



A STUDY OF DETENTION IN URBAN  
STORMWATER MANAGEMENT

by

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E R R A T A

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On page 52, under Part B, Section 2, Paragraph 2, Line 2, "...for side slopes of 2 to 1 are shown in Table 9." should read "...for side slopes of 2 to 1 are shown in Table 10."

On page 55, under Vertical Distribution of Tractive Force on Channel Side, the equation

$$\tau_z = [.749 + 0.0145 \ln (B/y)] \sin [\pi(z/y)^{0.45}] \text{ should read}$$

$$\tau_z = [.749 + 0.0145 \ln (B/y)] \sin [\pi(z/y)^{0.45}] \gamma_{ys}.$$

On page 58, Table 11 should read as follows:

Table 11

HYDROGRAPH FORMULATION FOR IMPULSE ANALYSIS

Item	Before Development		After Development	
	2-yr. Event	10-yr. Event	2-yr. Event	10-yr. Event
SCS Curve Number	60	60	75	75
24-hr. Rainfall (in.)	3.5	5.0	3.5	5.0
6-hr. Rainfall (in.)	2.50	3.75	2.50	3.75
6-hr. Runoff (in.)	0.17	0.64	0.65	1.48
Peak Discharge (cfs)	36	115	160	600
Time to Peak (min.)	77	89	65	39

On page 67, in Paragraph 4, the equation

$$\tau_{\max} = \gamma_{ys}[0.749 + 0.145 \ln (B/y)] \text{ should read as follows:}$$

$$\tau_{\max} = \gamma_{ys}[0.749 + 0.0145 \ln (B/y)].$$

On page 67, in Paragraph 4, the equation

$$\tau_{\max} = \gamma_{ys}[46.74 + 9.05 \ln (B/y)] \text{ should read as follows:}$$

$$\tau_{\max} = \gamma_{ys}[46.74 + 0.905 \ln (B/y)].$$

On page 69, in Figure 30, the calibration of the horizontal axis is incorrect, but the shapes and relative positions of the curves are correct.



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## ABSTRACT

The concept of stormwater detention was analyzed for effectiveness in controlling urban flooding and stream-channel enlargement in North Carolina. Model studies, statistical analyses and theoretical analyses were conducted using gaged watersheds in Charlotte, North Carolina. Existing conventionally developed watersheds were found to have effective stormwater detention unintentionally incorporated behind culverts and bridges.

Statistical analysis of long-term historical streamflows suggests that flood peaks diminished with urbanization until about 1950, when peaks began to rise again. Although detention can be shown to be present and effective by simulation and statistical analysis, stream channel enlargement and consequent high sediment loads are observable in the streams of the study area.

Impulse analysis of channels subjected to discharges from hydrographs with and without detention suggests that detention can worsen streambank degradation by intensifying the impulse intensity at the channel sides near the bottom to cause more severe slumping.

Stormwater detention is recommended for control of nuisance flooding immediately downstream of radical changes in land use. It should be incorporated in a comprehensive watershed management program wherein the drainage system is treated as a public utility so that economies of scale can be gained and facilities can be located strategically.



## ACKNOWLEDGMENTS

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The members of the Technical Advisory Committee of the North Carolina Sedimentation Control Commission debated these issues during the course of the project. The research was much affected by the arguments of Dr. Joe Phillips, Dr. Jim Stewart, Mr. Bill Weldon, Mr. Taylor Currin, Mr. Harry Morgan, and Mr. Cameron Lee.

Considerable resources, advice, and constructive criticism were provided to the investigator. Mr. Clark Readling, City Engineer of Charlotte, and his staff gave much support in mapping, field work, and record searching. Dr. Brendan Harley of Resource Analysis, Inc., made available the MITCAT Model and provided much helpful advice in its use. Mr. Ralph Heath, District Chief of the U. S. Geological Survey, and several members of his staff assisted in providing streamflow records, interpretations, and constructive arguments, especially when the need arose to use the original records. Dr. Joe Kleiss and others of the North Carolina State University Soil Science Department helped with soil identification and classification in the study area.

Several students contributed materially to the project. Wayne Meads, Bob Houghtalen, Charles Southerland, Tracy Hill, and Kevin Caldwell were full participants in the research.

Some who have been acknowledged do not agree with the findings of the project. The author respects and appreciates their arguments and claims no special insight to assert his own rightness. Time and experience will provide for the use of the best of all our opinions.

The project was made much better by these contributions, but responsibility for the shortcomings of the work belongs to the author.





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## SUMMARY

The original purpose of the research was to study the concept of stormwater detention as a means of controlling urban streambank deterioration and urban flooding, with particular emphasis on the determination of the best design storm to be specified in a stormwater-detention program. Implicit were the premises that stormwater detention would provide flood-control benefits, by peak-stage reduction, and sediment-control benefits, by prevention of stream-channel enlargement; and that these benefits would accrue significantly in both areas of concern by specifying a single design storm. As the project developed, it was found that these premises must be rejected, at least for conditions prevalent in the study area.

The project was executed as a group of concurrent studies using an existing stormwater runoff model validated for watersheds in the study area, design studies verified by modelling, statistical analysis of urban streamflow records and theoretical analyses of streambank degradation.

The model studies were conducted in the Charlotte, North Carolina, urban area where suitable records exist for a 41-square-mile watershed and several subwatersheds. The MITCAT Model was calibrated and validated for several storms on two subwatersheds, one of 360 acres, the other of 18.5 square miles.

The early model studies revealed that the runoff model could be calibrated reasonably only by simulating certain significant culverts as reservoirs in the model. After validation, removal of the culverts from the model showed that considerable peak-flow reduction was taking place behind existing culverts.

Streamflow records exist from 1924 to the present for Little Sugar Creek at Tyvola Road in Charlotte (41 square miles). It was expected that an upward trend in peak flows could be identified as urbanization became more dense in the watershed. Conventional statistical analyses, however, suggest that peak flows diminished through the early part of the record and that they have risen again since about 1950. It is during this latter period that culverts have been systematically enlarged on the mainstems and principal tributaries of the watershed as thoroughfares have been widened and reconstructed.

The streams of the watersheds carry large sediment loads. Assuming construction-related erosion to have been controlled effectively in recent years, the main source of the sediment is the deteriorating urban streambank. Extensive slumping can be observed along the streambanks of the watersheds.

Stormwater detention has been proposed as a means of prevention of streambank deterioration, the trouble having been associated with increasing peak flows with urbanization. But here are watersheds with considerable detention supplied, with peaks diminishing through the record and with streambank enlargement taking place anyway. Can detention be a part of the cause of streambank enlargement?

With detention, the hydrograph is made longer and flatter at constant volume. The flow does not run as high, but it does run at a lower rate for a





much longer period of time. The concept of impulse analysis, impulse being the product of force and the time over which it is applied, was used to study the differences in erosive force behavior in a stream channel with and without detention. It was found that the overall impulse was not very different with or without detention. But because the stage is held lower and the flow persists at a significant magnitude for a longer period of time, with detention the intensity of the impulse is greater near the bottom of the channel sides. The channel sides are undermined more severely, and slumping takes place when the slopes become structurally unstable.

The culvert is seen to be a main actor in watershed behavior. Should a typical stormwater-detention regulation be imposed, it can be shown that the entire effect to be expected from the regulation can often be overwhelmed by the enlargement of a single highway culvert. Culvert design decisions should be made in the context of total watershed management.

Flood control and sediment control are designed at different levels of risk. Effective sediment-control designs should be done for relatively frequent storms (the two to ten-year event), whereas flood-damage control generally is justified for rarer storms (50 to 100-year events). A regulation intended for both purposes using a single design storm is difficult to defend.

Stormwater detention is quite effective in preventing nuisance flooding immediately downstream of radical changes in land use. For this purpose detention is useful to protect off-site downstream interests, and it can be used profitably within projects for designing more economical drainage systems than those designed conventionally. The keys to gains in economy are to integrate the drainage design in the overall project planning from the outset and to make use of the storage area for other purposes in dry weather.

Stormwater detention is not recommended as a single or central policy in watershed management. It is one of several available tools, to be used where it does some good. In watershed management, it is important to provide for the efficient allocation of all resources. To specify one means of control, to be applied universally, is expressly inefficient. A potential mechanism for allocation is to treat the drainage system as a public utility, as water distribution and wastewater collection are accomplished. Under central planning and design, the several developments of a given watershed can be viewed as a unit, and drainage costs may be apportioned among those who benefit.

The streambank degradation issue is complex. Detention may not help. An urban stream apparently cannot look like a rural stream, a natural stream. It need not look like an open sewer. Between, there is something that makes sense, and it may not be the same thing for all reaches of a given stream. The River Seine is a thing of beauty, the subject of many artful renderings. It doesn't look like a natural stream, but it was right for the Left Bank. Instead of insisting on "natural" urban streams, it may be more appropriate to ask what an urban stream can look like and to find what an urban stream ought to look like.



## CONCLUSIONS

1. Urban watersheds discharge significantly more sediment than rural watersheds of the same size. The source of much of this sediment is the eroding urban streambank.
2. Streambank degradation is far more complex than is generally taken into account in storm-drainage design and management policies. It is unlikely that a suitable stream-stability policy can be developed to preserve urban streams in a natural state.
3. Watershed simulation models are not yet ready to be used generally by cities and other local jurisdictions as day-to-day decision-making tools because of their capriciousness, gargantuan appetite for data and file maintenance and because of some remaining gaps in logic and means of data acquisition.
4. Model selection and data acquisition should follow careful study of the capabilities of available models and the data required for calibration and validation. The purpose to which the model is to be put and the cost of data gathering and file maintenance ought to govern the choice.
5. The effect of temporary storage of surface runoff on the shape of the hydrograph is pronounced and highly significant. The location and magnitude of storage in relation to the size of the watershed is important in determining the degree of peak-flow reduction. In general, large facilities on the mainstem have a greater effect on the peak than small facilities distributed widely through the watershed.
6. Historically, conventionally developed urban watersheds have included significant temporary storage capacity without that intent. That storage typically has been concentrated behind culverts and bridges.
7. Urbanization generates more surface runoff but not necessarily higher peaks.
8. As stream crossings are "improved" during construction or expansion of traffic arteries, large quantities of previously available temporary storage are lost when the hydraulic capacities of culverts and bridges are increased. The effect is to increase peak flows downstream from the new construction.
9. The effect on peak flow of including storage distributed broadly at small scale in the watershed is lost a short distance downstream from each detention facility.
10. Detention measures intended for the control of sedimentation can aggravate flooding and conversely.
11. The detention design storm for control of streambank degradation should be a relatively frequent storm, perhaps the 2-year storm, with a permissible outflow much less than bank full, set by tractive-force consideration.



12. Detention facilities for flood-damage control must reduce flood stages below damaging levels in rare-event storms of recurrence intervals set by economic analysis--usually rarer than the 50-year event.
13. Programs of urban flood control and sediment control need to be coordinated carefully for they have the potential for mutual conflict.
14. Stormwater detention as a single or central policy for sediment control is overly simplistic, probably ineffective, possibly damaging. Its efficacy cannot be demonstrated now.
15. Stormwater detention as a single policy for flood control is uneconomical because of the large storage requirement.
16. Stormwater detention can be quite economical when done at small scale to control nuisance flooding just downstream from radical changes in land use. It is frequently less expensive in first cost and maintenance than conventional design approaches when it is integrated carefully with other features of the project plan.
17. In watershed management, it is important to provide for the efficient allocation of all resources available. To specify one tool to be applied universally in the watershed is expressly inefficient. A potential mechanism for this provision is to treat the drainage system as a public utility in parallel with the provision of water, sewerage, and streets.



## SECTION I

### INTRODUCTION

#### A. BACKGROUND

Urban stormwater management programs have historically been undertaken for two principal purposes: for mitigation of urban flooding and for control of urban sedimentation. Although urban sedimentation and flooding problems both occur as effects of urban runoff phenomena, their solutions have been pursued through separate programs, the objectives of which have not necessarily been mutually consistent.

The intent of this research has been to study stormwater management practices as they relate to emerging problems in North Carolina and to ask if an integrated set of stormwater management standards might be appropriate for the economical accomplishment of objectives in both areas.

As the project began, programs identified in other geographical areas were seen to be very different one from another, and these evidences of experience did not provide clear guidance for programs or appropriate levels of protection.

In the several counties of Maryland, where precedent-setting programs were established, a variety of design standards have emerged. Montgomery County, for example, requires the peak outflow from the two-year storm after development to be held to the peak outflow of the two-year storm from the same watershed under wooded conditions.

These program requirements, and others of similar basis, rely heavily on the concept of stormwater detention to bring about the desired results of stormwater management for sediment control, flood control or both.

It was implicitly assumed at the outset of this research that stormwater detention is a reliable practice for controlling the intensity of flooding and for reducing the rate of streambank degradation. Thus, the project was conceived largely as an investigation of an appropriate set of standards for stormwater detention that could form the basis for state regulations and local ordinances being considered in North Carolina.

As the research unfolded, it became increasingly clear that the behavior of urbanizing watersheds is so complex that one should not expect one management response of any kind to be effective alone. Stormwater detention can be seen to be a single tool among many for stormwater management. Problems differ from region to region and from watershed to watershed. It is unreasonable to expect widely different problems to yield to a single solution strategy. Within watersheds it is desirable to be able to distribute measures strategically rather than uniformly in order to allocate management resources efficiently.

The research questions were put too narrowly in the original proposal, but to expand to the comprehensive question of what should be an appropriate scheme for optimum management of all watersheds is to enlarge the scope of the project unreasonably. The investigator decided to stick to the original objective set and to interpret research findings in relation to the original questions.





## B. RESEARCH OBJECTIVES

As given in the proposal, the specific research objectives are:

1. To define the state of current and proposed U. S. practice concerning stormwater management programs relating to mitigation of sedimentation and flood damage resulting from urbanization.
2. To investigate currently available hydrologic models and to select an appropriate one for assessing the impact of given projects on the flow regimes of the watersheds in which they are situated.
3. To subject selected sets of program standards to analysis, using regional case studies from North Carolina, with regard for
  - a. probable degree of effectiveness in mitigating urban sedimentation and/or flood damage,
  - b. cost of implementation on a project basis, and
  - c. cost of implementation on a watershed basis.
4. To determine whether economies of scale are realizable through public investments in stormwater management facilities.
5. To determine whether performance objectives in the two areas of concern (urban sedimentation and flood damage) should be pursued in an integrated system, or whether they should be pursued in separable programs.
6. To propose an appropriate set of stormwater management performance objectives for design of drainage systems in North Carolina.

## C. DESCRIPTION OF RESEARCH EFFORT

The investigation consisted of four general activities, which proceeded in parallel. These were the literature review, modelling effort, statistical studies and desk-top design studies.

The literature was reviewed continually for source material on the general topic and specifically for work on flood-damage control practice, the mechanics of streambank degradation and for descriptions of current and proposed practices.

An urban-runoff simulation model was selected, calibrated, and validated for two watersheds in North Carolina. Studies were conducted to determine the relative importance of the various calibrating parameters, the effect of urbanization on flows, velocities and erosive flows during storm events, and the changes attainable through the use of detention policies in the watersheds. Extension to other geographic areas was investigated, but validation of those results was not possible for lack of data.

Statistical studies were conducted of rainfall-runoff records in North Carolina to detect trends in flooding associated with urbanization.



Desk-top design studies were made using various detention policies both to examine the differences in application of various standards at the same location and to provide input conditions for the simulation model for the assessment of changes in streamflow from the use of given measures for control.

There were other supporting research activities. Many natural, urban, and urbanizing stream systems were observed. Design professionals, public officials, and investigators in other fields, such as geology, geography, soil science and statistics, were consulted. Several conferences were attended. In some, presentations of current research and tentative conclusions were made to obtain reactions from researchers and users. During much of the research period, the investigator was a member of the Technical Advisory Committee of the North Carolina Sedimentation Control Commission. The content of the research was presented to and influenced by that group as they debated and formulated regulations for control of streambank degradation in urban areas.



## SECTION II

### STORMWATER DETENTION PRACTICES

Stormwater detention requirements have been imposed for the purposes of flood control and control of streambank deterioration, the idea being to reduce the outflow from a site in the design storm to some permissible discharge by causing stormwater to be stored temporarily on site. In effect, stormwater detention facilities are small-scale, flood-control impoundments.

Design-storm magnitudes vary widely from place to place, depending on the purpose of the regulation and the logic on which it is based. Those intended primarily for flood control generally range from the 10-year to the 100-year event. Those intended primarily for control of streambank erosion use the two to ten-year storms. The widely recognized program in Montgomery County, Maryland, perhaps the first of its kind, based the design requirement on the two-year storm. Given that streams in the region are observed to size themselves naturally to contain approximately the two-year storm at bank-full flow, it was reasoned that a stream would not tend to enlarge if the peak of the two-year storm was held to the pre-urbanization discharge. For various reasons, in other locations where similar programs were adopted, other design storms were used. The tentative regulation among the North Carolina regulations for sediment control set the ten-year storm as the design storm to be consistent with the risk level selected for other sediment-control features, such as sediment basins for construction sites, and to be compatible with the prevalent design storm for other storm-drainage elements.

The typical statement of the design objective is to the effect that "The peak discharge from the site in the n-year storm after development can be no greater than the peak discharge in the n-year storm from the same site prior to development." Some regulations require the post-development discharge to be equivalent to that from a wooded area of the same size, irrespective of the pre-development site condition.

Such requirements generally result in the distribution of a great many small-scale facilities distributed widely through the watershed, developed in a random pattern in space and time, and rarely coordinated in design.

No retrospective field evaluations of the effectiveness of detention programs were found in this research.



## SECTION III

### MODEL SELECTION, CALIBRATION AND VALIDATION

#### A. MODELS CONSIDERED

The second project objective was to investigate currently available hydrologic models and to select an appropriate one for assessing the impact of given projects on the flow regimes of the watersheds in which they are situated. Criteria for selecting the model were that:

1. The model is in the public domain, available at nominal handling cost.
2. It is sufficiently precise in reproducing storm hydrographs measured in North Carolina.
3. Input data requirements are at a minimum for satisfactory results.
4. The model is structured so as to be compatible with computer facilities at North Carolina State University.

The models given primary consideration were EPA-SWMM, Corps of Engineers HEC1, The Stanford Watershed Model (together with some of its progeny) and the MIT Catchment Model. Other models, such as the University of Cincinnati Urban Runoff Model and the Illinois Urban Drainage Analysis System, were reviewed in the literature and dropped from consideration for various reasons.

The Stanford Watershed Model synthesizes a continuous hydrograph using a large quantity of climatological data and watershed properties. A summary of the model structure is readily available in recent hydrology texts (e.g., Linsely, et al., 1979, and Viessman, et al., 1977). The data base for this model is formidable. There are some 19 parameters which must be measured in the field or on maps, 30 parameters which must be assigned based on records or experiential assumption, 12 parameters to be set by trial-and-error adjustment and a large number of clerical and output-control parameters.

For the most part, this project is concerned with direct runoff from short-term storm events. Simulation of base flow, runoff from snowmelt, etc., were of little use. The extensive data set and the need for calibration of parameters for which there is no local experience put the effective use of Stanford-type models outside the resources of the project.

The investigator had a peripheral involvement in the application of the EPA Storm Water Management Model (Metcalf and Eddy, et al., 1971) as a part of an urban-runoff-quality investigation (Colston, 1974). Based on that experience, it was reasoned that SWMM is best suited for densely developed watersheds of high degrees of imperviousness, where infiltration is not a dominant factor in estimating runoff quantities, and where the bulk of the drainage system is in pipe. In this study, the main concern is for watersheds where the old natural channels receive the runoff during urbanization. The channel cross-section templates available in SWMM did not seem to be sufficiently flexible for this use. The model finally selected had that flexibility, and it also had more optional submodels for infiltration which, as it turned out, was a troublesome process to calibrate.

The model HEC1, of the Corps of Engineers (Corps of Engineers, 1973), was obtained and applied to small urban watersheds in Charlotte and Durham. HEC1 is a lumped-parameter model based on the unit-hydrograph concept. It must be calibrated specifically for each watershed condition using historical data. The investigator shortly found that he was using the model well outside the limits for which it was developed and that the structure of the model was not amenable to asking the what-if question about changes in watershed composition after calibration.

The model which was selected for use in the project is MITCAT (Harley, 1975), developed for commercial use by Brendan Harley from the MIT Catchment Model. MITCAT is a deterministic, distributed-parameter model which produces the hydrograph of a given storm incident but does not carry over continuously from incident to incident because groundwater contributions to streamflow are not computed.

The watershed is viewed as an aggregation of catchment elements which receive rainfall, deduct hydrologic abstractions and pass the surface runoff to stream elements in an appropriate time pattern determined by the size, composition and topography of the catchment elements. Stream elements are connected together to form the stream network, and the flows are routed according to the kinematic wave equation through the system. Reservoir elements may be interposed in the network.

An advantage of MITCAT is that almost all of the calibrating parameters which are not measured directly are parameters for which independent observations exist. Examples are the Manning roughness coefficient and the Holtan infiltration parameters. So it is possible to judge whether calibrating parameters are within defensible ranges when the size and shape of the computed hydrograph approach those of the observed hydrograph.

Of the models reviewed before and during the project, MITCAT seemed best. Calibration and validation required the least effort in field work. Resolution of the modelling effort (the selection of the size and number of catchment elements) could be varied to suit the question under investigation. The impact of urbanization on the stream system and the effect of various detention policies could be modelled with reasonable fidelity. The selection criterion that the model be in the public domain is not satisfied by the selection of MITCAT for it is a proprietary model.

## B. AVAILABLE TEST WATERSHEDS

The term of the project was not long enough nor were resources sufficient to install gaging stations to collect runoff and rainfall data for calibrating and validating the model. The project was, therefore, dependent upon existing hydrologic data for the modelling effort. It was desirable that test watersheds have the following attributes:

1. That rainfall and streamflow gaging data existed with spatial coverage of rainfall sufficient for good correlation and temporal coverage sufficient for assessing statistical significance.
2. That detailed topographic mapping of the watersheds were obtainable.



3. That good records of drainage facilities were available.
4. That a reliable soil survey had been conducted.

The limiting attribute is the rainfall/runoff requirement. Very few watersheds have been gaged in urban areas sufficiently precisely to relate surface runoff hydrographs to the causative rainfall. The only suitable records uncovered by this investigator are those used for the earlier USGS investigation of the nature of urban flooding in North Carolina (Putnam, 1972). The data set for that study includes simultaneously collected rainfall and runoff observations for storm events occurring in some cases during periods exceeding ten years. The area of coverage is piedmont North Carolina. Among the data are the 41.0 square-mile watershed of Little Sugar Creek in central Charlotte, North Carolina, and several subwatersheds ranging in size from less than one square mile to 18.5 square miles. During the ten-year period of intensive data gathering some of the same storms were recorded at all of the gaging points. The record of the 41.0 square-mile watershed extends back to 1924. The watersheds are shown in Fig. 1.

Watershed mapping and records of storm-drainage system construction were provided by Mr. Clark Readling, City Engineer, and his staff. Recently completed but unpublished soil-survey information was provided by the Mecklenburg County Office of the Soil Conservation Service. Little Sugar Creek possesses the desirable attributes of test watersheds to the greatest degree.

The mountain and coastal plain regions of North Carolina are also important in the assessment of stormwater management policies. However, no data was found for model implementation in either region. Nor were satisfactory records found for similar regions in adjacent states. The decision was made to concentrate the modelling and statistical efforts in the piedmont and to draw inferences for the other regions where necessary.

### C. MODEL CALIBRATION AND VALIDATION

1. General Calibration/Validation Considerations. The purpose of calibration is to set parameters which must be estimated at values such that the hydrographs computed for given historical rainfall incidents match the hydrographs observed at the gaging station for the same storms. The model is then validated by running one or more storms varying only the parameters which reflect rainfall, antecedent wetness conditions and seasonal variations where they are known. The closeness of the computed hydrographs to the observed hydrographs for the validation storms is a measure of the fidelity of the modelling process. Upon satisfactory validation, one may vary storm severity and physical features within the watershed and estimate watershed response to the changes, within reasonable limits.

Among the parameters subject to calibration are the Manning roughness coefficient for overland flow, main channels and overbank flow, several infiltration parameters, and an average overland flow length. These are set for each catchment element and stream segment. Parameters measured in the field or from maps are subject only to observational error, and they should not be adjusted except to improve the precision of the measurement.

During calibration, estimated parameters are adjusted within defensible ranges to match the peak, volume, and shape of the computed hydrograph to those of the observed hydrograph.

