INFORMATION TO USERS

This manuscript has been reproduced from the microfilm master. UMI films the text directly from the original or copy submitted. Thus, some thesis and dissertation copies are in typewriter face, while others may be from any type of computer printer.

The quality of this reproduction is dependent upon the quality of the copy submitted. Broken or indistinct print, colored or poor quality illustrations and photographs, print bleedthrough, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send UMI a complete manuscript and there are missing pages, these will be noted. Also, if unauthorized copyright material had to be removed, a note will indicate the deletion.

Oversize materials (e.g., maps, drawings, charts) are reproduced by sectioning the original, beginning at the upper left-hand corner and continuing from left to right in equal sections with small overlaps.

ProQuest Information and Learning
300 North Zeeb Road, Ann Arbor, MI 48106-1346 USA
800-521-0600

UMI®

Mgex (Don)

CONSIDERATIONS ON TWO URBAN RUNOFF APPLICATIONS IN THE CANADIAN CONTEXT: THE MODELING OF STORAGE FACILITIES WITH LOW RELEASE RATES AND THE SCS METHOD

by

Robert Henry Pankratz

A Report
presented to the School of Graduate Studies and Research
of the University of Ottawa
in partial fulfillment of the requirements
for the degree of Master of Engineering
in Civil Engineering

Université d'Ottawa
BIBLIOTHÈQUES
LIBRARIES
University of Ottawa

© Robert H. Pankratz, Ottawa, Canada, 1987.

UMI Number: EC52413

INFORMATION TO USERS

The quality of this reproduction is dependent upon the quality of the copy submitted. Broken or indistinct print, colored or poor quality illustrations and photographs, print bleed-through, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send a complete manuscript and there are missing pages, these will be noted. Also, if unauthorized copyright material had to be removed, a note will indicate the deletion.

UMI®

UMI Microform EC52413
Copyright 2007 by ProQuest LLC
All rights reserved. This microform edition is protected against unauthorized copying under Title 17, United States Code.

ProQuest LLC
789 East Eisenhower Parkway
P.O. Box 1346
Ann Arbor, MI 48106-1346

ACKNOWLEDGEMENTS

There are several people who deserve special recognition for their assistance in the completion of this report. The author wishes to express sincere appreciation to his supervisor, Dr. Paul Wisner, for support, encouragement, guidance and for the wealth of his understanding of the subject matter as a whole.

Grateful acknowledgement is made of the Implementation of Stormwater Management (IMPSWM) program which provided financial support and also afforded many opportunities for practical professional development.

The author's parents and family also warrant special recognition for their faithful prayers, encouragement and financial assistance during the course of the author's studies.

Finally, the author is eternally indebted to the Lord Jesus Christ for His daily provision of grace; given both spiritually and academically.

PREFACE

This report addresses two widespread misapplications in the current state-of-the-art of stormwater management practice in Canada. Misapplications occur in the use of the popular SCS urban runoff procedures for Canadian conditions and in hydrologic sizing of on-site detention facilities. Both involve modeling of urban runoff phenomena and share widespread misunderstanding which leads to errant design practices at the consulting engineering level.

The SCS methods, well known and commonly used in the United States, have several drawbacks for Canadian conditions which are generally unknown and which may result in serious misapplications. In an attempt to better understand and objectively qualify these limitations, simulation studies were conducted using measured rainfall-runoff data. Issues addressed include historic event simulation capabilities and the potential for performance improvement through parameter optimization. Performance comparisons with another urban runoff model was also conducted.

On-site detention facilities with low release rates are becoming increasingly popular. Proponents advocate methods ranging from the very simple rational method to sophisticated continuous simulation computer models. Storage volumes can be alarmingly divergent and inconsistent and accuracy of the methods, has, in many cases, not been tested. Because of the lack of understanding of the critical hydrologic processes and parameters governing detention design, serious misapplication of modeling in this direction has occurred in Canada for quite some time. A comparison of design event and continuous simulation sizing techniques was conducted to determine the relative merits of each method and their respective ranges of application and confidence.

Each of these two aspects was treated independently and, therefore, each is presented as separate within the larger report. In this rather unconventional format, a separate table of contents, reference list and numbering system for figures, tables and pages was incorporated.

PRELIMINARY CONSIDERATIONS IN THE MODELING OF STORAGE FACILITIES WITH SMALL RELEASE RATES

by

Robert Henry Pankratz

A Report
presented to the School of Graduate Studies and Research
of the University of Ottawa
in partial fulfillment of the requirements
of the degree of Master of Engineering

A B S T R A C T

A fundamental stormwater management principle is the storage of excessive stormwater runoff at its source and its subsequent release at controlled outflow rates. On-site detention facilities with low release rates are becoming increasingly popular in Canada. To determine the required size of such facilities, proponents advocate methods ranging from the simple rational method to sophisticated continuous simulation computer models. Storage volumes determined by these various methods can produce alarmingly divergent and inconsistent results. The accuracy of such methods has, in many cases, not been adequately tested. Because detention facility design is not well understood, serious misapplication of modeling has occurred in Canada for quite some time.

A comparison of two categories of sizing methods - design event methods and continuous simulation techniques - was conducted to determine the relative merits of each method and their respective ranges of application and confidence. Detention facilities sized with single event procedures were subjected to long term continuous simulations using historic rainfall records to determine the actual recurrence frequency of overflows. URBHYD and STORM were selected to represent classes of single event and continuous simulation models, respectively.

It was possible to compare the results of both methodologies on the basis of three parameters: storage volumes, release rates and watershed imperviousness. The comparison revealed that for low release rates the single event design approach may significantly underestimate the required storage volume. It also became evident that defining a critical design storm event independently of the drainage basin is impossible.

Municipalities accepting release rates as low as 10 to 40 l/s/ha should require continuous simulation in the design process to ensure that adequate storage capacity is provided. Further research is necessary to determine the extent to which snowmelt events effect the design process.

Table of Contents

List of Tables

List of Figures

1. INTRODUCTION

1.1 Background

Increased stormwater runoff volumes, higher peak flows and faster watershed response rates are only some of the undesirable hydrologic by-products of urbanization. Increased flooding, erosion, sedimentation and higher pollutant loadings are often the result. Temporarily detaining excess runoff and releasing it at a controlled rate is a fundamental principle of Storm Water Management (SWM).

Benefits of temporarily detaining storm runoff are many and varied. In addition to reducing peak discharges to rates which can be conveyed in downstream drainage systems, storage facilities may retain water for consumptive, aesthetic or recreational purposes. Groundwater, often depleted as a result of high proportions of impervious cover, can be recharged. Storage facilities can reduce the kinetic energy of surface runoff in high relief watersheds, which would lead to excessive velocities causing soil and stream bed erosion. Sedimentation, flotation and oxidation processes, which occur in storage, will reduce pollutant concentrations in impounded water and may minimize pollutant loadings on downstream receiving waters when released in a controlled manner. Suspended solids tend to settle out in detention ponds reducing siltation in downstream sewers and the accompanied loss in capacity (Poertner and Reindl, 1980). Storage facilities aid treatment plant efficiency by temporarily holding untreated water, thereby reducing the number and volume of overflows. Downstream sewer systems are less likely to surcharge and cause basement and street flooding. Channel and pipe sizes downstream of a detention facility may be reduced at cost savings.

A common runoff storage technique is on-site detention (OSD), which has been applied since the early days of SWM and is described in many publications (APWA, 1981). OSD is a means of temporarily storing excess runoff at its source, upstream of the major receiving water. OSD facilities are designed primarily to reduce peak runoff flows and may or may not provide all of the above mentioned benefits.

In practice, OSD storage can be created on parking lots, on roofs, in parks (dry basins) or in small man-made ponds. Roof, parking lot and park storages have the obvious advantage of being dual land-use alternatives and thereby requiring no additional space dedicated soley for storage. Storage is effected by using commercially available devices to restrict outflow from roofs or into catchbasins. This land sharing feature makes OSD storage a popular and economical alternative.

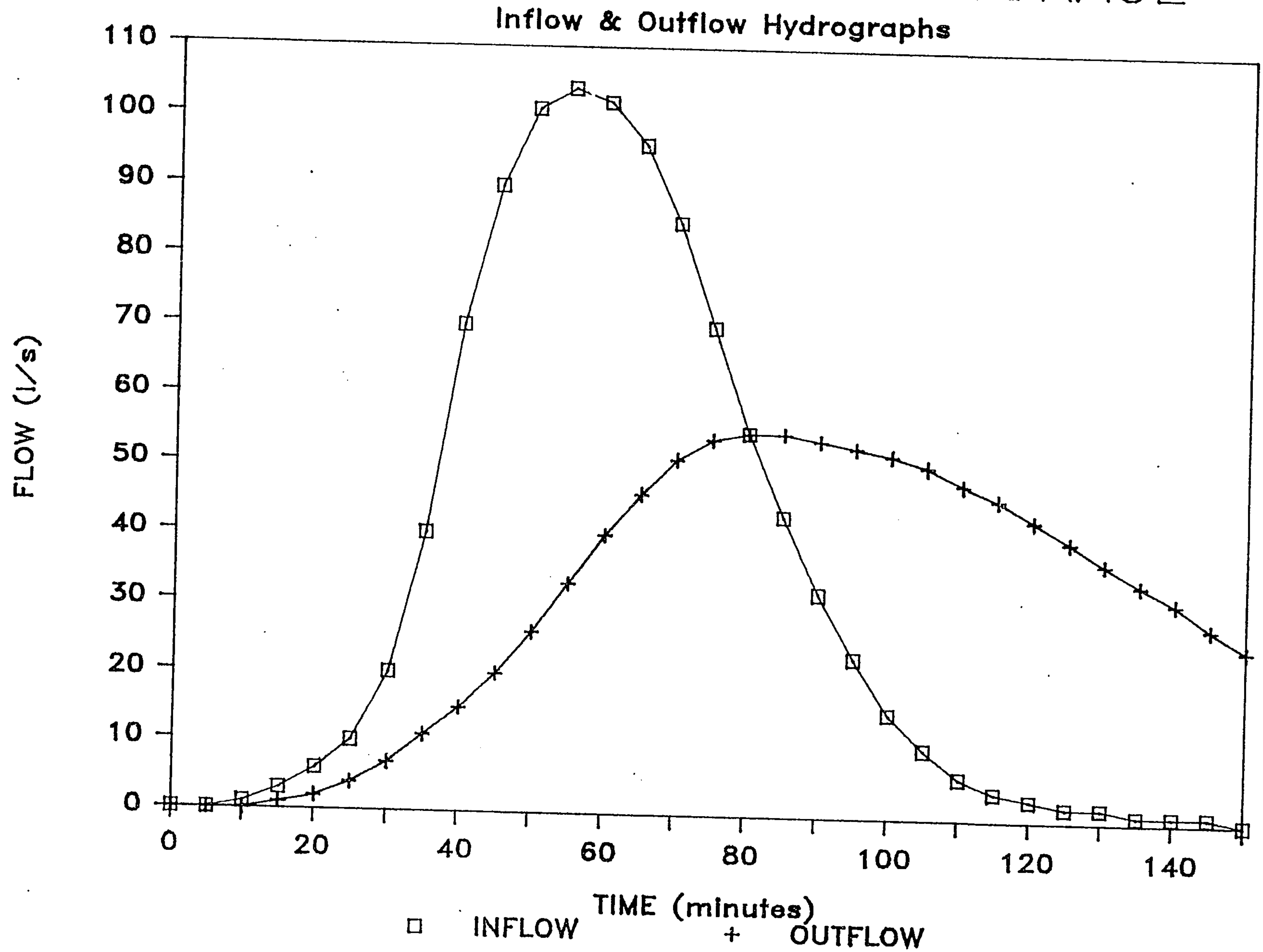
Sizing of OSD storage is a matter of generating inflow and outflow hydrographs and estimating the necessary storage volume (Figure 1). As with other components of urban drainage systems, detention facilities have, historically, been sized with the design storm approach and single event simulation. Beginning with various versions of the rational method event simulation has progressed to include the use of sophisticated computer models.

Although multi-event or continuous modeling is now a practical reality and is widely supported by recent literature, it is rarely used by practising engineers for the design of detention facilities. Continuous models can simulate responses over time periods comparable with the life span of the project. They eliminate the problems of estimating antecedent hydrologic conditions and inter-event times and are considered the most reliable means of checking a design.

1.2 Purpose of the Investigation

Research needs identified at the conference on Stormwater Detention Facilities - Planning, Design, Operation and

FIG. 1 — SIZING OSD STORAGE



Maintenance in 1982, included the following:

1. Review of available design methods including capabilities and limitations for all methods presently in use including continuous simulation models.

2. Development of generalized design parameters relating to basin size, configuration and performance incorporating catchment size, urbanization ratio, inflow-outflow ratio and runoff factors.

3. Meteorological data studies of the relationships between standardized rainfall data and real storm events and between individual events and groups of events. All known design procedures are based on design rainfall data and do not provide information on the effect of real storms on real basins. Sequences of events have a vital relevance to the drawdown time assumed in the design and operation practice of a detention basin.

This study investigates the validity of sizing OSD facilities by applying single event design methods in the light of continuous simulation. It endeavors to provide practising engineers with some general information about the relative accuracy of the two methods. Such a comparison has not yet been reported in the literature. An attempt is made to determine their compatibilities and limitations.

A further objective of this study is to develop a method of producing simple guidelines for typical design conditions on the basis of multi-event simulations. General design principles are sought which account for watershed area, imperviousness and OSD outflow rates.

1.3 Scope of the Investigation

The study at hand is broadly concerned with the hydrologic sizing of storm water detention facilities, specifically those with low release rates or outflow rates. For low release rate conditions, sophisticated hydraulic reservoir routing techniques are unwarranted. Validity of the present design storm approach of generating hydrographs for storage sizing is specifically addressed as well as the hydrological inaccuracies of this practice.

Storage volume estimations are compared by event modeling and continuous simulation. Continuous simulations were performed by the Hydrologic Engineering Center (HEC) Storage Treatment Overflow Runoff Model (STORM) which ignores overland and reservoir routing and models outflow as a constant release rate. STORM has undergone extensive calibration and validation testing as outlined in a report by Proctor and Redfern and MacLaren (1975). Long term continuous simulations were made for two cities: Montreal and Toronto. In keeping with STORM input requirements, the rainfall hyetograph series was entered as in one hour time intervals. Although coarse for hydrograph shape simulation, one hour intervals provide for adequate volume computation. Unfortunately, snowfall is measured by different field equipment and recorded in a different format. For this reason snowmelt computations were excluded from the analyses.

Single event simulations were performed with the URBHYD submodel of the OTTHYMO (University of OTTawa HYdrologic MOdel) package (Wisner, 1982). Synthetic 3-hour, 5-year and 10-year Chicago storms were considered to be typical of design storms presently used in Canadian practice. Storage volumes were estimated graphically using a linear outflow hydrograph approximation. More sophisticated reservoir routing techniques were considered unwarranted for the case of low release rates.

2. DESIGN PRACTICES: PAST AND PRESENT

2.1 Past Methods

Typically, detention design procedures of the past employed a "design storm" as input to a hydrograph generating technique which formulated both pre-development and post-development discharge rates. Most synthetic design storms, such as constant intensity, Chicago, Huff or SCS storms, are characterized by a specific recurrence frequency, duration and temporal distribution (Ormsbee, 1984).

2.2 Present Practice

In the United States, the most popular design storm return period for sizing detention facilities is the 100-year storm followed by the 10-year and then the 25-year events (Poertner and Reindl, 1980). In Canada, design storms commonly used for detention pond sizing and for pollution control studies are generally smaller: with 5 and 2-year recurrence intervals.

There are as many techniques for generating runoff hydrographs as there are storage sizing methods. The most popular, the rational method, was acceptable by 80% of the American public agencies polled. On the other hand, 20% of the agencies polled rejected the rational method as a valid design alternative. The second most popular procedure involved some form of unit hydrograph determined from recorded rainfall-runoff data. Following, these were the modified rational method and the SCS curve number procedure. Even less frequently used was some form of computer simulation model, among the most widely applied of which are: SCS TR-20, SWMM, HEC-1, HEC-2, ILLUDAS and STORM. All except STORM are single event models.

An in-house survey of 21 Canadian consultants, conducted in 1986 by the Implementation of Storm Water Management (IMPSWM) group University of Ottawa, revealed that 50% of those surveyed did not apply empirical methods for any of their storage sizing design work. OTTHYMO or HYMO, SWMM and ILLUDAS were used by 90%, 67% and 33% of the respondents, respectively. Less than one quarter of those surveyed applied other techniques, which would include continuous modeling.

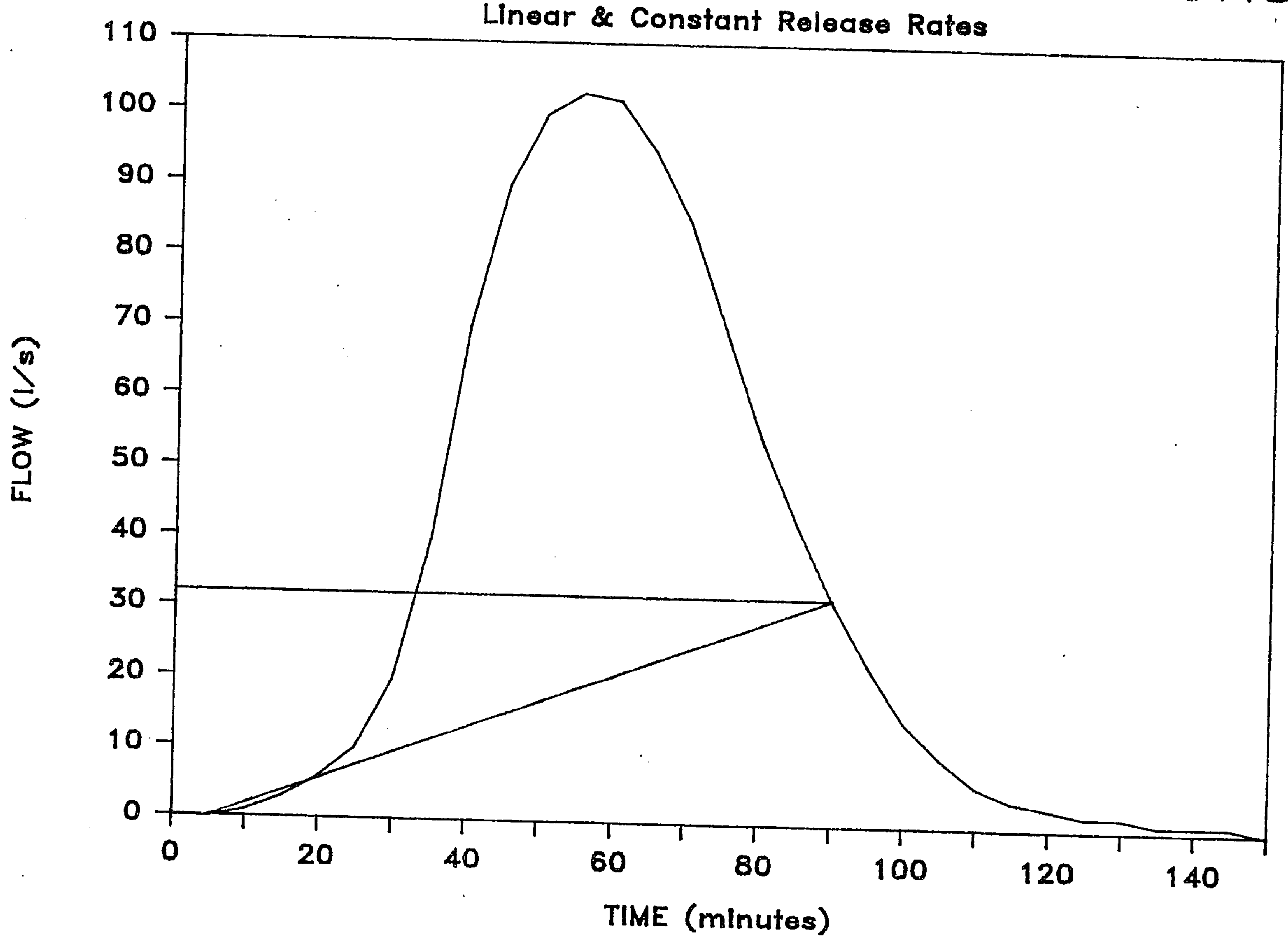
Most present day detention volume estimation is of the following form (Duru, 1981): an inflow hydrograph is routed hydraulically through the reservoir using either an orifice or weir flow equation and a simple stage-storage relation. The procedure itself does not yield a required storage volume explicitly, but forms one step in an iterative design process.

For OSD design, specific outflow-storage relations are often not available and simple assumptions, such as constant outflow, linear release rates or some other approximation, reflecting the hydraulics of the system, are made (Figure 2). Whether full reservoir routing is performed or simplifying assumptions are made depends on the degree of accuracy warranted by the specific project at hand. Although the methodology is old, the tools for generating hydrographs and routing them through reservoirs - the computer models - are new.

2.3 Current Trends in Canada

Examination of several recent projects indicates a significant variation in OSD minimum release rates used in design. Criteria of the Town of Markham require a minimum release rate of 40 l/s/ha for roof storage. In order to accommodate specific constraints, the trend in other municipalities is towards lower release rates. For a particular industrial park in Canada, an OSD was designed with a release rate of 17 l/s/ha (6.1 mm/hr). An even lower value of 6 l/s/ha (2.2 mm/hr) was reported for another project where the construction of a large storage facility was economically feasible. Lower release rates are becoming more viable and are being encouraged by the promotion of commercially available control devices such as the "Hydrobrake", roof controls, etc.

FIG. 2 — OUTFLOW APPROXIMATIONS



Low release rates are often used in quality control ponds to increase the detention time which allows the natural process of sedimentation to reduce pollutant concentrations. For example, if a 15 mm runoff is detained and released in 24 hours, the outflow rate should be less than 1 mm/hr (2.8 l/s/ha). Storage tanks recently designed in Metro Toronto to reduce basement flooding and for runoff quality control are based on small release rates of 5.5 mm/hour (15 l/s/ha). However, many quality control studies, as with quantity control, single event simulation is still used.

3. LITERATURE REVIEW OF DESIGN TECHNIQUES

3.1 Event Modeling Techniques

Modeling can be roughly divided into two classes: event and continuous simulation. Between these two poles are transitional methods which do not strictly fit in either category. The most applied class of methods, event modeling, can involve synthetic or historical storms but it always involves a *single rainfall event* which yields a single design hydrograph.

The rational method, although chiefly a peak flow technique, has been augmented to produce volume estimates and design hydrographs and was deemed the modified rational method - MRM (APWA, 1981; F.C. Houston, 1983). The MRM is characterized by artificial triangular or trapezoidal hydrograph shapes. Routing and volume estimation may involve assuming a linear outflow rate. A critical storm duration, one which produces the largest storage volume, is sought.

Going a step further, Hall et. al. (1978) advocated using a unit hydrograph procedure to generate inflow hydrographs for storm durations - varying between two and five times the lag time - to systematically determine the maximum required volume.

Results of the various event type storage estimation methods may differ significantly. Rossmiller (1982) found that several variations of the rational method produced higher storage volumes than SCS methods. The way in which inflow and outflow hydrographs are generated is one of chief relevance to the accuracy of the final storage volume estimated. Coyler (1982) suggests that "hydrograph formulation is the source of greatest error in the entire design process, owing to fundamental disagreement among competing methods and to imprecision of measurement of field data required."

In addition to hydrograph formulation, accuracy in reservoir routing is also important for sizing detention facilities. Boyd (1982) compared various linear release rate assumptions for a triangular inflow hydrograph and found that all three of the assumptions studied underestimated the required volumes by 5 to 50% when compared with conventional reservoir routing. The assumption of a constant outflow rate gave the worst underestimation. A release rate increasing linearly from zero to the maximum outflow rate gave the best volume estimation.

By virtue of their simplicity, event methods overlook some important hydrologic factors and, therefore, event-based designs may still need to be checked. Fok et. al. (1981)

advocate the testing of all detention facilities with the largest observed storm or 24-hour statistically based events in excess of this. They point out that the storm duration is a relevant parameter affecting volume.

3.2 Transition Methods

Between event modeling and strictly continuous simulation lies a whole category of methods that share characteristics of each to a greater or lesser extent.

Walesh et. al. (1979) have developed the Historic Storm Technique (HST) which consists of collecting hyetographs and antecedent moisture conditions for major storm events, generating hydrographs with an event model and selecting annual peak discharges from the output for statistical analysis resulting in a volume frequency distribution. The HST is restricted to basins where rainfall, not snowmelt, causes major flooding. Aitken and Goyen (1982) also reported a simplified detention sizing technique based on a runoff volume frequency distribution which was never put into practice. While representing an improvement over the design storm approach, the method is, nevertheless, inferior to continuous modeling.

Verworn (1984) tested basins designed with a simplified event sizing technique by routing through the basins a series of 81 hydrographs simulated from 21 years of record. Each of the detention basins were sized for various combinations of release rates and storm recurrence intervals. The number of overflows (failures) gave a better indication of the actual recurrence interval of violations than did the design recurrence interval.

Findings indicated that the simplified event sizing method sometimes overestimated and sometimes underestimated basin volumes. Those basins designed for lower outflow rates tended to to be underdesigned and less safe. Verworn reasoned that there is a a higher chance of a large runoff event finding a low outflow basin full or partly filled leading to an overflow. Of course, such antecedent conditions cannot be accounted for in simplistic design techniques. Because he found no simple relationship between the design storm and the actual recurrence interval of the overflows, Verworn determined the need for new design procedures which consider rainfall variation, catchment characteristics and basin release rates.

Ormsbee (1984) has suggested finding one year in a long period of continuous simulation which is statistically similar to the entire period of record. The representative year could subsequently be applied independently. This "poor man's" continuous simulation technique may encounter difficulties if substantial start-up time is required to reach modeling equilibrium or if a typical year is not to be found.

3.3 Continuous Modeling Techniques

In contrast to event models, continuous simulation models use long records of continuous meteorological data to formulate long-term streamflow records which lend themselves to statistical analysis. Antecedent soil moisture conditions, depression storage, detention basin water levels and other sensitive parameters are monitored and continuously adjusted internally throughout the simulation period. Single event methods typically do not specify antecedent conditions nor can they account for dynamically changing watershed parameters.

Although continuous simulation is generally considered to produce results technically superior to event modeling, consultants have had economic concerns about applying continuous models, and rightly so. Continuous modeling requires more meteorological input data and, consequently, consumes more computer time.

Several continuous simulation computer models are currently available. In Canada, the more popular are: STORM, Continuous SWMM (Huber et al., 1980) and QUALHYMO (Rowney and Wisner, 1985).

Raasch (1979) proposed an urban stormwater detention sizing (USDS) technique based on analytical and statistical analysis of continuous hydrologic simulation results. By generating long term runoff files for specific land uses, soil types and meteorologic combinations, annual peak volumes are identified and analyzed to produce volume-probability relationships. Subsequent application of USDS involves using the runoff storage volume relationship appropriate for the land use, basin release rate and time required to drain the detention facility. In some respects, the methodology is similar to the present study, however, no comparison was made with single event sizing.

Padmanabhan and Delleur (1982) proposed a conjunctive use of the STORM and ILLUDAS models. STORM, a continuous model which simulates lumped storage in a basin, defines rainfall events in terms of system characteristics and not simply as meteorologically dependent. STORM provides overflow frequencies for various combinations of treatment rates and total basin storage volumes on a macroscopic scale. Individual detention sizes and locations are then analyzed with ILLUDAS.

Literature reviews reveal that there are less detention pond sizing methods based on continuous simulation than methods involving event simulation.

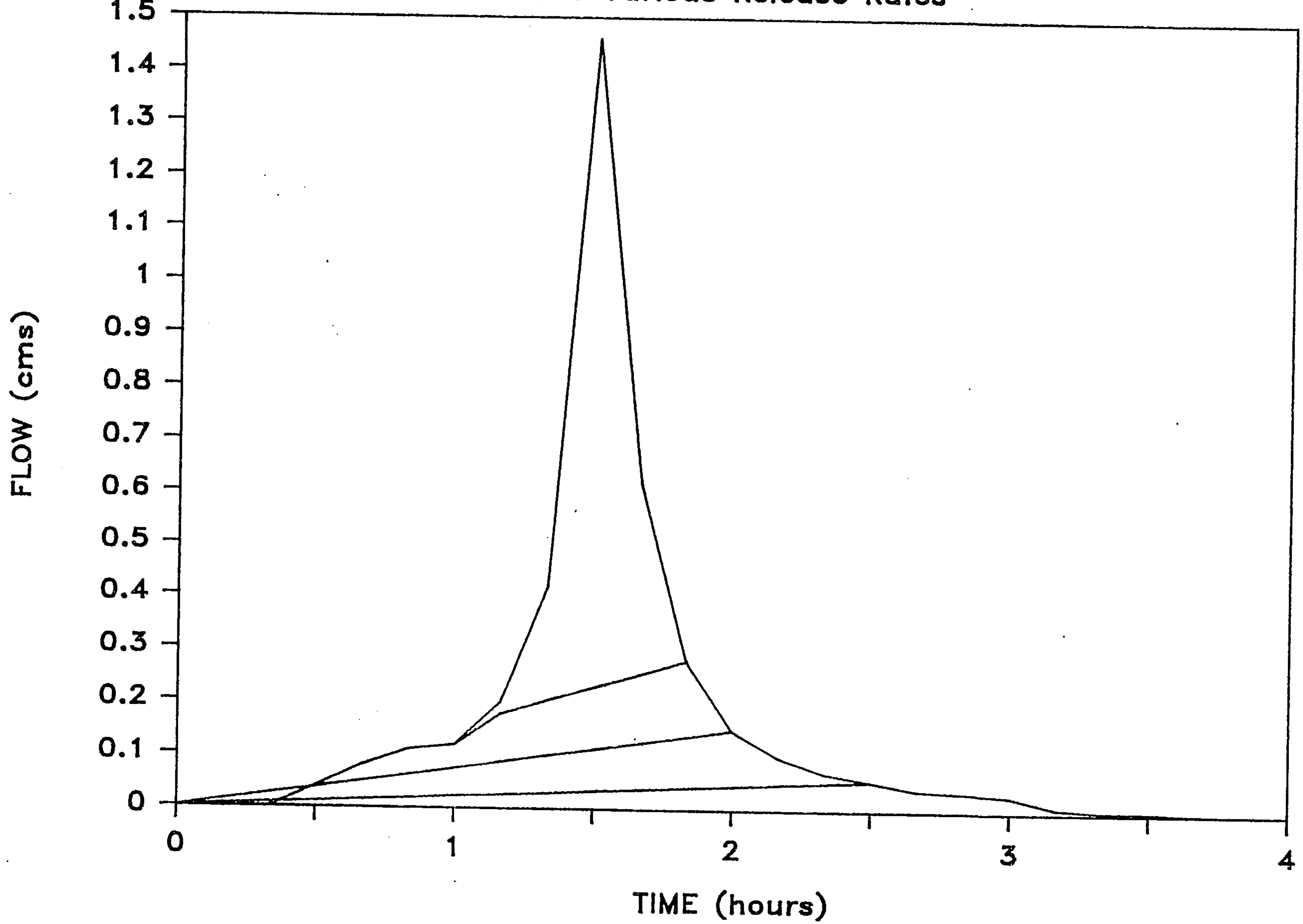
4. HYDROLOGIC CONSIDERATIONS

As the release rate from an OSD storage is reduced, the shape of the inflow hydrograph and the outflow-storage relationship become less important for storage volume computation (Figure 3). The detailed hydrograph modeling performed in some projects becomes unwarranted and an approximate runoff volume estimation becomes more important. SCS has recognized this difference in the degree of modeling sensitivity used in sizing storage volumes. TR-55 (SCS, 1975) states that "when only a small volume is available for temporary storage, the shape of the outflow hydrograph is very sensitive to the rate of rise of the inflow hydrograph. Conversely, when a large volume is available for storage, the shape of the inflow hydrograph has little effect in the outflow hydrographs which, in this case, is controlled by the hydraulics of the structural system."

On the other hand, one should ask whether one event simulation is indeed valid for both of the above conditions. Woolridge (1982) discovered that "the critical storm duration for the design of storage tanks is about 30 to 40% longer than the critical duration for the calculation of discharge." The storm duration giving the maximum storage varies according to local climate and is, therefore, site specific. Unfortunately, design storms which give the highest peak flows are often indiscriminately used for

FIG. 3 — STORAGE COMPUTATION

For Various Release Rates



storage volume estimation. Events with lower intensities and higher runoff volumes or sequences of events may be more critical than high intensity storms.

Walesh (1979) has given the following definition for design storms:

> "The overall idea of a design storm is to provide a means of estimating a discharge or volume of specified recurrence interval for planning or design purposes. That is, a recurrence interval is assigned to the design storm, a rainfall-runoff procedure is used to convert the rainfall to runoff, and the recurrence interval of the design storm is transferred to the resulting runoff discharge or volume."

Sieker (1979) showed that the postulate of transferring the storm recurrence interval to the hydrograph is false. James et. al. (1982) stated that "storage design is more reliable if based on continuous modelling rather than discrete event modelling" and proceeded to point out several drawbacks of using design storms. Synthetic design storms represent an aggregation of storms of all types, cyclonic, thunderstorms etc., into a single distribution which bears no resemblance whatsoever to any of the observed storms. The probability of generating a specific runoff volume is actually the joint probability of the initial conditions and the rainfall intensity. Padmanabhan and Delleur (1982) also indicate that the number of overflows from a detention is a function of the storage and release rate. The design storm approach considers only the input with no regard for the system characteristics - thus no scope for considering alternatives or economic analysis is possible.

Another major weakness of design storm approach according to Ormsbee (1984) stems from its inability to generate statistical information such as flow duration curves and probability distributions of outflows and overflows, which are direct functions of the available storage volume and the release rate. From this perspective, design storms are simply inapplicable.

Marsalek (1978) conducted a study in which he found that the popular Chicago design storm generated runoff peaks that were 80% larger than produced by corresponding historic storms of identical return periods and that the ISWS (Illinois State Water Survey) storm overestimated by about 27%. Return storms of between 1 and 10 years were used in both cases. Although not reported, errors in runoff volume would also be expected.

Wenzel and Voorhees (1978) studied design storms by comparison with recorded events and continuous models and found that "surcharging and antecedent conditions dominated tending to render the design storm concept of doubtful value."

According to Fok et. al. (1981), the primary advantage of continuous modeling is to better determine the frequency of antecedent conditions such as soil moisture levels and antecedent detention water levels. Such probabilities are generally unknown in the design storm approach. Continuous simulation affords the opportunity to observe the frequency of overflow violations for a particular OSD design.

Although design storms have many serious limitations, there exists no widely accepted alternative. The trend, however, has been gradually away from single event input to a series of historical storms or to continuous modeling.

Linsley and Crawford (1974) advocate continuous simulation for water quality modeling because pollutant accumulation depends on antecedent dry-days. No reliable probabilities can be assigned to single events because the method is not suitable for statistical analyses. It is the higher frequency events which account for the majority of the combined sewer overflows and thus the largest annual pollutant loads. Therefore, designing a system for rare events is uneconomical.

This is certainly not an exhaustive list of all the researchers who advocate continuous simulation over design storms. Johansen and Harremoes (1979), Urbonas (1979) and Padmanabhan and Delleur (1982) as well as many others, have, through much study, also come to this opinion.

5. STUDY METHODOLOGY

To test the performance of OSD's sized by event methods using continuous simulation, the fundamental differences in input requirements, procedures, assumptions and output of the two methods had to be taken into account. Two representative models were selected for simulation:

1. URBHYD, a submodel of OTTHYMO (Wisner, 1982), and
2. STORM (Hydrologic Engineering Center, 1976).

These two models are similar in some respects. Both are algorithmically simple, both perform lumped hydrologic simulation, and both have been well tested and applied.

URBHYD, an event model, is simply a runoff hydrograph generating tool. Used as a stand alone model in this study, URBHYD does not simulate storage or reservoir routing. STORM, a continuous model, generates only hourly runoff volumes, but also simulates storage and outflow. Whereas URBHYD is applied with a design storm, be it synthetic or historical, STORM requires a continuous hourly precipitation record. It is imperative to have a general understanding of each model before analyzing, interpreting and comparing results.

5.1 Event Simulation

5.1.1 The URBHYD Model

URBHYD, a submodel of the OTTHYMO package is widely applied across Canada and is representative of event hydrograph simulation methods currently applied in practice. A quasi-linear parallel reservoir model, it simulates runoff from pervious and impervious areas separately and adds the two routed hydrographs at the outlet point. Previous comparisons show that hydrographs generated by URBHYD are comparable with those generated by discretized SWMM simulations. SWMM itself, was previously compared with HVM-Dorsch and RRL. Because of these comparisons, conclusions of this present study are not considered to be restricted to a particular single event model.

Input to URBHYD consists of the watershed area, the impervious ratio, infiltration parameters for pervious areas, hydraulic parameters for routing - slope, length and roughness - and the rainfall hyetograph in a user specified time step. Output includes the runoff hydrograph which, in OSD sizing, represents inflow to the detention facility.

5.1.2 Event Simulation Strategy

To be consistent with present practice, 5 and 10-year Chicago design storms of three hour duration were derived from Intensity Duration Frequency (IDF) curves for Toronto and Montreal. Since the watersheds were hypothetical, hydrographs were generated for the following characteristics:

a) imperviousness = 20%, 40%, 60%, 80%

b) watershed area = 2, 5, 10, 25, 100 Ha.

Watershed slope, which has no effect on runoff volume, was arbitrarily set to 1.0% for all URBHYD runs. All other parameters were set to the default values recommended in the user manual (see Table 1).

Preliminary indications showed that watershed area had no significant effect on the depth of runoff generated per unit area. Consequently, all subsequent simulations were for a 10 ha area and imperviousness became the primary watershed variable.

To be consistent with the units in the STORM model, volumes were converted into mm per unit area and release rates into mm per hour. Storage requirements were computed for the

TABLE 1 - URBHYD PARAMETER VALUES

MODEL VARIABLE	UNITS	VALUE	COMMENTS
DT	hours	0.167	Computational time step
DA	Ha	10.0	Drainage area
XIMP	%	0.2,0.4, 0.6,0.8	Directly connected impervious area ratio
TIMP	%	0.2,0.4, 0.6,0.8	Total impervious area (ratio of total watershed area)
NI	-	18	Number of rainfall increments (3 hours @ 10 min. time step)
FO	mm/hr	76.2	Initial infiltration rate (default)
FC	mm/hr	13.21	Final infiltration rate (default)
DCAY	1/hr	4.14	Horton decay constant (default)
F	mm	0.0	Accumulated soil moisture prior to start of rain (default)
DPSI	mm	1.57	Depression storage for impervious area (default)
DPSP	mm	4.67	Depression storage for pervious area (default)
STI,STP	hr	0.0	Storage coefficient of linear reservior fo impervious and pervious areas (to be computed based on the slope, length and roughness to follow)
SLI,SLP	%	1.0	Slope of impervious and pervious areas respectively
LGI,LGP	m	258.2	Length of impervious and pervious areas respectively, by default: $1.5L^2$ = Area
MNI,MNP	imperial	0.013, 0.250	Manning roughness coefficient for impervious and pervious areas.

following release rates:

1. 2 mm/hr = 5.56 l/s/ha,
2. 5 mm/hr = 13.9 l/s/ha,
3. 10 mm/hr = 27.8 l/s/ha,
4. 20 mm/hr = 55.6 l/s/ha.

Hydrographs generated by URBHYD were plotted and storage volumes for the above release rates were calculated graphically based on the straight-line outflow approximation (Figure 3).

5.2 Continuous Simulation

5.2.1 The STORM Model

The Storage Treatment Overflow Runoff Model - STORM - was selected for continuous simulation because of its conceptual simplicity and its unique definition of storm events. STORM simulates quality and quantity of runoff from urban and non-urban watersheds. STORM lumps both the watershed and the storage facilities in the watershed. Statistical information on the quality and quantity of washoff and overflows is provided. The model is designed for use with many years of continuous hourly precipitation records but may be used for individual events also.

Quality aspects were completely ignored in this study. Runoff quantity computations are accomplished in one of three ways: by the coefficient method, by the SCS curve number method and by a combination of the two. This third alternative was selected, namely the coefficient method for impervious areas and the SCS method for pervious areas.

As mentioned above, one of the reasons for selecting STORM was its unique manner of defining rainfall events. The implied philosophy is that a design storm cannot truly be defined without consideration of the drainage system to which it will be applied. Intensity and duration are not the only factors involved in determining runoff or overflows. Antecedent moisture and water levels strongly influence the severity of an event.

STORM defines an event as the rainfall occurring between the time *storage* is first utilized until it empties. In other words, an event does not occur unless runoff of an intensity greater than the detention facility release rate is generated whereby the untreated or excess runoff is diverted into storage. The event ceases when the storage is allowed to empty due to the termination or reduction of runoff.

STORM input consists of rainfall data, evaporation and infiltration parameters, imperviousness, and storage size-treatment rate combinations. Among many other statistics, output includes the dates and times of events and a tally of the total number of events and overflows for each treatment rate-storage combination.

5.2.2 Continuous Simulation Strategy

Fourty-one and 26 years of continuous hourly rainfall data for Montreal and Toronto, respectively, provided input for the continuous simulation portion of the investigation. Since release rates and storage are both measured on a per unit area basis by STORM, watershed area was not a relevant parameter and results were interpreted as generally applicable for all areas. To be consistent with single event modeling, default parameters were selected for STORM in all cases of unknown hydrologic parameters. Matching parameters with URBHYD was attempted wherever possible, especially for imperviousness ratios and Horton infiltration parameters. A list of parameters and their values is presented in Table 2.

Various combinations of release rates and storage volumes (Table 2) were analyzed for both Montreal and Toronto rainfall data to determine the number and magnitude of overflows. Comparison of the two methods could have been accomplished by entering the release rates and computed storage volumes from single event modeling directly into STORM and obtaining the number overflows as output.

TABLE 2 - STORM PARAMETER VALUES

CARD	MODEL VARIABLE	UNITS	VALUE	COMMENTS
E2	AREA	Ha	100	Area of subasin (does not affect results since storage and treatment rate are measured per unit area)
E3	RECVRT	mm/day	0.1,0.4 1.0,3.0 3.4,3.8 3.8,3.4 3.1,1.3 0.4,0.1	Pan evaporation rates for each month of the year (default values used)
E4	LOSSEQ	-	3	Infiltration losses will be computed by the coefficient method for impervious areas and by the SCS method for pervious areas
	CPERV	ratio	0.20	Pervious area runoff coefficient
	CIMP	ratio	0.95	Impervious area runoff coefficient (default)
	DEPRS	mm	0.50	Maximum depression storage capacity
	EERC	-	3.0	Evapotranspiration exponent
	EPRC	-	3.0	Percolation exponent
E5	DEPR	mm	4.0	Max. initial abstraction capacity
	ACTIA	mm	4.0	Starting initial abstraction capacity
	SACT	mm	76.2	Starting soil moisture retention capacity
	SMAX	mm	76.2	Max. retention capacity
	RATEIN	mm	76.2	Max. infiltration capty.
	PERCMX	mm/hr	13.2	Max. percolation rate
F1	FIMP	%	20,40, 60,80	Percent imperviousness of this land use.
T2	TRATER	mm/hr	1,2,5, 10,20	Treatment rate
T3	CAPR	mm	2,5,10, 25,40,100	Storage capacity to be analyzed for designated treatment rate.

However, in favour of a more general approach, release-rate storage simulations were selected to represent a broader range of conditions and situations. Release rates modeled tend to be low reflecting the objectives of the investigation in light of present OSD design practices.

6. SIMULATION RESULTS

6.1 Results of Event Modeling

Graphically estimated storage volumes based on hydrographs generated by URBHYD are presented in Table 3 for the specified release rates. As the release rate of the detention facility decreases, storage approaches the total volumetric runoff. Volumes are fairly similar for both cities, as expected. It is evident that storage capacities generated with event modeling depend on the design storm return period and the release rate of the detention facility.

6.2 Results of Continuous Simulation

STORM output, because of its sheer volume, can be best presented graphically. Along with many other statistics, STORM computes the number of overflows during the simulation period for each OSD - defined by a release rate and a storage capacity - as one measure of its effectiveness. Consequently, the number of overflows is a function of the storage volume and the release rate of the OSD, and when plotted against these two parameters, produces a family of iso-release rates. Montreal iso-release rates are illustrated in Figures 4A to 4C for various impervious ratios; Toronto results follow in Figures 5A to 5D.

TABLE 3 - RESULTS OF SINGLE EVENT MODELING

IMPERV-IOUSNESS (%)	ESTIMATED STORAGE VOLUMES (mm) FOR THE FOLLOWING RELEASE RATES 2 mm/hr 0.056 cms	5 mm/hr .139 cms	10 mm/hr .278 cms	20 mm/hr .556 cms	TOTAL RUNOFF VOLUME (mm)
MONTREAL: 5-YEAR STORM					
20.0	9.8	6.4	3.7	1.0	13.2
40.0	16.3	12.1	8.1	4.0	19.5
60.0	22.7	18.2	12.5	7.5	25.8
80.0	29.2	24.7	19.1	12.8	32.1
MONTREAL: 10-YEAR STORM					
20.0	15.0	10.8	7.3	3.0	17.4
40.0	21.0	16.7	11.3	6.8	24.1
60.0	27.7	23.1	17.1	11.3	30.8
80.0	34.6	29.4	23.0	16.0	37.5
TORONTO: 5-YEAR STORM					
20.0	13.3	8.9	5.6	2.4	16.5
40.0	21.0	16.0	9.9	5.9	23.3
60.0	27.1	22.2	15.5	10.1	30.2
80	34.1	29.4	22.4	14.0	37.1
TORONTO: 10-YEAR STORM					
20.0	18.8	14.2	10.2	5.8	22.1
40.0	26.5	21.3	14.1	10.0	29.6
60.0	33.9	28.4	21.1	13.9	37.1
80.0	41.4	36.7	28.3	17.7	44.6

