most general model available and it is also the most complicated. It has 21 parameters which require carefull calibration prior to application of the model in design.

The SWM model is used to simulate the hydrologic cycle using rainfall and evapotranspiration time series data and parameters which describe the hydrologic response of the drainage basin such as slope, area, etc. The inputs to this model includes soil group classification, soil moisture and precipitation.

The disadvantage of this model is the difficulty associated with the calibration and the vast data required to get reasonable results.

The Stormwater Management Model (SWMM)

In so far as complex systems are concerned, this is the most complete model.

The SWMM model is basically a design oriented model for sewers. It is also used to simulate peak flows and quality of runoff. Data requirements are just as intense as for SWM. Like the SWM, this model is also very difficult to calibrate.

This approach has however been accepted in the municipal engineers practice and is commonly used in the design of storm sewer systems. It should be recognised, however that it has very limited usefulness in analysis or prediction of runoff hydrographs in partially developed or developed areas without extensive sewers. It has therefore little value in this study.

This model is also very difficult to calibrate.

Hydrologic Engineering Centre-1(HEC-1)

This model is basically used to simulate ordinary flood hydrographs associated with precipitation. It is a single event model in the sense that a single hypothetical or recorded storm is used with other basin physiographic features.

This model has many subroutines which make it a very flexible model. It can be used as a planning model or to simulate flow quantities. Besides having many subroutines, the data requirements are not vast and the parameters of the model have well defined relationships to physical conditions of a watershed.

Illinois Urban Drainage Area Simulator(ILLUDAS)

This model is used to simulate stormwater runoff from both impervious and pervious areas separately. It is also a single event model which was adapted from the British Road Research Laboratory.

Inputs to this model include soil cover complex, basin physiographic features, initial abstractions, and the recieving subsystem parameters such as water levels, etc.

This model is also extensively used to design sewers.

2.3 Model Selection And Comments

Having reviewed the capabilities of various models a model which could do the job in minimum time with reasonable accuracy using parameters which are easily related to phsical conditions in the basin was chosen. Based on these criteria, HEC-1 was selected for assessing the impacts of development. Further support for this selection was provided by Beard and Chang(1979) who also recommended HEC-1 based on the following criteria

(i) The algorithms used in the program are accepted in the engineering profession.

(ii) This model has been used quite extensively for similar jobs and has proved satisfactory and reliable.

(iii) The model has automatic calibration capability and

(iv) The fact that the model has few parameters, makes it easy to calibrate and to relate these parameters to physical conditions of the watershed.

It should be noted that all candidate models only approximate the physical conditions of the basin. For this reason, no model really reproduces what the actual response is. This is supported by the study done by Dracup (1973) who used all available models and concluded that none of those models had a 100 percent fit in both peak flows and hydrograph fit.

The computational procedure in HEC-1 starts with a known rainfall input and rainfall abstractions for each subarea. The subarea hydrograph is computed from a derived unit hydrograph for the excess precipitation. The subarea hydrographs are then routed to a point of interest or design point and combined to produce the total outfall hydrograph for the whole watershed. Routing of hydrographs is done by a number of different methods including the Muskingum method, modified puls and several other methods.

Chapter III

DESIGN STORMS

A design storm is a rainfall event either historical or artificial which is used as a basis for determining the design input for a proposed drainage or water-related system. In the assessment of effects of urbanization, design storms are used as inputs to the predictive models (Beard and Chang 1979).

For urban areas, the average frequency of rainfall occurence used for design determines the degree of protection afforded by a given system. ASCE (1979), recommends the following ranges of frequencies:

(i) for storm sewers in residential areas, 2 to 5 years.

(ii) for storm sewers in commercial districts and high value disticts 10 to 50 years depending on economic justification.

(iv) for flood protection works, 50 years or more.

For these reasons the design storms used in this study were developed for return periods ranging from 2 to 100 years.

Among many methods available for deriving design storms, the Keifer and Chu method (1957) was used. This method requires the conversion of depth duration rainfall data to intensity-duration data and fitting an equation of the following form:

$$i=a/(Td^{\wedge}b+c) \text{-----------------------------} (3.1)$$

Where i is the rainfall intensity mm(in.) per hour.

td is storm duration in minutes and

a, b, and c are constants for each return period.

The intensity-duration data for Truro Creek was available from Atmospheric Environment-Canada for Winnipeg airport for period between 1944 and 1981. The various constants for Equation 3.1 were derived by using SAS, (1982) which is a general statistical package for various kinds of statistical analysis such as regression and curve fitting. The constants for this analysis are given in Table 3.1

Table 3.1 Design Storm Model Constants

Return Period (years)	a	b	c
2	625.63	0.70	2.95
5	1537.05	0.87	6.81
10	2282.44	0.93	9.31
25	2696.71	0.92	9.27
50	3111.61	0.93	9.67
100	3883.49	0.95	10.55

As seen from Table 3.1, these constants appear to increase with increasing return period.

The design storms were then discretized to 5 minute interval and the total duration was 180 minutes for each storm. 180 minutes duration was chosen such that a comparison with already derived design storms for 5 year and 25-year using a different approach could be done. The Keifer and Chu model(1957) was used to discretize storms by using Equations 3.2 and 3.3 given as follows:

$$iB=a((1-b)(Tb/r)^b+c/((Tb/r)^b+c)^2 \qquad (3.2)$$

$$iA=a((1-b)(Ta/(1-r))^b+c)/((Ta/(1-r))^b+c)^2 \qquad (3.3)$$

Where iA and iB are intensities after and before peak intensity

Tb and Ta are times in minutes before and after peak intensity.

r is the ratio of time to peak to Td, usually taken as a value just above one third Td.

Table 3.2 shows the derived design storms using Equations 3.2 and 3.3 with the constants in Table 3.1. The intensities for these storms are presented in imperial units as inches per hour in Table 3.2.

The derived design storm intensities for the 5-year and the 25-year storms were then compared with those derived by Zurek(1980) for the City of Winnipeg. The derived intensities for the 5-year design storm are very similar to those derived by Zurek(1980) with differences of about 2 percent in total storm depth and about 1 percent difference in peak intensity. While for the 25-year design storm, the difference in total storm depth is also 1 percent and the difference in peak intensities is 3 percent. However, for the 25-year storm the peak intensities do not appear at the same time perhaps due to the differing assumptions in the ratio of time to peak and td. The comparisons are shown in Table 3.3.

In real life, however, actual storms may never exactly match the synthetic storms because of such things as large antecedent rainfall mass before the peak intensity or the presence of multiple peak intensities of rainfall during a storm event. The derived storm intensities could be regarded as purely hypothetical storms which are suitable for application in the vicinity of Winnipeg Airport.

Table 3.2 Derived Design Storms(inches per hour)

Time (minutes)	Return Period (years) 2	5	10	25	50	100
0	0.00	0.00	0.00	0.00	0.00	0.00
5	0.21	0.12	0.09	0.12	0.13	0.12
10	0.23	0.13	0.10	0.14	0.14	0.13
15	0.25	0.15	0.12	0.16	0.16	0.15
20	0.27	0.16	0.13	0.18	0.19	0.17
25	0.29	0.19	0.16	0.21	0.22	0.22
30	0.32	0.21	0.19	0.25	0.26	0.25
35	0.36	0.25	0.23	0.30	0.32	0.31
40	0.41	0.31	0.29	0.38	0.38	0.40
45	0.49	0.40	0.39	0.50	0.54	0.55
50	0.61	0.55	0.56	0.72	0.78	0.81
55	0.82	0.85	0.93	1.17	1.28	1.37
60	1.34	1.65	1.98	2.34	2.70	2.96
65	4.08	5.57	6.52	7.77	8.67	9.56
70	2.06	2.88	3.48	4.21	4.70	5.22
75	1.31	1.64	1.92	2.36	2.65	2.88
80	0.99	1.11	1.25	1.56	1.72	1.86
85	0.80	0.83	0.80	1.13	1.24	1.32
90	0.60	0.65	0.68	0.87	0.94	0.99
95	0.59	0.53	0.54	0.69	0.75	0.63
100	0.53	0.45	0.45	0.57	0.62	0.53
105	0.43	0.39	0.37	0.48	0.52	0.45
110	0.42	0.34	0.32	0.42	0.44	0.39
115	0.40	0.30	0.28	0.36	0.39	0.34
120	0.38	0.27	0.25	0.32	0.34	0.30
125	0.35	0.25	0.22	0.29	0.30	0.27
130	0.33	0.23	0.20	0.26	0.27	0.24
135	0.31	0.21	0.18	0.24	0.25	0.22
140	0.30	0.30	0.16	0.22	0.23	0.20
145	0.28	0.18	0.15	0.20	0.21	0.18
150	0.27	0.17	0.14	0.19	0.19	0.17
155	0.26	0.16	0.13	0.17	0.18	0.16
160	0.25	0.15	0.12	0.16	0.17	0.14
165	0.24	0.14	0.11	0.15	0.16	0.14
170	0.23	0.13	0.10	0.13	0.14	0.12
175	0.22	0.12	0.10	0.13	0.14	0.14
180	0.21	0.12	0.10	0.13	0.14	0.14

Table 3.3 Comparison of Derived Intensities and those Derived by Zurek (1980)

Time (minutes)	5-Year Storm (in/hr) a	5-Year Storm (in/hr) b	25-Year Storm (in/hr) a	25-Year Storm (in/hr) b
0	0.0	0.0	0.0	0.0
5	0.12	0.12	0.12	0.12
10	0.13	0.13	0.13	0.14
15	0.15	0.13	0.15	0.14
20	0.17	0.15	0.17	0.16
25	0.19	0.19	0.21	0.21
30	0.25	0.21	0.23	0.25
35	0.30	0.25	0.23	0.30
40	0.35	0.31	0.26	0.38
45	0.50	0.40	0.29	0.50
50	0.92	0.55	0.33	0.72
55	2.09	0.93	0.42	1.17
60	5.65	1.65	0.53	2.34
65	2.90	5.57	0.76	7.77
70	1.58	2.88	1.24	4.21
75	1.08	1.64	2.96	2.36
80	0.80	1.11	7.86	1.56
85	0.60	0.83	3.93	1.13
90	0.50	0.65	2.29	0.87
95	0.45	0.53	1.54	0.69
100	0.40	0.45	1.17	0.57
105	0.35	0.39	0.92	0.57
110	0.31	0.34	0.75	0.42
115	0.29	0.30	0.65	0.36
120	0.27	0.27	0.58	0.32
125	0.25	0.25	0.51	0.29
130	0.23	0.23	0.45	0.26
135	0.22	0.18	0.40	0.25
140	0.21	0.30	0.35	0.22
145	0.20	0.18	0.30	0.20
150	0.19	0.17	0.26	0.19
155	0.18	0.16	0.22	0.17
160	0.17	0.15	0.19	0.16
165	0.16	0.14	0.17	0.15
170	0.15	0.13	0.16	0.13
175	0.14	0.12	0.15	0.13
180	0.0	0.12	0.14	0.13

a = design storm intensities by Zurek (1980)
b = design storm intensities by the author

Chapter IV

MODEL PARAMETER CALIBRATION VERIFICATION AND VALIDATION

Modelling runoff for a drainage basin using HEC-1 requires a complete definition of the unit hydrograph and loss rate criteria. The unit hydrograph parameters required are time of concentration and hydrograph attenuation coefficient. For routing subbasin flows to outlet, routing coefficients are also required.

HEC-1 has a capability to determine all these parameters automatically. The first of these parameters, however, was derived by using known formulae. The hydrograph attenuation coefficient and routing coefficients were derived by hydrograph analysis. The loss rate parameters were derived by using the HEC-1 subruotine OPTIM which is used for optimizing parameters.

4.1 Hydrograph Analysis.

The aim of this analysis was to obtain a relationship between peak flows resulting from isolated storms and the recession flows, using graphical techniques shown in Appendix B.

Generally for an isolated storm, the resulting hydrograph can be synthesised into three major components; the rising limb, the crest segment, and the falling limb. The rising limb is influenced by the precipitation excess resulting from the storm and the recession flow depends on the basin storage. The point of inflexion on a semi log plot of discharge versus time indicates the time inflow to the channel ceases and the flows thereafter are as a result of withdrawal from basin storage (Linsley et al. 1982).

In HEC-1 the flow at the beginning of recession is designated QRCSN. For this analysis, 4 hydrographs resulting from isolated storms were used. These hydrographs were ploted on semi log graphs and the discharges at the beginning of recession were noted by the straight line departure from the curve. These recession flow values for the individual isolated storms were then divided by their corresponding peak discharges with the aim of finding on an average the ratio of QRCSN to peak flows(Qpeak). Table 4.1 shows the results of this analysis.

Table 4.1 Hydrograph Analysis Results

Storm Date	Total depth	Qpeak	QRCSN	QRCSN/Qpeak
	(Inches)	(cfs)	(cfs)	
20-05-84	0.48	11.0	3.0	0.27
12-06-84	0.35	19.0	7.4	0.39
16/17-06-84	2.42	113.0	28.0	0.26
21-06-84	2.63	207.0	64.0	0.31
			Sum	1.23
			Mean	0.31

The results of this analysis showed that on the average, the recession flows begin at 31 percent of peak flow(QRCSN=0.31QPeak).

4.2 Time of Concentration.

Another important parameter in runoff analysis is the time of concetration(TC). TC is the time required by a particle of water to flow from the most remote point in the watershed in terms of flow to the outlet or point of interest in a basin.

Many formulae exist for estimating this parameter. Among these are the Soil Conservation Service formula (McCuen et al. 1984), the Kirpich formula (Kibler,1982), Linsley et al. formula (Linsley et al. 1982), and McCuen et al. formula (McCuen et al.,1984). These formulae are presented in Table 4.2

Table 4.2 Time of Concentration Formulae

Method	Formula
SCS	$TC=0.000877Lf^{0.80}\ (1000/CN-9)^{0.70}\ S^{-0.50}$ (hours)
Kirpich	$TC=0.0078L^{0.77}\ S^{-0.35}$ (hours)
Linsley	$TC=0.35((LLc/S^{0.50})^{0.38}$ (hours)
McCuen	$TC=0.01462Lf^{0.5552}\ i^{-0.7164}\ Sfm^{-0.2070}$ (hours)

Where

Lf is length in feet.
CN is soil cover complex number(Kibler,1982)
S is slope in feet per foot.
i is 2-year storm depth over a period of one hour duration
Sfm is slope in feet per mile.

The values of time of concentration were estimated by using equations of Table 4.2 and some of the physiographic features of Table 1.1. The results of this analysis are given in Table 4.3.

Table 4.3 Time of Concentration Results

Method	subasin1 TC(Hours)	subbasin2 TC(Hours)	total basin TC(Hours)
SCS	5.96	4.70	10.66
Kirpich	2.83	2.83	5.660
Linsley	3.13	3.27	6.40
McCuen	3.16	3.16	6.32

An examination of the above results shows quite a range of values with the SCS being significantly different from the others which are quite close. This difference is attributed to the differing assumptions in derivation of these various formulae. Most of these equations are empirical. However, McCuen et al. formula applies to urban areas and was derived statistically. It was therefore chosen for assessing flows using HEC-1.

4.3 Loss Rate Parameters.

The loss rates affect both peak runoff and volumes of runoff as discussed earlier. Loss rates can either be computed by using initial and uniform loss rates such as using the Horton equation or by a function which replaces the loss rates to rainfall intensity and accumulated loss.

In HEC-1, the loss rate function is given by

ALOS=(AK+DLTK)(RAIN^ERAIN)--------------------(4.2)

Where

AK=STRKR/(RTIOL)^0.1CUMUL -----------------(4.2)

DLTK=0.2DLTKR[1-CUMULLTKR]^2 -----------------(4.3)

for CUMUL/DLTKR less than 1, otherwise zero.

ALOSS=loss rate in inches per hour.

AK=basic loss coefficient.

DLTK=Incremental loss coefficient.

RAIN=rainfall in inches per hour.

ERAIN=exponent of rainfall.

STRKR=initial loss rate inches per hour.

RTIOL=ratio of loss coefficient(AK) to AK after loss of 10 inches more of accumulated loss.

CUMUL=accumlated loss, inches.

DLTKR=initial accumulated loss,inches.

In Equations 4.1 and 4.2, only STRKR, DLTKR, ERAIN, and RATIOL need to be defined for computation. HEC-1 is capable of automatically deriving values for these parameters given recorded storm and recorded hydrograph as input. In this study, the process of calibration was done in two steps. The first step was for full model calibration and the second step involved keeping the parameters which did not change significantly constant and recalibrating the model. These steps were done following recommendations of Beard and Chang (1979).

The results of both optimization procedures are shown in Tables 4.3, and 4.4, respectively.

Table 4.4 Initial Full Model Calibration Results

Storm Date	Duration (hours)	STRKR (in/hr)	ERAIN	DLTKR (inches)	RATIOL
20-05-84	9.0	0.25	0.50	0.61	2.20
12-06-84	7.0	0.29	0.53	0.69	2.04
16-06-84	7.0	0.84	0.53	1.95	2.21
21-06-84	3.0	0.80	0.52	2.01	1.98
26-06-84	7.0	0.20	0.50	0.50	2.30
22-06-84	4.0	0.59	0.55	1.60	2.89
	Sum	2.97	3.10	7.36	13.62
	Mean	0.50	0.52	1.23	2.27

Note that the values of ERAIN and RATIOL are fairly constant and thus were kept constant at 0.52 and 2.27, respectively for the next calibration.

Table 4.5 Final Optimization Results

Storm Date	STRKR (in/hr)	DLTKR (inches)
20-06-84	0.26	0.50
16-06-84	0.83	0.52
12-06-84	0.30	0.51
06-06-84	0.24	0.50
26-06-84	0.42	0.58
21-06-84	0.80	2.21
Sum	2.85	4.73
Mean	0.48	0.79

The results of the final optimization gave values for STRKR=0.49 in/hr, ERAIN=0.52, DLTKR=0.79 inches, and RATIOL=2.27, respectively. As far as the parameter DLTKR is concerned, the initial optimization gives 1.23 and 0.79 represents a reasonable compromise in the final optimization.

These values were then used in the subsequent analysis of the effects of urbanization and development on catchment response.

4.5 Routing Coefficients.

The Muskingum method was used to derive routing coefficients from the recorded hydrograph resulting from an isolated storm.

Basically, flows in channels are attenuated by storage effect. Storage itself is related to discharge by the following formula

$$S=KQ^x \text{------------------------------} (4.4).$$

Where

S=storage

Q= discharge

K= ratio of storage to discharge

x= Muskingum coefficient

In a given reach, change in storage is the difference between inflow I and outflow Q given by the following formula

$$I-Q=dS/dt \text{---------------------------} (4.5).$$

Substituting Equation 4.5 into Equation 4.4 yields the following equation

$$I-Q=KXQ^{(X-1)}dQ/dt \text{-------------------} (4.6).$$

For recession flow, I is zero and thus the reduced linealized equation becomes

$$\log(-\Delta Q/\Delta t)=-(X-2)\log Q-\log(KX) \text{------} (4.7).$$

Equation 4.7 was used to derive the routing coefficients by plotting the recession flow on a log-log graph as shown in Figure B.5 of Appendix B for recession flows resulting from an isolated storm.

The results of this analysis gave a K value of 2.83 hours and an x value of 0.42.

In HEC-1, hydrograph routing by the Muskingum method is done as follows:

Q(2)=(CA-CB).I(1)+(1-CA).(Q1)+CB.I(2)-------------(4.8)

Where

CA=2.(TRHR)/(2.AMSK.(1-X)+TRHR)-------------------(4.9)

CB=(TRHR-2.AMSK.X)/(2.AMSK.(1-X)+TRHR)-----------(4.10)

Where:

Q(2)=outflow at end of interval

I(1)=inflow at beginning of interval

I(2)=inflow at end of interval

TRHR=routing interval in hours

AMSK=Muskingum coefficient K

X=Muskingum coefficient X

The output for a typical routing is illustrated in Appendix D.

4.6 Sensitivity Analysis.

The aim of this section was to examine the relative importance of the derived parameters in predicting peak flows and volumes of runoff. Sensitivity analysis also addresses the question of accuracy in estimating the parameters.

For this analysis I1 and I2 stand for impervious proportions as percentage for the two subbasins 1 and 2, respectively. In this study, the two parameters investigated were STRKR and DLTKR because for each storm event, these parameters changed significantly. Besides the two loss rate parameters, the effect of time of concentation was also investigated. TC was varied between 2.0 and 8.0 hours. STRKR was varied from 0.2 inches per hour to 2.0 inches per hour while DLTKR was varied from 0.2 inches to 2.50 inches. The results of this analysis using the derived design storms shown in Table 3.2 are shown in Figures 4.1 through 4.5 and their respective values are shown in Tables 4.6 through 4.10. Examination of Figures 4.4 and Tables 4.6 through 4.10 shows that DLTKR is not a sensitive parameter in estimating both peak flows and runoff volumes while STRKR is a very Sensitive parameter. Quantitatively, a change of 67 percent in DLTKR value from low to high results in a corresponding change in both peak flows and runoff volumes of only 17 percent while a similar change in STRKR value results in a change of 142 percent which means that STRKR is more sensitive than DLTKR.

The accuracy in predicting peak flows and runoff volumes therefore depends on the accurate estimate of the initial loss which in general is a very difficult parameter ot predict with accuracy for a future anticipated storm event. Hence HEC-1, like many other runoff models, is severely restricted in its accuracy by the general inability to predict this parameter for a furture event.

As far as TC s concerned, a change of 50 percent from high to a lower value results in a corresponding increase in peak flows by about 19 percent. However, for Truro Creek watershed, a significant change in TC will be as a result of sewer construction or lining of channels. For the proposed stage of development, it is unlikely that TC will be greatly affected due to the fact that the channel slope is very small.