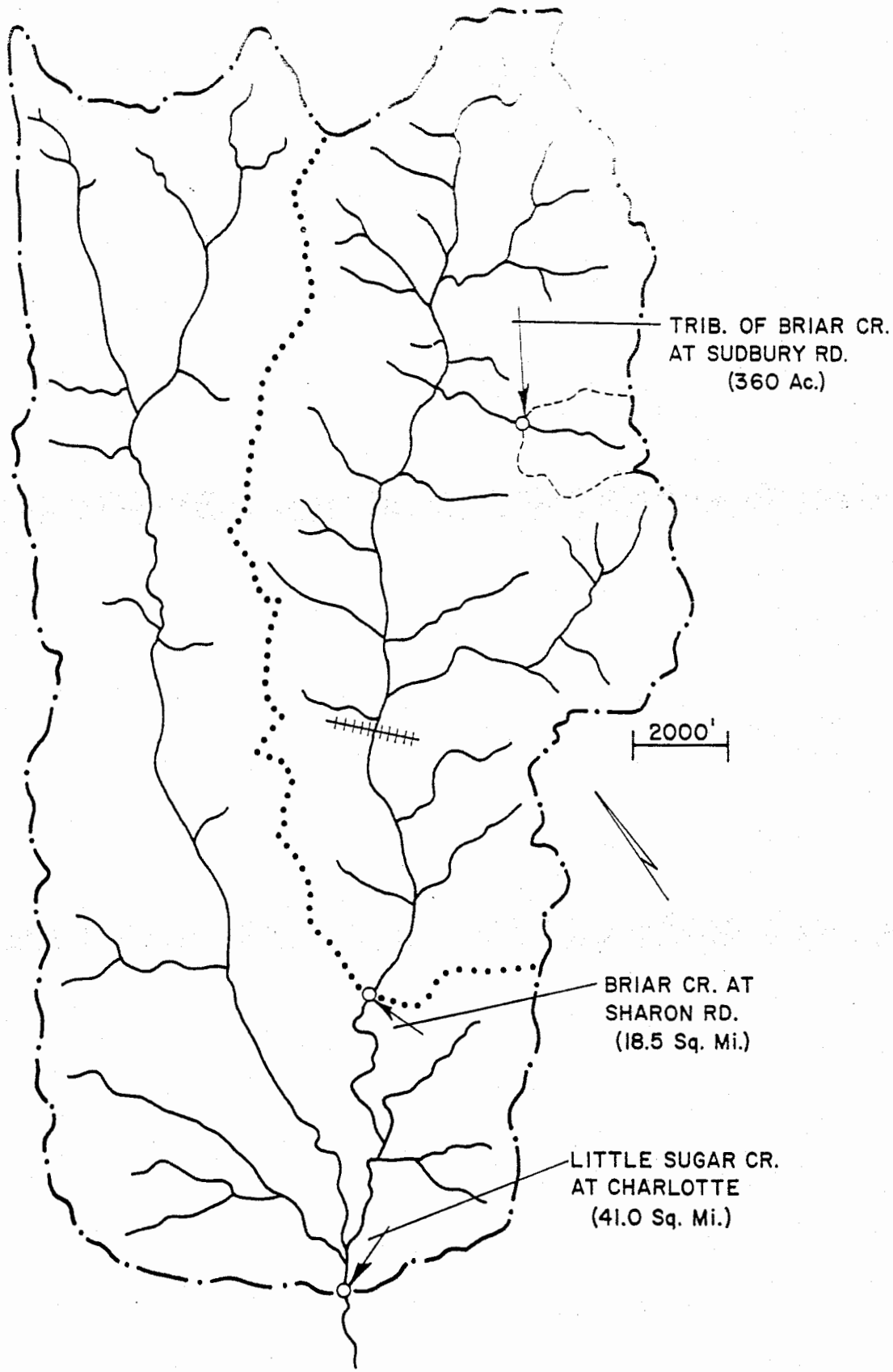


FIG. I CHARLOTTE WATERSHEDS

2. The Sudbury Watershed. MITCAT was applied to the watershed of the unnamed tributary of Briar Creek at Sudbury Road in Charlotte, N. C. (For convenience the watershed is called "Sudbury" throughout the report.) The drainage area is approximately 360 acres at the gaging station, which is located at latitude 35°-13'-26" and longitude 80°-46'-01". The topography is rolling. Overland flow slopes used in the model vary from 0.012 to 0.11 averaging 0.04. The mainstem of the stream system falls 80 feet in the 5400-ft. distance from the most remote ridge point to the outlet. The soils are of the Cecil classification. The watershed is almost fully developed at about four dwellings per acre, with some scattered commercial and institutional usage. Imperviousness ranges from 15 to 30 percent among the catchment elements, averaging 19 percent. The watershed is shown in Fig. 2.

Four storms were selected from the record for calibration/validation. Three were used for calibration having been chosen for their severity and temporal pattern. The dates and total rainfall for the calibration storms are: 9 June 68, 1.71 in.; 23 Aug. 67, 1.84 in.; and 23 Jul 70, 1.38 in. They are listed in descending order with respect to peak flow. The single validation storm occurred 10 Aug. 67, with 1.47 in. of rainfall. These storms and their hydrographs are shown in Figs. 3, 4, 5, and 6, together with the modelling results.

In the early attempts to calibrate the model, the effect of temporary storage in the watershed was neglected. Using defensible values for infiltration parameters and for overland and channel roughness coefficients, computed hydrographs were obtained with significantly high peak flows and early times-to-peak. Peaks were delayed in time by using larger Manning coefficients, and peak flows were reduced by adjusting the infiltration coefficients. The peak coordinates of the computed and observed hydrographs could not be aligned in this manner without losing agreement of computed and observed surface-runoff volumes. Further, the parameters of hydraulic roughness and infiltration necessary to align the peaks were absurd.

The hydraulic roughness and infiltration parameters were returned to defensible levels, and temporary storage was considered. Ten culverts were modelled as reservoirs based on field observations of hydraulic performance of the culverts and map observations of the stage-storage relationship. The results were much improved. Further fine tuning of roughness parameters yielded the final calibration results shown in Figs. 3, 4, and 5.

For the validation run, the input data were prepared using all parameters set in the calibration process. Only rainfall and antecedent moisture conditions were made specific to the storm of interest. Validation results, which may be taken as an estimate of model fidelity, are shown in Fig. 6.

In the Sudbury modelling experience, the importance of the effects of infiltration and temporary storage on the size and shape of the hydrograph is demonstrated. In MITCAT, optional infiltration submodels are included based on the commonly used Horton, SCS, and Holtan techniques. Of these three, only the Holtan method (Holtan, 1961) could be applied satisfactorily by this investigating group. The Holtan expression is:

$$F = a F_p^{1.4} + fc$$

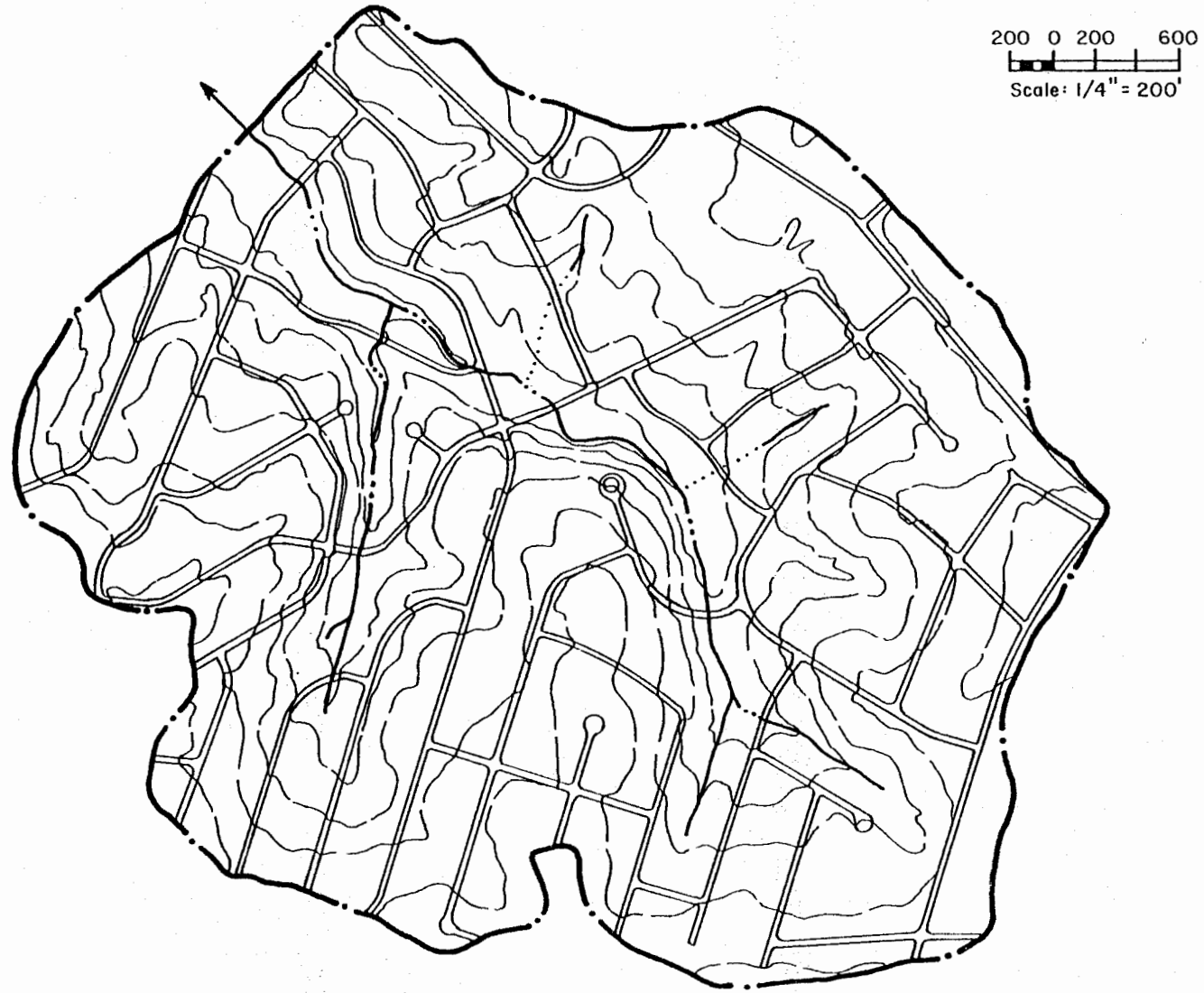


FIG. 2 WATERSHED OF TRIBUTARY OF BRIAR CREEK  
AT SUDBURY ROAD, CHARLOTTE, N.C. (360 Ac.)

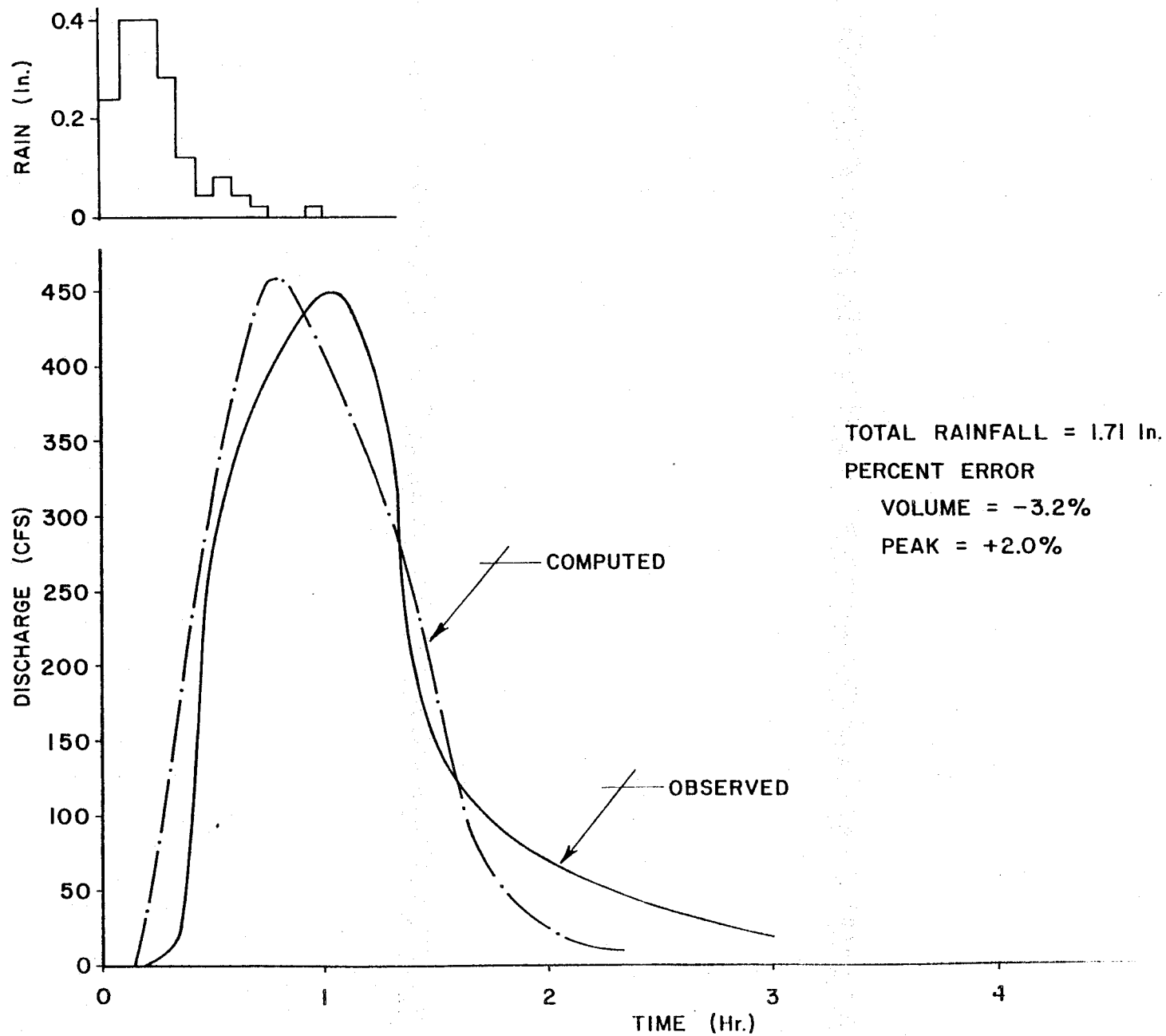


FIG. 3 STORM HYDROGRAPH  
TRIBUTARY OF BRIAR CREEK AT SUDBURY ROAD  
JUNE 9, 1968

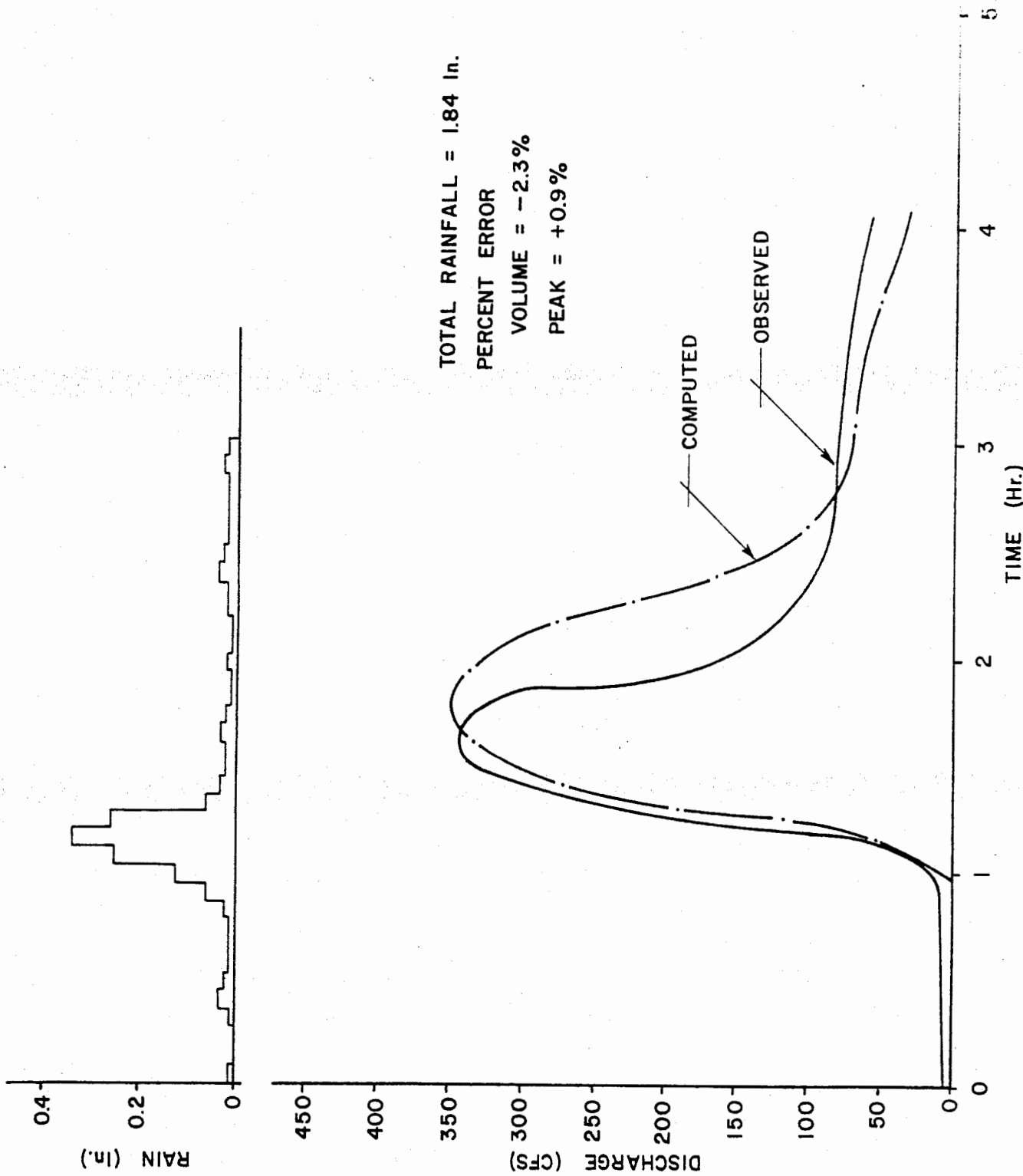
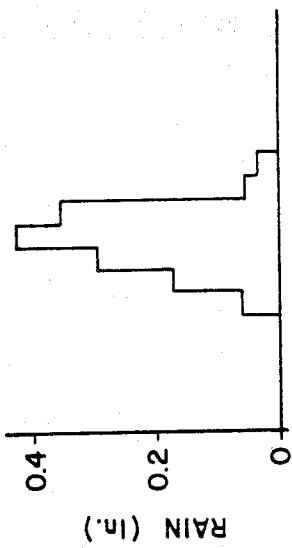


FIG. 4 STORM HYDROGRAPH  
 TRIBUTARY OF BRIAR CREEK AT SUDBURY ROAD  
 AUGUST 23, 1967



TOTAL RAINFALL = 1.38 in.  
 PERCENT ERROR  
 VOLUME = -4.7%  
 PEAK = +7.4%

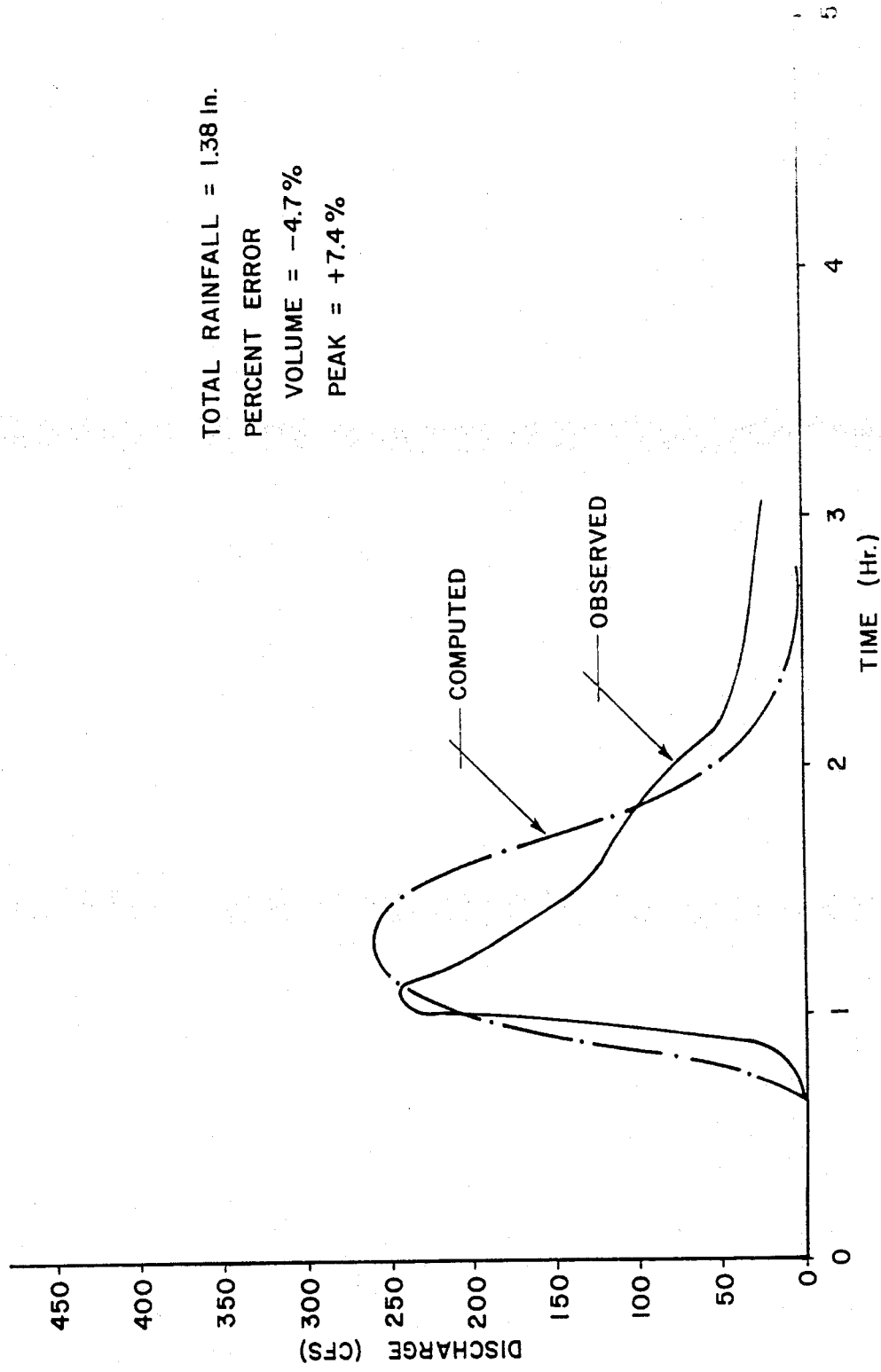
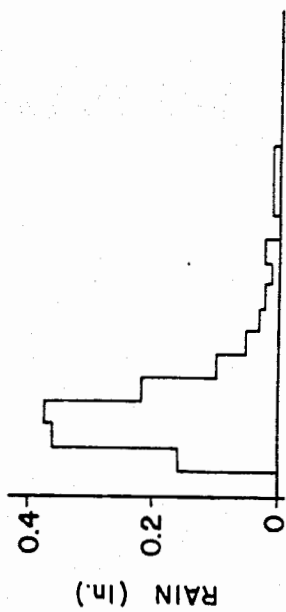


FIG. 5 STORM HYDROGRAPH  
 TRIBUTARY OF BRIAR CREEK AT SUDBURY ROAD  
 JULY 23, 1970



TOTAL RAINFALL = 1.47 in.  
 PERCENT ERROR  
 VOLUME = -2.9%  
 PEAK = +10.5%

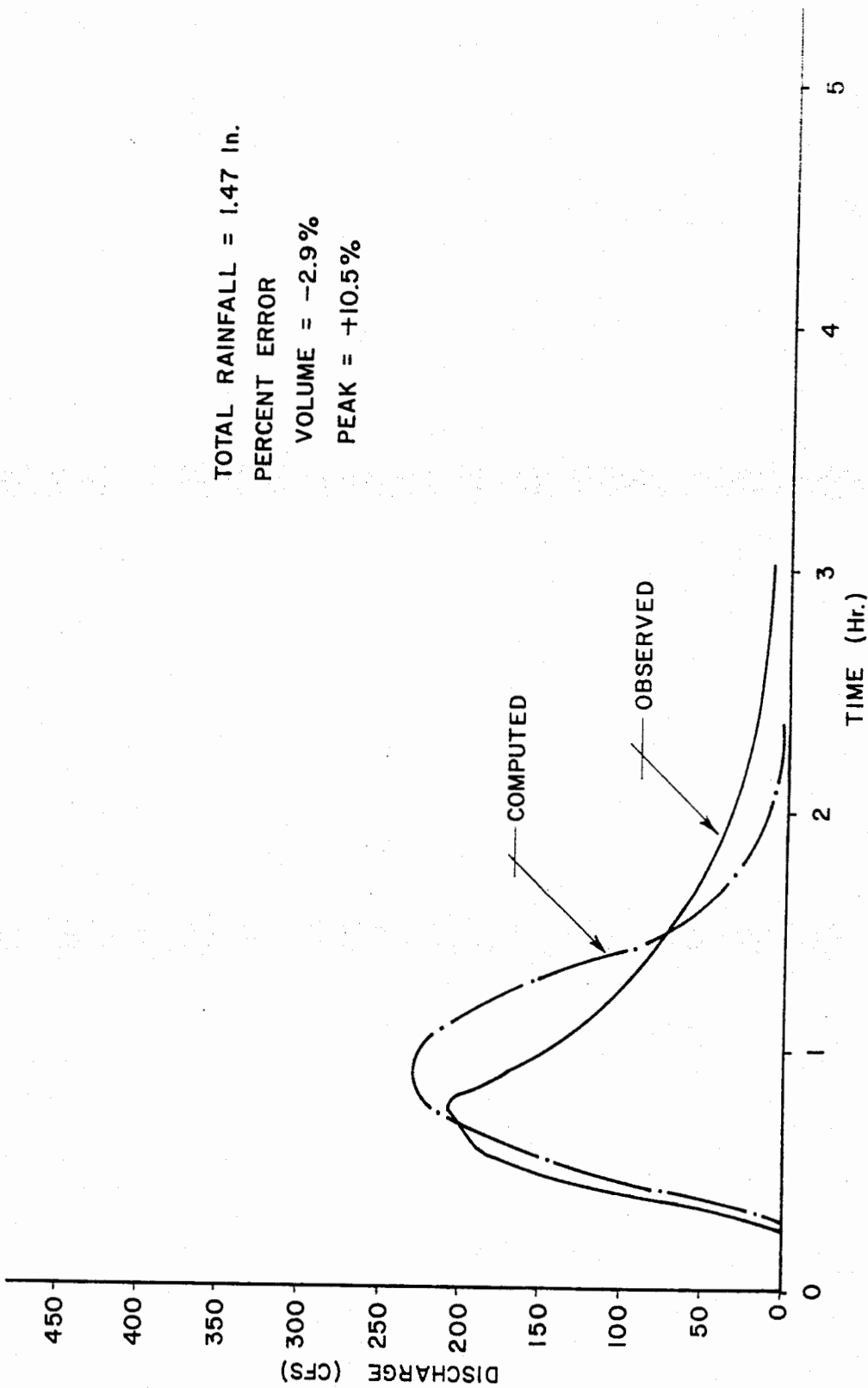


FIG. 6 STORM HYDROGRAPH  
 TRIBUTARY OF BRIAR CREEK AT SUDBURY ROAD  
 AUGUST 10, 1967



where  $f$  = infiltration capacity (in./hr.)