FIG. 4A – MONTREAL 1943–83

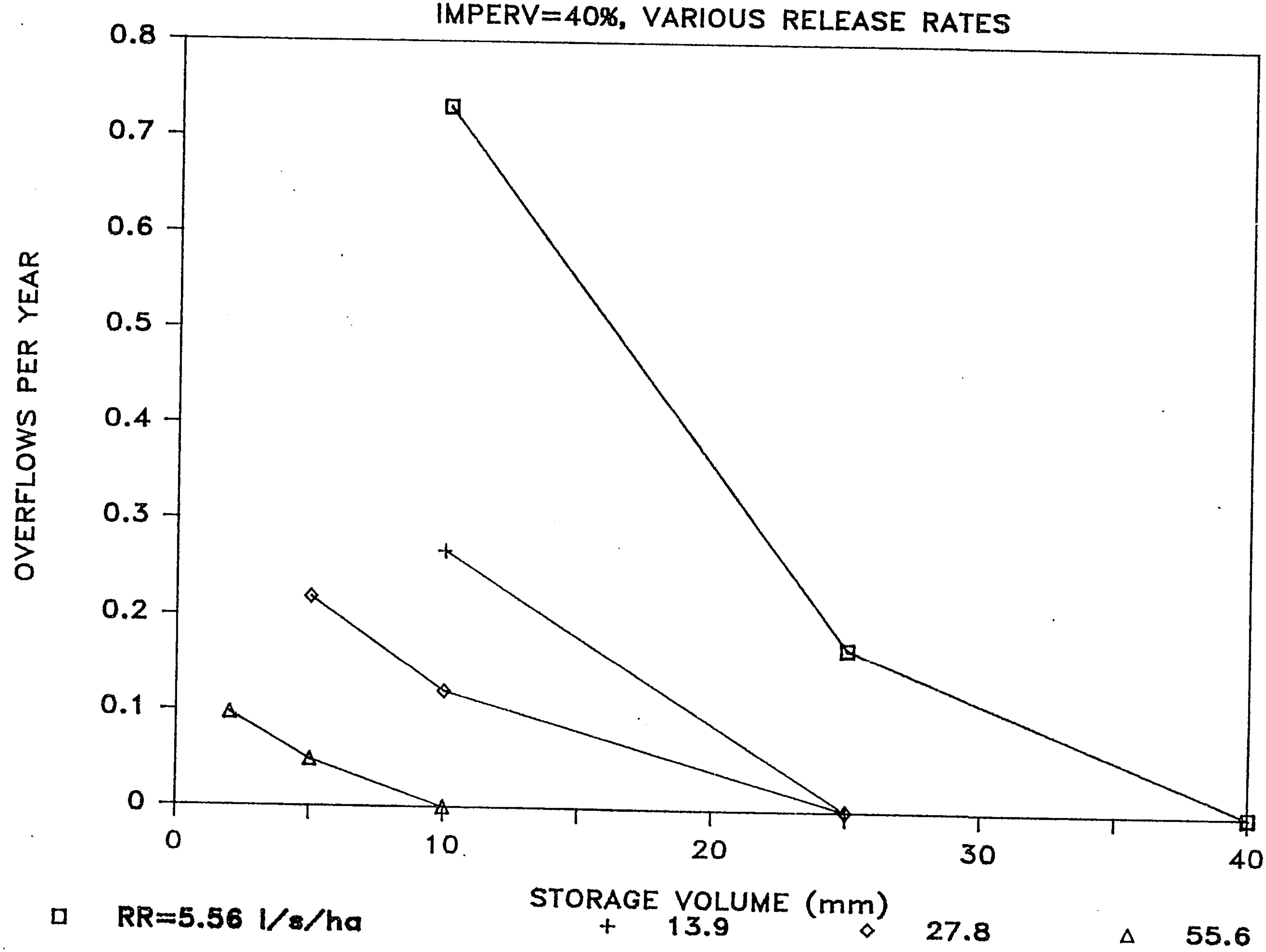


FIG. 4B — MONTREAL 1943–83

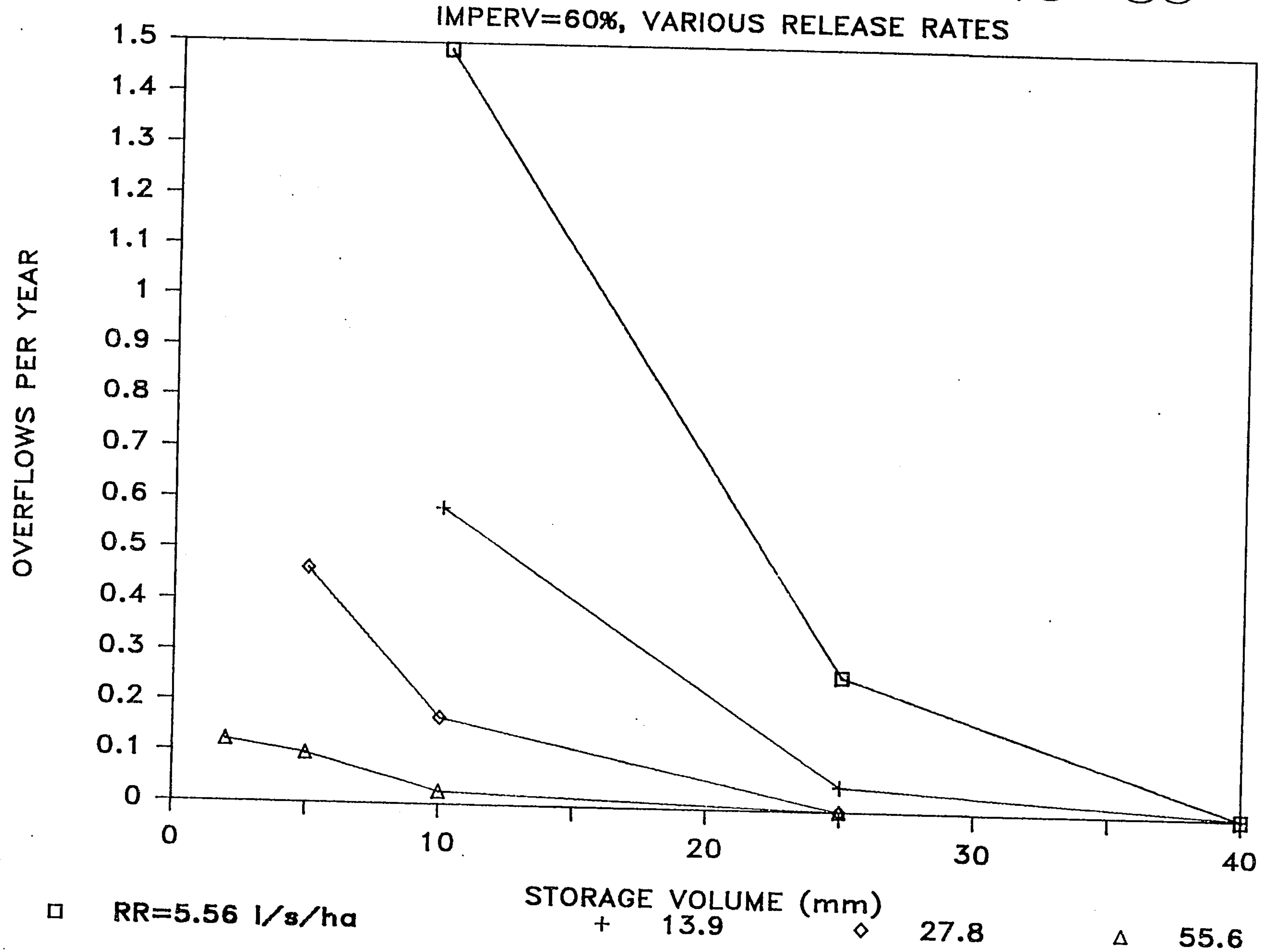


FIG. 4C — MONTREAL 1943—83

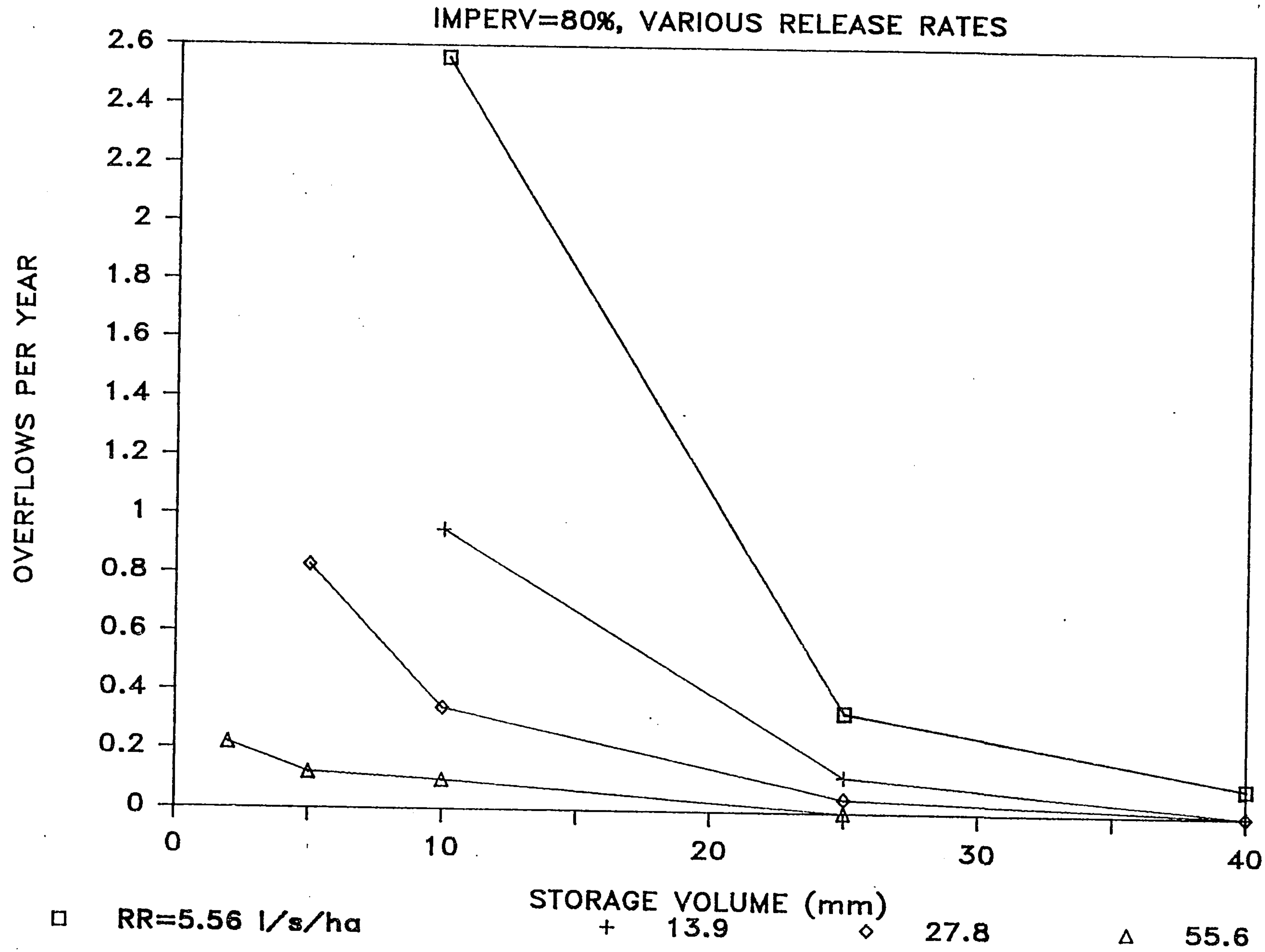


FIG. 5A — TORONTO 1960—85

IMPERV=20%, VARIOUS RELEASE RATES

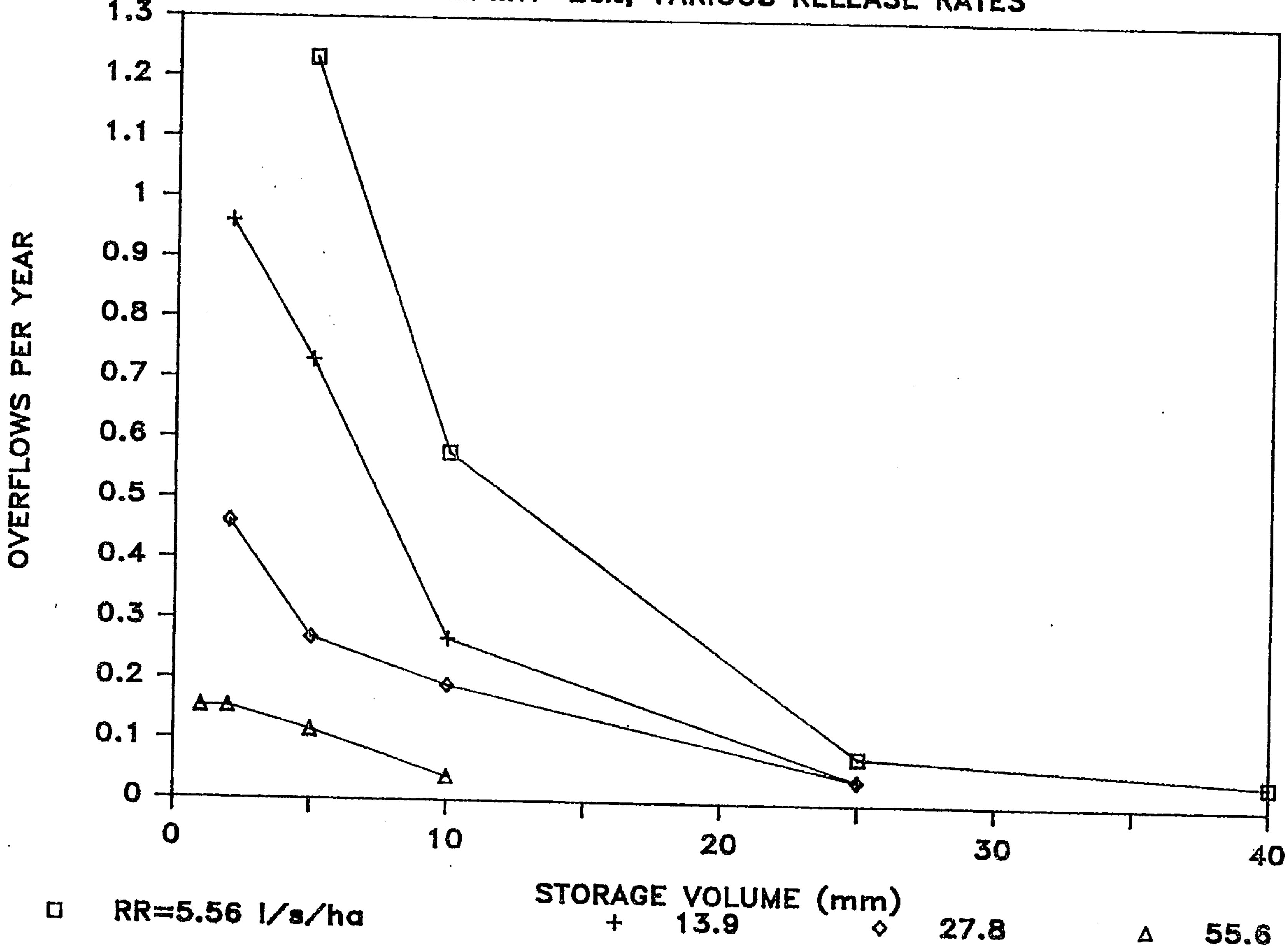


FIG. 5B — TORONTO 1960—85

IMPERV=40%, VARIOUS RELEASE RATES

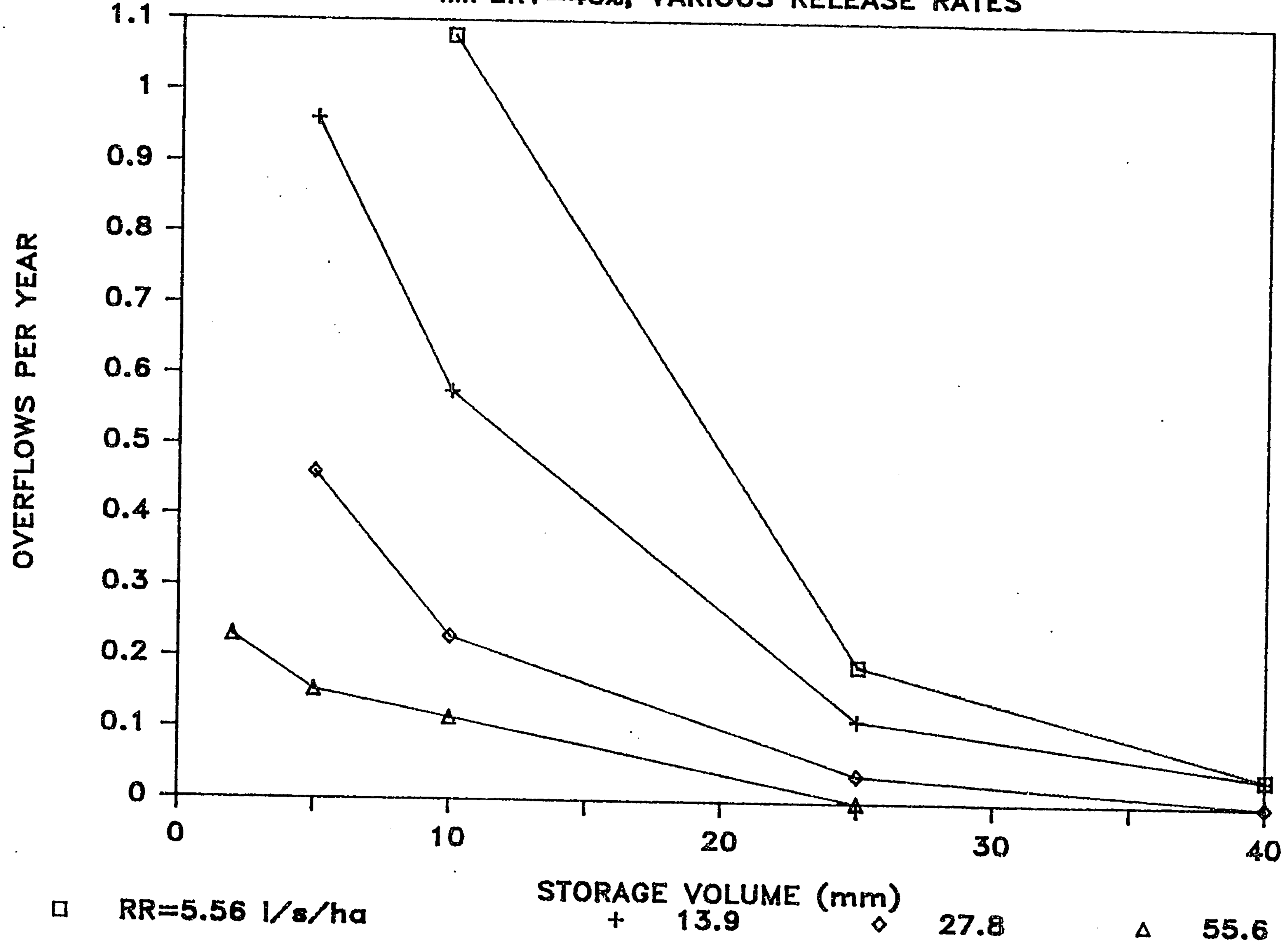


FIG. 5C — TORONTO 1960–85

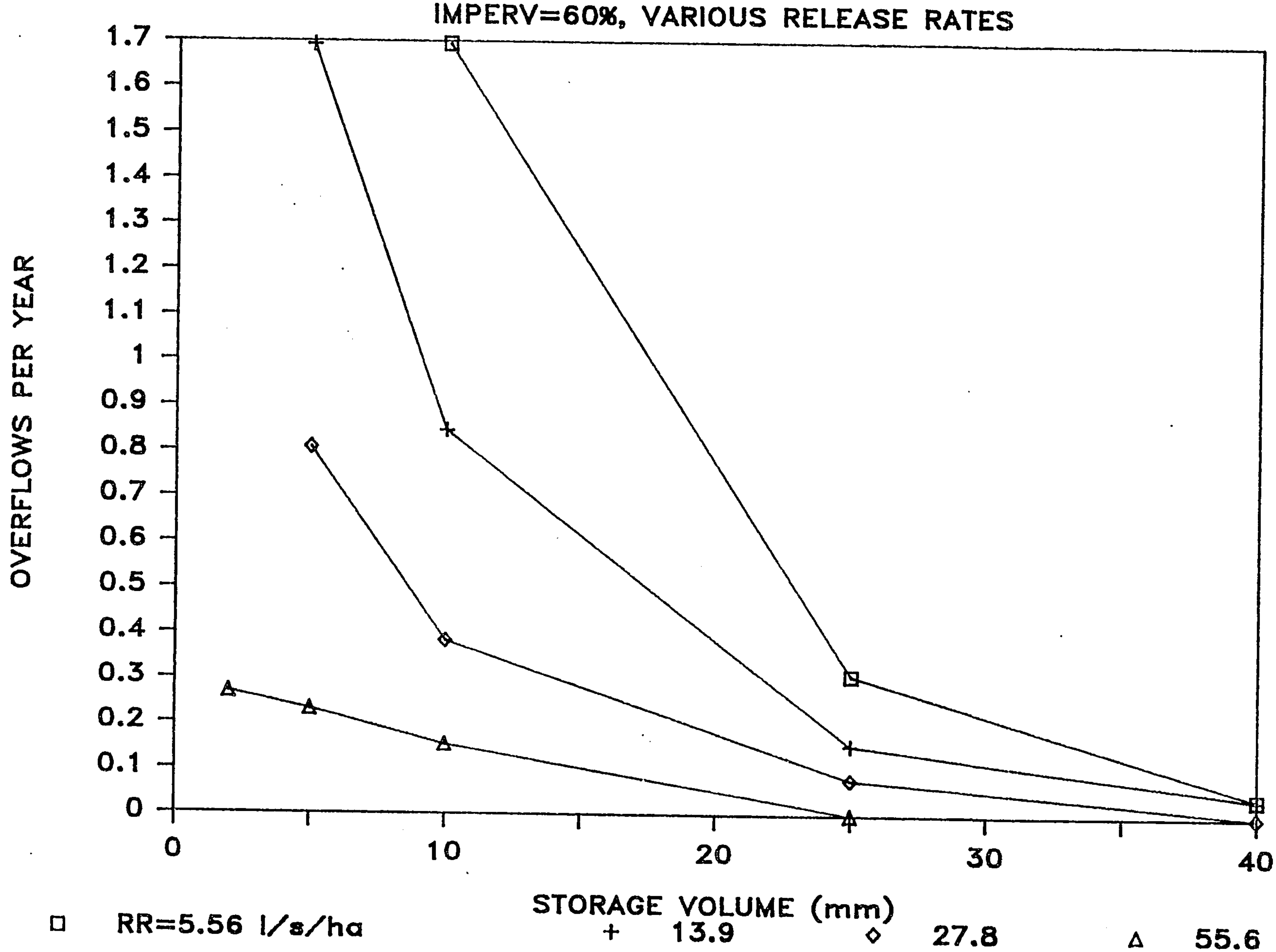
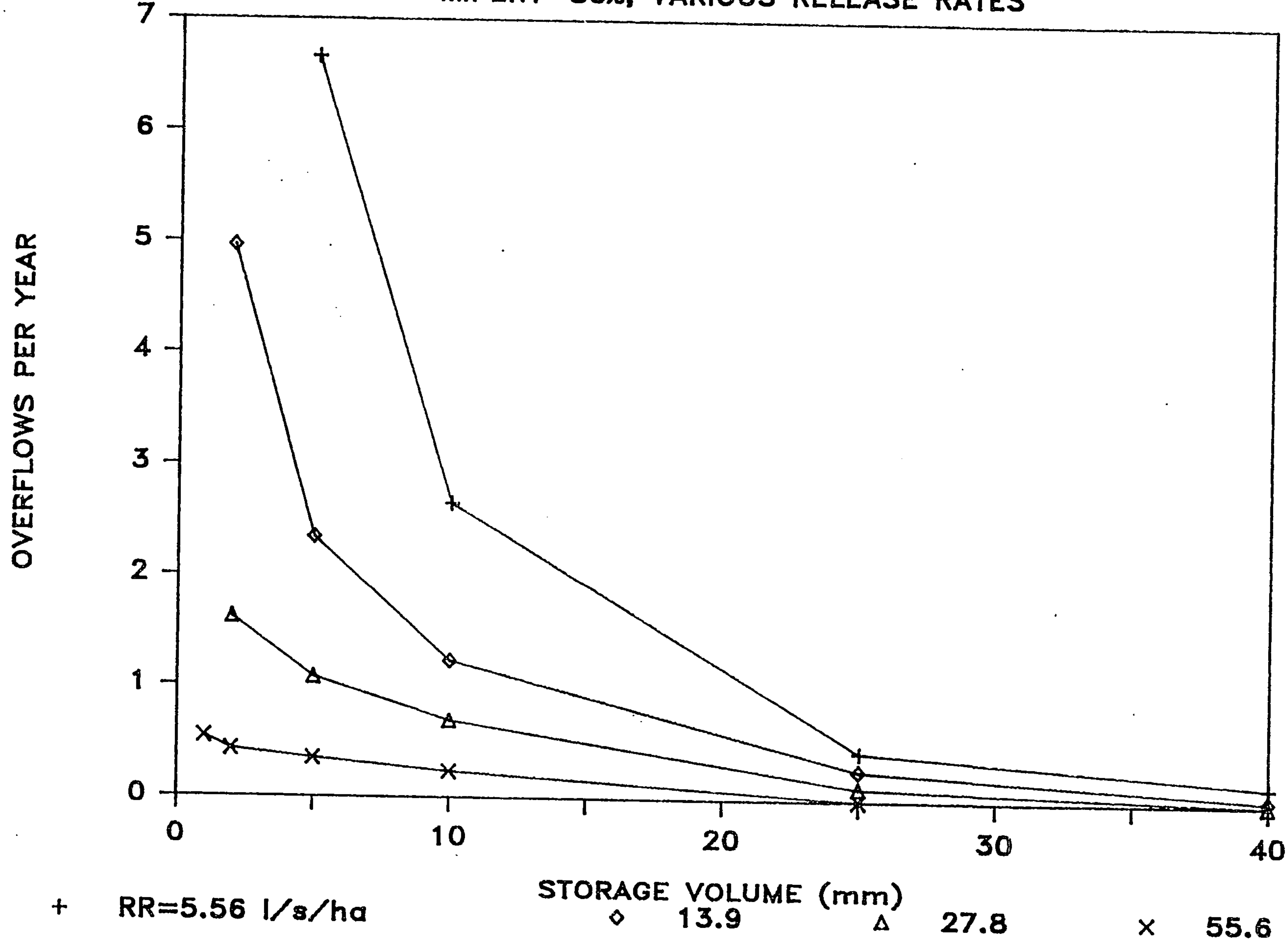


FIG. 5D — TORONTO 1960–85

IMPERV=80%, VARIOUS RELEASE RATES



Fundamentally different from URBHYD output, STORM provides the frequency or recurrence of failures (overflows) of a detention facility with a specified size and release rate. Results form Montreal and Toronto are not easily compared since the simulation periods encompass different time spans.

In addition to the number of events and overflows, STORM provided much statistical data detailing the durations, time spans and depths of events and critical (overflow) events. After compilation of duration and time scale data from the 26 year Toronto simulations the following observations were made:

A larger difference was generally noted between the total duration of the rainfall event and the number of actual hours that rain fell for overflow events than for non-overflow events, especially at the lower release rates tested. In other words, overflow events generally experienced intermittent dry periods more often than non-overflow events did.

The total hours of actual rainfall and the total rainfall depths for overflow events are generally larger than for non-overflow events.

Overflow events tended to use storage facilities for a longer period of time also. Recall that STORM defines an event on the basis of storage use. Thus, the overall duration (dry periods included) of events which caused overflows was longer than those which did not. The average overflow event duration for Toronto simulations was 6.8 hours; non-overflow events were considerably shorter on average.

7. ANALYSIS AND DISCUSSION OF RESULTS

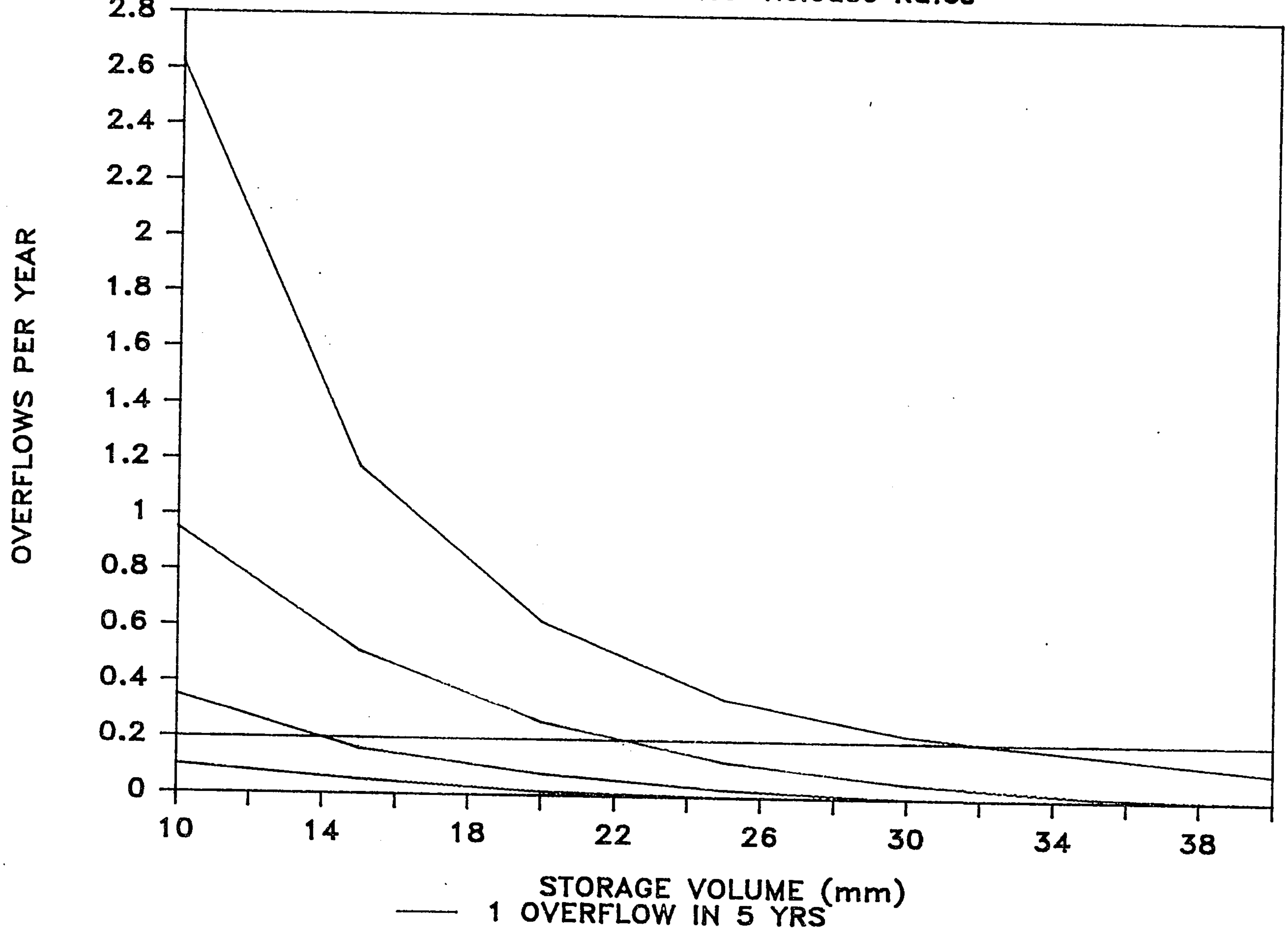
7.1 Procedure for Comparing Results

To enable a proper comparison between STORM and URBHYD results to be conducted, a number of assumptions were made. First, event modeling, by assumption, transfers the return period of the design storm to the formulated hydrograph. The frequency of failure (overflow) for an OSD designed with this hydrograph is then the recurrence interval of the design storm. In other words, a detention facility sized with a 5-year design storm should overflow only once in five years or 8.2 times in 41 years or 5.2 times in 26 years. Similarly, an OSD sized for a 10-year storm should, on average, overflow 4.1 times in 41 years and 2.6 times in 26 years.

By interpolating the iso-release rates of Figures 4 and 5 at overflow frequencies representing one failure in 5 and 10 years, it is possible to determine the volume of storage necessary, according to continuous simulation, to ensure these recurrence intervals. Graphically, overflow frequencies of one in 5 and 10 years were plotted as horizontal lines on Figures 4 and 5 (see example, Figure 6). Intersections with the iso-release rate curves define the so-called "design" storage volumes given by continuous simulation.

FIG. 6 — STORM VOLUMES

Xsections of Iso-Release Rates



In this manner, storage volumes for specific recurrence intervals, release rates and impervious ratios were estimated from STORM output. Since the single event volumes formulated with URBHYD output were functions of the same three parameters, graphical representation was essential to clearly portraying and comparing the differences between the two methods (Figures 7 and 8).

7.2 Analysis

According to Figures 7 and 8, the following three situations were observed:

1. Event and continuous simulation give similar volume estimates,
2. Event volume estimates are higher,
3. Volumes determined with continuous simulation are higher.

The first situation is illustrated by the intersections of the curves representing event modeling with the curves representing continuous simulation. At these points there is no difference in the required storage volumes given by the two methods. Although obscure and ill-defined in some cases, intersections generally occur in the range of 10 to

FIG. 7A — MONTREAL 40% IMP.

1 OVERFLOW IN 5 & 10 YEARS

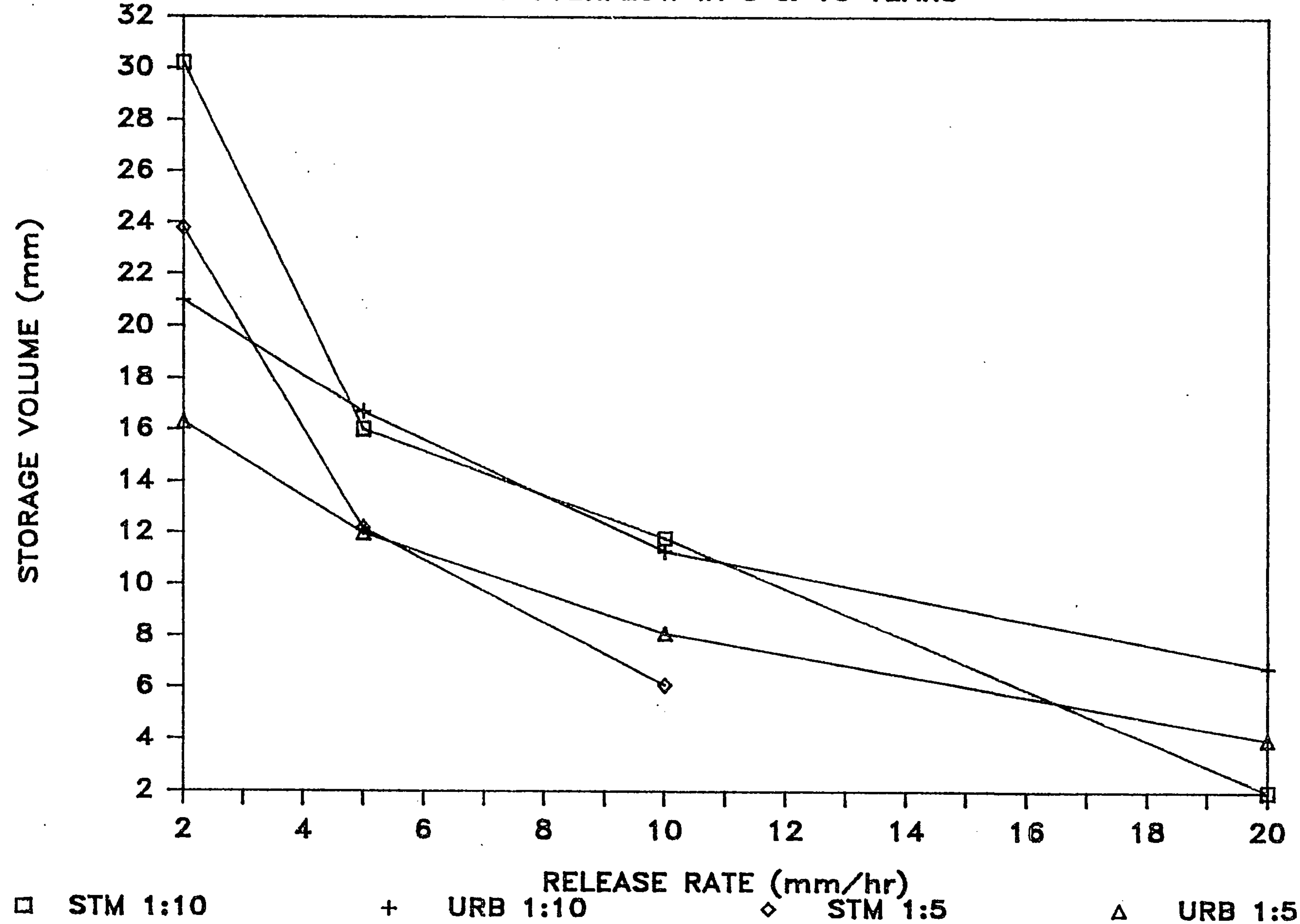


FIG. 7B — MONTREAL 60% IMP.

1 OVERFLOW IN 5 & 10 YEARS

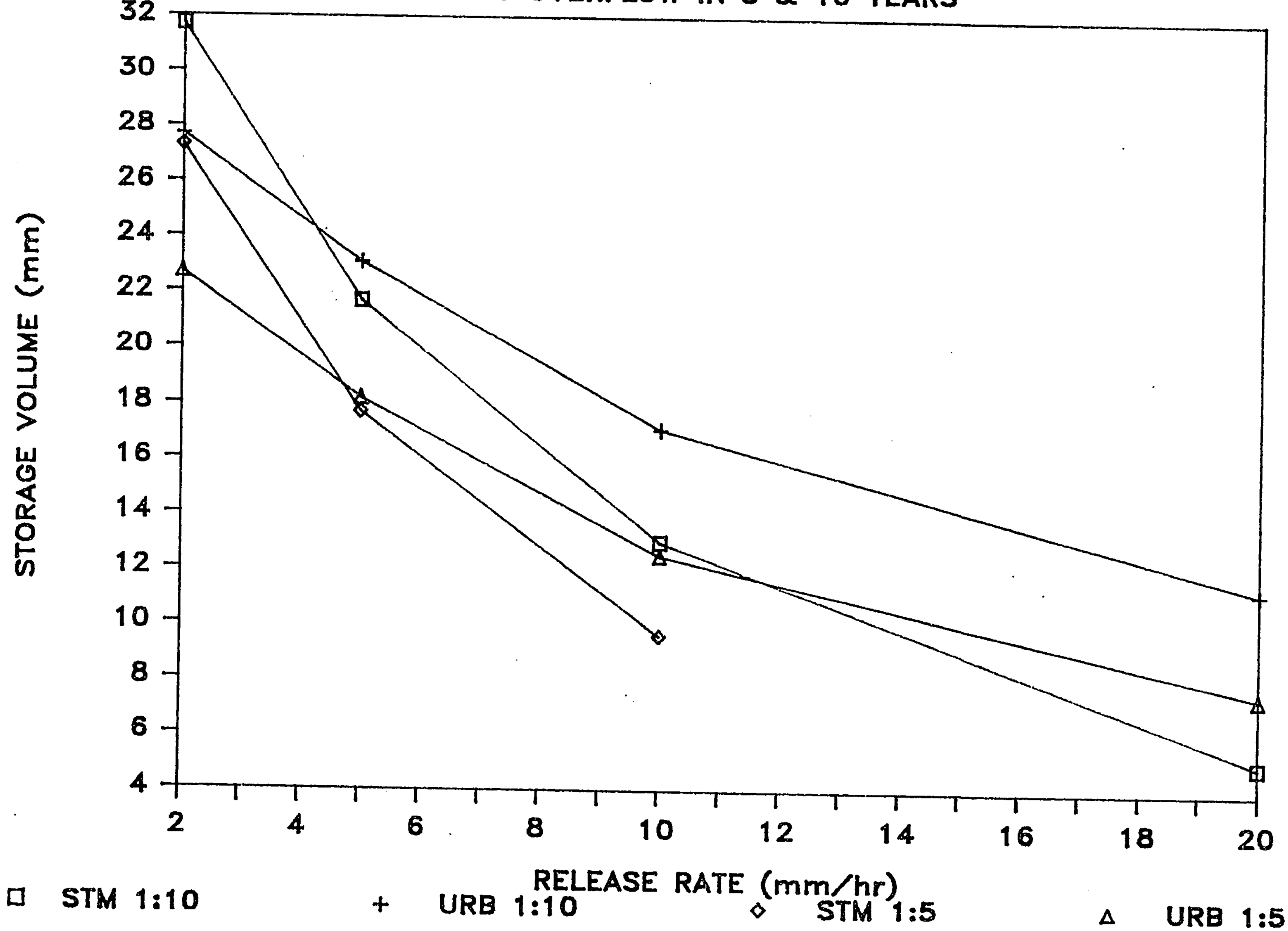


FIG. 7C — MONTREAL 80% IMP.

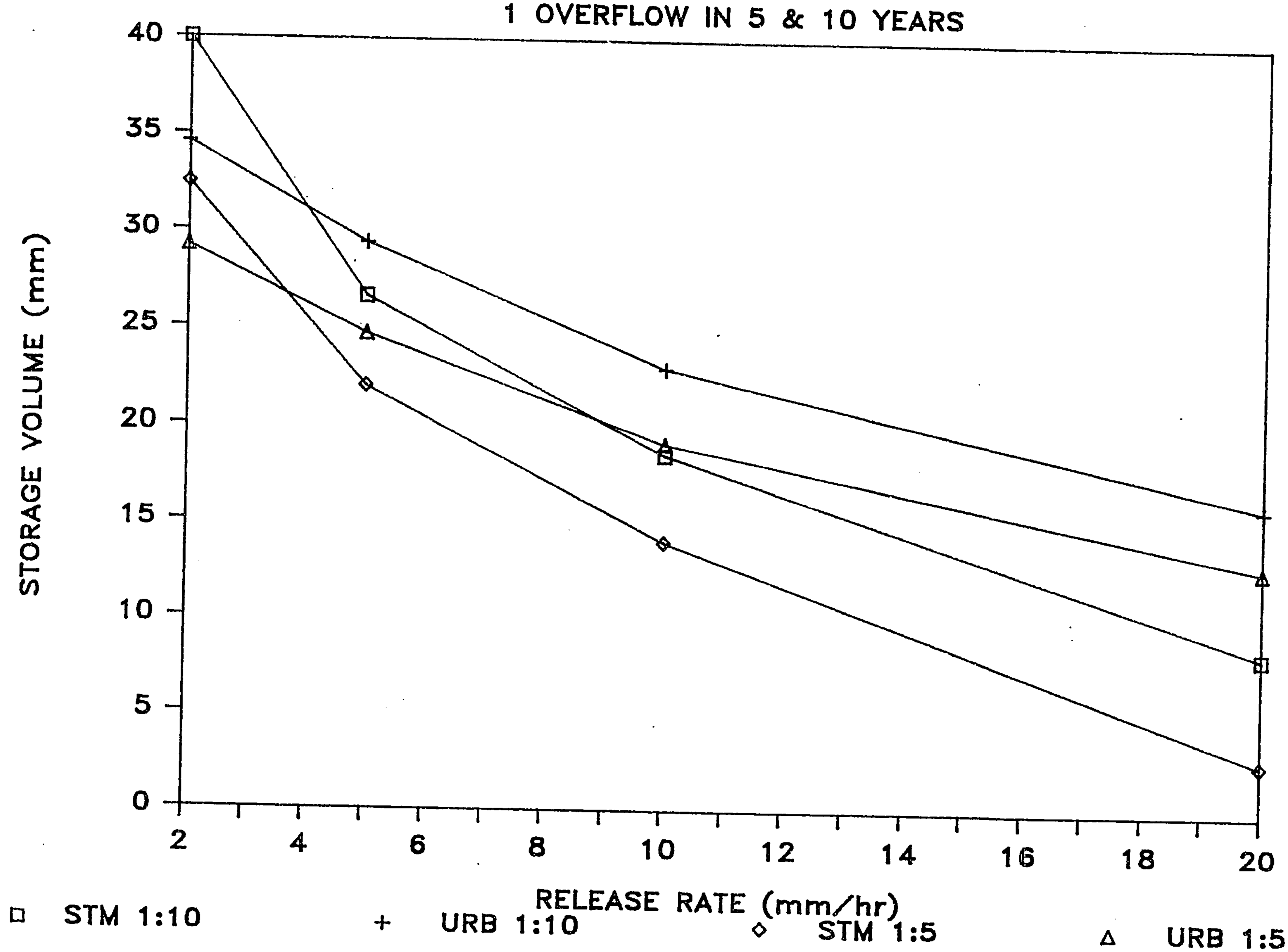


FIG. 8A — TORONTO 20% IMP.

1 OVERFLOW IN 5 & 10 YEARS

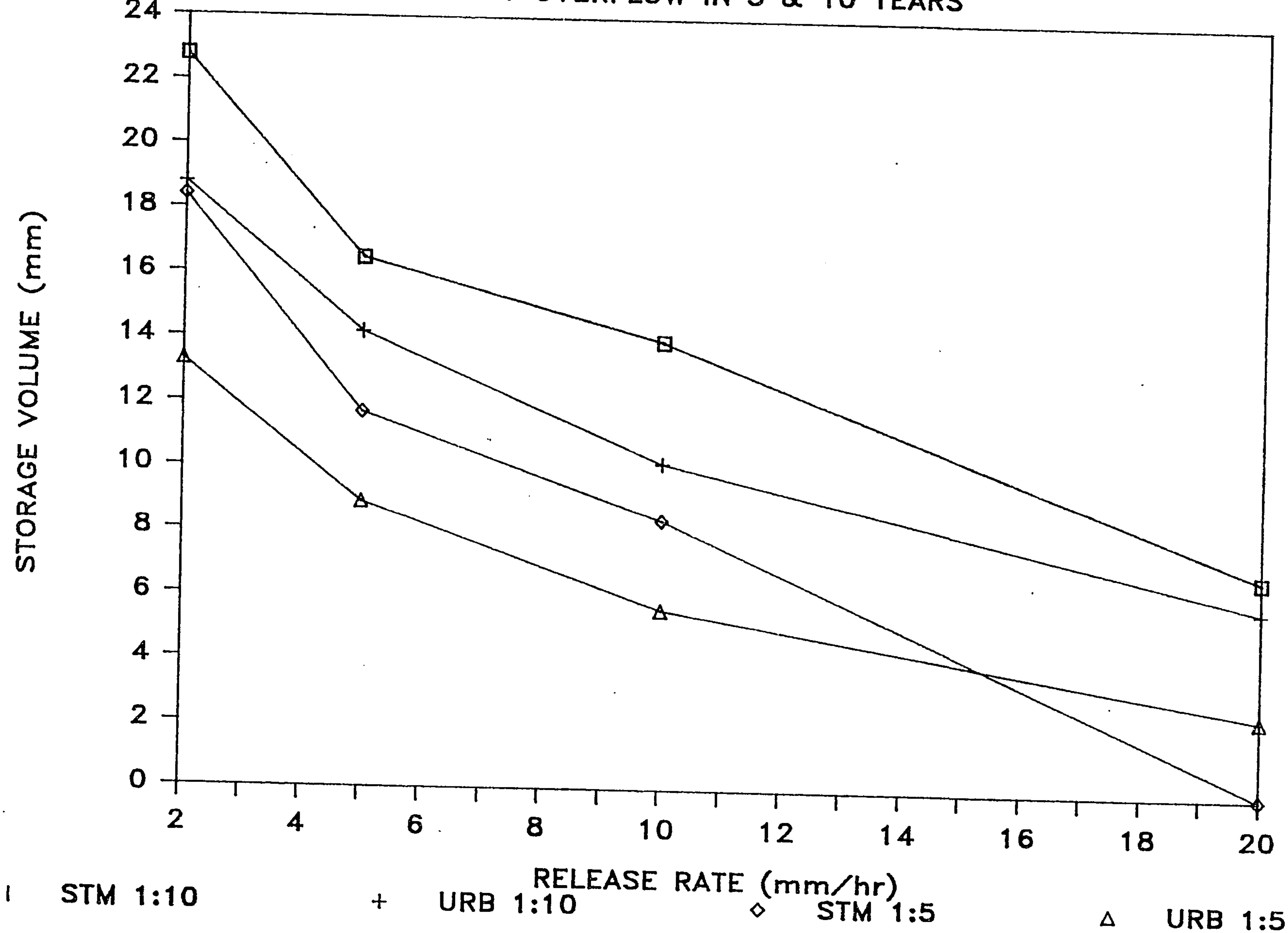


FIG. 8B — TORONTO 40% IMP.

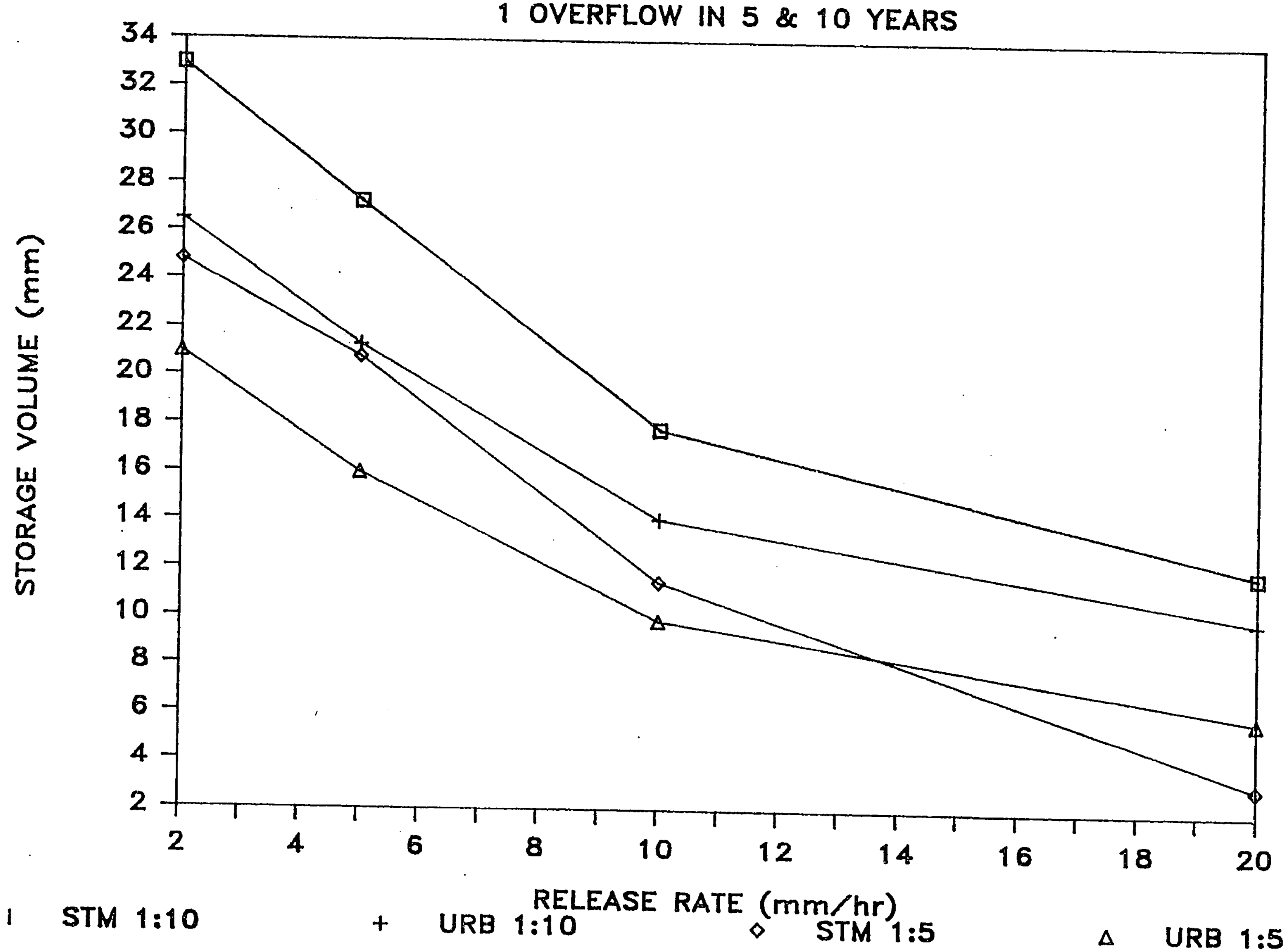


FIG. 8C — TORONTO 60% IMP.

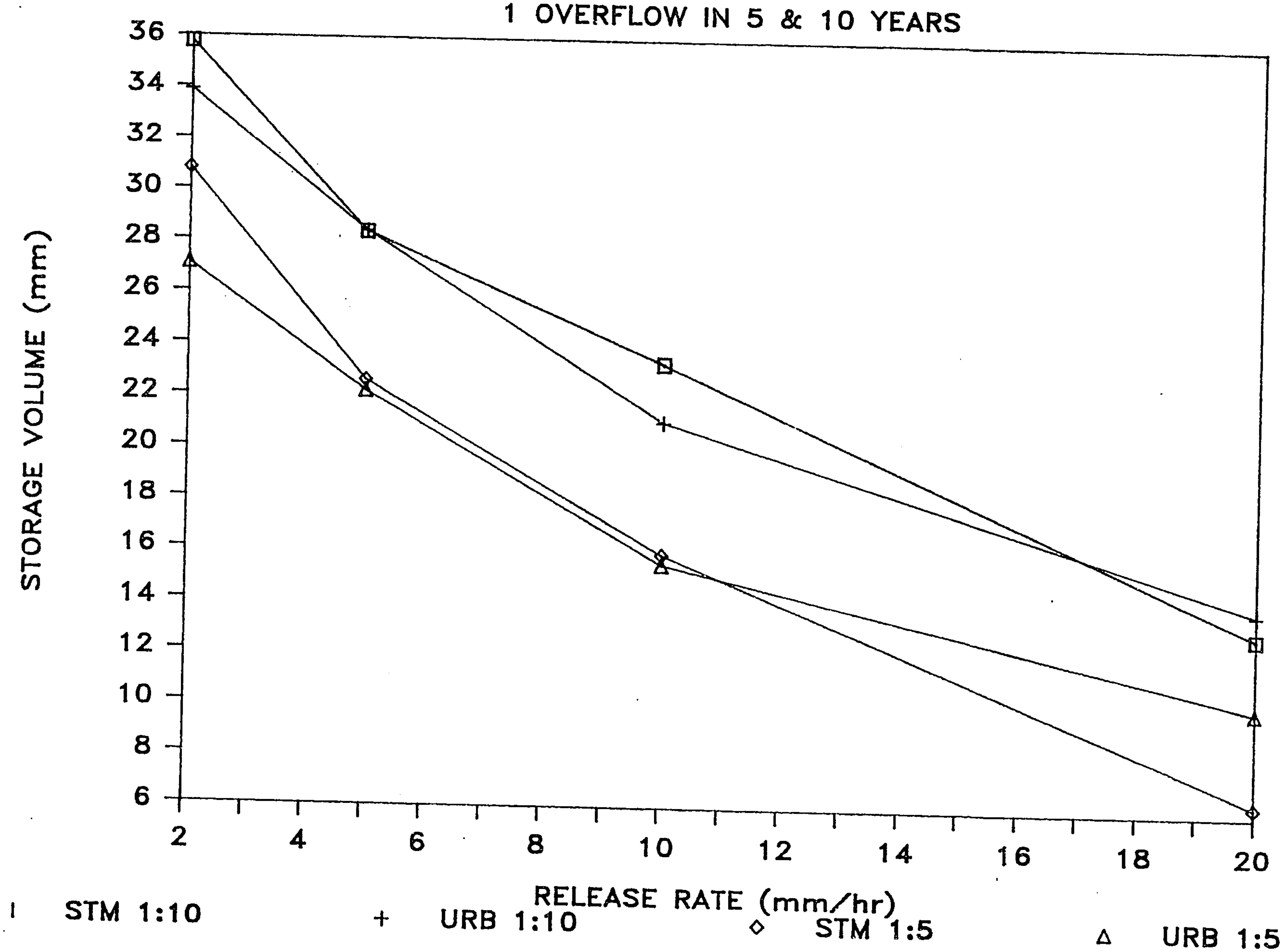
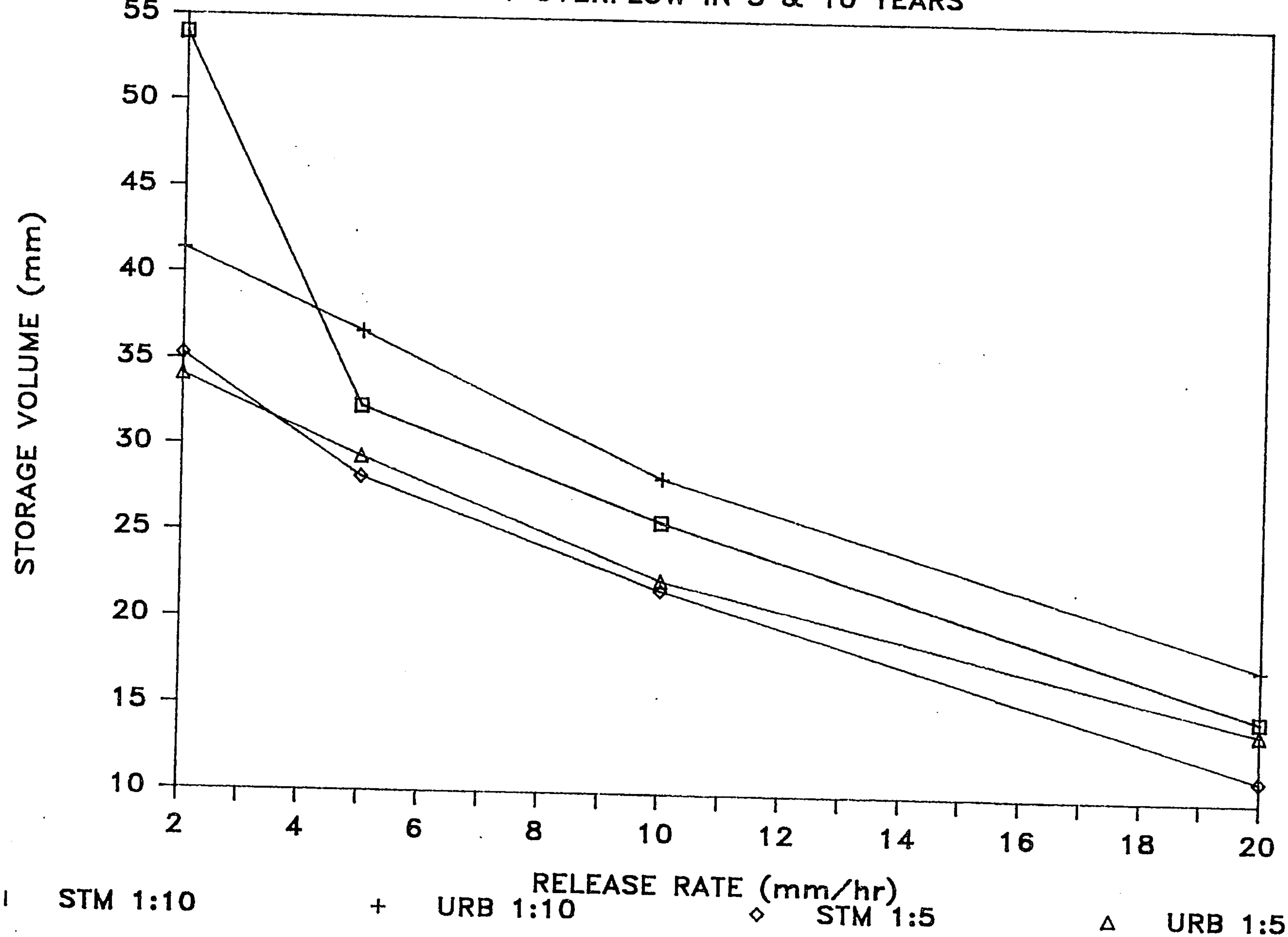


FIG. 8D — TORONTO 80% IMP.