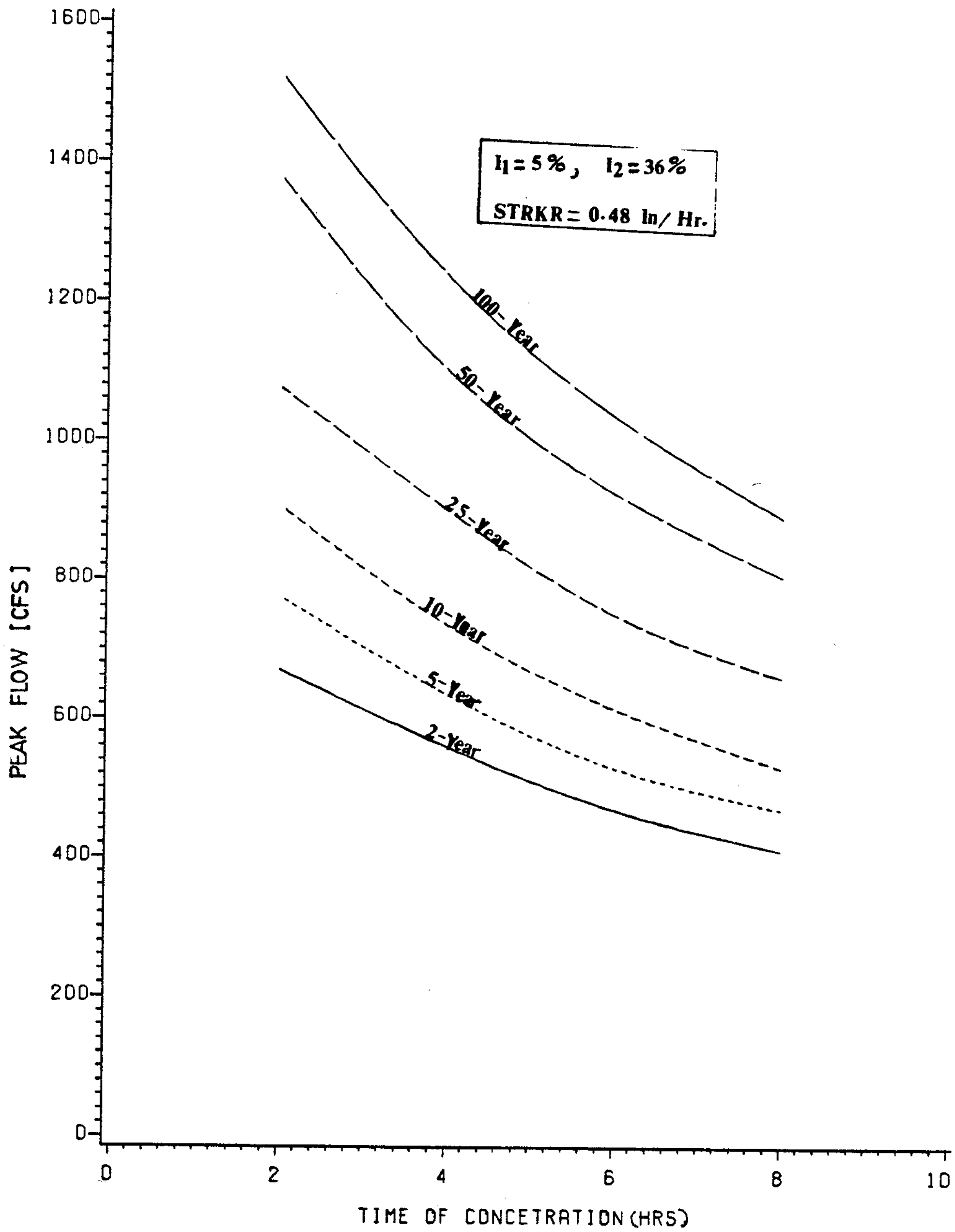

Figure 4.1 Variation of Peak Flows with Time of Concentration

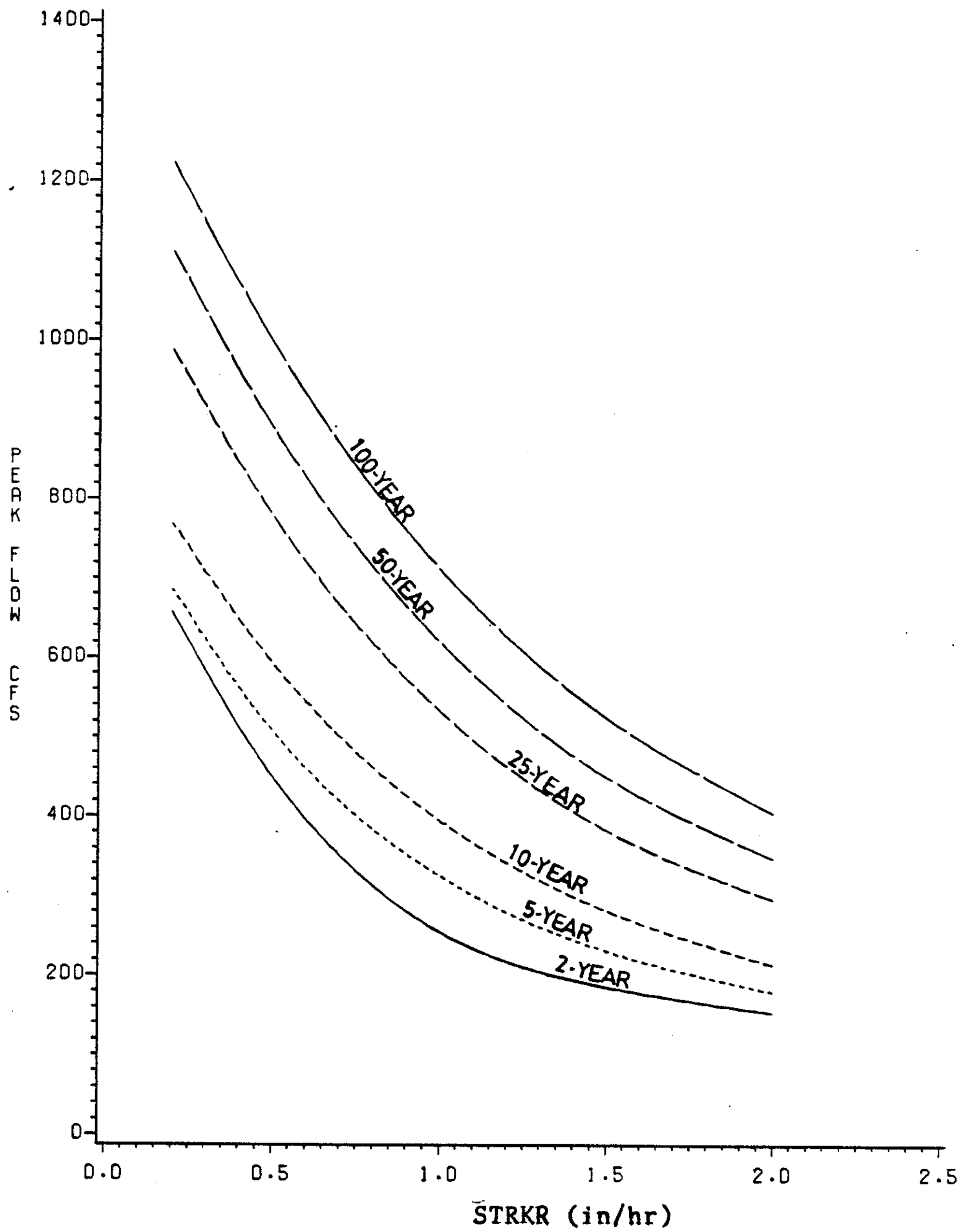

Figure 4.2 Variation of Peak Flows with STRKR

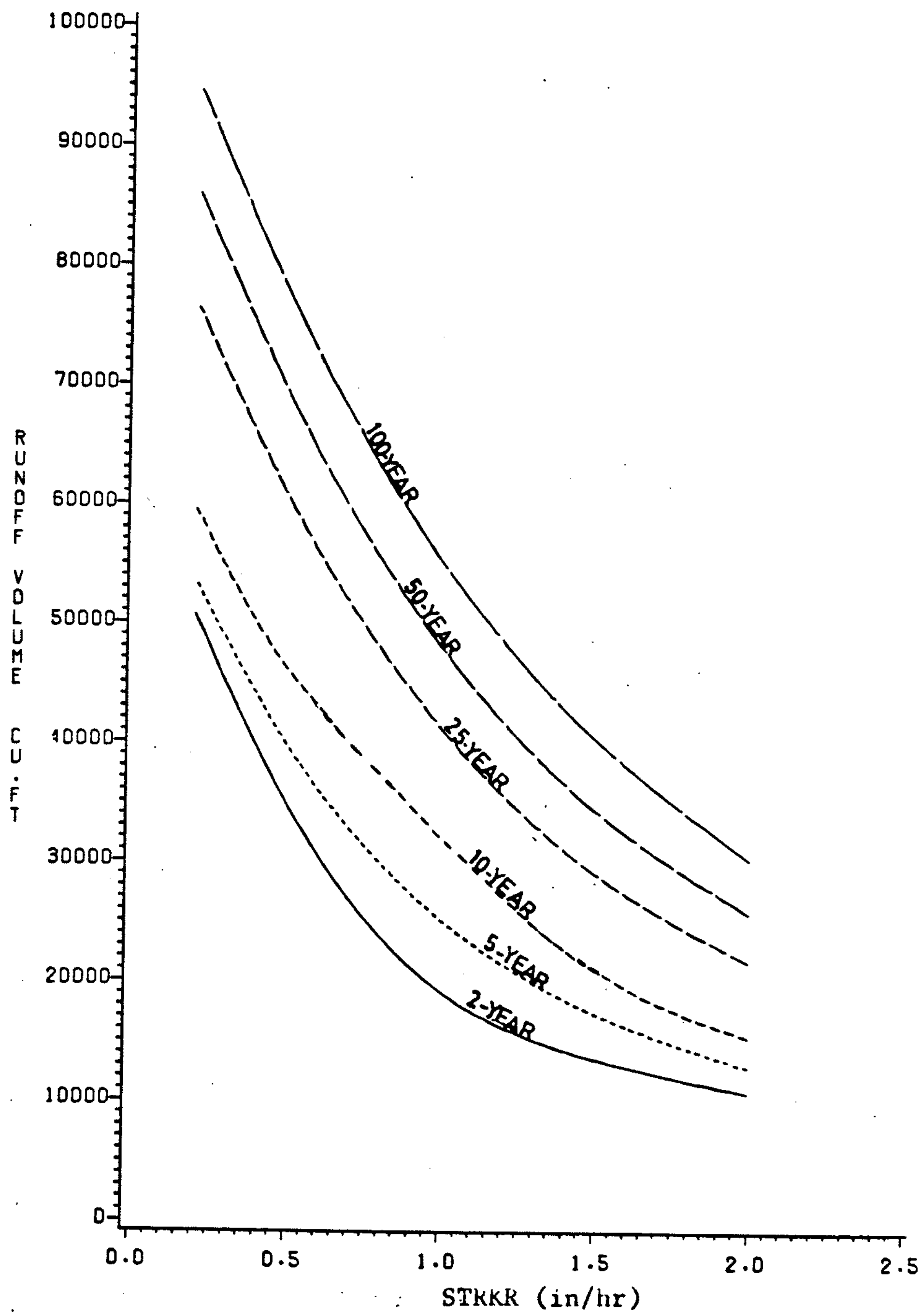

Figure 4.3 Variation of Runoff Volume with STRKR

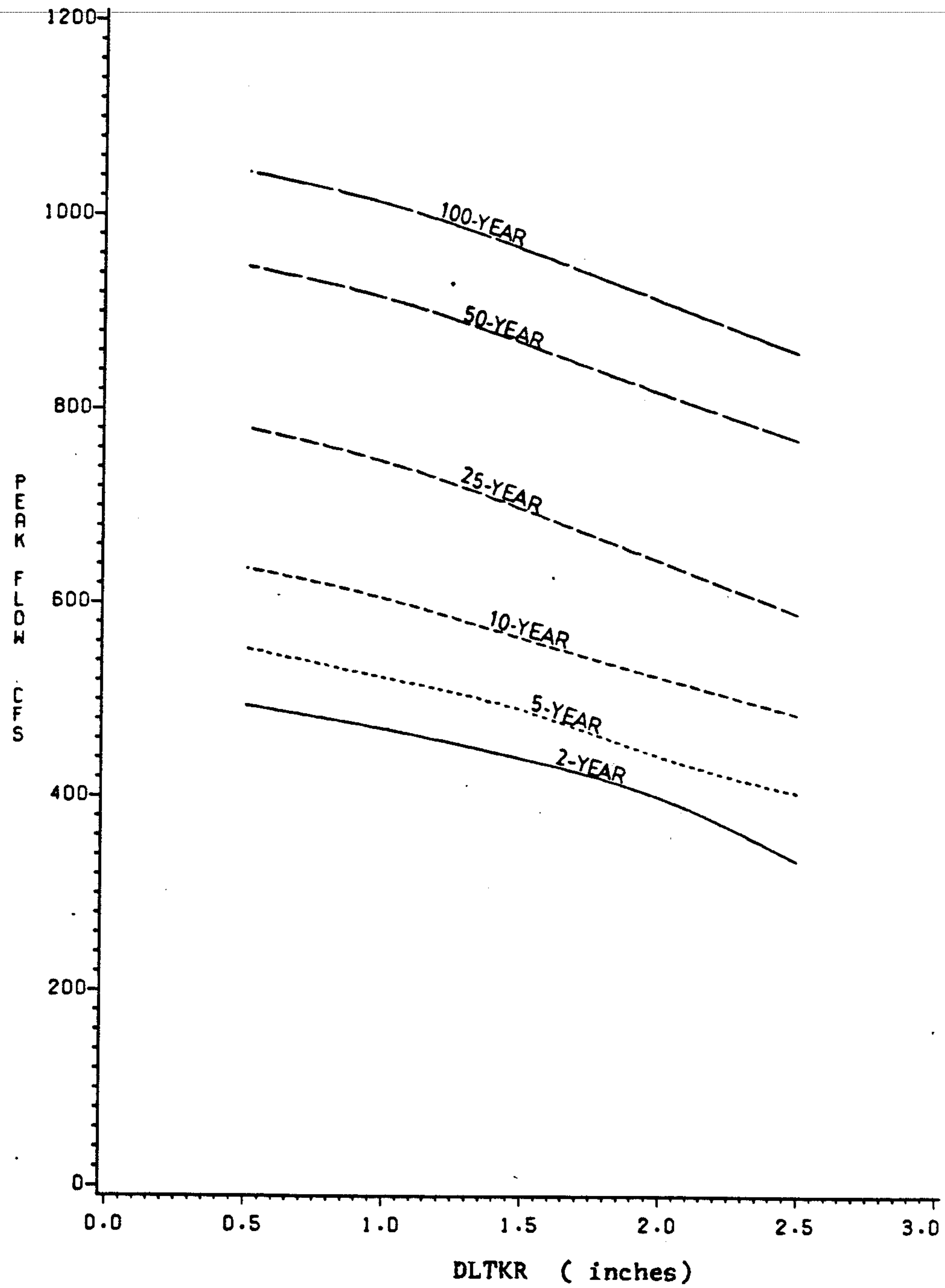

Figure 4.4 Variation of Peak Flows with DLTKR.

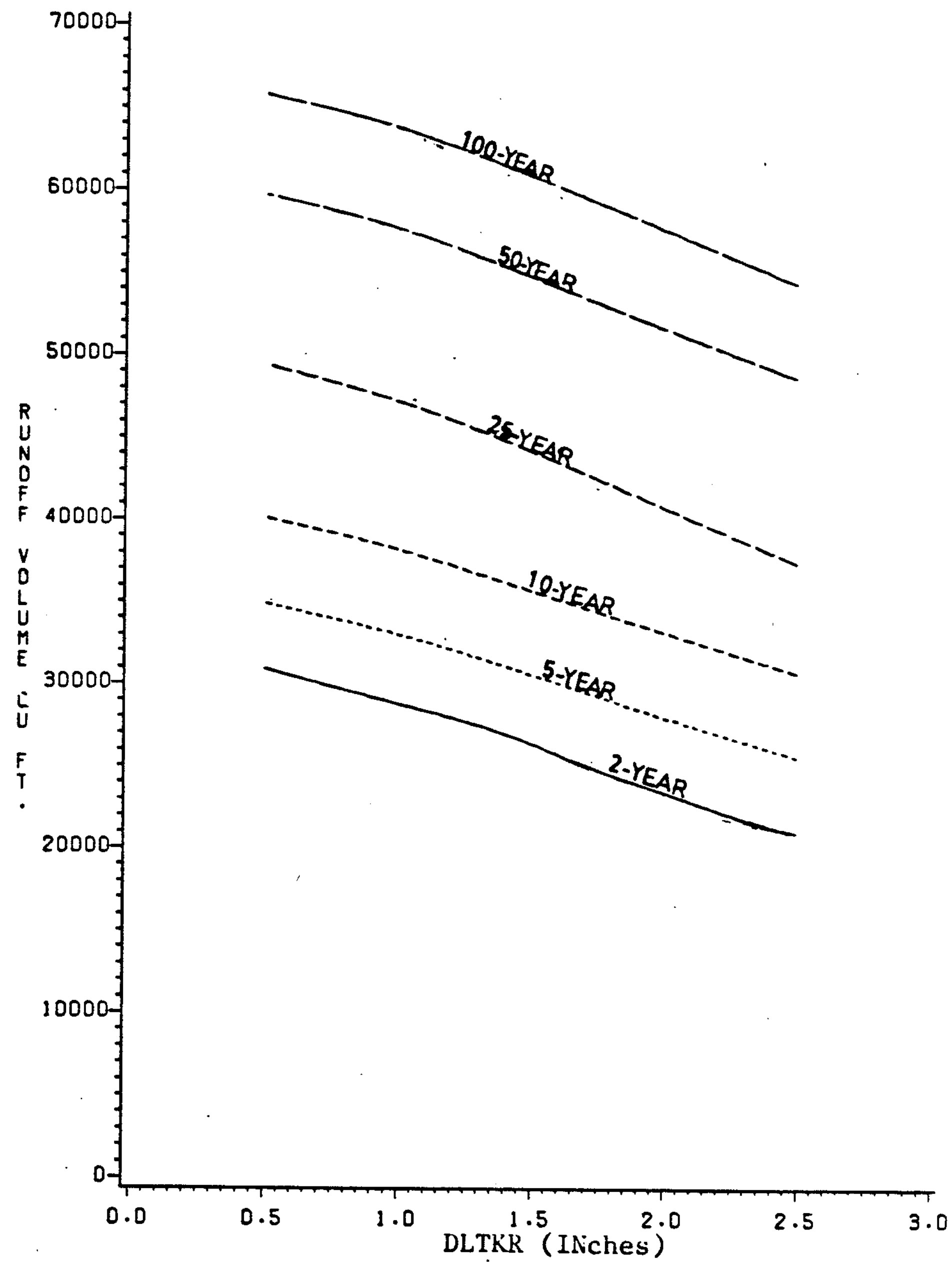

Figure 4.5Variation of Runoff Volume with DLTKR.

Table 4.6 Variation of Peak Flows(cfs) with TC

I1=5%, I2=36%, STRKR=0.48 in/hr

Return Period	TC (hours)			
(years)	2.0	4.0	6.0	8.0
2	670.0	560.0	470.0	411.0
5	775.0	635.0	530.0	470.0
10	905.0	735.0	615.0	530.0
25	1075.0	900.0	750.0	660.0
50	1375.0	1100.0	925.0	805.0
100	1520.0	1235.0	1035.0	890.0

Table 4.7 Variation of Peak Flows(cfs) with STRKR

I1=5%, I2=35%(expected condition)

STRKR	Return Period(years)					
(in/hr)	2	5	10	25	50	100
2.00	154.00	180.00	214.00	296.00	348.00	405.00
1.50	186.00	231.00	282.00	383.00	450.00	525.00
1.00	255.00	325.00	395.00	533.00	620.00	712.00
0.50	450.00	507.00	593.00	779.00	890.00	997.00
0.20	658.00	686.00	769.00	988.00	1112.0	1223.0

Table 4.8 Variation of Runoff Volume(cu. ft.) with STRKR

I1=5%, I2=36% (expected condition)

STRKR	Return Period (years)					
(in/hr)	2	5	10	25	50	100
2.00	10800	12900	15400	21700	25800	30300
1.50	13600	17400	21200	29100	34400	40400
1.00	19200	25200	33600	41200	48000	55200
0.50	34700	39600	46000	60400	69000	77300
0.20	50700	53400	59500	76400	86000	94600

Table 4.9 Variation of Peak Flows(cfs) with DLTKR

I1=5%, I2=36% (expected condition)

DLTKR	Return Period (years)					
(inches)	2	5	10	25	50	100
2.50	334.0	404.0	485.0	589.0	769.0	860.0
2.00	400.0	442.0	524.0	644.0	818.0	913.0
1.50	422.0	490.0	564.0	698.0	869.0	966.0
1.00	470.0	523.0	605.0	746.0	915.0	1012.0
0.50	494.0	553.0	636.0	780.0	947.0	1044.0
0.20	500.0	561.0	644.0	792.0	956.0	1053.0

Table 4.10 Variation of Runoff Volumes(cu. ft) with DLTKR

	DLTKR (inches)					
Return Period (years)	2.50	2.00	1.50	1.00	0.50	0.20
2	21000	22900	26400	28800	30900	31900
5	25600	28000	30600	33000	34900	35400
10	30700	33200	35700	38200	40100	40600
25	37400	40800	44200	47200	49400	50200
50	48700	51700	54800	57700	59700	60700
100	54400	57700	60900	63800	65800	66400

4.7 Model Verification and Validation.

Calibration is concerned with tuning various parameters until the model reproduces observed data. Validation is a test of calibrated parameters on other data apart from those used in calibration.

HEC-1 was used to simulate flows for May 20, 1984, June 8, 1984, June 16, 1984 and June 21, 1984 storm events. For all these storm events, the hydrograph fit was not perfect but peak flows for both observed and computed flows occured at about the same time. For the May 20, 1984 storm event, the difference between the simulated and observed peak flows was 9 percent, for June 16, 1984 storm event, the difference was 24 percent. In order to get a reasonable close fit in hydrograph peak flow for June 21, 1984 storm event, trial values of STRKR were used until there was a match between observed and recorded hydrographs at a STRKR value of 1.15 inches per hour which is larger than the value in the optimization process for the same date by 0.35 inches per hour. The higher STRKR value is unusual as the occurence of the June 16 1984 storm a few days earlier would indicate that a lower rather than a higher initial loss rate might be anticipated for this storm. Perhaps another reason for is that for June 21, 1984 storm event, the rainfall was characterized by a high intensity short duration which might mean that only a portion of the basin contributed to the peak flow before rain stopped. On the other hand the antecedent precipitation depth 4 days earlier was only 0.35 inches which may suggest that the soil was almost dry which resulted in a higher STRKR value.

The comparisons were done by plotting the computed and observed hydrographs on the same graph for a particular storm event and by comparing the differences between the predicted peak flow and the observed peak flow. Figures 4.7 through 4.9 show the respective comparisons of simulated and observed hydrographs.

Validation was done by utilizing the June 8, 1984 storm and the recorded hydrograph which happens to have multiple peaks. The results showed a difference in peak flows of 36 percent. In so far as the general fit is concerned, the computed flows follow the observed flows very well except for the differences in peaks. From this analysis it was concluded that the derived time parameters are within the desired accuracy shown by phase match of the computed and observed peaks.

The model does not appear to reproduce peak flows very well simply because of different moisture levels in the the basin. It was suggested earlier in this study that antecedent moisture levels can vary significantly from one storm event to another. Hence the choice of the above parameters to fit the peak flows is justified as long as care is taken in using the correct values for the antecedent moisture conditions.

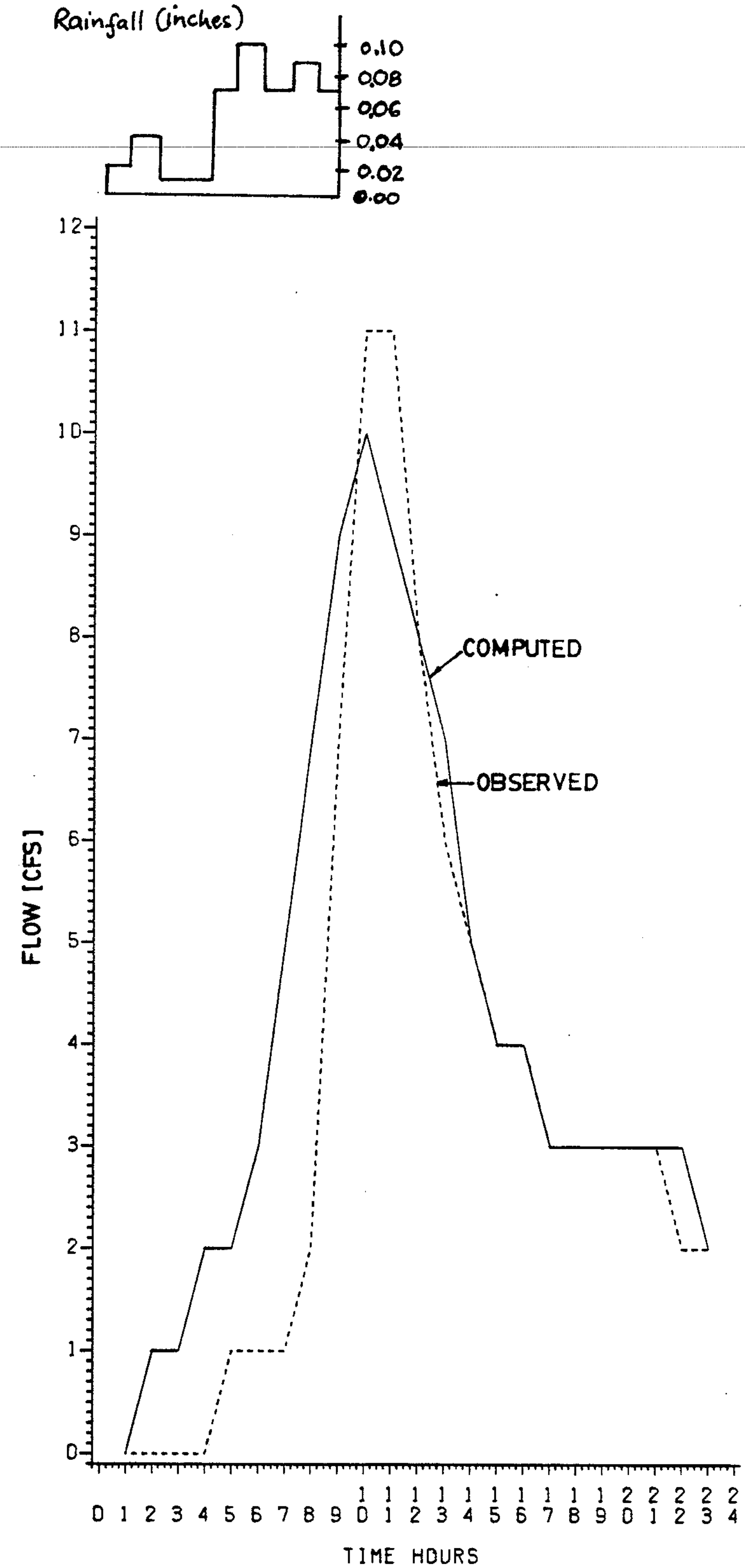

Figure 4.6 Comparison of Observed and Computed Flows for May 20, 1984 Storm event.

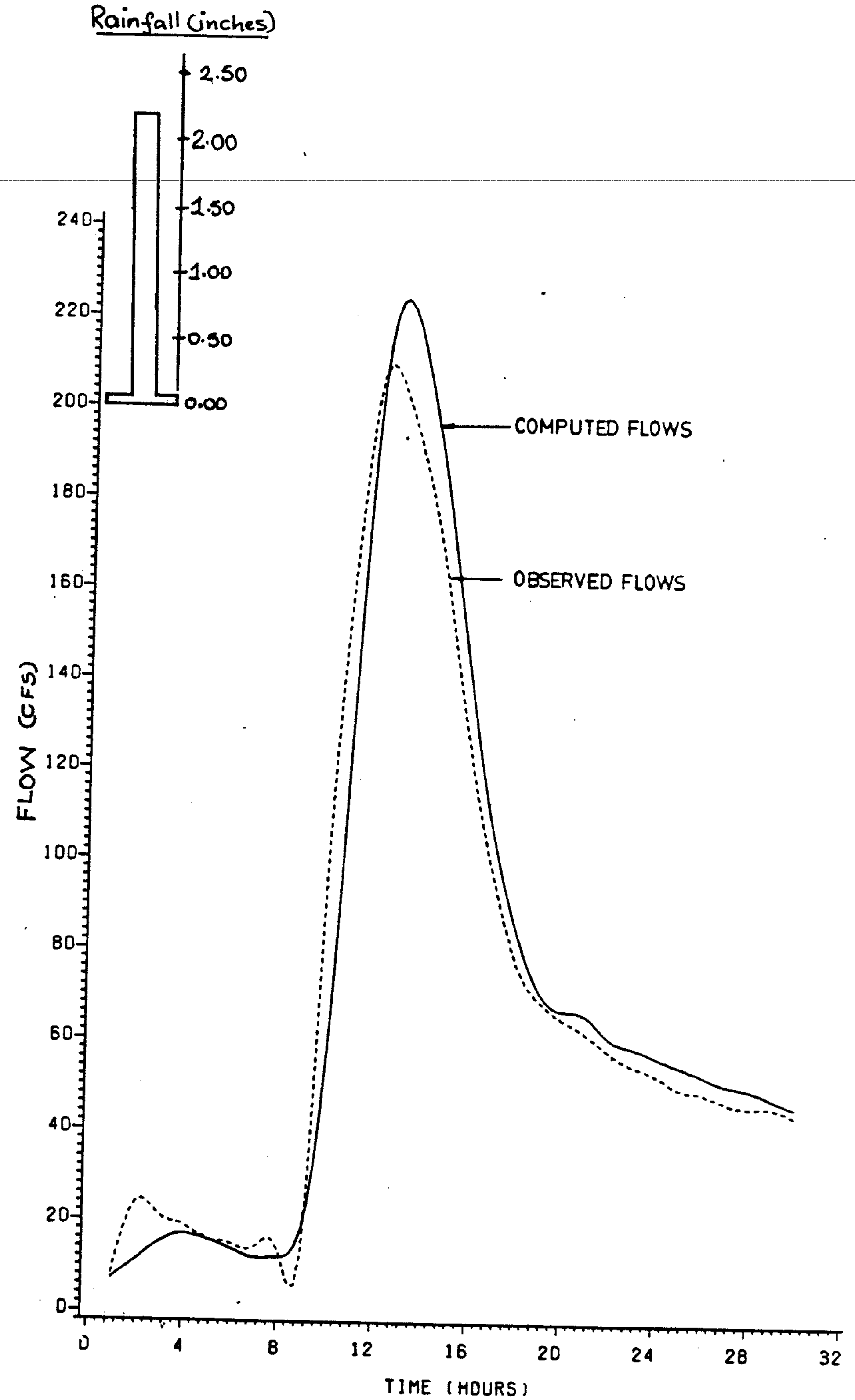

Figure 4.7 Comparison of Observed and Computed Flows for the June 21,1984 Storm Event

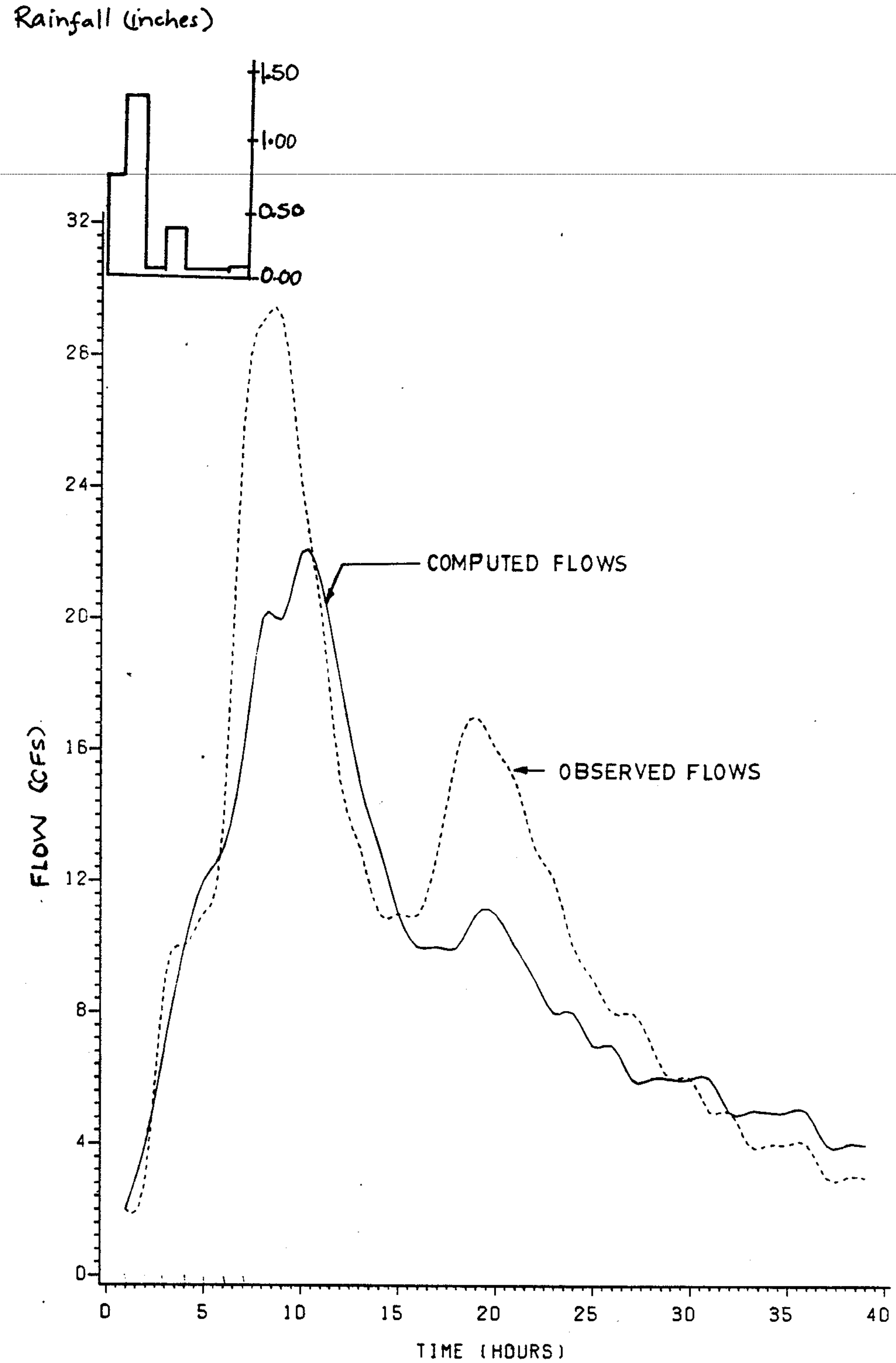

Figure 4.8 Comparison of Observed and Computed Flows for June 16, 1984 Storm event.