$a$  = index of surface-connected porosity

$F_p$  = available storage in the soil layer/s (inches)

$f_c$  = constant rate of infiltration after prolonged wetting (in./hr.)

The upper soil layer is viewed as a reservoir receiving the infiltrated water from above and passing it to the soil below at a rate,  $f_c$ . The index of surface-connected porosity has been viewed as being seasonally influenced. The available storage at the beginning of the storm is set by consideration of the soil type and the antecedent moisture conditions. In the model, the Holtan equation is used to conduct a mass balance of infiltrated stormwater, and  $F_p$  is updated after each time interval. The value  $f_c$  can be estimated as the limiting infiltration capacity of the lower soil layers.

It was found that the model results were quite sensitive to the parameters selected for  $f_c$  and the initial values of  $F_p$ . It can be observed readily that for relatively impervious soils the value of  $f_c$  is quite small so estimates of upper-zone soil storage are crucial. In more pervious soils, the distinction between water held in upper-zone soil storage and that infiltrated to lower zones is less pronounced so some calibrating flexibility exists in defining this condition. For the Charlotte watersheds, SCS soil surveys augmented by field observations of topsoil depth were used to obtain estimates of  $f_c$  and dry-weather values of  $F_p$ . The work of Lutz was used for inferential estimates of the effect on the initial value of  $F_p$  of antecedent moisture conditions (Lutz, 1970).

The lack of a reliable infiltration submodel with which modelling results may be reproduced and cross-validated by different investigators is a formidable impediment to the regular use of watershed models for planning and engineering purposes. It is a fertile topic for further research.

It was an early finding that existing temporary storage in the watershed must be included for reasonable validation of the model. There are at least two implications of this on watershed management. The first is that temporary storage in urban watersheds is an important part of the input data set. From studies later to be discussed, it may be judged that storage facilities near the outlet on major tributaries have the greatest effect. The second implication is that if temporary storage is important in model validation, it must likewise be an important fact of watershed behavior, especially from the standpoints of flooding and streambank degradation. Considerable attention is paid to this in the following studies.

3. The Watershed of Briar Creek at Sharon Road. MITCAT was also applied to the much larger watershed of Briar Creek at the Sharon Road crossing where the watershed area is about 18.5 square miles. The gaging station is at latitude  $35^{\circ}-10'-47''$ , longitude  $80^{\circ}-49'-46''$ . The stream was gaged from April 1962 to March 1973. For part of this time hydrographs and multiple rainfall observations were obtained at five-minute intervals during storms. The Sudbury watershed is a typical subcatchment of the Briar-Sharon watershed. Slopes of the various stream elements range from 0.002 to 0.08. The average slope of the mainstem is approximately 0.0028. The soils are of the Cecil and Cecil-Urban groups. It can be observed that disturbance of these soils in the urbanization process generally reduces the upper-zone storage capacity markedly, even in

those areas which would not be counted as being impervious in the determination of the percent imperviousness. Impervious cover ranges from 2 to 45 percent. The watershed is shown in Fig. 1.

The calibration storms for Briar at Sharon occurred on the dates of 7 Jun 68 and 12 Mar 68. Rainfall for these storms were observed at as many as six points in and near the watershed. The spatial variation of rainfall was taken into account within the model. The validation storm occurred on 23 Aug 67.

Results of the calibration/validation efforts are shown in Figs. 7, 8, and 9. Validation of the model for the watershed of Briar Creek at Sharon Road was not as good as that of the Sudbury watershed. The reasons for the poorer response are not altogether clear. Certainly, spatial variation in rainfall that was not detected in the coarse rain-gage network is one factor. The inadequacy of the infiltration submodel to reproduce the surface runoff history is another. There was evidence of numerical instability of the kinematic-wave procedure within the model for some runs. Consultation with the author of the model confirmed that there had been some difficulty with that routine in other applications and that subsequent versions of MITCAT have built-in checks and remedies for the condition. Given scarce time resources and abundant unrelated calibrating problems, the investigator did not pursue the question of numerical instability.

Although validation of the model was not as close as desirable for the larger watershed, it was sufficient to conclude that trends in hydrograph shape, given changes in watershed storage, were discernible. Specific values for stage and flow yielded by the model would not be valid without further improvement in the modelling effort.

Conclusions regarding the effect and distribution of storage in the watershed will be presented in more detail subsequently.

#### 4. Summary of Findings on Model Applications

- a. Watershed simulation models are not yet ready to be used generally by cities and other local jurisdictions because of their capriciousness, gargantuan appetite for data and file maintenance, and because of some remaining gaps in logic and means of data acquisition.
- b. Model selection and data acquisition should follow careful study of the capabilities of various models and the data required for calibration and validation. The purpose to which the model is to be put and the cost of data gathering and file maintenance ought to govern the choice.

Further work is needed before models can become regular and reliable tools for watershed management. Now, model development has progressed to the point where the necessity for further refinements in measuring watershed parameters has stalled the current state of the art. More precise descriptive submodels of the hydrologic abstractions, such as infiltration and depression storage, are needed. Subsequently, uniform methods and tests are needed for the measurement of input parameters so that results may be reproduced by independent observers.

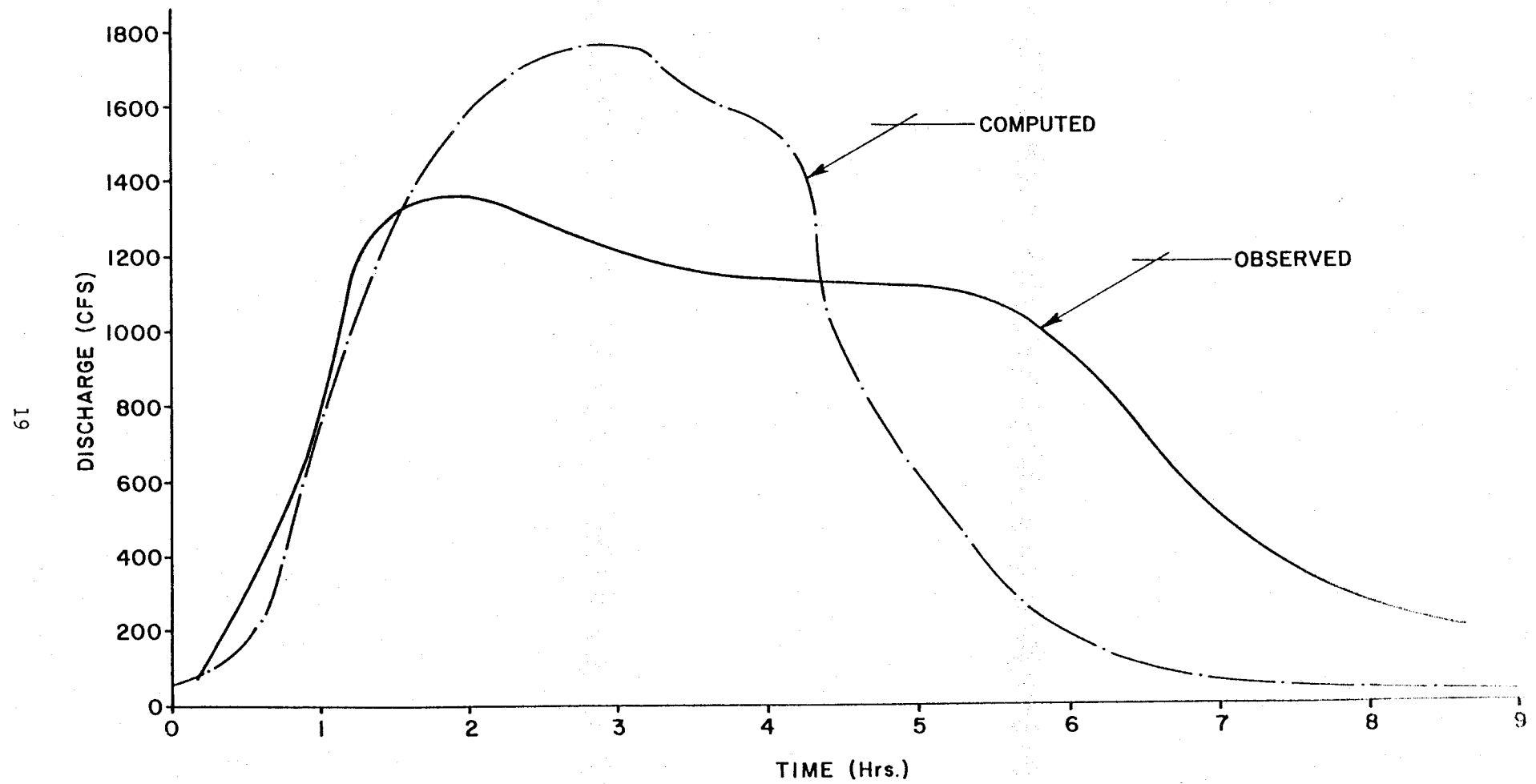


FIG. 7 STORM HYDROGRAPH - JUNE 9, 1968  
BRIAR CREEK AT SHARON ROAD

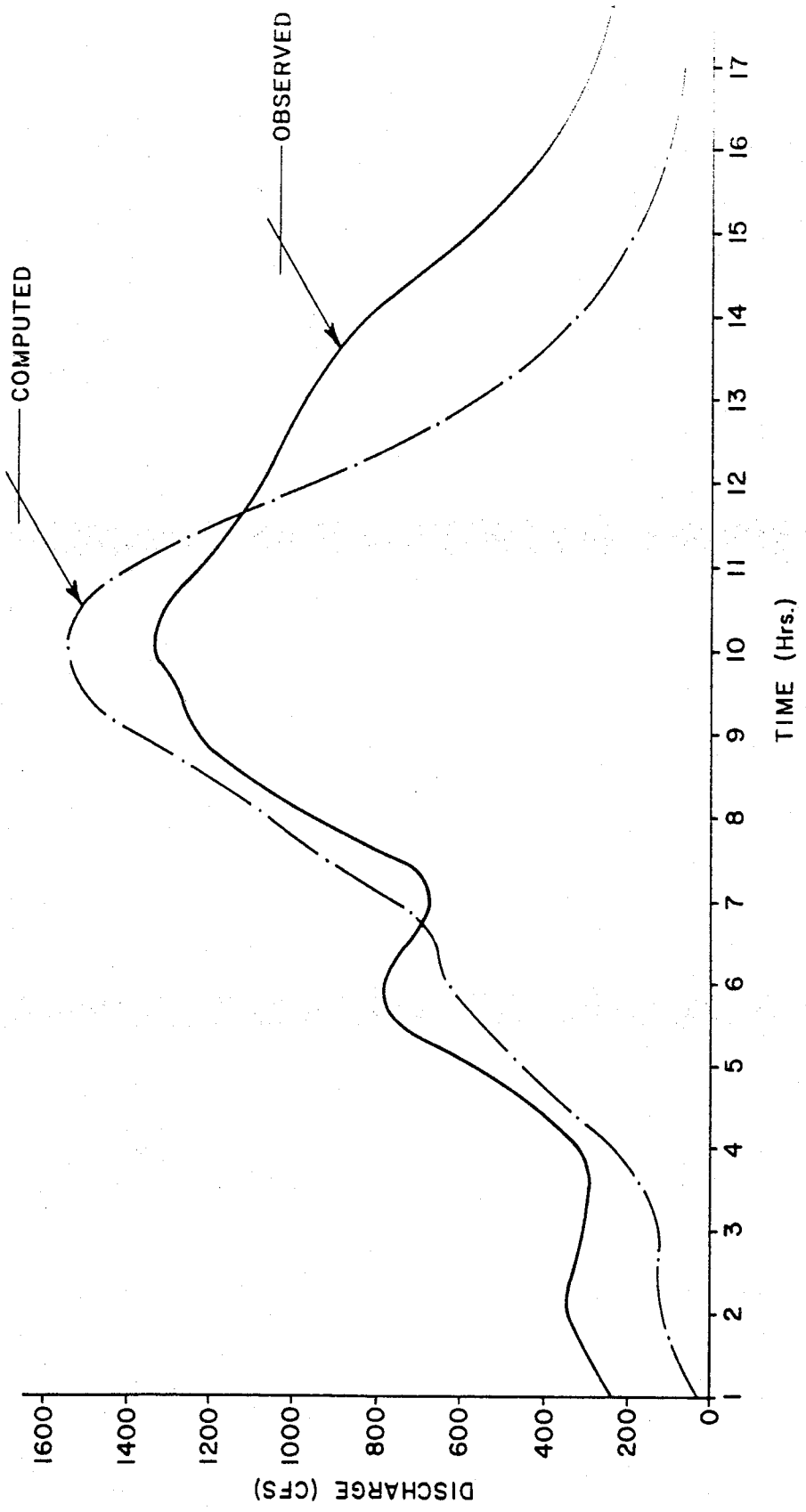


FIG. 8 STORM HYDROGRAPH - MARCH 12, 1968  
BRIAR CREEK AT SHARON ROAD

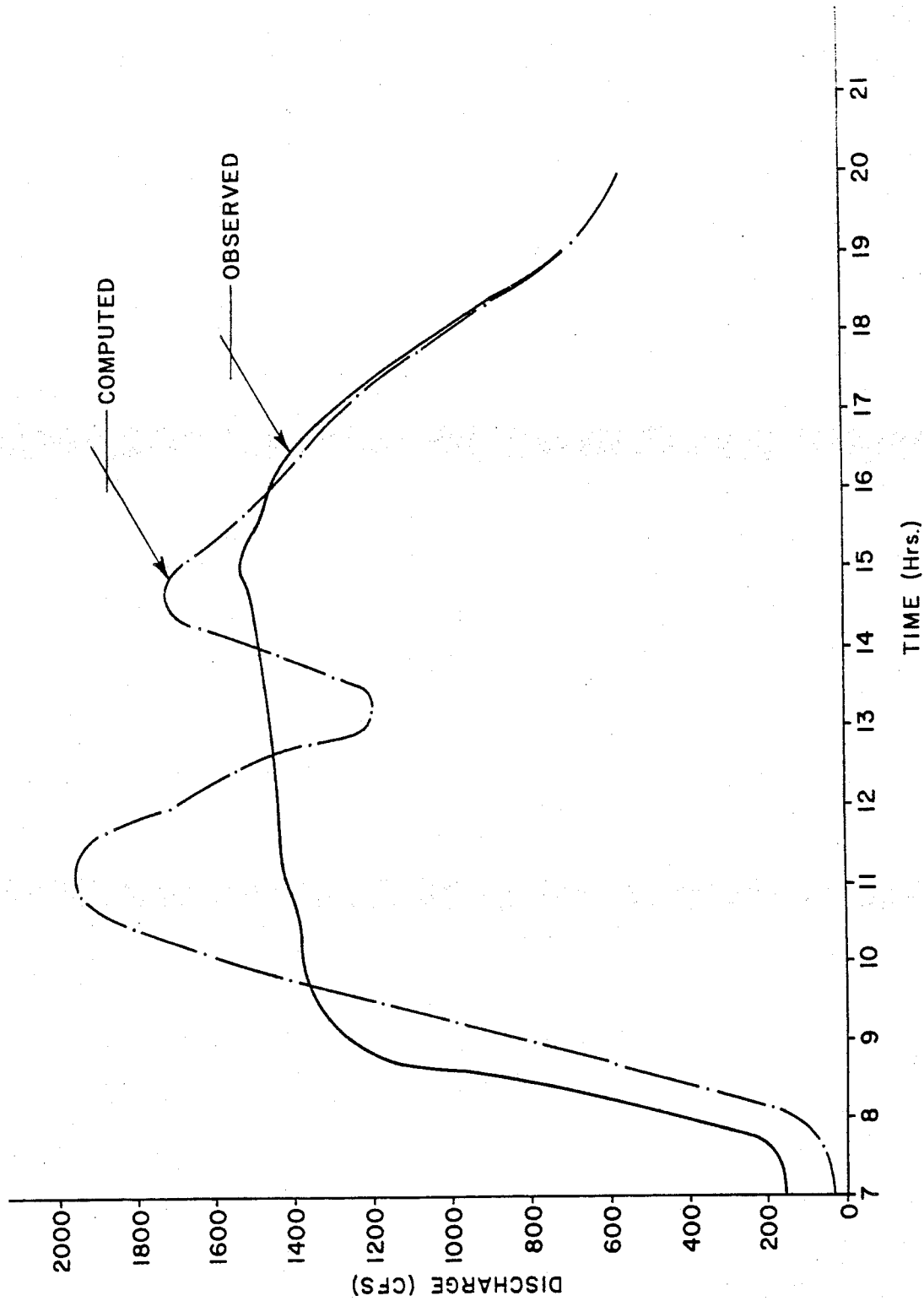


FIG. 9 STORM HYDROGRAPH - AUGUST 23, 1967  
BRIAR CREEK AT SHARON ROAD

Model developers should account for downstream effects in routing algorithms, particularly for tailwater submergence of culverts and for backwater in tributaries caused by high stages in the mainstems into which they empty. Many urban streams have significantly wide flood plains. It is unclear in the stream-segment-routing procedures of many models whether the storage effect of the flood plain in damping the hydrograph is adequately modelled.

The rainfall/runoff data base is absolutely necessary for calibration and validation of any model. Rather than to advocate widespread gathering of such data, the investigator recommends that coordinated rain and streamflow gages be installed where the need for modelling is established. Adequate lead time would have to be allowed to gather a sufficient set of storms, and continued operation of the gages is recommended where models are to be used for management decisions.

## SECTION IV

### MODEL STUDIES OF URBANIZATION AND DETENTION

The MITCAT model, as calibrated and validated for the two study watersheds, was used to study the effect on hydrograph shape of existing storage in the watershed and of new storage that might be added in response to new stormwater detention policies.

#### A. EFFECT OF EXISTING STORAGE ON DISCHARGE

In the description of the calibration process, it was reported that it was necessary to include the principal culverts and bridges as storage facilities in order to be able to calibrate the model adequately with defensible values of independently observable parameters. This led naturally to the question of the extent to which existing structures in conventionally developed watersheds serve to counter the effect of urbanization on storm flows.

1. Basis of the Study. The watersheds were studied in a historical storm and under conditions intended to simulate a ten-year event. The hyetograph for the ten-year event was contrived such that for any duration the accumulated rainfall approximates the ten-year values given in U. S. Weather Bureau Technical Paper 40 extrapolated to include durations less than thirty minutes using Technical Paper No. 25 as a guide. The hyetograph, shown in Fig. 10, has the property of being a ten-year rainfall for any selected elapsed time, up to four hours. Thus, the storm should produce a peak which crudely approximates the ten-year flow event for the larger watershed and any subcatchment. The investigator is aware of the arguments for and against the use of hyetographs of one pattern or another to produce a given discharge event and of the fact that the return period of the flood is not necessarily or even usually equal to that of the rainfall that produced it. But here, the motive for the study is to examine differences in behavior of a given watershed and its subcatchments in a common storm under differing development conditions, so the crudeness of the approximation is acceptable. In point of fact, the peak computed in the model for the smaller watershed is well within the range of estimates produced independently by the Rational Method, the Soil Conservation Service Small Watershed Method and the statistical analysis reported by USGS (Putnam, 1972).

For the historical storm on the Sudbury watershed and the ten-year storm on both watersheds, three watershed development conditions were simulated. The present condition of development was taken as a base. Then, the bridges and culverts were removed from the model to assess the effect of those structures on the hydrograph. Finally, the watershed was "de-urbanized" by returning the percent imperviousness to zero, roughening the overland flow n-value and adjusting infiltration parameters to reflect the existence of a permeable forest floor. The bridges and culverts were left out. This is not to say that the actual rural condition was being simulated, for the real data for that condition are not to be found. Rather, it is a probable rural condition, a situation that might have been.

2. The Sudbury Watershed. For the 360-acre watershed, the effect of removing nine culverts is shown in Fig. 11 along with the computed and observed hydrographs with the culverts incorporated, for the storm of 9 June 68. The

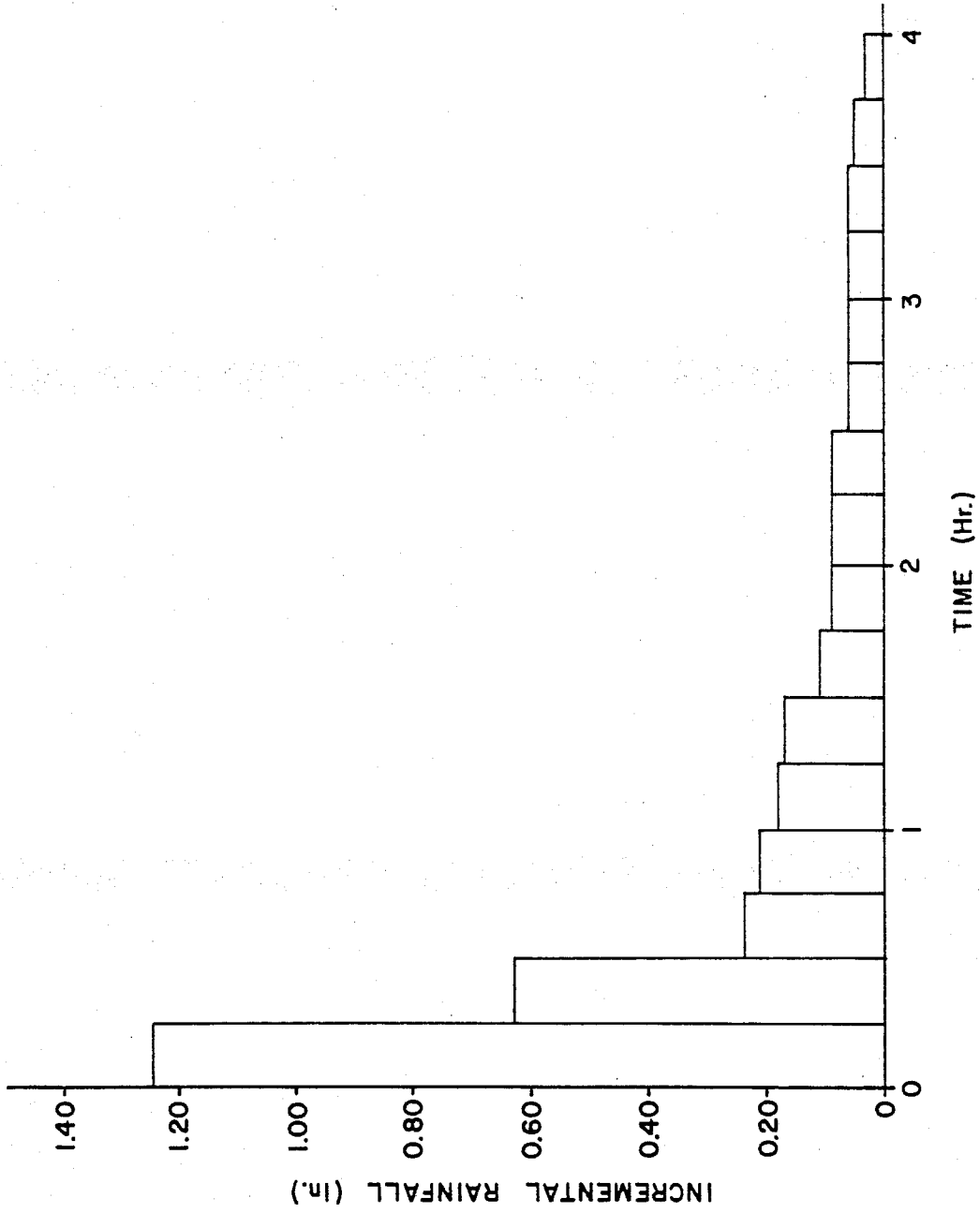


FIG. 10 TEN-YEAR DESIGN HYETOGRAPH FOR CHARLOTTE, N.C.