1 OVERFLOW IN 5 & 10 YEARS



40 l/s/ha and are greatly dependent on the watershed imperviousness.

The second situation generally occurs for higher release rates. Under these conditions, event simulations formulate storage volumes larger than continuous simulations would indicate are necessary. Here, event modeling produces safe, conservative designs.

For the third situation, the detention facility sized by single event modeling will, according to continuous simulation, overflow more often than the return period for which it was designed. In fact, in the majority of cases, the storage volume dictated by continuous simulation is greater than the total volumetric runoff under the design hydrograph. This region of underestimation by event modeling usually occurs for lower outflow rates and is especially pronounced at lower impervious ratios. Under these circumstances, continuous modeling techniques should be implemented to ensure that OSD facilities are designed with sufficient capacity.

7.3 Discussion

These findings agree with those of Verworn (1984), cited above. Recall that he found no simple relationship between the return period of the design storm and the actual frequency of overflows and, therefore, recommended that new design procedures be developed.

Verworn's explanations apply here also. Detention facilities with low release rates will require longer emptying times, hence incurring a greater probability of a large successive rainfall finding the facility full or partly filled and thus resulting in an overflow. Conversely, higher outflow rates require less time to empty and induce a smaller risk of successive events causing overflows. Antecedent detention water level considerations are foreign to all notions of the design storm approach.

By similar reasoning, it is possible to explain the observed higher frequency of storage underestimation and the larger magnitude of discrepancies by event models at lower impervious ratios. Antecedent soil moisture conditions play a far more important role in watersheds with higher proportions of pervious area. A given rainfall volume will produce drastically different effects depending on whether the soil is saturated or not. It follows that event modeling accuracy increases for zones of higher imperviousness. Pervious area simulation generally requires more parameters to model, is less understood than impervious area simulation and is much more dependent on antecedent conditions. For low impervious ratios, just as for low release rates, successive events, which cannot be accounted for by event modeling, significantly influence the outcome.

Other observed trends can also be explained by the above rational. For example, intersection points - points of equal storage - generally decrease with increasing imperviousness, likely as a result of decreasing accuracy in single event simulation runoff volume for lower impervious ratios.

A few curves do not follow the general trends and may be categorized as curves which have more than one point of intersection or curves which have no intersections in the plotted range. These two deviant situations are likely the result of interpolation errors in Figures 4 to 5; the consequence of using a small number points used to draw the iso-release rate curves. The anomalies, arising only in the Toronto results, may also indicate that the simulation period was too short.

Boyd (1982), cited above, found that approximating the outflow hydrograph by a *constant release rate* underestimated the required volume by some 50% for triangular inflow hydrographs. To re-iterate, the constant release rate assumption proved to be the worst of the three methods examined, whereas an increasing linear outflow gave the best results.

Boyd's findings not only agree with this study, they increase the significance of the volume differences noted between single event and continuous modeling. Since STORM assumes a constant outflow rate in its simulations and since single event storage volumes were computed assuming the increasing linear release rate, any errors incurred by the use of these methods serve only to strengthen and intensify the findings of this study. In other words, if single event volumes were accurate but STORM volumes were low, then for low release rate situations, where event modeling already underestimates, the discrepancy between the two methods would be even more significant.

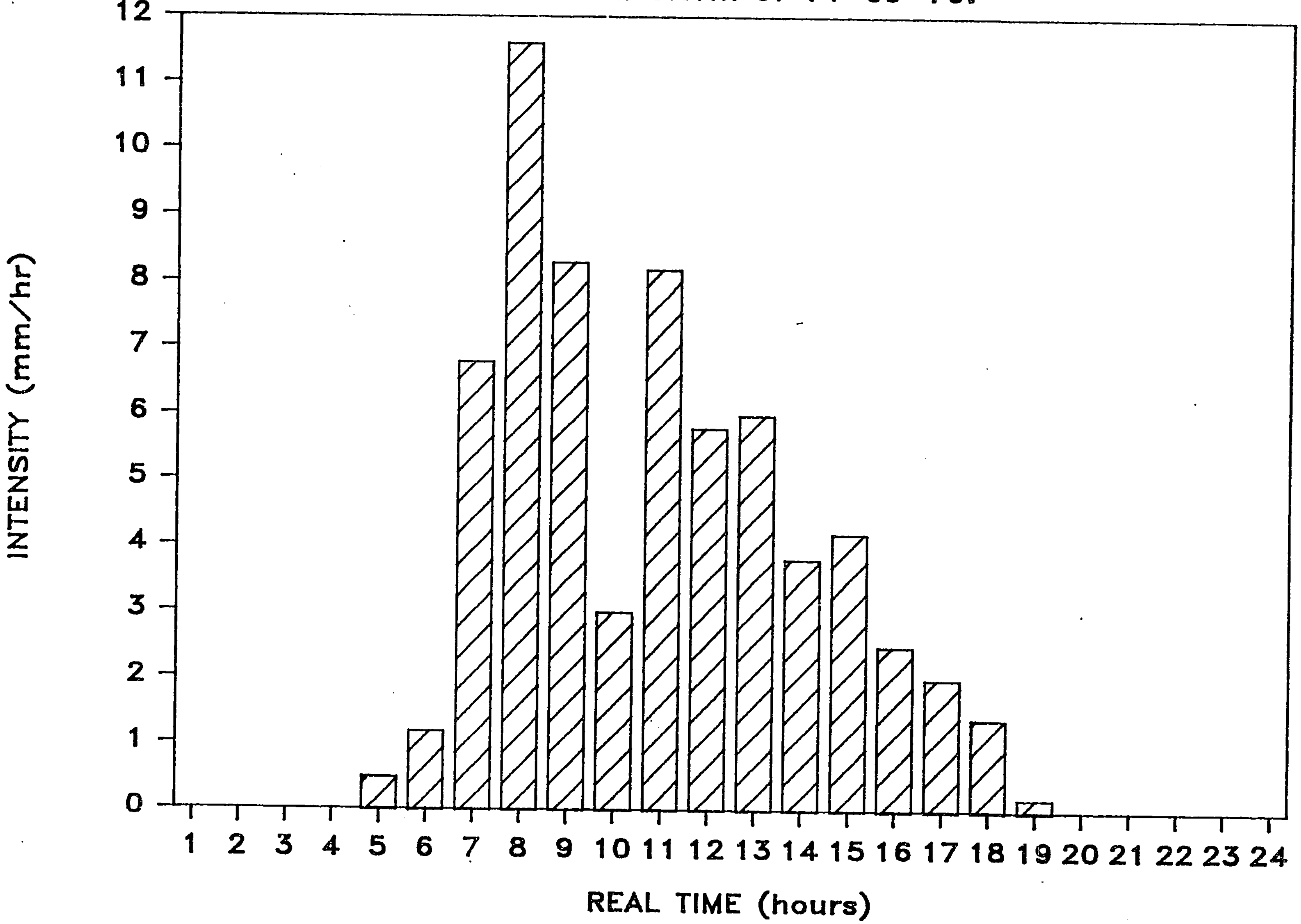
7.4 Overflow Event Analysis

Inspection of the durations of STORM events, both overflow and non-overflow events, reveals much about the types of storms that cause overflows and also provides further insight into the differences in single event and continuous simulation results. Three main types of storms have been categorized as causing overflows.

Firstly, storms with a lower intensities and longer durations than design storms can cause overflows (Figure 9). As noted above, overflow events in STORM tended to be longer - 6.8 hour average for Toronto - than design storm durations

FIG. 9 — LOW INTENSITY EVENT

Montreal Storm of 14-09-79.

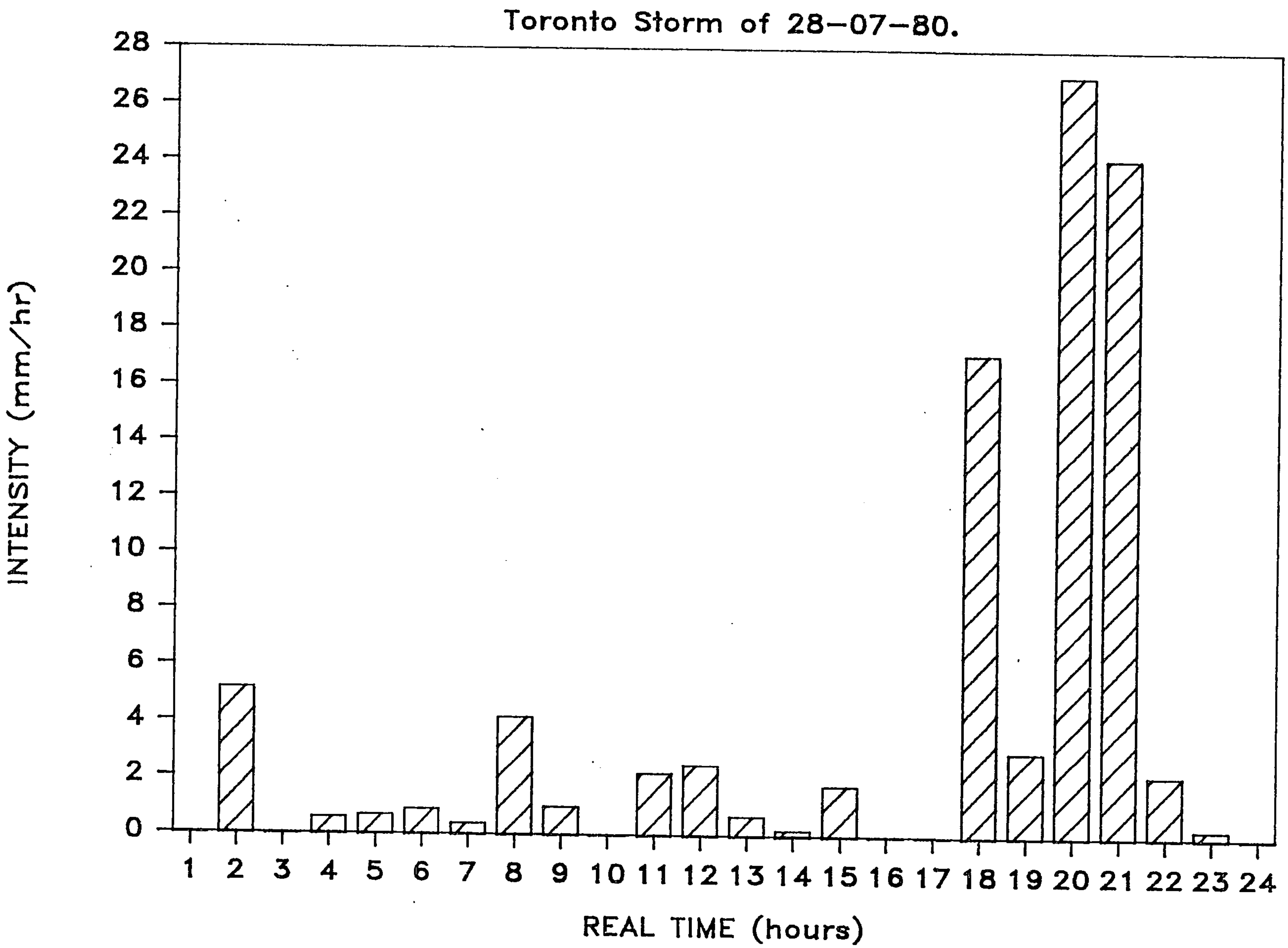


used in many single event studies. Assuming the IDF curves are accurate, a longer storm would necessarily mean a lower intensity than the 3-hour Chicago design storms used in the present research.

In the second category, overflows can be caused by a series of events in close succession (Figure 10). In this situation, a detention facility with low release rates will not be emptied between events and accumulation of runoff water will eventually cause overflows. As observed and noted above, overflow events tend to have longer intermittent dry periods than non-overflow events, especially for OSD's with low release rates. This strongly implies that the critical rainfalls are often a series of events characterized dry periods between successive rainfalls. For lower release rates storage empties less quickly and the probability of more rain falling before storage is empty is higher.

The last category, short duration, high intensity storms similar to those generally used in single event simulation, can also cause overflows. STORM output indicated that overflow events tend to be larger than non-overflow events, as expected. Even though on average critical storms were longer than 6 hours, in some cases these storms were in the order of 3 hours.

FIG. 10 — SERIES OF EVENTS



Snowmelt events, which were not considered in this study, can also cause OSD facilities to overflow. Snowmelts tend to be long, low intensity events occurring during a specific season of the year. To model effects of snowmelt in Canadian cities, it is advisable to assume that snow has been removed from a percentage of the catchment.

8. CONCLUSIONS AND RECOMMENDATIONS

During the course of this investigation, the following conclusions were reached:

1. For storage analysis, it is impossible to define a design storm which is independent of the catchment and the detention system to which it will be applied.

2. Detention facilities designed for a specific storm return period generally do not overflow at that design frequency in actual practice.

3. At low release rates, 10 to 40 l/s/ha, the single event design approach may significantly underestimate the required storage capacity. For larger release rates, single event design may be conservative.

4. Real storms which cause actual overflows can be long, low intensity events, short high intensity events or a series of storms in close succession.

The following recommendations encompass design and future studies:

Snowmelt events, an aspect not considered in this investigation, should be studied to assess their impacts on storage volume estimation. Designers using the methodology presented herein should also consider snowmelt effects where applicable.

In conducting similar studies, a minimum of 25 years of continuous precipitation data should be used to generate meaningful iso-release rate curves. The number of overflows occurring in a given simulation period is a discrete integer. The chances of having curves with discontinuities increases as the modeling period decreases. Discontinuities may pose interpretation and analysis problems. In light of these potential problems, twenty-five years is recommended as the shortest period for this type of simulation.

Municipalities which accept release rates in the order of 10 to 40 l/s/ha (3.5 to 14 mm/hr) for OSD facilities, should include continuous simulation in the design process. From this viewpoint, many existing OSD projects may be under designed. It is still possible to use single event design methods confidently, provided the release rate is relatively high, say above 40 l/s/ha. For lower release rates, a study, similar to the one described herein, carried out once for a particular location yields results which may be applied generally for many OSD projects in that region.

For similar reasons, continuous simulation is required for many studies involving runoff quality control. Reviews show that single event modeling is still used in some studies. As recommended by Linsley and Crawford (1974), pollution studies should also consider small, frequent rainfalls which wash off streets and flush out pipes, contributing large loadings to receiving water bodies. Continuous modeling helps to overcome difficulties in the assessment of runoff volumes for these smaller events which usually cause problems for event models.

REFERENCES

Aitken, A.P. and Goyen, A.G., "Simplifications in Stormwater Detention Design", Proceedings of the Conference on Stormwater Detention Facilities - Planning, Design, Operation and Maintenance, ASCE, New York, 1982, pp 176-185.

American Public Works Association, "Urban Stormwater Management - Special Report No. 49", American Public Works Association, Chicago, 1981.

Boyd, M.J., "Preliminary Design Procedures for Detention Basins", Urban Stormwater Quality, Management and Planning, Proceedings of the Second International Conference on Urban Storm Drainage held at Urbana, Illinois, 1981, Water Resources Publications, Littleton, Colorado, 1982, pp 370-378.

DeGroot, W., Editor, "Research Needs - Quality Issues", Proceedings of the Conference on Stormwater Detention Facilities - Planning, Design, Operation and Maintenance, ASCE, New York, 1982, pp 368-373.

Duru, O.J., "On-Site Detention: A Storm Water Management or Mismanagement Technique?", 1981 International Symposium on Urban Hydraulics, Hydrology and Sediment Control, Lexington, Kentucky, July 27-30, 1981, pp 297-302.

Fok, A.T.K., Vatagodakumbura, S., Rowney, A.C., "Some Considerations of Detention Lake Design", 1981 International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, Kentucky, 1981, pp 323-331.

Hall, M.J., Prus-Chacinski, T.M., Riddell, K.J., "Some Aspects of the Design of Stormwater Balancing Ponds for Catchment Areas Subject to Urbanization", Urban Storm Drainage, Proceedings of the International Conference held at the University of Southampton, London, 1978, pp 421-433.

Houston, F.C., "Simplified Sizing of Small Urban Detention Ponds", 1983 International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, Kentucky, July 25-28, 1983.

Huber, C.W., Heany, J.P., Nix, S.T., "Storm Water Management Model (SWMM) Version 3.", Department of Environmental Engineering Sciences, University of Florida, Florida, 1980.

Hydrologic Engineering Center, US Army Corps of Engineers, "Storage, Treatment, Overflow, Runoff Model (STORM) Users Manual, Version 2.0", Davis, California, 1976.

James, W. and Robinson, M., "Continuous Models Essential for Detention Design", Proceedings of the Conference on Stormwater Detention Facilities - Planning, Design, Operation and Maintenance, ASCE, New York, 1982, pp 163-175.

Johansen, L. and Harremoes, P., "The Use of Historical Storms for Urban Drainage Design", 1979 International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky, 1979, pp 61-70.

Lin, A.C., "A Simplified Method for Sizing Storm Runoff Retention Units", Proceedings of the Symposium on Surface on Surface Water Impoundments, ASCE, New York, 1981, pp 254-262.

Linsley, R. and Crawford, N., "Continuous Simulation Models in Hydrology", Geophysical Research Letters, Vol. 1, No. 1, 1974, pp 59-62.

MacLaren, J.F. Ltd., "Review of Canadian Storm Sewer Design Practice and Comparison of Urban Hydrology Models", for Canada-U.S. Agreement Task 3/4 Project 4; Canada-Ontario Agreement Project No. 74-8-31, Research Report No. 26; Ottawa, Ontario, 1975.

Malcom, H.R., "Some Detention Design Ideas", Proceedings of the Conference on Stormwater Detention Facilities - Planning, Design, Operation and Maintenance, ASCE, New York, 1982, pp 138-145.

Marsalek, J., "Synthesized and Historic Storms for Urban Drainage Design", Urban Storm Drainage, Proceedings of the International Conference held at the University of Southampton, London, 1978, pp 87-99.

Ormsbee, L.E., "Systematic Planning of Dual Purpose Detention Basins", 1984 International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, Kentucky, 1984, pp 143-151.

Padmanabhan, G., and Delleur, J.W., "An Alternative to the 'Design Storm Approach' for Urban Stormwater Management", 1982 International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, Lexington, Kentucky, 1982, pp 267-274.

Poertner, H.G. and Reindl, J., "United States Practices in Detention of Urban Runoff", Proceedings of the Symposium on Surface on Surface Water Impoundments, ASCE, New York, 1980, pp 945-954.

Raasch, G.E., "Urban Stormwater Detention Sizing Technique", 1979 International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky, 1979, pp 55-60.

Rossmiller, R.L., "Rational Formula Revisited", Proceedings of the Conference on Stormwater Detention Facilities - Planning, Design, Operation and Maintenance, ASCE, New York, 1982, pp 146-162.

Rowney, A.C., Wisner, P., "QUALHYMO An Alternative Continuous Quality Quantity Simulation Model", Proceedings of the Conference on Stormwater and Water Quality Management Modelling, Edited by W. James, Toronto, Canada, 1985.

Sieker, F., "Investigation of the Accuracy of the Postulate - Total Rainfall Frequency equals Flow Peak Frequency", Proceedings of the International Conference on Storm Drainage, University of Southampton, 1979, pp 31-41.

Soil Conservation Service, Engineering Division, "Technical Release 55. Urban Hydrology for Small Watersheds", Washington, D.C., 1975.

Storage, Treatment, Overflow, Runoff Model "STORM" - Quantity and Quality of Runoff User's Manual, Version 2.0, Hydrologic Engineering Center, U.S. Army Corps of Engineers, 1976.

Urbonas, B., "Reliability of Design Storms in Modelling", Proceedings of the 1979 International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky, 1979, pp 27-35.

Urbonas, B. and Glidden, M., "Development of Simplified Detention Sizing Relationships", Proceedings of the Conference on Stormwater Detention Facilities - Planning, Design, Operation and Maintenance, ASCE, New York, 1982, pp 186-195.

Verworn, H-R, "How Accurate are Simplified Detention Basin Design Procedures?", Proceedings of the Third International Conference on Urban Storm Drainage, Volume 1 - Analysis and Design of Stormwater Systems, Goteborg, Sweden, 1984, pp 1-9.

Walesh, S.G., "Summary Seminar on the Design Storm Concept", Proceedings of the SWMM User's Group Meeting, 1979, pp 47-54.

Walesh, S.G., Lau, D.H. and Liebman, M.D., "Statistically-Based Use of Event Models", 1979 International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Kentucky, 1979, pp 75-81.

Wenzel, H.G. and Voorhees, M.L., "Evaluation of the Design Storm Concept", Fall Meeting of A.G.U., San Francisco, 1978, 27 pp.

Woolridge, G., "Performance of Storage Tanks in Storm Drainage Systems", Urban Stormwater Quality, Management and Planning, Proceedings of the Second International Conference on Urban Storm Drainage held at Urbana, Illinois, 1981, Water Resources Publications, Littleton, Colorado, 1982, pp 350-359.

CONSIDERATIONS IN THE APPLICATION OF THE SCS METHOD FOR URBAN RUNOFF IN CANADA

by

Robert Henry Pankratz

A Report
presented to the School of Graduate Studies and Research
of the University of Ottawa
in partial fulfillment of the requirements
of the degree Master of Engineering

A B S T R A C T

One of the most popular urban runoff models in Canada and the United States is the Soil Conservation Service (SCS) Curve Number method. However, some limitations of this procedure may restrict its application under Canadian conditions. Although in existence for some 30 years, the method has not been subjected to direct verification by comparison with observed rainfall-runoff measurements from monitored urban catchments. A recent collection of published rainfall-runoff data for urban watersheds worldwide has made such testing possible.

SCS procedures are very structured and well defined. The unit hydrograph used in SCS procedures is actually only a specific case of the more general Nash unit hydrograph and can, in this respect, be calibrated. Tests were conducted on six urban catchments selected to represent both North American and European conditions. A similar investigation was carried out for a quasi-linear parallel reservoir model called URBHYD. In all cases observed runoff volumes were matched prior to calibration.

Hydrograph shapes were only marginally improved by the calibration of Nash unit hydrograph parameters whereas URBHYD simulations represented relevant and consistent improvement. The SCS procedure, which relies on a single composite curve number to generate runoff, had particular difficulty simulating the fast early responses typical of urbanized watersheds.

The curve number method provides a conservative design tool but should be used with caution for urban hydrograph simulation. Models which account for the unique responses of pervious and impervious areas, generally simulate real storm hydrographs better. The minimum storm size requirement specified in SCS guidelines precludes its use for many water quality and detention pond studies in Canada where smaller storms are generally used for design.

Table of Contents

List of Tables

List of Figures

1. INTRODUCTION

In the past, a major limitation in the development of urban hydrology was the lack of reliable and accessible measurements for the testing and verification of models. At the International Symposium on Comparison of Urban Drainage Models held at Dubrovnik, Yugoslavia in 1986, a collection of rainfall-runoff data for urban watersheds was assembled. The publication "Urban Drainage Catchments" - UDC (Maksimovic and Radojkovic, 1986) contains measured rainfalls with corresponding hydrographs for 20 experimental urban watersheds complete with detailed pipe system layouts and other relevant data. The data compiled at Dubrovnik provides an excellent opportunity for the study and testing of urban hydrologic models.

The Implementation of Storm Water Management (IMPSWM) program at the University of Ottawa, coordinated by Professor Paul Wisner, is extensively applying the UDC data for the study of various models. One of these models is the SCS unit hydrograph.

Runoff control studies in many states of the USA are conducted with the Soil Conservation Service (SCS) Technical Release 55, a simplified method based on the SCS rural unit hydrograph. This popular unit hydrograph is also incorporated in various user-friendly microcomputer models (eg. TR-20). Models such as SWMM are used in pollution control studies or relief sewer studies.

In Canada, however, no standard procedure exists. Initially, SCS methods were applied directly or in conjunction with the HYMO model. In 1982, an urban hydrology model, URBHYD (Wisner, 1982), was developed at the University of Ottawa. URBHYD is presently applied in many provinces in Canada although HYMO is still used in some jurisdictions. SCS applications in Canada include smaller, more frequent storms; right down to the 2-year return level. According to an SCS representative (Miller, 1987), application in the USA is restricted to large storms, a constraint emphasized in the 1986 revision of Technical Release 55 (SCS, 1986).

The popular SCS procedure which, although originally developed for rural hydrologic design, was later adapted and extended for application to small urban watersheds. A recent study by Water Resources Council indicated that the SCS procedure is among the most widely used hydrologic methods in North America (McCuen et. al., 1983) and is described in SCS publications (SCS, 1975, 1986) or textbooks (Viessman et al., 1977; APWA, 1981; McCuen, 1982). Despite its popularity, the design method has not been extensively compared with other models on the basis of observed rainfall-runoff data.

The Nash unit hydrograph was also chosen for study and as a comparison model since it represents a more general form of the SCS unit hydrograph. The objective of the study was to determine if a Nash unit hydrograph could be found which was superior to the SCS hydrograph for modeling urban watersheds. The rainfall-runoff transformation was not the object of investigation since in the comparison both models employed the SCS curve number (CN) relation with calibrated CN's.

As an alternative to the lumped CN unit hydrograph approach, a quasi-linear parallel reservoir model was tested. URBHYD, a submodel of the OTTHYMO program (Wisner, 1982), generates runoff from pervious and impervious areas separately. Since its development in 1982, URBHYD has undergone extensive testing on many watersheds in various parts of the world.

The study was not concerned with a comprehensive comparison of model structures in terms of input requirements, execution time, flexibility, applicability, etc. but rather concentrated on the comparison of performance for recorded UDC rainfall-runoff data and design storm input.

2. DESCRIPTION OF SCS PROCEDURES

2.1 Historical Development

The U.S. Soil Conservation Service developed a synthetic unit hydrograph in the early 1950's which, when combined with a simple rainfall loss function based on a "curve number" was initially used in the design of small dams for predominantly rural watersheds. The principles of this model were later incorporated in the SCS TR-20 computer model (SCS, 1970). More recently, the method was expanded for use in urban watersheds. The SCS Technical Release TR-55 (SCS, 1975) provides tables and charts for relatively simple hand computation of "design hydrographs" and peak discharges. TR-55 methodology was updated by SCS in 1986.

The use of a standard unit hydrograph from coast to coast, the simplicity of the model, the consistency of approach in more than three decades and the availability of curve number parameters and design storms explain the popularity of SCS procedures in the United States, especially in states where SCS is the leading storm water management agency.

2.2 Generating Runoff

The SCS procedure for computing runoff is known as the Runoff Curve Number (CN) method. Watershed runoff volume is considered to be a function primarily of the rainfall and associated losses due to infiltration characteristics such as soil type, soil moisture, antecedent moisture condition, cover type, impervious surfaces and depression storage. Infiltration responses of the watershed are represented by the initial abstraction and potential maximum soil retention capacity. Infiltration, rainfall and runoff are then related by the following expression:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad (1)$$

in which Q is the volume of runoff (mm),

P is the volume of rainfall (mm),

I_a is the initial abstraction (mm) and

S is the potential maximum retention capacity of the soil (mm).

The equation is valid only for $P > I_a$.

Initial abstraction represents evaporation, interception, depression storage and infiltration; in short, all losses before runoff can begin. Studies conducted on small agricultural watersheds yielded the following empirical relationship between initial abstraction and soil retention capacity:

$$I_a = 0.2\ S \tag{2}$$

Substituting this expression into equation (1) results in a relationship for runoff in terms of rainfall and soil retention capacity only. Retention capacity, S, expresses the infiltration properties of the soil and the land cover. The runoff curve number is defined in terms of the soil retention capacity by the following expression:

$$S = 1000/CN - 10, \quad \text{or} \quad S = 25400/CN - 254 \tag{3}$$

in imperial (inches) or metric units (mm), respectively.

Whereas retention capacity has no upper limits the curve number varies between 0 and 100 and is determined on the basis of the land use, land cover, antecedent moisture conditions and the hydrologic soil type. Tables describing typical rural and urban land use scenarios are available to aid in the selection of an appropriate CN. Area weighted curve numbers are averaged in the computation of a lumped CN representing the entire watershed.

2.2.1 Urban Adjustments

When the methodology is extended to urban watersheds, certain adjustments are required to maintain a lumped CN approach that represents both pervious and impervious areas. Adjustment factors, reflecting the influence of both connected and unconnected impervious areas, are recommended in the 1986 edition of TR-55 for estimating a "composite CN". While a distinction is made between the connected and unconnected impervious areas for watersheds where less than 30% of the total area is impervious, for larger imperviousness ratios no special correction is made.

2.2.2 Limitations of TR-55

TR-55 warns users about several limitations in the methodology. Firstly, curve numbers were developed to represent average watershed conditions for "design storm" conditions, modeling accuracy decreases if historical storms are used as input (SCS, 1986). Secondly, the curve number equation does not contain any expression for time to account for the intensity and duration of actual storm events and, therefore, must be used with caution in these applications. Another assumption, the relationship for initial abstraction in equation (2), was based on agricultural watershed data and may not apply for urban catchments. Lastly, the CN procedure is less accurate for total runoff less than 0.5 inches (12.5 mm).

This last limitation may prove a severe restriction. As an example, a 2-year return storm, as is commonly used in Canada for the design of detention facilities in a watershed with an imperviousness of about 30% is likely to have a runoff of less than the new accepted limit. In the prairie provinces a larger storm is necessary to meet this new restriction.

2.3 Overland Routing

The rainfall-runoff transformation in the SCS procedure is based on a linear synthetic unit hydrograph which, when convoluted with the rainfall excess distribution determined by equation (1), produces a design hydrograph. Originally developed for rural conditions, the SCS unit hydrograph was later borrowed for application to urban watersheds.

The time to peak of the unit hydrograph is conventionally taken as:

$$t_p = 0.67\ t_c \qquad (4)$$

where t_c is the time of concentration.

SCS provided two options for computing the time to peak of the unit hydrograph: the lag method and the velocity method.

2.3.1 The Lag Method

The Lag Method, developed for overland flow, assumes homogeneous conditions over the entire length of the flow path and expresses lag time as a function of slope, flow length and maximum soil retention capacity, S. The assumption is evidence that the method was originally intended for rural application.

To reflect the quicker responses observed in urbanized watersheds, adjustment factors were developed in terms of percentage imperviousness and the percentage of hydraulic length replaced by channels (McCuen and Miller, 1983). Although no mention of the lag method was made in the latest edition of TR-55 (1986), the method was retained for this study to assess applications made in the past decade.

2.3.2 The Velocity Method

The Velocity Method differentiates between the individual components of the runoff flow path such as overland, channel and pipe flow, and account for the travel time associated with each individual component as the ratio of hydraulic length to velocity. The outlet time of concentration is simply the sum of travel times for all components along the flow path. Intuitively, the velocity method appears to be the superior approach, as substantiated by McCuen (1984b) and others.

The 1986 edition of TR-55 presents a kinematic wave equation for travel time estimates in the upper reaches of a basin where sheet flow may be expected. The method is limited to a flow path less than 300 feet (90 m), after which sheet flow is said to become "shallow concentrated flow" and velocities can be estimated from slope and roughness.

It was intended that TR-55 would be applied with a design storm of magnitude and type defined by the return period and location. The complete methodology together with a simple routing method are incorporated in the SCS TR-20 computer program (SCS, 1973).

2.4 Description of the Nash Unit Hydrograph Model

The linear cascade of reservoirs, as proposed by Nash (1957), is another popular unit hydrograph model initially applied to rural watersheds. Outflow from a series of N reservoirs is:

$$Q_N = \frac{(t/t_p)^{N-1} \exp\{-t/t_p\}}{t_p\,\Gamma(N)} \tag{5}$$

in which N is the number of linear reservoirs in cascade.
t_p is the time to peak of the unit hydrograph, and
$\Gamma(N)$ is the gamma function of N.

The parameters, N and t_p, uniquely define the shape of the unit hydrograph making a whole family of shapes possible. If N=4.75 the Nash unit hydrograph very closely resembles the SCS unit hydrograph. In other words, the SCS unit hydrograph is a special case of the Nash model.

3. PREVIOUS STUDIES

3.1 Previous SCS Studies

3.1.1 Initial Abstraction

Studies conducted by IMPSWM (Wisner, 1986) in 1981 for the Seymaz watershed in Switzerland found that the initial abstraction was much less than 0.2S. Morel-Seytoux et. al. (1982) reported similar findings and pointed out that initial abstraction has been shown to vary with rainfall intensity and, as such, can not be a constant value.

3.1.2 Application with Historic Storms

Prior to SCS self-imposed limits, Kent (1966) stated that the procedures "are primarily for establishing safe limits in design, and for comparing the effectiveness of alternate systems of measures within a watershed project. They are not used to recreate specific features of an actual storm." Problems arise because SCS methods are commonly used to simulate historic events - a practice which has never been adequately tested. Some of the most widely applied computer models - HYMO and TR-20 - use the CN procedure. Tschantz (1985) has even developed a microcomputer program which allows the user to enter a recorded historical storm and apply SCS procedures to simulate runoff hydrographs for urban watersheds.

There is a need for general methods which can apply both design and historic storms. Marsalek (1978), Johansen et. al. (1979), Urbonas (1979), James et. al. (1982) and Padmanabhan et. al. (1982) all recommended the general use of observed events over design storms for a number of reasons. The presupposition that the return period of the design storm equals the return period of the runoff peak and volume is weak and was most frequently and vigorously challenged by different researchers.

3.1.3 Peak Discharge

Simulation results with observed rainfall-runoff data are not available in any of the SCS publications. On the other hand, peak discharges obtained with SCS methods for design storms of various frequencies were compared with a statistical distribution of gauged flows (from the same watershed).

A previous paper by McCuen (1983) presented the results of a comparison of various peak discharge methods including SCS (McCuen, 1983). SCS results showed a greater scatter (less accuracy) than the results from either the rational or USGS methods but also accounted for the lowest bias.

In still another study, McCuen et al. (1984a) investigated the accuracy of the urbanization factors introduced in TR-55 (1975) to adapt the previously rural SCS procedure to urban conditions. SCS urban peak flow estimates for various return periods were compared with peak discharge rates generated by a Log-Pearson Type III distribution for annual maximum flood series data on 51 urban watersheds. Results indicated that, with minor modification, the urbanization adjustment factors are acceptable for estimating peak discharge rates for urban catchments with areas less than 1600 ha.

Previous studies have tested various components of the urban SCS procedures and have investigated the accuracy of simulated peak flows by statistical means. However, no study has yet undertaken the comparison of the SCS method with other urban runoff models for observed rainfall-runoff data.

3.1.4 Time to Peak

Studies by Espey, Huston and Associates (1979), and McCuen et al. (1984a, 1984b) have reviewed SCS procedures considering the CN selection process, calculation of the time of concentration and validity of urbanization adjustment factors for estimating peak runoff rates. While these studies pointed out limitations in the curve number selection process (Espey et. al., 1979) or limitations in time of concentration formulas (McCuen, 1984b), they did not evaluate the shape of the SCS unit hydrograph nor did they compare results with other models.

Espey et al. found that for over half of the watersheds studied, the curve numbers optimized by HEC-1 were greater than the corresponding values given by SCS tables. Also, lag times, computed by the SCS lag method, appeared high in comparison with times given by other methods.

McCuen et al. (1984b) tested both the lag method and the overland flow component of the velocity method with several other techniques for estimating time of concentration. The investigation was conducted with data from 48 urban watersheds obtained from a U.S. Geological Survey data base. The lag method and the overland flow component of the velocity method gave times in the same order of magnitude. For nonhomogeneous land uses where a combination of overland, channel and pipe flow is possible, it was found that a velocity based method has the potential to give the most accurate results. The SCS velocity method falls into this category of methods.

3.2 Previous Nash Model Studies

Bielawski (1984) compared the Zoch single linear reservoir model with the Nash model on a 300 ha urban catchment situated in Warsaw, Poland. Using summertime precipitation and flow data he concluded that linear reservoirs were satisfactory for the Warsaw catchment and that the Nash model gave better results than the Zoch model. He found that N was, in general, lower than 4.75.

4. DESCRIPTION OF THE WATERSHEDS STUDIED

After screening the UDC watershed data, six catchments - three from North America and three from Europe - were selected for the study. Each catchment catalogued in UDC included six recorded rainfall-runoff events. In addition to this, three extra storms and hydrographs from the Malvern basin, brought the total number of events up to 39.

Catchments were chosen to represent a wide range of climatic and geographic conditions thereby giving the results more general credibility. Watershed characteristics and locations and rainfall data are presented in Table 1; detailed descriptions and sewer system layouts are provided in UDC. Large ranges in catchment area, imperviousness ratios, rainfall volumes and observed runoff coefficients are immediately evident. Total rainfalls range from 3 to 60 mm with the larger volumes generally occurring in North America.

TABLE 1 - CHARACTERISTICS OF STUDY CATCHMENTS

CATCHMENT LOCATION	AREA (Ha)	IMPERV (%)	AVG SLOPE (%)	NO. OF EVENTS	RANGE OF TOTAL RAINFALL (mm)	RANGE OF RUNOFF COEFF. (%)
MALVERN, BURLINGTON	23.30	34.	2.0	9	3.8-15.2	28.0-37.5
KING'S CREEK, FLORIDA	5.97	49.	0.7	6	10.8-57.1	37.6-80.4
GRAY HAVEN, BALTIMORE	9.40	52.	1.0	6	30.2-55.9	39.9-72.8
ST. DENIS, PARIS	255.00	33.	0.9	6	4.3-14.7	13.8-19.7
MUNKERISPARKEN, DENMARK	6.44	33.	1.0	6	2.7-14.5	27.5-35.6
MISKOLC, HUNGARY	25.34	16.	5.0	6	4.3-26.0	14.4-41.3
RANGE/TOTAL	6-255	16-52	0.7-5	39	2.7-57.1	13.8-80.4

5. STUDY METHODOLOGY

5.1 Evaluation Criteria

McCuen (1983) defines the following criteria for model selection:

1. Conceptually rational,
2. Flexible in application to various design problems,
3. Widely applicable geographically,
4. Requiring minimal and easily obtainable inputs,
5. Consistent results from different modeler and
6. Conceptually simple to apply.

Although the above criteria are significant, relevant and generally tend to describe both the SCS and Nash models, a more objective measure must be defined for distinguishing between the strengths and weaknesses of each model.

Singh (1979) addresses the questions of what a good model is and how to choose between them. He emphasizes the need for objective criteria in judging goodness of fit. A following form is suggested:

$$E = a(\text{error in peak flow}) + b(\text{error in volume}) + c(\text{error in time to peak}) + d(\text{error in shape}) \quad (6)$$

where E is the objective error function and

a,b,c,d are positive weighting coefficients summing to unity.

All errors represent deviations from the observed hydrograph and are summations over all studied events. Errors in any of the four measurements above can be expressed as a standard error, for example:

$$\text{error in } Q_p = \sqrt{\frac{1}{n-1-p} \sum \frac{(Qpobs-Qps)^2}{Q_{pobs}}} \qquad (7)$$

where Q_{pobs} is the observed peak flow for each event,

Q_{ps} is the simulated peak flow for each event,

n is the total number of events for a watershed,

p is the number of free model parameters and

$n\text{-}1\text{-}p$ represents the total degrees of freedom.

The standard error is thus adjusted for the complexity of the model, reflected in the number of parameters that can be calibrated. It is expected that a more complex model will out-perform a simpler one.

Errors in peak flow and the time to peak were the criteria used to compare the performance of both models. Previous experience with relations of the form of equation 7 show that standard errors of this type may suffice for a first model comparison. Visual appearance can also be used as a subjective means of comparing simulated hydrograph shapes.

5.2 Model Study Procedures

Two model testing strategies are recognized. The most frequently used strategy is to simulate measured runoff hydrographs with real event input. SWMM (1971), ILLUDAS (1974) and OTTHYMO (1982) etc. have all been tested using this strategy and their user manuals illustrate comparisons of modeled and observed hydrographs. The SCS TR-20 computer program can also be used with historical storms and is valid for application to urban catchments provided the proper adjustment factors are applied to CN and time of concentration values. In order to test the basic assumptions behind any method, it must eventually be compared with measurements.

The second model testing strategy has already been used by McCuen et al. (1984); that is to compare peak flows generated for design storms of various return periods with peaks estimated statistically from recorded stream gauge data.

5.3 Verification of SCS Method

Due to a lack of previous work in the former direction, a study of SCS procedures based indirectly on the simulation strategy was undertaken. The OTTHYMO model (Wisner, P'ng, 1982) was used for simulation purposes since it includes the Nash model with the following input parameters: N, t_p, CN and I_a. The advantage of using OTTHYMO being that both SCS and Nash unit hydrographs may be simulated and each step of the procedure - rainfall, runoff and routing - can be analyzed separately. In calibration, CN and I_a determine the runoff volume, whereas N and t_p are strictly unit hydrograph shape parameters. This investigation, being more concerned with the shape of the unit hydrograph, examined the improvements obtained in shape by the "optimization" of N and t_p. By setting CN and I_a to match the runoff volume, the study will focus on hydrograph shapes and peak flows only. This also enables rainfall events smaller than the 12.5 mm limit to be included.

A microcomputer version, MINI-OTTHYMO, (Wisner, Consuegra, Sabourin, 1986) capable of plotting observed and simulated hydrographs on the monitor, was a useful aid for the trail and error calibration process.

Testing of the SCS unit hydrograph model was conducted in the following manner:

Curve numbers were calculated using equation (1) in order to ensure that the observed runoff coefficient was matched and, therefore, the runoff volume would be correct. This also served to isolate the performance of the unit hydrograph from the rainfall-runoff transformation. Initial abstractions were determined from preliminary calibration results and were held constant for each catchment even though CN's varied from event to event.

The number of linear reservoirs, N, was held constant at 4.75 for all events since this defined the unique SCS unit hydrographs shape. The time to peak of the unit hydrograph for each catchment was estimated by using both the SCS lag and velocity methods.

Computer simulations were conducted for all 39 events and resulting hydrographs were compared with observed runoff hydrographs.

SCS type II design storms for Metro Toronto rainfall depths were then applied to three of the catchments using the parameters obtained above.

5.4 Calibration of the Nash Model

The same 39 rainfall events were also used to calibrate the Nash model as summarized in the following strategy:

Curve numbers and initial abstractions found to match observed runoff volumes for SCS simulations were retained for the Nash unit hydrograph simulations.

Hydrographs were calibrated to temporally match the observed peak flow for each event by varying t_p, the unit hydrograph time to peak.

The number of linear reservoirs, N, was calibrated in an attempt to match observed peak flows.

A compromise was affected between calibrated values of N and t_p.

Calibrated values of N and t_p were applied with Toronto 5 and 100-year SCS type II storms for three catchments.

5.5 Comparison with the URBHYD Model

It is expedient to compare the results obtained with the SCS and Nash models, both of which were originally intended for rural watersheds, with a model developed specifically for urban conditions. URBHYD, a quasi-linear model simulating runoff for pervious and impervious areas separately (Table 2), was selected to run all of the events. URBHYD has been extensively tested and found to be consistent with the EPA-SWMM model (Consuegra and Wisner, 1986).

To be consistent, URBHYD parameters were set to match the observed runoff volumes as was done in the previous simulations. This was accomplished by assuming a value for depression storage and calibrating the imperviousness ratio so as to obtain the measured runoff. Default values were assigned to all other parameters.

TABLE 2 - SCS MODEL PARAMETERS

CATCHMENT	AREA (Ha)	RANGE OF CN*	INITIAL ABSTN (mm)	LAG METHOD t_p(hr)	VELOCITY METHOD t_p(hr)
MALVERN, BURLINGTON	23.3	87-97	0.0	0.45	0.19
KING'S CREEK, FLORIDA	5.97	91-99	1.0	0.48	0.15
GRAY HAVEN, BALTIMORE	9.40	82-93	0.2	0.26	0.16
ST. DENIS, PARIS	255.0	79-91	1.0	1.50	0.33
MUNKERISPARKEN, DENMARK	6.44	90-98	0.2	0.33	0.12
MISKOLC, HUNGARY	25.34	81-95	0.2	0.21	0.09

* Computed to match observed runoffs by equations 1 and 3.

6. RESULTS

6.1 Results of SCS Verification

In order to accurately reproduce observed runoff volumes, it was necessary to significantly reduce the initial abstraction from 0.2S. Higher values of I_a did not account for the quick responses of observed hydrographs. Through preliminary calibration, an acceptable range was found to be 0 to 1.0 mm.

One curve number could not accurately replicate the large ranges of runoff coefficients observed in a some of the watersheds of this study, as illustrated in Figure 1 for the Malvern catchment in Canada.

The empirical lag method for estimating response time consistently and significantly overestimated the time to peak. Consequently, all further findings reported for the SCS model are based on the time to peak as computed from the time of concentration determined by the velocity method. All subsequent references to SCS results imply the use of the velocity method unless explicitly stated otherwise.

Hydrograph peaks simulated by the SCS unit hydrograph were, in general, later than observed peaks as evident in Figure 2

FIG. 1 — RUNOFF COEFFICIENT & CN

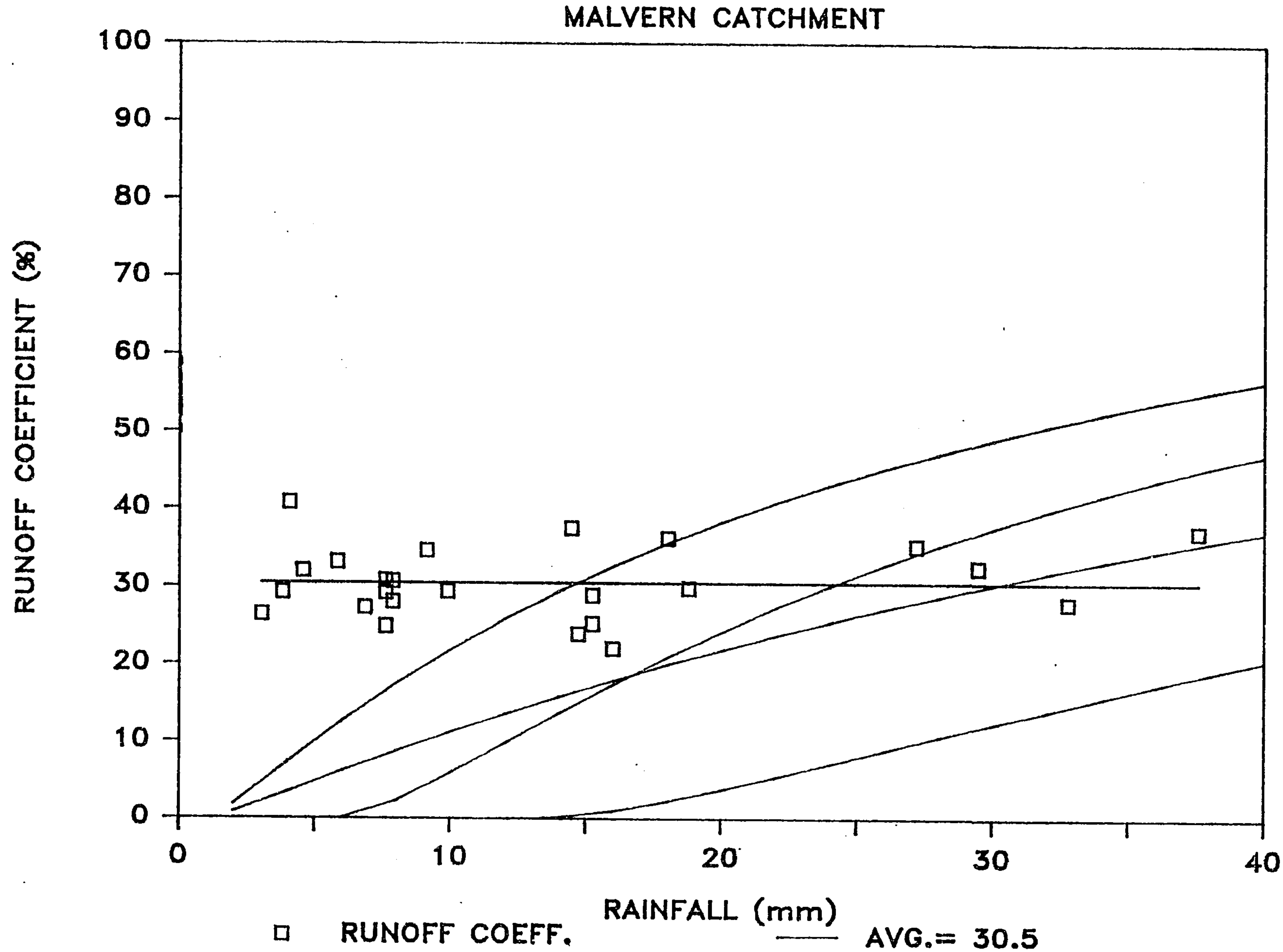
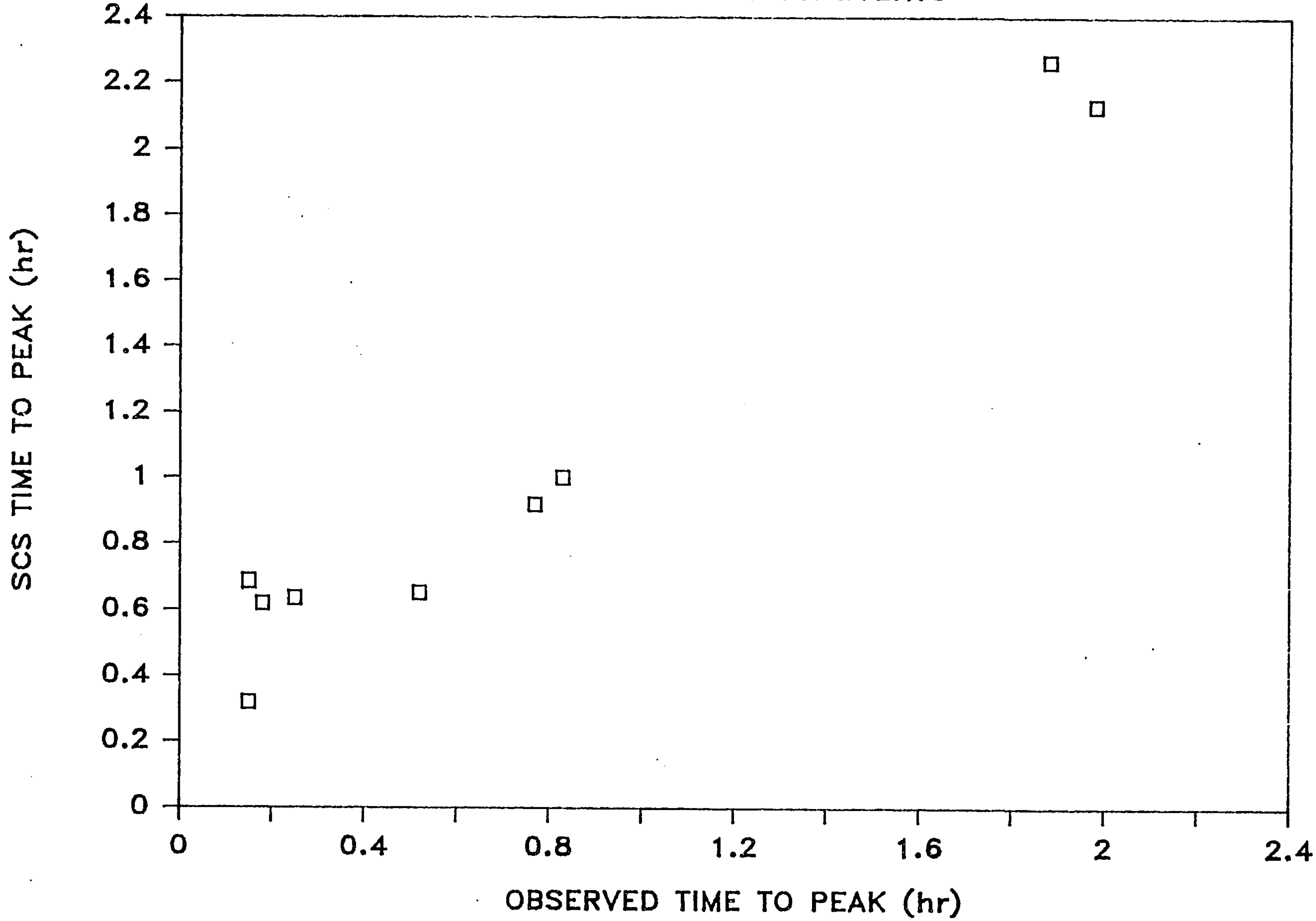


FIG. 2 — SCS TIME TO PEAK

FOR NINE MALVERN EVENTS



for the Malvern catchment. However, simulated hydrograph shapes were reasonable for about 45% of the events. Simulated hydrographs for all watersheds and all events may be found in Appendix A.