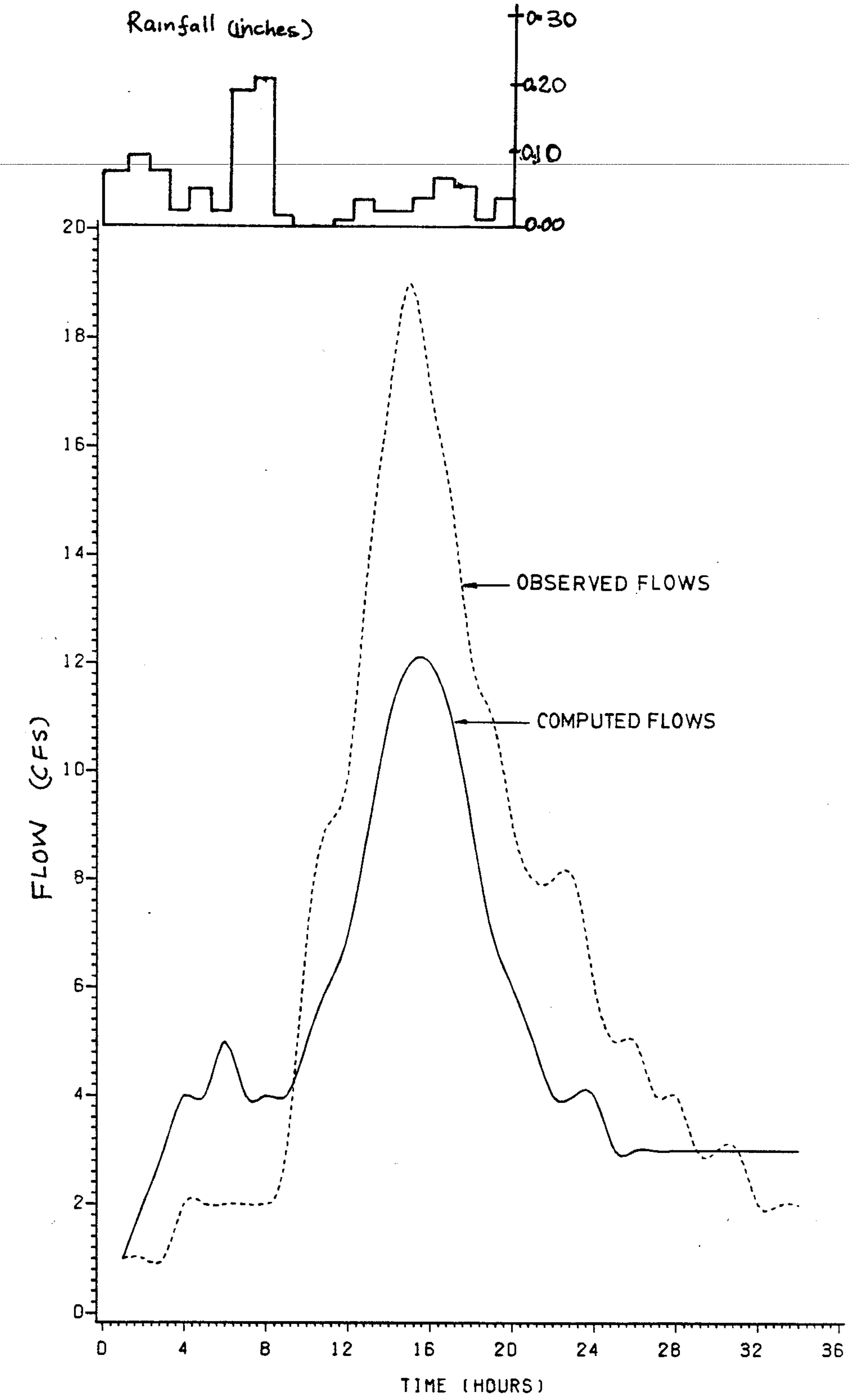

Figure 4.9 Comparison of Observed and Computed Flows for June 8, 1984 Storm event.

Chapter V

ASSESSING URBAN RUNOFF CHANGES

In assessing the runoff changes associated with urbanization, the following conditions were assumed

(i) There is a one to one ratio between the frequency of the design storm and that of the resulting flood. In the strict sense this in not true as the probability of other parameters such as antecedent soil moisture also affect the frequency or probability of a runoff event. The assumption was used however, to facilitate a comparison within the bounds of the thesis.

(ii) The rainfall resulting from the storm event is uniform over the entire basin.

(iii) Since the slope of the basin is so small, the only changes to time of concentration will be as a result of sewer construction.

(iv) Runoff due to snowmelt is negligible during the period of study which covers mostly summer months.

This section deals with several approaches to the assessment of urban runoff changes. Emphasis was on proportion of imperviousness and moisture levels in so far as it affects the prediction of flood frequencies of ungauged locations. The moisture level is depicted by the initial infiltration losses. The higher the losses, the drier the initial soil conditions and vice versa for the lower infiltration losses. In both situations, comparisons were done based on simulated quantities for the existing and expected conditions. The expected condition include increasing impervious proportion of subbasin 1 from 2 percent to 5 percent while increasing subbasin 2 impervious proportion from 10 percent to 36 percent. The derived flood frequencies were compared with recorded flood frequency results for rural as well as urbanized conditions. Frequencies generated in the model were then compared with those generated by applying statistical models.

5.1 Simulation of Rural and Urban Runoff Quantities.

In assessing changes in urban runoff, runoff quantities of peak flows and volumes were simulated for the existing conditions for the design storms of 2, 5, 10, 25, 50, and 100-year recurrence intervals. The existing condition assumed 2 percent of subbasin one with impervious cover and 10 percent of subbasin 2 with impervious cover.

In this section, I, shall stand for impervious proportion and thus I1 stands for the impervious proportion of subbasin 1 while I2 stands for impervious proportion for subbasin 2.

In both of these analyses, calibrated parameters were used. The runoff quantities were also simulated at a STRKR value of 1.75 inches per hour in correspondence to experimental results of Watson(1969) on similar type of soil for an initially dry surface. The soils tested by Watson ranged from silty clay loam to heavy clay. For this basin the soil consists of fine textured black earth with an underlying stratum of clay and hence approximates the soils tested by Watson and justifies the use of 1.75 inches per hour as STRKR for an initially dry soil surface. In real situations, however, the basin moisture keeps on changing due to differing amounts of antecedent precipitation and climatic conditions. For these reasons, the initial losses do not remain constant.

The results of simulation of urban runoff quantities at different initial losses are shown in Tables 5.1 and 5.1 for peak flows and runoff volumes, respectively.

In typical urban areas, development seems never to come to an end. Practically all urban areas will experience inreasing development. Because of this, urban runoff quantities were simulated for various degrees of development. At the same time in recognition of the fact that initial losses do not remain constant, rural and urban runoff quantities were simulated for differing initial losses in an analysis under the heading of sensitivity analysis.

Table 5.1 Comparison of Rural and Urban Peak Flows(cfs) at Different Initial Losses

tr (years)	QPr1	QPu1	GF1	QPr2	QPu2	GF2
2	65.0	170.0	2.60	401.0	459.0	1.14
5	103.0	206.0	2.00	460.0	517.0	1.10
10	140.0	248.0	1.77	547.0	600.0	1.08
25	212.0	340.0	1.60	727.0	786.0	1.08
50	264.0	399.0	1.51	870.0	930.0	1.07
100	324.0	465.0	1.43	944.0	1000.0	1.06

Where QP r1 and QP u1 are Rural and Urban Peak Flows
at STRKR=1.75 inches per hour
Qp r2 and QP u2 are Rural and Urban Peak Flows
at STRKR=0.48 inches per hour
GF1 is QP u1/QP r1 (growth factor)
GF2 is QP u2/QP r2 (growth factor)

Table 5.2 Comparison of Rural and Urban Runoff Volumes in cubic feet at Different Values of STRKR

tr (years)	Vr1	Vu1	GF1	Vr2	Vu2	GF2
2	4700	12200	2.60	24900	28500	1.14
5	7600	15200	2.60	29200	32700	1.12
10	10400	18300	1.76	34400	37900	1.10
25	15900	25400	1.60	42000	46200	1.08
50	20500	30100	1.47	52600	56700	1.08
100	24600	35400	1.44	59200	63400	1.07

Where V r1 and V u1 are Rural and Urban Runoff Volumes
at STRKR=1.75 inches per hour
V r2 and V u2 are Rural and Urban Runoff Volumes
at STRKR=0.48 inches per hour
GF1 is V u1/V r1 (growth factor)
GF2 is V u2/V r1 (growth factor)

5.2 Estimation of Rural and Urban Peak Flows with Statistical Models

The aim of this section was to generate flood flows for various recurrence intervals by making use of statistically derived equations for comparison with the flood flows generated by HEC-1.

The statistical models were developed by regression techniques on urban flow data ranging in length from 5 years to 43 years. The equations derived, only apply to flood frequencies of up to and including the 50-year return period as discussed by Espey and Winslow (1974). For this reason flood frequencies using the statistical models were only derived for return periods of up to 50 years.

The statistical models are shown in Table 5.3. The other parameters relating to these models are shown in Table 5.4. The values of peak flows for rural conditions were estimated by assuming a Φ value of 1.0 with an assumed 4 percent overal impervious proportion while for urban condition, a value of 1.1 was assumed for Φ with 14 percent overal impervious proportion. These values together with other basin physiographic features for Truro Creek of Table 1.1 and the corresponding rainfall depths were substituted into equations of Table 5.3 for estimation.

Table 5.3 Derived Flood Frequency Equations For Urban Watersheds (after Espey and Winslow, 1974)

Equation	Correlation Coefficient (logs)	Average Absolute Percentage error
$Q_{2.33}=169A^{0.77} I^{.29} S^{.42} R^{1.80} \Phi^{-1.17}$	0.97	30
$Q_5 =172A^{.80} I^{.27} S^{.43} R^{1.73} \Phi^{-1.21}$	0.97	31
$Q_{10} =178A^{.82} I^{.26} S^{.44} R^{1.71} \Phi^{-1.32}$	0.96	32
$Q_{20} =243A^{.84} I^{.24} S^{.48} R^{1.62} \Phi^{-1.38}$	0.96	32
$Q_{50} =297A^{.85} I^{.22} S^{.50} R^{1.57} \Phi^{-1.61}$	0.96	34
$Q_5 =1.13Q_{2.33}^{1.03}$	0.99	8
$Q_{10} =1.24Q_{2.33}^{1.05}$	0.99	16
$Q_{20} =1.34Q_{2.33}^{1.06}$	0.99	22
$Q_{50} =1.47Q_{2.33}^{1.08}$	0.97	28

Where A= Drainage area in square miles

S= slope in feet per foot

Rt= Rainfall depth for return peiod t for 6 hours duration in inches.

I= Impervious proportion as a percentage

Table 5.4 Classification of Channel Urbanization Factor (after Espey and Winslow, 1974)

Value of Φ	Classification
	(a) For Φ_a
0.6	Extensive channel improvements and storm sewer system, closed conduit system
0.8	Some channel improvement and storm sewers; mainly cleaning and enlargement of existing channel
1.0	Natural channel conditions
	(b) For Φ_b
0.0	No vegetation
0.1	Light channel vegetation
0.2	Moderate channel condition
0.3	Heavy channel vegetation

Where Φ = dimensinless urbanization factor

$$\Phi = \Phi_a + \Phi_b$$

The results of this analysis are shown in Figures 5.1 and 5.2. On Figure 5.1, flood frequency values for the rural and urban conditions for Houston , Texas are also plotted to facilitate visual comparison of the generated flood frequencies. The comparison in Figure 5.1 is to show the similarity and differences in general shape of the changes in flows between the Truro Creek system described in this thesis and the basin used in Houston study. The most interesting feature of the observation for the Houston study is that there is significant increase in peak flows from higher return periods. This issue is contrary to the findings of this thesis and many other studies and is discussed in more details in chapter 6. The flood frequencies for the basin in Houston were derived by fiting data to the Log-Pearson Type III method for both rural and urban conditions. In the Houston case, the urbanized condition consisted of 27 percent impervious proportion which corresponds to data collected between 1950 and 1972 while the rural conditions included the period between 1937 and 1950. Figure 5.1 shows that the two basins show similar forms in the changes in peak flows for the assumed development conditions.

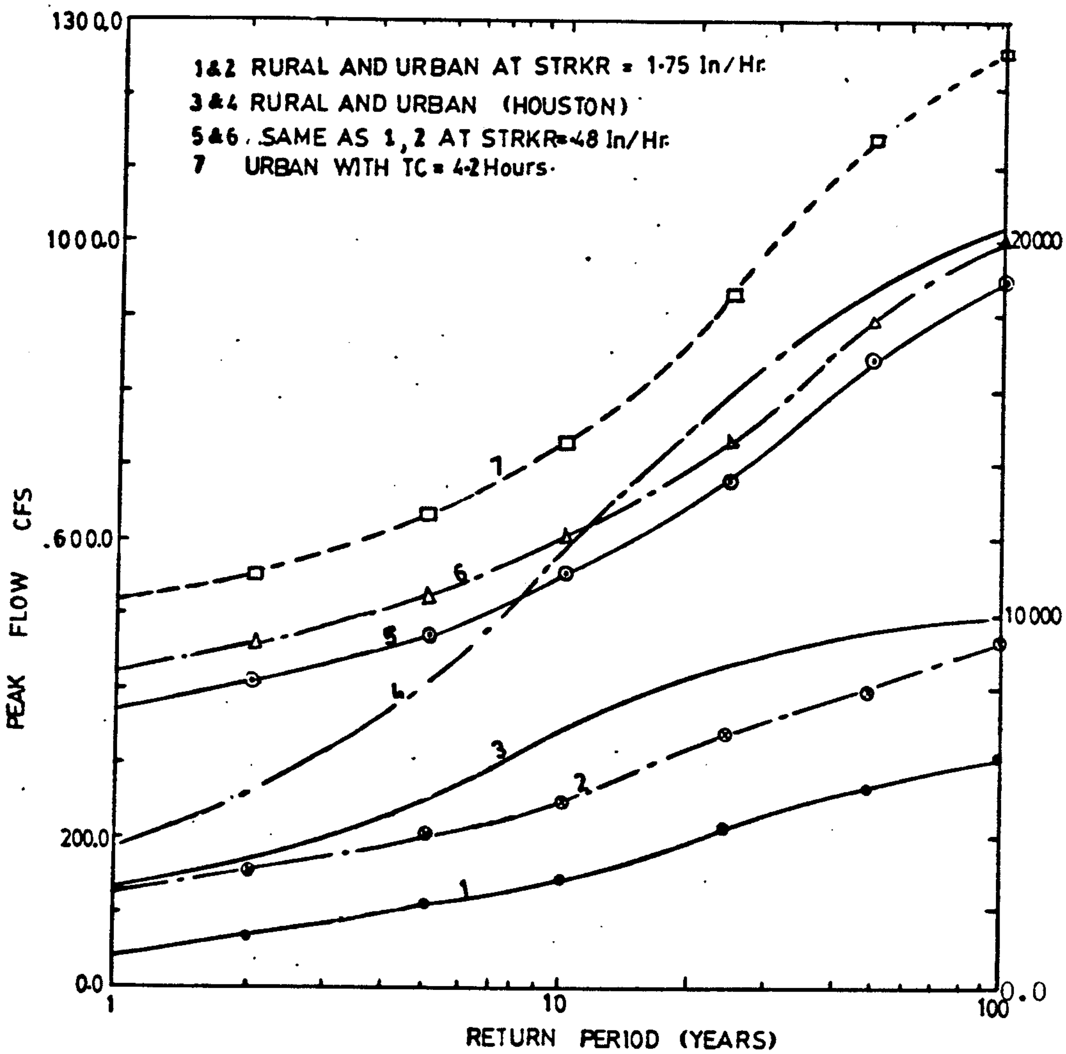

Figure 5.1 Comparison of Results Obtained by Using HEC-1 on Truro Creek and Recorded Flood Frequencies for Houston, Texas.

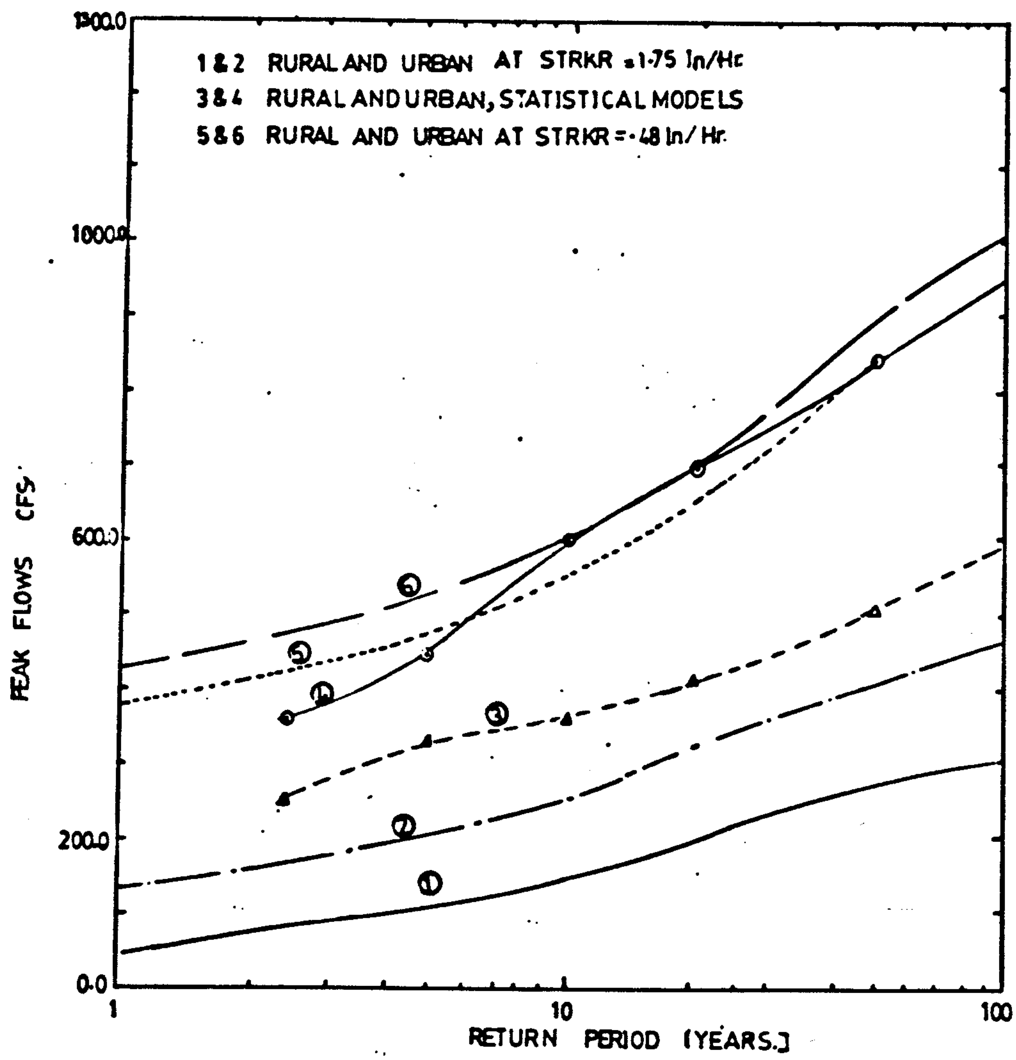

Figure 5.2 Comparison of Results Obtained by Using HEC-1 and Statistical Models for Truro Creek Watershed.

5.3 Sensitivity Analysis.

The aim of this section was to explore the extent to which runoff quantities will be modified for differing basin development and to examine the effects of moisture conditions in predicting flood frequencies for Truro creek.

Several conditions of development were examined. For a hypothetical condition, the impervious proportion was varied for both subbasins.

Theoretically, subbasin 1 will not undergo any major development because it is outside the building limit of the airport. Subbasin two is already partially developed and any kind of development can take place in the future.

Three conditions were examined at different STRKR values ranging from 0.20 to 2.00 inches per hour. These conditions are as follows;

(i) I1=2 percent, I2=10 percent (existing condition)

(ii) I1=5 percent, I2=36 percent (expected condition)

(iii) I1=5 percent, I2=100 percent(most likely future condition)

In this analysis, the expected condition corresponds to the proposed development extent while the most likey future condition refers to some distant future development condition. The other conditions examined were on proportion of impervious ranging from 20 percent to 100 percent in equal increments for both subbasins, and another condition involved keeping I1 constant at 5 percent while varying I2 form 20 percent to 100 percent.

It should be noted that STRKR values of less than 0.40 represents a wet soil condition and STRKR value above 1 represents a dry soil condition.

The results of this analysis are shown in Tables 5.5 through 5.11. Comparison of rural and urban runoff quantities was carried out by taking the ratios of rural runoff quantities to urban runoff quantities as shown in Tables 5.12 through 5.19.

A graphical summary of the results are shown in Figures 6.3 through 6.15.

Table 5.5 Results of Estimation of Peak Flows using Statistical Models for Rural and Urban Conditions

Return Period (years)	QP ural (cfs)	QP urban (cfs)	QP urban/QP rural
2.33	249.0	359.0	1.44
5	330.0	484.0	1.47
10	362.0	597.0	1.65
25	413.0	684.0	1.66
50	505.0	845.0	1.67

Where QP rural and QP urban are for peak flows for rural and urban conditions, respectively.

Table 5.6 Peak Flows(cfs) Versus STRKR for Rural Conditions

Return Period	STRKR (incher per hour)				
(years)	2.0	1.5	1.0	0.50	0.20
2	46.0	84.0	165.0	391.0	626.0
5	71.0	134.0	246.0	454.0	656.0
10	98.0	181.0	315.0	540.0	738.0
25	159.0	265.0	440.0	720.0	955.0
50	202.0	325.0	521.0	827.0	1078.0
100	252.0	396.0	612.0	933.0	1188.0

Table 5.7 Runoff Volumes(cu. ft.) Versus STRKR for Rural Conditions

Return Period	STRKR(inces per hour)				
(years)	2.0	1.50	1.00	0.50	0.20
2	3300	6400	12800	30100	48000
5	5400	10500	19300	35400	50900
10	7500	14100	24500	41900	57000
25	12300	20600	34200	55700	73700
50	15700	25300	40500	64000	83200
100	19600	30900	47600	72300	91700

Table 5.8 Variation of Peak Flows(cfs) with STRKR for I1=5% and I2=36% condition

Return Period	STRKR(inches per hour)				
(years)	2.00	1.50	1.00	0.50	0.20
2	154.0	186.0	255.0	450.0	659.0
5	180.0	231.0	325.0	507.0	686.0
10	214.0	282.0	395.0	593.0	769.0
25	296.0	393.0	533.0	779.0	988.0
50	348.0	450.0	620.0	890.0	1112.0
100	405.0	525.0	712.0	997.0	1223.0

Table 5.9 Variation of Runoff Volumes(cubic feet) with STRKR for I1=5% and I2=36% condition

Return Period (years)	STRKR (inches per hour) 2.00	1.50	1.00	0.50	0.20
2	10800	19200	19200	34700	50700
5	12900	25200	25200	39600	53400
10	15400	33600	33600	46000	59500
25	21700	41200	41200	60400	76400
50	25800	48000	48000	69000	86000
100	30300	55200	55200	77300	94600

Table 5.10 Variation of Peak Flows with STRKR for I1=5% and I2=100% Condition

Return Period (years)	STRKR(inces per hour) 2.0	1.50	1.0	0.50	0.20
2	387.0	408.0	451.0	577.0	723.0
5	416.0	448.0	505.0	622.0	747.0
10	466.0	509.0	579.0	707.0	831.0
25	596.0	651.0	743.0	906.0	1056.0
50	673.0	736.0	841.0	1022.0	1188.0
100	748.0	821.0	937.0	1129.0	1294.0

Table 5.11 Variation of Runoff Volumes(cubic feet) with STRKR for I1=5% and I2=100% condition

Return Period (years)	STRKR(inches per hour) 2.0	1.50	1.00	0.50	0.20
2	24700	27800	32100	43800	55900
5	27600	31100	37000	47900	58300
10	31000	35400	42500	54200	64000
25	40400	46000	55200	69700	81800
50	46000	52600	62900	78700	91600
100	51600	59200	70500	87200	100200

Table 5.12 Growth Factors for Peak Flows(QP urban/QP rural) at Diffefernt STRKR Values for I1=5% and I2=36% Urban Condition

Return Period (years)	STRKR(inches per hour) 2.00	1.50	1.00	0.50	0.20
2	3.25	2.21	1.55	1.15	1.05
5	2.54	1.72	1.32	1.12	1.05
10	2.18	1.56	1.25	1.10	1.04
25	1.86	1.45	1.21	1.08	1.03
50	1.72	1.38	1.19	1.08	1.03
100	1.61	1.33	1.16	1.07	1.03

Where QP urban/QP rural refers to the ratio of ur- ban peak flow to the rural peak flow(Growth Factor)

Table 5.13 Growth Factors for Runoff Volumes (V urban/V rural) for I1=5% and I2=36% Urban Condition

Return Period (years)	STRKR(inches per hour) 2.00	1.50	1.00	0.50	0.20
2	3.27	2.13	1.50	1.15	1.06
5	2.39	1.66	1.31	1.12	1.05
10	2.05	1.50	1.25	1.10	1.04
25	1.76	1.41	1.20	1.08	1.04
50	1.64	1.36	1.19	1.08	1.03
100	1.55	1.31	1.16	1.07	1.03

Where V urban/V rural refers to the growth factor

Table 5.14 Growth Factors for Peak Flows(QP urban/QP rural) Versus STRKR Values for I1=5% and I2=100% Condition

Return Period (years)	STRKR(inches per hour) 2.00	1.50	1.00	0.50	0.20
2	8.41	4.85	2.73	1.48	1.15
5	5.86	3.34	2.05	1.37	1.14
10	4.75	2.81	1.84	1.31	1.13
25	3.75	2.46	1.69	1.31	1.11
50	3.32	2.26	1.61	1.26	1.10
100	2.96	2.07	1.53	1.24	1.09

Where QP urban/QP rural refers to the peak flow growth factor

Table 5.15 Growth Factors for Runoff Volumes (V urban/V rural) Versus STRKR for I1=5% and I2=100% Condition

Return Period (years)	STRKR(inches per hour) 2.00	1.50	1.00	0.50	0.20
2	7.79	4.34	2.51	1.21	1.16
5	5.11	2.96	1.92	1.46	1.16
10	4.13	2.51	1.73	1.35	1.13
25	3.28	2.23	1.61	1.29	1.11
50	2.93	2.08	1.55	1.23	1.10
100	2.63	1.92	1.48	1.21	1.09

Where V urban/V rural refers to ratio of urban runoff volumes to rural runoff volumes.
tr is return period

Table 5.16 Growth Factors for Peak Flows(QP urban/QP rural) with STRKR for I1=I2=100% Urban Condition.