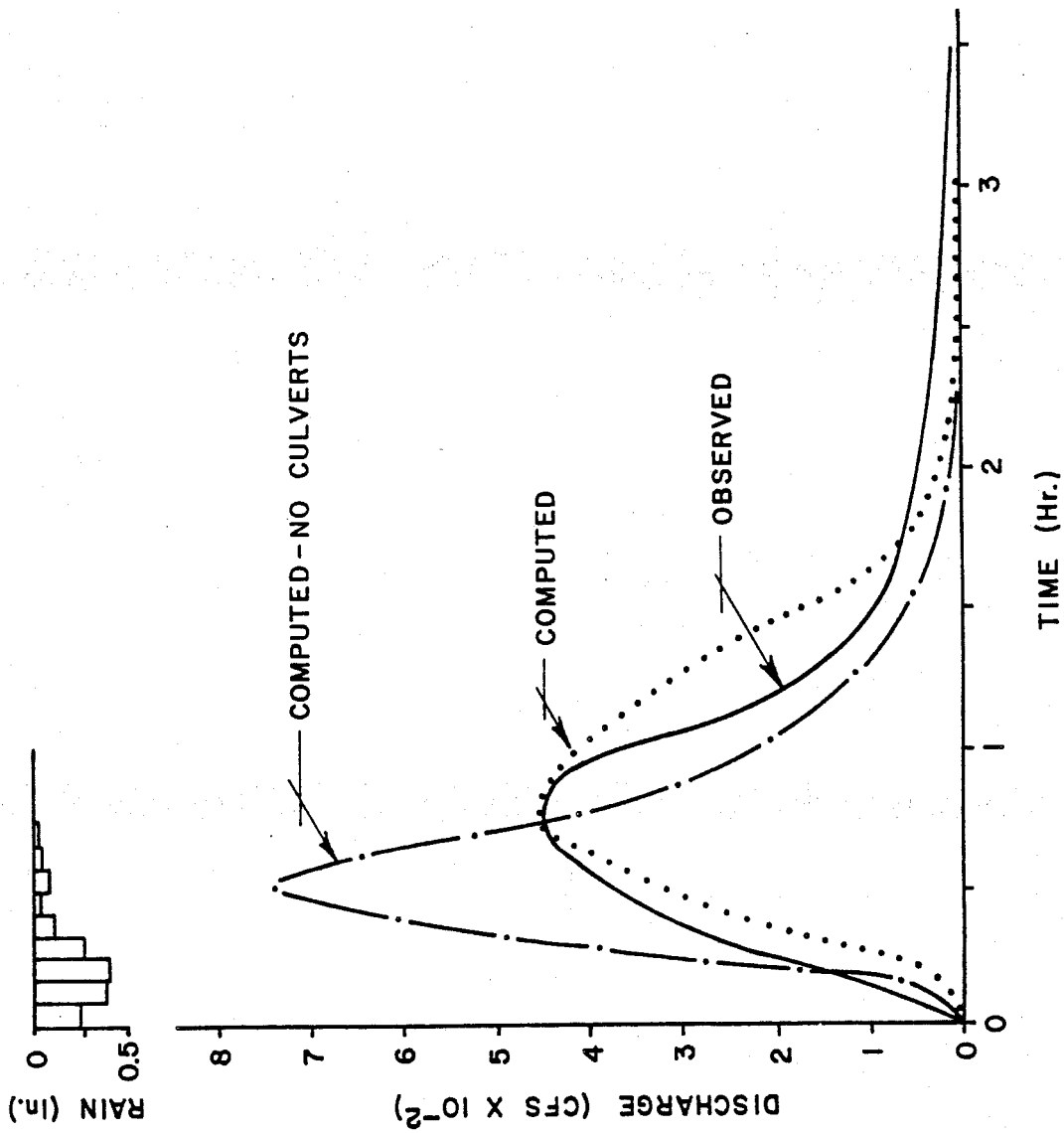


FIG. 11 STORM HYDROGRAPH  
 TRIBUTARY OF BRIAR CREEK AT SUDBURY ROAD  
 (360 Ac.)  
 JUNE 9, 1968

storm was something less than a five-year storm. The results indicate that these structures collectively held the peak to about 450 cfs as compared to the potential 740 cfs which might have occurred had they not been in place, a reduction of 40 percent.

The ten-year storm hyetograph was passed through the watershed under "rural" conditions, present conditions and present conditions without culverts, as described above. The results, given in Fig. 12, indicate that urbanization might have caused peaks to increase from about 500 cfs to a potential of 900 cfs, and that the culverts held the increase to about 650 cfs. The potential increase of 80 percent was held to 30 percent.

Different investigators would, without doubt, obtain different numerical results from similar analyses, but the results would still lead to the conclusion that the existing culverts are providing significant peak reduction.

3. The Briar Creek Watershed (at Sharon Road). Similar studies were conducted on the less well validated 18.5 square-mile watershed. Accepting that the trends might be instructive in spite of the error of validation, the ten-year storm was likewise passed through the watershed under the same three conditions of development. The results, shown in Fig. 13, are more drastic. The large railroad fill and several roadway crossings immediately above the gaging site apparently provide the dominant control. Under ten-year storm conditions, the conjectural rural peak is about 3600 cfs, the potential urban peak is 4300 cfs and the effect of the culverts and bridges is to hold the peak to a protracted 2700 cfs. Were the model to be believed, the present peak is lower than the rural peak.

Can peak discharges decline with urbanization? There is some statistical evidence in the affirmative to be discussed in Section V.

4. Implications for Watershed Management. Apparently, under conventional urban development conditions in Charlotte, stormwater detention has been built in unintentionally, and it is significant in reducing peak discharges in storms of return periods of five to ten years. It is readily observable in the watershed that the channels have deteriorated, and continue to deteriorate, in spite of the storage having been in effect. From the long flattened shape of the hydrograph of the larger watershed, observable both in the modelled storm here and in the recorded hydrographs given earlier, it may be seen that a large quantity of storage would be necessary to get further significant reductions in peak discharge.

The study also raises questions about the enlargement of culverts and bridges during thoroughfare improvements. Frequently, as street improvements are made, the old "undersized" culverts and bridges are made bigger to accommodate larger estimated discharges to reflect lower accepted design risks or to alleviate ponding conditions upstream. These enlargements are likely to remove significant storage from the system and to cause higher stages downstream for the same design storm. In a flood, relative to a given point on the stream, the water can be held temporarily upstream or it can be released downstream. Whether one or the other is objectionable depends mainly on local circumstances. But the street culvert is a main actor in urban watershed management. Considerable thought should be given to its effect elsewhere in the watershed.

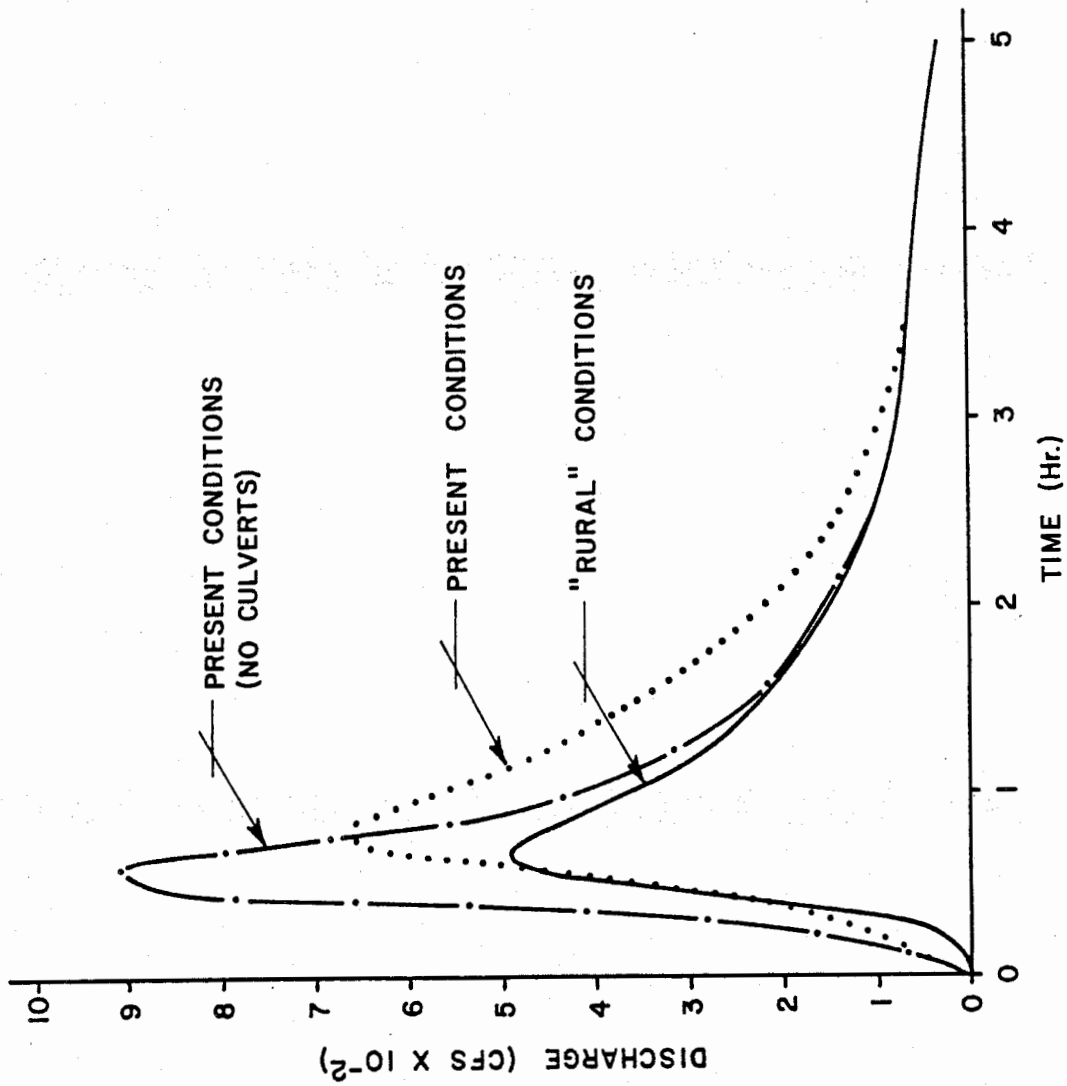


FIG. 12 10-YEAR STORM SIMULATION  
 TRIBUTARY OF BRIAR CREEK AT SUDBURY ROAD  
 (360 Ac.)

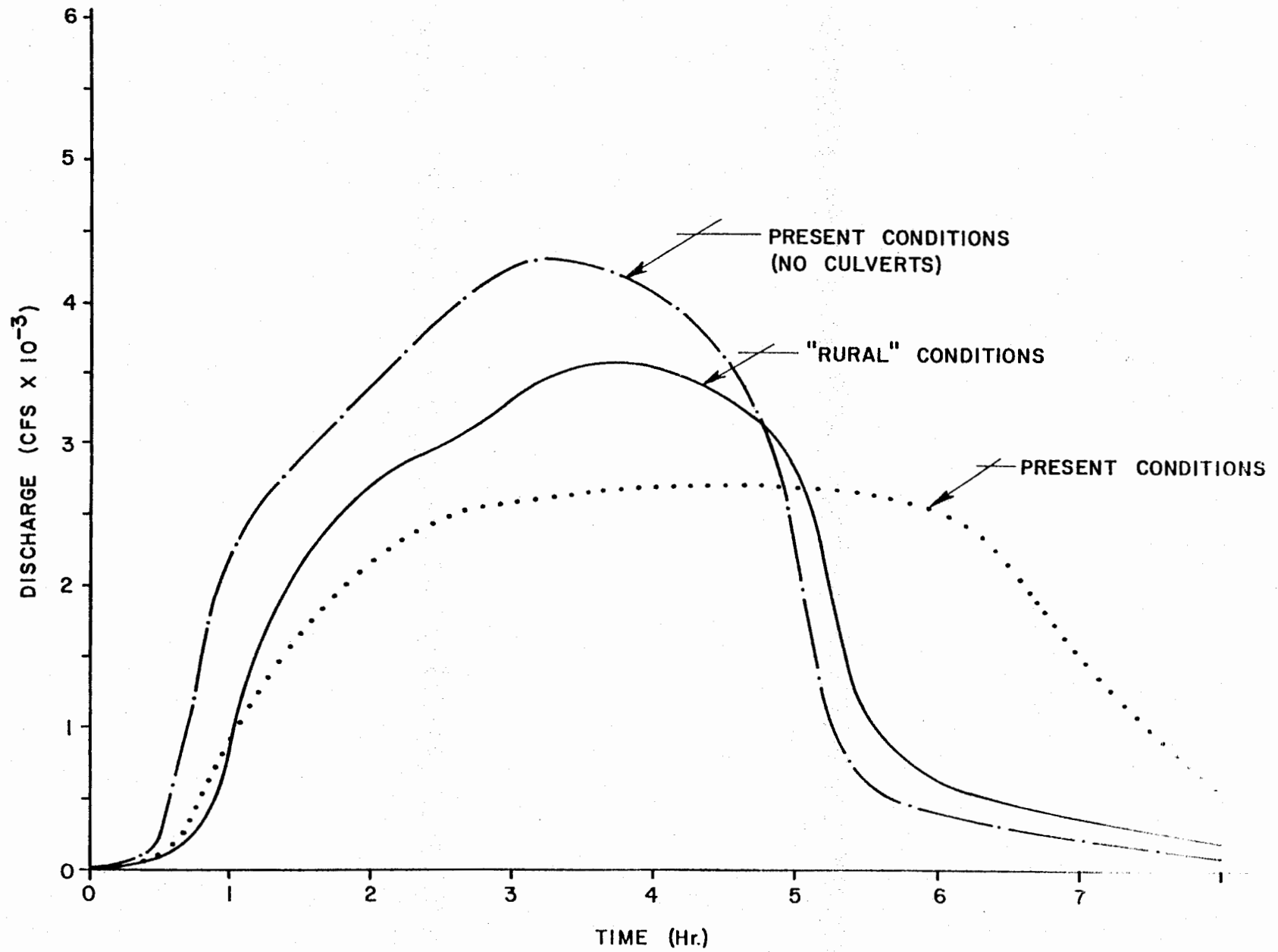


FIG. 13 10-YEAR STORM SIMULATION  
BRIAR CREEK AT SHARON ROAD  
(18.5 Sq. Mi.)

## B. EFFECT OF STORAGE ON DISCHARGE AT DOWNSTREAM LOCATIONS

To determine the effect of stormwater storage on discharge at locations some distance downstream of the storage location, the model was used to execute studies in the 360-acre Sudbury watershed and the 18.5 square-mile watershed of Briar Creek at Sharon Road.

1. Hypothetical New Development in the Sudbury Watershed. Two shopping centers were assumed to have been constructed in the upper reaches of the Sudbury Watershed, one at 30 acres, the other at 16.5 acres. Using the present conditions of the watershed as a base, the two shopping centers were included by adjusting the appropriate model elements to reflect decreased overland flow roughness, complete imperviousness and paved-channel delivery of runoff to the discharge point. Stormwater detention facilities were designed for each shopping center to control the runoff in the ten-year storm to an outflow equivalent to that from the same site with a rational runoff coefficient of 0.3. The model was then run to include the shopping centers without detention facilities and again to include shopping centers with detention facilities.

The rainfall used in the model was the "ten-year" declining-intensity storm discussed above. Passing this storm through one of the shopping centers yields the results given in Fig. 14. The effect, then, of storage on flows just downstream of the site is pronounced, as expected.

The results of the full watershed model runs are given in Fig. 15. At the 360-acre watershed outlet, about 4000 feet downstream from the shopping centers, little difference is detected by the model between any of the three situations: present conditions, shopping centers in with no control, and shopping centers in with adequate detention. Table 1 shows the specific peak-discharge values obtained from the model.

Table 1

### PEAK-DISCHARGE COMPARISONS OF SHOPPING CENTER STUDY

Location	Watershed area	Present conditions	Shopping centers without detention	Shopping centers with detention
Shopping Center A	30 ac	91 cfs	128 cfs	82 cfs
Shopping Center B	16.5 ac	52 cfs	72 cfs	51 cfs
Watershed Outlet	360 ac	665 cfs	688 cfs	642 cfs

2. Hydrograph Response at Sharon Road to Storage at Sudbury. The larger watershed, Briar Creek at Sharon Road (18.5 sq mi), and its subwatershed, Tributary of Briar Creek at Sudbury Road (360 ac or 0.56 sq mi), were used to study the effect of a single storage facility on peak discharge well below the point of storage.

"DARBROOK" SHOPPING CENTER

SUDBURY BASIN  
10-YEAR DESIGN STORM  
DETENTION FACILITY

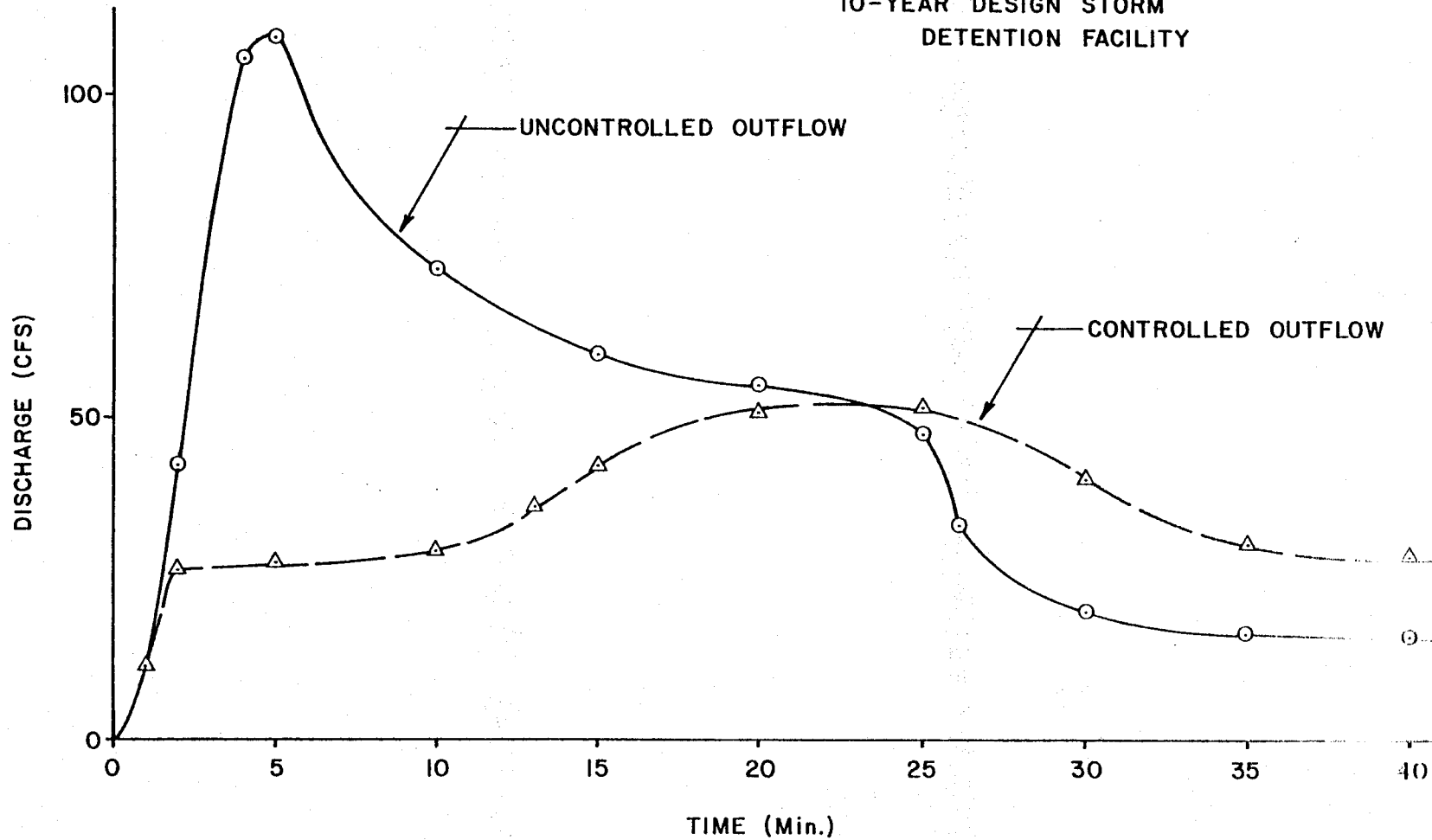
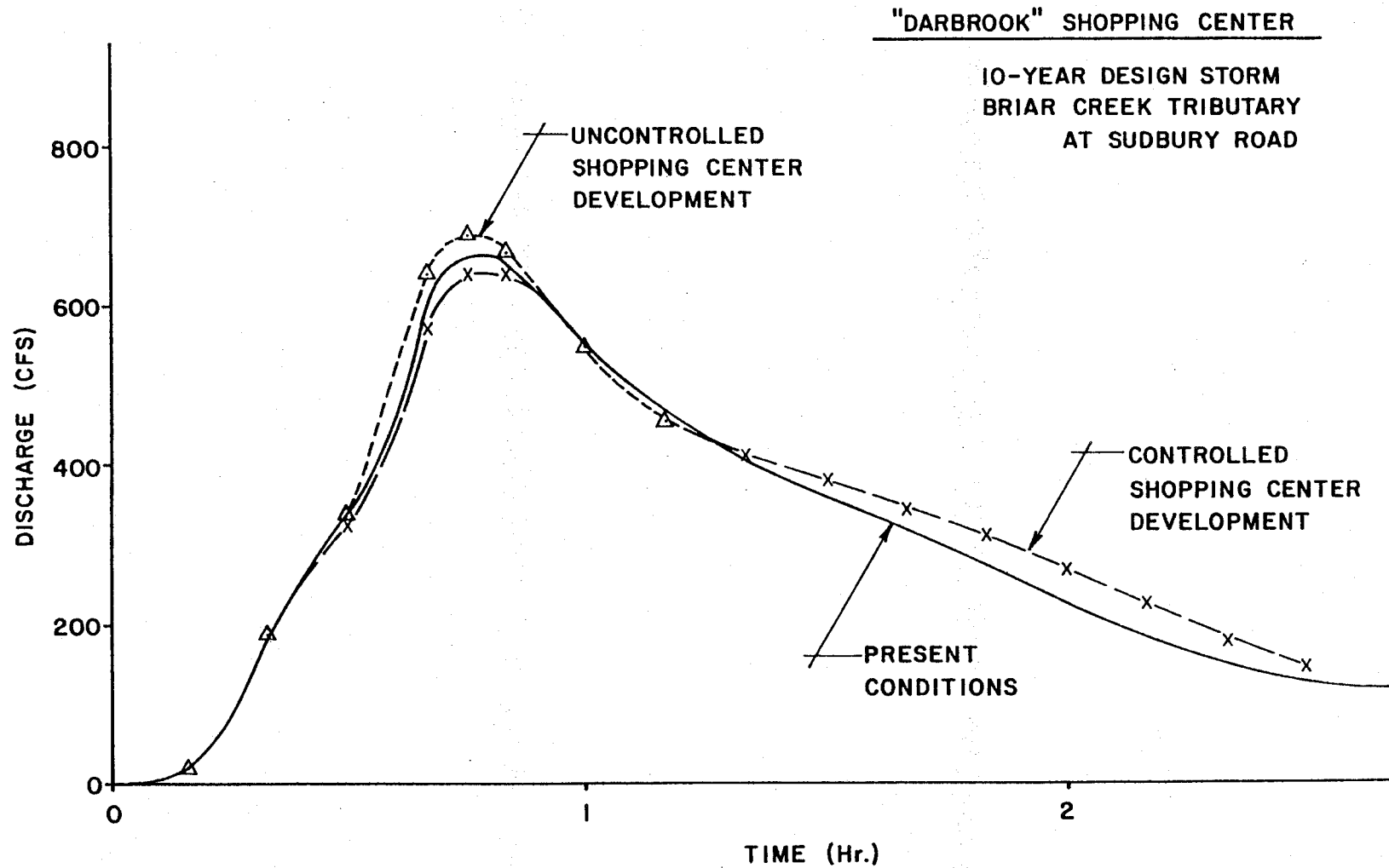


FIG. 14 EFFECT OF DETENTION ON DISCHARGE IMMEDIATELY BELOW HYPOTHETICAL SHOPPING CENTER



**FIG. 15 EFFECT OF UPSTREAM SHOPPING-CENTER DETENTION ON DISCHARGE AT WATERSHED OUTLET**

A detention facility was included at Sudbury Road to reduce the peak discharge at Sudbury Road from 684 cfs to 230 cfs when the ten-year declining-intensity storm was passed through the system. This is roughly equivalent to designing for an effective reduction of the Rational runoff coefficient from 0.6 to 0.2.