Ratios of simulated peak flows to observed peaks are shown in Figure 3 plotted against observed runoff volumes. Model performance differed greatly from one watershed to another confirming the supposition that validation of a model on a small number of watersheds may lead to biased conclusions.

In practical applications of SCS methodology, a watershed is assumed to be represented by a single curve number. Simulations conducted with average watershed curve numbers show a significant deterioration in performance over the results in Appendix A. For example, Figures 4A to 4F illustrate this deterioration for the French watershed when applying an average CN of 86.8. The use of a single lumped curve number is then a source of potential error. A single curve number can not replicate the range of observed runoff coefficients encountered in practice (see Figure 1).

6.2 Nash Model Results

A significant effort was made to improve the SCS unit hydrograph results by calibrating the shape parameters (N

FIG. 3 — PEAK FLOWS VS RUNOFF

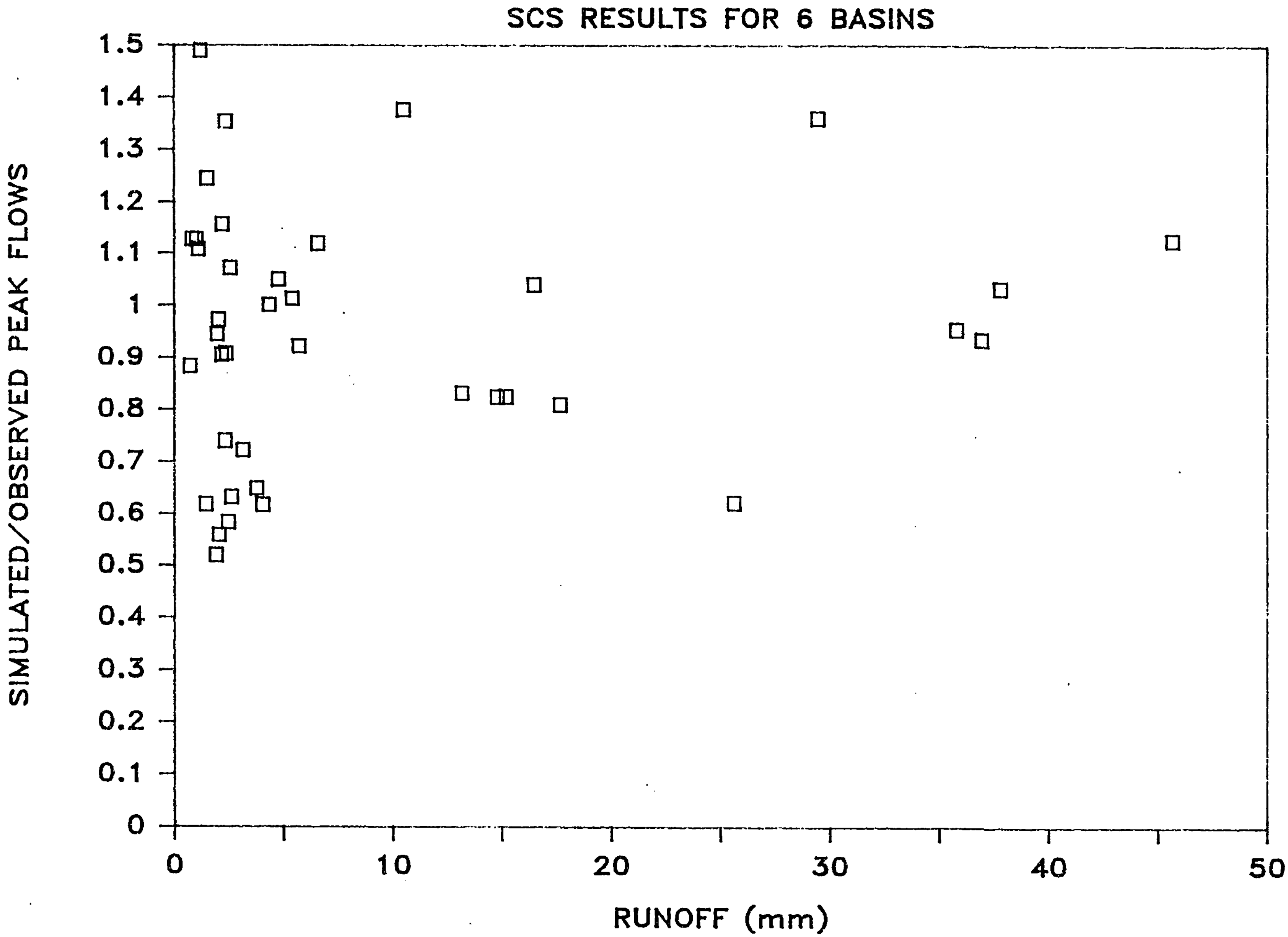
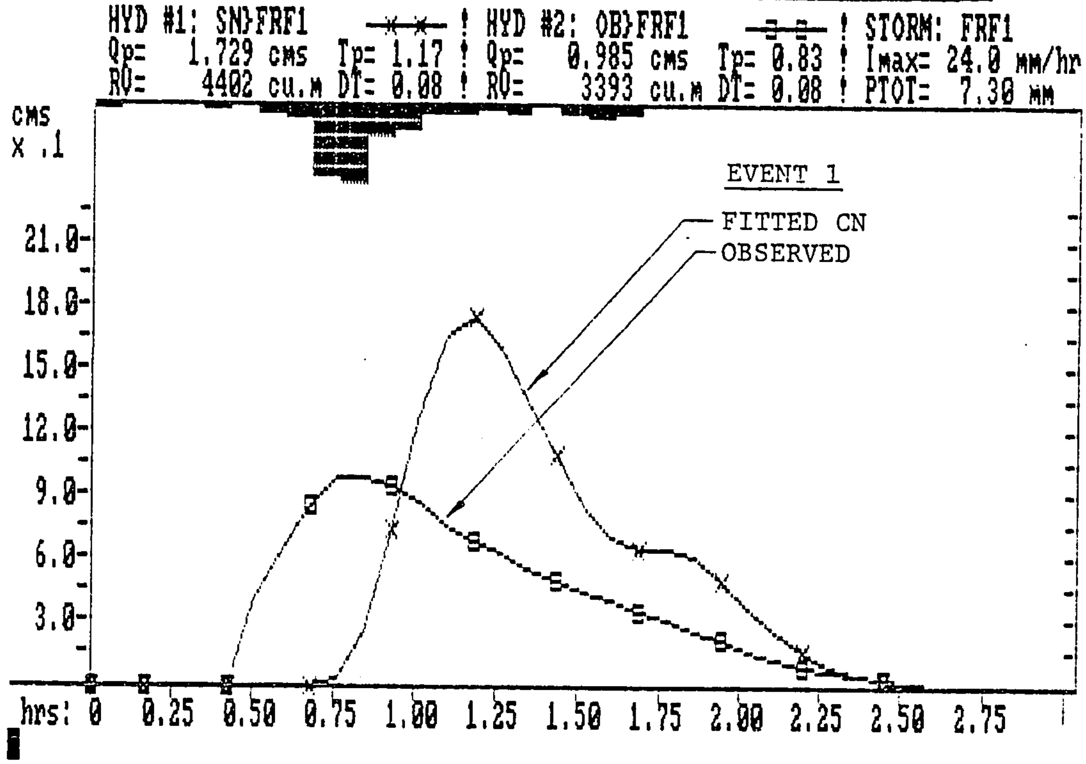


FIG. 4A - FITTED AND CONSTANT CN SIMULATION COMPARISON



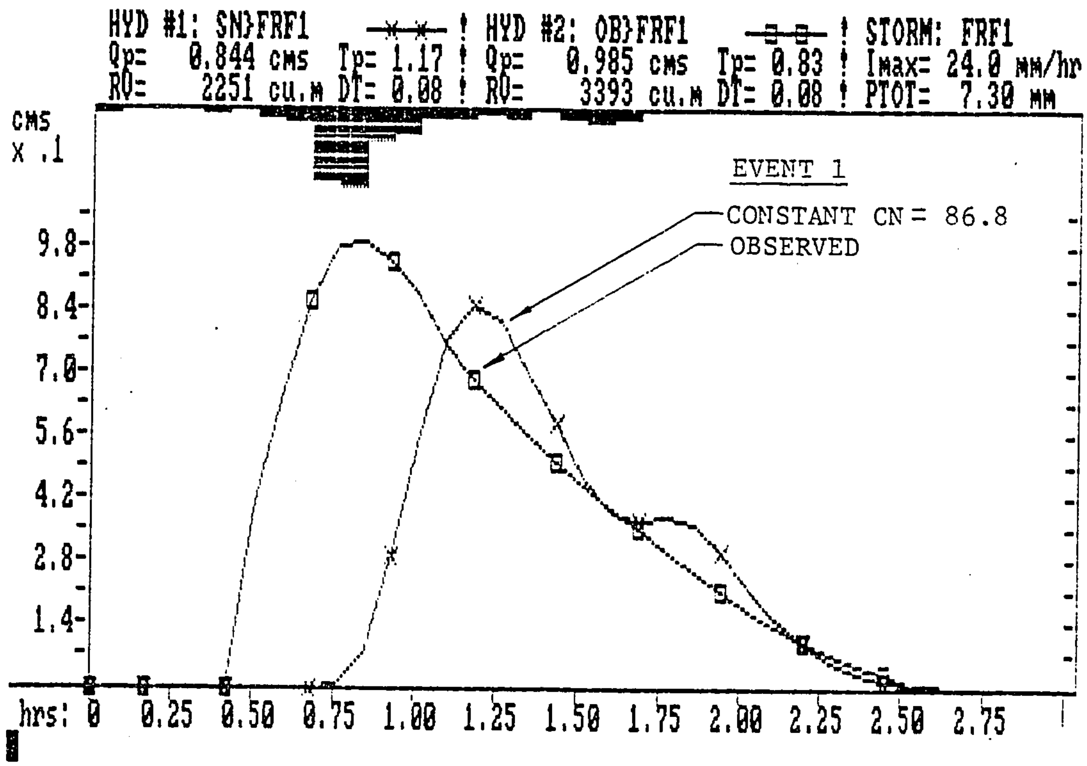
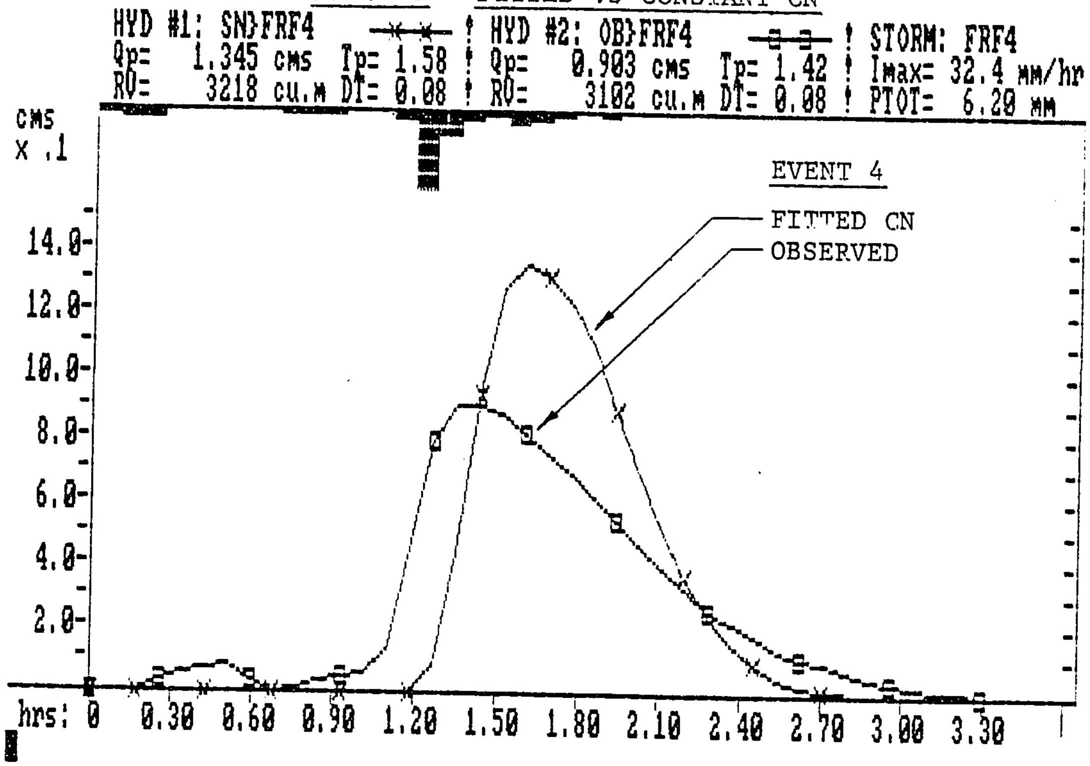


FIG. 4B - FITTED VS CONSTANT CN



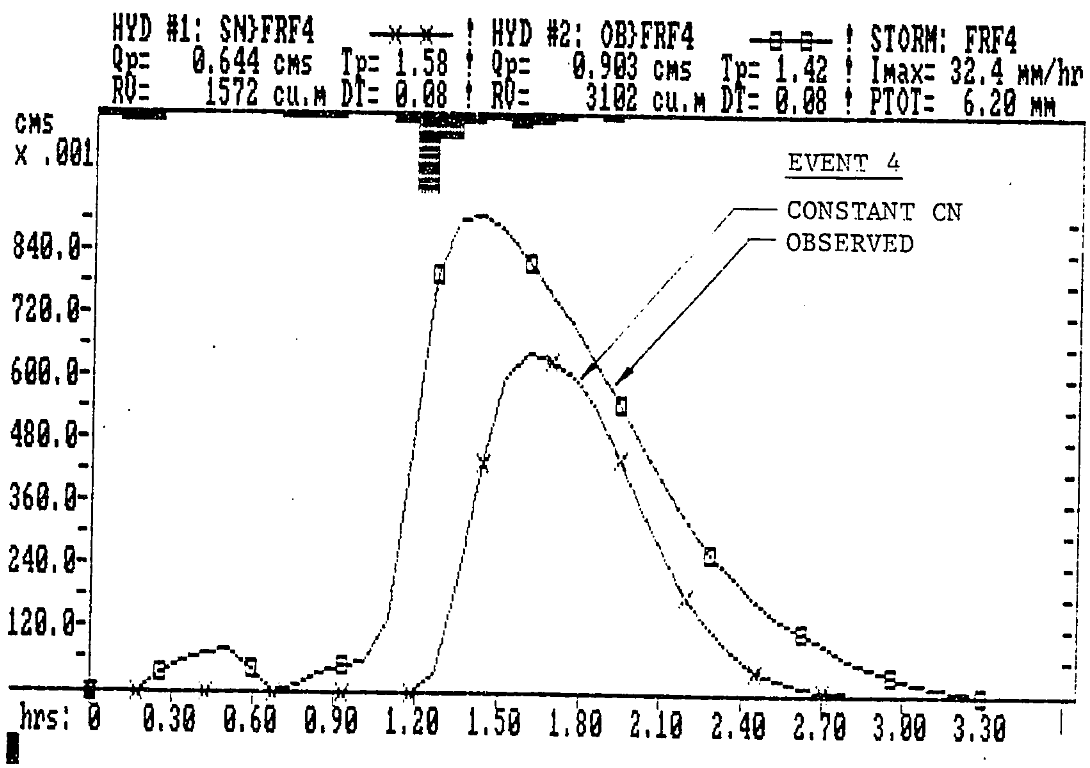
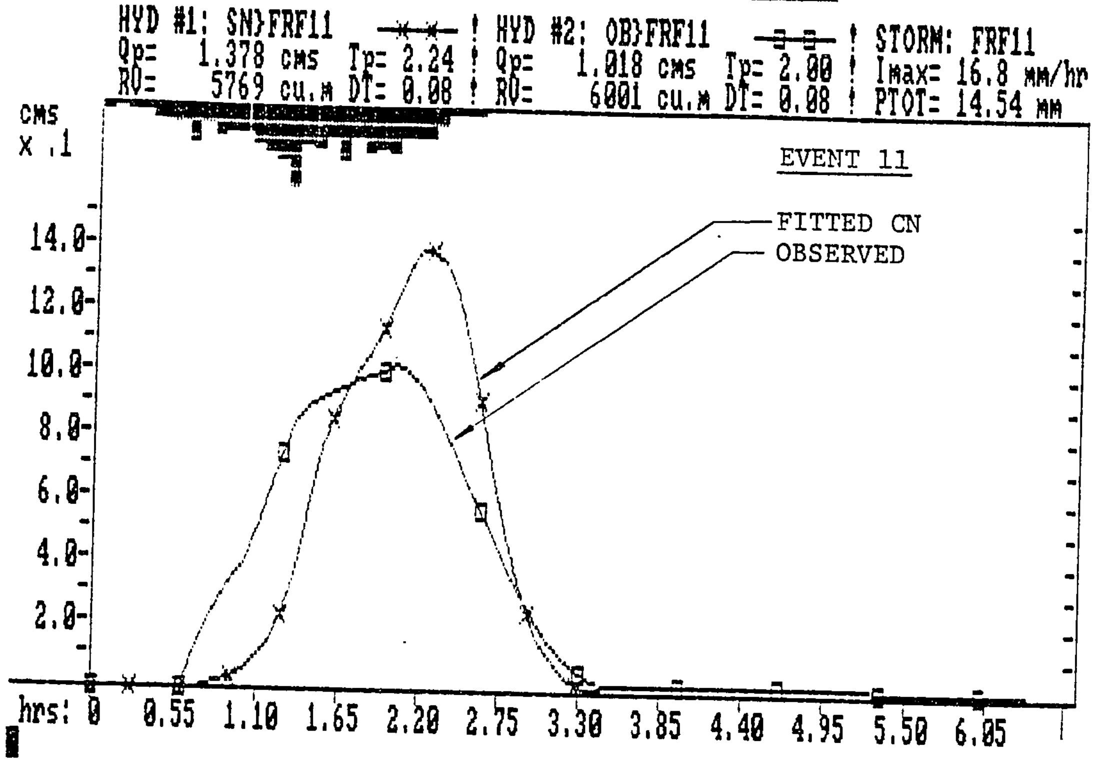


FIG. 4C - FITTED VS CONSTANT CN



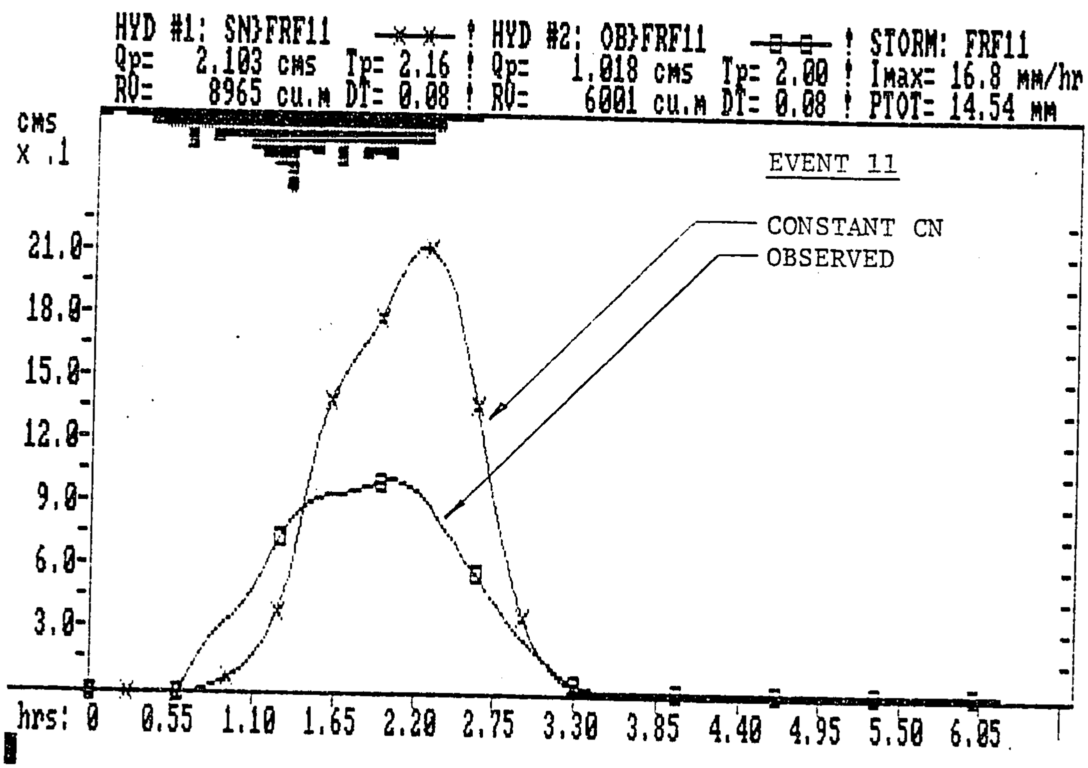
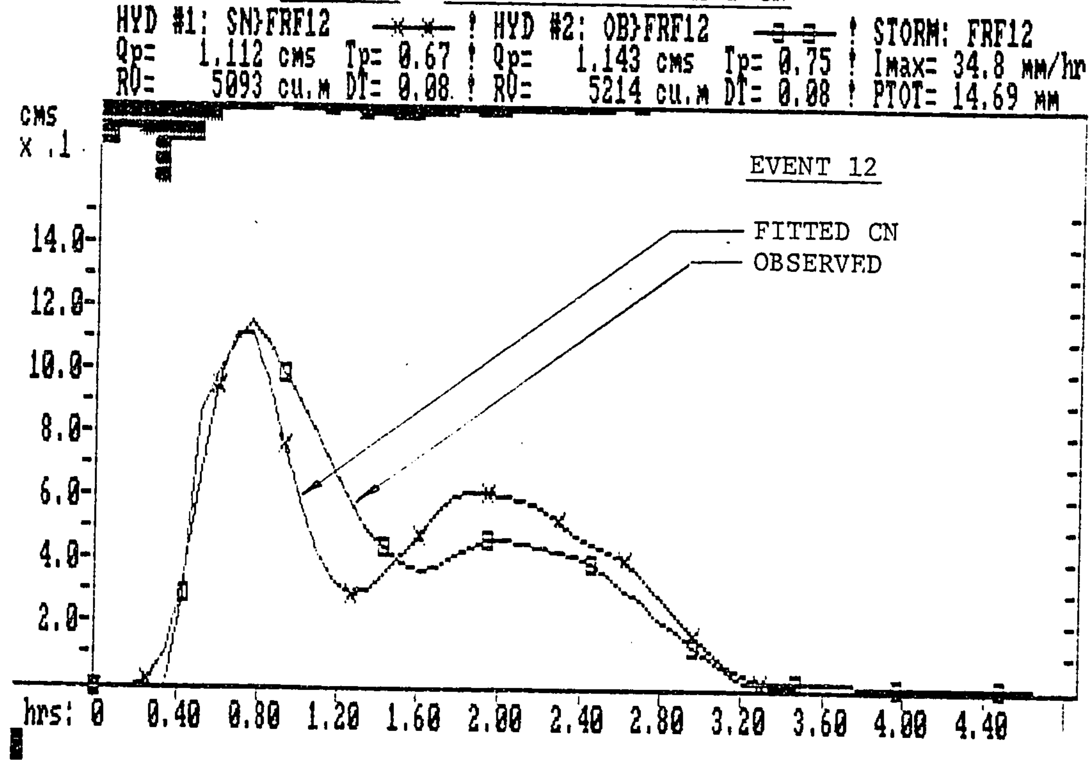


FIG. 4D - FITTED VS CONSTANT CN



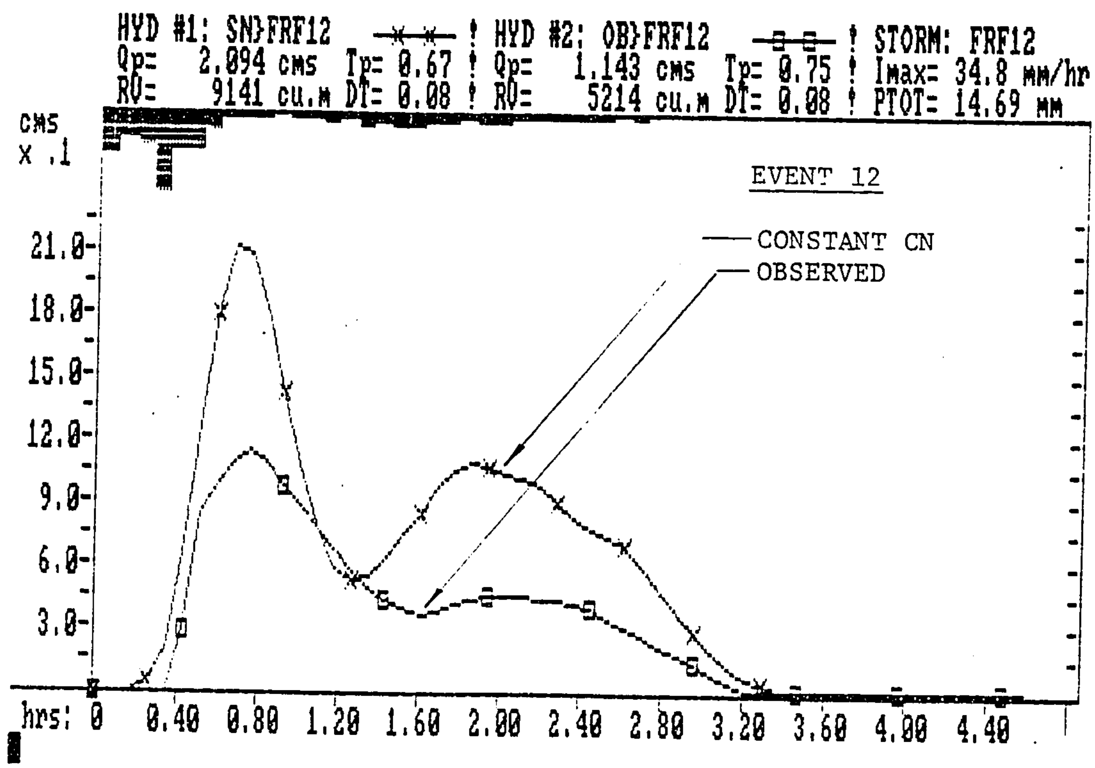
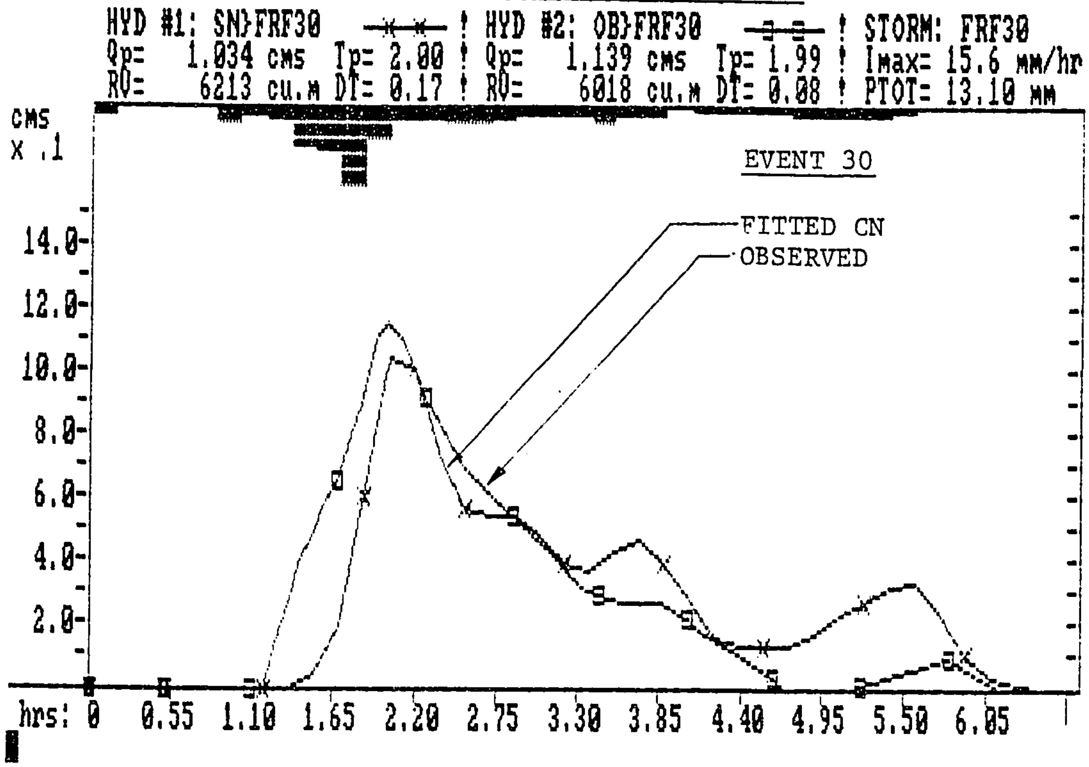


FIG. 4E - FITTED VS CONSTANT CN



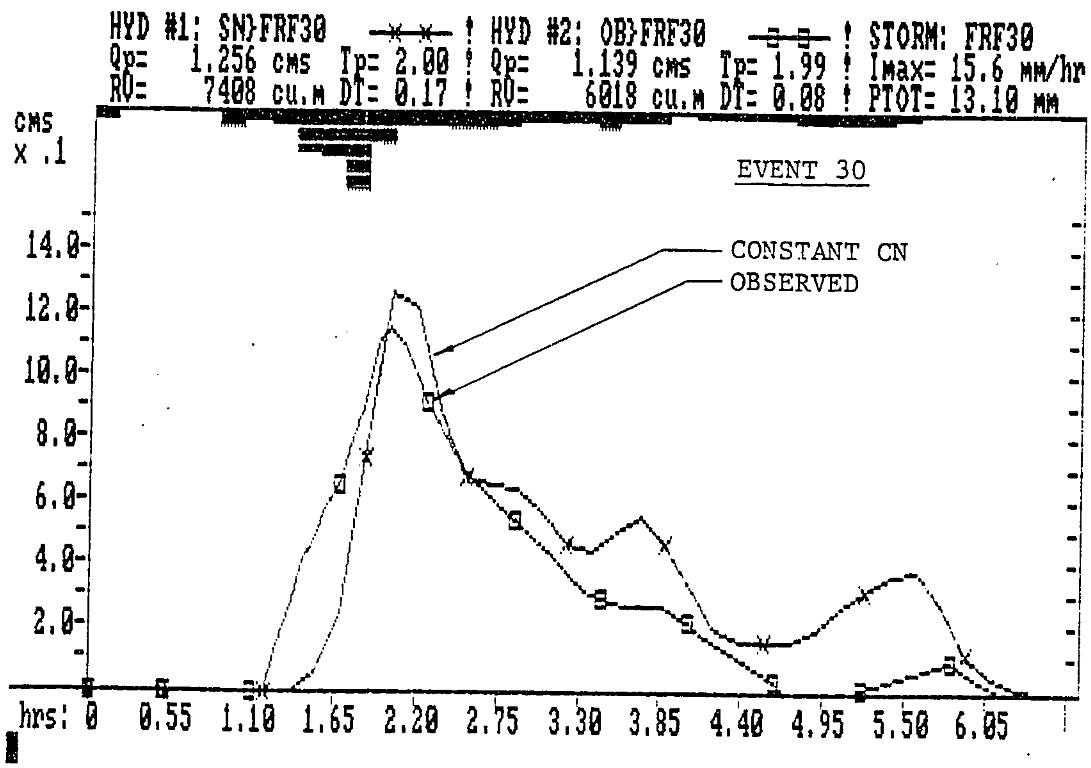
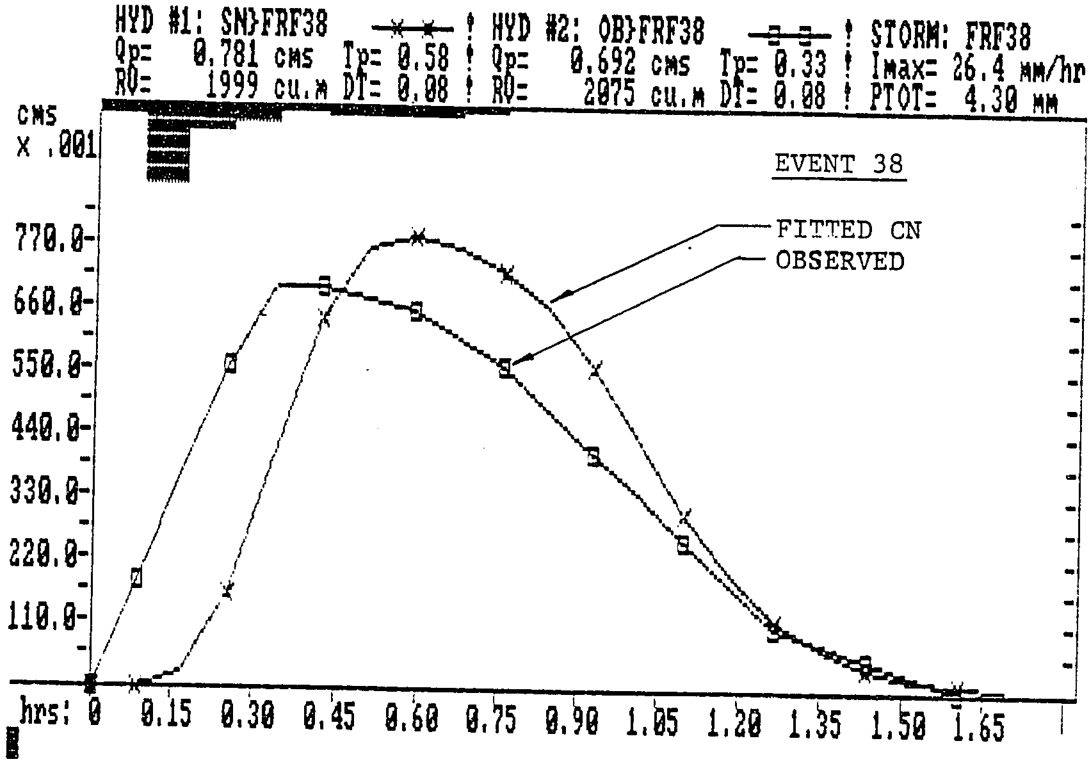
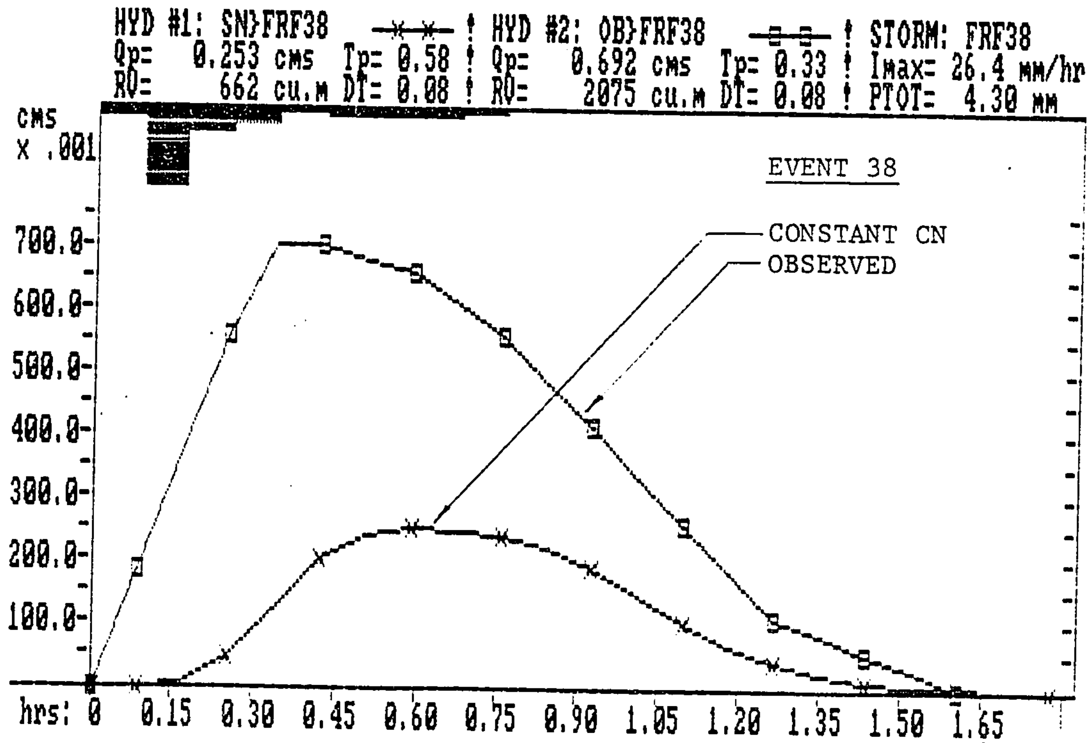


FIG. 4F - FITTED VS CONSTANT CN





and t_p) of the Nash unit hydrograph. Since N and t_p are held constant or defined by equations in the SCS method, it was thought that by calibration an improvement in unit hydrograph performance could be achieved. Optimized N values were found to be highly dependent on individual events and also varied considerably from catchment to catchment. The time to peak was significantly lower than estimates given by the SCS velocity method. After N and t_p were optimized separately for each event, watershed averages were taken as representative of the true calibrated values (Table 3) and are henceforth referred to as the "calibrated" or "optimized" Nash parameters.

A complete set of hydrograph simulations including all watersheds and all events for the calibrated Nash model can be found in Appendix A. Calibrated parameter values are presented in Table 4. A complete event by event listing of observed and simulated peak flows and times to peak for both SCS and Nash models is found in Table 5. Model results were compared in terms of ratios of simulated to observed peak flows which are summarized in Table 6. Standard peak flow errors were also computed for comparison purposes and are discussed in detail below.

Figures 5 to 8 illustrate the sensitivity of peak flows to variations in N and t_p for the Danish watershed. Generally, as the number of Nash reservoirs, N, increases, the peak

TABLE 3 - NASH MODEL CALIBRATION PARAMETERS

CATCHMENT	AREA (Ha)	RANGE OF CN	INITIAL ABSTN (mm)	NASH CALIB. PARAMETERS	
				N	t_p(hr)
MALVERN, BURLINGTON	23.3	87-97	0.0	2.1	0.05
KING'S CREEK, FLORIDA	5.97	91-99	1.0	3.0	0.01
GRAY HAVEN, BALTIMORE	9.40	82-93	0.2	3.5	0.07
ST. DENIS, PARIS	255.0	79-91	1.0	1.4	0.10
MUNKERISPARKEN, DENMARK	6.44	90-98	0.2	3.3	0.06
MISKOLC, HUNGARY	25.34	81-95	0.2	1.8	0.04

TABLE 4 - SCS AND NASH RESULTS FOR ALL EVENTS

CATCHMENT	EVENT NO.	OBSERVED Q_p (cms)	OBSERVED t_p (hr)	SCS (velocity method) Q_p	SCS (velocity method) t_p	OPIMIZED NASH Q_p	OPIMIZED NASH t_p
Malvern, Burlington	2	0.711	1.98	0.77	2.00	0.51	2.13
	8	0.135	1.88	0.18	2.17	0.16	2.26
	12	0.901	0.52	1.28	0.50	0.91	0.65
	15	0.140	0.77	0.18	0.84	0.15	0.92
	16	0.770	0.15	0.60	0.17	0.40	1.88
	21	0.428	0.15	0.36	0.57	0.28	0.69
	22	0.188	0.25	0.20	0.52	0.17	0.64
	23	0.201	0.18	0.15	0.52	0.12	0.62
	24	0.249	0.83	0.23	0.87	0.18	1.00
King's Creek, Florida	1	0.895	0.28	0.90	0.33	0.94	0.33
	2	0.753	1.25	0.72	1.16	0.72	1.25
	3	0.493	0.47	0.66	0.42	0.67	0.42
	4	0.441	2.97	0.51	2.74	0.50	2.74
	5	0.391	0.13	0.25	0.17	0.24	0.25
	6	0.451	0.88	0.40	0.42	0.37	0.50
Gray Haven, Baltimore	1	2.261	1.58	2.51	0.60	2.33	0.70
	2	2.212	1.58	2.28	0.65	2.07	0.70
	3	0.864	0.38	0.87	0.70	0.71	0.75
	4	0.828	0.38	0.77	0.70	0.68	0.75
	5	0.763	0.48	0.70	0.50	0.63	0.60
	6	2.466	0.22	1.63	0.20	1.53	0.35
Munkerisparken Denmark	1	0.103	1.17	0.079	1.05	0.063	1.10
	3	0.111	1.03	0.082	0.95	0.070	1.00
	5	0.129	1.47	0.089	1.40	0.072	1.45
	6	0.162	1.78	0.174	1.70	0.143	1.75
	7	0.063	0.22	0.057	0.15	0.052	0.20
	8	0.307	1.13	0.394	1.05	0.322	1.10
St. Denis, Paris	1	0.985	0.83	1.13	1.30	1.28	1.17
	4	0.903	1.42	1.10	1.40	1.29	1.58
	11	1.018	2.08	1.23	1.99	1.25	2.49
	12	1.143	0.83	0.94	0.58	1.11	0.67
	30	1.139	2.00	0.94	1.90	1.00	2.00
	38	0.692	0.42	0.64	0.30	0.81	0.58
Miskolc, Hungary	1	1.290	0.07	1.33	0.08	1.38	0.13
	2	1.177	0.05	1.10	0.05	1.11	0.10
	3	0.481	0.07	0.53	0.08	0.54	0.13
	4	1.795	0.18	1.70	0.28	1.66	0.32
	5	2.011	0.05	2.09	0.12	2.25	0.17
	6	0.475	0.12	0.58	0.15	0.59	0.20

TABLE 5 - COMPARISON OF SCS AND NASH SIMULATIONS

CATCHMENT	Average Q_{psim}/Q_{pobs}		Range of Q_{psim}/Q_{pobs}		Std Q_p Errors	
	SCS	Nash	SCS	Nash	SCS	Nash
Malvern, Burlington	0.825	1.052	0.52-1.16	0.73-1.42	0.31	0.30
King's Creek, Florida	0.984	0.995	0.61-1.36	0.64-1.34	0.29	0.30
Gray Haven, Baltimore	0.844	0.973	0.62-1.03	0.73-1.11	0.24	0.17
Munkerisparken Denmark	0.784	0.942	0.56-1.05	0.69-1.28	0.36	0.32
St. Denis, Paris	1.204	1.006	0.91-1.49	0.71-1.22	0.36	0.28
Miskolc, Hungary	1.071	1.046	0.92-1.24	0.93-1.22	0.16	0.15

TABLE 6 - RESULTS OF RUNNING DESIGN STORMS

Catchment	CN*	Model **	N	t_p hr	5-Year Storm Q_p cms	5-Year Storm Peak Ratio+	5-Year Storm Runoff Coeff.	100-Year Storm Q_p cms	100-Year Storm Peak Ratio	100-Year Storm Runoff Coeff.
Malvern,	86	1	4.75	0.44	1.08	1.00	0.46	2.50	1.00	0.54
Burlington	86	2	4.75	0.19	1.59	1.46	0.46	3.56	1.42	0.54
	86	3	2.10	0.05	1.69	1.56	0.46	3.67	1.51	0.54
	-	4	-	-	1.16	1.07	0.46	2.99	1.20	0.54
St. Denis,	86	1	4.75	1.49	5.05	1.00	0.46	11.7	1.00	0.54
Paris	86	2	4.75	0.33	13.8	2.72	0.46	31.6	2.70	0.54
	86	3	1.40	0.10	12.5	2.47	0.46	28.7	2.45	0.54
	-	4	-	-	12.0	2.38	0.46	22.6	1.93	0.54
King's Creek	89	1	4.75	0.48	0.31	1.00	0.52	0.67	1.00	0.61
Florida	89	2	4.75	0.15	0.48	1.59	0.52	1.03	1.54	0.61
	89	3	3.00	0.10	0.50	1.64	0.52	1.05	1.55	0.61
	-	4	-	-	0.40	1.31	0.52	0.90	1.34	0.61

NOTE: Peak Ratios are with respect to the first peak (model 1).

* Composite CN determined from SCS tables.

** Legend of Methods Used:
1 - SCS model, time to peak computed by the lag method
2 - SCS model, time to peak computed by the velocity method
3 - Nash unit hydrograph model, time to peak and N calibrated
4 - URBHYD model

+ Ratio of peak flow divided by method 1 peak.