Return Period (years)	STRKR(inches per hour) 2.00	1.50	1.00	0.50	0.20
2	20.43	11.19	5.70	2.40	1.50
5	13.41	7.10	3.87	2.10	1.45
10	10.56	5.72	3.29	1.92	1.40
25	8.10	4.83	2.90	1.78	1.34
50	6.91	4.29	2.68	1.68	1.29
100	6.08	3.87	2.50	1.64	1.29

Where QP urban/QP rural refers to the growth factors

Table 5.17 Growth Factors for Runoff Volumes (V urban/V rural) with STRKR for I1=I2=100% Urban Condition.

Return Period (years)	STRKR(inches per hour) 2.00	1.50	1.00	0.50	0.20
2	21.85	11.27	5.63	2.40	1.50
5	13.89	7.14	3.89	2.12	1.47
10	10.69	5.65	3.25	1.90	1.40
25	8.02	4.79	2.88	1.77	1.34
50	6.90	4.28	2.68	1.69	1.30
100	6.02	3.82	2.48	1.63	1.29

Where V urban/V rural refers to the growth factors.

Table 5.18 Growth Factors for Peak Flows (QP urban/QP rural) Versus Impervious Proportion for I1=I2 Condition

Return Period (years)	Impervious Proportion(%) 20	40	60	80	100
2	1.18	1.50	1.70	2.06	2.34
5	1.17	1.39	1.62	1.84	2.06
10	1.12	1.28	1.44	1.60	1.77
25	1.12	1.28	1.44	1.60	1.77
50	1.10	1.27	1.38	1.55	1.69
100	1.10	1.23	1.34	1.48	1.60

QP urban/QP rural refers to the growth factor.

Table 5.19 Growth Factors for Runoff Volumes

(V urban/V rural) Versus Impervious Proportion for I1=I2.

Return Period	Impervious Proportion(%)				
(years)	20	40	60	80	100
2	1.06	1.16	1.26	1.35	1.45
5	1.06	1.13	1.20	1.28	1.36
10	1.05	1.10	1.17	1.23	1.30
25	1.04	1.09	1.14	1.20	1.25
50	1.04	1.09	1.13	1.18	1.22
100	1.03	1.07	1.12	1.16	1.20

Where V urban/V rural refers to the growth factors.

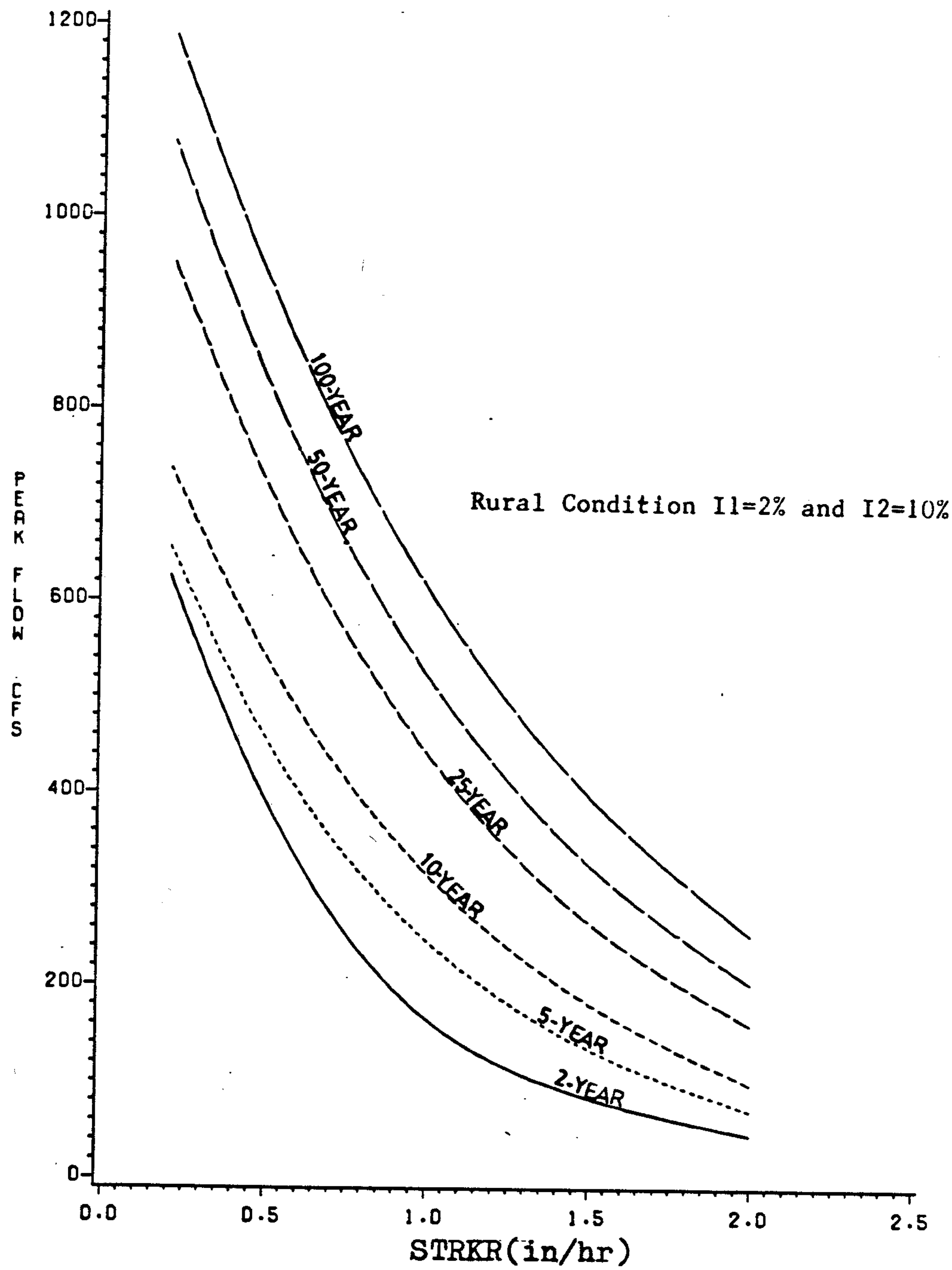

Figure 5.3 Peak Flow Variation with STRKR for Rural Conditions.

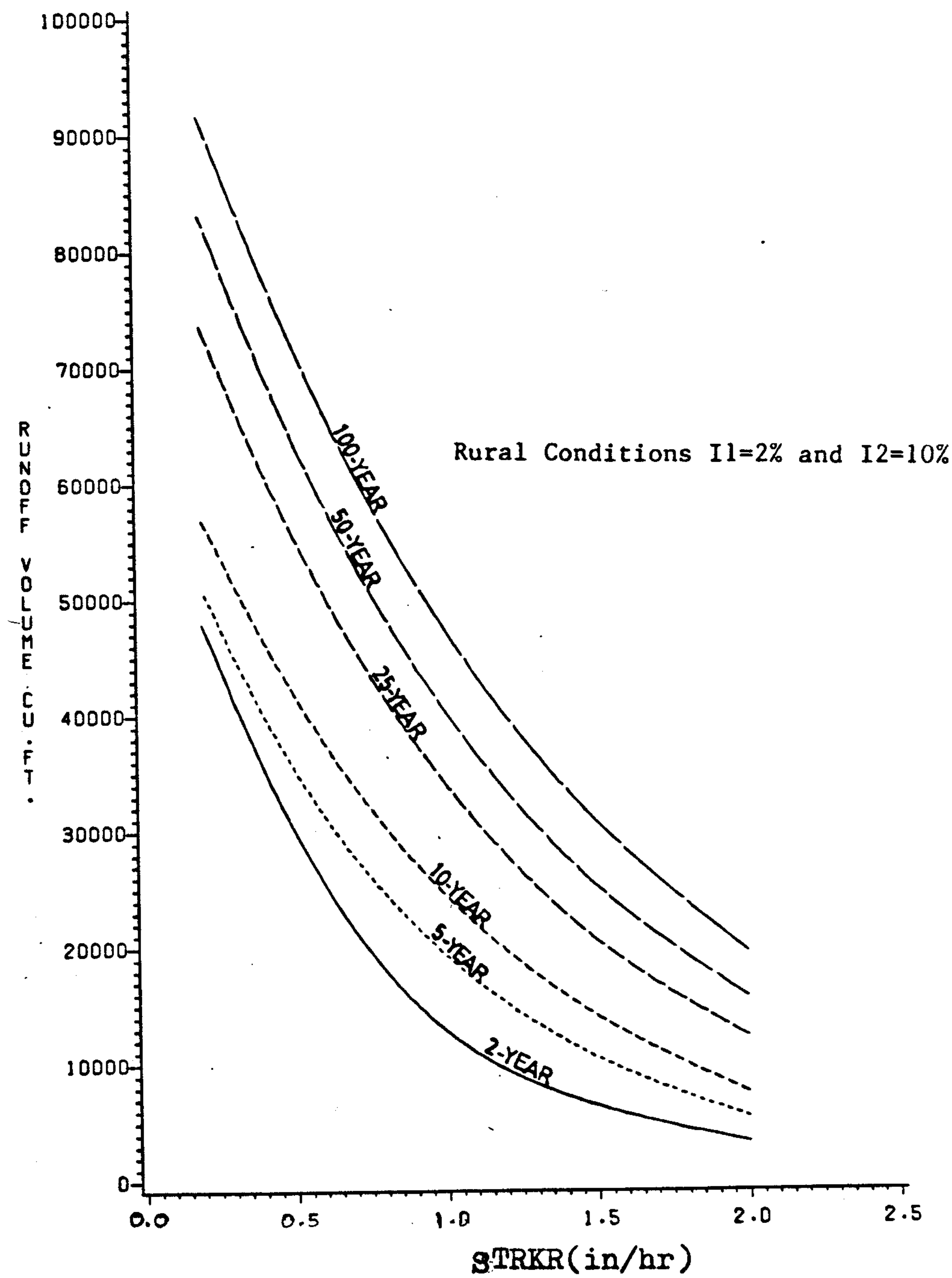

Figure 5.4 Runoff Volume Variation with STRKR for Rural Conditions.

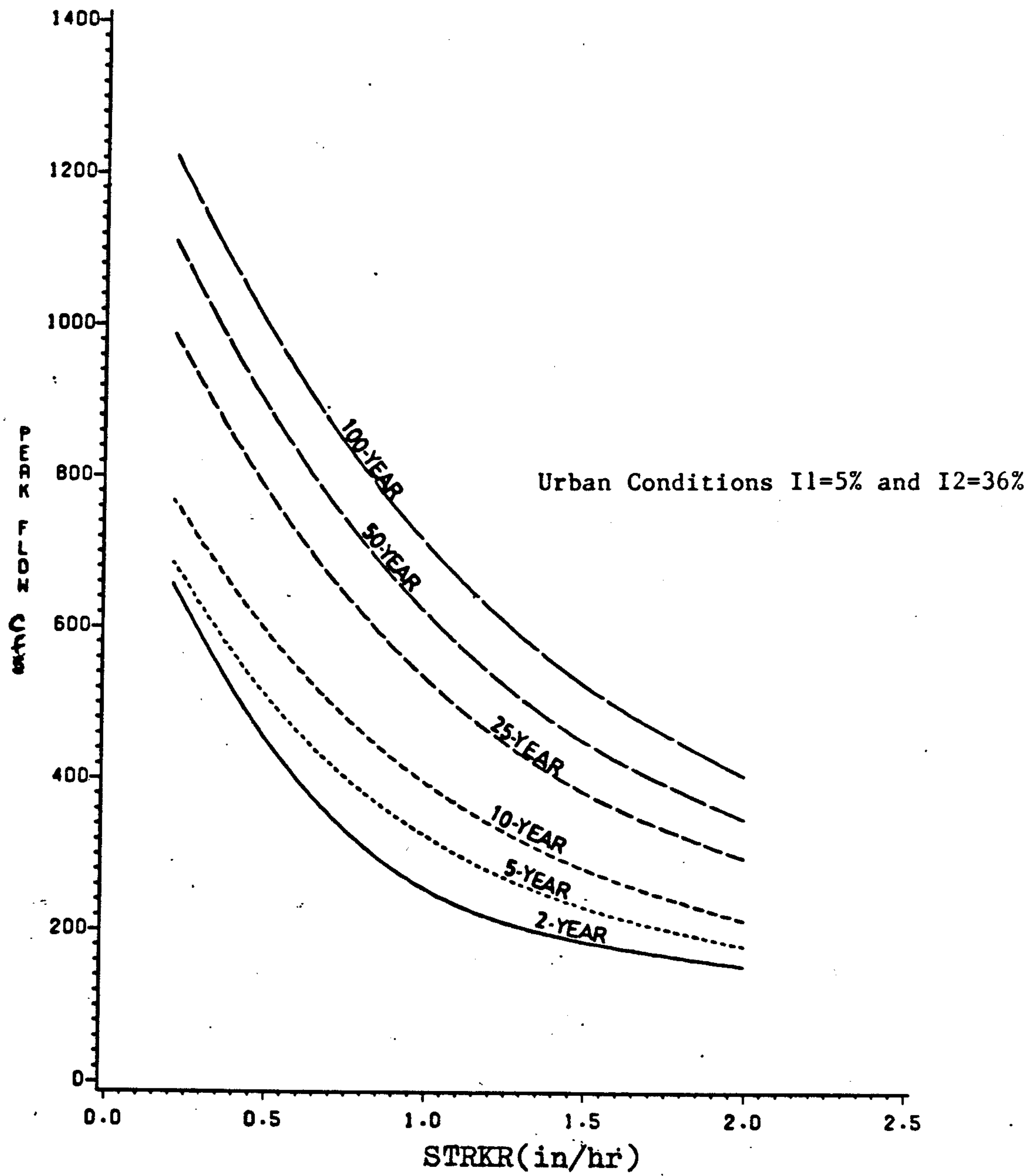

Figure 5.5 Peak Flow Variation with STRKR for Urban Condition I1=5% and I2=36%.

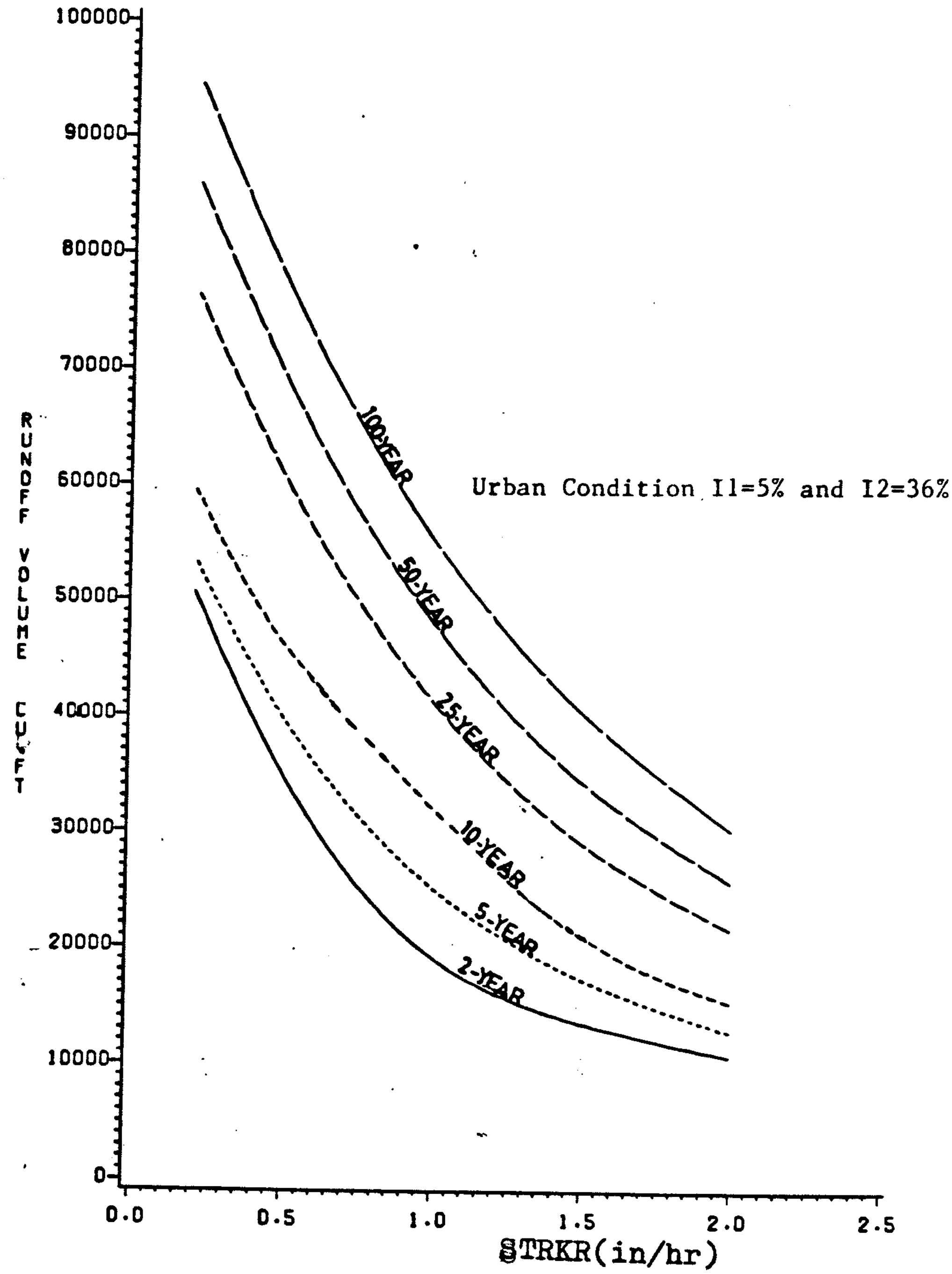

Figure 5.6 Runoff Volume Variation with STRKR for Urban Condition I1=5% and I2=36%.

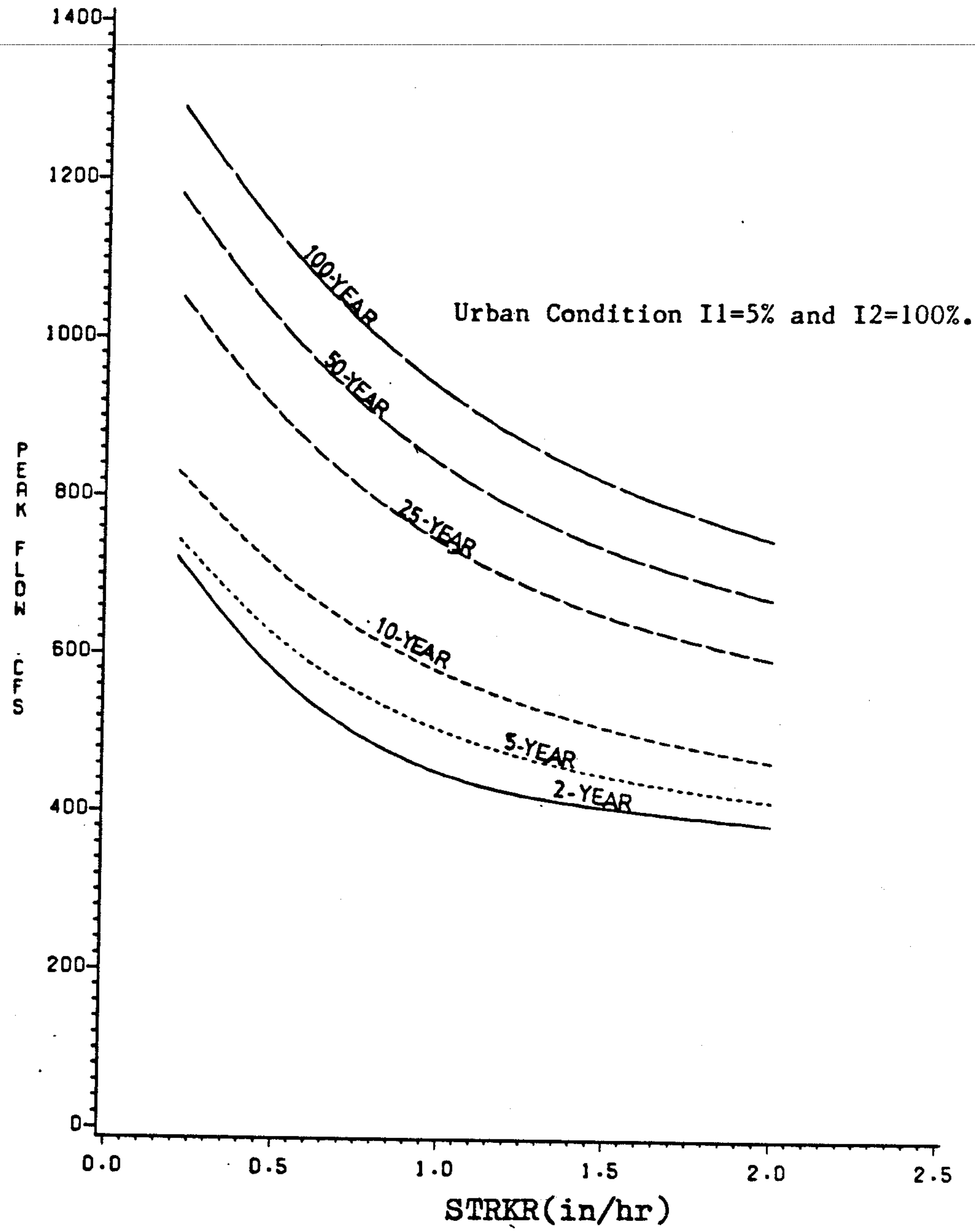

Figure 5.7 Peak Flow Variation with STRKR for Urban Condition I1=5% and I2=100%.

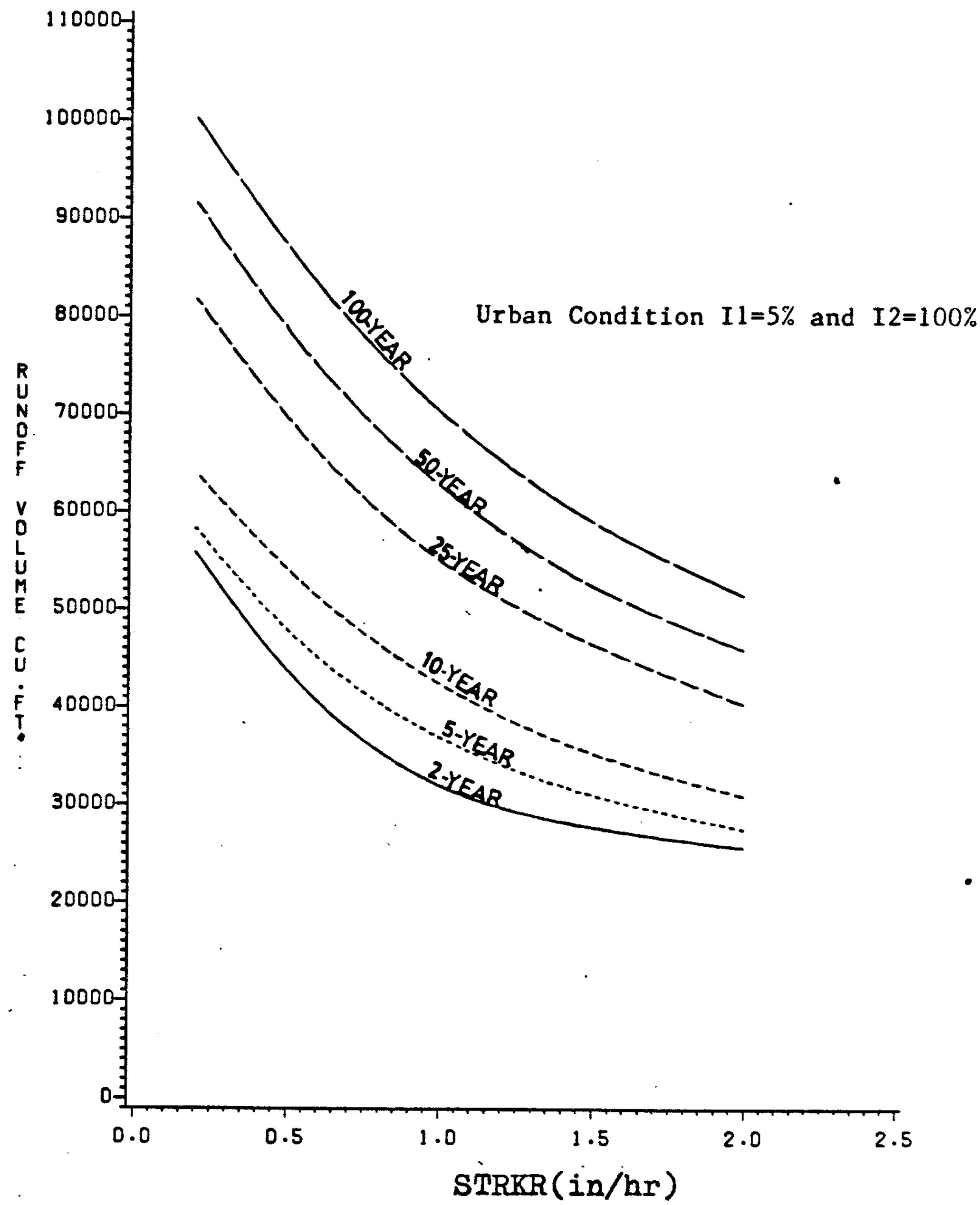

Figure 5.8 Runoff Volume Variation with STRKR for Urban Condition I1=5% and I2=100%.