The storm was passed through the Sudbury watershed, with and without the detention facility, to obtain two hydrographs. In the model network of the 18.5 square-mile watershed, the Sudbury subcatchment element was removed and replaced by each of the hydrographs, using a feature of MITCAT which provides for hydrograph input to be made compatible in time with the synthesis of other discharges within the watershed. The results of the two runs are shown in Fig. 16.

In Table 2, computed discharge values are shown for various crossings of the mainstem along Briar Creek from Sudbury Road to Sharon Road. Examining the outflow values at each crossing, with and without detention at Sudbury, shows that there is a reduced effect of Sudbury storage as one proceeds downstream.

3. Storage-Location Strategies. The model was used to study storage-location strategies in the Sudbury watershed. As the study progressed, it became apparent that it was too site-specific to generalize the quantitative results. For purposes of the study, the Sudbury watershed was taken up to 30 percent imperviousness to represent a typical office-institutional development in the 360-acre watershed. It was presumed that outflow from the watershed, and perhaps within the watershed, was to be held to the equivalent of that from a wooded area. Systems of reservoirs were devised for the ten-year storm by desk-top preliminary design, then modeled for verification, according to the following strategies:

- a. Locate detention facilities wherever feasible to limit the outflow from each control point to that expected if the area were wooded.
- b. Locate detention facilities at a lesser number of selected points in the watershed deemed most suitable by virtue of their potential for storage, designed under the same specifications as (a), above.
- c. Locate one detention facility at the watershed outlet to protect the downstream area, designed under the same conditions.

Detention requirements for the design criteria to be satisfied by each of the strategies is given in Table 3. Hydrograph results are shown in Fig. 17.

In the design process, it was not possible to utilize existing storage locations behind culverts to meet the objective. Considerably more storage was required than was available. The storage provided would have to be made available by excavation. The reservoirs used in the study were vertical-sided depressions with essentially flat bottoms; hence, the area and depth requirements would seem to be near minimum. These depths are probably not acceptable in developed areas such as those assumed in the study.

A simplified desk-top study was executed to compare three general storage-location strategies under more tightly controlled circumstances. Storage-location strategies can be grouped in three main categories, illustrated as follows where it is desired to effect a 40 percent reduction in peak outflow at the watershed outlet:



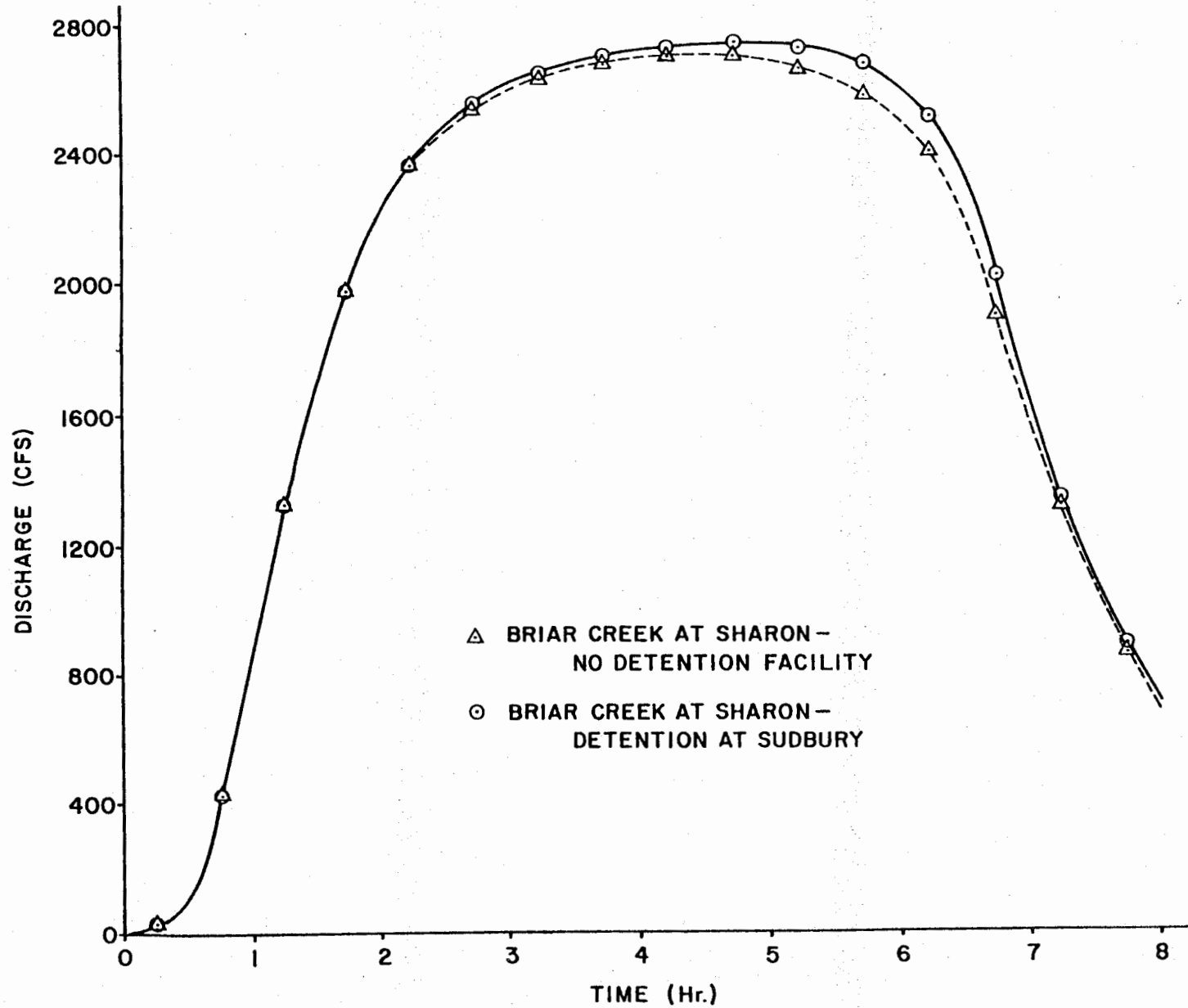


FIG. 16 HYDROGRAPH RESPONSE OF BRIAR CREEK AT SHARON ROAD TO DETENTION AT SUDBURY

Table 2

## EFFECT OF DETENTION AT SUDBURY ROAD ON PEAK DISCHARGE AT POINTS DOWNSTREAM

Crossing	With detention at Sudbury		Without detention at Sudbury		Change in Outflow	
	Inflow peak (cfs)	Outflow peak (cfs)	Inflow peak (cfs)	Outflow peak (cfs)	Net (cfs)	Relative (%)
Country Club Dr.	2168	2012	2014	1911	101	5.0
Central Ave.	2536	2513	2316	2297	216	8.6
Commonwealth Ave.	2640	2580	2439	2401	179	6.9
Railroad	3185	2635	3153	2542	93	3.5
Providence Rd.	2781	2771	2733	2723	48	1.7
Sharon Rd.	2773	2754	2730	2711	43	1.6

34

Table 3

## DETENTION REQUIREMENTS IN STORAGE LOCATION STUDY OF SUDBURY WATERSHED

Strategy	No. of Reservoirs	Total storage Volume (ac/ft)	Total area Req'd (ac)	Avg storage Depth (ft)
a. Distributed storage	9	33	4.8	6.8
b. Selected sites	5	35	4.3	8.2
c. Single site	1	46	4.2	11.1

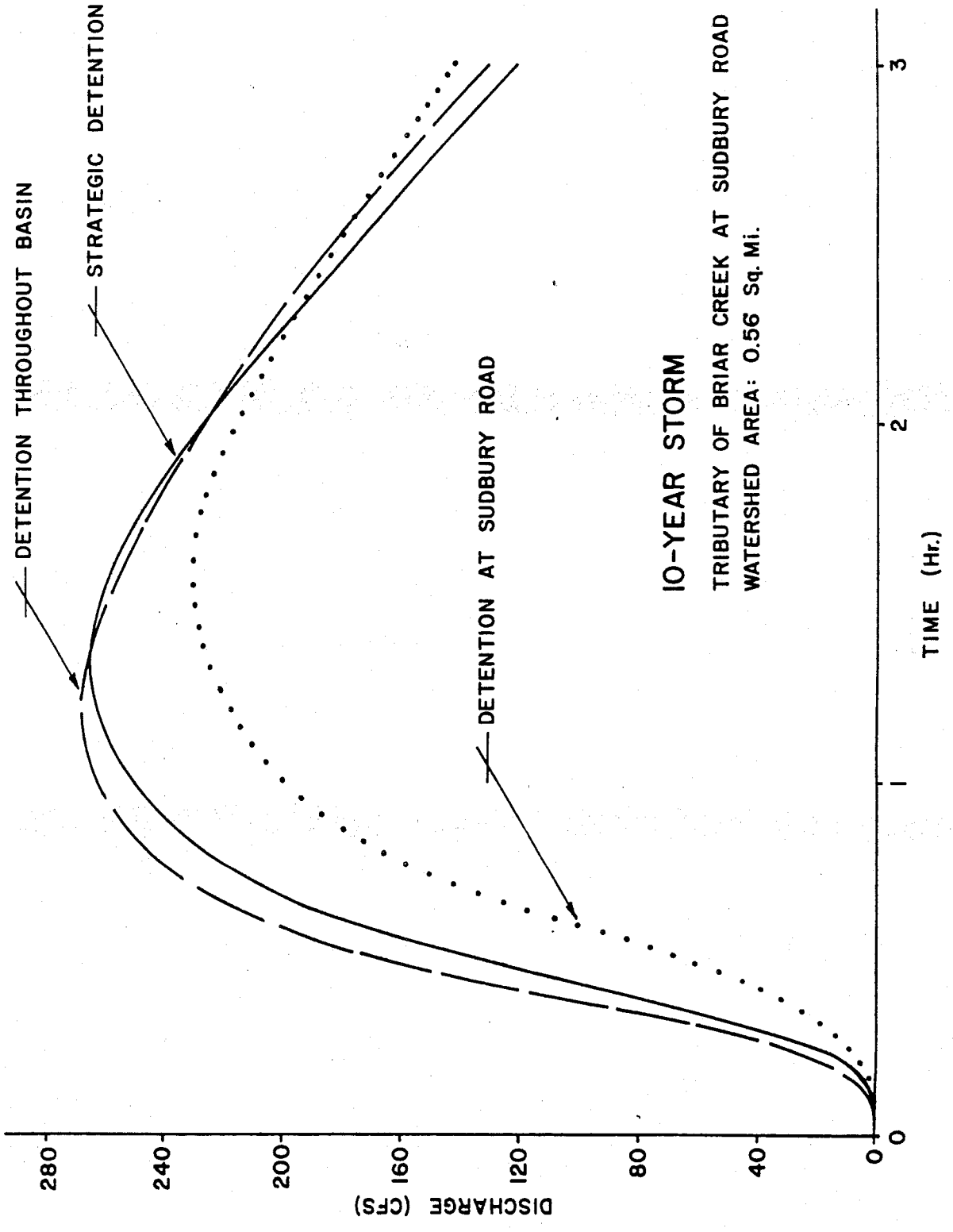


FIG. 17 EFFECT OF LOCATION OF DETENTION FACILITIES ON WATERSHED HYDROGRAPH

- a. Locate a single reservoir at the outlet designed for 40 percent reduction in peak discharge.
- b. Locate several reservoirs throughout the watershed, each designed to reduce the local peak by 40 percent.
- c. Locate reservoirs such that their aggregate contributory drainage area comprises 40 percent of the total watershed, and design each for virtually complete catchment of the floodwater volume.

A hypothetical watershed of 320 acres, subdivided into two watersheds of 130 acres (40 percent of total) and 190 acres, was selected as shown in Fig. 18. Reservoir Site A is at the outlet of the 130-acre watershed; Site B of the 190-acre watershed and Site C just below the confluence so as to be fed by the total watershed of 320 acres.

Hydrographs were formulated for each of the three sites in the same manner as those for the channel-impulse analysis of Section VI of this report. Based on a 6-hour, ten-year storm totaling 3.75 inches of rain and an SCS curve number of 75, hydrographs were fitted to the step-function such that the hydrographs at A and B sum approximately to the hydrograph at C, ordinate by ordinate. The same stage-storage expression was used for all three storage sites:

$$S = 500 Z^{3.25}$$

At locations A, B, and C, reservoirs were contrived to effect 40 percent reduction in peak outflow. At location A another reservoir was sized for 99 percent reduction in peak outflow. The elements of these designs are shown in Table 4.

Table 4

SUMMARY OF RESERVOIR DESIGNS FOR STORAGE  
LOCATION STUDY IN HYPOTHETICAL WATERSHED

Item	Location			
	A	B	C	A
Target Percent Reduction	40	40	40	99
Inflow Hydrograph				
Peak Discharge (cfs)	95	130	224	95
Peak Time (min)	148	158	150	148
Allowable Outflow (cfs)	57	78	134	1
Pipe Selection*	2 @ 20"	1 @ 32"	3 @ 23"	1 @ 3.5"
Routing Results				
Peak Outflow (cfs)	57	76	131	1.13
Peak Stage (ft)	7.17	8.15	9.43	10.73
Storage Volumes (ac ft)	6.92	10.49	16.86	25.65
Storage Area (ac)	3.14	4.19	5.81	7.77

\*The pipes were considered as culverts under inlet control. Non-standard sizes were used to obtain close agreement between target outflow and routed outflow.

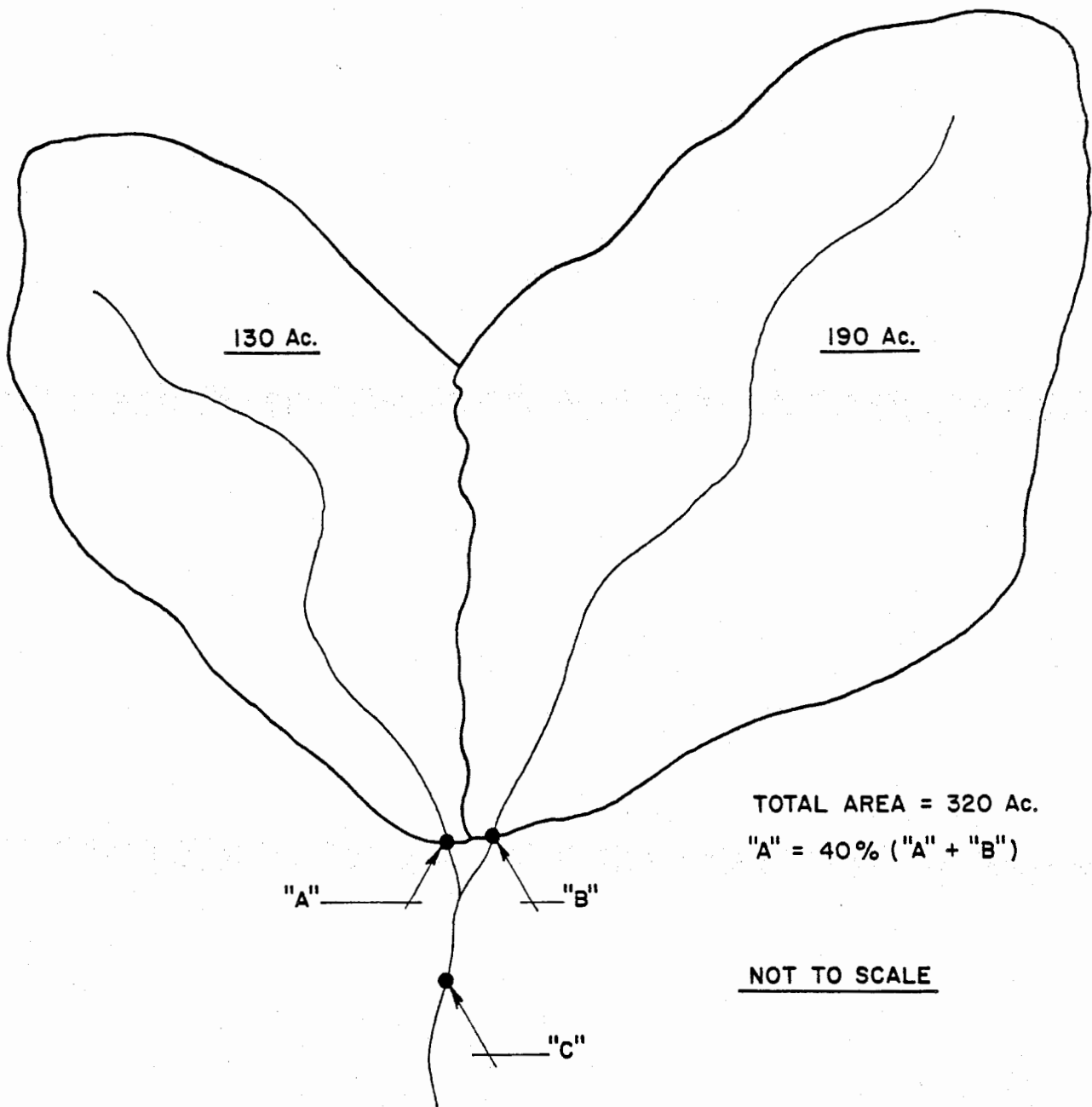


FIG. 18 HYPOTHETICAL 320-ACRE WATERSHED FOR STORAGE-LOCATION ANALYSIS

The effect of reservoirs at A and B was obtained by summing the outflow hydrographs, ordinate by ordinate. The effect of complete catchment at A is to deduct that hydrograph from the flow at C, which leaves the inflow at B as the only flow at C, effectively.

The results are summarized in Table 5.

Table 5

RESULTS OF STORAGE LOCATION STUDY IN HYPOTHETICAL WATERSHED

Item	Reservoir Location		
	A & B	C only	A only
Uncontrolled peak at C (cfs)	224	224	224
Controlled peak at C (cfs)	133	131	130
Percent Reduction	40.6	41.5	42.0
Volume Detained (ac ft)			
at A	6.92	0	25.65
at B	10.49	0	0
at C	0	16.86	0
TOTAL	17.41	16.86	25.65
Area Inundated (ac)			
at A	3.14	0	7.77
at B	4.19	0	0
at C	0	5.81	0
TOTAL	7.33	5.81	7.77

In this restricted study, the three strategies produce essentially the same effect at point C, about 41 percent reduction in peak discharge. The single-reservoir strategy requires least volume (but not much less than the two-reservoir schemes) and inundates significantly less area than either of the other schemes. The complete capture scheme appears to require significantly more storage.

The results are, of course, dependent upon the specific stage-storage relationships for the sites. But the volume-requirement comparisons ought to be generalizable to the extent of saying that the complete capture approach will require the most storage to achieve a given level of peak reduction and that the other two schemes will require about the same volume when the locations are near the outlet. It is also generalizable that one reservoir will require less surface area than two of the same total volume given generally observed stage storage functions. Reservoirs are essentially of the shape of inverted pyramids; a foot of depth up high will contain much more water than a foot down low.

Other studies have dealt with the effect on watershed outflow of distributing storage throughout the watershed at small scale. One (McKuen, 1974) shows that detention placed on tributaries near the watershed outlet can increase

peak outflow by delaying water which would have been discharged early in the hydrograph so as to coincide with the peak discharge. Another (Lumb, *et al.*, 1974) cites the fact that while storage reduces peak flow at a point, it does not reduce volume. Cumulative downstream effects make distribution of storage throughout the watershed less effective in downstream reaches.

Thus, it can be observed that the response of a watershed to randomly located storage facilities, as generated by a typical stormwater detention ordinance, is unpredictable. The location of storage facilities is important; the interaction of storage facilities one with another can be such that they are mutually supportive, damaging or that they counteract each other so as to have no discernible effect.

### C. SENSITIVITY OF DISCHARGE AND RUNOFF VOLUME TO WATERSHED IMPERVIOUSNESS

The soils of the Charlotte study watersheds are of the Cecil soil type. It is of interest regionally to study urbanization effects on other soil types. The Cecil soils are tight soils with shallow topsoils and relatively low capacity for upper-zone storage of infiltrated water. The rate of infiltration from the upper zone to deeper zones is also quite low. By contrast, a Norfolk soil, typical of the sandy soils of eastern North Carolina, is characterized as having a relatively high capacity for storage of water in the upper zone and a high rate of infiltration to deeper zones.

The model was used to examine the differences in response expected from Norfolk-type soils as compared to Cecil-type soils. The Sudbury watershed was used as calibrated for present conditions in the watershed. The storm was the ten-year declining-intensity rainfall. Percent imperviousness was varied from 2 to 98 percent in separate model runs. The peak discharge and runoff volume for each level of imperviousness were the parameters of interest.

For the Norfolk-soil sensitivity, the same runs were executed, except that the infiltration parameters were set to represent the sandy soil.

The infiltration parameters used for the two soil types are shown in Table 6. A more detailed description of the Holtan infiltration method is found in Section III of this report and in the references.

Table 6

#### HOLTAN INFILTRATION PARAMETERS FOR IMPERVIOUSNESS STUDY

Soil Type	Available Storage Fp (in)	Ultimate Infiltration Rate fc (in/hr)
Cecil	0.76	0
Norfolk	1.5	0.2

The results of the sensitivity analysis are shown in Figs. 19 and 20.

# BRIAR CREEK AT SUDBURY ROAD

10-YEAR DESIGN STORM  
PRESENT STORAGE CONDITIONS  
AREA: 0.56 Sq. Mi.