FIG. 5 — PEAK FLOW VRS. N

EVENT 5, CN=93, IA=1.0 mm, TP=10 min

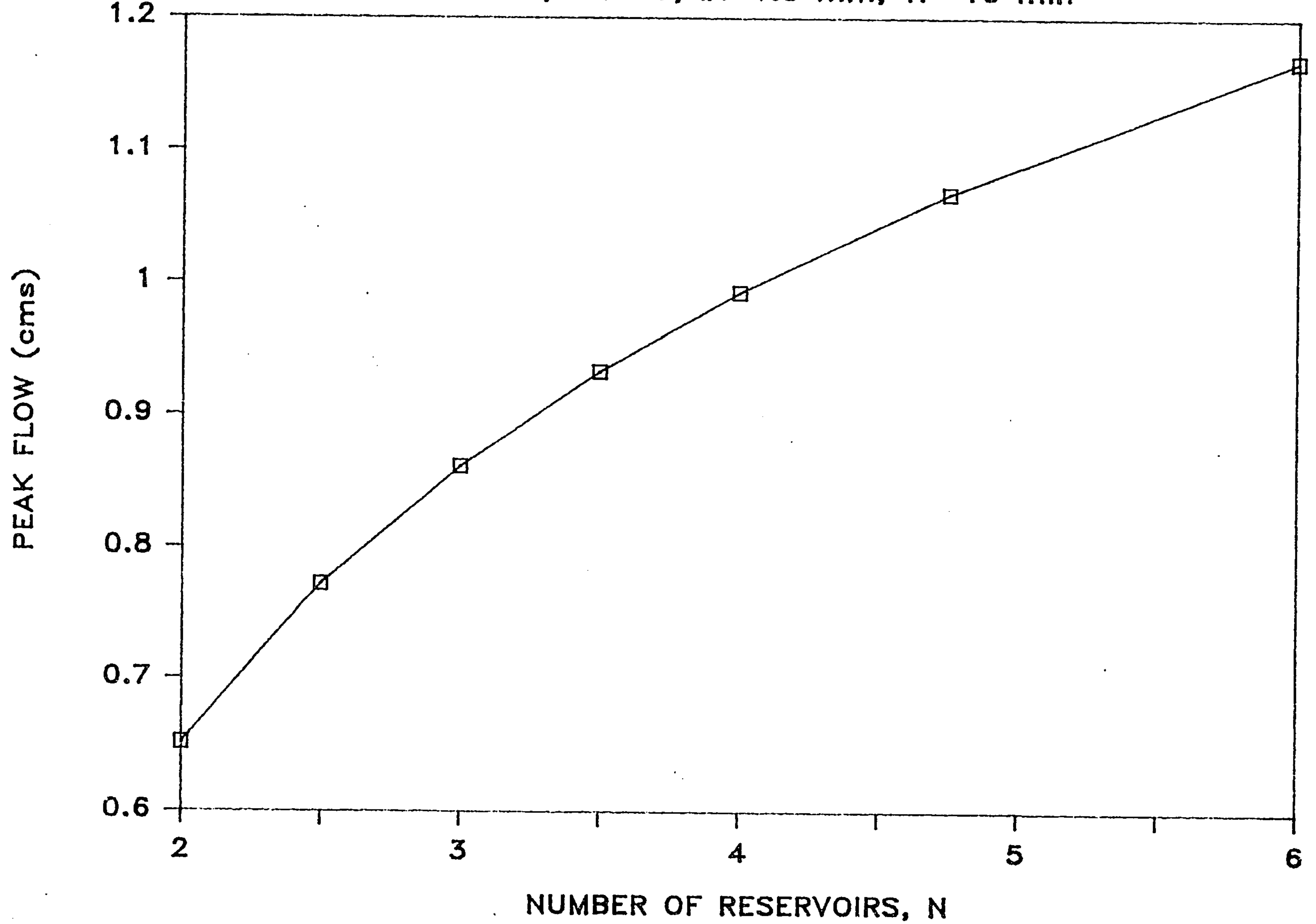


FIG. 6 — PEAK FLOW VRS. TP

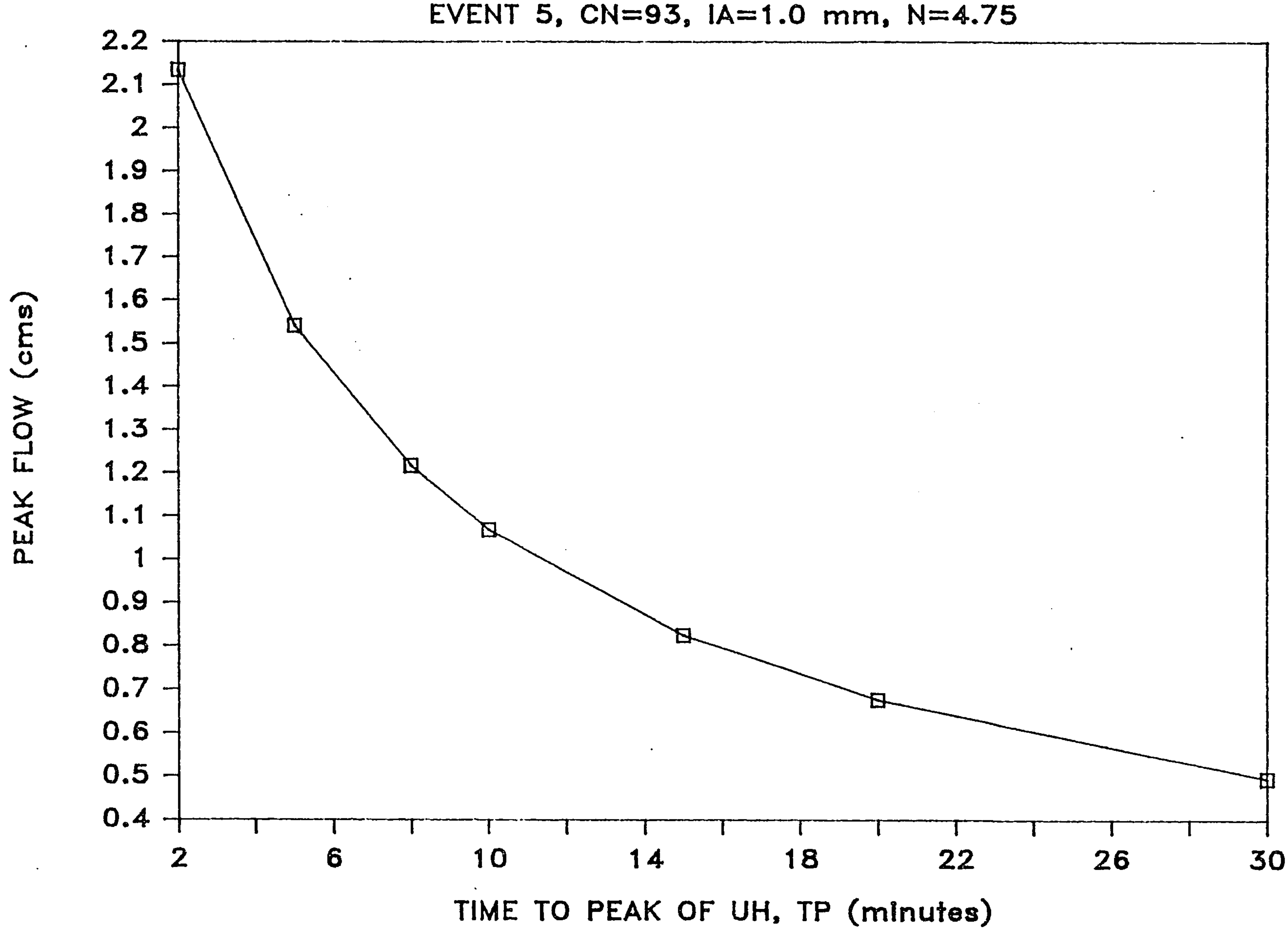


FIG. 7 — PEAK FLOW VRS. N, TP

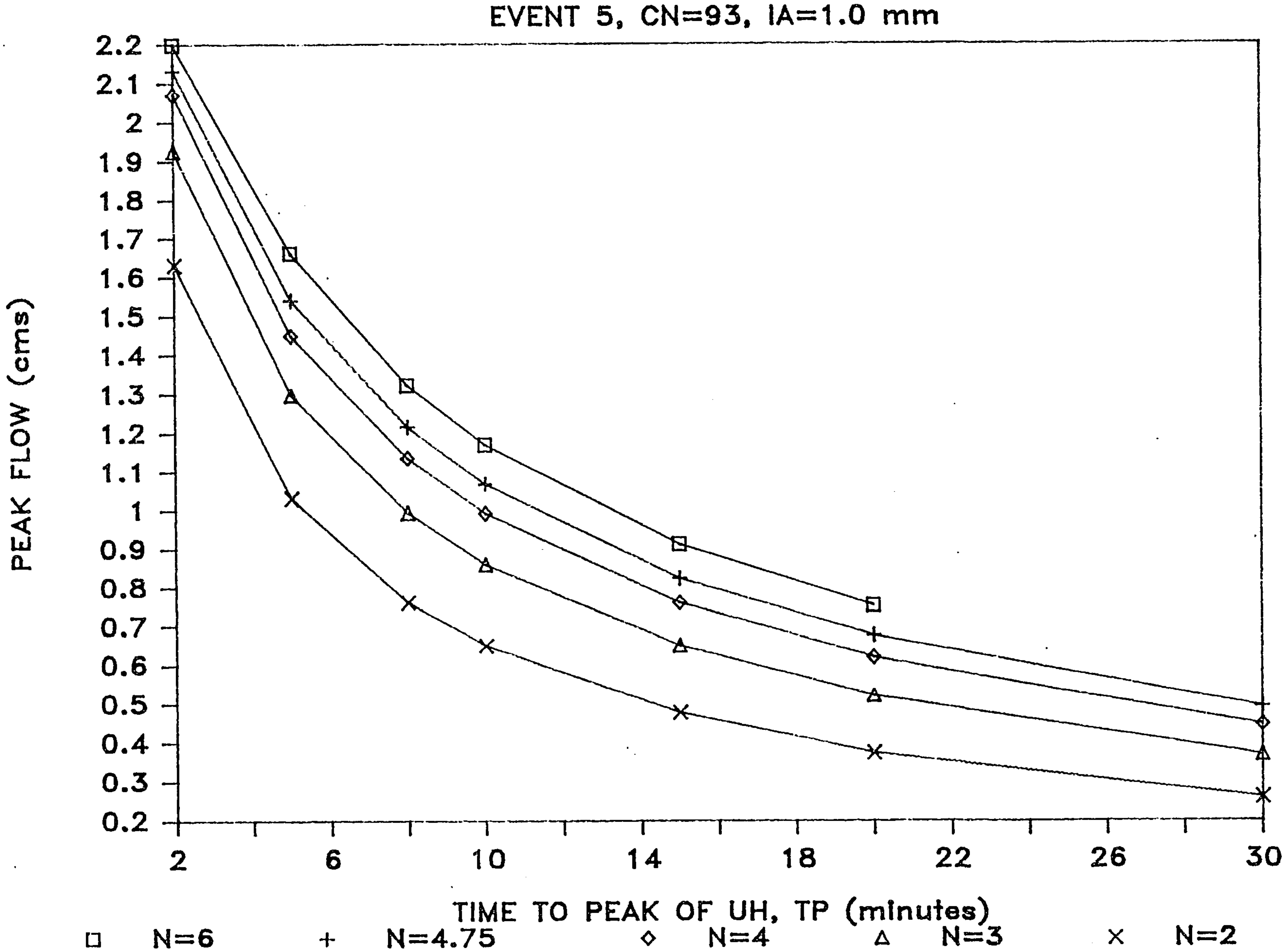
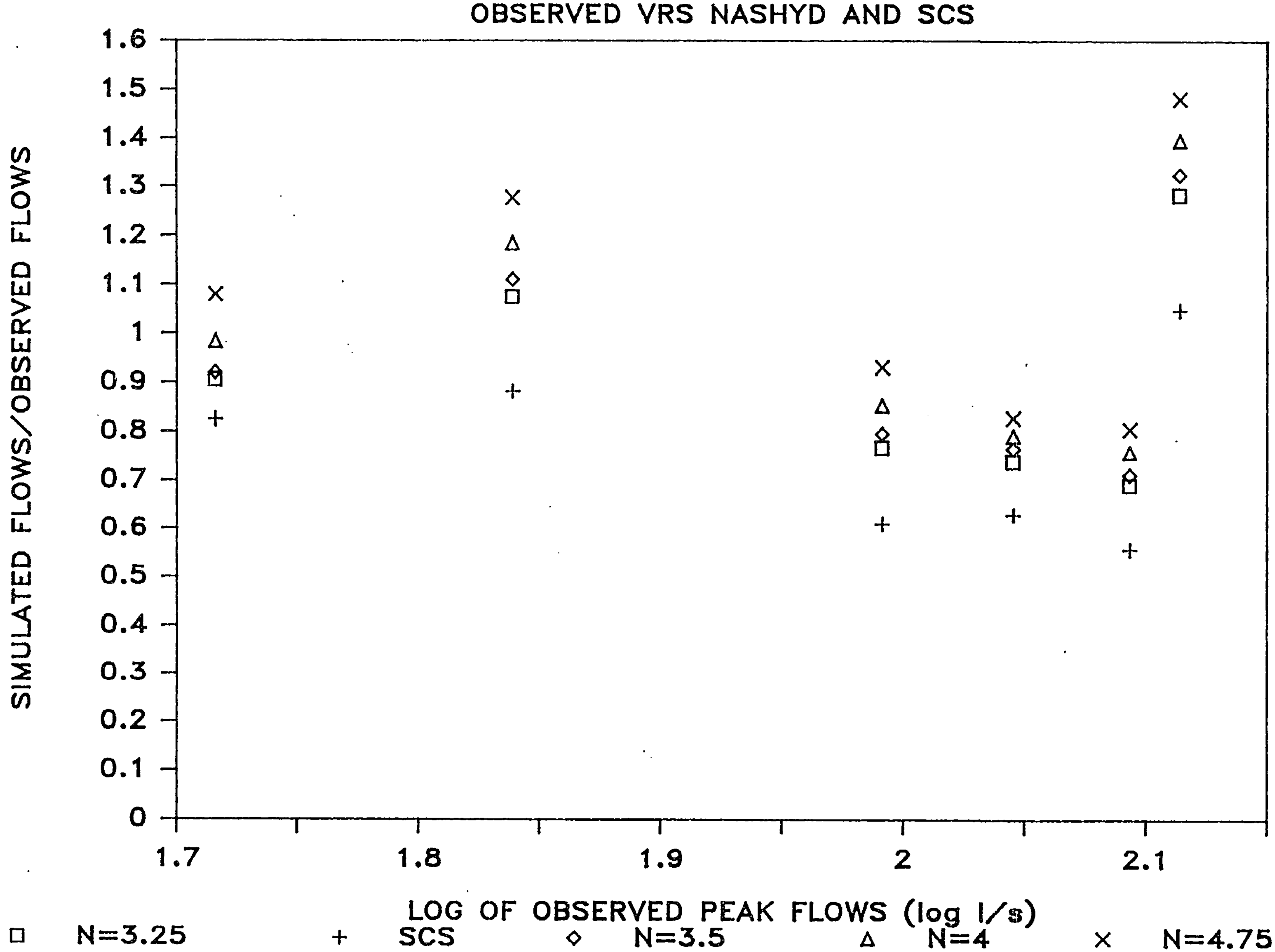


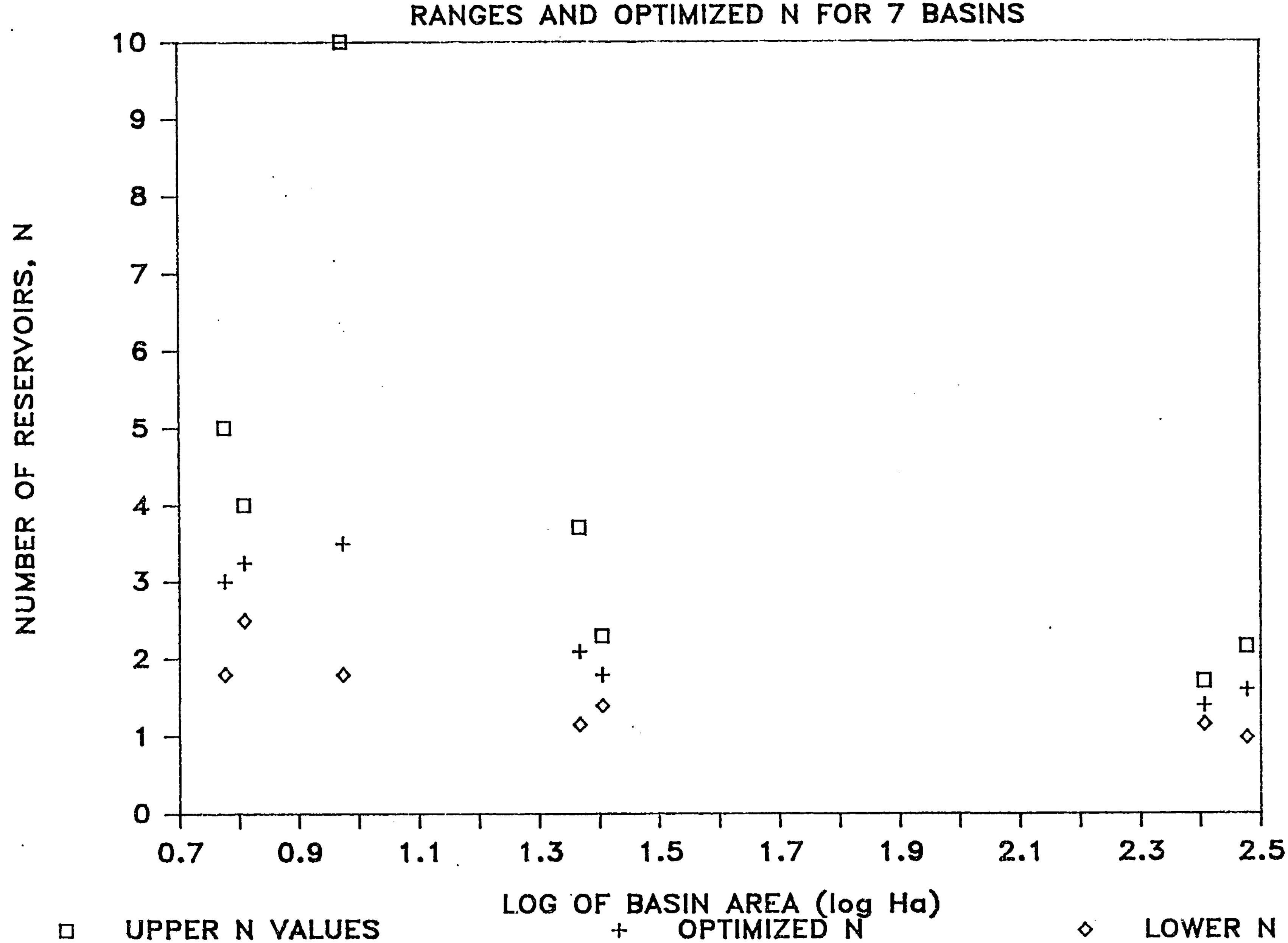
FIG. 8 — DENMARK PEAK FLOWS



flow also increases (Figure 5). Counteracting this, as the time to peak increases, the peak of the unit hydrograph decreases (Figure 6). This is an important observation which suggests that various combinations of N and t_p can generate identical unit hydrograph peaks (Figure 7). Figure 8, particularly, illustrates the changes in peak flows for various values of N. Inspection of the figure reveals points designated "SCS" which are not the same as points designated "N=4.75". The variance is a result of using different times to peak; those computed by the velocity method for SCS points and optimized t_p values for the latter group of Nash points. However, N is equal to 4.75 for the points designated "SCS".

Figure 9 indicates the range of calibrated N for 39 events along with results obtained by Bielawski (1986) plotted against basin area. The downward trend of N with area in Figure 9 appears to be very weak. Bielawski calculated N and t_p from first and second moments of excess rainfall and direct runoff distributions for a 300 ha watershed in Warsaw and found that N ranged from 0.98 to 2.16 with t_p ranging between 13 and 52 minutes. For comparison, the largest catchment of this study, St. Denis, France, is 255 ha in area and produced an optimum time to peak of 6 minutes with N = 1.4.

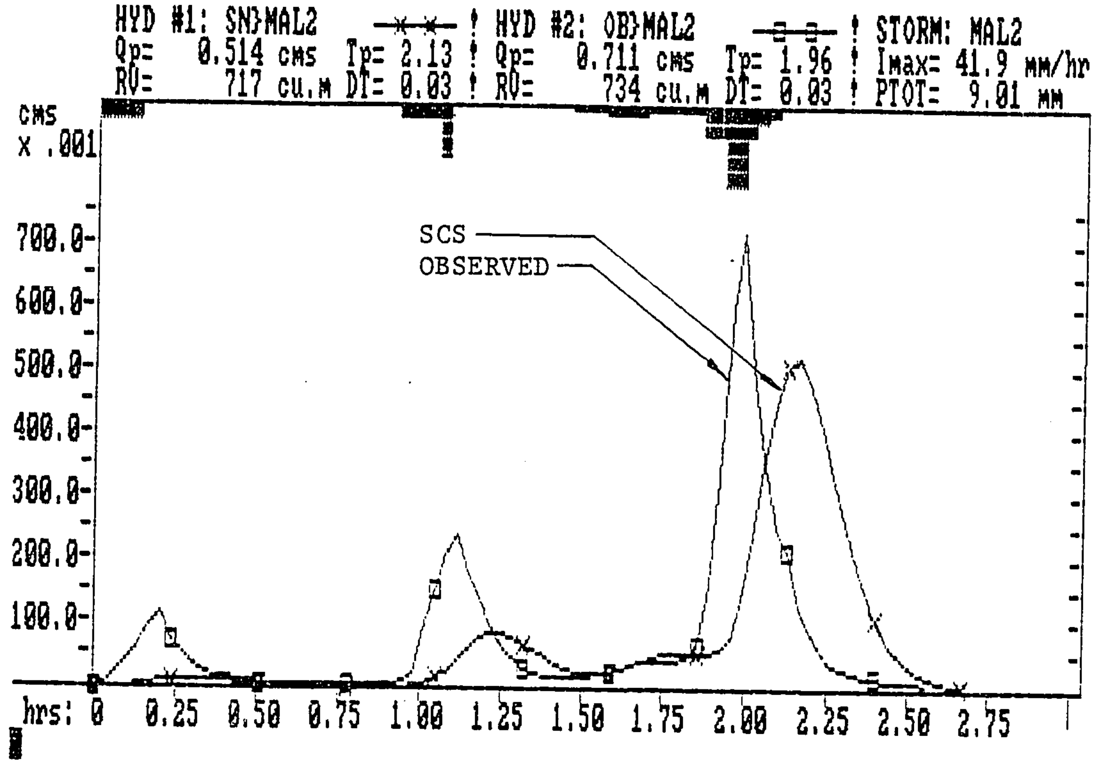
FIG. 9 — N VRS. BASIN AREA



For Malvern, a catchment where the SCS unit hydrograph performance was weak, a reduction of N and t_p produced significant improvements (Figures 10A to 10I). Both the peak flows and the times to peak were generally improved for this basin. Figures 11A and 11B clearly demonstrate that improvements in the peak flows were not as significant as the improvements in the initial peaks of multi-peak hydrographs. In other words, the response times have also improved, as specifically illustrated in Figure 12. For watersheds where SCS unit hydrograph parameters performed acceptably, calibration of N and t_p gave only marginally improved results. Results for the catchment in Florida, for example, were nearly identical for both models. It is noteworthy that the Florida catchment experienced the largest rainfalls in this study.

The overall performance of both models for each catchment is illustrated in Figures 13 and 14 in terms of the criteria established in equation (7). Mean standard *peak flow* errors as defined by equation (7) are presented in Table 6 and plotted in Figure 13. Recall that the number of degrees of freedom in the equation depend on the number of events used in calibration and on the number of free model parameters. Accordingly, with only the time to peak as variable, the standard error for the SCS unit hydrograph is subject to four degrees of freedom. On the other hand, the Nash model,

FIG. 10A MALVERN EVENT 2



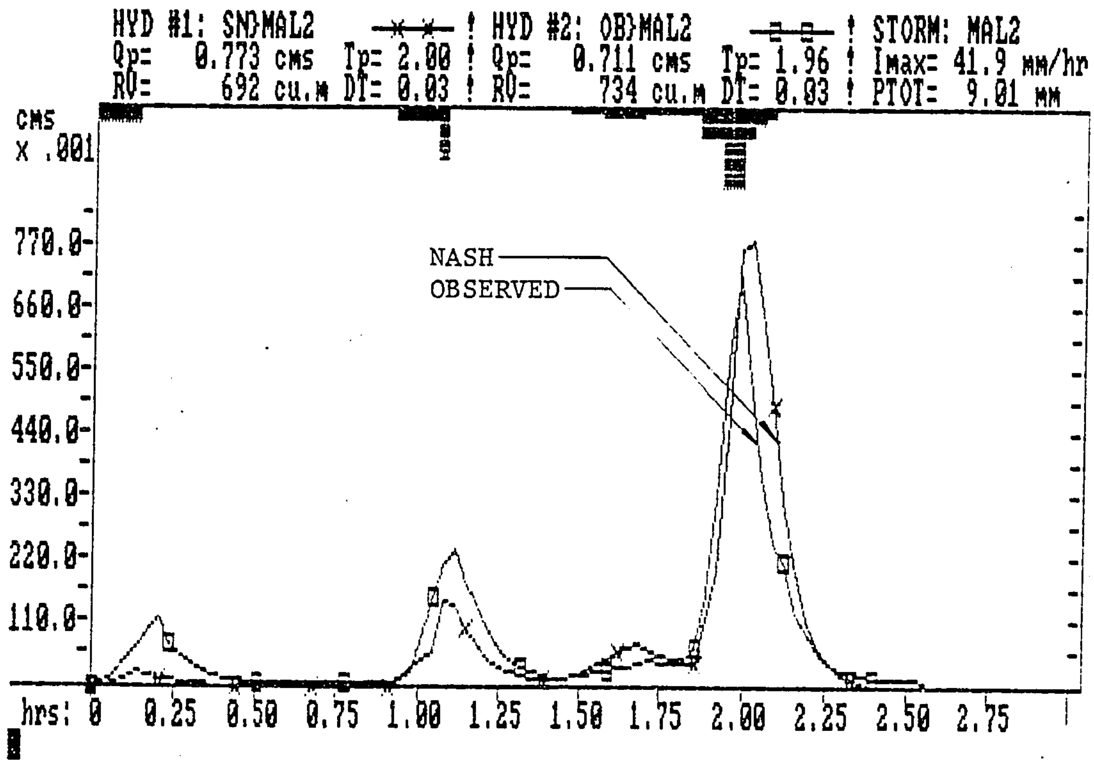
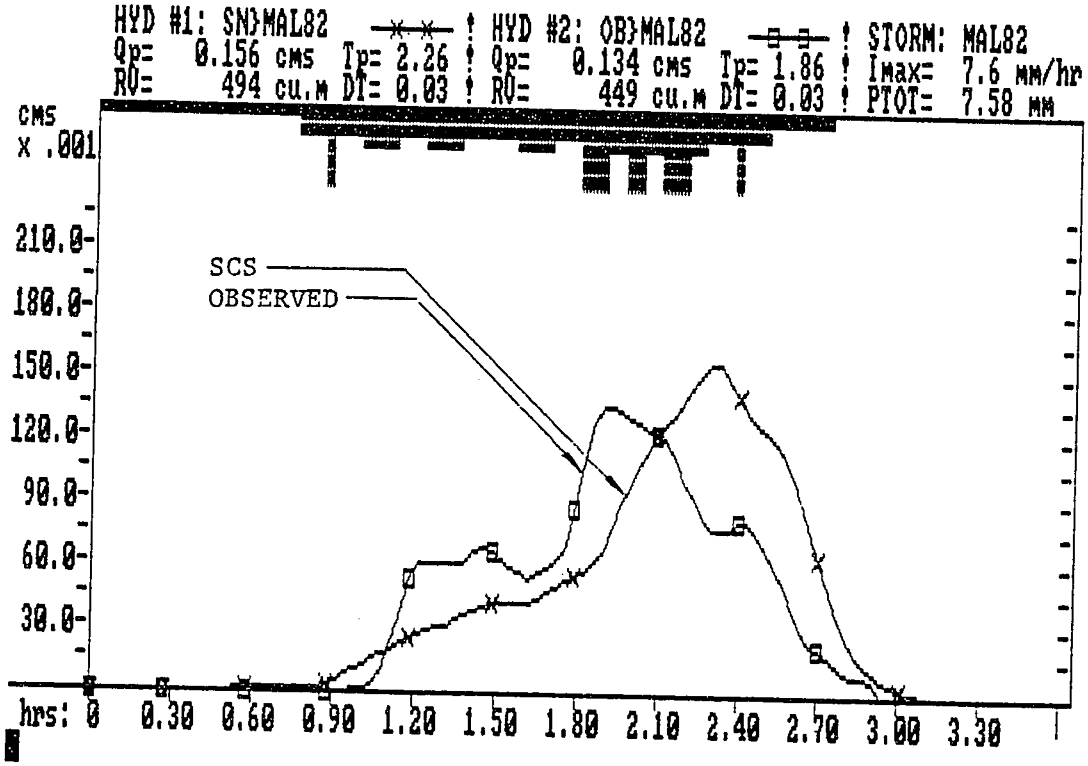


FIG. 10B MALVERN EVENT 8



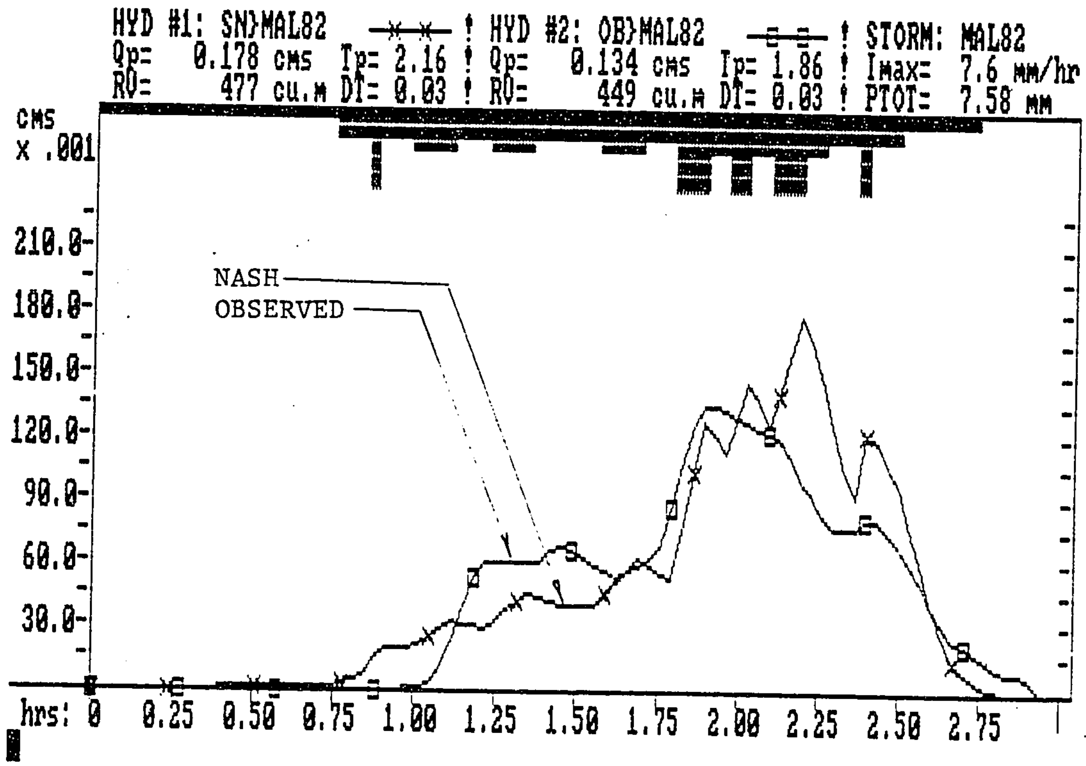
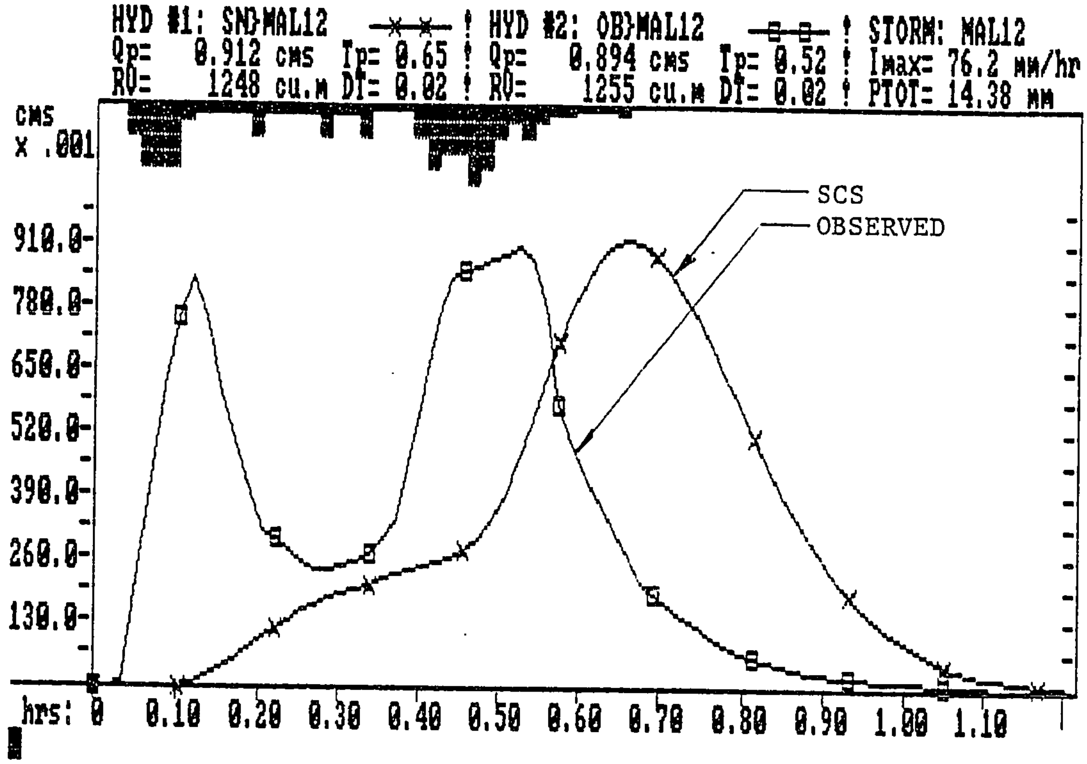


FIG. 10C MALVERN EVENT 12



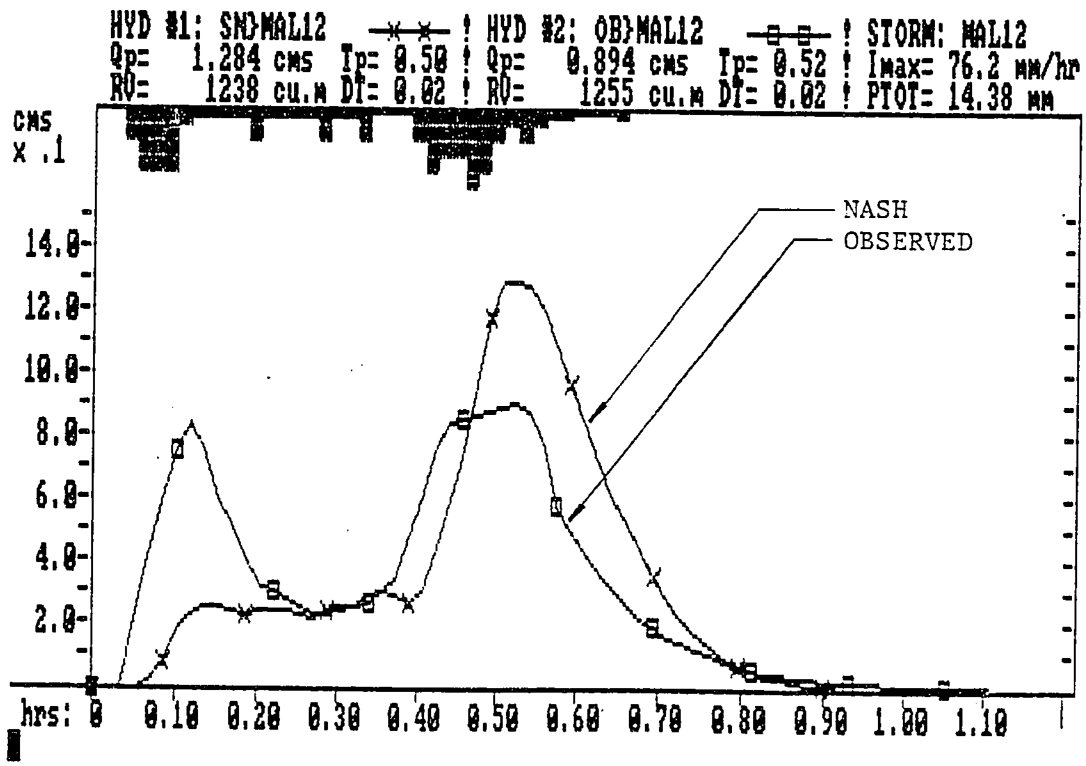
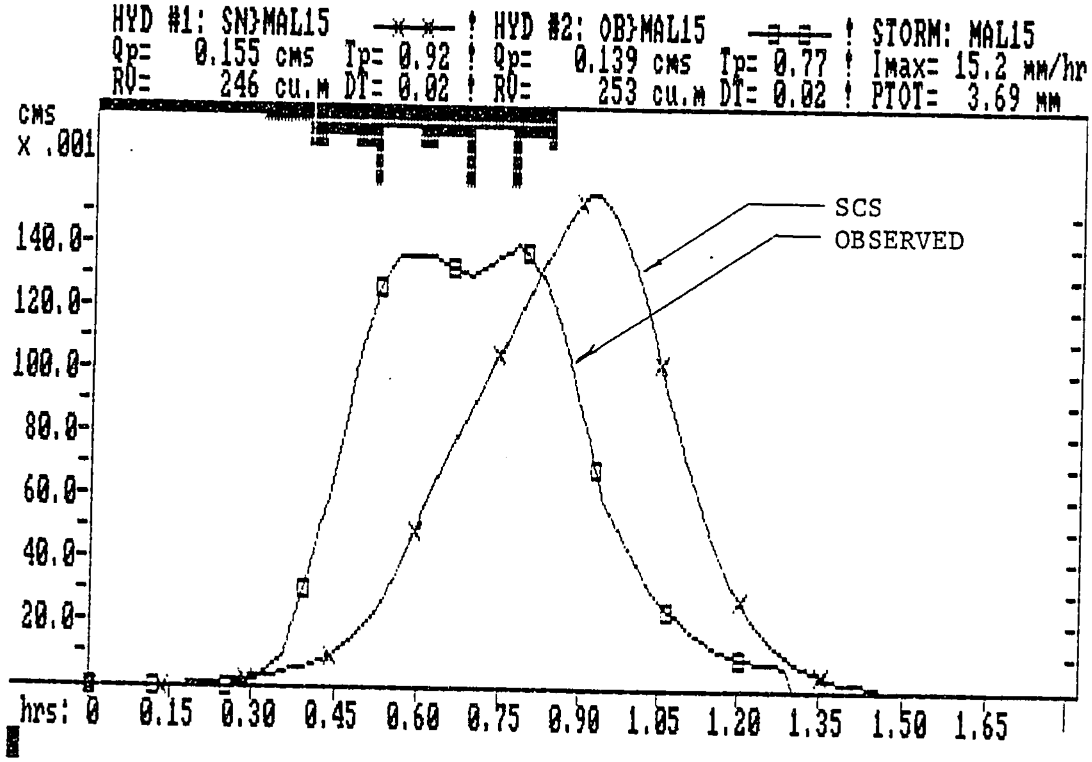


FIG. 10D MALVERN EVENT 15



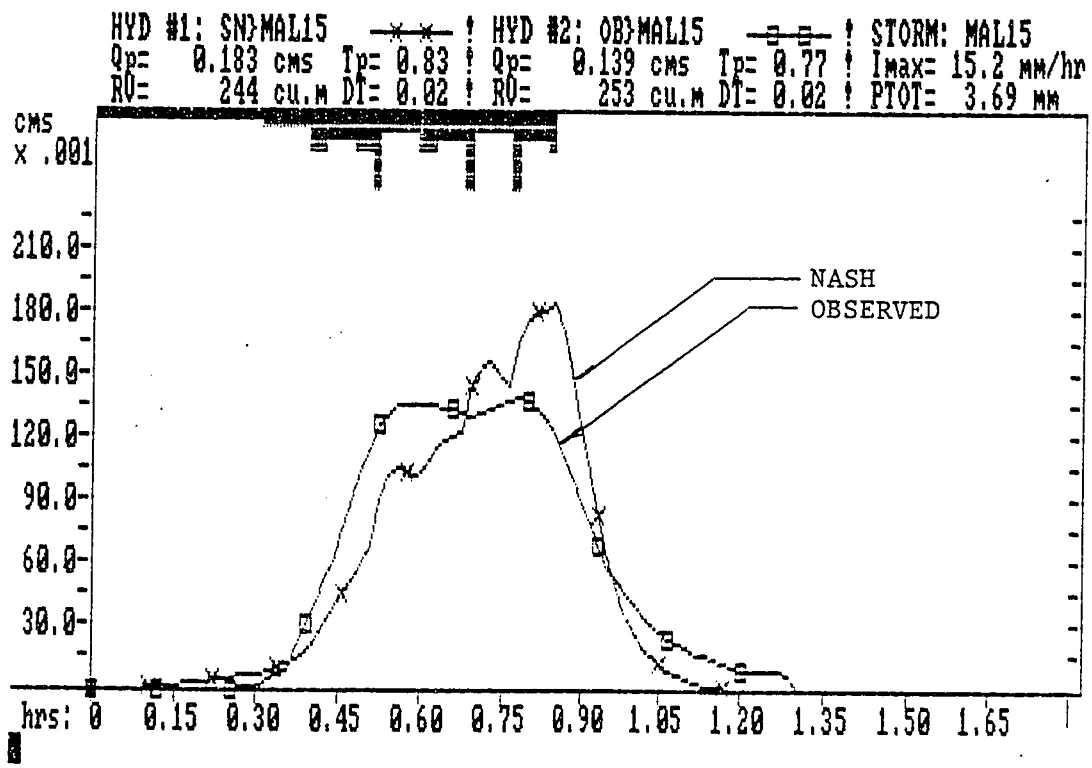
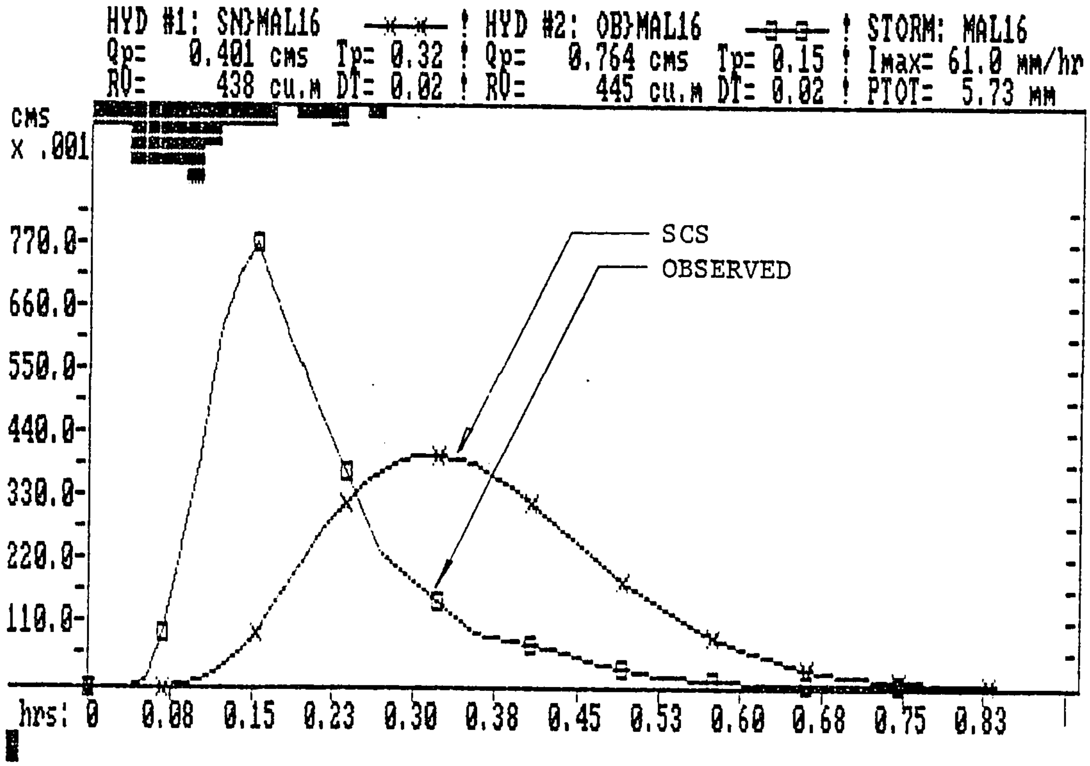


FIG. 10E MALVERN EVENT 16



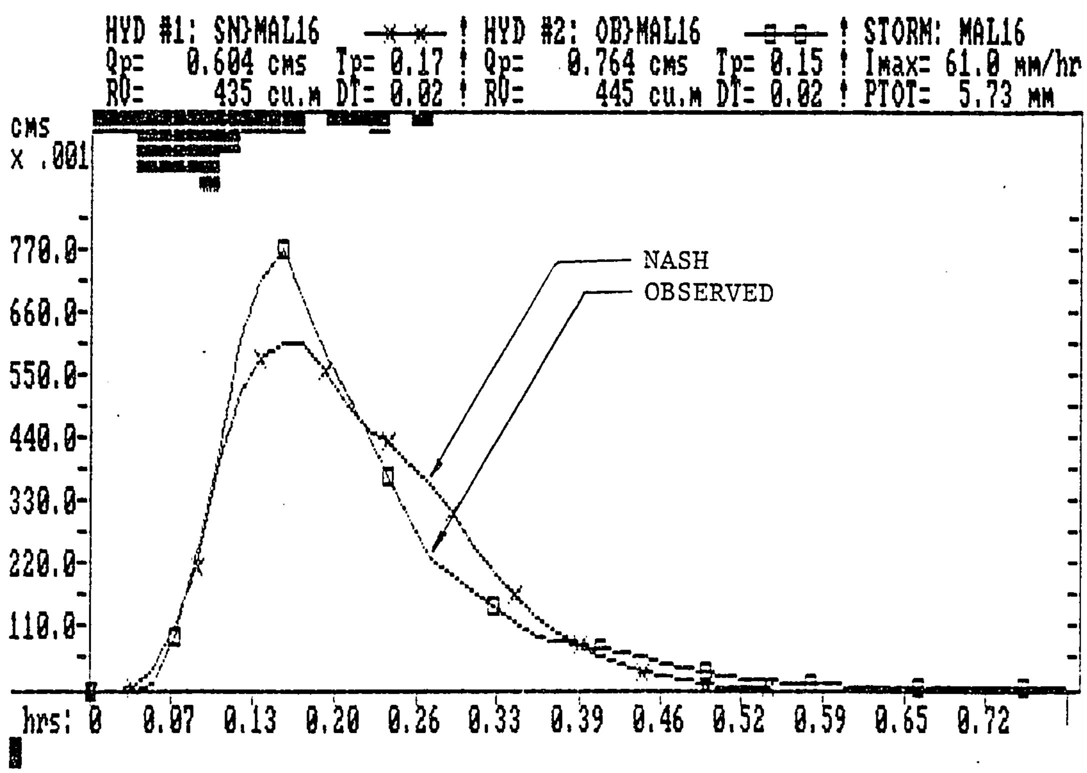
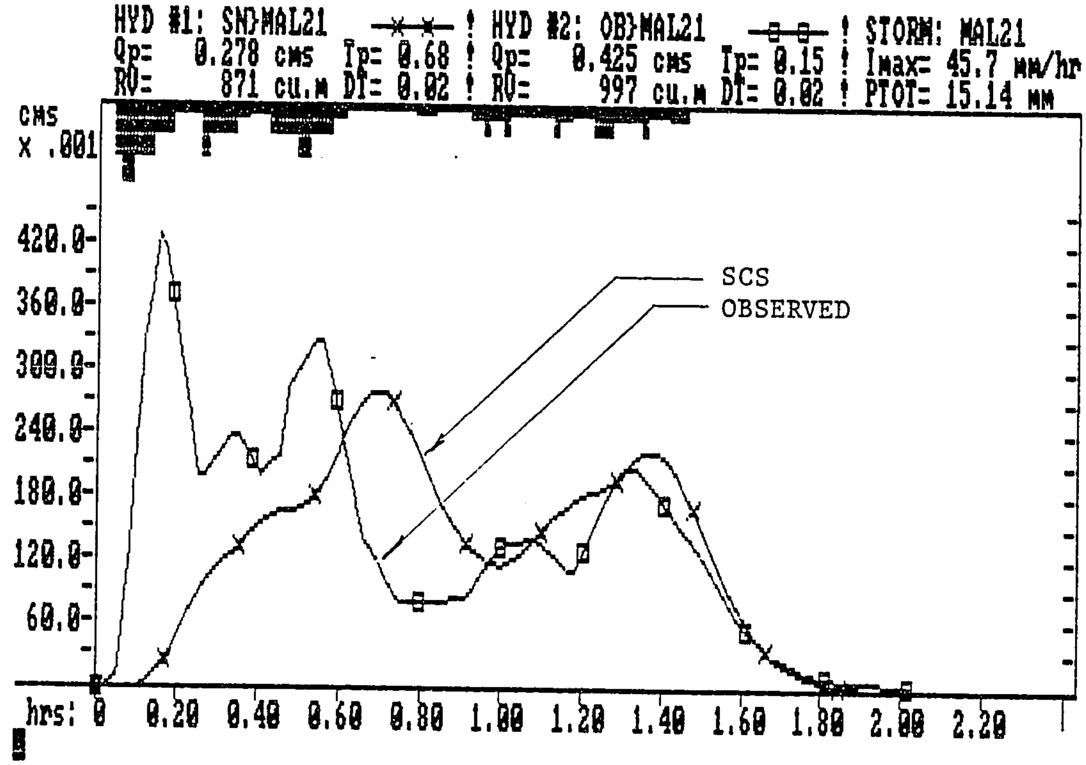


FIG. 10F MALVERN EVENT 21



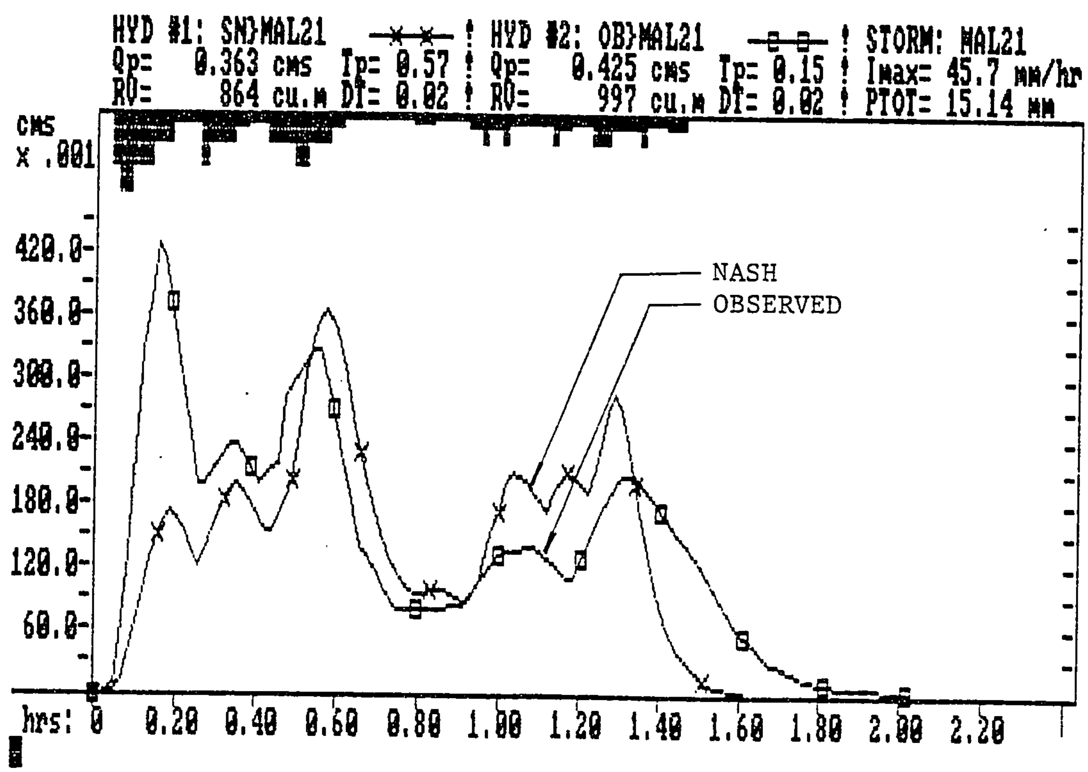
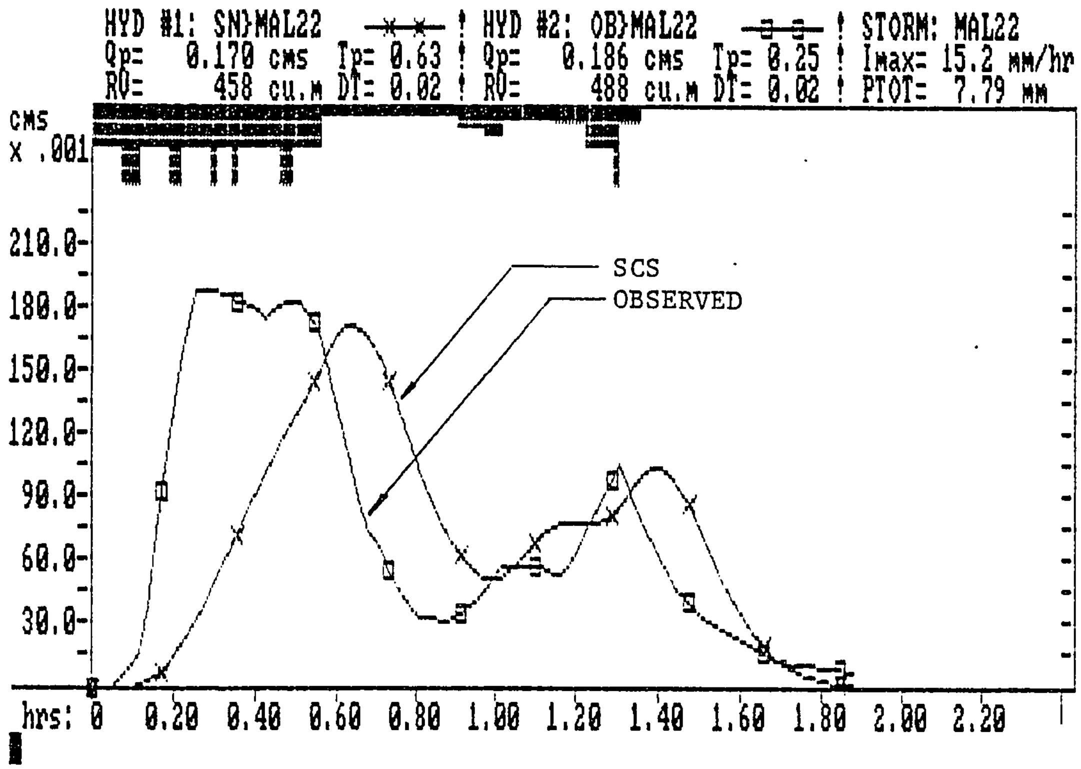


FIG. 10G MALVERN EVENT 22



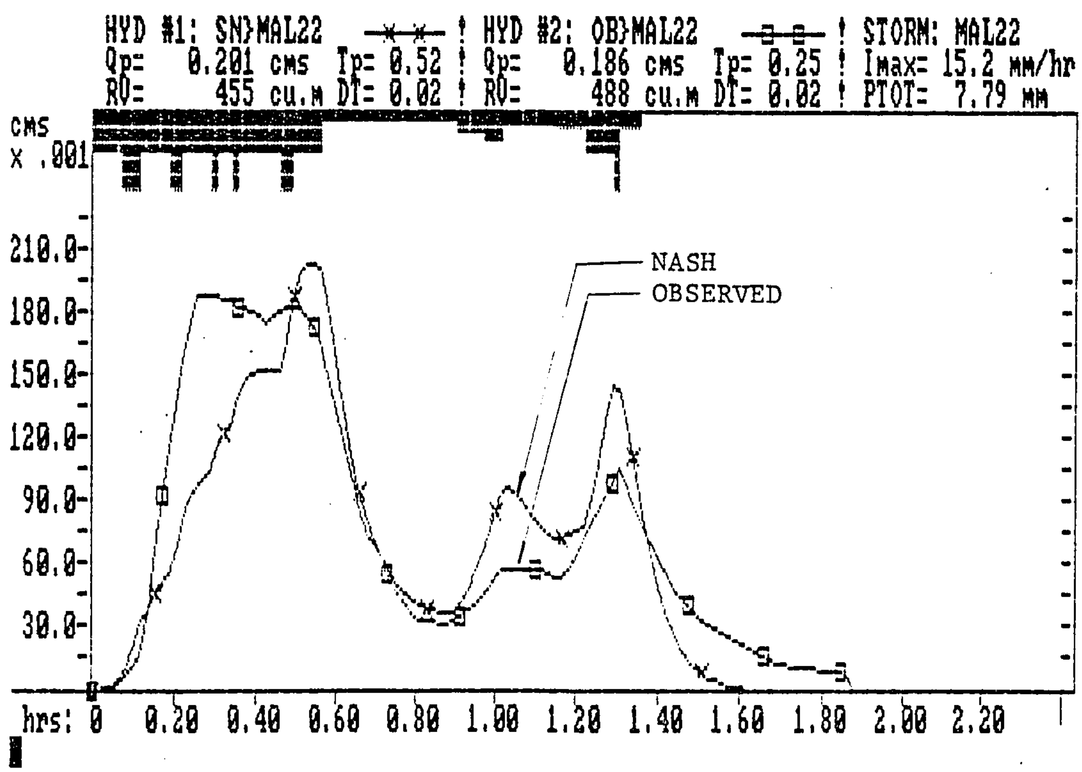
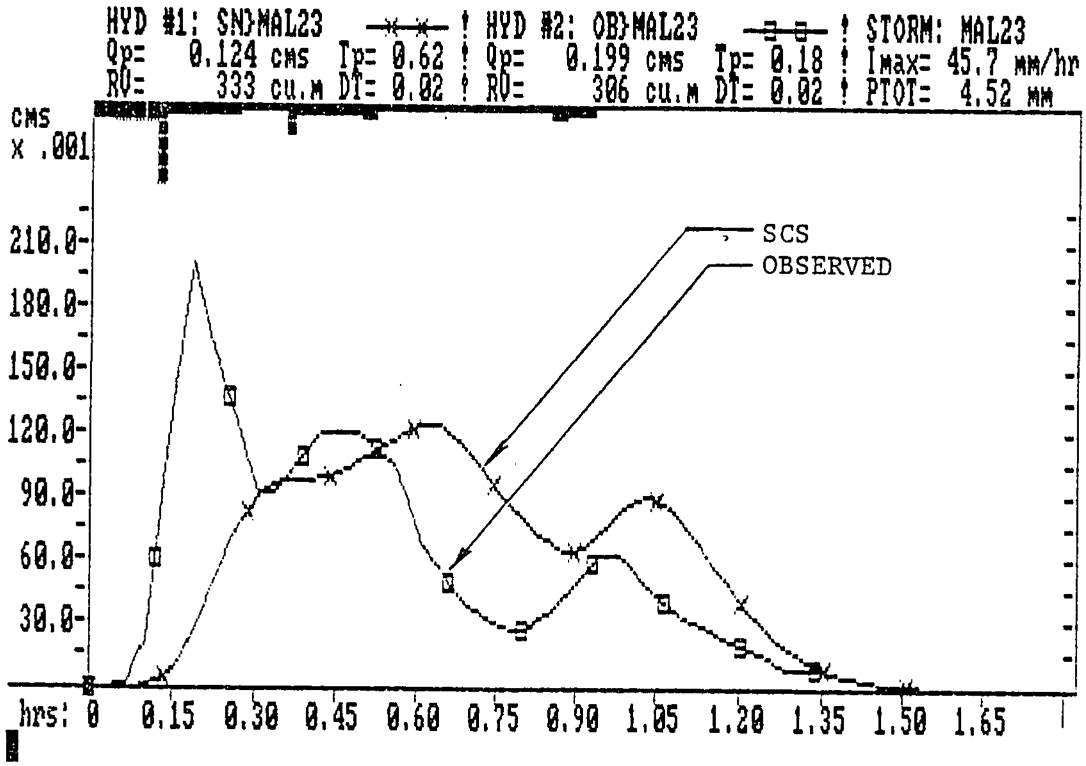