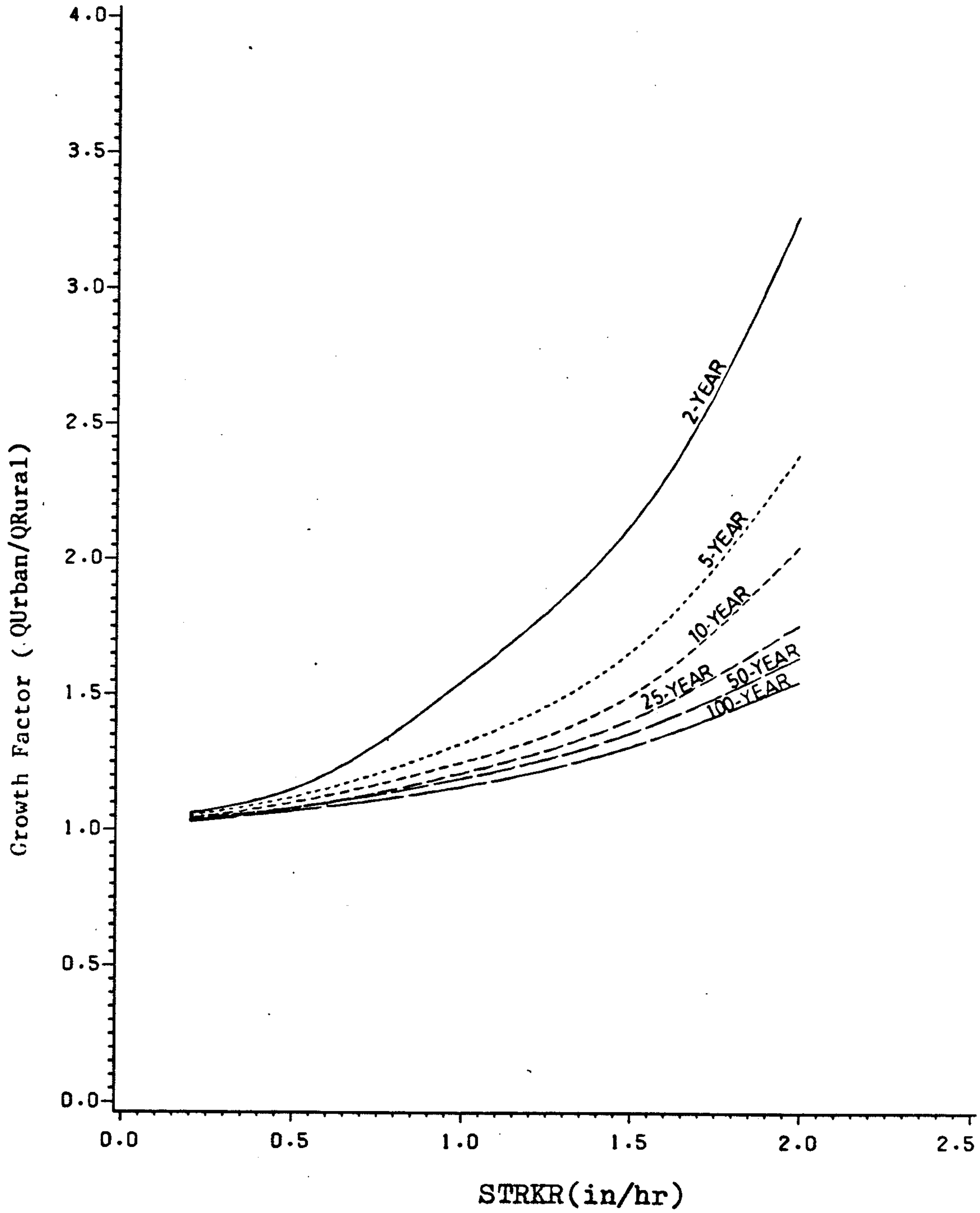

Figure 5.9 Variation of Growth Factors(QUrban/QRural) with STRKR for Urban Condition I1=5% and I2=36%.

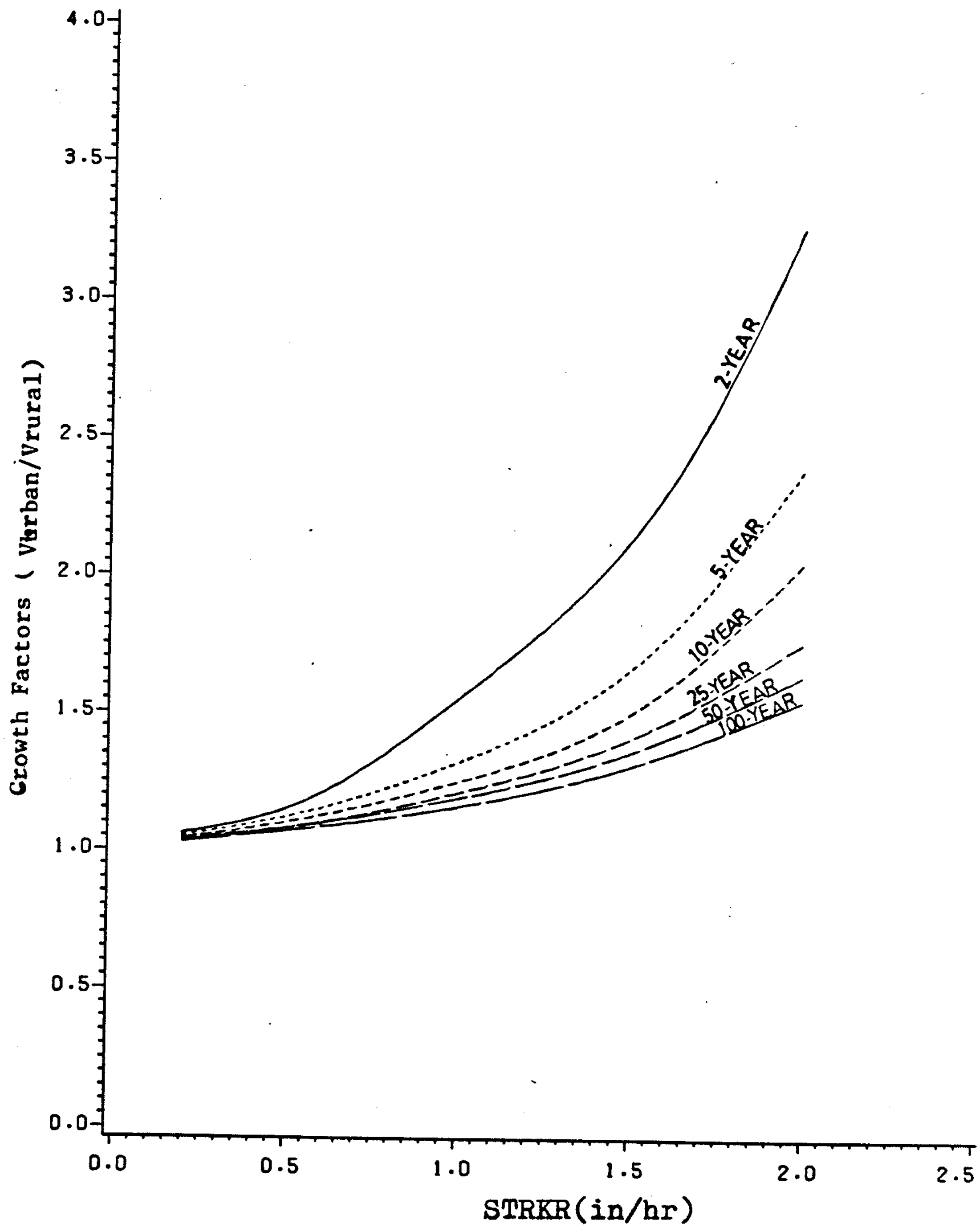

Figure 5.10 Variation of Growth Factors(Vurban/Vrural) with STRKR for Urban Condition I1=5% and I2=36%.

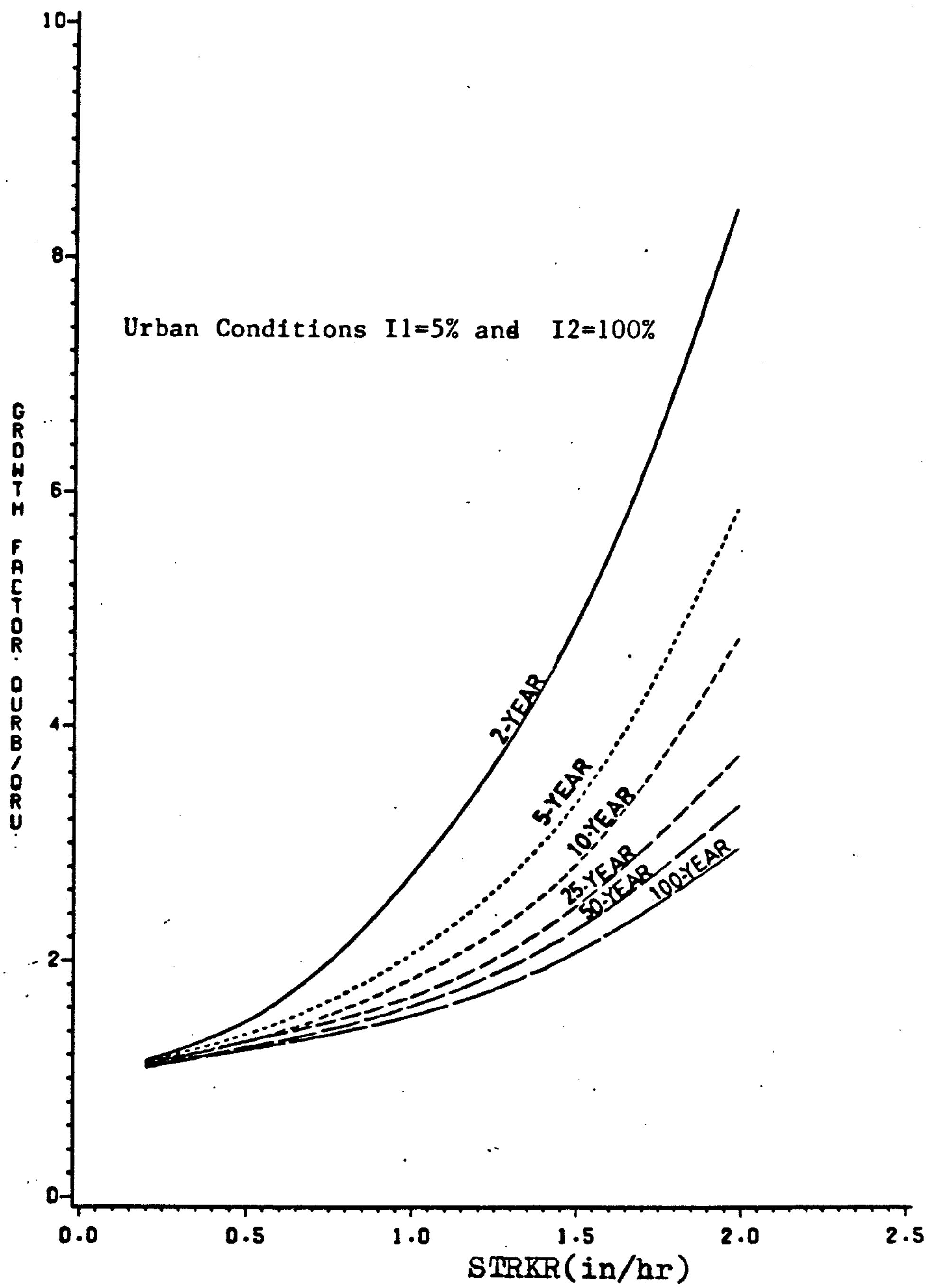

Figure 5.11 Variation of Growth Factors(Q_{Urban}/Q_{Rural}) with STRKR for Urban Condition I1=5% and I2=100%.

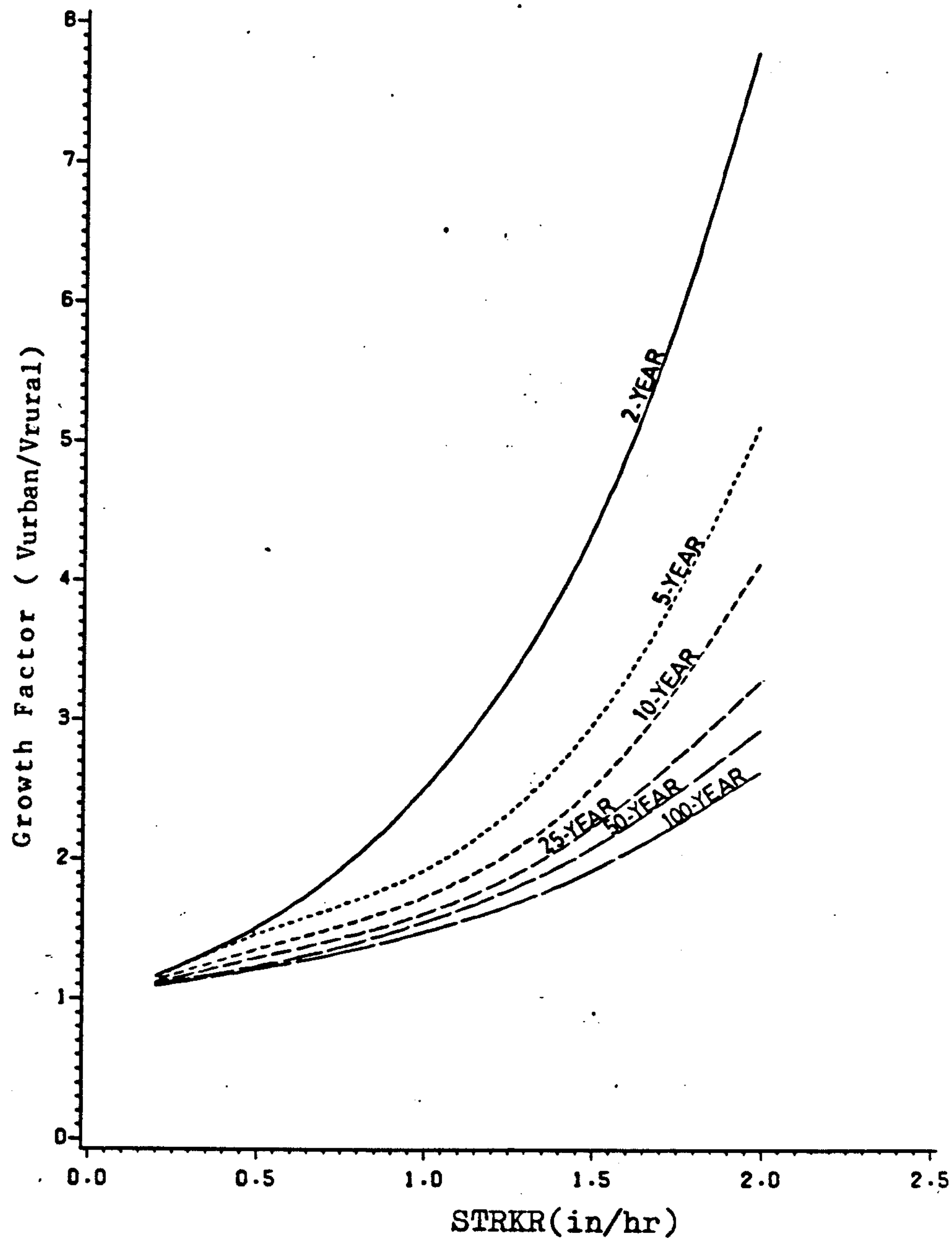

Figure 5.12 Variation of Growth Factors(Vurban/Vrural with STRKR for Urban Condition I1=5% and I2=100%.

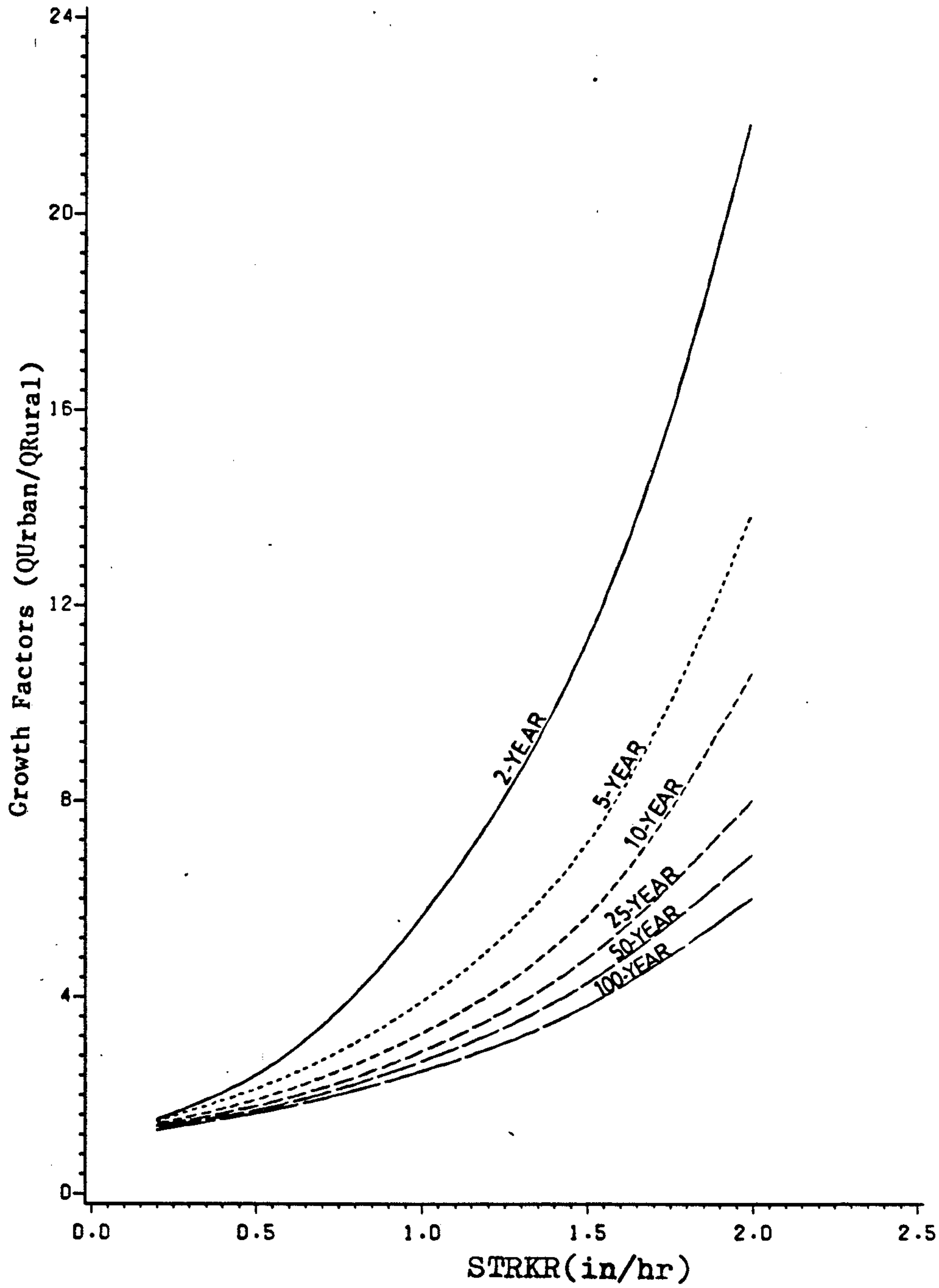

Figure 5.13 Variation of Growth Factors(QUrban/QRural) with STRKR for Urban Condition I1=I2=100%.

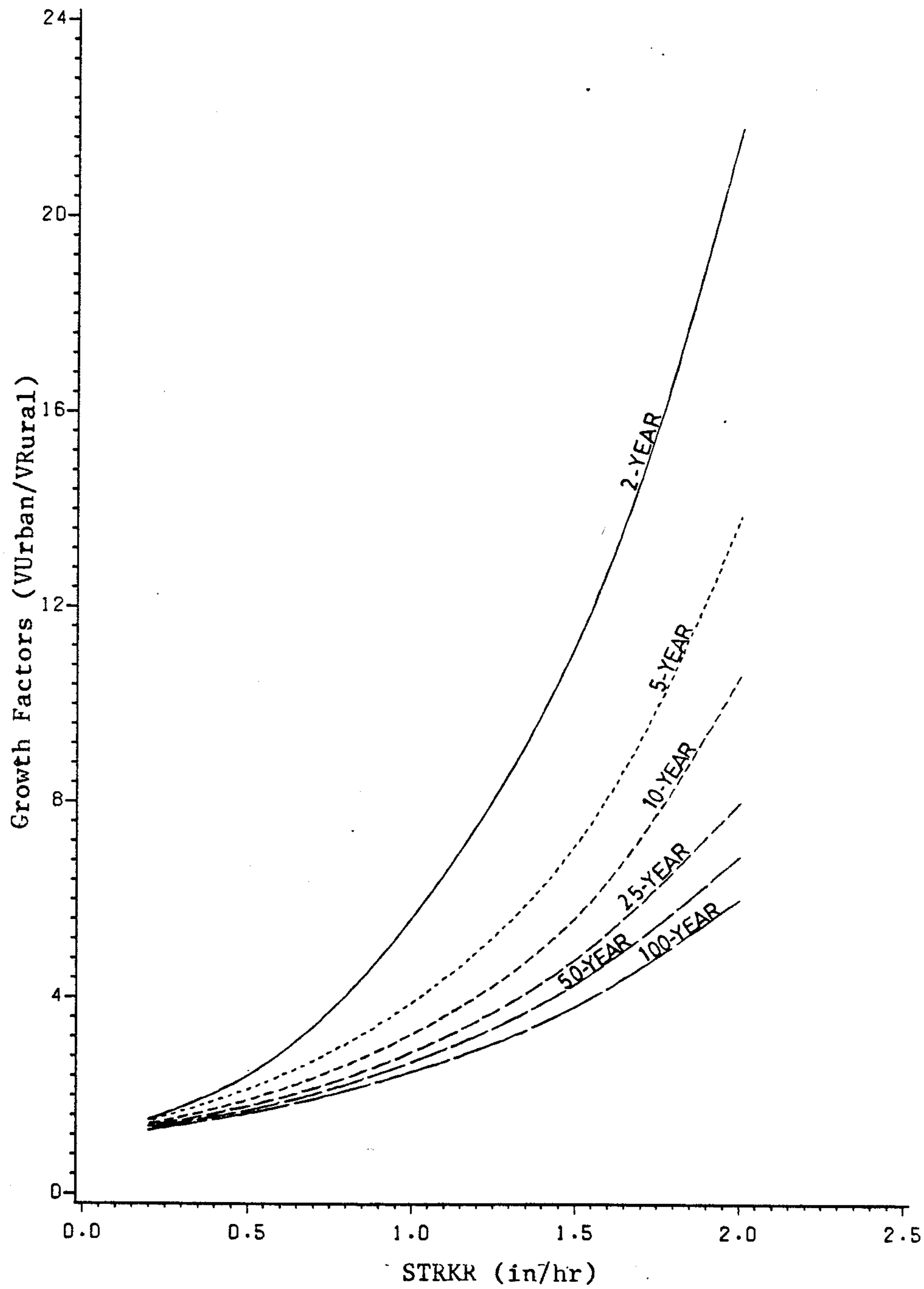

Figure 5.14 Variation of Growth Factors for Runoff with STRKR for Urban Condition I1=I2=100%

Chapter VI

DISCUSSION OF RESULTS

In general, the impacts of any development on catchment response are represented either as percentage growth above the rural peak flows and runoff volumes or in the form of growth factors which depict the ratio of urban runoff quantities to the rural runoff quantities. The other approach concerns plotting of flood frequencies of rural and urban peak flows on the same graph. In this study, comparisons were based on growth factors and plotting of frequencies on the same graphs.

The results of this research have shown some interesting facts about predicting urban catchment response to development. More especially the results have shown some basic facts about the ungauged watersheds.

First of all, at lower infiltration losses, the effects of development are less pronounced than at higher infiltration losses for similar levels of development. The results of Table 5.12 clearly show this trend. These results were obtained by simulating urban and rural runoff quantities. The results have shown higher growth factors for the more frequent floods than for the rare flood events. The aim of this section is therefore to discuss these findings and to to discuss the comparisons of these findings with those of other researchers.

The first interesting fact concerns the extremities of basin moisture conditions in a given time interval. The two extremities are for an initially dry soil surface condition and the other one is for an initially wet soil surface condition. In this study it was assumed that the value of 1.75 inches per hour as initial infiltration loss represents a dry condition while the value of 0.48 inches per hour represents a fairly wet soil condition. The results obtained by simulating runoff quantities at these two extremities shows that for a dry condition, the growth in runoff is higher than for a wet soil condition. This perhaps suggests that in humid areas, urbanization has less pronounced effects than for drier areas.

At STRKR value of 1.75 inches per hour, the growth factor is for the 2-year flood is 2.60, while for the 100-year flood, the growth factor is 1.43 where as at a STRKR value of 0.48 inches per hour, these growth factors are 1.14 and 1.06, respectively. The value of 1.75 inches per hour was based on experimental results of Watson(1969) as explained earlier. The STRKR value of 0.48 inches per hour was obtained through parameter optimization.

The results based on growth factors as already shown in Tables 5.12 through 5.19 for different levels of development at different initial loss rates show that urbanization has less effects on the rare floods than on the more frequent ones. The reason for this is that rare events the total precipitation is quite high and the initial loss and infiltration losses constitute a very small proportion of the available moisture. Changes in these values consequently affect the flows only very slightly. Further more, rare events are also associated with a 'worst' combination of parameters, i.e. high antecedent moisture giving small initial loss, which simulate closely the reduced initial loss and infiltration conditions resulting from the inrease in impervious proportion resulting from urbanization.

The results of increasing impervious areas in a watershed showed the same trend in growth of runoff quantities. Higher growths are associated with higher impervious areas.