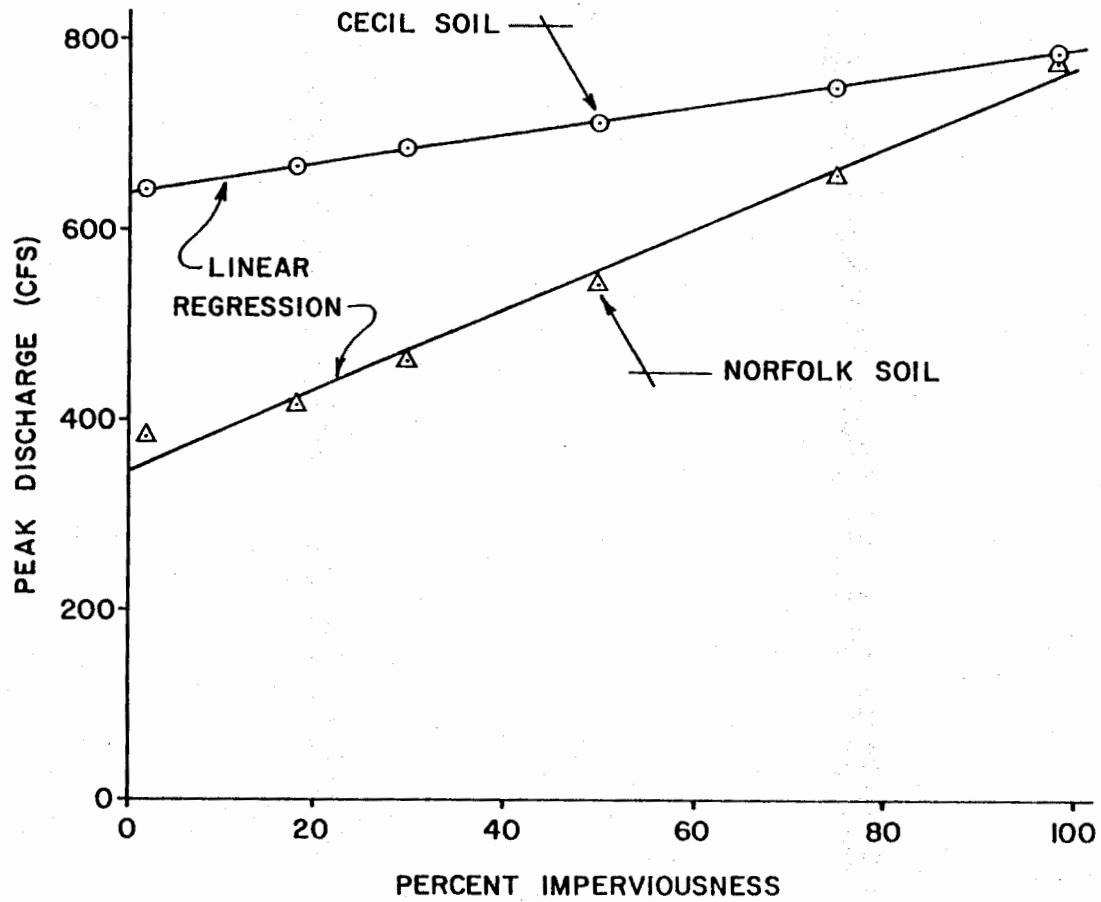


FIG. 19 SENSITIVITY OF PEAK DISCHARGE TO WATERSHED IMPERVIOUSNESS



# BRIAR CREEK AT SUDBURY ROAD

10-YEAR DESIGN STORM  
PRESENT STORAGE CONDITIONS  
AREA: 0.56 Sq. Mi.

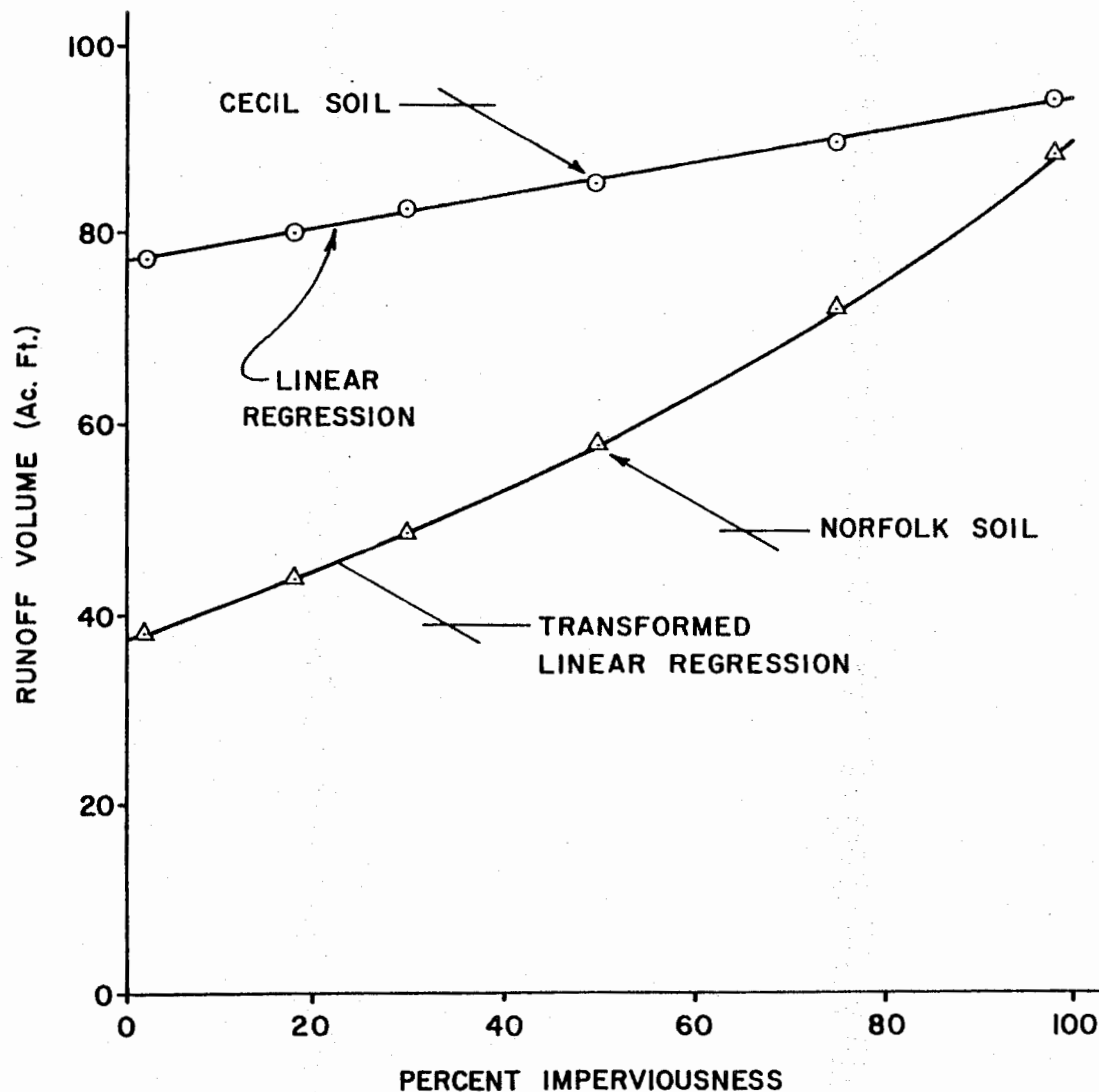


FIG. 20 SENSITIVITY OF RUNOFF VOLUME TO WATERSHED IMPERVIOUSNESS

The curves tend to converge to a maximum value at 100 percent imperviousness for both peak discharge and runoff volume. Since the soil having the higher infiltration capacity generates less runoff and peaks at a lower discharge, the effect of increasing the extent of impervious cover is greater in the more porous soil.

One may conclude that the effect of urbanization on runoff behavior is more severe in regions where the soils have higher infiltration capacities.

## SECTION V

### STATISTICAL STUDIES OF URBANIZATION AND WATERSHED RESPONSE

Early in the project work, the extensive set of streamflow data for Charlotte was studied with the intent of defining the actual upward trend of peak discharge with time in an urbanizing watershed. Data exists back to 1924 for the 41.0 square-mile watershed of Little Sugar Creek at Tyvola Road. Initially, the data set was divided in half, and the means were computed for the two halves. The mean for the earlier half was found to be larger than that of the more recent half of the streamflow record, in spite of the fact that urbanization was taking place throughout the record. Described here are some of the studies that lead one to conclude that flood discharges do not necessarily increase with urbanization.

#### A. TESTS OF HYPOTHESIS

Having found that the mean annual flood based on the recent half of the data was less than that of the earlier half, the data were divided into three subsets, based on interviews with local municipal officials:

1. Set A (1924-1939) - The watershed was substantially rural, although the northwestern sector was urban throughout the record. There was at least one channelization project undertaken in 1936.
2. Set B (1940-1960, 1946 missing) - During the intermediate period, there are remembered instances of small watershed flooding within the study watershed. By today's standards, culverts and bridges were systematically undersized.
3. Set C (1961-1975) - During the last set, there have been concerted efforts to improve channel and structure performance by channelization and structure enlargement throughout the watershed to mitigate local flooding.

To these sets, statistical hypothesis-testing procedures were applied as follows:

1.  $H_0: \mu_A < \mu_B$  - Can the hypothesis be rejected that Set A has a smaller mean than Set B?
2.  $H_0: \mu_B > \mu_C$  - Can the hypothesis be rejected that the middle part of the record has a greater mean than the last part?

The data are given in Table 7.

For analysis, the data were transformed by taking the natural logarithm. The transformed sets were more nearly normally distributed. The test used is the t-test as described in Ostle, 1963, p. 123, p. 119, and p. 122.

The test for equality of variances indicated that the variances were within reasonable limits. The hypothesis test, in this case, is a one-tailed t-test in which the value of t is computed as

Table 7

ANNUAL MAXIMUM FLOODS  
LITTLE SUGAR CREEK, NEAR CHARLOTTE, N. C.  
DRAINAGE AREA: 41.0 SQ. MI.

<u>Year</u>	<u>Flow (cfs)</u>	<u>Year</u>	<u>Flow (cfs)</u>
1924	3310	1950	2160
1925	4760	1951	2240
1926	3240	1952	4360
1927	2980	1953	2850
1928	7030	1954	3370
1929	5340	1955	4450
1930	3160	1956	3160
1931	2630	1957	2780
1932	5860	1958	5060
1933	4760	1959	2900
1934	5860	1960	3300
1935	3300	1961	3960
1936	8470	1962	5780
1937	4380	1963	4040
1938	1020	1964	2770
1939	3520	1965	4660
1940	2230	1966	3920
1941	4680	1967	4010
1942	6600	1968	3890
1943	3040	1969	2140
1944	4840	1970	3060
1945	3430	1971	5190
1946	Missing	1972	4480
1947	1800	1973	8440
1948	3360	1975	3630
1949	3730	1975	7800

Table 8

SUMMARY OF STATISTICAL PARAMETERS FOR ANNUAL MAXIMUM FLOOD DATA  
LITTLE SUGAR CREEK, NEAR CHARLOTTE

	Subsets			Entire record
	A 1924-1939	B 1940-1960	C 1961-1975	
Arithmetic mean (cfs)	4339	3517	4518	4069
Arithmetic Std. deviation (cfs)	1838	1181	1720	1605
Mean of natural logarithms	8.27744	8.11411	8.35442	8.23603
Std. dev. of natural logarithms	0.49111	0.32767	0.35829	0.39975
Skew coeff. of natural logarithms	-1.15	0.093	0.26	-0.41
Geometric mean (cfs)	3934	3341	4249	3775
10-yr. flood (ln-Pearson-III)	6740	5100	6780	6170
100-yr. flood (ln-Pearson-III)	8150	7340	10460	8470
Sample size	16	20	15	51

$$t = \frac{(\bar{X}_1 - \bar{X}_2)}{S_d}$$

where

$$S_d^2 = \frac{S_1^2}{n_1} + \frac{S_2^2}{n_2}$$

and

$$S_p^2 = \frac{(n_1 - 1) S_1^2 + (n_2 - 1) S_2^2}{n_1 + n_2 - 2}$$

in which  $\bar{X}$  = mean of sample (mean of the logarithms in this case)

S = sample standard deviation

n = number of observations in sample

t = test statistic

Numerical test results are given in Table 9.

Table 9

RESULTS OF HYPOTHESIS TESTS OF ANNUAL FLOOD DATA FOR  
LITTLE SUGAR CREEK AT TYVOLA ROAD, CHARLOTTE, N. C.

Item	$H_o: \mu_A < \mu_B$	$H_o: \mu_B > \mu_C$
Degrees of freedom	34	33
t observed	1.194	2.063
Probability of greater t	0.12	0.023

The test results show that the hypothesis may be rejected; or that, in fact, the probability is small that these relationships between the means of Samples A and B and those of Samples B and C could have occurred randomly. It appears that flood discharges were higher in the earlier and later parts of the record than they were in the middle part of the record.

## B. MOVING-AVERAGE ANALYSIS

The moving-average procedure provides a means to follow more nearly continuously the trends identified by hypothesis testing. The procedure is not held in high regard by many because of its obvious limitations. Here, it is used to study the trends in the data set rather than to place any sort of numerically measured confidence in the results.

1. Annual Peaks. A moving average of fifteen-year subsets of annual peaks was passed through the record from 1924 through 1975. Again, the flows were transformed using the natural logarithm so that the moving average is a fair estimate of the trend of the two-year flood as computed from fifteen-year samples. The trend is shown in Fig. 21.

The essentials of the hypothesis tests are reflected in the moving average. Flood discharges were higher in the early and late parts of the record and lower during the period 1950 through 1960.

2. Floods Above the Base. Among the criticisms of moving-average procedures is the observation that an extraordinarily large event or a small one will affect significantly the magnitude of the moving average in any subset in which the extreme value is included. In order to guard against the false interpretation of a trend, as a result of extreme events being included in the data set, a new sample was drawn. For each year in the record, the floods above an arbitrary base (2100 cfs) were obtained. The annual peak, which was a part of the previous data set, was discarded unless it was the only peak available. The remaining values were averaged for each year's observation, and the moving average was computed.

The results of this analysis are also shown in Fig. 21. It is noteworthy to observe that the trends are preserved when different source data are used.

3. Rainfall. There is some concern that the trends observable in the floods may be caused by corresponding trends in rainfall. Analyses of rainfall trends were conducted for several rainfall records in the region.

Flooding events in the study watershed appear to result from rainfall events of four to eight hours' duration. Moving-average analyses were conducted for rainfalls of four, six, and eight hours' duration. In each analysis, rainfall records were scanned to find for each year the maximum value of rainfall of the duration being analyzed at the location in question to form the data set. The moving-average analysis was then conducted independently for the three durations at each of several locations. The trends which emerged were not significantly different from level, but they were consistently reversed from those found for the discharges in Little Sugar Creek. (See Fig. 22.)

It was concluded that the trends in flood peaks do not result from trends in rainfall.

4. Conclusions of the Statistical Studies. Based on statistical analysis of rainfall and streamflow records, it is apparent that during the early period for which records are available, flood discharges were diminishing with time, in spite of urbanization of the watershed. During the period since about 1960, flood discharges have increased markedly.

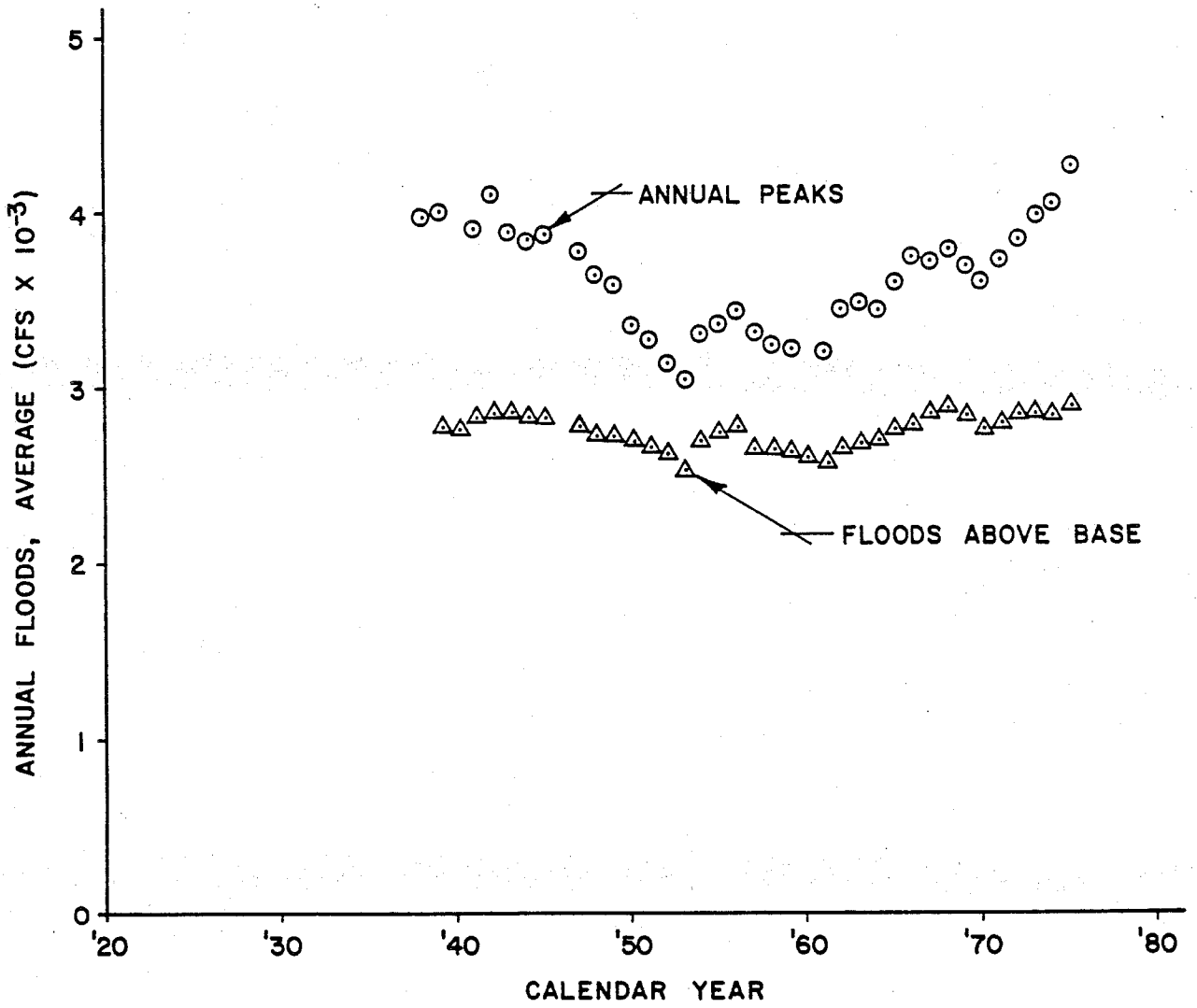


FIG. 21 MOVING AVERAGE (LOG-TRANSFORMED) RECORD OF LITTLE SUGAR CREEK, CHARLOTTE (41.0 Sq. Mi.)

DURATION OF STORM EVENTS

- x 8-Hr. RAINFALL
- △ 6-Hr. RAINFALL
- 4-Hr. RAINFALL

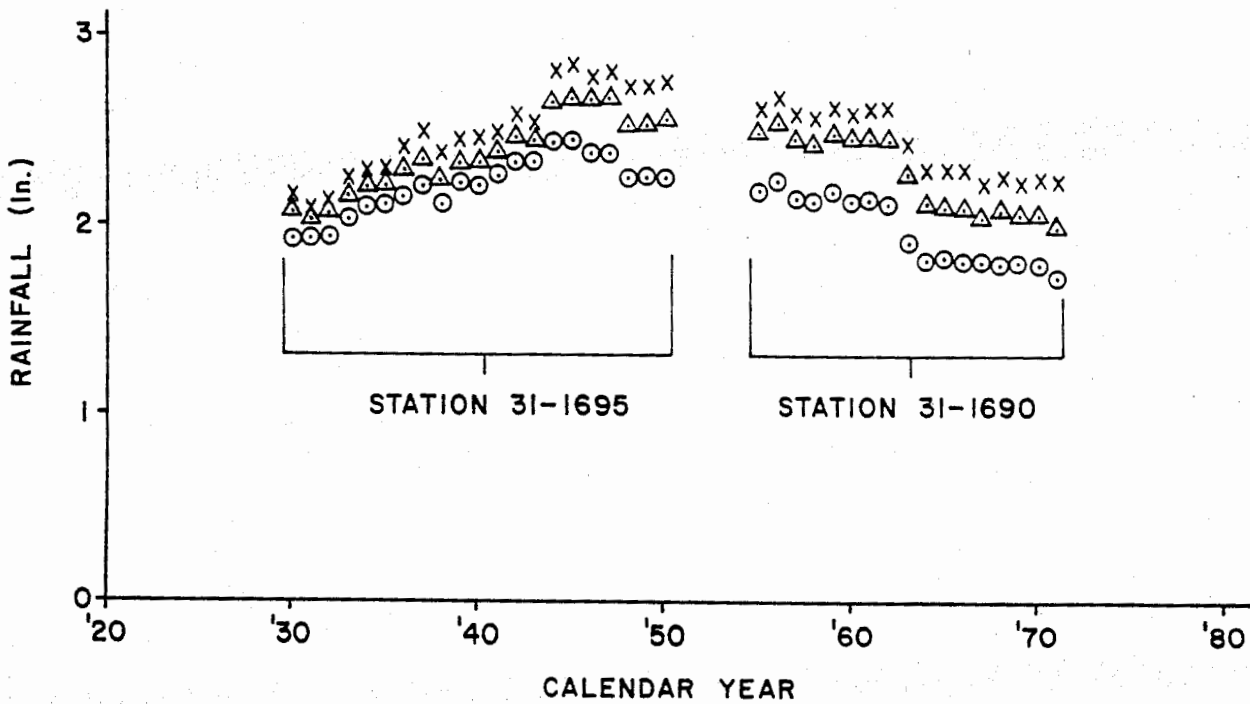


FIG. 22 15-YEAR MOVING AVERAGE OF RAINFALL AT TWO STATIONS IN CHARLOTTE, N.C.



The resources available in this project have not been adequate for detailed analysis of the causative factors behind the trends. The writer proposes to conduct further studies under separate sponsorship to try to determine the reasons for the trends. Based on the model studies discussed elsewhere in this report, the roadway culverts and bridges appear to be a major factor in the response of the watershed to changes in its urban composition.

The first part of the document discusses the importance of maintaining accurate records of all transactions. It emphasizes that every entry should be supported by a valid receipt or invoice. This ensures transparency and allows for easy verification of the data.

In the second section, the author outlines the various methods used to collect and analyze the data. This includes both primary and secondary data collection techniques. The primary data was gathered through direct observation and interviews with key stakeholders. Secondary data was obtained from existing reports and databases.

The third section provides a detailed description of the data analysis process. This involves identifying trends, patterns, and anomalies within the dataset. Statistical tools and software were used to facilitate this process, ensuring that the results are both accurate and reliable.

Finally, the document concludes with a summary of the findings and their implications. It highlights the key insights gained from the study and offers recommendations for future research and practice. The author notes that while the study has provided valuable information, there are still several areas that require further investigation.

## SECTION VI

### STUDIES OF URBANIZATION, STORMWATER DETENTION AND TRACTIVE FORCE

Stormwater detention is widely adopted for the purpose of reducing stream-bank degradation in urbanizing areas by preventing the increase of peak flows for storms of given return periods. Studies described elsewhere in this report indicate that effective temporary stormwater storage may accompany conventional urbanization of watersheds and that deterioration of streams persists in spite of storage.

The purpose of the studies of this section is to examine the time distribution of the force which tends to erode the streambank during the passage of a hydrograph as that force distribution is changed by urbanization and detention.

#### A. THE IMPULSE IDEA

As the storm flow passes through a given channel, the tractive force applied at the channel sides varies in intensity, vertically and in time, as the depth of flow changes. To study the differences in impacts of various shapes and sizes of hydrographs, it is necessary to account both for the magnitude of the force and the time over which it is applied.

A suitable concept is the impulse idea of mechanics. Impulse, the product of an applied force and the time of application, may be equated to the change in momentum of the object to which the force is applied.

$$I = Ft = \Delta mV$$

for a constant force applied over a finite time increment.

Given that the force varies with time, as in the channel, impulse is

$$I = \int_0^T Fdt = \Delta mV$$

The rightmost term, the total change in momentum, may be conceived in the channel as being reduced by the "breakaway" force required to dislodge the particles from the side of the channel, leaving a residual change in momentum which sets the particles on their way in the complex turbulence near the channel side.

At this stage of study, the fate of the particles is not of interest. Rather, the analytical resource provided by the impulse idea is used in the accounting of the force-time aggregation as a measure of the relative impacts of the several hydrographs on the sides of the channel. Thus, the differences in magnitude and vertical distribution of applied impulse constitute an estimate of the nature of the response of the channel as changes occur in the shape and size of the storm hydrograph. The impulse is viewed as a crude surrogate for the quantity of material eroded.

Two aspects of the impulse are studied:

1. For each hydrograph of interest, the total force per unit length of channel is integrated over the time base of the hydrograph to obtain the total impulse. The larger the impulse, the greater is the tendency to erode.
2. For each hydrograph of interest, the impulse intensity, defined here as impulse per unit area of channel side, is estimated as it is distributed vertically along the side of the channel. These results show whether one hydrograph shape, as compared to another, is more likely to cause slumping of the channel banks.

#### B. TRACTIVE FORCE DISTRIBUTION

As a basis for analysis, the work of Lane (Lane, 1955) was adapted to the present purpose. Expressions were obtained to relate tractive force at a specified depth to the maximum tractive force and to estimate the total applied force at the channel side.

1. Maximum Tractive Force. From Fig. 1 of the Lane reference, an expression was derived by statistical regression analysis to yield a good fit for maximum tractive force as a function of readily observable parameters:

$$\tau_{\max} = \gamma s [0.749 + 0.0145 \ln (B/y)]$$

where  $\tau_{\max}$  = Maximum tractive force

$\gamma$  = Unit weight of water

$y$  = Depth of flow

$s$  = Water surface slope

The equation is specific to a channel side slope of 2 to 1. See also the definition sketch, Fig. 23.