FIG. 10H MALVERN EVENT 23



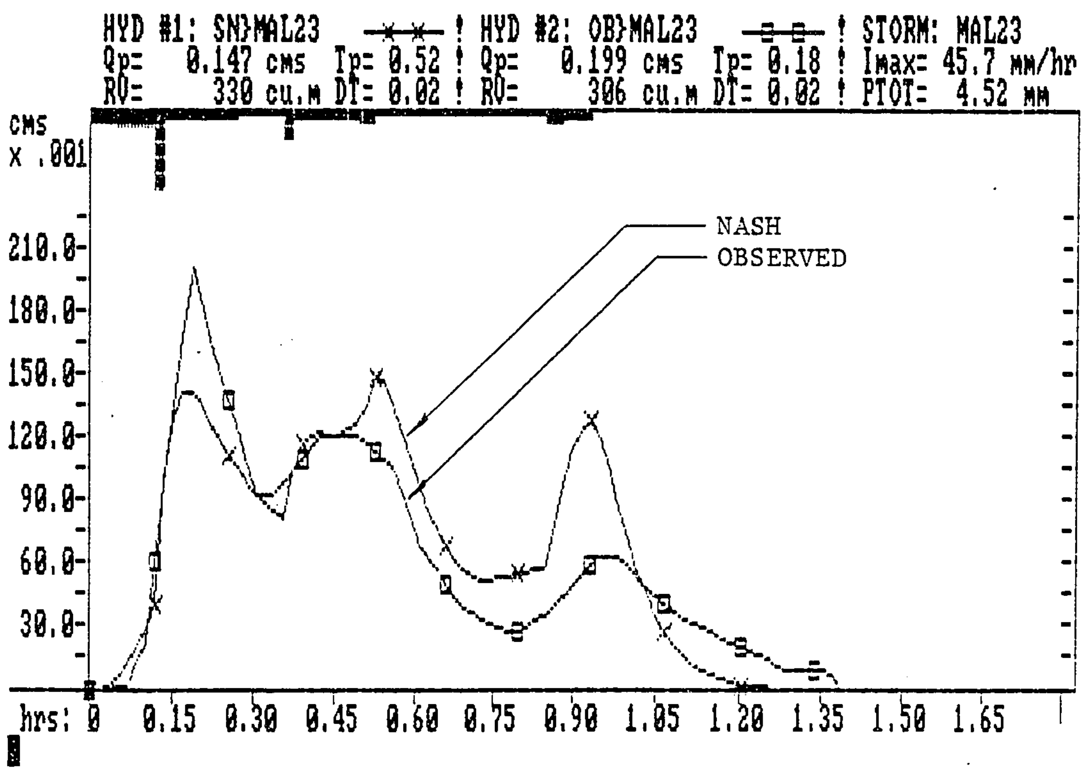
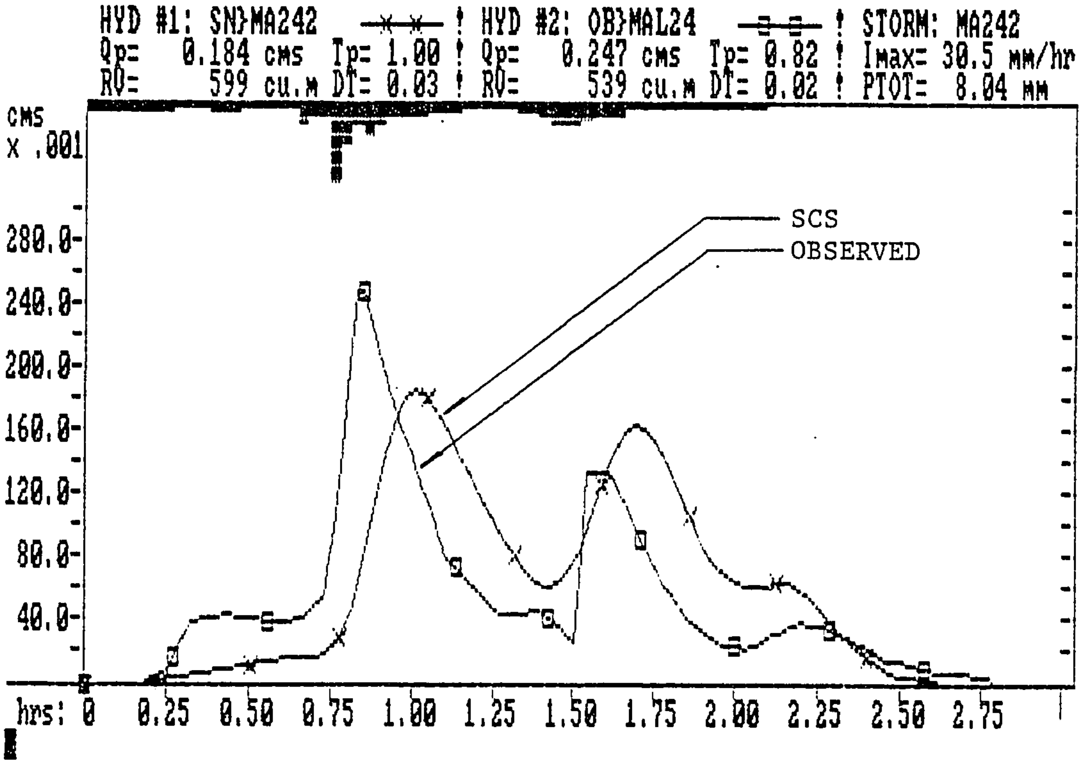


FIG. 10I MALVERN EVENT 24



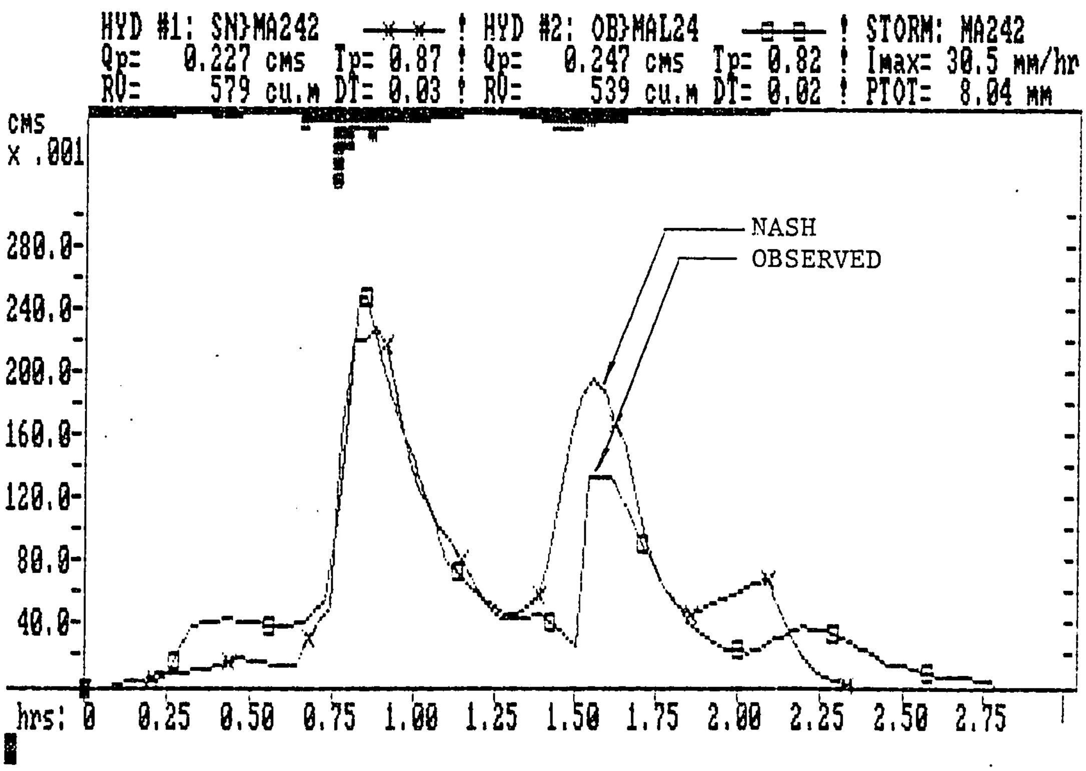


FIG. 11A — MALVERN PEAK FLOWS

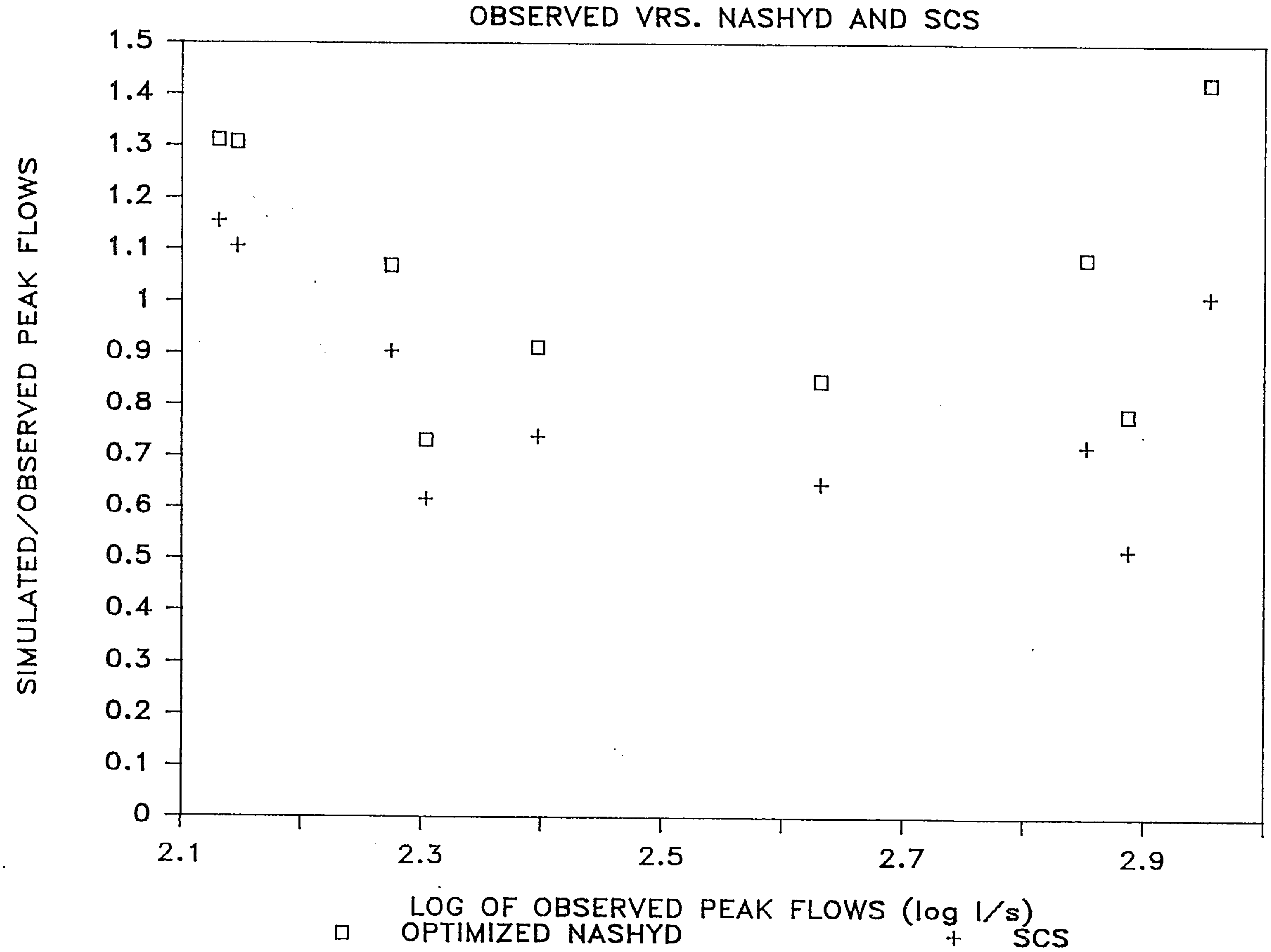


FIG. 11B — MALVERN: FIRST PEAKS

OBSERVED VRS. NASHYD AND SCS

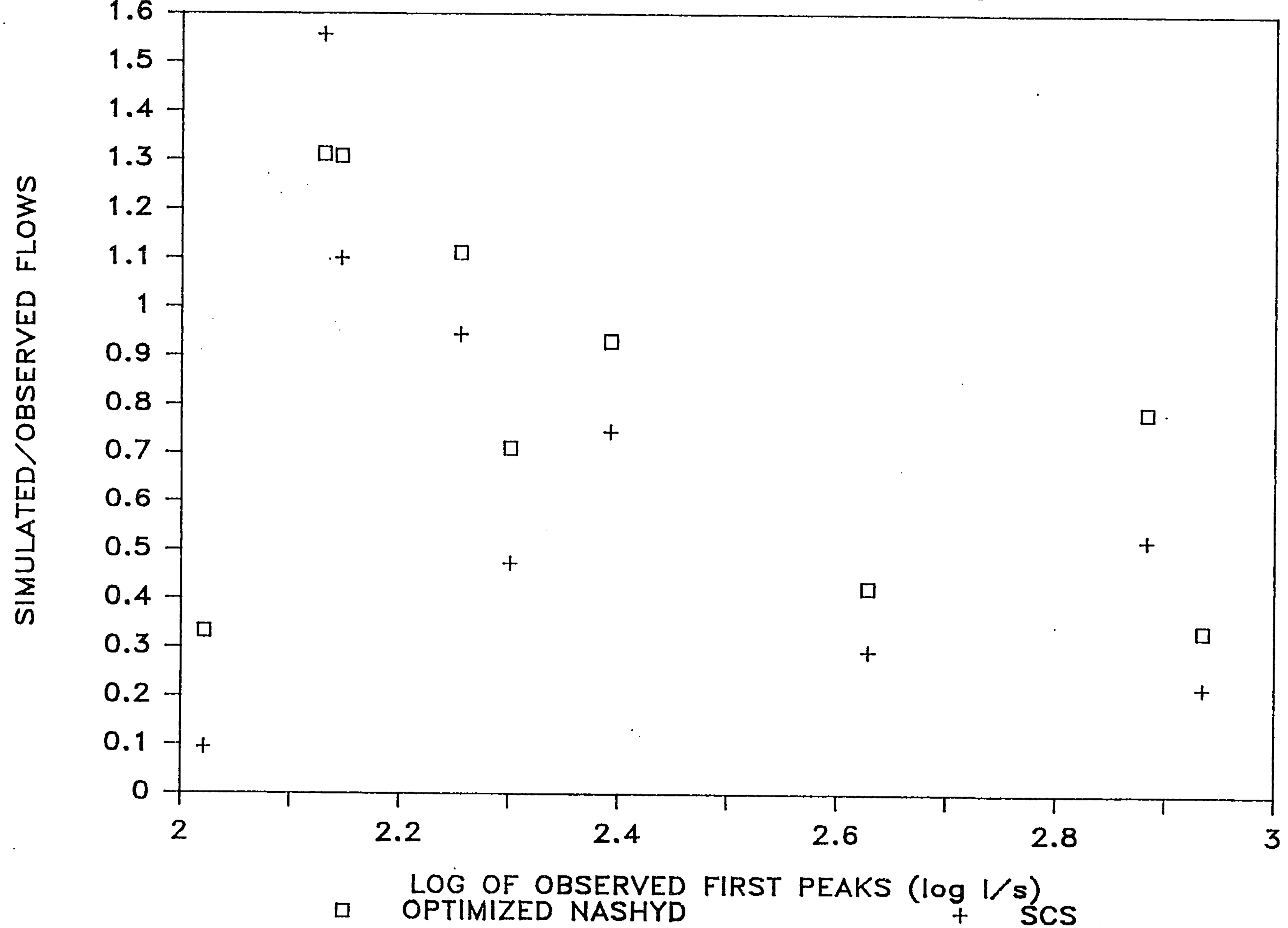


FIG. 12 – MALVERN TIME TO PEAKS

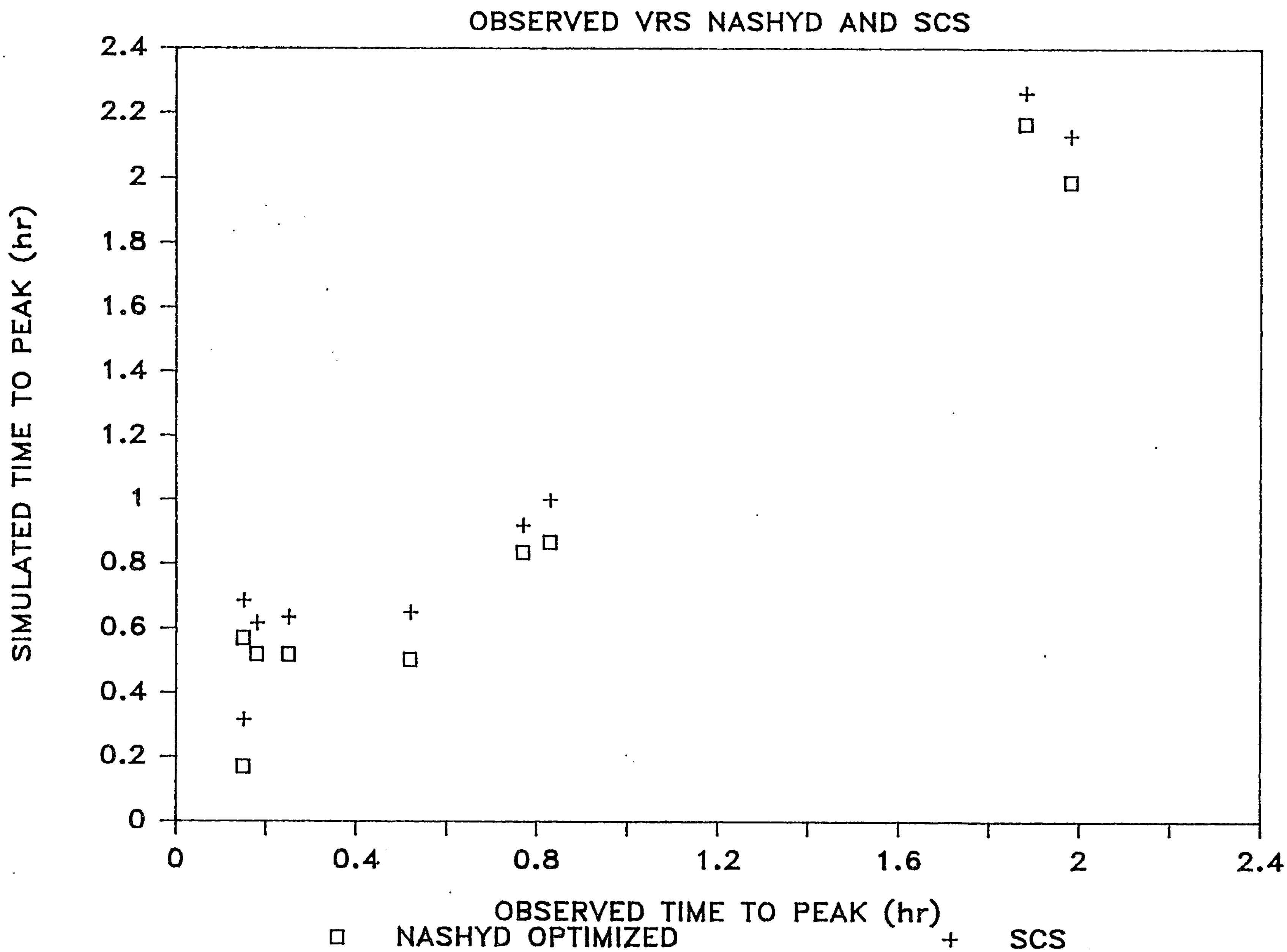


FIG. 13 – PEAK FLOW ERROR

(DOF: NASHYD 3, SCS 4)

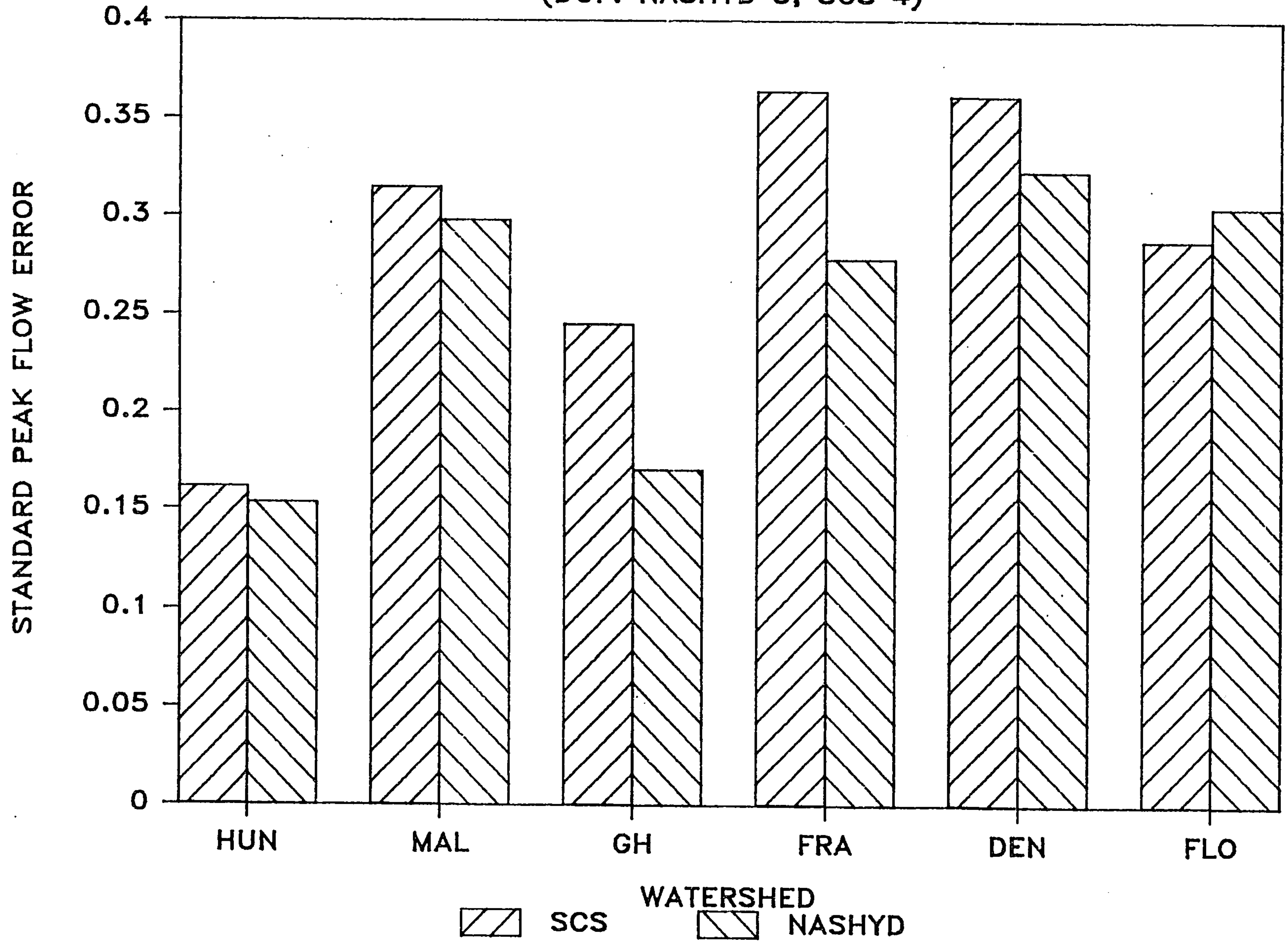
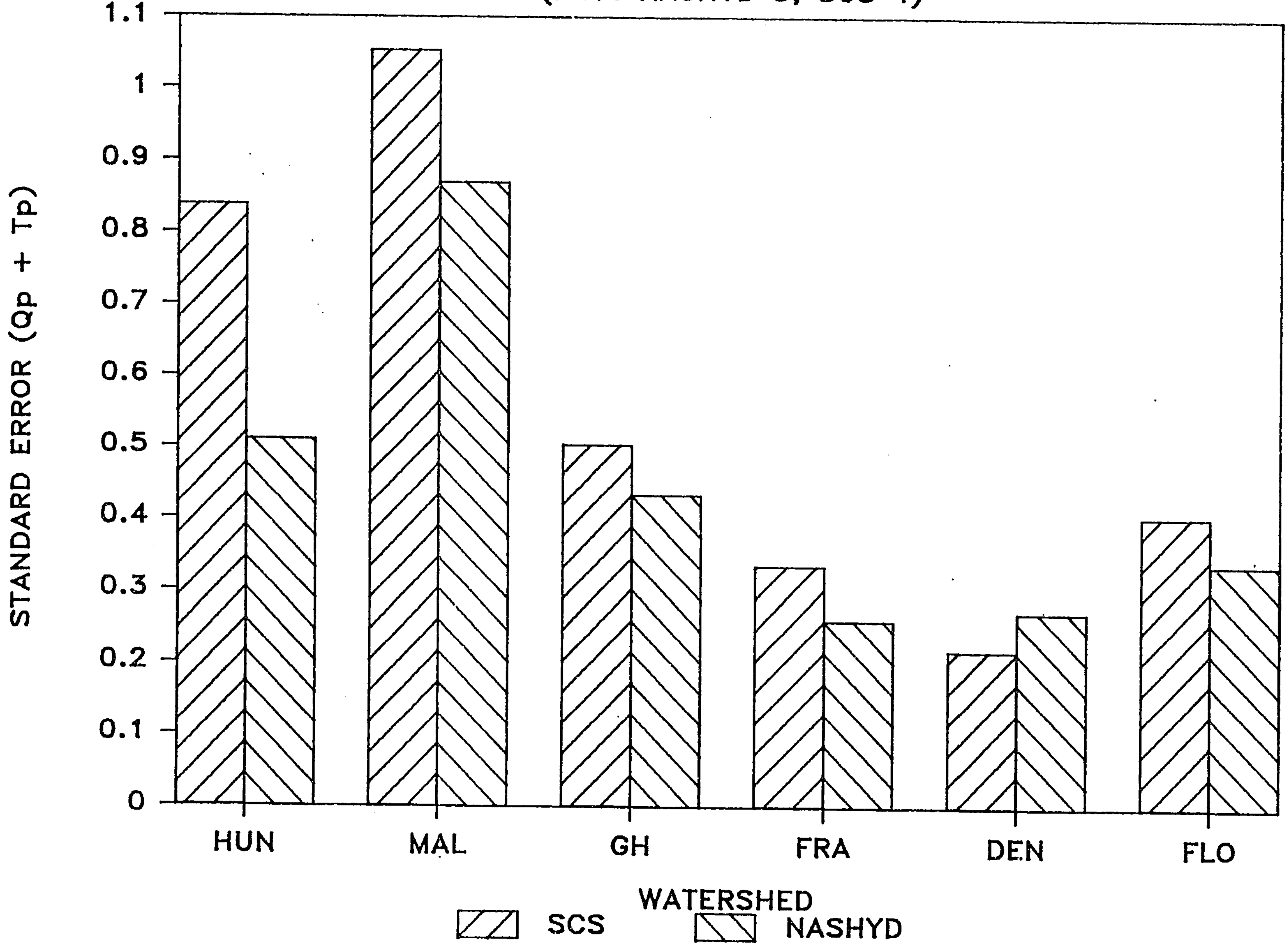


FIG. 14 — ERROR FUNCTION (Eqn 8)

(DOF: NASHYD 3, SCS 4)



having both parameters free for calibration, is relegated to only 3 degrees of freedom.

Standard errors for an equally-weighted combination of peak flow errors and errors in time to peak in equation (6) are plotted in Figure 14. In this case, the coefficients *a* and *c* of equation (6) are both set to 0.5:

$$Std\ Err = 0.5 * \sqrt{\frac{1}{n-1-p} \sum \frac{(Qpobs-Qps)^2}{Q_{pobs}}} + 0.5 * \sqrt{\frac{1}{n-1-p} \sum \frac{(tpobs-tps)^2}{t_{pobs}}} \qquad (8)$$

In general, optimized Nash parameters represent a marginal improvement in the simulation of observed hydrographs. Inspection of the hydrographs in Appendix A indicates that calibrated Nash parameters do not consistently improve results over the SCS unit hydrograph. By observing the SCS and optimized Nash unit hydrographs for each of the six catchments (Figure 15A to 15F) results can be better understood. For all but Malvern and Gray Haven, the unit hydrograph peak flows for both models are approximately equal. This similarity in peaks is possible because, as discussed earlier in the sensitivity analysis, various combinations of N and t_p can generate the same unit hydrograph peak. It is significant to note that the SCS unit hydrograph gave its worst performance for the Malvern catchment. On the other hand, Gray Haven results for the optimized Nash model were only marginally better than SCS.

FIG. 15A UNIT HYDROGRAPHS FOR MALVERN

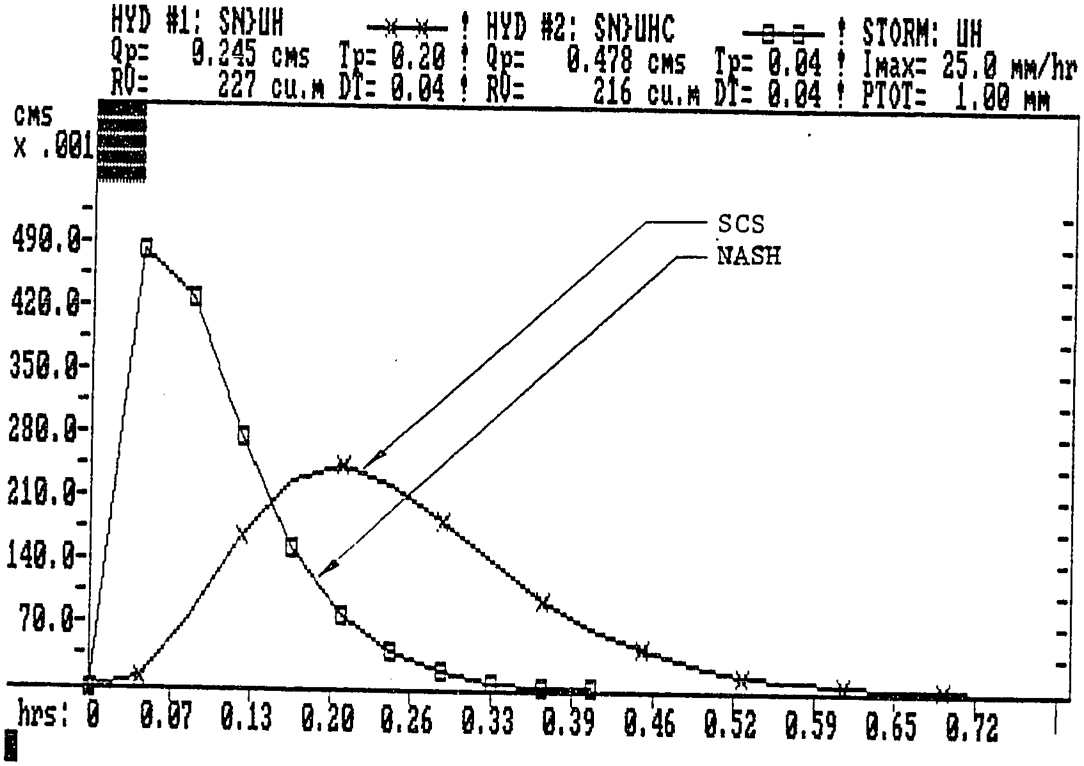


FIG. 15B UNIT HYDROGRAPHS FOR KINGS CREEK

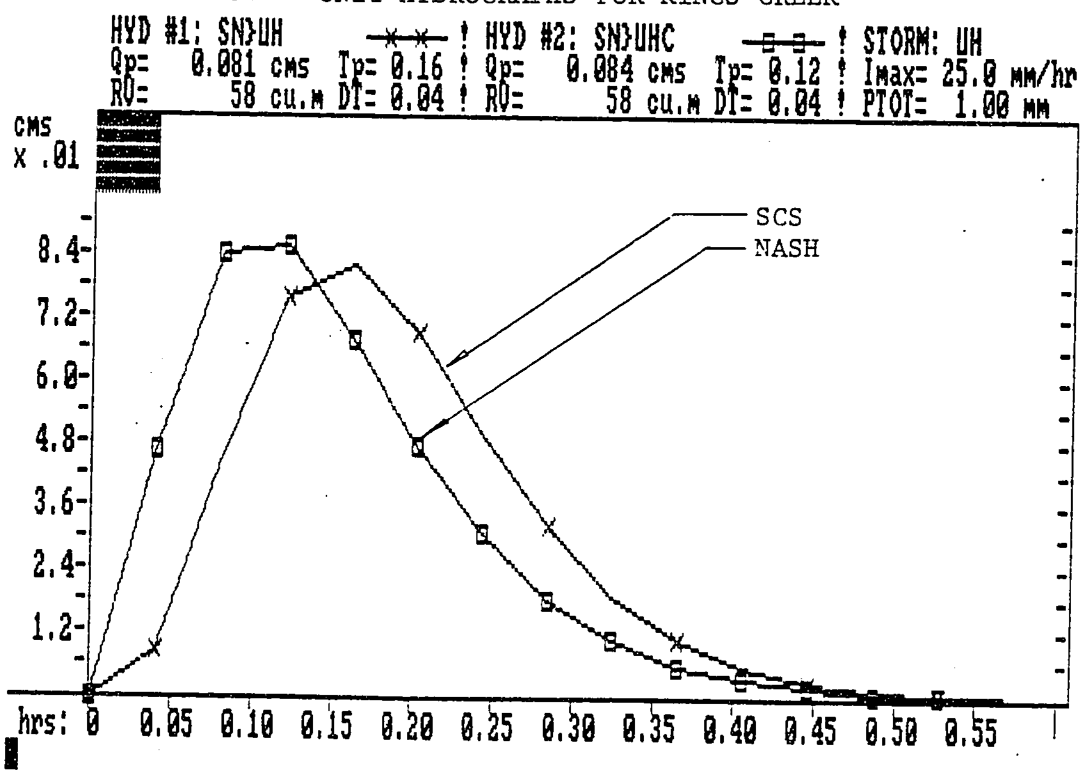


FIG. 15C UNIT HYDROGRAPHS FOR GRAY HAVEN

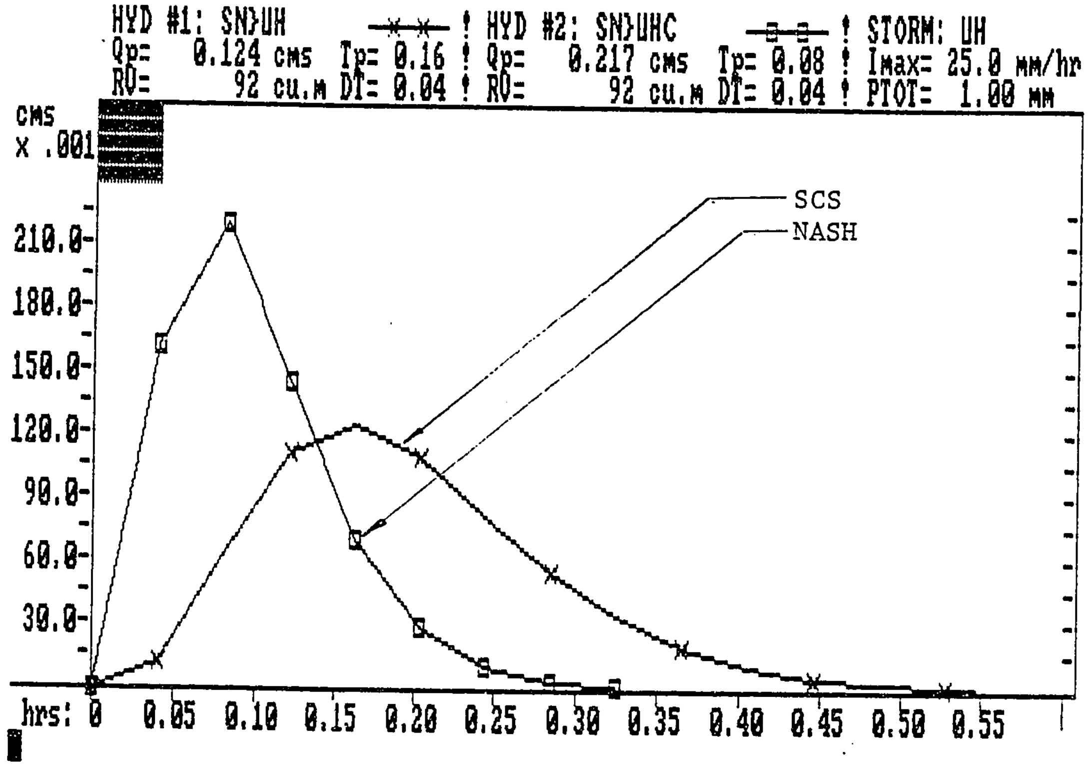


FIG. 15D UNIT HYDROGRAPHS FOR DENMARK

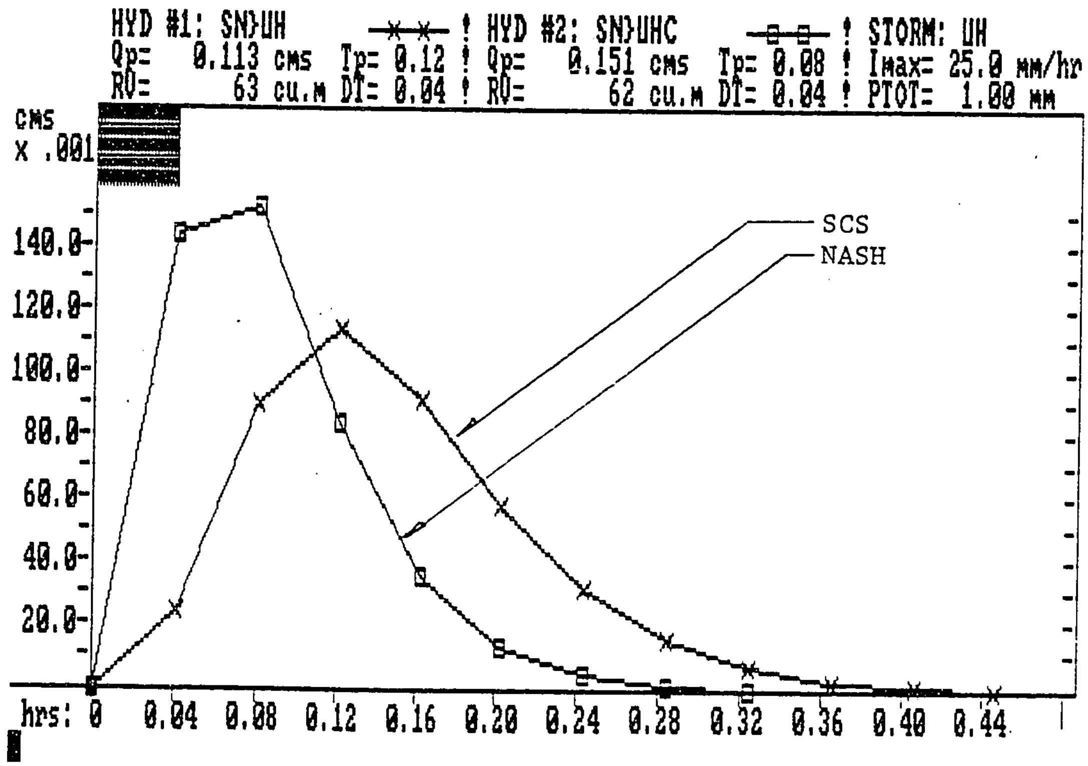


FIG. 15E UNIT HYDROGRAPHS FOR FRANCE

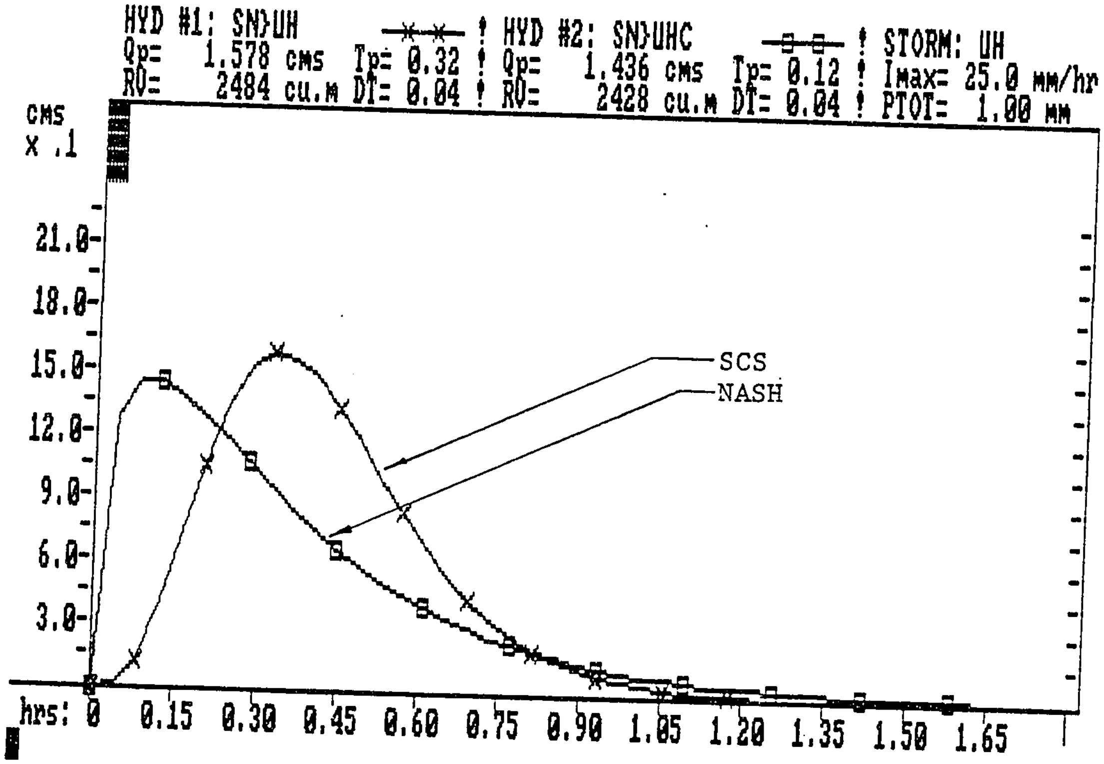
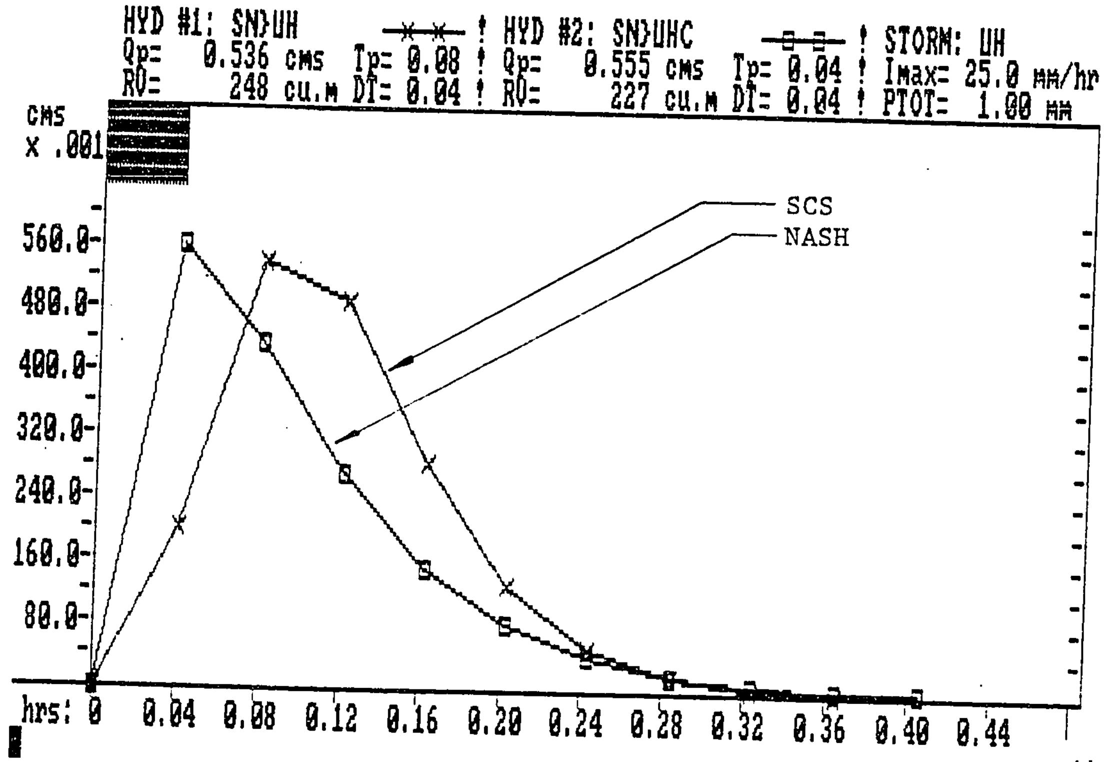


FIG. 15F UNIT HYDROGRAPHS FOR HUNGARY



6.3 Comparison With URBHYD

For comparison URBHYD simulations were generated for two of the six catchments; Malvern in Canada and St. Denis in France. Measured event runoff volumes were matched by varying the imperviousness ratio, yet simulations were still largely conducted with default parameters.

URBHYD represented an improved simulation of observed hydrographs, not only over SCS, but also over the optimized Nash model, as exemplified in Figures 16A to 16F for the Malvern watershed. Results for the French watershed are found in Appendix B. Additional simulation results with URBHYD also performed well (Wisner and Consuegra, 1986).

6.4 Results With SCS Design Storms

The SCS method was developed for use with larger rainfalls, specifically, to be applied with SCS design storms. In light of this, the SCS model was compared with the Nash model with optimized N and t_p for design storm conditions. SCS type II design storms were generated for rainfall depths defined for Toronto 5-year and 100-year return periods. Peak discharge rates, peak ratios and runoff coefficients for three catchments are presented in Table 6.