There is however, a conflict between return period and growth factors. Espey and Winslow (1974) used Log-Pearson Type III method to derive flood frequencies for Whiteoak Bayou watershed in Houston, Texas for both rural and urban conditions as explained earlier. These findings are in complete disagreement with those found by other research results on the effects of urbanization on catchment response. The results of Espey and Winslow show higher growth factors for the rare floods than the frequent ones. There is no explanation for this trend in growth. The results of simulation of urban runoff quantities with HEC-1 and the frequencies defined by Log-Pearson Type III method by Espey and Winslow are shown in Figure 5.1. On this figure, the numbers 3 and 4 correspond to the flood frequency curves of Houston, Texas while 1, 2 ,5 and 6 are results obtained by using HEC-1 for Truro Creek. The right hand side of Figure 5.1 corresponds to the frequencies of Houston. There are two different scales to enable visual comparisons of the trend in growth of the runoff peaks from rural to urban conditions. According to the results of Whiteoak Bayou watershed, a 27 percent change of area to impervious cover resulted in a growth factor of about 2 for for the 100-year flood while for the 5-year flood, the growth factor was 1.60 showing that the higher the return period, the higher the growth factor which is in conflict with the other findings. However, the results of this research supports the findings of other researchers who demonstrated that urbanization has little effects on rare floods than on more frequent ones.

The results obtained by using statistically derived equations, however, supports the fact that urbanization has more pronounced effects on the rare floods than on the more frequent ones. This is evident in Figure 5.2 which shows the flood frequencies generated by HEC-1 at a STRKR value of 0.48 inches per hour and at 1.75 inches per hour as well as the flood frequencies derived by using statistically derived equations for both the rural and urban condition corresponding to the expected condition.

In Figure 5.2, numbers 1 and 2 correspond to the frequencies generated by HEC-1 at a STRKR value of 1.75 inches per hour while 3 and 4 correspond to the frequencies generated by statistical models and 5 and 6 correspond to the frequencies generated by HEC-1 at a STRKR value of 0.48 inches per hour. Examination of this figure reveals the diversity of results. There are two extremities of results. The lower extremity concerns the results obtained at a higher STRKR value while the upper extremity corresponds to a lower STRKR value and the results of the statistical models are in between these two extremities. This perhaps suggests the importance of a good estimate of the initial losses. The runoff quantities significantly depend on the initial losses

6.1 Limitations and Difficulties in Runoff Trends in Developing Basins.

6.1.1 Antecedent Moisture Conditions

Typically, the infiltration capacity varies from low during early spring to high in summer months (Brater, 1969). In this watershed, the highest precipitation amounts occur in the June-July months when the infiltration capacity is high. The fact that the infiltration capacity varies makes the whole process of predicting the effects of urbanization on catchment response difficult. The differing antecedent moisture conditions result in different runoff volumes for the same rainfall depths due to differing abstractions.

One of the limitations of predicting urban runoff quantities lies on the fact that continous monitoring of infiltration capacity of the soils in the basin is not possible for calibrating the predictive model. The question therefore arises as to what magnitude of values to use in assessment of runoff resulting from urbanization. In some models such as ILLUDAS, an account is made of the antecedent moisture so that the available soil storage capacity can be modified when simulating peak flows. Although this has proved good in estimating peak flows from urban areas, the prediction of frequencies seems to be quite difficult without the accurate knowledge of the basin moisture conditions. For these reasons perhaps a better approach would be to get the range of infiltration capacities for the basin during the period in question and to base the whole argument on the average values. It is the idea of the author that once the initial loss rates can be predicted based on recorded storms and runoff for the season in study, the average growth factors obtained would approximate the natural conditions of the basin.

Referring to the calibration results, it can be shown that based on the recorded storm events and streamflow data the initial losses in this basin would range between 0.70 inches per hour to as high as 1.2 inches per hour. It would thus be more appropriate to base the catchment response on an average of the two expected extreme values of initial loss rates between which the initial infiltration loss may be expected to range.

Considering the two values of STRKR of 0.70 inches per hour and 1.2 inches per hour, the values of peak flows corresponding to the various return periods for Truro Creek were obtained from HEC-1 and are shown in Table 6.1. The results shown in this Table also show the corresponding growth factors. These two extreme values for STRKR were then averaged and the results of this exercise are shown in Table 6.2. The results show the average growth factors of 38 percent for the 2-year flood, 26 percent for the 5-year flood, 21 percent for the 10-year flood, 18 percent for the 25-year flood, 16 percent for the 50-year flood and 14 percent for the 100-year flood.

Table 6.1 Derived Flood Frequency Values(cfs) at two

Two Different STRKR Values

Return Period	STRKR 1.20 in/hr			STRKR 0.70 in/hr		
(years)	Qru	Qurb	GF	Qru	Qurb	GF
2	133.0	227.0	1.71	300.0	372.0	1.24
5	201.0	288.0	1.43	371.0	434.0	1.17
10	261.0	350.0	1.34	450.0	514.0	1.14
25	370.0	477.0	1.29	608.0	681.0	1.12
50	443.0	552.0	1.25	705.0	782.0	1.11
100	526.0	637.0	1.21	805.0	883.0	1.10

Where Q ru and Q urb represent the rural and ur-
ban

peak flows
GF is the growth factor(Q urb/Q ru)

Table 6.2 Results of Average Runoff Values(cfs) and the

Corresponding Growth Factors

Reurn Period (years)	QPru (cfs)	Qpurb (cfs)	GF
2	217.0	300.0	1.38
5	286.0	361.0	1.26
10	356.0	432.0	1.21
25	489.0	579.0	1.18
50	574.0	667.0	1.16
100	666.0	760.0	1.14

Where Q Pru and Q Purb represent the rural and
urban peak flows
GF is the growth factor(QP urb/Q Pru)

Table 6.3 Comparison of Results with Statistical Results

	Rural Flows		Urban Flows	
Return Period (years)	HEC-1 (cfs)	STAT. (cfs)	HEC-1 (cfs)	STAT. (cfs)
2	217.0	----	300.0	-----
2.33	-----	249.0	-----	359.0
5	286.0	330.0	361.0	484.0
10	356.0	362.0	433.0	597.0
20	-----	413.0	-----	684.0
25	498.0	-----	579.0	-----
50	574.0	505.0	667.0	845.0
100	666.0	-----	760.0	-----

Where STAT refers to the values obtained by using statistical models.

Table 6.4 Comparison of Growth Factors

Return Period	Growth Factors	
(years)	HEC-1	STAT.
2	1.38	----
2.33	----	1.44
5	1.26	1.46
10	1.21	1.65
20	----	1.66
25	1.18	----
50	1.16	1.67
100	1.14	----

Where STAT refers to the values obtained by using statistical models.

Comparison of these results with those obtained by using statistical models show that for the rural conditions, the peak flows are in agreement while for the urban condition the statistical model shows higher flows hence higher growth factors. As pointed out earlier the statistical models tend to predict higher growth for the rare floods for reasons which are not quite explicit. However, with the above obtained growth factors and the corresponding quantities, it could perhaps be said that the values obtained will represent the actual values to be experienced by the basin.

7.2 Sensitivity Analysis Results

The results of sensitivity analysis on the degree of development represented by the impervious proportion, show that for increasing impervious cover, the effects are similar to the condition where the soil remains wet giving rise to more runoff quantities because of less abstractions as already shown in Tables 5.14 through 5.17.

Based on previous extreme values of STRKR, the effects of urbanization could be as high as 2.8 times for the 2-year flood and as high as 1.6 for the 100-year flood for 100 percent impervious cover of subbasin 2 keeping the impervious proportion of subbasin 1 at 5 percent. With the other return period growth factors being in between these values. For a hypothetical condition both subbasins could be 100 percent covered with impervious areas giving growth factors of 5.8

for the 2-year flood and 2.5 for the 100-year flood with the growth factors for other return periods lying in between the two values as pointed out earlier.

The latter results perhaps indicates the maximum change that could be expected from the complete development of Truro Creek watershed at Assiniboine golf course. However, due to the fact that an area of this watershed can not reach 100 percent impervious area coverage, these changes in urban runoff may never be experienced.

Chapter VII

CONCLUSIONS AND RECOMMENDATIONS

(i) This analysis has shown that for a generally wet watershed, where infiltration losses are fairly small, urbanization has less pronounced effects on both peak flows and runoff volumes.

(ii) The effects of extending Winnipeg airport will be most pronounced for the 2-year flood, 5-year flood, and the 10-year flood while this extention will have little effects on the rare floods.

(iii) Simulation of urbanization effects using statistically derived equations, however, has a complete reverse of the effects. The more rare floods show greater growth than the frequent ones.

(iv) In ungauged watersheds, it is very difficult to give absolute numbers for the urbanization effects due to differing moisture levels in the basin which affects the initial losses. For these basins a range of expected growth is desired based on range of expected initial losses.

(v) Any further development on sub-basin 2 leading to almost 100 percent of impervious cover, will result in twice as much flows for the 2-year flood and up to 50 percent growth for the 100-year flood.

(vi) The extent to which urbanization affects runoff quantities depends on soil moisture conditions of the basin during period of precipitation.

Recommendations

Perhaps the most logical approach for predicting effects of urbanization is to monitor the basin infiltration capacity during the study period and to base the analysis on field results.

Apart from this analysis, it is recommended to study the effects of of drainage density in so far as the sewer installation affects the urban cathment response because this is the only possible way of reducing basin response time.

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Appendix A

RAINFALL INTENSITY-DURATION DATA

Table A-1 Rainfall Intensity Duration Frequency Data for Winnipeg International Airport.

Atmospheric Environment Service

Rainfall Intensity, Duration, Frequency Values

Prepared by the Hydrometeorological Division, Canada Climate.

Climate	Sta.Name	Year	5 min. (mm)	10 min. (mm)	15 min. (mm)	30 min (mm)	1 Hr. (mm)	2 Hr. (mm)	6 Hr. (mm)	12 Hr. (mm)	24 Hr. (mm)
5023222	Winnipeg A	1944	14.7	18.8	29.1	37.3	39.9	41.7	45.0	49.0	75.9
5023222	Winnipeg A	1945	13.7	15.5	19.8	27.2	31.5	33.8	42.9	45.2	48.3
5023222	Winnipeg A	1946	5.6	6.1	7.1	8.6	10.2	14.2	20.6	21.3	25.1
5023222	Winnipeg A	1947	8.4	11.4	13.5	19.6	22.1	28.2	45.0	45.0	45.2
5023222	Winnipeg A	1948	9.1	11.7	13.2	17.3	30.5	33.3	33.3	36.6	37.3
5023222	Winnipeg A	1951	5.8	6.1	9.1	15.7	16.0	17.9	21.1	21.1	36.3
5023222	Winnipeg A	1952	9.1	17.8	22.4	28.4	29.2	43.7	43.7	43.7	43.7
5023222	Winnipeg A	1953	10.2	15.5	19.3	24.6	30.2	31.2	41.7	43.4	43.4
5023222	Winnipeg A	1954	9.7	12.2	12.7	16.0	18.3	21.1	25.9	28.7	35.8
5023222	Winnipeg A	1956	6.1	9.7	13.5	17.8	18.1	18.3	29.2	33.8	55.1
5023222	Winnipeg A	1958	8.9	10.9	15.7	17.8	18.5	19.8	35.8	50.3	67.1
5023222	Winnipeg A	1959	9.4	15.5	18.8	19.6	19.6	22.4	37.6	42.2	44.7
5023222	Winnipeg A	1960	4.3	5.60	7.40	8.1	8.1	11.7	27.9	32.8	41.1
5023222	Winnipeg A	1961	6.1	8.60	10.20	13.2	17.3	19.8	21.8	26.4	34.0
5023222	Winnipeg A	1962	11.2	14.70	19.60	27.6	35.8	56.1	82.3	83.1	83.8
5023222	Winnipeg A	1963	7.6	11.90	16.50	16.5	21.1	24.1	38.9	50.5	52.8
5023222	Winnipeg A	1964	6.9	12.20	17.50	22.4	33.5	36.8	38.1	52.3	59.2
5023222	Winnipeg A	1965	6.1	9.10	11.70	14.5	15.7	16.8	17.8	21.1	30.2
5023222	Winnipeg A	1966	9.1	12.7	15.2	22.6	37.8	4.0	68.3	73.2	76.2
5023222	Winnipeg A	1967	12.2	24.1	25.9	31.7	33.0	57.9	63.2	63.5	63.5
5023222	Winnipeg A	1968	17.8	24.6	25.3	39.4	39.4	39.4	48.3	61.2	84.3
5023222	Winnipeg A	1969	7.1	10.4	12.7	15.2	21.8	23.4	25.4	39.1	49.3
5023222	Winnipeg A	1970	11.2	20.8	29.0	37.8	41.1	49.8	54.9	60.5	62.2
5023222	Winnipeg A	1971	4.6	6.1	8.4	11.7	14.5	19.8	25.4	29.0	31.0
5023222	Winnipeg A	1972	9.1	16.5	20.3	35.6	35.6	35.8	35.8	35.8	35.8
5023222	Winnipeg A	1974	9.4	16.3	18.3	25.1	28.7	33.0	37.1	38.9	55.4
5023222	Winnipeg A	1973	6.3	10.4	14.5	19.8	29.7	40.4	45.7	45.7	45.7
5023222	Winnipeg A	1975	9.4	14.5	17.8	22.6	27.9	27.9	44.7	53.8	54.4
5023222	Winnipeg A	1976	15.0	15.7	18.0	21.0	22.1	24.1	26.2	33.3	42.7
5023222	Winnipeg A	1977	7.4	12.4	15.2	19.8	21.6	32.5	50.3	57.7	61.7
5023222	Winnipeg A	1978	10.6	17.6	21.6	24.5	28.0	41.7	52.6	52.6	60.4
5023222	Winnipeg A	1979	10.6	19.1	25.4	36.3	39.3	39.8	40.7	40.7	40.7
MEAN EXTREME			9.1	13.6	17.2	22.4	26.1	31.9	39.6	44.1	50.7
STANDARD DEVIATION			3.1	4.6	6.3	8.3	9.0	13.0	14.3	14.5	15.3
YEARS OF RECORD			32	32	32	32	32	32	32	32	32

RETURN PERIOD YEARS **RAINFALL AMOUNTS (MM)**

RETURN PERIOD YEARS	5 min.	10 min.	15 min.	30 min	1 Hr.	2 Hr.	6 Hr.	12 Hr.	24 Hr.
2	8.7	12.8	16.3	21.2	24.8	29.9	37.4	41.9	48.4
5	11.8	17.7	22.6	29.6	33.9	43.0	51.9	56.6	63.8
10	13.9	21.0	26.9	35.1	39.9	51.7	61.5	66.3	74.0
25	16.5	25.1	32.2	42.2	47.6	62.7	73.6	78.4	87.0
50	18.4	28.1	36.2	47.4	53.3	70.0	82.6	87.7	96.6
100	20.3	31.1	40.1	52.6	58.9	79.0	91.5	96.7	106.1

PRETURN PERIOD RAINFALL AMOUNTS EXPRESSED AS MM/HR WITH 50% CONFIDENCE LIMITS

	5 MIN	10 MIN.	15 MIN.	30 MIN.	1 HR	2 HR	6 HR	12 HR	24 HR
2	104.1+/- 4.1	77.0+/- 3.2	65.0+/- 2.8	42.3+/- 1.8	24.75+/- .99	14.95+/- .71	6.24+/- .24	3.49+/- .13	2.02+/ .07
5	141.5+/- 7.4	106.3+/- 5.8	98.6+/- 5.6	59.1+/-3.5	33.9+/- 1.80	21.51+/- 1.29	8.65+/- .47	4.75+/- .24	2.66+/- .13
10	166.3+/- 10.2	125.8+/- 8.0	107.5+/-6.9	70.3+/-4.6	39.95+/- 2.48	25.87+/-1.76	10.25+/-.66	5.52+/-.33	3.09+/- .17
25	197.6+/- 13.9	150.3+/-10.9	128.9+/-9.5	84.4+/-6.2	47.59+/-3.39	31.36+/-2.43	12.27+/-.89	6.55+/-.45	3.62+/- .24
50	220.8+/- 16.7	168.5+/-13.1	144.7+/-11.4	94.8+/-7.5	53.26+/-4.69	35.43+/-2.93	13.75+/-1.08	7.31+/-.55	4.02+/-.29
100	243.9+/-19.5	186.6+/-15.3	160.5+/-13.5	105.2+/-8.8	58.9+/-4.77	39.48+/-3.42	15.25+/-1.26	8.06+/-.64	4.42+/-.34

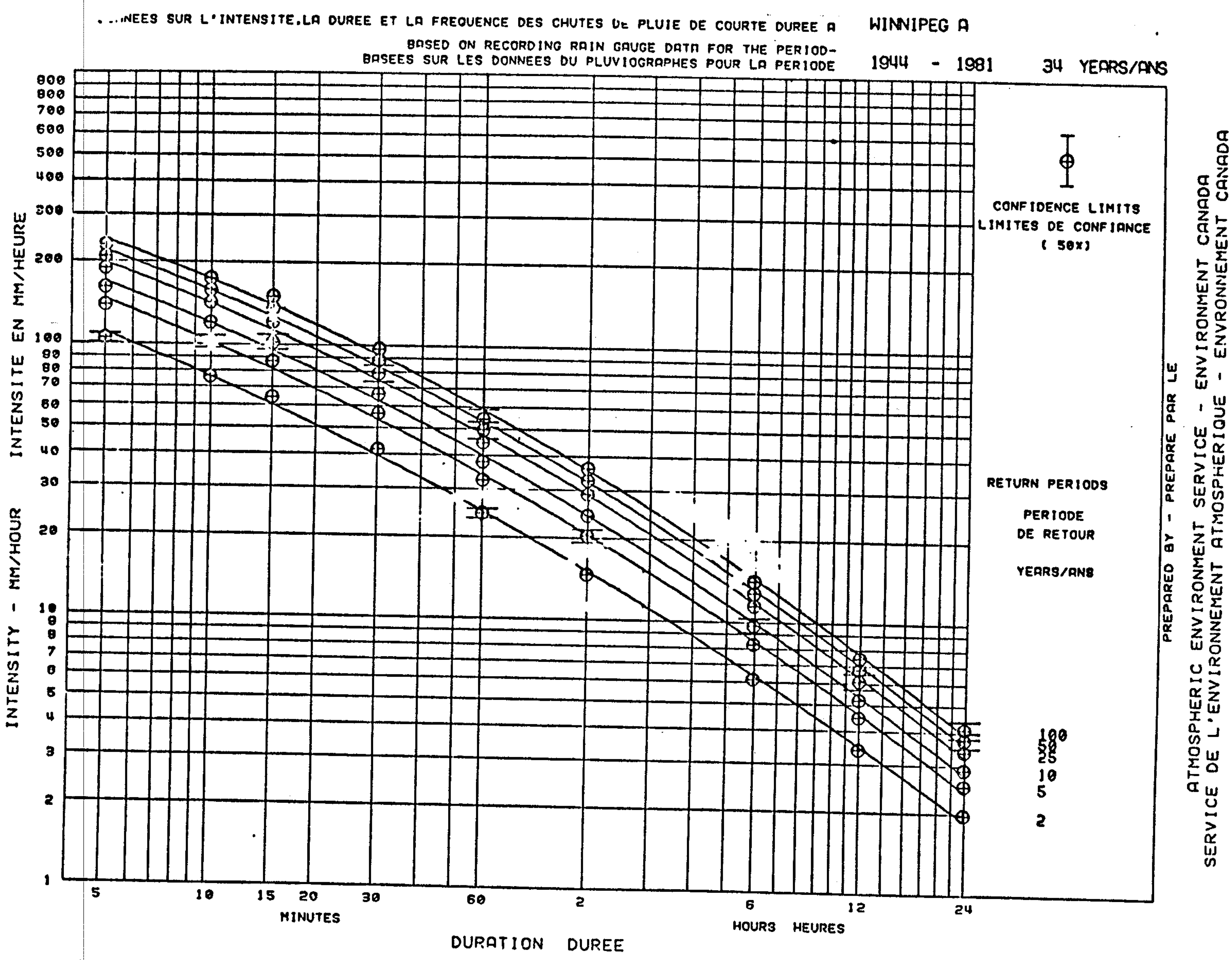


Figure A-1 Rainfall Intensity Duration Curves for Winnipeg International Airport.

Appendix B

HYDROGRAPH ANALYSIS GRAPHS

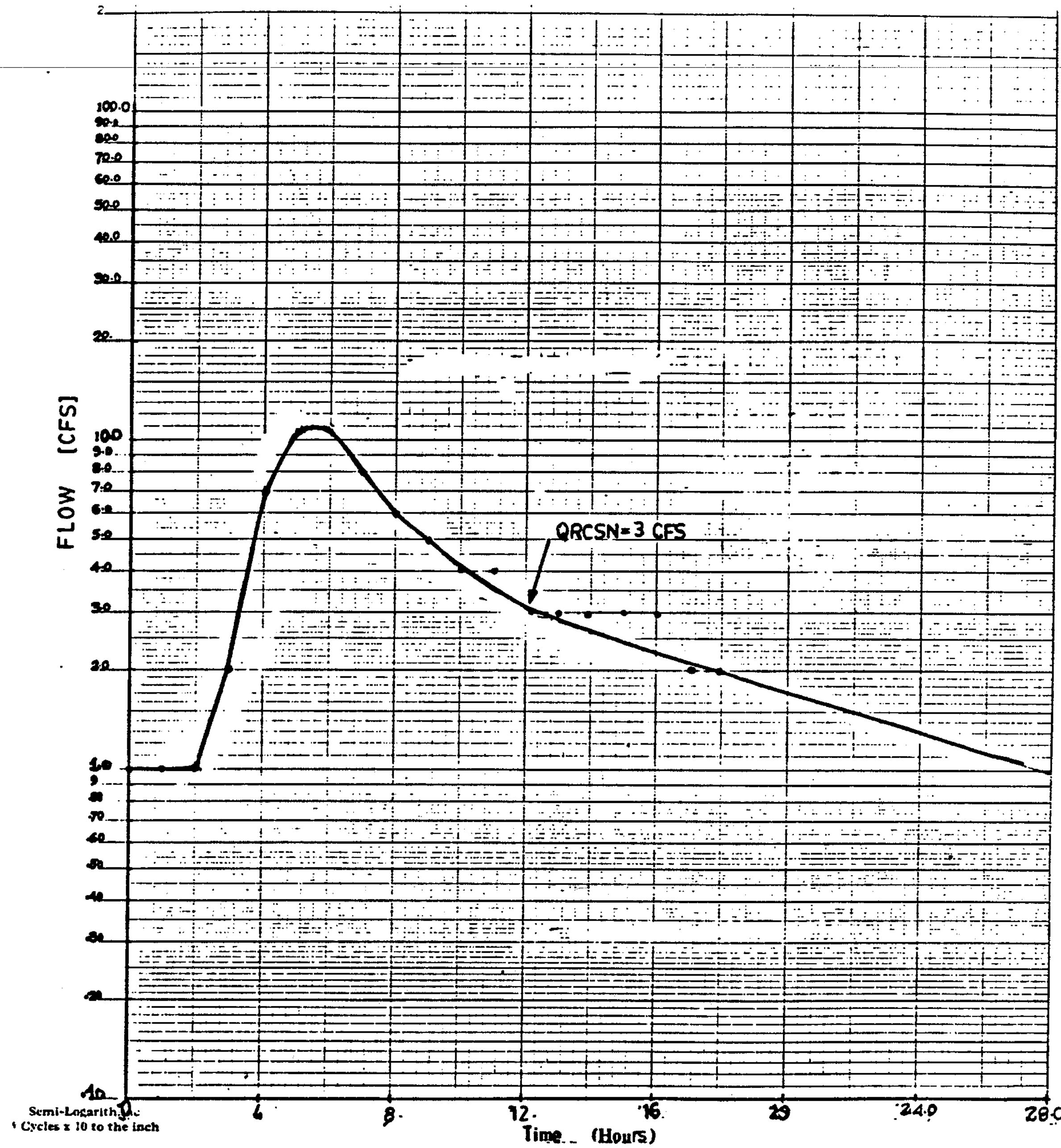


Figure B-1 Runoff Hydrograph for May 20, 1984 Storm Event.

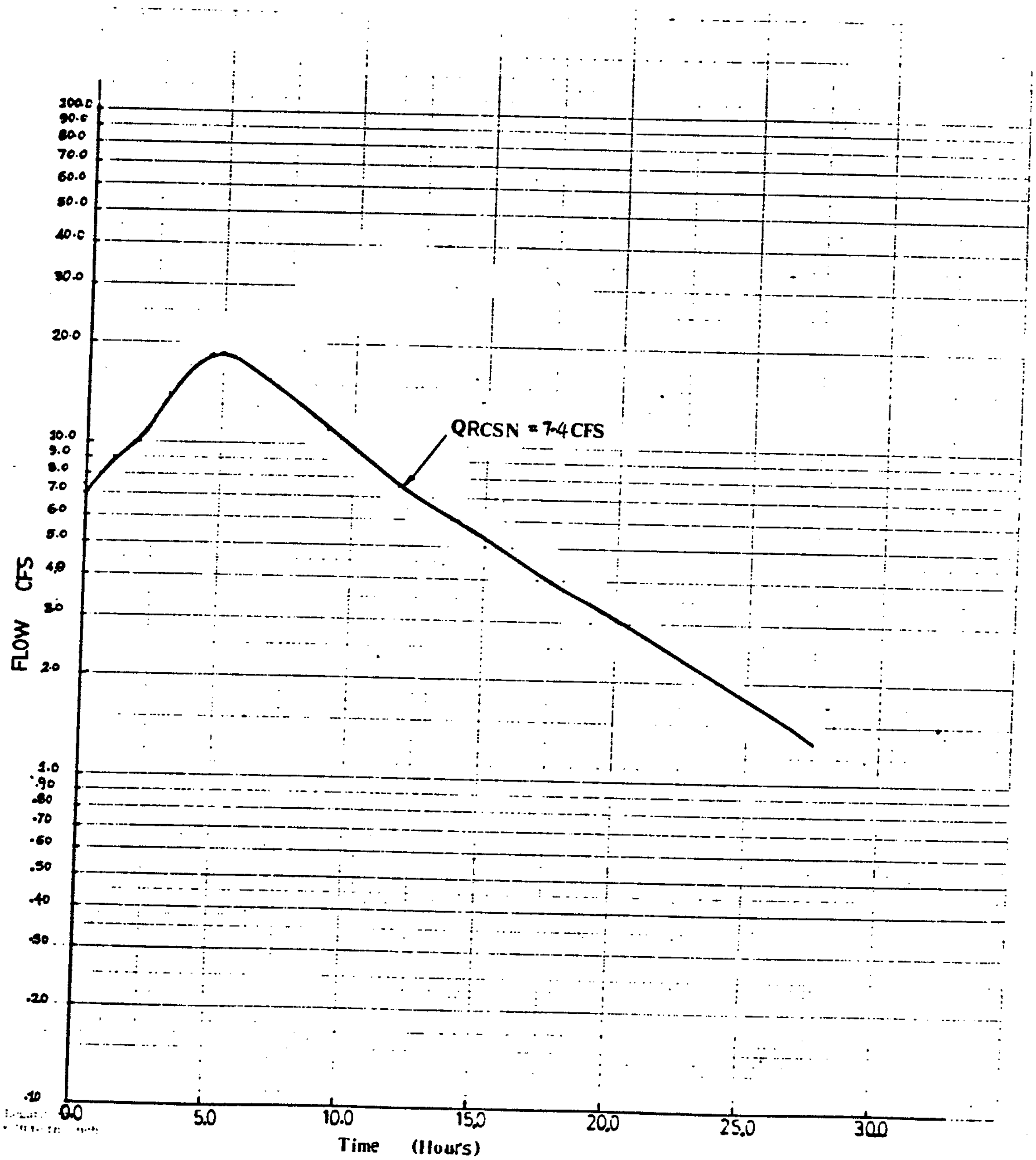


Figure B-2 Runoff Hydrograph for June 12, 1984 Storm Event.