2. Distribution of Tractive Force. An expression was also obtained to relate tractive force at a given vertical location to the maximum tractive force.

The values for tractive force at given levels for various width-depth ratios, as presented by Lane, for side slopes of 2 to 1 are shown in Table 9. It may be noted that the maximum tractive force occurs at a level of 0.2 y, measured from the bottom, for a wide range of width-depth ratios and that the tractive force at constant vertical location is irregular with changing width-depth ratio. For this purpose, the magnitude of tractive force at a given vertical location,  $z/y$ , was considered to be independent of width-depth ratio. Mean values of tractive force, with one-standard-deviation limits, are shown in the rightmost column of the table.

An expression was obtained so as to follow closely the vertical distribution of mean tractive force:

$$\tau_z = 735 \sin [\pi(z/y)^{0.45}]$$

where  $\tau_z$  = mean tractive force, divided by  $\gamma y s \times 10^{-3}$

$z$  = location of point of interest, measured from bottom

$y$  = depth of flow

The expression is plotted with the Lane data in Fig. 23.

Table 10

VALUES OF TRACTIVE FORCE, VERSUS DEPTH AND CHANNEL WIDTH, USED TO FORMULATE EQUATION

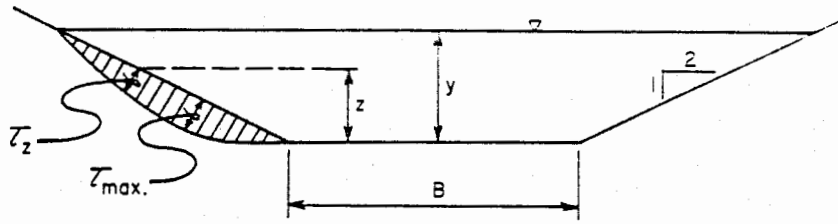
Depth (z/y)	B/y = 0	B/y = 2	B/y = 4	B/y = 8	Mean $\pm$ Std. Dev.
0	0	0	0	0	0
0.05	440	670	660	640	602.5 $\pm$ 109
0.1	550	740	750	720	690 $\pm$ 94
0.15	600	760	765	760	721 $\pm$ 81
0.2	640	760*	770*	770*	735* $\pm$ 64
0.3	650*	730	740	730	712 $\pm$ 42
0.4	610	680	680	660	658 $\pm$ 33
0.5	550	600	590	580	580 $\pm$ 22
0.6	470	500	490	470	482 $\pm$ 15
0.7	380	380	380	360	375 $\pm$ 10
0.8	260	250	260	240	253 $\pm$ 10
0.9	130	130	130	120	128 $\pm$ 5
1.0	0	0	0	0	0

\*Maximum

Note: Body of table contains values of  $\tau$  divided by  $\gamma y s \times 10^{-3}$ .

In terms of the maximum tractive force, the expression becomes

$$\tau_z = \tau_{\max} \sin [\pi(z/y)^{0.45}]$$



DEFINITION SKETCH

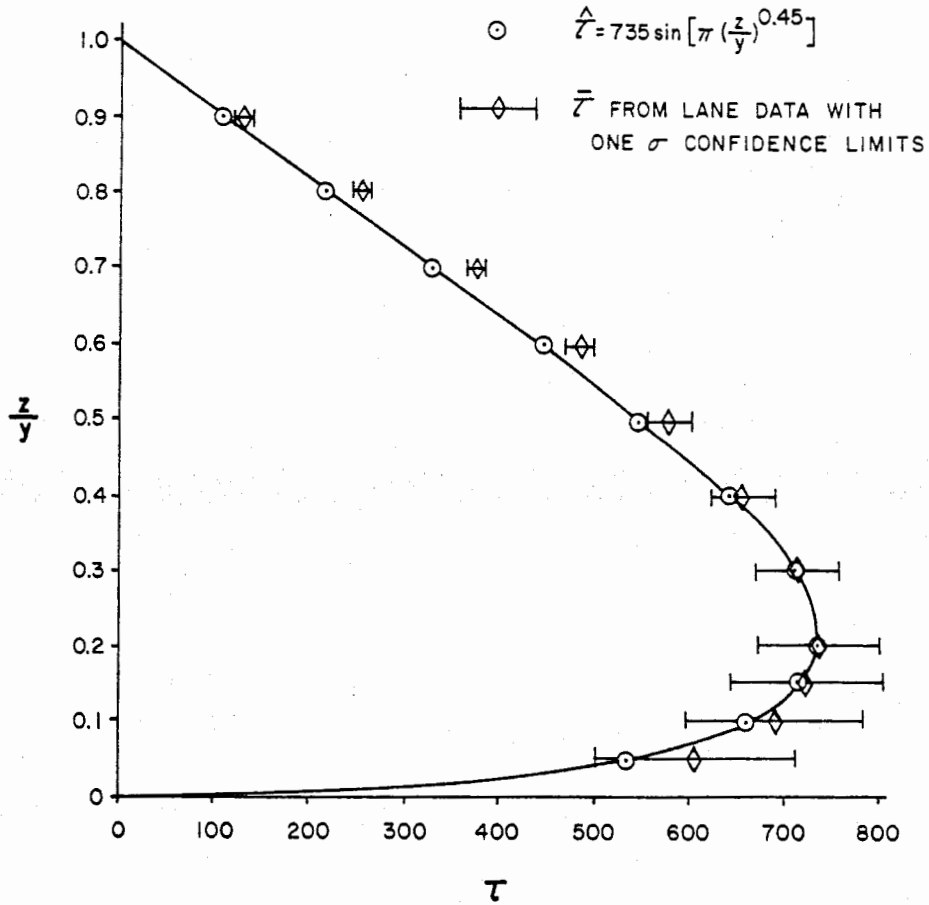


FIG. 23 TRACTIVE FORCE EXPRESSION COMPARED TO LANE DATA

3. Total Force Per Unit Length of Channel. Tractive force, as described by Lane, is, in fact, a shear stress applied tangentially to the channel lining. The total force applied along a channel strip of unit length may be found by integrating the expression of the vertical distribution of tractive force. Setting the results in terms of depth of flow and maximum tractive force gives

$$F = 0.622 \tau_{\max} y$$

where  $F$  is the force per unit length of channel.

Substituting the expression for maximum tractive force yields the alternate form

$$F = \gamma y^2 s [0.466 + 0.00902 \ln (B/y)]$$

In summary, the data reported by Lane (Lane, 1955) may be adequately represented in further calculations by any of the following expressions, which are specific to the sides of channels of two-to-one side slopes:

Maximum Tractive Force:

$$\tau_{\max} = \gamma s [0.749 + 0.0145 \ln (B/y)]$$

Vertical Distribution of Tractive Force on Channel Side:

$$\tau_z = \tau_{\max} \sin [\pi(z/y)^{0.45}]$$

or,

$$\tau_z = [0.749 + 0.0145 \ln (B/y)] \sin [\pi(z/y)^{0.45}]$$

where the argument of the sine function is in radians.

Total Force Per Unit Length of Channel (on one side):

$$F = 0.622 \tau_{\max} y$$

or,

$$F = \gamma y^2 s [0.466 + 0.00902 \ln (b/y)]$$

where  $\gamma$  = Unit weight of water (62.4 lb-force/ft<sup>3</sup>)

$y$  = Normal depth of flow (ft)

$z$  = Vertical location of interest (ft from bottom)

s = Slope of water surface (dimensionless)

B = Bottom width of channel (ft)

$\tau$  = Tractive force (lb/ft<sup>2</sup>)

F = Force per unit length of channel (lb/ft).

### C. COMPARISON HYDROGRAPHS

For the purpose of comparing the effects of urbanization and detention on the magnitude and distribution of the impulse applied on the channel side, six hydrographs were formulated for the smaller watershed used in the modelling study. The study hydrographs were then passed through a trapezoidal channel section to observe the differences in impact on the channel.

1. The Study Watershed. The channel of interest is located at the outlet of the 0.56 square-mile watershed of the tributary of Briar Creek at Sudbury Road, as shown in Fig. 24.

2. The Study Channel. A trapezoidal channel section having a bottom width of 8.0 ft, side slopes of two to one, channel slope of one percent and a Manning n-value of 0.030, was assumed to have been constructed at the outlet of the watershed. Uniform steady flow was assumed for a given time increment in the calculations. These are typical of conditions expected to prevail in a newly constructed channel in the study area.

3. The Study Storms. The analysis was conducted for storms of recurrence intervals of two and ten years. Hydrographs were derived to represent the watershed discharge behavior in the two and ten-year storms under rural and developed conditions.

The procedure adopted was to estimate the peak discharge and runoff volume for the storm of interest and to set the time to peak such that the volume of runoff was preserved under the hydrograph shaped according to the pattern hydrograph selected.

Peak flows were obtained by using Soil Conservation Service small-watershed techniques (SCS, 1973). The curve number selected for rural conditions was 60, for urban, 75. The 24-hour rainfall accumulations used as inputs for the method were obtained from published weather data (USWB, 1961).

The runoff volume was estimated for the six-hour storm, using the SCS curve number method (SCS, 1972).

The pattern hydrograph used was a step-function approximation of the SCS dimensionless hydrograph (SCS, 1972, p. 16.3), as compared in Fig. 25.

For  $0 \leq t \leq 1.25 t_p$ :

$$Q = \frac{Q_p}{2} \left[ 1 - \cos \left( \frac{\pi t}{t_p} \right) \right]$$



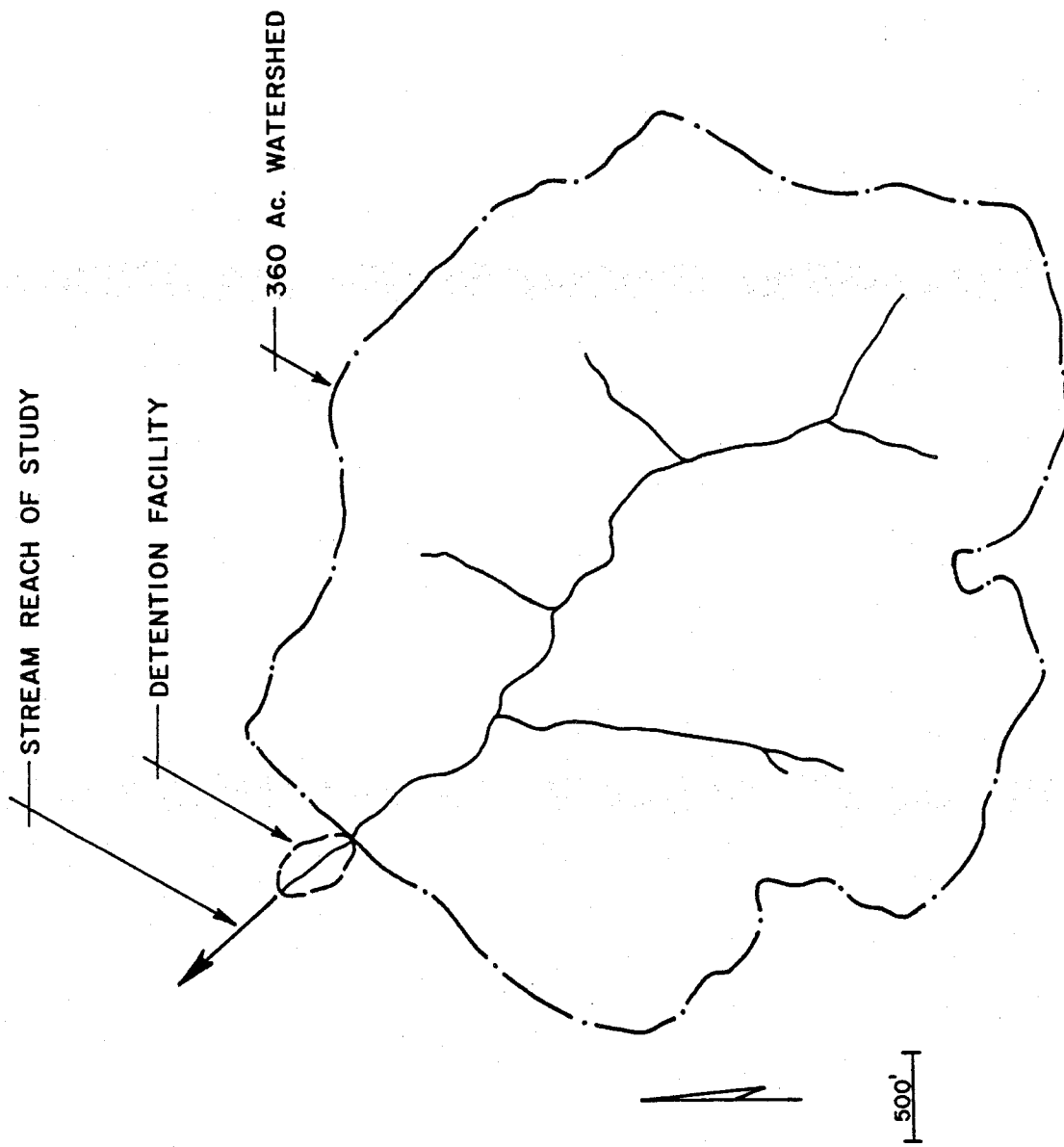


FIG. 24 LOCATION MAP FOR CHANNEL STUDIES

For  $t > 1.25 t_p$ :

$$Q = 5.37 Q_p e^{-1.42 \frac{t}{t_p}}$$

where  $Q_p$  = Peak flow

$t_p$  = Peak time

$t$  = Time of interest

$Q$  = Flow at time of interest

and the argument of the cosine is in radians.

The total volume of runoff under the hydrograph is, in consistent units (Briggs, 1977):

$$\text{Vol} = 1.37 Q_p t_p$$

The time to peak, then, may be set to preserve a given runoff volume under a hydrograph which peaks at a given flow by

$$t_p = \frac{\text{Vol}}{1.37 Q_p}$$

By these means, the entries of Table 11 were obtained.

Table 11  
HYDROGRAPH FORMULATION FOR IMPULSE ANALYSIS

Item	Before Development		After Development	
	2-yr. Event	10-yr. Event	2-yr. Event	10-yr. Event
SCS Curve Number	60	60	75	75
24-hr. Rainfall (in.)	3.5	3.5	5.0	5.0
6-hr. Rainfall (in.)	2.50	2.50	3.75	3.75
6-hr. Runoff (in.)	0.17	0.64	0.65	1.48
Peak Discharge (cfs)	36	115	160	600
Time to Peak (min.)	77	89	65	39

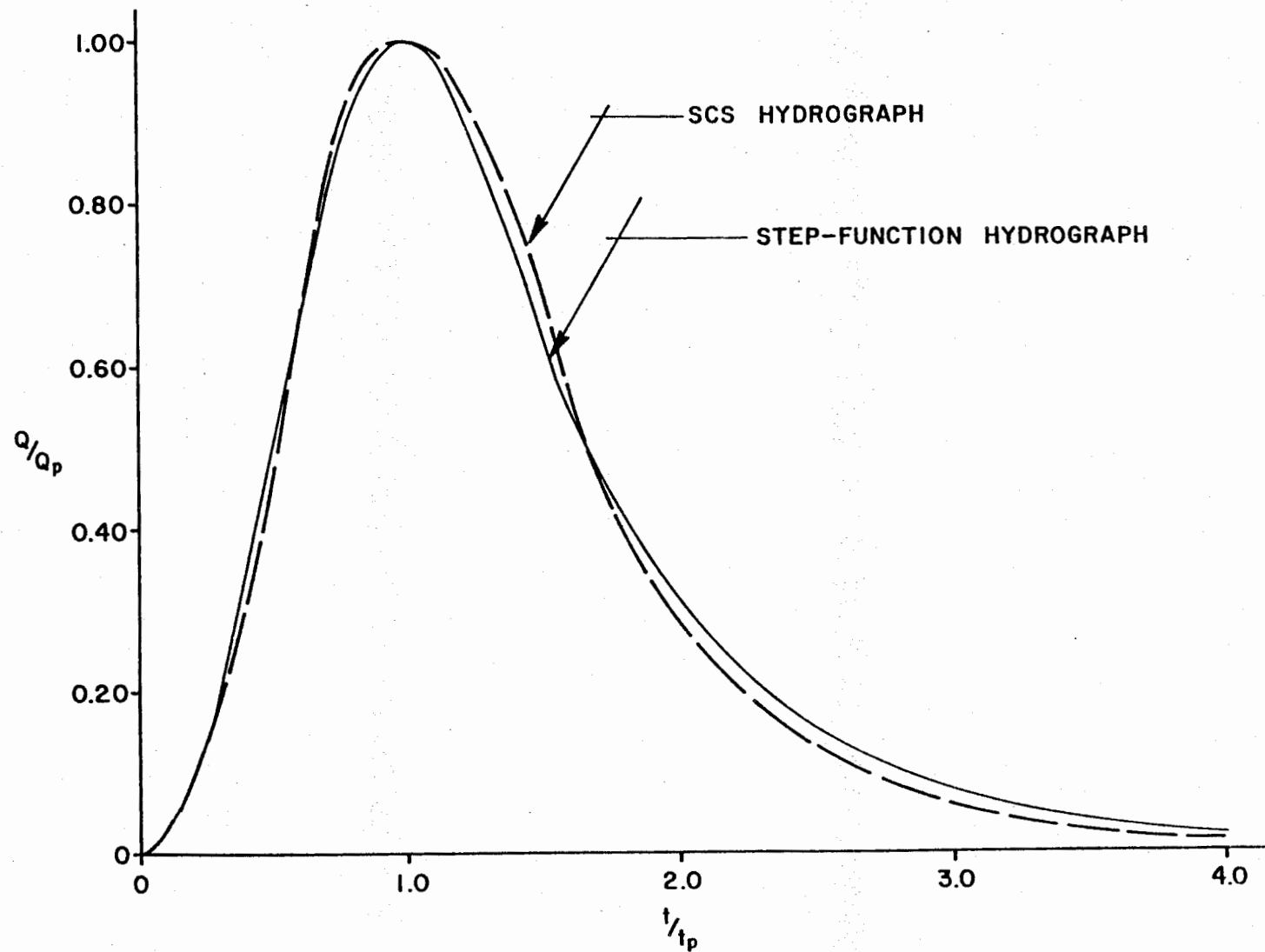


FIG. 25 COMPARISON OF STEP FUNCTION AND SOIL CONSERVATION SERVICE DIMENSIONLESS HYDROGRAPH

4. The Detention Basins. Facilities were sized for both the two and ten-year storms according to the design criterion that the peak discharge from the watershed after development shall not exceed the peak discharge from the watershed prior to development, in the storm of interest. Thus, for the two-year storm, a basin was contrived to hold the estimated peak of 115 cfs after development to no more than 36 cfs and for the ten-year storm to hold the estimated peak of 600 cfs to no more than 160 cfs.

The detention site is located at a street crossing of the stream near the watershed outlet. The detention facilities were developed by sizing the culvert under the roadway such that water would be stored in the area upstream of the culvert. The stage-storage relationship, typical for local topography, is

$$S = 500 Z^{3.25}$$

where Z is stage above the invert of the culvert involved (ft.)

and S is storage (ft<sup>3</sup>).

The results of these designs are shown in Table 12.

Table 12  
THE SIZING OF DETENTION FACILITIES FOR STUDY STORMS

Item	2-yr. design	10-yr. design
Inflow peak (cfs)	160	600
Time to peak (min)	65	39
Allowable outflow (cfs)	36	115
Pipe selection (reinforced concrete, headwall condition)	2 @ 15"	2 @ 24"
Routing results:		
Peak outflow (cfs)	36	105
Maximum stage (ft)	8.49	11.24

5. The Study Hydrographs. The hydrographs prepared for the channel study are shown in Fig. 26 and Fig. 27. They have the following properties:

- a. The hydrograph for conditions before development peaks at the best estimate for the storm of interest under rural conditions. The volume under the hydrograph is the estimated runoff for the six-hour storm of stated recurrence interval under rural conditions.
- b. The hydrograph for conditions after development peaks at the best estimate for the storm of interest under mixed urban conditions typical

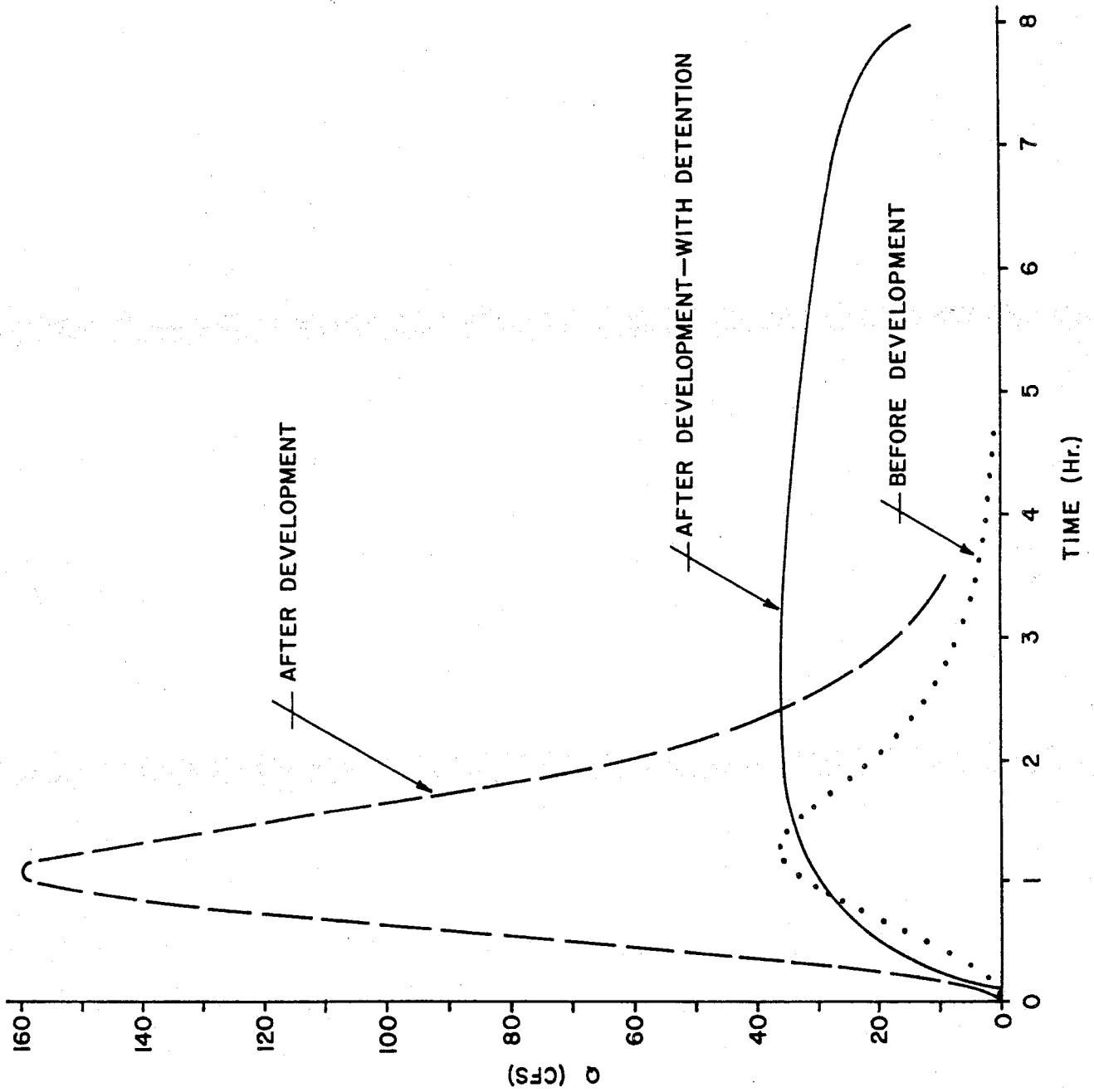


FIG. 26 2-YEAR STORM HYDROGRAPHS FOR CHANNEL STUDIES

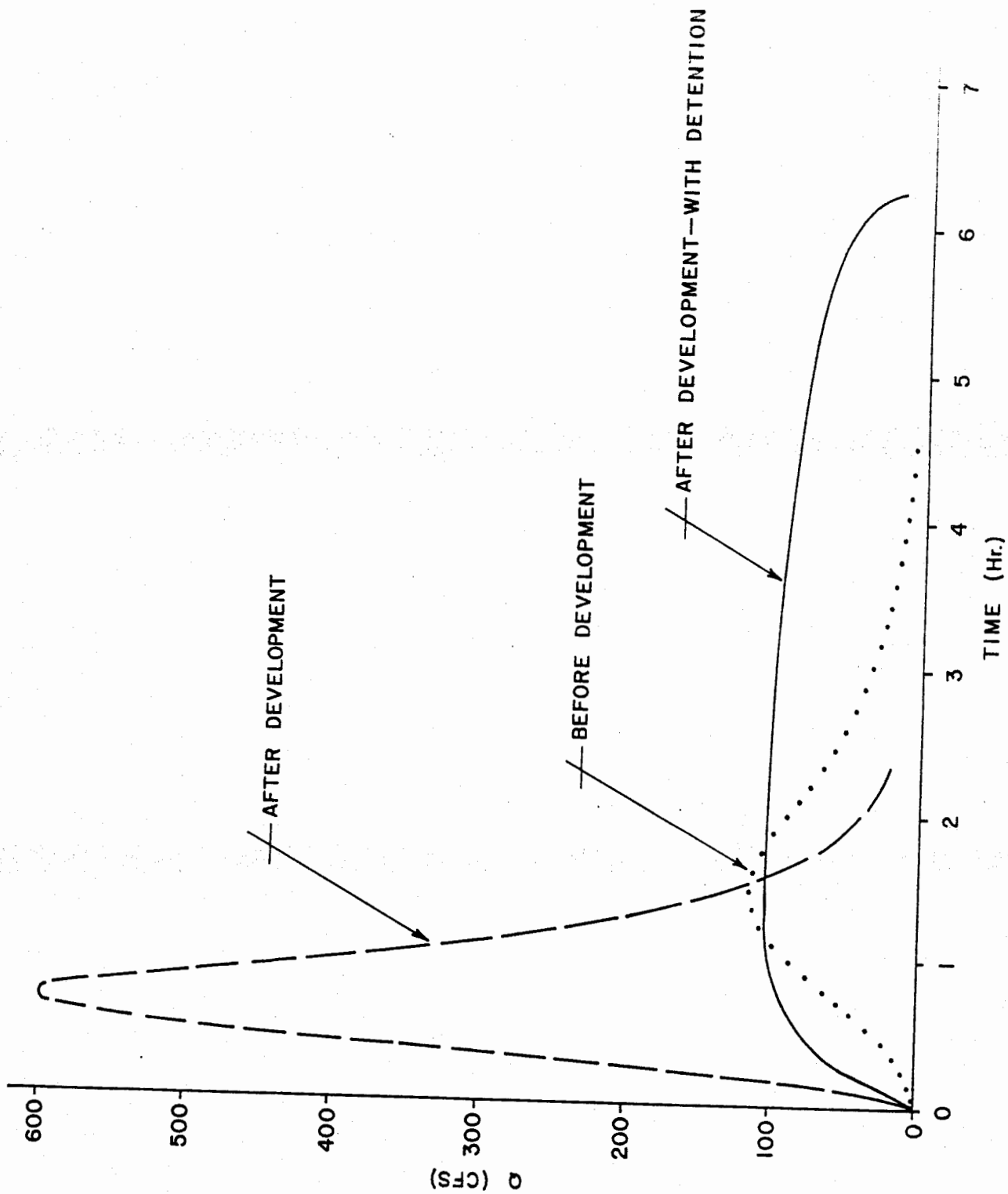


FIG. 27 10-YEAR STORM HYDROGRAPHS FOR CHANNEL STUDIES

of the study area. The volume under the hydrograph is the estimated runoff for the six-hour storm of stated recurrence interval under urban conditions.