FIG. 16A - MALVERN EVENT 2

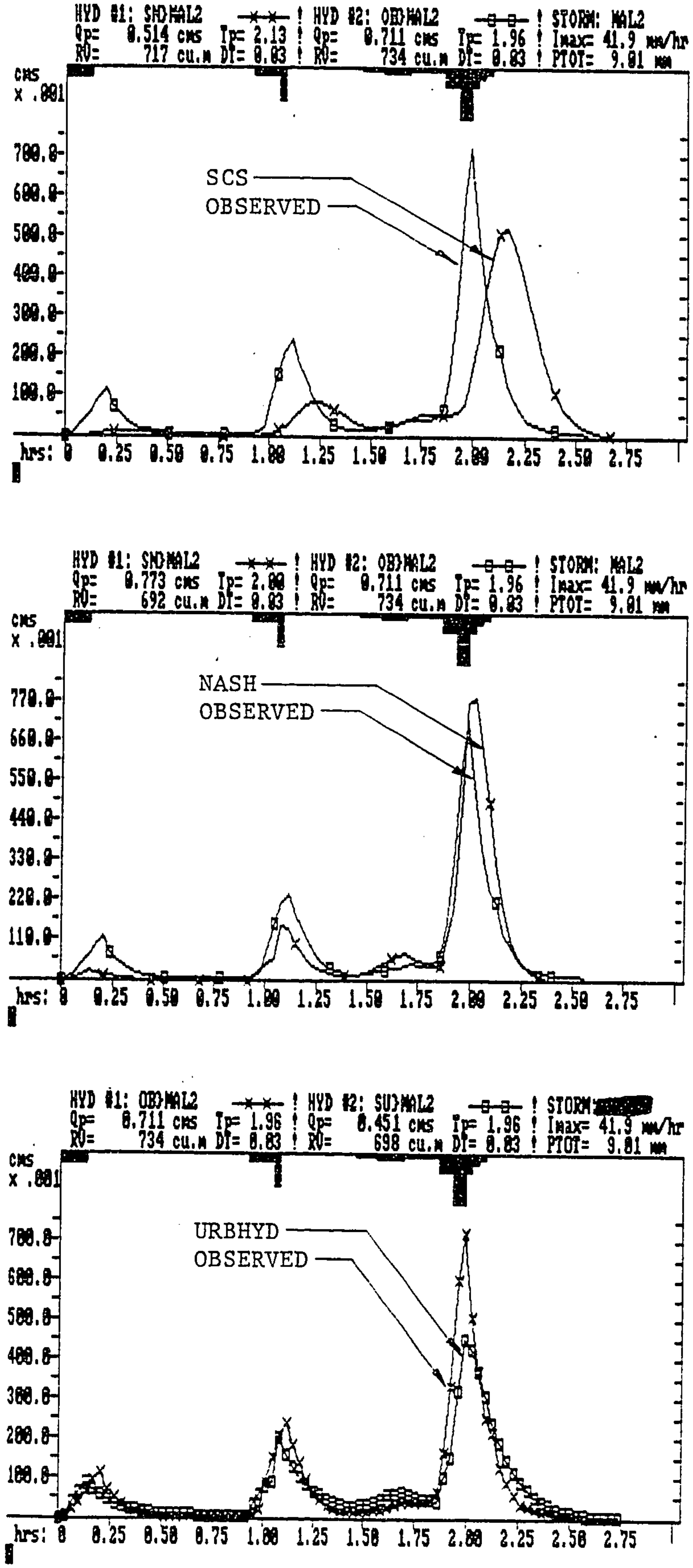


FIG. 16B - MALVERN EVENT 8

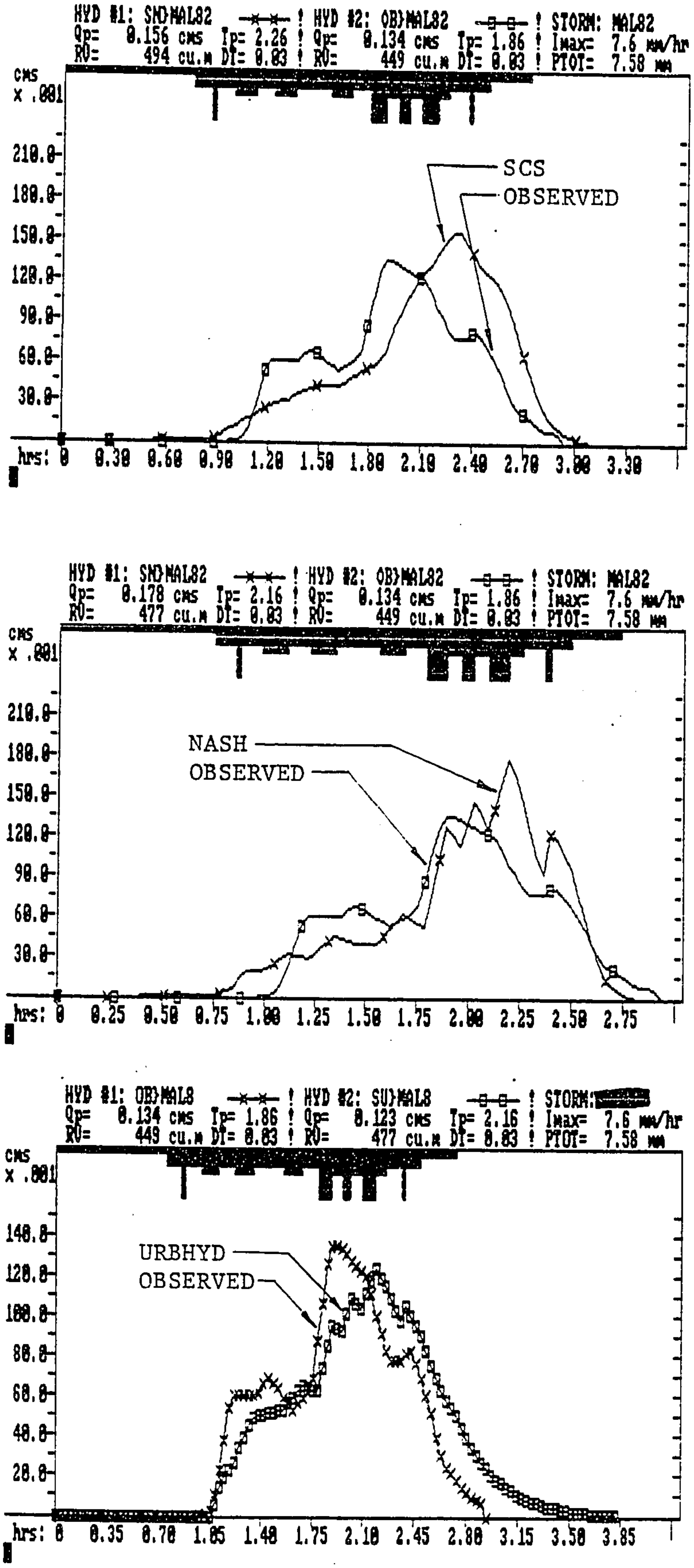


FIG. 16C - MALVERN EVENT 12

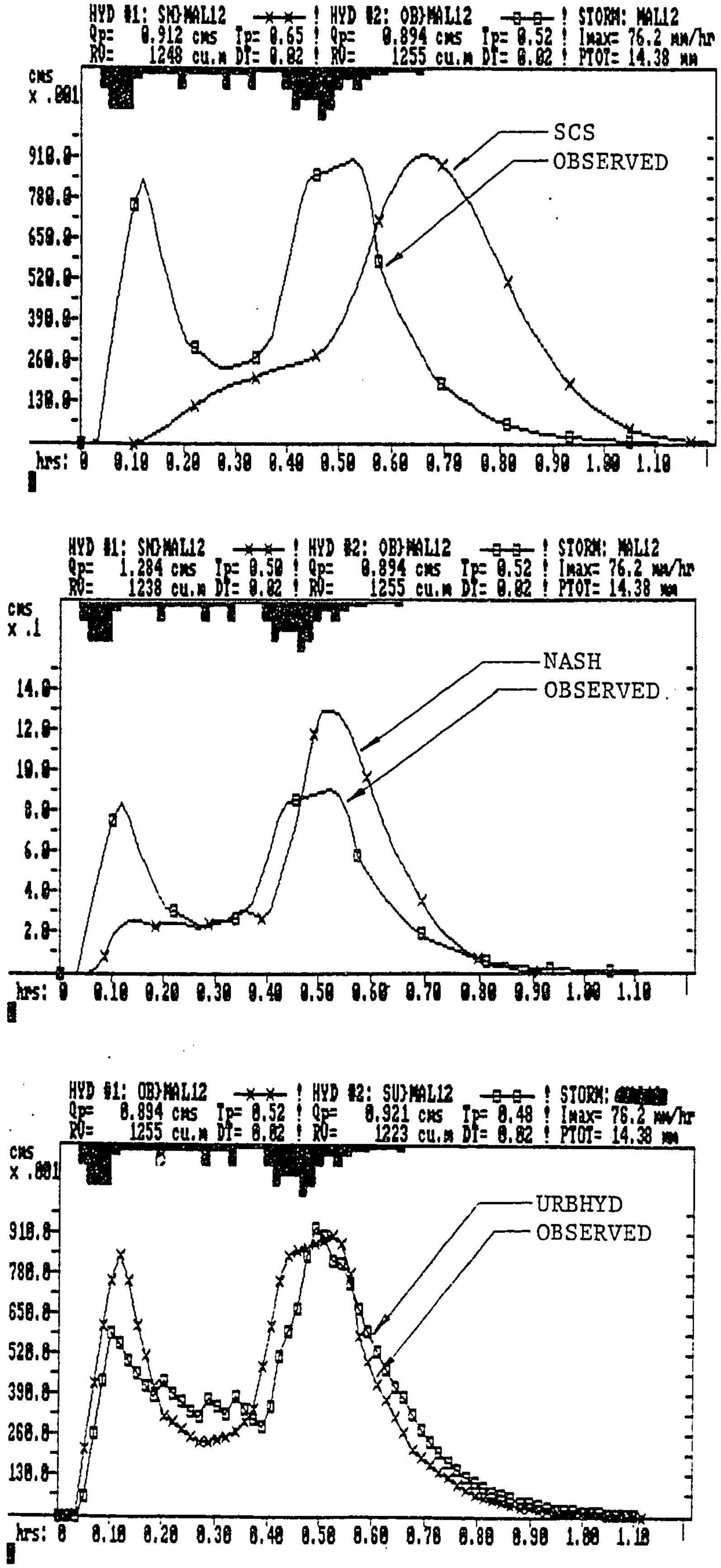


FIG. 16D - MALVERN EVENT 16

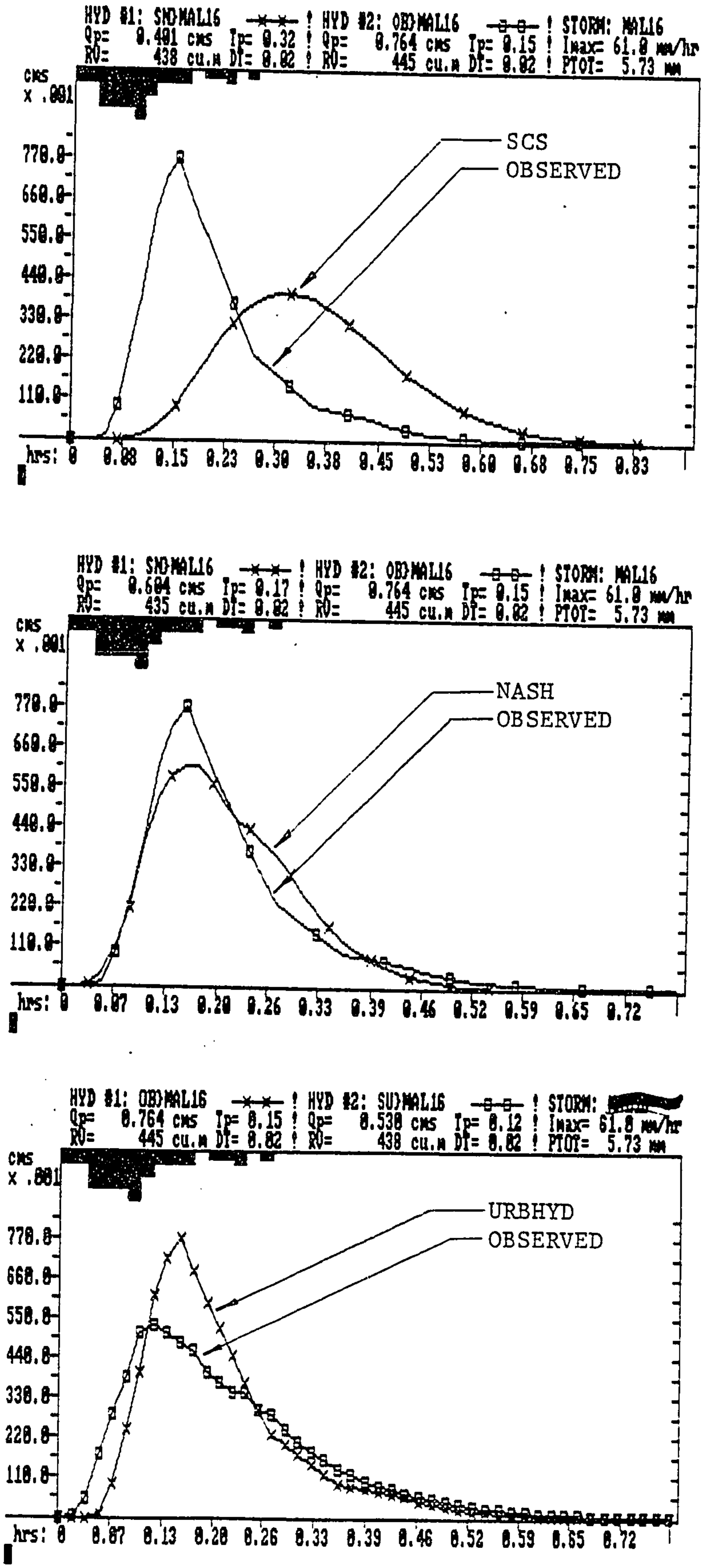
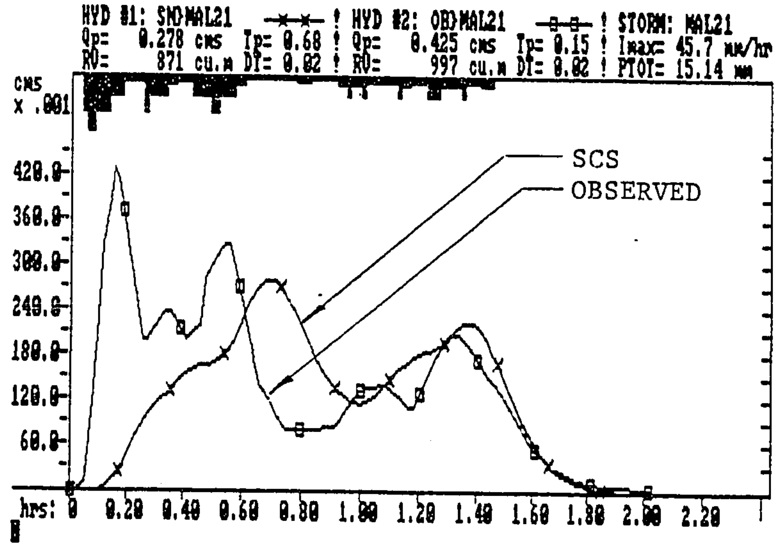
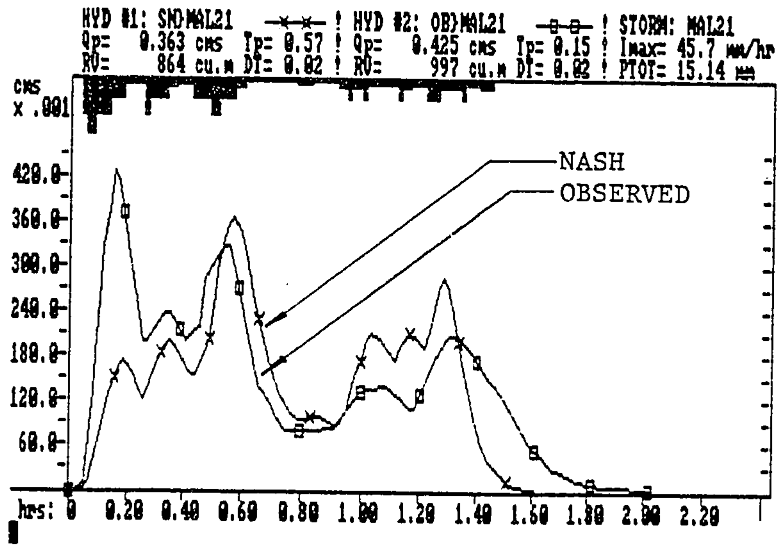


FIG. 16E - MALVERN EVENT 21





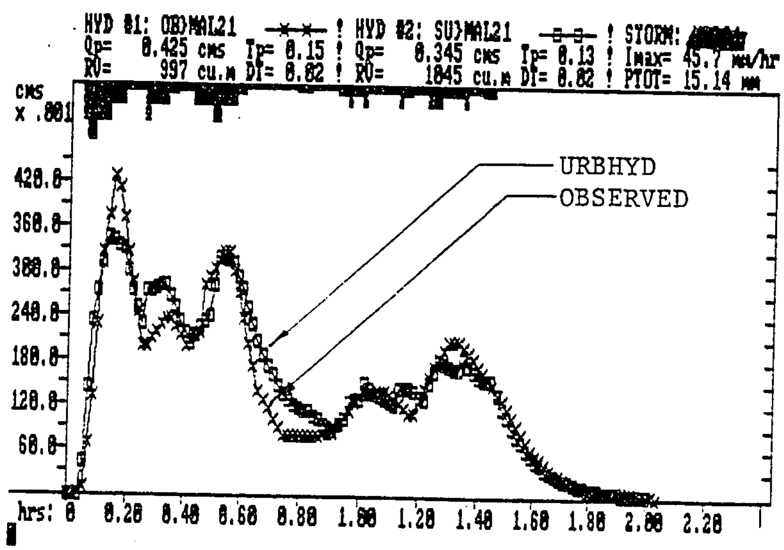
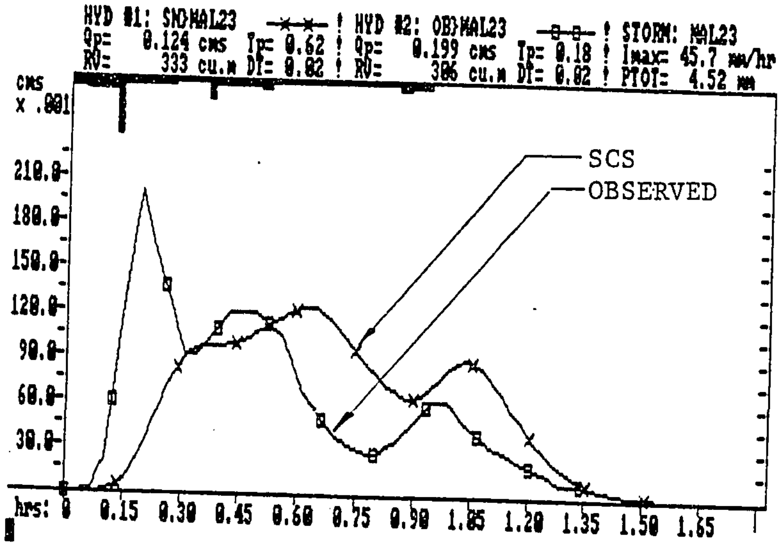
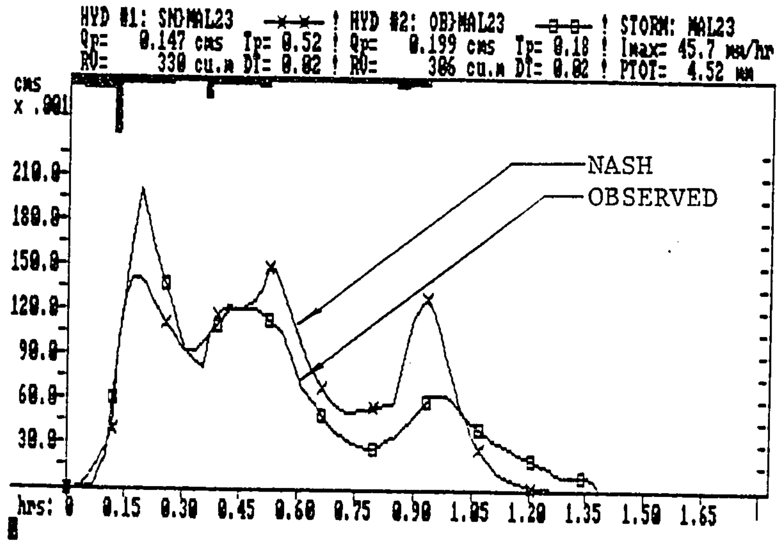
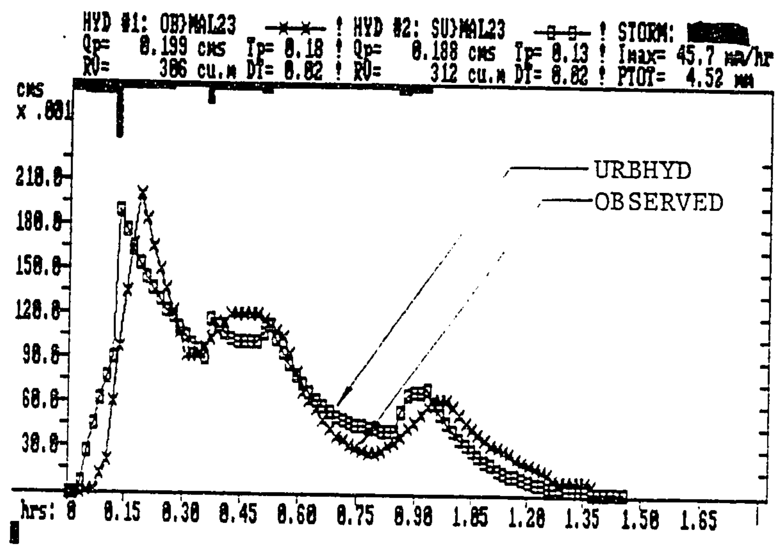


FIG. 16F - MALVERN EVENT 23







When the lag method is used to compute the time to peak for the SCS unit hydrograph, peak flows are considerably lower than with other methods. However, with the more accurate velocity method, the SCS unit hydrograph generated peak flows that were generally within 10% of the Nash unit hydrograph results demonstrating that for very large storms optimization of the Nash model may be unnecessary.

In the same table, results are also compared with the quasi-linear model, URBHYD (Wisner, 1982) - also part of the MINI-OTTHYMO package. Runoff volumes are the same for the sake of comparison. Simulations done with the URBHYD model were acceptable using default values making calibration, other than matching of observed runoff volume, unnecessary. URBHYD peak flows are lower than Nash and SCS results. By comparison SCS results were very conservative for St. Denis, France and for the 5-year storm at Malvern.

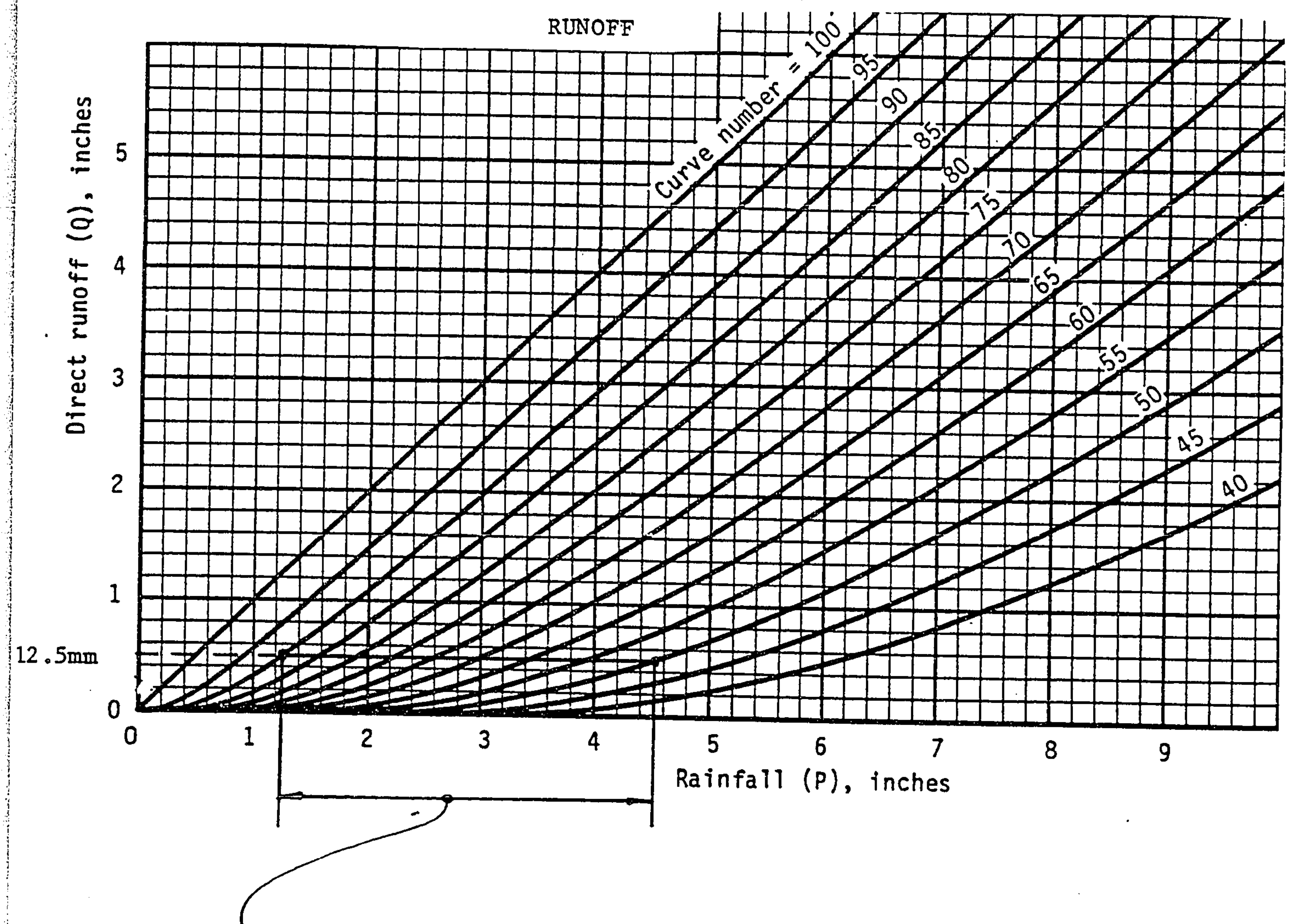
7. DISCUSSION

Prior to other remarks, it is suggested that caution be exercised in applying curve numbers for Canadian conditions. Figure 17 illustrates the range of total precipitation necessary to generate the threshold runoff (12.5 mm) for a range of curve numbers specified in SCS tables for typical urban land-use scenarios. A substantial portion of this range of required rainfall depths is larger than the 5 and 10-year storms used for the design of urban drainage systems in many parts of Canada.

The SCS unit hydrograph model may give acceptable results for real storm input on certain watersheds, even when the total runoff is less than the 12.5 mm limit, provided appropriate selection of I_a and CN is made and provided the velocity method is used to estimate the time to peak. Acceptable results were obtained for the Hungarian, Danish and French watersheds, all of which had one or more rainfall events which were under the 12.5 mm limit.

However, caution must still be observed. Model results must be compared with recorded hydrograph data for the specific basin and specific range of anticipated runoff volumes prior to design applications. As a preliminary recommendation, users may select an initial abstraction of 1 mm and calculate the curve number by equations (1) and (3) for an

FIGURE 17 - TOTAL RAINFALL REQUIRED TO PRODUCE 12.5 mm RUNOFF



Total rainfall depths required to produce 12.5 mm (0.5 in) runoff for various CN values as recommended in TR-55 (1986).

	IMPERV. (%)	CURVE NUMBERS SOIL GROUP A	B	C	D
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82

average runoff coefficient compatible with the catchment imperviousness ratio (see Figure 1).

The study clearly demonstrates the necessity of testing models for a wide range of conditions. If the SCS model had been tested on only one watershed, such as King's Creek, Florida, it would have provided a very biased, albeit optimistic, assessment and very different conclusions would have been reached. Models may perform adequately for certain conditions, but may have serious limitations for another range of conditions.

The degree of performance improvement possible over the SCS unit hydrograph by optimizing N and t_p in the Nash model is limited. One reason for this limitation is because the unit hydrograph models in this study employed the SCS rainfall-runoff transformation to estimate the rainfall excess distribution. This loss equation creates an initial deficit in runoff supply due to its structure and the initial abstraction requirement. When the peak intensity occurs at the beginning of the rainfall, the effect may be poor simulation of the resulting hydrograph shape. The curve number method is not appropriate for urban runoff simulations because of its inability to reproduce the fast responses typical of urban watersheds. Since the unit hydrograph shape parameters N and t_p work in tandem, fixing N to 4.75 as in the SCS unit hydrograph renders it very difficult to find a value of t_p which will produce a good hydrograph simulation. In other words, the time to peak will tend either to generate a hydrograph which will match the peak discharge but not react quickly enough to simulate early peaks or visa-versa, but not both. From a calibration viewpoint, when N becomes fixed only one parameter, t_p, is left to influence the hydrograph shape and the flexibility to reproduce many and various shapes is greatly reduced.

In contrast, the runoff generating algorithm of URBHYD is superior for replication of early peaks and quick responses.

8. CONCLUSIONS

The following conclusions were drawn from the results of the study:

1) Only limited improvement in hydrograph shape can be achieved by optimizing the Nash unit hydrograph parameters N and t_p for any single catchment and a systematic improvement is not possible for all watersheds. Although a general improvement in hydrograph peak flows and peak times is evidenced by Figures 12 and 13, improvement primarily results from calibration of the Nash model, whereas the SCS unit hydrograph parameters are either fixed or defined by equations. Nevertheless, improvement was generally marginal and inconsistent.

2) The performance of models which lump pervious and impervious areas applied with real storm events is inferior to that of models which generate hydrographs from pervious and impervious areas separately; such as URBHYD or SWMM. Compared with the latter models, the SCS method, applied with SCS design storms and the velocity method, gives results which appear to be more conservative.

3) Because of the widespread application of SCS procedures, more comparisons with measurements and other models are recommended. Previous projects designed using the lag

method of computing the time to peak may need to be reviewed. The use of a single curve number for a watershed and $I_a = 0.2S$ may also lead to substantial errors.

4) In light of the overall results, use of either the SCS or Nash unit hydrograph models for urban runoff simulation requires caution. Specifically, the SCS runoff generating method is not able to replicate the quick responses typically observed in urban watersheds.

ACKNOWLEDGEMENTS

Many of the computer simulations documented were conducted in an International workshop. Messrs. Despotovic, Maza and Gayer participated as representatives of the University of Belgrade, VITUKI (Budapest) and INCYTH-CRA (Mendoza, Argentina) respectively. The author would like to thank other workshop participants Messrs. Paul Pilon and Gaetan Thibault (University of Ottawa) for assistance with computer simulations and Mr. Norman Miller (SCS, Washington) for his useful comments.

REFERENCES

Bielawski, Z., "The Nash Cascade or a Single Linear Reservoir for Modelling Urban Runoff", Proceedings of the Third International Conference on Urban Storm Drainage, Volume 1 - Analysis and Design of Stormwater Systems, Goteborg, Sweden, 1984, pp 203-211.

Bielawski, Z., "The Nash Model Parameters for an Urban Catchment in Poland", Urban Drainage Modelling; Proc. of International Symposium on Comparison of Urban Drainage Models with Real Catchment Data, Pergamon Press, Oxford, 1986.

Consuegra, D. and Wisner, P., "Considerations on the Testing and Validation of a Model for Master Drainage Plans", Urban Drainage Modelling; Proc. of International Symposium on Comparison of Urban Drainage Models with Real Catchment Data, Pergamon Press, Oxford, 1986.

Consuegra, D. and Wisner, P., "Verification of Default Parameters in Urban Hydrologic Models", a paper submitted to the Fourth International Conference on Urban Storm Drainage (1987), Lausanne, Switzerland, 1986.

Espey, Huston and Associates, Inc., "An Evaluation of the Effects of Urbanization on Flood Discharges", A report prepared for The Hydrologic Engineering Center, U.S. Army Corps of Engineers, 1979.

Hjelmfelt, A.T., "Empirical Investigation of Curve Number Technique", Journal of the Hydraulics Division, Vol. 106, HY9, September 1980, pp 1471-1476.

James, W., and Robinson, M., "Continuous Models Essential for Detention Design", Proceedings of the Conference on Stormwater Detention Facilities - Planning Design Operation and Maintenance, ASCE, New York, 1982, pp 163-175.

Johansen, L. and Harremoes, P., "The Use of Historical Storms for Urban Drainage Design", Proceedings of the International Symposium on Urban Storm Runoff, University of Kentucky, 1979, pp 61-70.

Kent, K.M., "Estimating Runoff from Rainfall in Small Watersheds", Presented at Meeting of American Geophysical Union, Los Angeles, California, September 1966.

Maksimovic C., Radojkovic, M., "Urban Drainage Catchments", Pergamon Press, Oxford, 1986.

Marsalek, J., "Synthesized and Historical Storms for Urban Drainage Design", Proceedings of the International Conference on Urban Storm Drainage, London, 1978, pp 87-99.

McCuen, R.H. "A Guide to Hydrologic Analysis Using SCS Methods", Prentice-Hall Inc., Englewood Cliffs, N.J., 1982.

McCuen, R.H., "Conceptual and Empirical Comparison of Methods for Predicting Peak-Runoff Rates", Transportation Research Record 921, National Research Council, National Academy of Sciences, Washington, 1983, pp 32-39.

McCuen, R.H. and Miller, N., "Predicting Hydrologic Effects of Urbanization by Using Soil Conservation Service Methods", Transportation Research Record 922, Transportation Research Board, Washington, D.C., 1983, pp 40-46.

Morel-Seytoux, H.J., "Verdin, J.P. and Woo, D.C., "Methodology and Evaluation of an Extension of the Soil Conservation Service Rainfall-Runoff Approach for Urban Watersheds", International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, 1982, pp 33-37.

Padmanabhan, G. and Delleur, J.W., "An Alternative to the Design Storm Approach for Urban Stormwater Management", International Symposium on Urban Hydrology, Hydraulics and Sediment Control, University of Kentucky, 1982, pp 267-274.

Singh, V.P., "A Note on a Systematic Evaluation of Urban Runoff Models", Proceedings of the International Symposium on Urban Storm Runoff, University of Kentucky, 1979, pp 37-45.Soil Conservation Service, Engineering Division, "Hydrology. National Engineering Handbook, Section 4", Washington, D.C., 1972, 1985.

Soil Conservation Service, Engineering Division, "Technical Release 55. Urban Hydrology for Small Watersheds", Washington, D.C., 1975, 1986.

Tschantz, B.A., "SCS User-Defined Storm Hydrograph (SCS-USH)", Proceedings of the Specialty Conference on Hydraulics and Hydrology in the Small Computer Age, Volume 2, ASCE, New York, 1985, pp 1274-1279.

Urbonas, B., "Reliability of Design Storms in Modelling", Proceedings of the International Symposium on Urban Storm Runoff, University of Kentucky, 1979, pp 27-35.

Wisner, P., "IMPSWM (IMPlementation of StormWater Management) Urban Drainage Modelling Procedures", 2nd Ed., University of Ottawa, 1982.

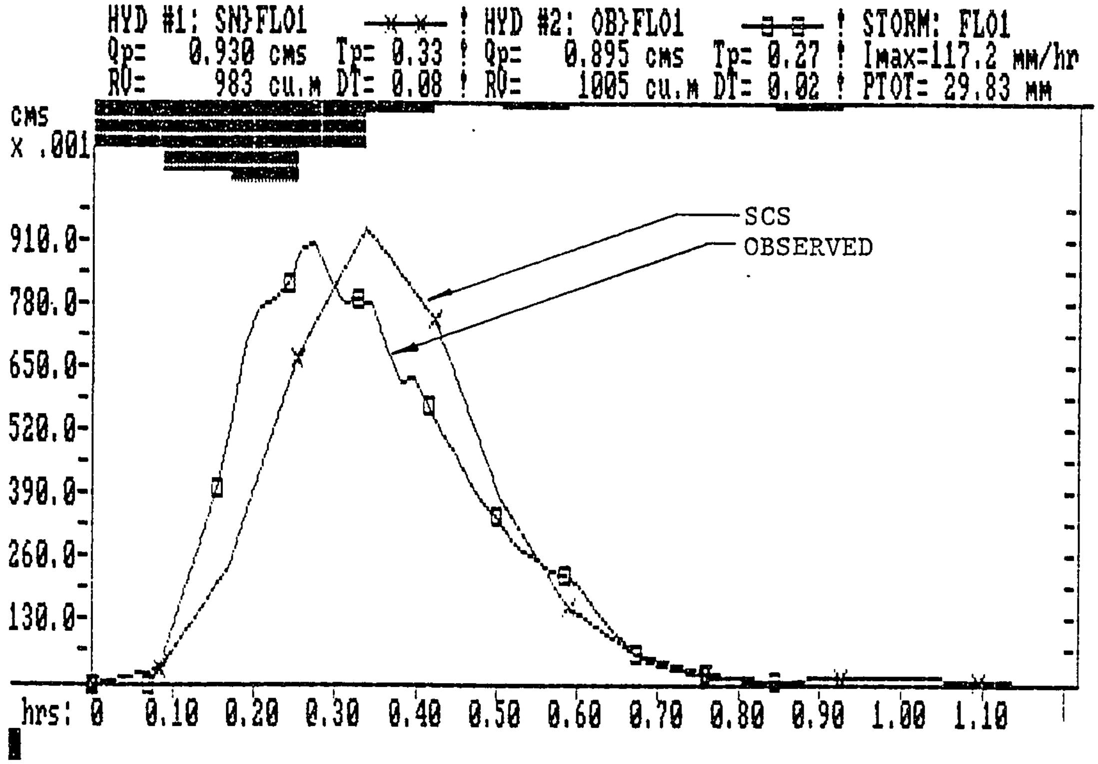
Wisner, P., Consuegra, D., Sabourin, J.F., "MINI-OTTHYMO User Manual (FASTHYMO)", University of Ottawa, 1987.

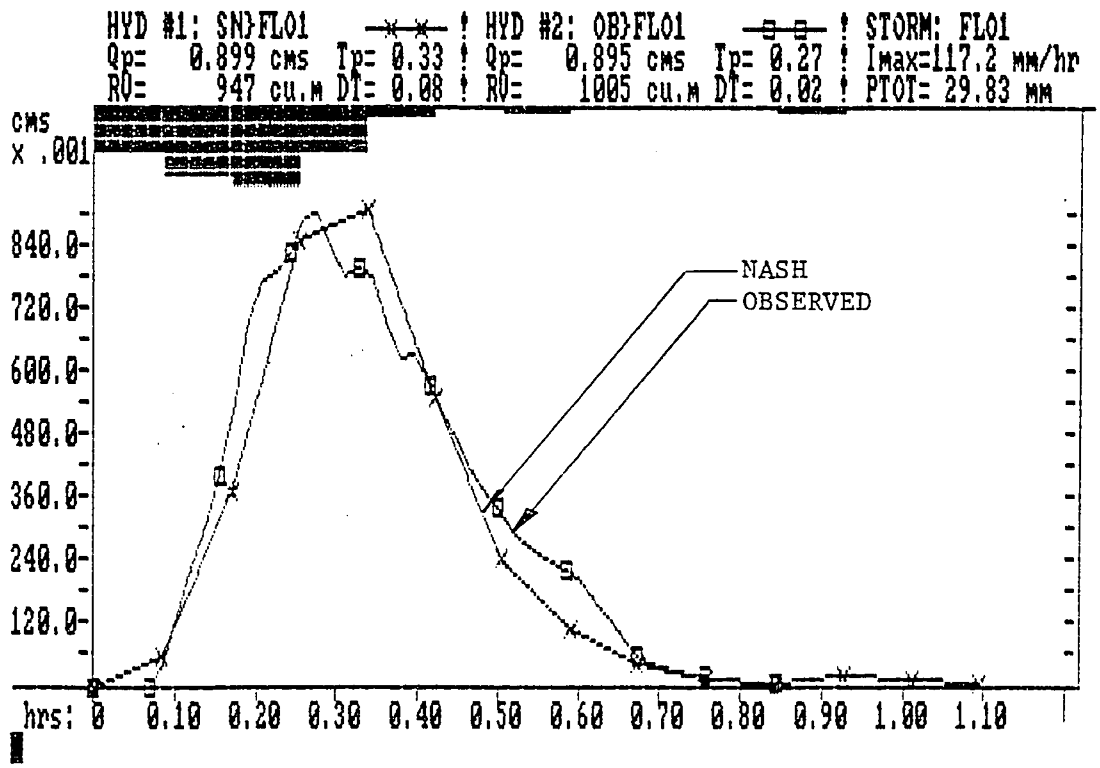
APPENDIX A

SCS and Nash Model Simulated Hydrographs
for:

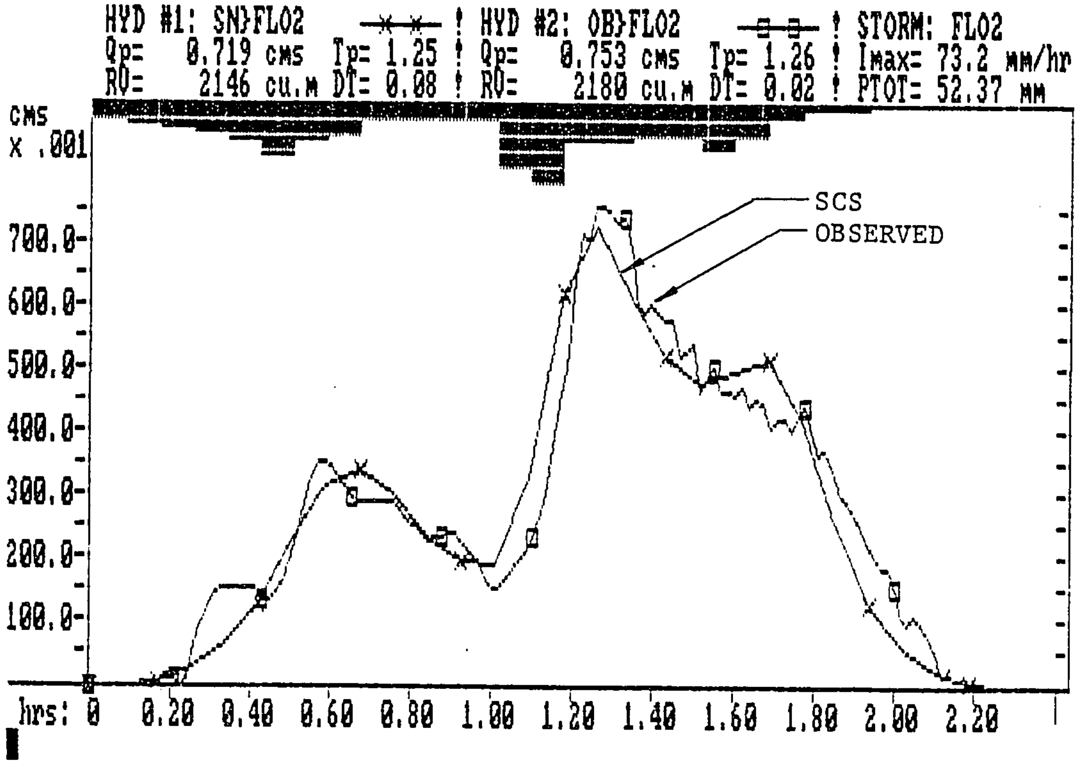
King's Creek, Florida
Gray Haven, Baltimore
St. Denis, Paris
Munkerisparken, Denmark
Miskolc, Hungary