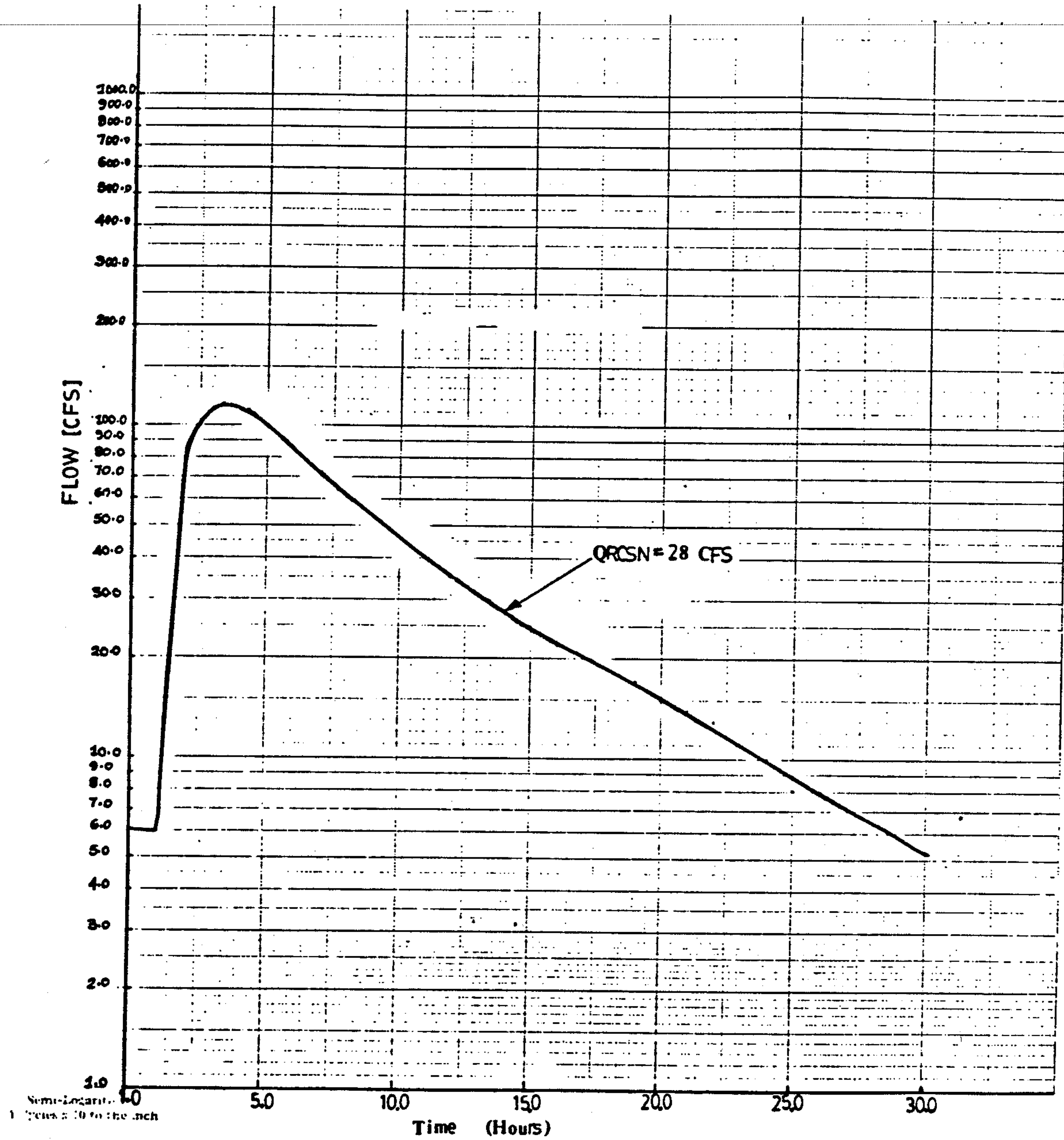


Figure B-3 Runoff Hydrograph for June 16, 1984 Storm Event.

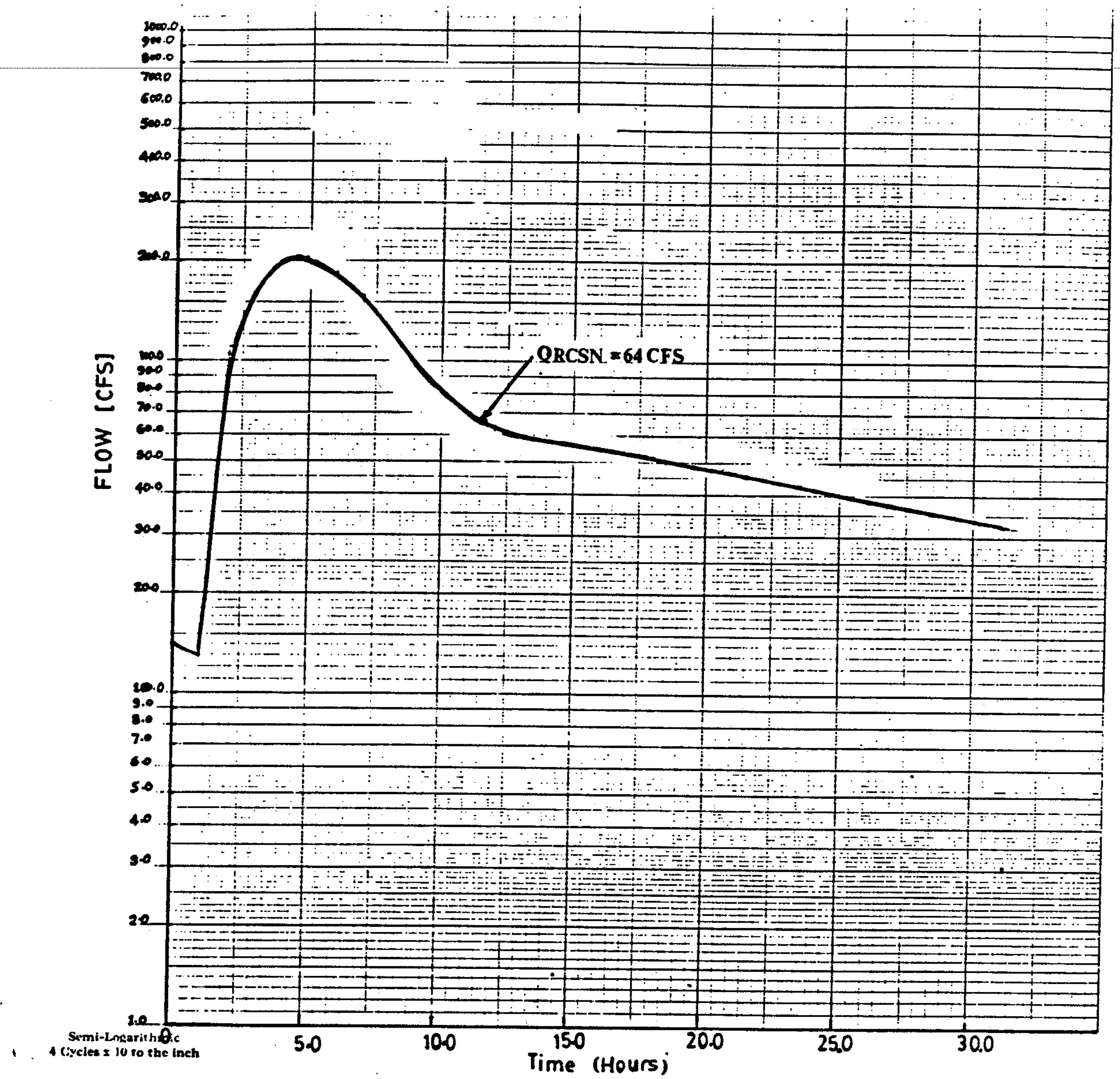


Figure B-4 Runoff Hydrograph for June 21, 1984 Storm Event

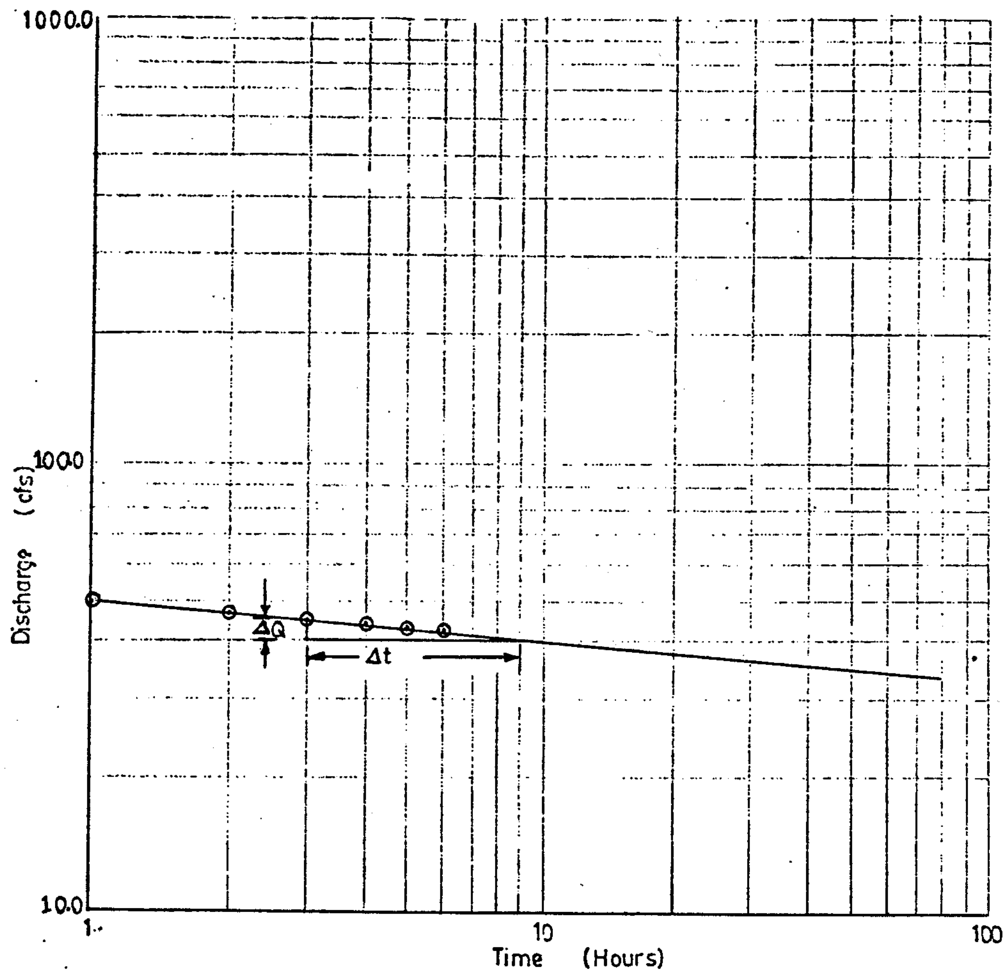


Figure B-5 Log-Log Graph of Recession Flows for June 21, 1984 Storm Event.

Appendix C

MODEL VERIFICATION AND VALIDATION TABLES

Table C-1 Comparison of Observed and Computed Flows for May 20, 1984 Storm Event.

OVN

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	95	0	0	2	1	0

COMPARISON OF COMPUTED AND OBSERVED FLOWS AT STATION 2

INTERVAL	COMPUTED	OBSERVED	RESIDUAL	SUM COMP	SUM OBSD	SUM RESDL
1	0.	0.	-0.	0.	0.	-0.
2	1.	0.	-1.	1.	0.	-1.
3	1.	0.	-1.	2.	0.	-2.
4	2.	0.	-2.	4.	0.	-4.
5	2.	1.	-1.	6.	1.	-5.
6	3.	1.	-2.	9.	2.	-7.
7	5.	1.	-4.	14.	3.	-11.
8	7.	2.	-5.	21.	5.	-16.
9	9.	7.	-2.	29.	12.	-17.
10	10.	11.	1.	39.	23.	-16.
11	9.	11.	2.	48.	34.	-14.
12	8.	8.	-0.	57.	42.	-15.
13	7.	6.	-1.	63.	48.	-15.
14	5.	5.	-0.	68.	53.	-15.
15	4.	4.	-0.	73.	57.	-16.
16	4.	4.	0.	77.	61.	-16.
17	3.	3.	-0.	80.	64.	-16.
18	3.	3.	0.	83.	67.	-16.
19	3.	3.	0.	85.	70.	-15.
20	3.	3.	0.	88.	73.	-15.
21	3.	3.	0.	91.	76.	-15.
22	3.	2.	-1.	93.	78.	-15.
23	2.	2.	-0.	95.	80.	-15.

**********	**********		**********			**********		**********
	ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	
	2	95	0	0	2	1	0	

********************** COMPARISON OF COMPUTED AND OBSERVED FLOWS AT STATION 2 **********************

INTERVAL	COMPUTED	OBSERVED	RESIDUAL	SUM COMP	SUM OBSD	SUM RESDL
1	7.	8.	1.	7.	8.	1.
2	11.	24.	13.	17.	32.	15.
3	15.	21.	6.	33.	53.	20.
4	17.	19.	2.	50.	72.	22.
5	16.	16.	0.	65.	88.	23.
6	14.	15.	1.	79.	103.	24.
7	12.	14.	2.	91.	117.	26.
8	18.	13.	-5.	109.	130.	21.
9	58.	103.	45.	167.	233.	66.
10	135.	167.	32.	301.	400.	99.
11	204.	207.	3.	505.	607.	102.
12	224.	203.	-21.	730.	810.	80.
13	207.	185.	-22.	937.	995.	58.
14	176.	157.	-19	1113.	1152.	39.
15	137.	122.	-15.	1250.	1274.	24.
16	105.	96.	-9.	1355.	1370.	15.
17	84.	77.	-7.	1438.	1447.	9.
18	71.	69.	-2.	1510.	1516.	6.
19	67.	65.	-2.	1577.	1581.	4.
20	66.	62.	-4.	1643.	1643.	0.
21	63.	60.	-3.	1706.	1703.	-3.
22	61.	58.	-3.	1767.	1761.	-6.
23	59.	55.	-4.	1826.	1816.	-10.
24	57.	53.	-4.	1883.	1869.	-14.
25	55.	50.	-5.	1938.	1919.	-19.
26	53.	49.	-4.	1991.	1968.	-23.
27	51.	47.	-4.	2042.	2015.	-27.
28	50.	46.	-4.	2092.	2061.	-31.
29	48.	46.	-2.	2140.	2107.	-33.
30	46.	44.	-2.	2186.	2151.	-35.
31	45.	42.	-3.	2230.	2193.	-37.

Table C-2 Comparison of Computed and Observed Flows for June 21, 1984 Storm Event.

********** ********** ********** ********** **********

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	95	0	0	2	1	0

*************************** COMPARISON OF COMPUTED AND OBSERVED FLOWS AT STATION 2 *******************

INTERVAL	COMPUTED	OBSERVED	RESIDUAL	SUM COMP	SUM OBSD	SUM RESDL
1	1.	1.	-0.	1.	1.	-0.
2	2.	1.	-1.	3.	2.	-1.
3	3.	1.	-2.	5.	3.	-2.
4	4.	2.	-2.	9.	5.	-4.
5	4.	2.	-2.	13.	7.	-6.
6	5.	2.	-3.	18.	9.	-9.
7	4.	2.	-2.	22.	11.	-11.
8	4.	2.	-2.	25.	13.	-12.
9	4.	3.	-1.	29.	16.	-13.
10	5.	7.	2.	34.	23.	-11.
11	6.	9.	3.	40.	32.	-8.
12	7.	10.	3.	47.	42.	-5.
13	9.	14.	5.	55.	56.	1.
14	11.	17.	6.	66.	73.	7.
15	12.	19.	7.	79.	92.	13.
16	12.	17.	5.	91.	109.	18.
17	11.	15.	4.	101.	124.	23.
18	9.	12.	3.	110.	136.	26.
19	7.	11.	4.	117.	147.	30.
20	6.	9.	3.	123.	156.	33.
21	5.	8.	3.	127.	164.	37.
22	4.	8.	4.	132.	172.	40.
23	4.	7.	3.	135.	179.	44.
24	4.	6.	2.	139.	185.	46.
25	3.	5.	2.	142.	190.	48.
26	3.	5.	2.	146.	195.	49.
27	3.	4.	1.	149.	199.	50.
28	3.	4.	1.	152.	203.	51.
29	3.	3.	-0.	155.	206.	51.
30	3.	3.	0.	158.	209.	51.
31	3.	3.	0.	161.	212.	51.
32	3.	2.	-1.	164.	214.	50.
33	3.	2.	-1.	166.	216.	50.
34	3.	2.	-1.	169.	218.	49.

Table C-3 Comparison of Computed and Observed FLOws for June 16, 1984 Storm Event.

********** ********** ********** ********** **********

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	95	0	0	2	1	0

********************* COMPARISON OF COMPUTED AND OBSERVED FLOWS AT STATION 2 ***********************

INTERVAL	COMPUTED	OBSERVED	RESIDUAL	SUM COMP	SUM OBSD	SUM RESDL
1	2.	2.	-0.	2.	2.	-0.
2	4.	3.	-1.	6.	5.	-1.
3	7.	9.	2.	13.	14.	1.
4	10.	10.	0.	23.	24.	1.
5	12.	11.	-1.	35.	35.	-0.
6	12.	12.	-0.	47.	47.	-0.
7	13.	14.	1.	61.	61.	0.
8	16.	26.	10.	77.	87.	10.
9	20.	29.	9.	97.	116.	19.
10	22.	24.	2.	119.	140.	21.
11	21.	20.	-1.	140.	160.	20.
12	18.	15.	-3.	158.	175.	17.
13	15.	13.	-2.	173.	188.	15.
14	13.	11.	-2.	186.	199.	13.
15	11.	11.	-0.	197.	210.	13.
16	10.	11.	1.	208.	221.	13.
17	10.	13.	3.	217.	234.	17.
18	10.	16.	6.	227.	250.	23.
19	11.	17.	6.	238.	267.	29.
20	11.	16.	5.	249.	283.	34.
21	10.	15.	5.	259.	298.	39.
22	9.	13.	4.	268.	311.	43.
23	8.	12.	4.	277.	323.	46.
24	8.	10.	2.	284.	333.	49.
25	7.	9.	2.	291.	342.	51.
26	7.	8.	1.	298.	350.	52.
27	6.	8.	2.	304.	358.	54.
28	6.	7.	1.	311.	365.	54.
29	6.	6.	0.	316.	371.	55.
30	6.	6.	0.	322.	377.	55.
31	6.	5.	-1.	328.	382.	54.
32	5.	5.	-0.	333.	387.	54.
33	5.	4.	-1.	338.	391.	53.
34	5.	4.	-1.	343.	395.	52.
35	5.	4.	-1.	348.	399.	51.
36	5.	4.	-1.	353.	403.	50.
37	4.	3.	-1.	357.	406.	49.
38	4.	3.	-1.	361.	409.	48.
39	4.	3.	-1.	366.	412.	46.

Table C-4 Comparison of Computed and Observed Flows for June 8, 1984 Storm Event.

Appendix D

EXAMPLE OF DATA INPUT AND COMPUTATION OUTPUT

```
*****************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
*****************************

              TRURO CREEK NEAR ASSINIBOINE GOLF COURSE
           STREAM ROUTING COMPUTATION MUSKINGUM METHOD
                USING      MUSKINGUM METHOD

                        JOB SPECIFICATION
           NQ   NHR  NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
          150     0     5     0     0     0     0     2     4     0
                             JOPER   NWT
                                 5     0

**********   **********   **********   **********   **********

                     SUB-AREA RUNOFF COMPUTATION
              ISTAQ  ICOMP  IECON  ITAPE   JPLT   JPRT  INAME
                  1      0      0      0      2      1      0

                            HYDROGRAPH DATA
   IHYDG   IUHG  TAREA   SNAP  TRSDA  TRSPC  RATIO  ISNOW  ISAME  LOCAL
       0      0   4.44    0.0    0.0    0.0    0.0      0      0      0

                              PRECIP DATA
                      NP  STORM    DAJ    DAK
                      37   1.99    0.0    0.0
                            PRECIP PATTERN
   0.01   0.01   0.01   0.01   0.01   0.02   0.02   0.02   0.03   0.05
   0.08   0.16   0.54   0.29   0.16   0.10   0.07   0.06   0.05   0.04
   0.03   0.03   0.02   0.02   0.02   0.02   0.01   0.01   0.01   0.01
   0.01   0.01   0.01   0.01   0.01   0.01   0.01

                               LOSS DATA
   STRKR  DLTKR  RTIOL  ERAIN  STRKS  RTIOK  STRTL  CNSTL  ALSMX  RTIMP
    3.00   1.07   2.27   0.53    0.0   1.00    0.0    0.0    0.0   0.05

                          UNIT HYDROGRAPH DATA
                      TC=  3.10    R=  3.65   NTA=  0

                             RECESSION DATA
              STRTQ=    0.0    QRCSN=   -0.31   RTIOR= 1.42

   UNIT HYDROGRAPH100 END-OF-PERIOD ORDINATES, LAG=   3.03 HOURS, CP= 0.64  VOL= 0.64
      2.      8.     18.     30.     44.     58.     74.     91.    109.    128.
    147.    167.    188.    209.    231.    253.    275.    298.    321.    344.
    366.    387.    406.    424.    441.    457.    471.    483.    495.    504.
    513.    520.    525.    528.    531.    531.    529.    523.    513.    502.
    490.    479.    468.    458.    447.    437.    427.    418.    408.    399.
    390.    381.    373.    364.    356.    348.    340.    333.    325.    318.
    311.    304.    297.    290.    283.    277.    271.    265.    259.    253.
    247.    242.    236.    231.    226.    220.    215.    211.    206.    201.
    197.    192.    188.    184.    180.    175.    172.    168.    164.    160.
    157.    153.    150.    146.    143.    140.    136.    133.    130.    127.

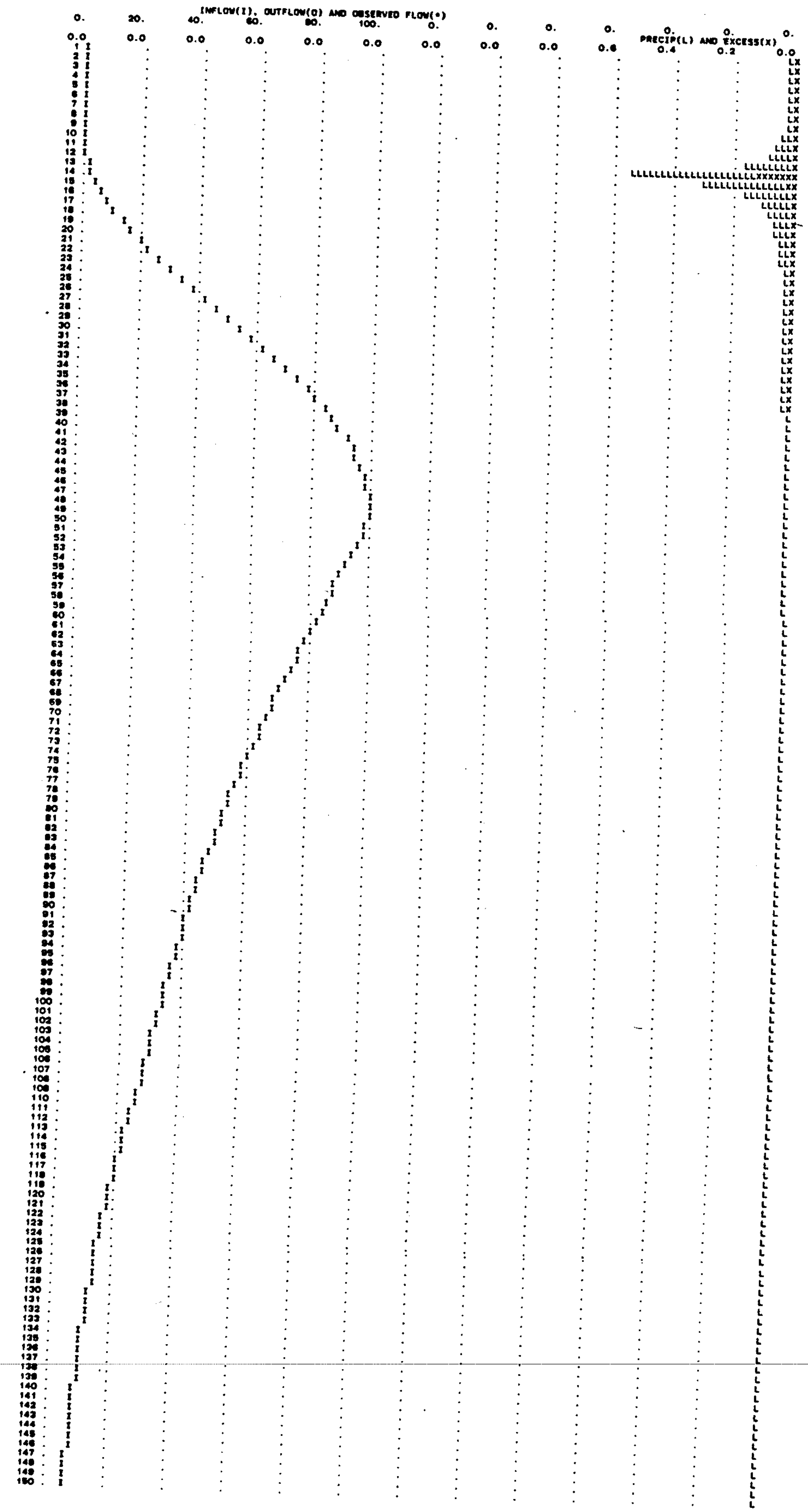
                      END-OF-PERIOD FLOW
                  TIME   RAIN   EXCS     COMP Q
                     1   0.01   0.00         0.
                     2   0.01   0.00         0.
                     3   0.01   0.00         0.
                     4   0.01   0.00         0.
                     5   0.01   0.00         0.
                     6   0.02   0.00         0.
                     7   0.02   0.00         0.
                     8   0.02   0.00         0.
                     9   0.03   0.00         0.
                    10   0.05   0.00         0.
                    11   0.08   0.00         0.
                    12   0.16   0.01         1.
                    13   0.54   0.12         1.
                    14   0.29   0.01         2.
                    15   0.16   0.01         4.
                    16   0.10   0.01         6.
                    17   0.07   0.00         8.
                    18   0.06   0.00        11.
                    19   0.05   0.00        13.
                    20   0.04   0.00        16.
                    21   0.03   0.00        19.
                    22   0.03   0.00        23.
                    23   0.02   0.00        26.
                    24   0.02   0.00        30.
                    25   0.02   0.00        34.
                    26   0.02   0.00        37.
                    27   0.01   0.00        41.
                    28   0.01   0.00        45.
                    29   0.01   0.00        49.
                    30   0.01   0.00        54.
                    31   0.01   0.00        58.
                    32   0.01   0.00        62.
                    33   0.01   0.00        66.
                    34   0.01   0.00        70.
                    35   0.01   0.00        74.
                    36   0.01   0.00        77.
                    37   0.01   0.00        80.
                    38   0.0    0.0         83.
                    39   0.0    0.0         86.
                    40   0.0    0.0         88.
                    41   0.0    0.0         91.
                    42   0.0    0.0         93.
                    43   0.0    0.0         95.
                    44   0.0    0.0         96.
                    45   0.0    0.0         97.
                    46   0.0    0.0         98.
                    47   0.0    0.0         98.
                    48   0.0    0.0         98.
                    49   0.0    0.0         98.
                    50   0.0    0.0         98.
                    51   0.0    0.0         97.
                    52   0.0    0.0         96.
                    53   0.0    0.0         94.
                    54   0.0    0.0         92.
                    55   0.0    0.0         91.
                    56   0.0    0.0         89.
                    57   0.0    0.0         87.
                    58   0.0    0.0         86.
                    59   0.0    0.0         84.
                    60   0.0    0.0         82.
                    61   0.0    0.0         80.
                    62   0.0    0.0         78.
                    63   0.0    0.0         77.
                    64   0.0    0.0         75.
                    65   0.0    0.0         74.
                    66   0.0    0.0         72.
                    67   0.0    0.0         70.
                    68   0.0    0.0         69.
                    69   0.0    0.0         67.
                    70   0.0    0.0         66.
                    71   0.0    0.0         64.
                    72   0.0    0.0         63.
                    73   0.0    0.0         62.
                    74   0.0    0.0         60.
                    75   0.0    0.0         59.
                    76   0.0    0.0         58.
                    77   0.0    0.0         56.
                    78   0.0    0.0         55.
                    79   0.0    0.0         54.
                    80   0.0    0.0         53.
                    81   0.0    0.0         51.
                    82   0.0    0.0         50.
                    83   0.0    0.0         49.
                    84   0.0    0.0         48.
                    85   0.0    0.0         47.
                    86   0.0    0.0         46.
                    87   0.0    0.0         45.
                    88   0.0    0.0         44.
                    89   0.0    0.0         43.
                    90   0.0    0.0         42.
                    91   0.0    0.0         41.
                    92   0.0    0.0         40.
                    93   0.0    0.0         39.
                    94   0.0    0.0         38.
                    95   0.0    0.0         37.
                    96   0.0    0.0         36.
                    97   0.0    0.0         35.
                    98   0.0    0.0         35.
                    99   0.0    0.0         34.
                   100   0.0    0.0         33.
                   101   0.0    0.0         32.
                   102   0.0    0.0         32.
                   103   0.0    0.0         31.
                   104   0.0    0.0         30.
                   105   0.0    0.0         29.
                   106   0.0    0.0         29.
                   107   0.0    0.0         28.
                   108   0.0    0.0         27.
                   109   0.0    0.0         26.
                   110   0.0    0.0         25.
                   111   0.0    0.0         24.
                   112   0.0    0.0         24.
                   113   0.0    0.0         23.
                   114   0.0    0.0         22.
                   115   0.0    0.0         21.
                   116   0.0    0.0         21.
                   117   0.0    0.0         20.
                   118   0.0    0.0         19.
                   119   0.0    0.0         19.
                   120   0.0    0.0         18.
                   121   0.0    0.0         17.
                   122   0.0    0.0         17.
                   123   0.0    0.0         16.
                   124   0.0    0.0         16.
                   125   0.0    0.0         15.
                   126   0.0    0.0         14.
                   127   0.0    0.0         14.
                   128   0.0    0.0         13.
                   129   0.0    0.0         13.
                   130   0.0    0.0         13.
                   131   0.0    0.0         12.
                   132   0.0    0.0         12.
                   133   0.0    0.0         11.
                   134   0.0    0.0         11.
                   135   0.0    0.0         11.
                   136   0.0    0.0         10.
                   137   0.0    0.0         10.
                   138   0.0    0.0          9.
                   139   0.0    0.0          9.
                   140   0.0    0.0          9.
                   141   0.0    0.0          8.
                   142   0.0    0.0          8.
                   143   0.0    0.0          8.
                   144   0.0    0.0          8.
                   145   0.0    0.0          7.
                   146   0.0    0.0          7.
                   147   0.0    0.0          7.
                   148   0.0    0.0          7.
                   149   0.0    0.0          6.
                   150   0.0    0.0          6.

                   SUM   1.99   0.16       6000.

                PEAK   6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
        CFS      98.      68.       40.       40.          6003.
     INCHES               0.14      0.17      0.17          0.17
      AC-FT                34.       41.       41.            41.
```