- c. The hydrograph for conditions after development with detention is the outflow hydrograph obtained by routing the hydrograph for conditions after development through a detention basin designed to hold the peak outflow to no more than the peak of the hydrograph before development. The volume under this hydrograph is equal to the runoff for after-development conditions.

#### D. TOTAL IMPULSE COMPARISONS

Each of the six hydrographs was passed through the study channel to obtain the estimated total impulse applied at the channel side.

From above, the total impulse is

$$I = \int_0^T F dt$$

or, numerically

$$I = \sum_{t=0}^{t=T} F \Delta t$$

Also from above,

$$F = \gamma y^2 s [0.466 + 0.00902 \ln (B/y)]$$

For water,  $\gamma = 62.4 \text{ lb/ft}^3$ . Substituting the value yields

$$F = y^2 s [29.1 + 0.574 \ln (B/y)]$$

So,

$$I = \sum_{t=0}^{t=T} \{y^2 s [29.1 + 0.574 \ln (B/y)]\} \Delta t$$

If time is measured in minutes, linear quantities in feet, and force in pounds, impulse will be obtained in units of pound-minutes per foot length of channel (one side).

Individual cases were executed by the following procedures:

1. Generate the hydrograph of interest in increments of  $\Delta t$ .
2. For each discharge, determine the normal depth of flow in the channel of interest.
3. For each time increment, determine the force per foot length of channel.
4. Obtain the total impulse as the sum of  $F \cdot \Delta t$  through the hydrograph.

The results of the analysis for the six hydrographs formulated for the study are presented in Table 13.

Table 13  
SUMMARY OF RESULTS OF TOTAL IMPULSE COMPARISONS

Item	Rain (in)	Runoff (in)	Qp (cfs)	Impulse (lb·min/ft)	$\frac{I}{I \text{ before}}$
2-yr. storm before development	2.50	0.17	36	23.9	1
2-yr. storm after development	2.50	0.65	160	106	4.4
2-yr. storm after development with detention	2.50	0.65	36	97	4.1
10-yr. storm before development	3.75	0.64	115	101	1
10-yr. storm after development	3.75	1.48	600	255	2.5
10-yr. storm after development with detention	3.75	1.48	105	248	2.5

One may conclude that the use of detention does not reduce significantly the total impulse applied at the channel side.

From the Lane research (Lane, 1955), it is reasonable to estimate that the native material of which the streambanks of the study area are substantially composed can withstand a tractive force of about  $0.26 \text{ lb/ft}^2$  without severe erosion. It is instructive to plot maximum applied tractive force versus time for each of the cases analyzed, as shown in Figs. 28 and 29. Only in the two-year storm under estimated rural conditions does the discharge remain largely non-erosive. This is in sensible agreement with the notion that the two-year storm can be contained within the streambanks without excessive erosion. While it seems to be true for the case of rural conditions, the two-year storm after development with detention appears to be quite erosive.



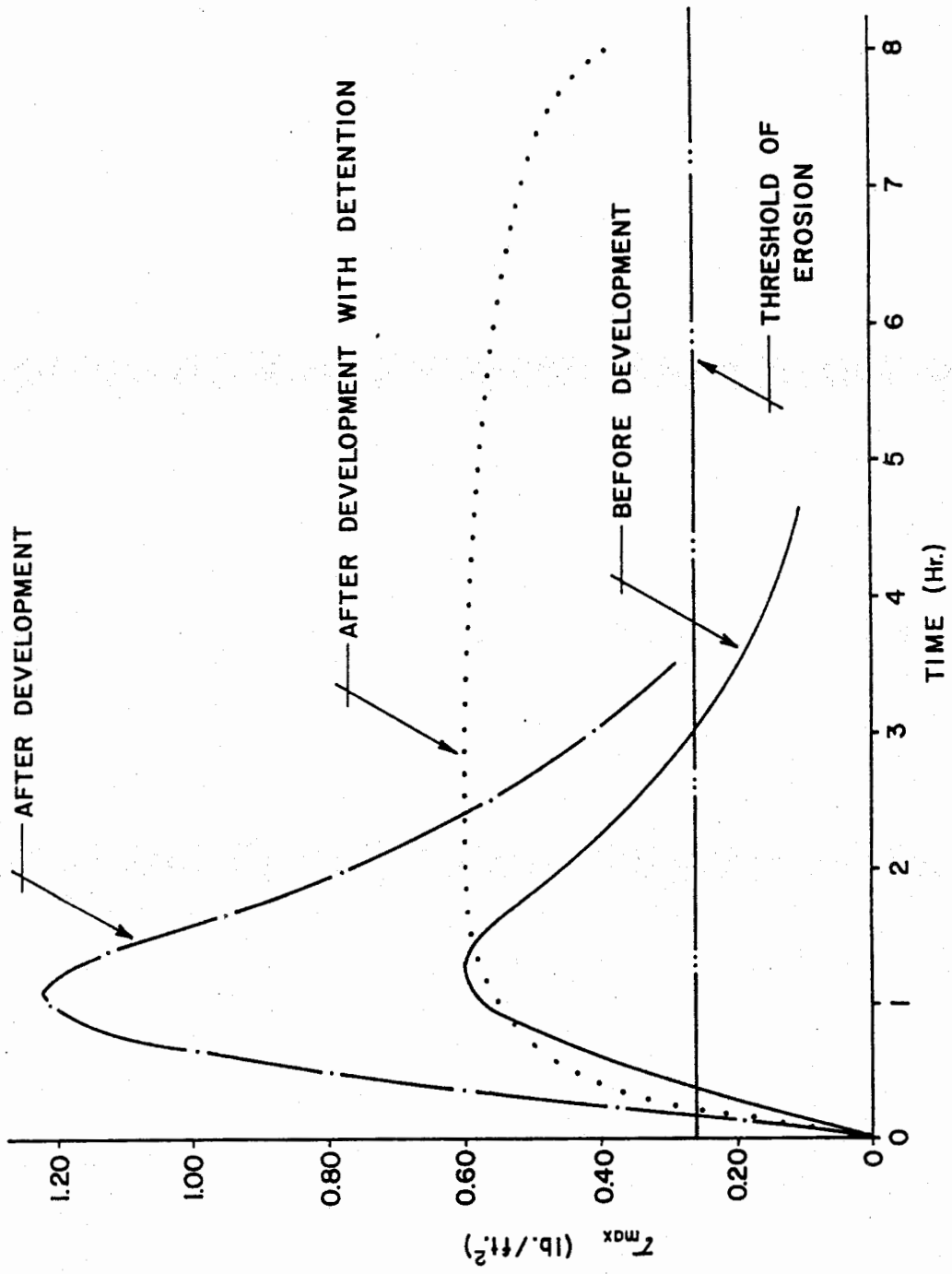


FIG. 28 MAXIMUM APPLIED TRACTIVE FORCE VERSUS TIME (2-YR. STORMS)

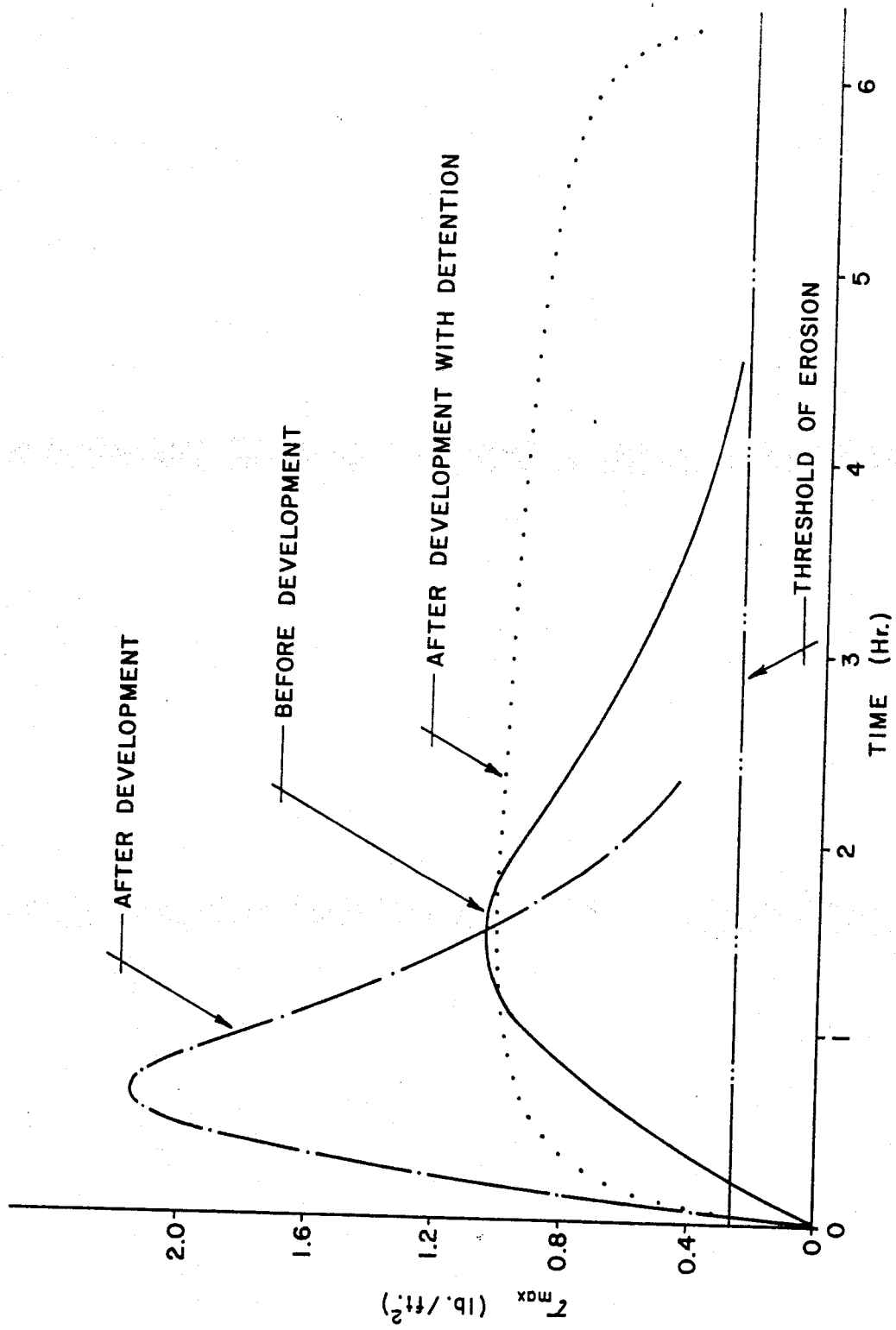


FIG. 29 MAXIMUM APPLIED TRACTIVE FORCE VERSUS TIME (10-YR. STORMS)

## E. VERTICAL DISTRIBUTION OF IMPULSE INTENSITY

The total impulse applied at the channel side appears to change little as a result of introducing detention to reduce peak flow. It is important to analyze the vertical distribution of the impulse intensity as it is applied along the channel side.

A consequence of detention is that maximum depth of flow is reduced as peak discharge is reduced. Hence, the relatively unchanged total impulse is applied in a smaller depth range. It is reasonable to expect that the intensity of the applied impulse is greater near the bottom with detention than without it.

If impulse in this context is the product of force and time during the passage of the hydrograph, the intensity of that impulse at a specified vertical location may be defined as the product of tractive force (stress) and time, the tractive force being the stress applied at the vertical location of interest through time. Impulse intensity is conveniently expressed in units of pound-minutes per square foot.

From above,

$$\tau_{\max} = \gamma y s [0.749 + 0.145 \ln (B/y)]$$

For  $\gamma = 62.4 \text{ lb/ft}^3$

$$\tau_{\max} = y s [46.74 + 9.05 \ln (B/y)]$$

In terms of  $\tau_{\max}$ , the tractive force at a specified vertical location,  $z$  (where  $z < y$ ), is

$$\tau_z = \tau_{\max} \sin [\pi(z/y)^{0.45}]$$

the argument of the sine being in radians.

The impulse intensity at a given vertical location,  $z$ , is

$$I_z = \int_0^T \tau_z dt$$

Or, numerically,

$$I_z = \sum_{t=0}^{t=T} \tau_z \Delta t$$

Individual cases were analyzed by the following procedure:

1. Generate the hydrograph of interest in increments of  $\Delta t$ .

2. For each discharge, determine the normal depth of flow in the channel of interest.
3. For each time increment, determine the maximum tractive force and from that the shear stress applied over the time increment at each of several selected vertical locations.
4. For each vertical location, obtain the impulse intensity as the sum of  $\tau_z \cdot \Delta t$  through the hydrograph.

The results of the analysis of the vertical distribution of impulse intensity are shown graphically in Fig. 30.

It is striking to note the degree to which the applied impulse is intensified near the bottom of the channel by the introduction of detention. One may conclude that the tendency is to undercut and steepen the slopes of streambanks downstream from effective detention facilities from which the discharge is sufficiently large to produce shear stresses greater than those that can be withstood by the bank material. This conclusion is in agreement with visual evidence in the study area. Over time, streambanks tend to be undercut or to become steeper until they become unstable and slumping occurs.

#### F. IMPLICATIONS IN WATERSHED MANAGEMENT

If one accepts the validity of the foregoing analysis, there are some important implications in watershed management:

1. The increase in runoff volume which accompanies urbanization is a dominant contributing factor to the deterioration of streambanks in urbanizing areas.
2. The introduction of detention in watersheds for the purpose of preventing or correcting streambank deterioration is likely to have the opposite effect.
3. Where detention is used for the purpose of flood control, an attendant detrimental effect is the intensification of streambank deterioration.
4. Measures intended to control streambank deterioration by hydrograph manipulation should include provisions for reducing the volume of runoff.
5. Measures intended to control streambank deterioration by protection of channel sides may be designed by using tractive-force concepts. Economic and aesthetic enhancement may be realizable by treating the bottom part of the channel sides.

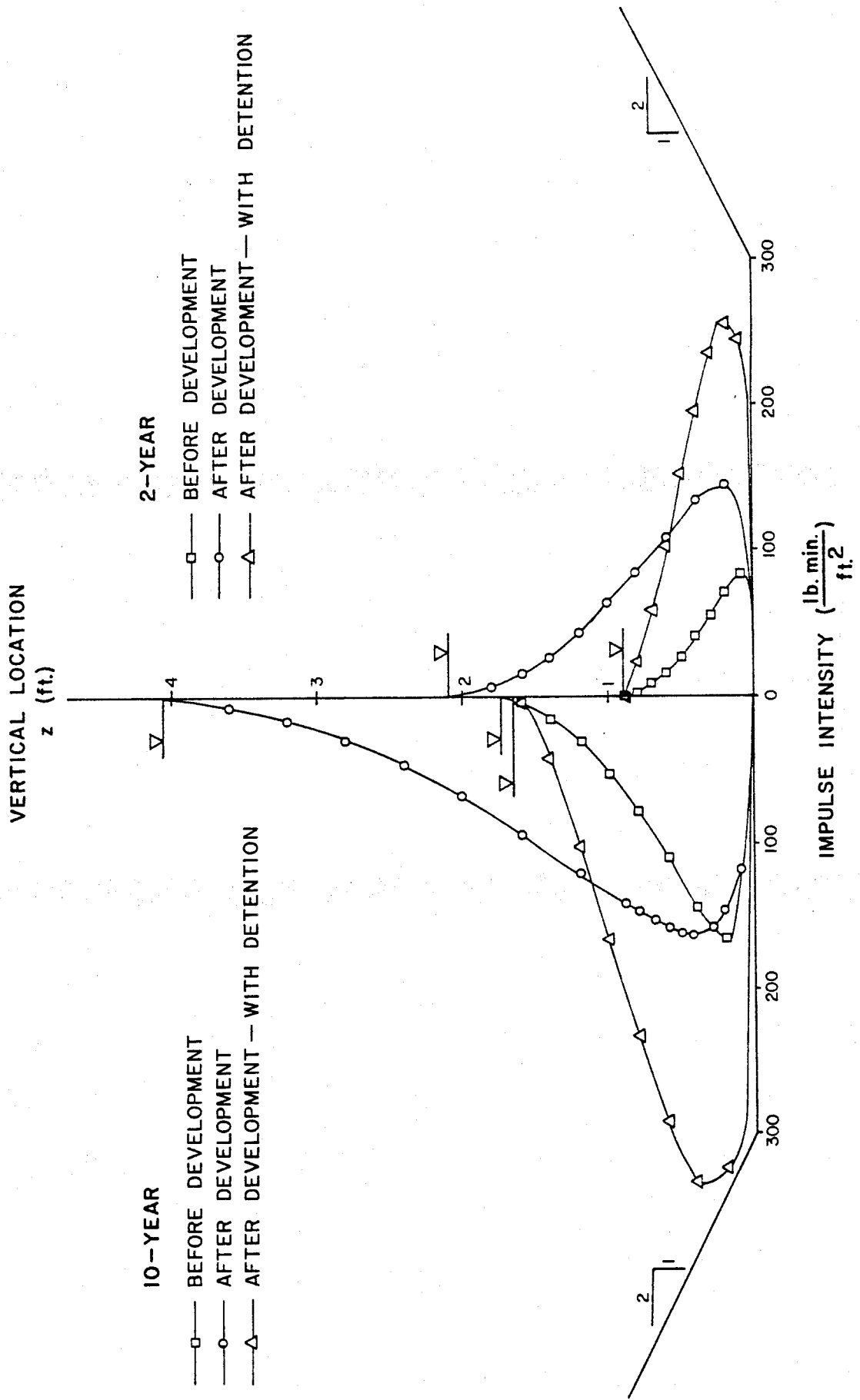


FIG. 30 RESULTS OF IMPULSE DISTRIBUTION ANALYSIS



## SECTION VII

### INTERACTION OF FLOOD CONTROL AND SEDIMENT CONTROL

Detention is frequently proposed as a means of flood control and sediment control; sometimes in a combination program, sometimes separately. When programs are established for both purposes, the interaction between them becomes important.

Detention regulations normally provide for a single design storm. Effective control of damaging floods generally implies a design storm at least as rare as the fifty-year event. Effective sediment control, i.e., the control of stream-bank erosion, requires substantial reduction of discharge in frequently occurring storms in the return-period range of two to ten years. (Indeed, the analyses of the preceding section witness against the use of detention for this purpose, but sediment-control detention programs are likely to persist.)

Consider a detention facility designed to reduce substantially the ten-year peak discharge. When hit by the 100-year storm, the emergency spillway is likely to be activated. The flood-control implications are that upstream stage is higher than it would be in the absence of any facility, and the peak outflow is likely to be very nearly as high as the peak inflow. The facility designed for the ten-year storm aggravates conditions of flooding in the 100-year storm.

Consider a flood-control facility designed for the 100-year storm. The outlet device is sized to release a discharge which will hold downstream damage to an acceptable level. Generally, the upstream storage is minimized in achieving this objective. Under ten-year storm conditions, the outlet device is usually much too large to reduce the hydrograph peak to the extent specified in sediment-control programs.

Some specially designed facilities can meet both objectives, but it is unrealistic to expect typical detention programs to be effective in controlling both flood damage and streambank erosion.





## SECTION VIII

### ECONOMIC CONSIDERATIONS

Detention is one tool, one means, available for stormwater management. It is frequently possible to reduce overall drainage costs in a given project by incorporating detention, whether or not it is required by external pressures. There are two key factors in determining whether its use is economical. The planning and design of the detention facility must proceed integrally with the rest of the project plans, and the detention storage area must be usable for other purposes.

In order to reduce outflow from given detention facilities, the pipes used to cause the water to be stored are smaller than those that would be required to pass the unattenuated peak discharges. The conveyances (pipes and channels) downstream from detention facilities need not be as large as conventionally sized units.

Whether or not the savings thus anticipated are realizable depends on the cost of providing the storage. Certainly, if the storage is placed in single-purpose, maintenance-intensive facilities, the cost will be great. But if the water can be stored harmlessly in areas used for some other purpose in fair weather, the storage may be "free."

Where large parking lots are required, instead of building an ugly hole somewhere that fills to six feet deep in the design storm and collects litter in fair weather, consider storing the water to curb height in a depression at one side. Maintenance consists of sweeping the detention depression with the rest of the lot.

Where culverts are to be installed under a roadway passing through a park, consider causing the water to back up to roadway height in the design storm. Let the roadway be the emergency spillway.

The savings obtained by such carefully designed devices can be remarkable.

1000  
1000

## SECTION IX

### POLICY RECOMMENDATIONS

This has been a study of the use of detention in urban stormwater management. The original purpose was to arrive at a "best" design storm for use in a comprehensive stormwater detention program; however, in examining the premises on which such programs are based, it was found that stormwater detention has limited usefulness.

Stormwater detention is quite effective in preventing nuisance flooding immediately downstream of radical changes in land use.

Stormwater detention is one very good tool for use in urban stormwater management. It can be effective and economical in many instances. But, as one would not use a hammer exclusively to build a house, one should not rely on stormwater detention as the single, or even the central, policy for stormwater management.

In watershed management, it is important to provide for the efficient allocation of all resources available. To specify one measure to be applied universally is expressly inefficient. A potential mechanism for allocation is to treat the drainage system as a public utility, in parallel with the provision of water, sewer, and streets. Where strategic detention facilities or major channels are needed, they can be designed centrally with the whole watershed being accounted for in the plan. The cost of those facilities may be spread among those who benefit. This is done for arterial streets, interceptor sewers and trunk mains. Local drainage costs could be borne locally as in the case of other utilities in the subdivision package. A scheme for distributing drainage costs has been suggested (Riordan, et al., 1978).



## SECTION X

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