OVF

STATION 1



		SUM 1.99	0.75	10070.	

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	164.	114.	67.	67.	10070.
INCHES		0.57	0.70	0.70	0.70
AC-FT		56.	69.	69.	69.

OVN

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	1	0	0	2	1	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	2	0	2.830	0.420	0.0	0.

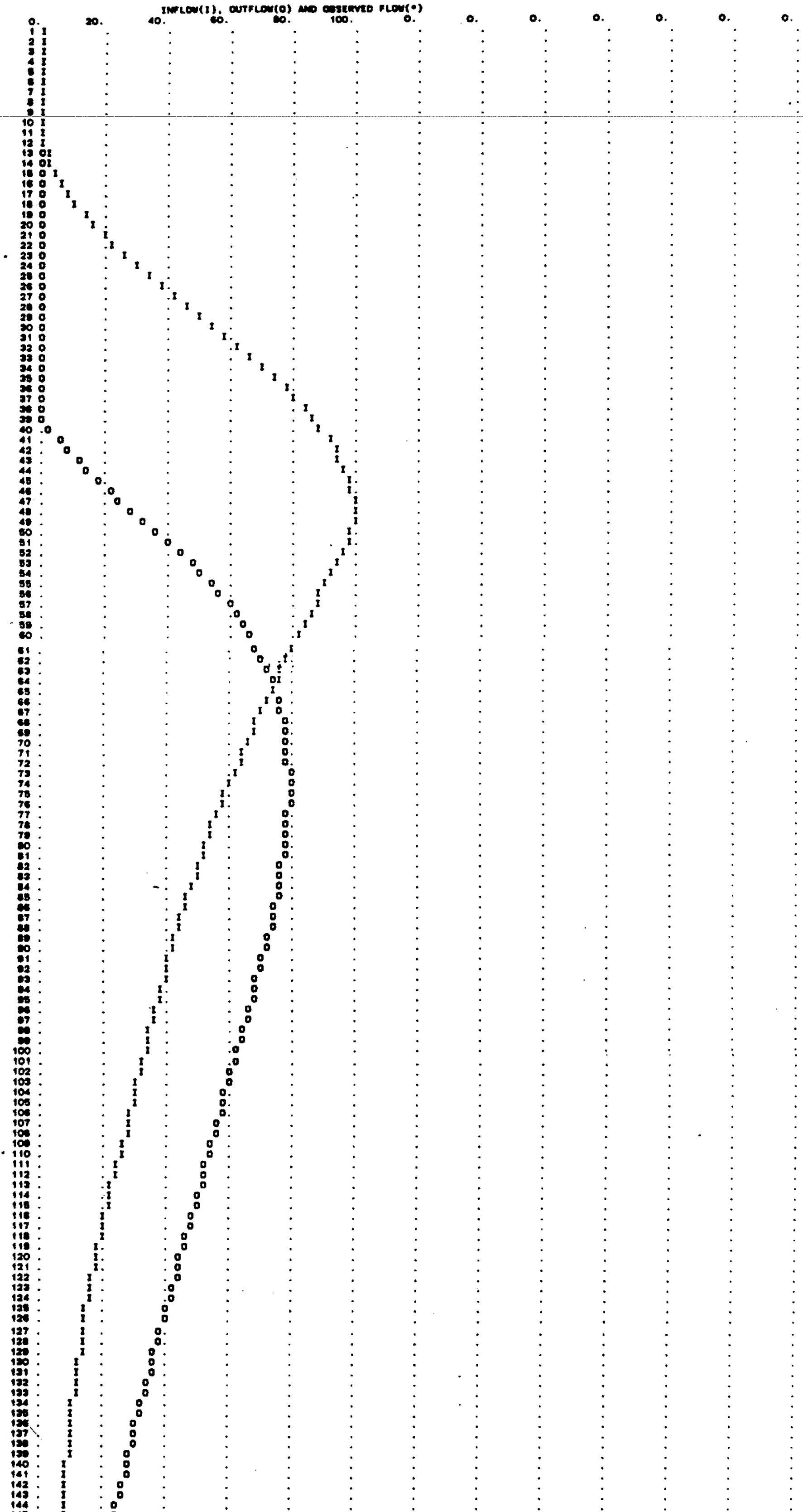
ROUTED FLOWS AT 2

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	3.
6.	8.	11.	14.	18.	21.	24.	28.	32.	35.
39.	43.	47.	51.	54.	57.	60.	62.	65.	67.
69.	70.	72.	73.	74.	75.	76.	77.	78.	78.
79.	79.	79.	79.	79.	79.	79.	79.	78.	78.
77.	77.	76.	76.	75.	74.	74.	73.	72.	71.
71.	70.	69.	68.	67.	66.	65.	64.	63.	63.
62.	61.	60.	59.	58.	57.	56.	55.	54.	54.
53.	52.	51.	50.	49.	49.	48.	47.	46.	45.
44.	43.	42.	41.	40.	39.	39.	38.	37.	36.
35.	34.	33.	33.	32.	31.	30.	29.	29.	28.
27.	26.	26.	25.	24.	24.	23.	22.	22.	21.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	79.	64.	38.	38.	5708.
INCHES		0.14	0.17	0.17	0.17
AC-FT		32.	39.	39.	39.

DVF

STATION 2



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*DIAP

**********          **********          **********          **********          **********

                          SUB-AREA RUNOFF COMPUTATION
                    ISTAQ   ICOMP   IECON   ITAPE   JPLT    JPRT   INAME
                        2       0       0       0      2       1       0

                                 HYDROGRAPH DATA
         IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
             0       0    1.85     0.0     0.0     0.0     0.0       0       0       0

                                   PRECIP DATA
                             NP   STORM    DAJ     DAK
                             37    1.98    0.0     0.0
                                 PRECIP PATTERN
  0.01      0.01      0.01      0.01      0.01      0.02      0.02      0.03      0.03      0.05
  0.08      0.16      0.34      0.23      0.16      0.10      0.07      0.05      0.05      0.04
  0.03      0.03      0.03      0.02      0.02      0.02      0.01      0.01      0.01      0.01
  0.01      0.01      0.01      0.01      0.01      0.01      0.01

                                    LOSS DATA
      STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
       2.00    1.07    2.27    0.52    0.0     1.00    0.0     0.0     0.0     0.36

                              UNIT HYDROGRAPH DATA
                          TC=  3.16    R=  3.85    NTA=  0

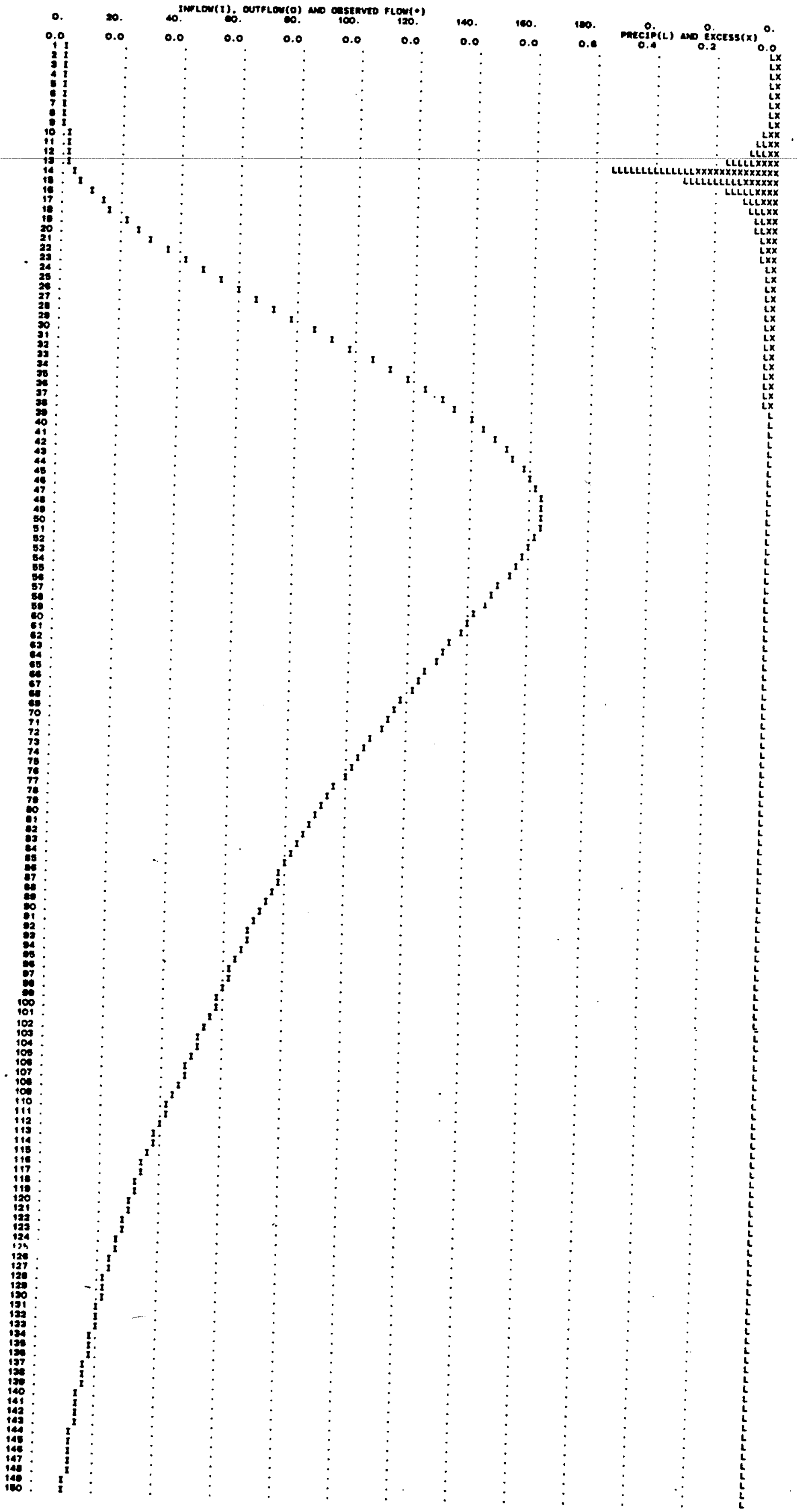
                                 RECESSION DATA
                    STRTQ=    0.0     QRCSN=   -0.31    RTIOR= 1.42

     UNIT HYDROGRAPH100 END-OF-PERIOD ORDINATES, LAG=   3.03 HOURS, CP= 0.54  VOL= 0.84
     1.        4.        8.       13.       18.       24.       31.       38.       45.       53.
    61.       70.       78.       87.       96.      105.      115.      124.      134.      143.
   153.      161.      169.      177.      184.      190.      196.      201.      206.      210.
   214.      217.      219.      220.      221.      221.      220.      218.      214.      209.
   204.      200.      195.      191.      186.      182.      178.      174.      170.      166.
   163.      159.      155.      152.      148.      145.      142.      139.      135.      132.
   129.      126.      124.      121.      118.      115.      113.      110.      108.      105.
   103.      101.       98.       96.       94.       92.       90.       88.       86.       84.
    82.       80.       78.       77.       75.       73.       71.       70.       68.       67.
    65.       64.       62.       61.       60.       58.       57.       56.       54.       53.

                              END-OF-PERIOD FLOW
                         TIME    RAIN   EXCS     COMP Q
                            1    0.01   0.00        0.
                            2    0.01   0.00        0.
                            3    0.01   0.00        0.
                            4    0.01   0.00        0.
                            5    0.01   0.00        0.
                            6    0.02   0.01        0.
                            7    0.02   0.01        0.
                            8    0.03   0.01        1.
                            9    0.03   0.01        1.
                           10    0.05   0.02        1.
                           11    0.08   0.03        1.
                           12    0.16   0.08        2.
                           13    0.34   0.26        3.
                           14    0.23   0.10        5.
                           15    0.16   0.06        7.
                           16    0.10   0.04       10.
                           17    0.07   0.03       13.
                           18    0.05   0.02       17.
                           19    0.05   0.02       21.
                           20    0.04   0.01       26.
                           21    0.03   0.01       31.
                           22    0.03   0.01       36.
                           23    0.03   0.01       41.
                           24    0.02   0.01       47.
                           25    0.02   0.01       53.
                           26    0.02   0.01       59.
                           27    0.01   0.00       66.
                           28    0.01   0.00       72.
                           29    0.01   0.00       78.
                           30    0.01   0.00       85.
                           31    0.01   0.00       92.
                           32    0.01   0.00       99.
                           33    0.01   0.00      105.
                           34    0.01   0.00      112.
                           35    0.01   0.00      118.
                           36    0.01   0.00      124.
                           37    0.01   0.00      129.
                           38    0.0    0.0       135.
                           39    0.0    0.0       139.
                           40    0.0    0.0       144.
                           41    0.0    0.0       148.
                           42    0.0    0.0       152.
                           43    0.0    0.0       155.
                           44    0.0    0.0       158.
                           45    0.0    0.0       160.
                           46    0.0    0.0       162.
                           47    0.0    0.0       163.
                           48    0.0    0.0       164.
                           49    0.0    0.0       164.
                           50    0.0    0.0       164.
                           51    0.0    0.0       162.
                           52    0.0    0.0       160.
                           53    0.0    0.0       158.
                           54    0.0    0.0       156.
                           55    0.0    0.0       153.
                           56    0.0    0.0       151.
                           57    0.0    0.0       148.
                           58    0.0    0.0       146.
                           59    0.0    0.0       143.
                           60    0.0    0.0       140.
                           61    0.0    0.0       137.
                           62    0.0    0.0       135.
                           63    0.0    0.0       132.
                           64    0.0    0.0       129.
                           65    0.0    0.0       127.
                           66    0.0    0.0       124.
                           67    0.0    0.0       121.
                           68    0.0    0.0       119.
                           69    0.0    0.0       116.
                           70    0.0    0.0       114.
                           71    0.0    0.0       111.
                           72    0.0    0.0       109.
                           73    0.0    0.0       106.
                           74    0.0    0.0       104.
                           75    0.0    0.0       101.
                           76    0.0    0.0        99.
                           77    0.0    0.0        97.
                           78    0.0    0.0        95.
                           79    0.0    0.0        93.
                           80    0.0    0.0        91.
                           81    0.0    0.0        89.
                           82    0.0    0.0        86.
                           83    0.0    0.0        85.
                           84    0.0    0.0        83.
                           85    0.0    0.0        81.
                           86    0.0    0.0        79.
                           87    0.0    0.0        77.
                           88    0.0    0.0        75.
                           89    0.0    0.0        74.
                           90    0.0    0.0        72.
                           91    0.0    0.0        70.
                           92    0.0    0.0        68.
                           93    0.0    0.0        67.
                           94    0.0    0.0        65.
                           95    0.0    0.0        64.
                           96    0.0    0.0        62.
                           97    0.0    0.0        61.
                           98    0.0    0.0        60.
                           99    0.0    0.0        58.
                          100    0.0    0.0        57.
                          101    0.0    0.0        56.
                          102    0.0    0.0        54.
                          103    0.0    0.0        53.
                          104    0.0    0.0        52.
                          105    0.0    0.0        50.
                          106    0.0    0.0        49.
                          107    0.0    0.0        47.
                          108    0.0    0.0        46.
                          109    0.0    0.0        44.
                          110    0.0    0.0        43.
                          111    0.0    0.0        41.
                          112    0.0    0.0        40.
                          113    0.0    0.0        39.
                          114    0.0    0.0        37.
                          115    0.0    0.0        36.
                          116    0.0    0.0        35.
                          117    0.0    0.0        33.
                          118    0.0    0.0        32.
                          119    0.0    0.0        31.
                          120    0.0    0.0        30.
                          121    0.0    0.0        29.
                          122    0.0    0.0        28.
                          123    0.0    0.0        27.
                          124    0.0    0.0        26.
                          125    0.0    0.0        25.
                          126    0.0    0.0        24.
                          127    0.0    0.0        24.
                          128    0.0    0.0        23.
                          129    0.0    0.0        22.
                          130    0.0    0.0        21.
                          131    0.0    0.0        20.
                          132    0.0    0.0        20.
                          133    0.0    0.0        19.
                          134    0.0    0.0        18.
                          135    0.0    0.0        18.
                          136    0.0    0.0        17.
                          137    0.0    0.0        17.
                          138    0.0    0.0        16.
                          139    0.0    0.0        15.
                          140    0.0    0.0        15.
                          141    0.0    0.0        14.
                          142    0.0    0.0        14.
                          143    0.0    0.0        13.
                          144    0.0    0.0        13.
                          145    0.0    0.0        13.
                          146    0.0    0.0        12.
                          147    0.0    0.0        12.
                          148    0.0    0.0        11.
                          149    0.0    0.0        11.
                          150    0.0    0.0        11.
```

OVF

STATION 2



OVN

********** ********** ********** ********** **********

COMBINE HYDROGRAPHS

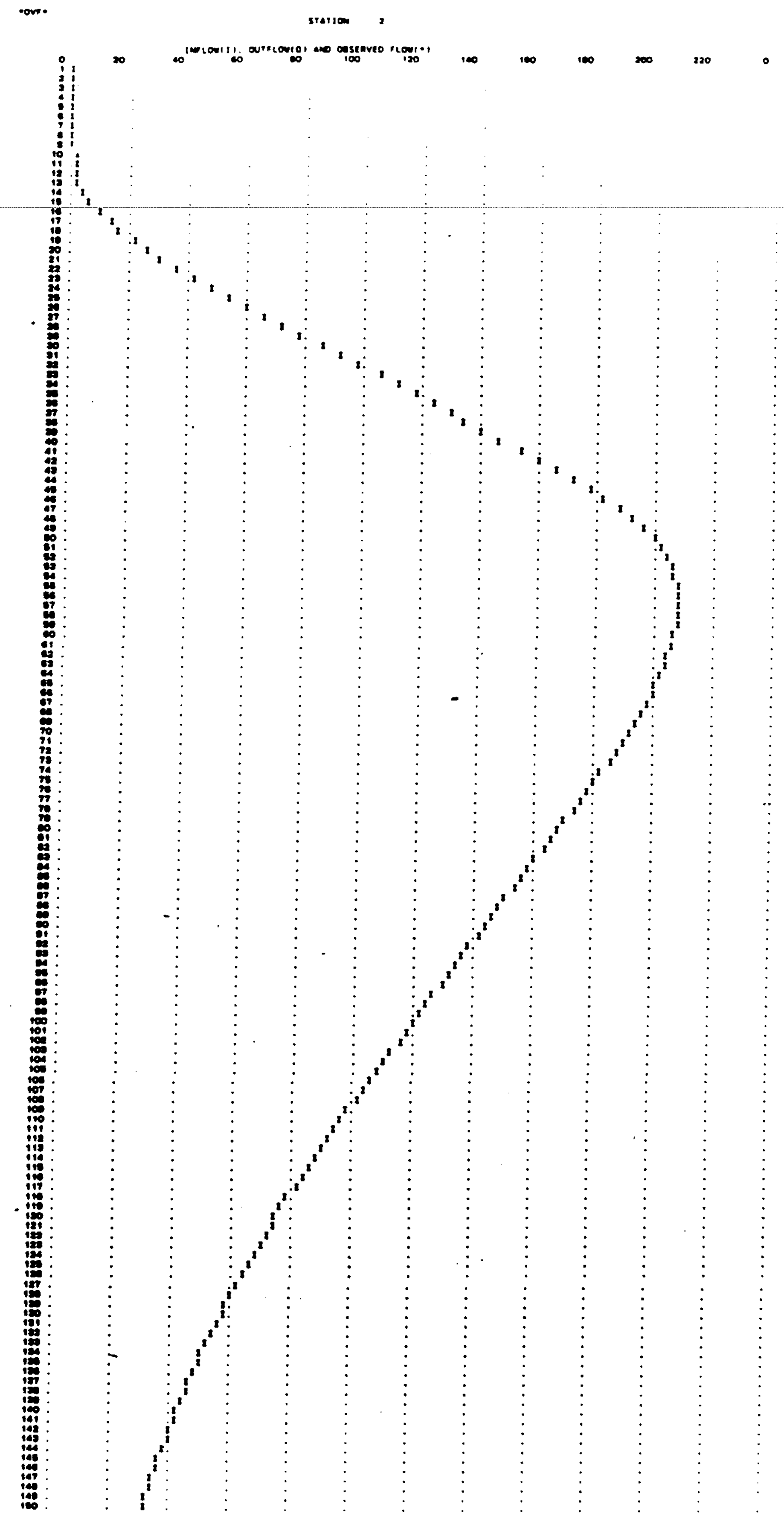
25-YEAR DESIGN HYDROGRAPH AT ASSINIBOINE GOLF COURSE

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
2	2	0	0	2	1	1

SUM OF 2 HYDROGRAPHS AT 2

0.	0.	0.	0.	0.	0.	0.	1.	1.	1.
1.	2.	3.	5.	7.	10.	13.	17.	21.	26.
31.	36.	41.	47.	53.	59.	65.	72.	78.	85.
92.	99.	105.	112.	118.	124.	129.	135.	140.	147.
154.	160.	166.	172.	178.	183.	188.	192.	196.	199.
202.	204.	205.	207.	207.	208.	208.	208.	207.	207.
206.	205.	204.	202.	201.	199.	198.	196.	194.	192.
190.	188.	185.	183.	181.	178.	176.	173.	171.	168.
166.	163.	161.	158.	156.	153.	151.	148.	146.	143.
141.	139.	136.	134.	131.	129.	127.	124.	122.	120.
117.	115.	113.	110.	108.	106.	103.	101.	99.	96.
94.	92.	90.	87.	85.	83.	81.	79.	77.	75.
73.	71.	69.	67.	66.	64.	62.	60.	59.	57.
55.	54.	52.	51.	49.	48.	47.	45.	44.	43.
42.	40.	39.	38.	37.	36.	35.	34.	33.	32.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	208.	166.	105.	105.	15778.
INCHES		0.25	0.32	0.32	0.32
AC-FT		82.	109.	109.	109.



OVF
STATION 2
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0 20 40 60 80 100 120 140 160 180 200 220 0

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	99.	68.	40.	40.	4.44
ROUTED TO	2	79.	64.	38.	38.	4.44
HYDROGRAPH AT	2	164.	114.	67.	67.	1.85
2 COMBINED	2	208.	166.	105.	105.	6.29