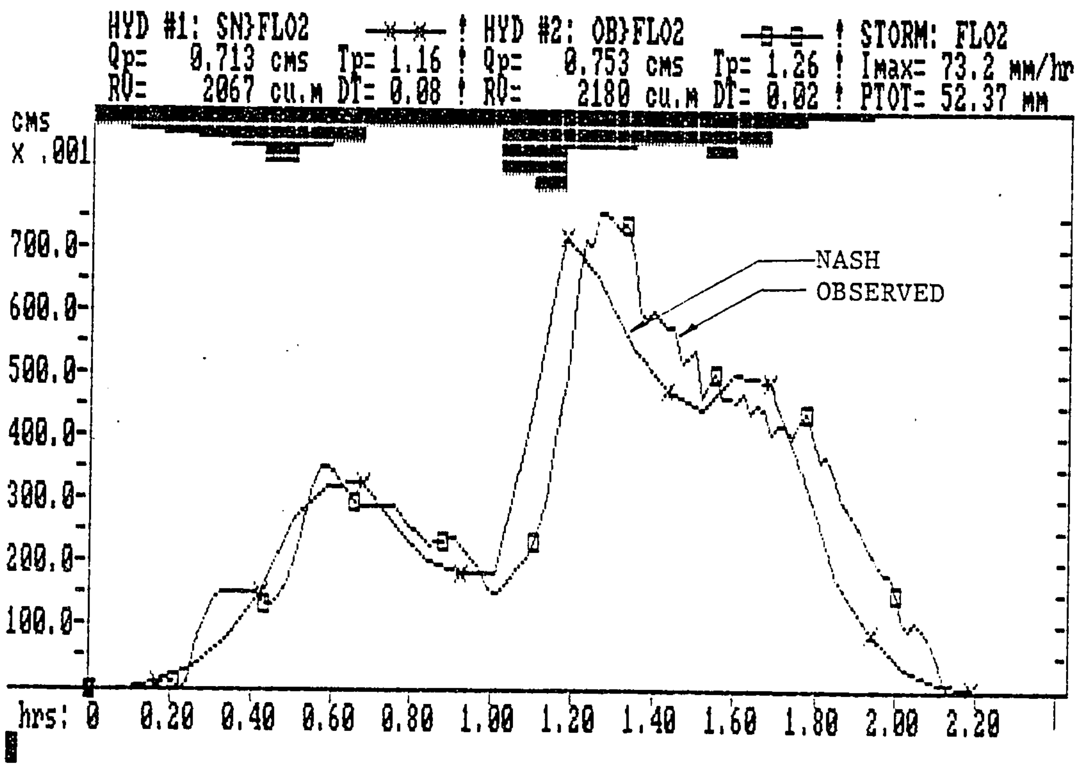
KINGS CREEK EVENT 1



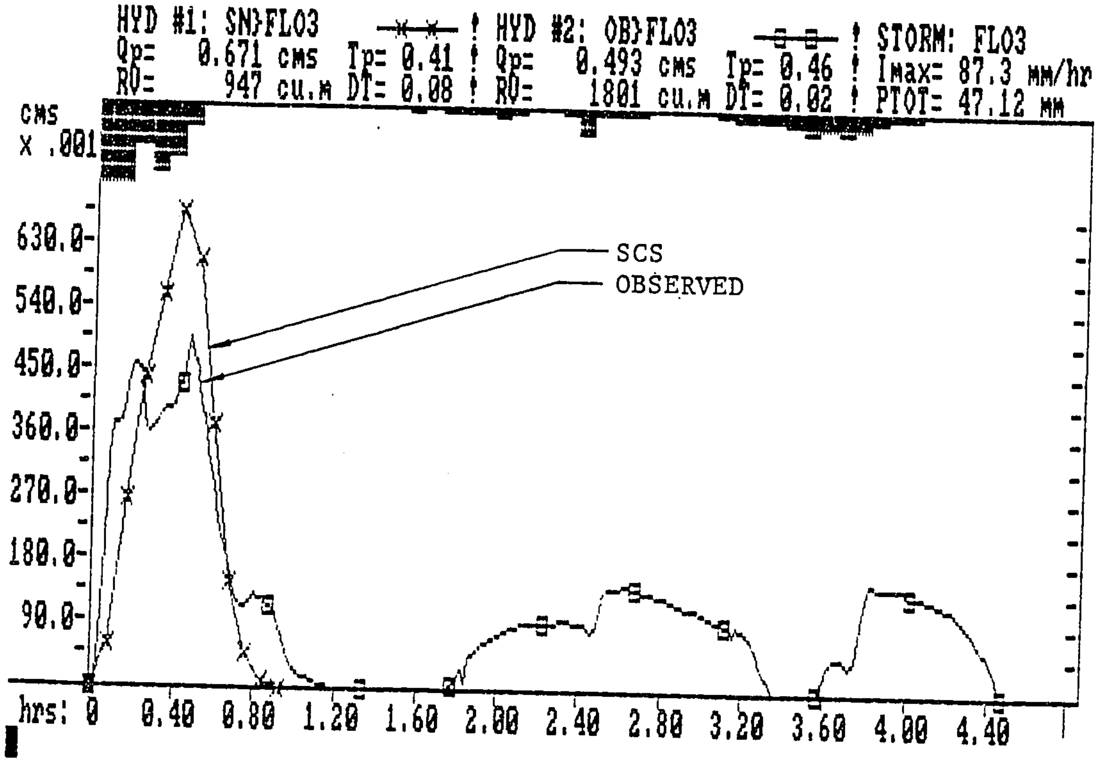


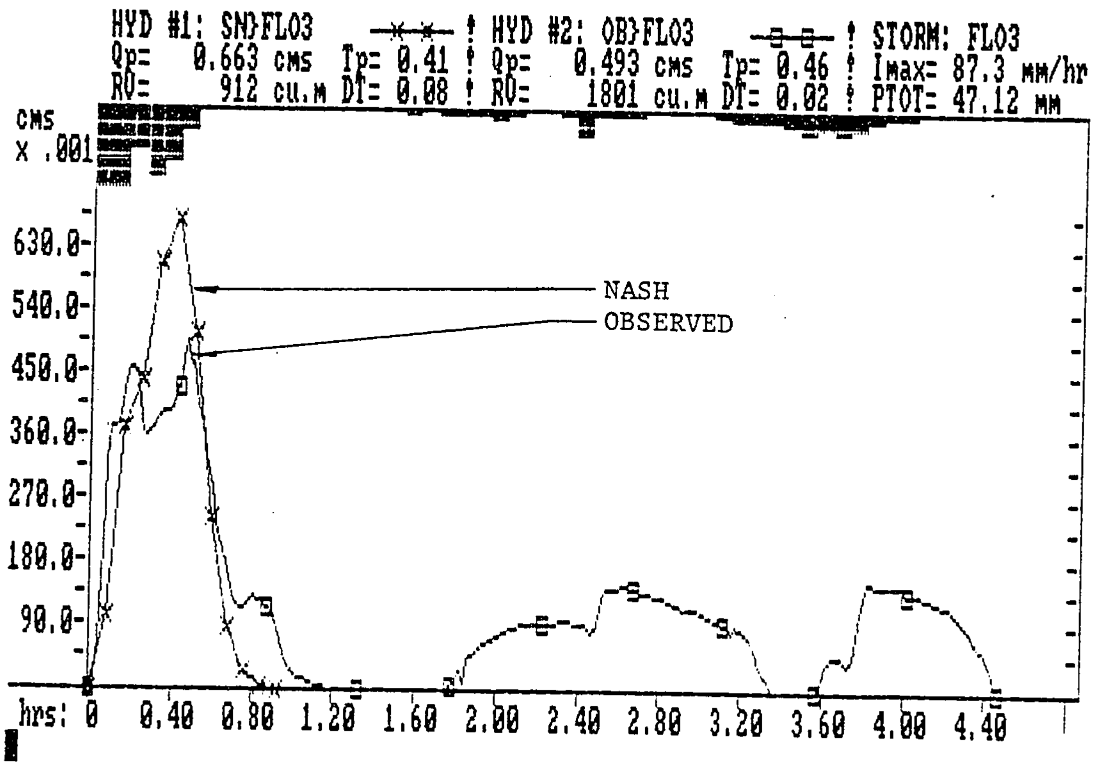
KINGS CREEK EVENT 2



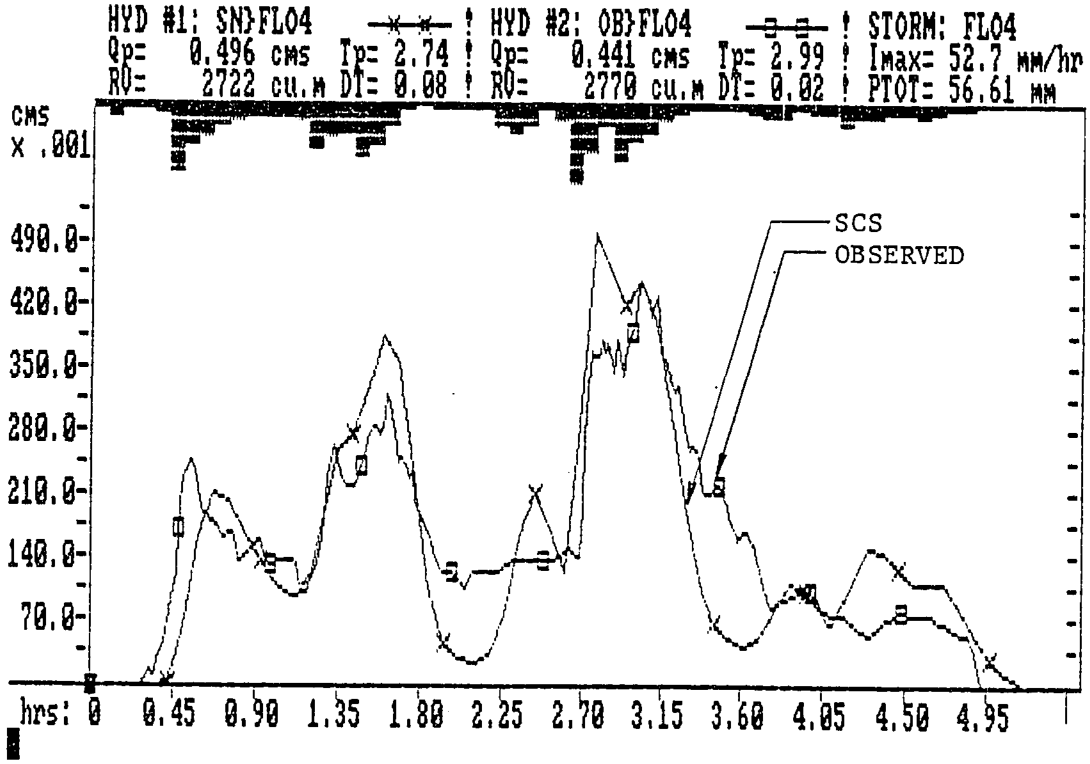


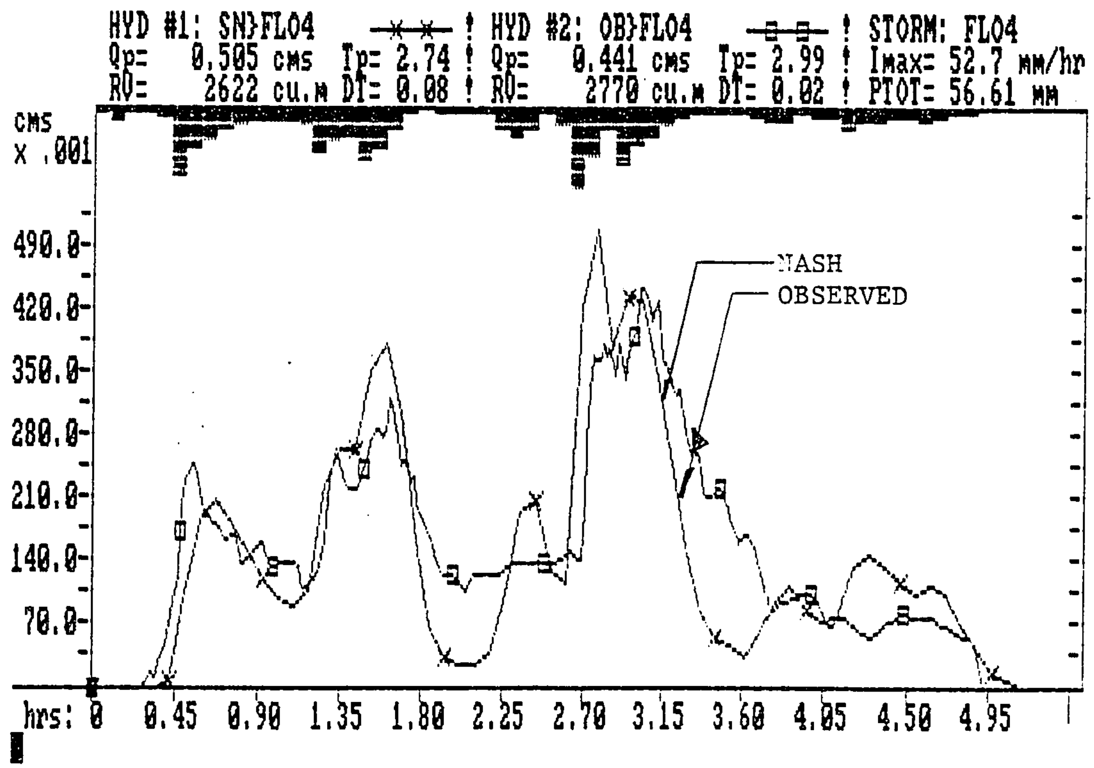
KINGS CREEK EVENT 3



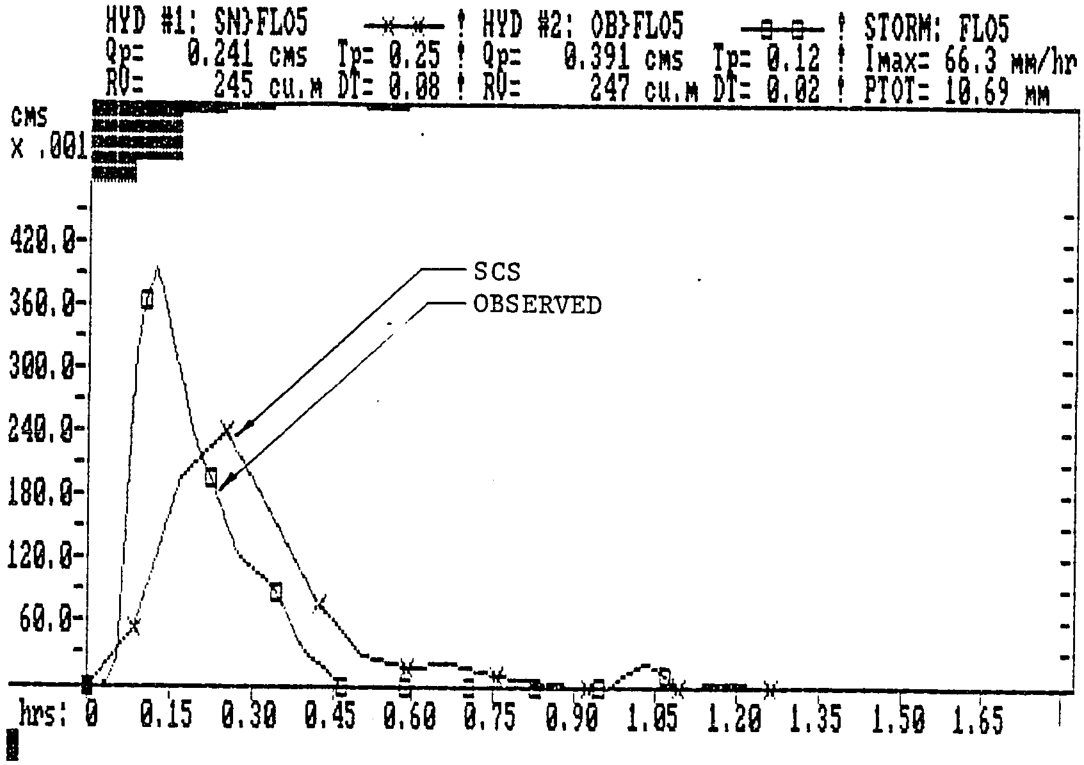


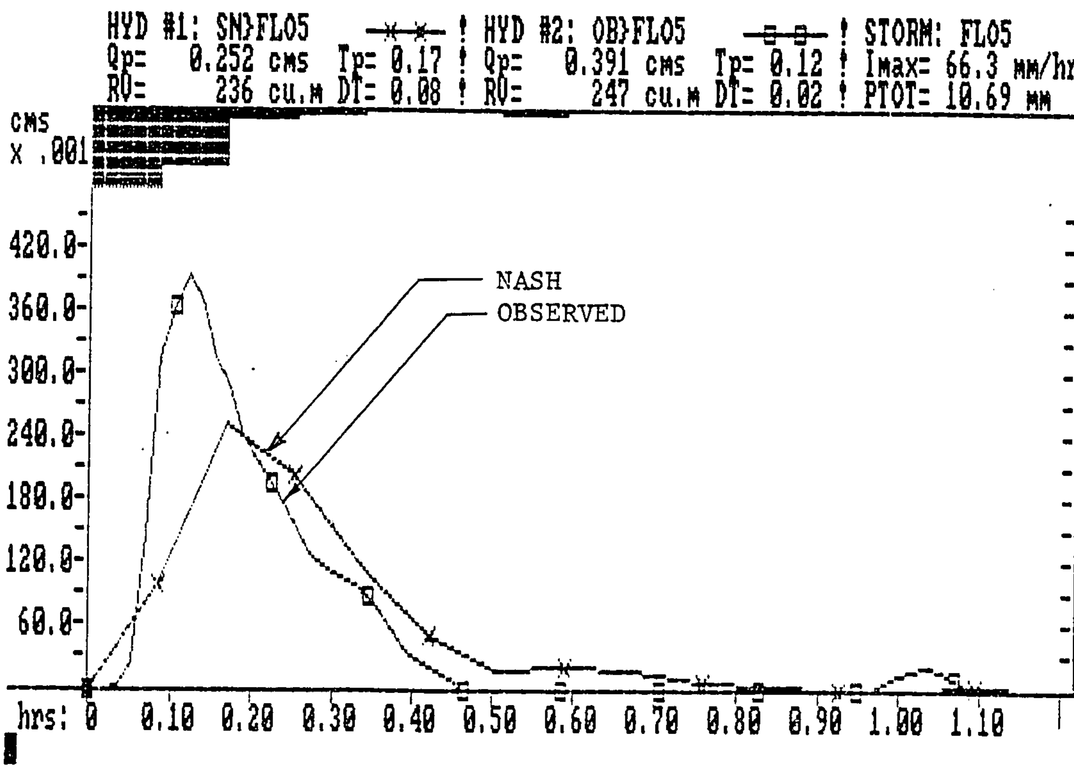
KINGS CREEK EVENT 4



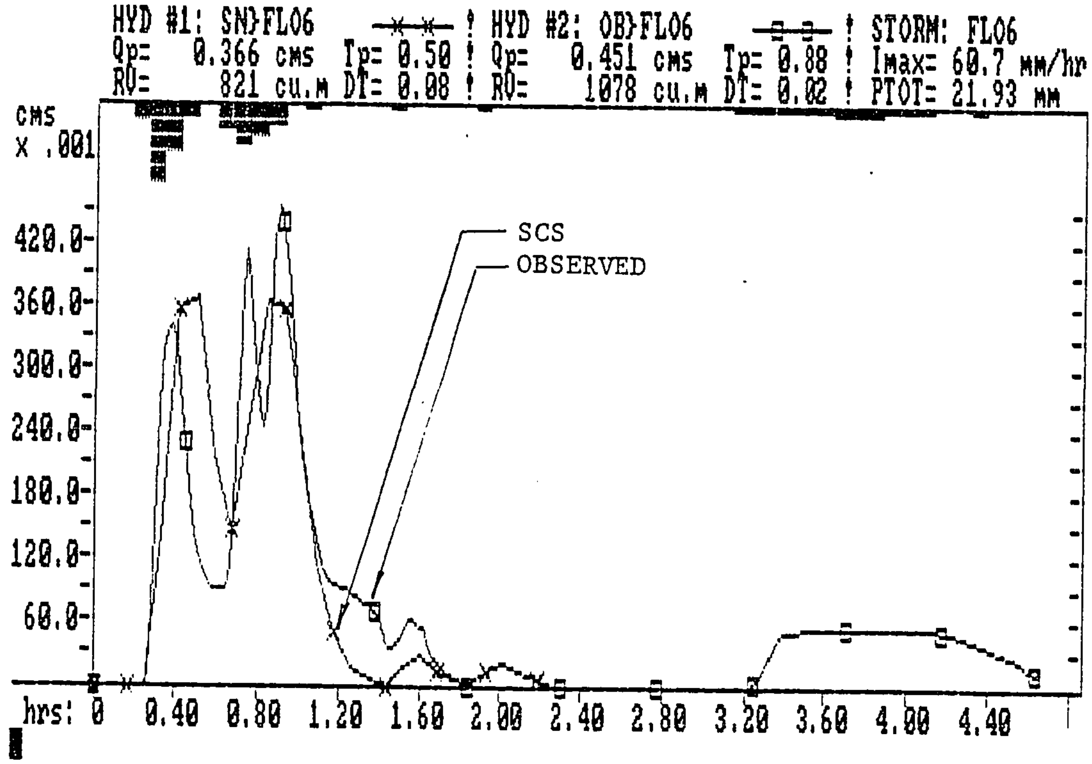


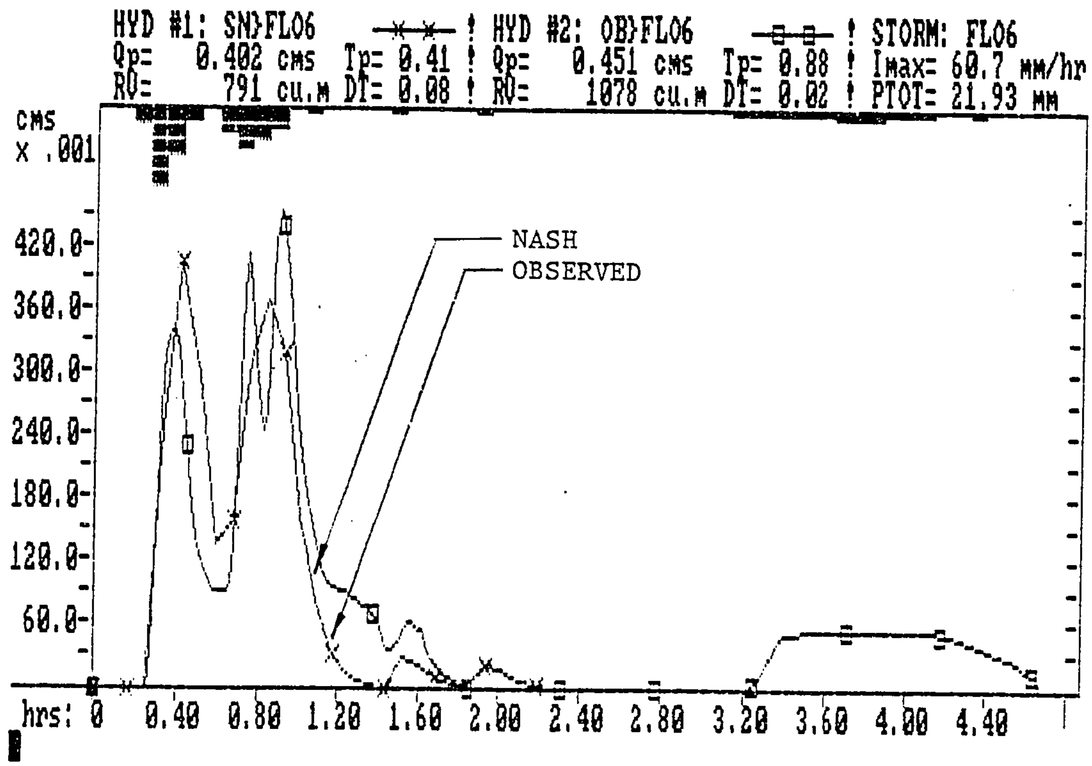
KINGS CREEK EVENT 5



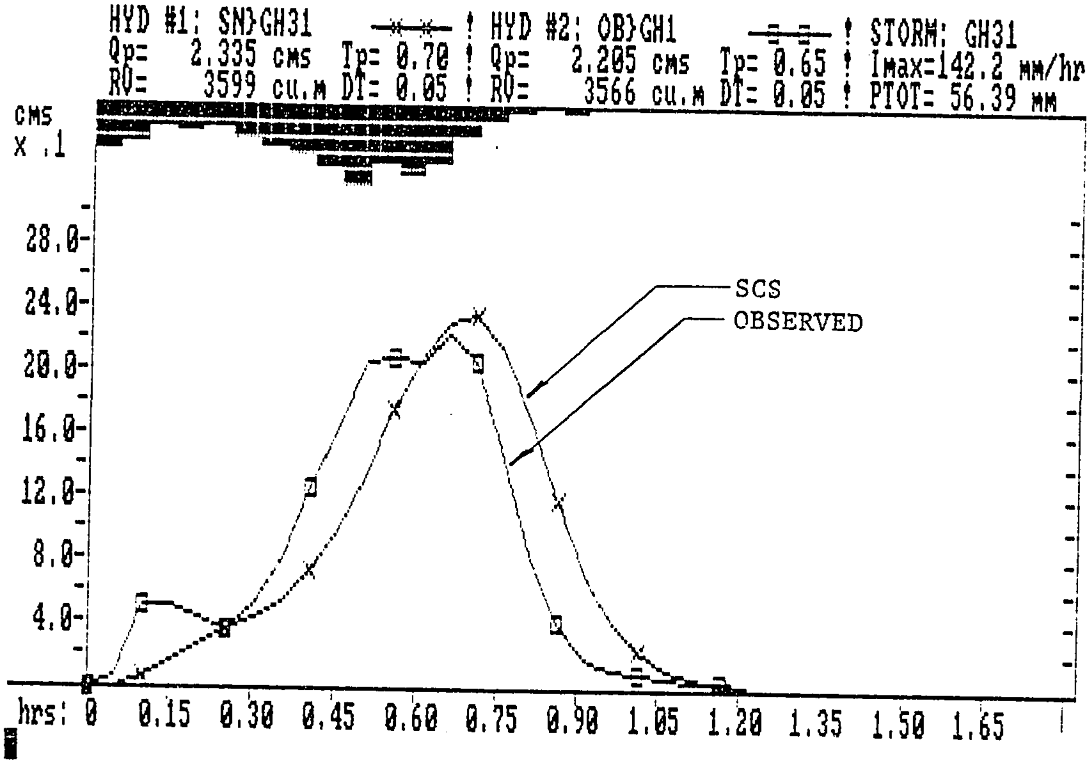


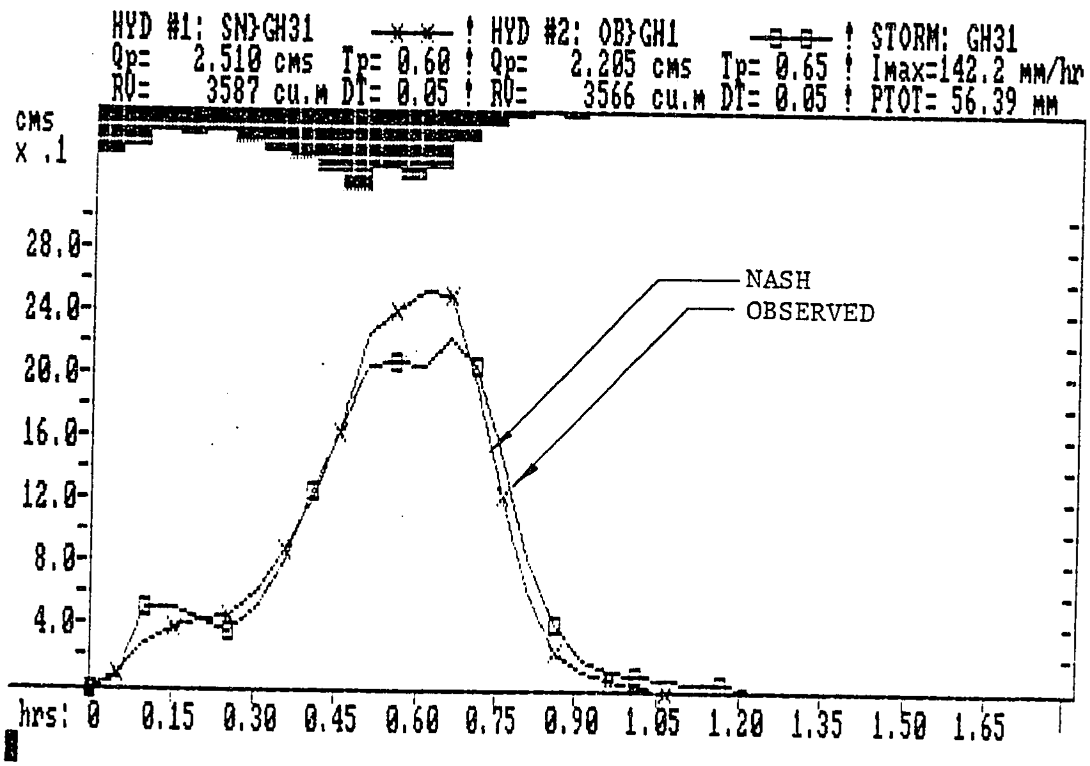
KINGS CREEK EVENT 6



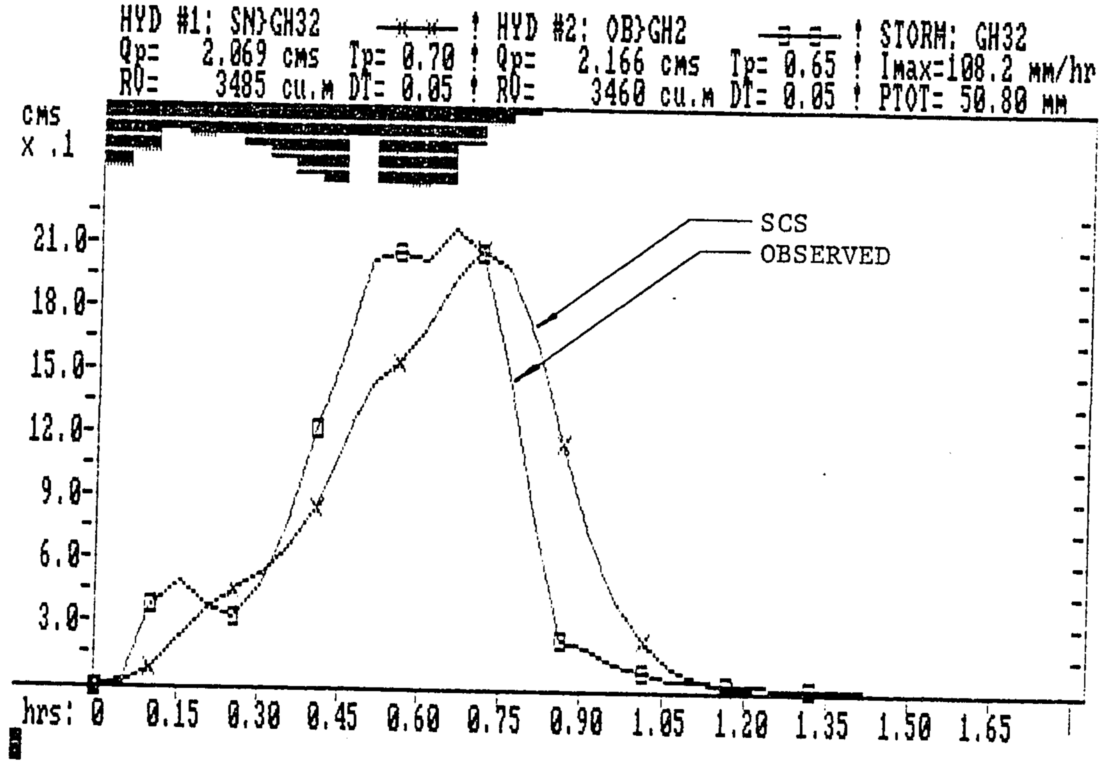


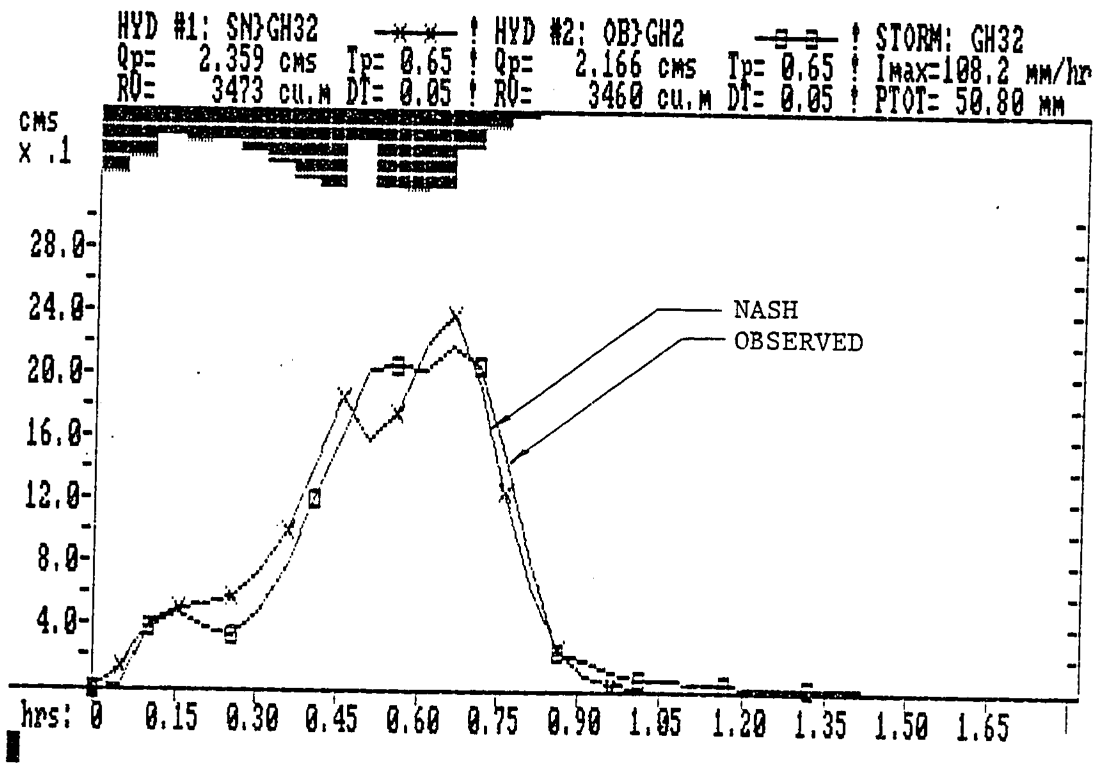
GRAY HAVEN EVENT 1



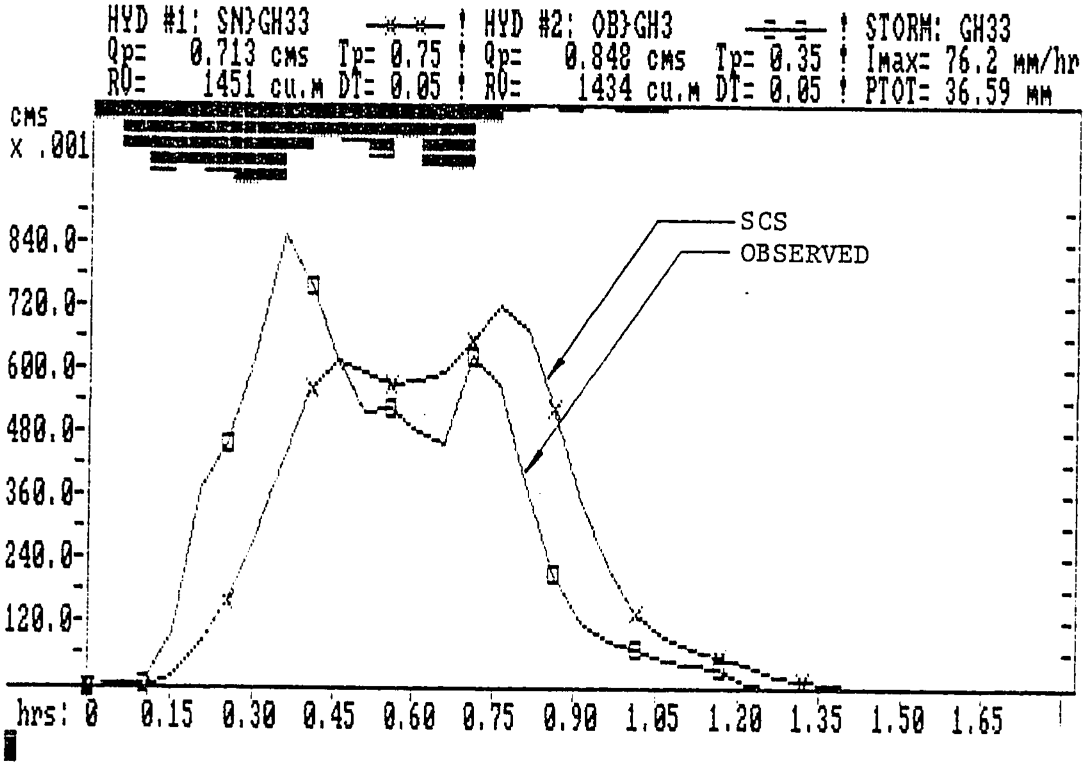


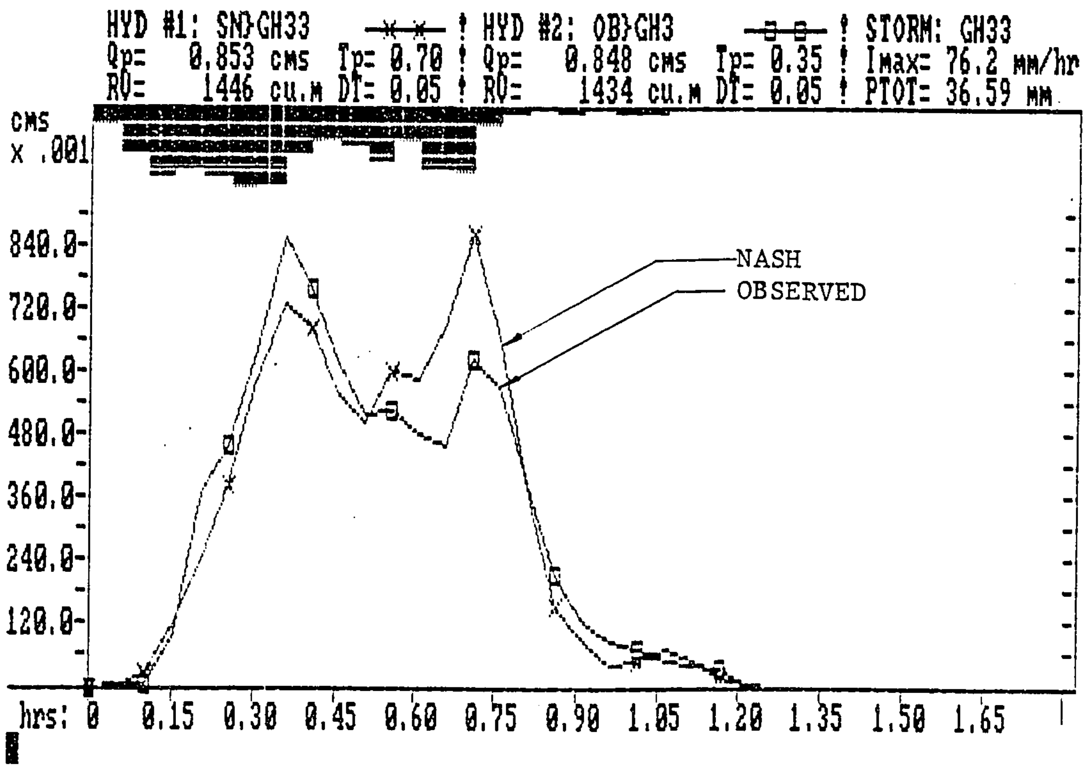
GRAY HAVEN EVENT 2



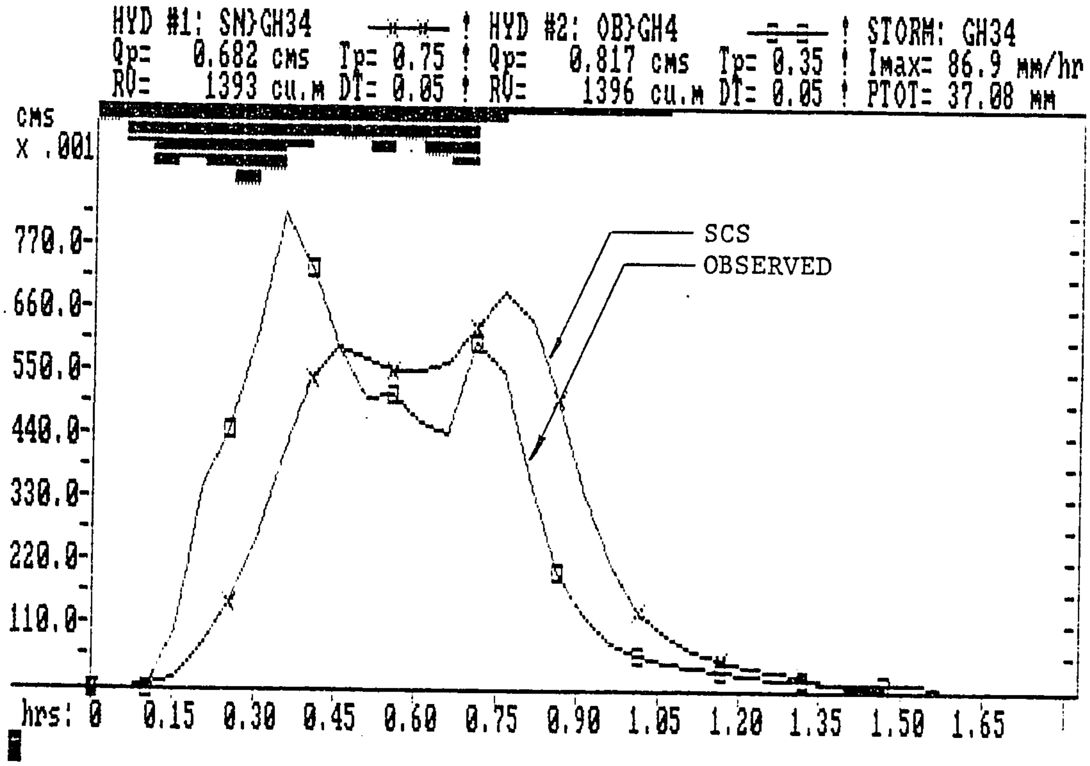


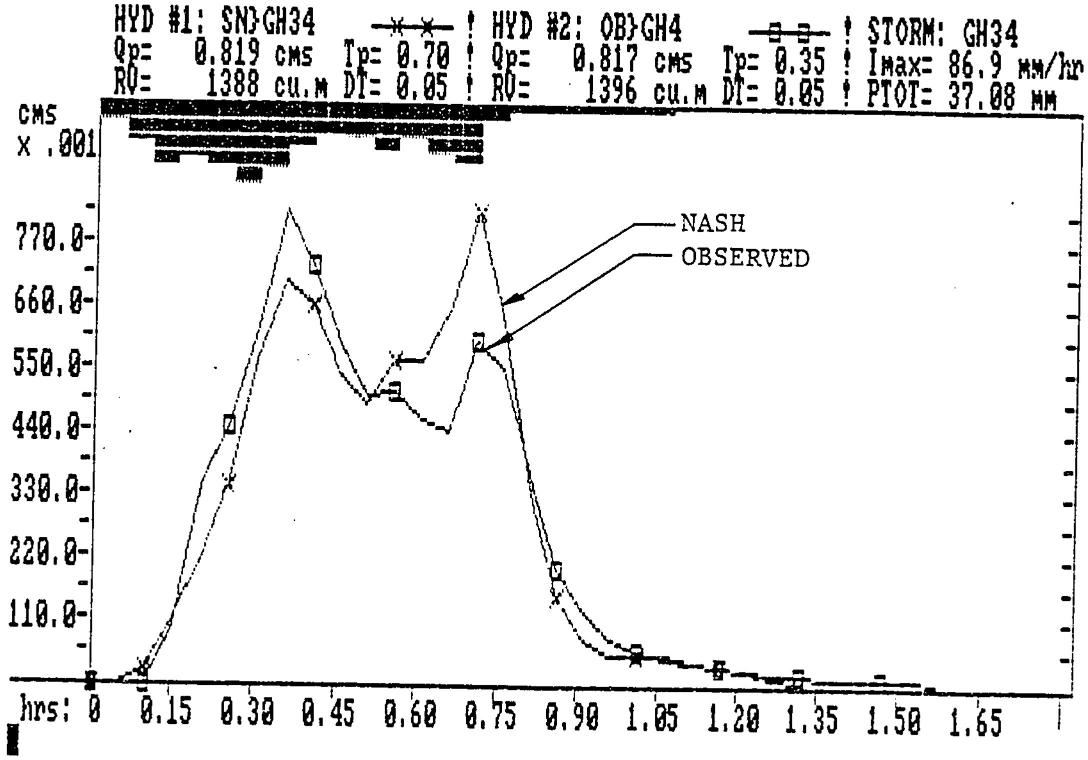
GRAY HAVEN EVENT 3



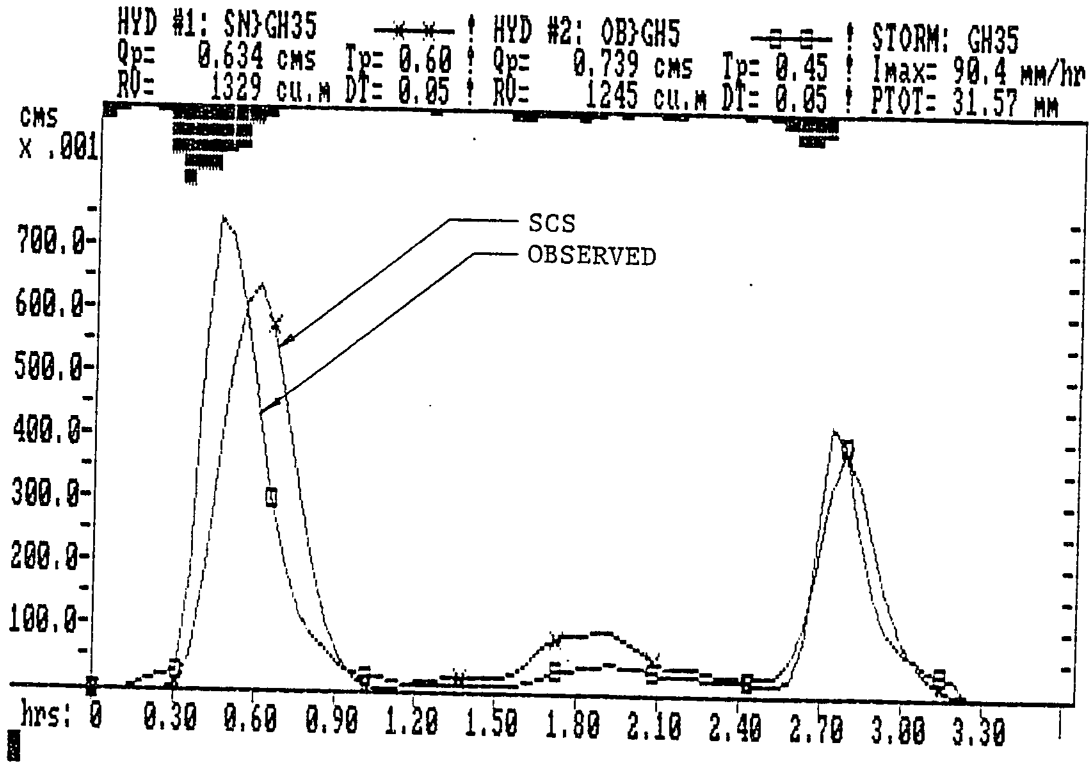


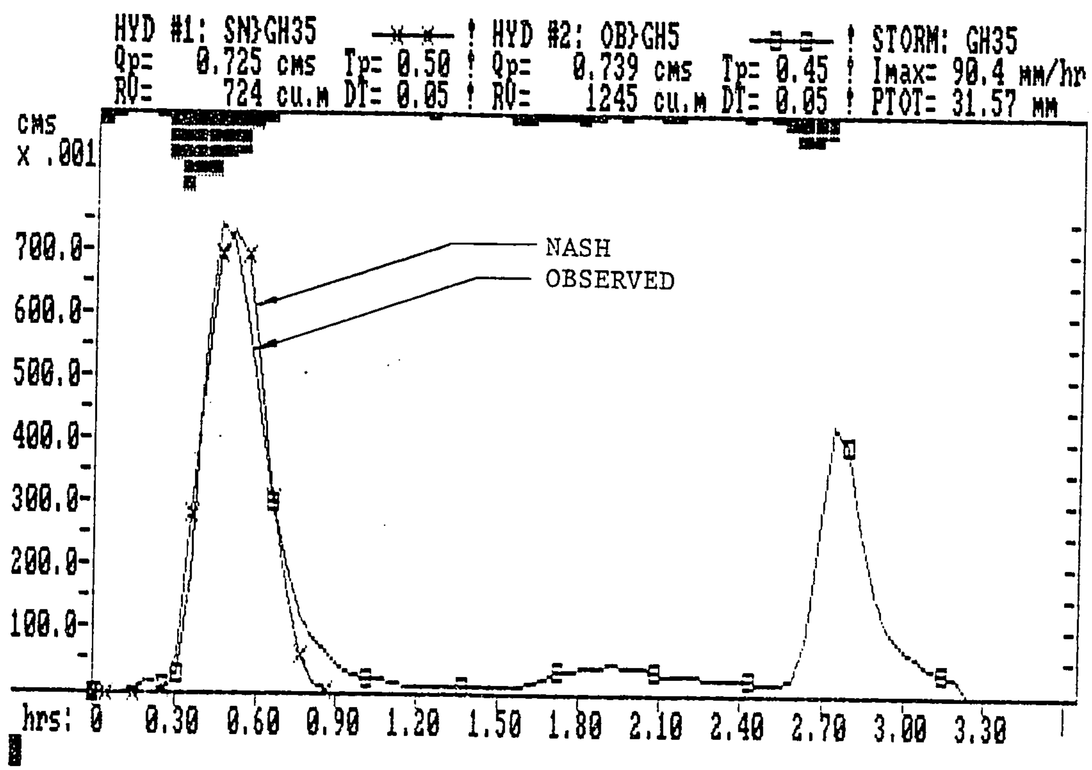
GRAY HAVEN EVENT 4



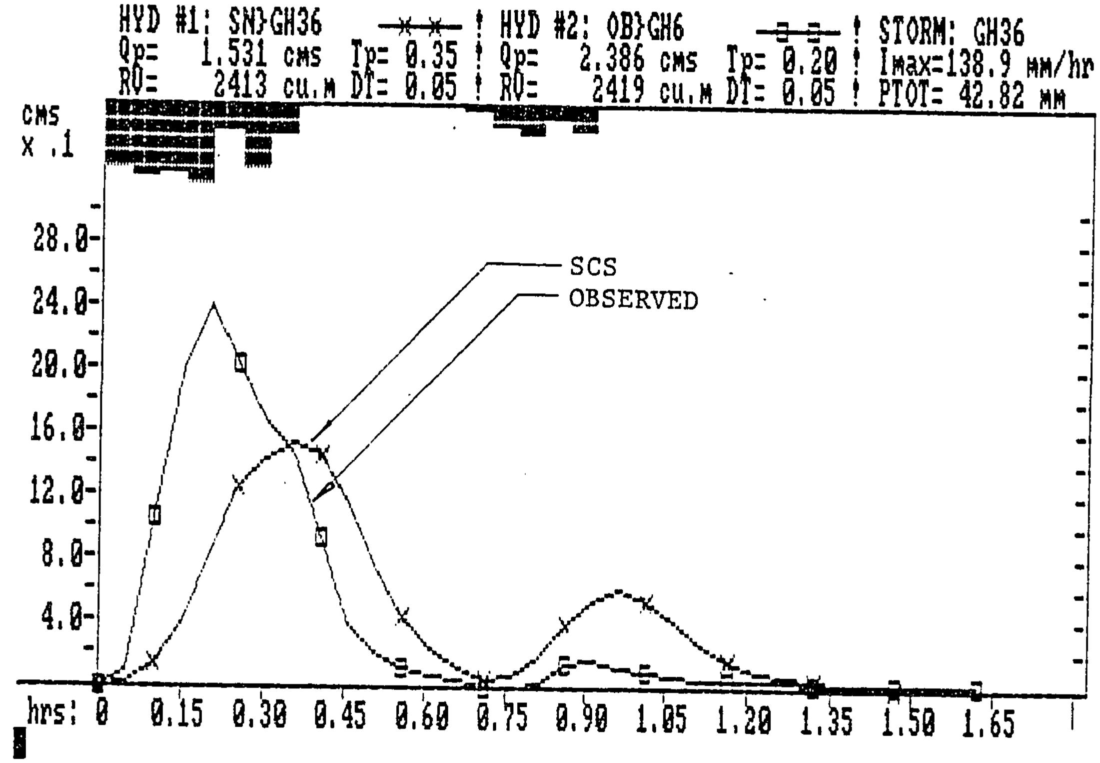


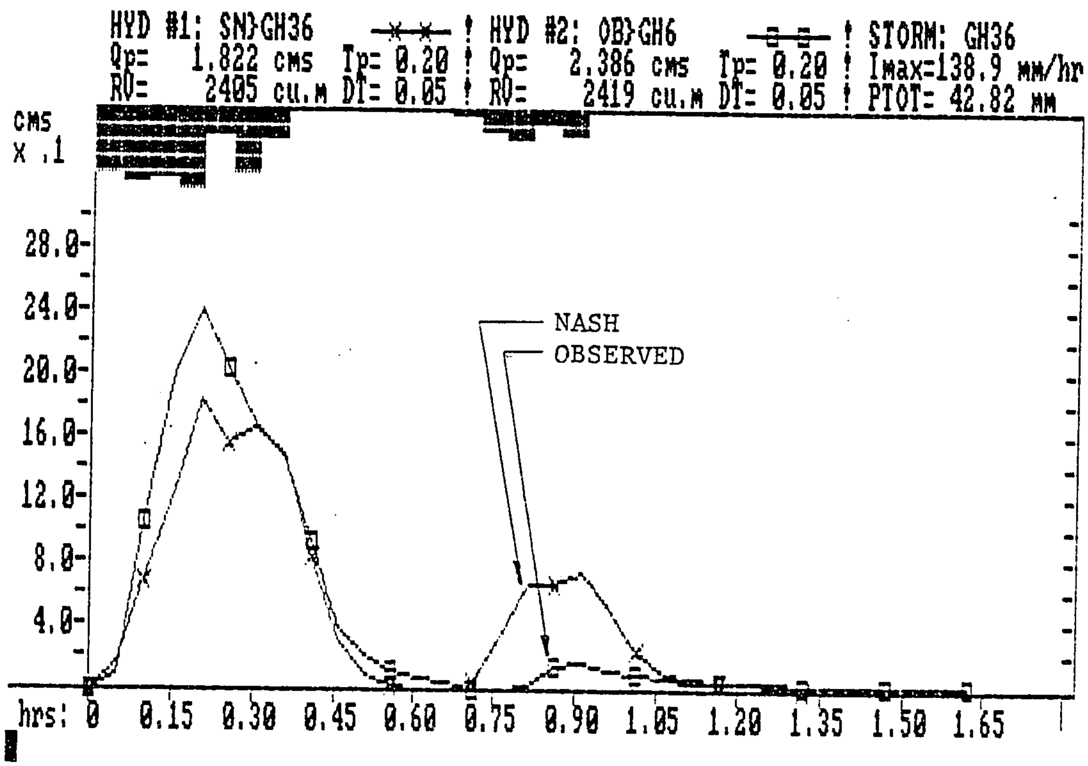
GRAY HAVEN EVENT 5



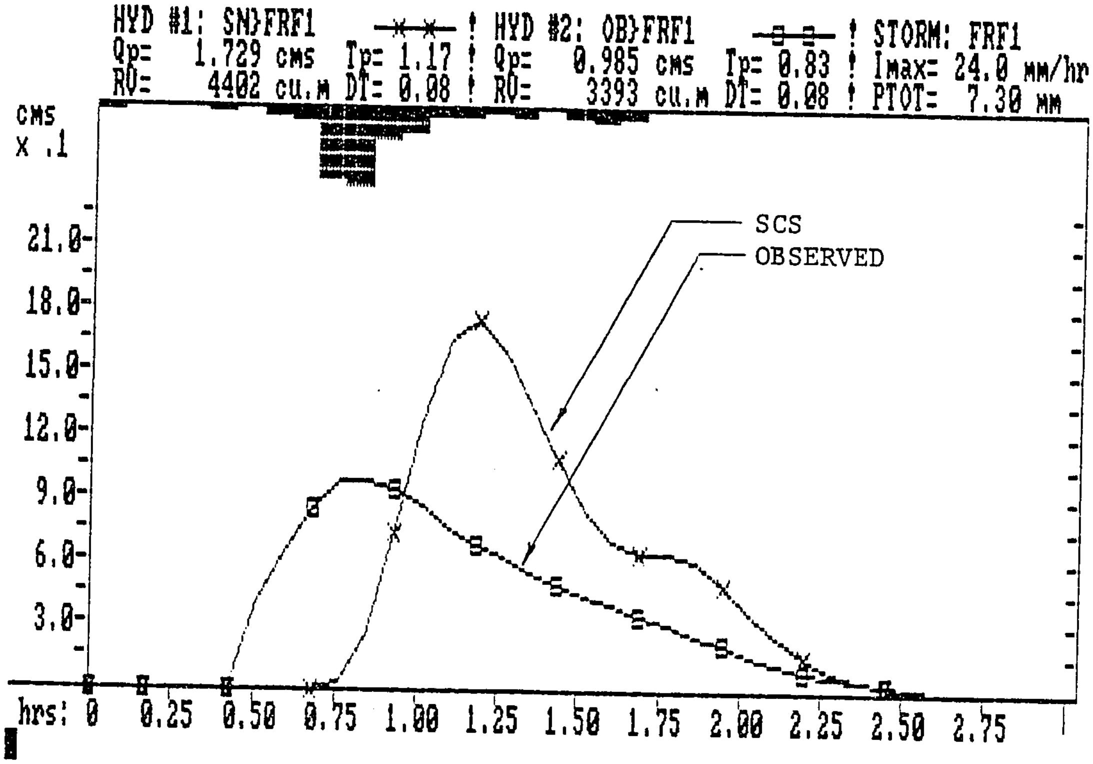


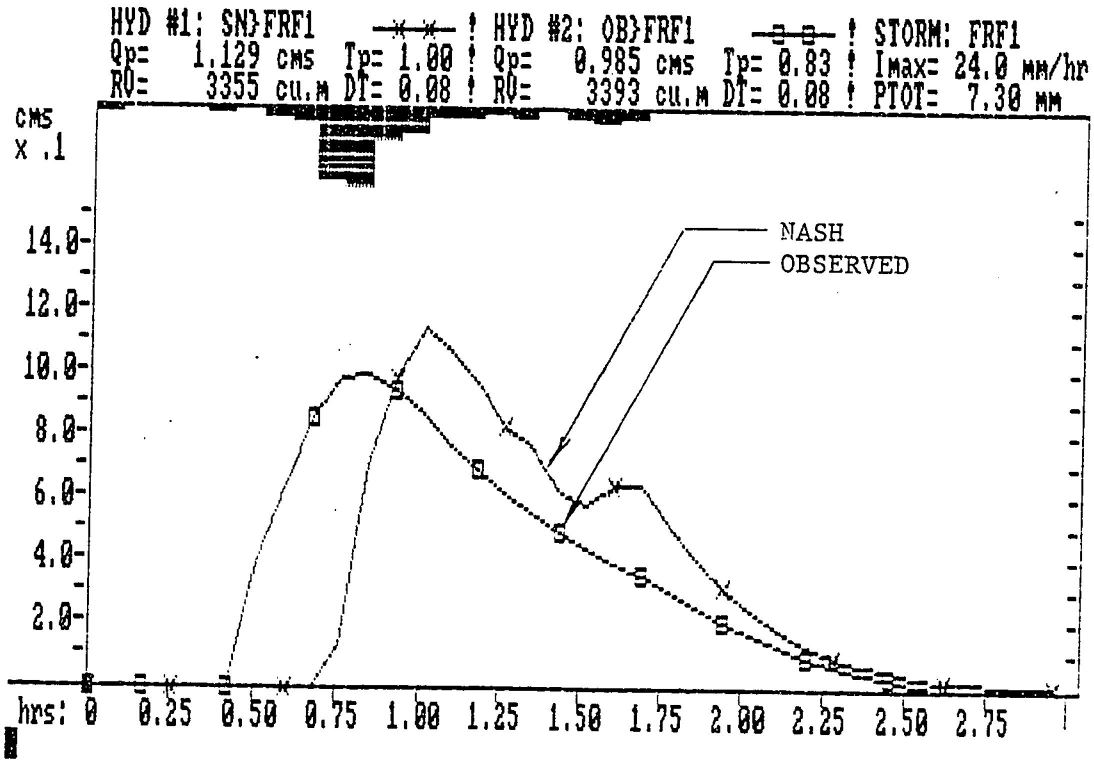
GRAY HAVEN EVENT 6



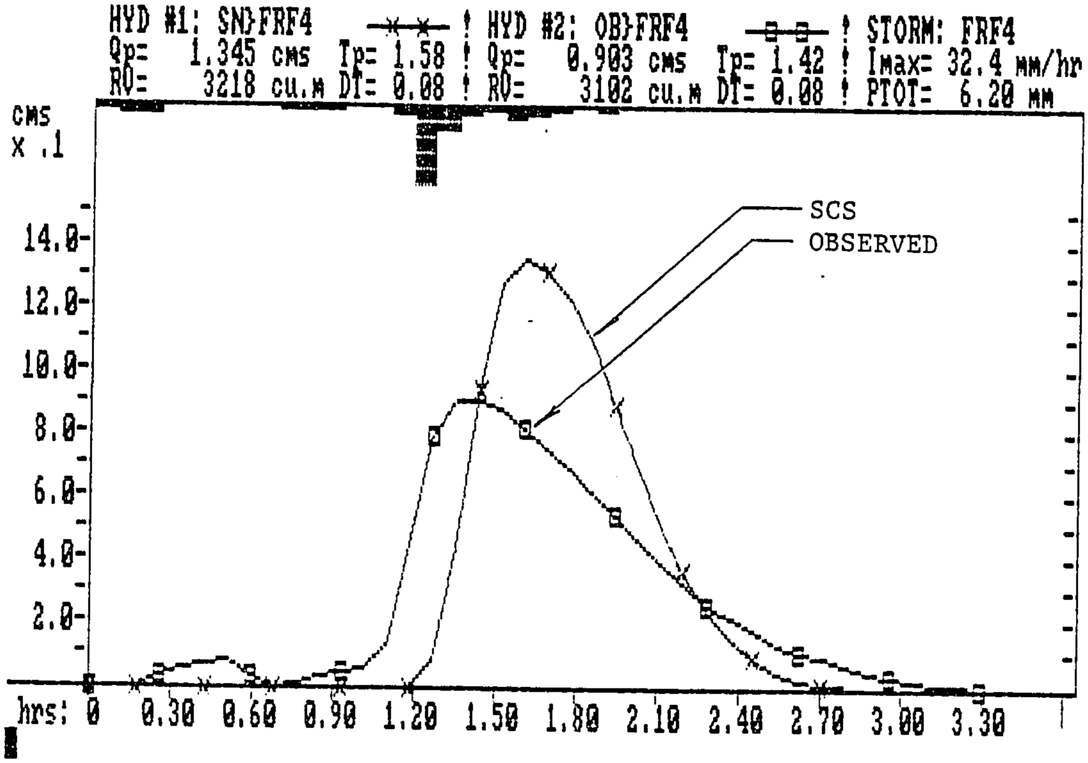


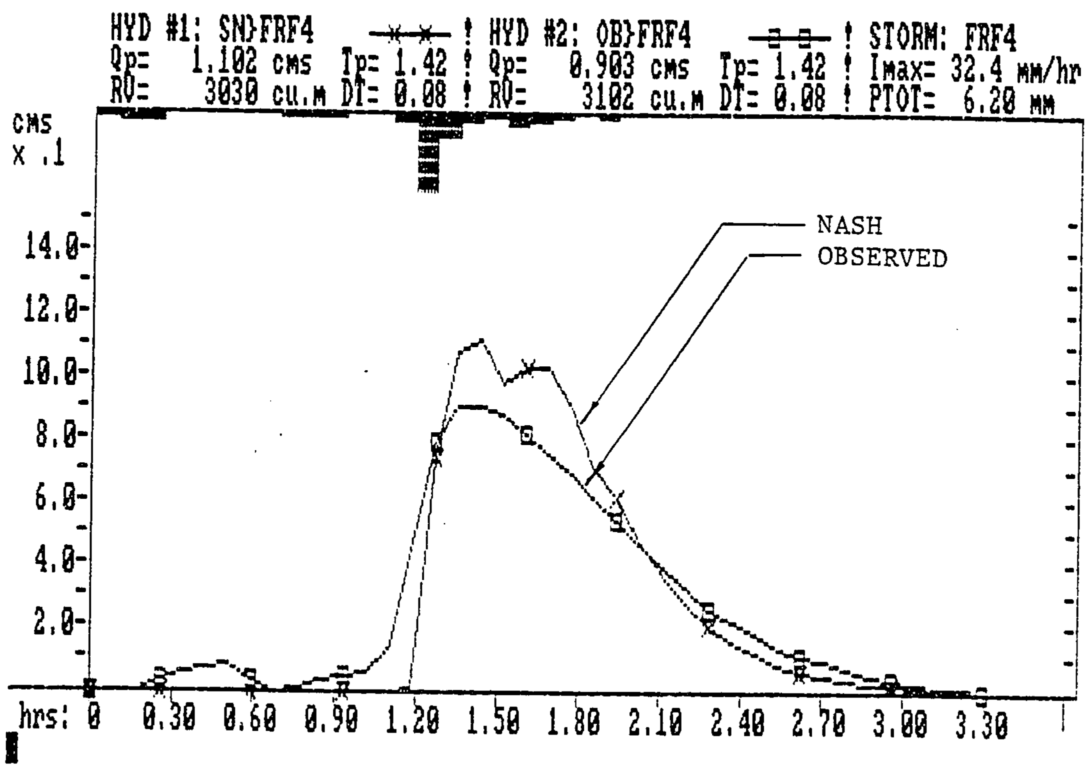
FRANCE EVENT 1



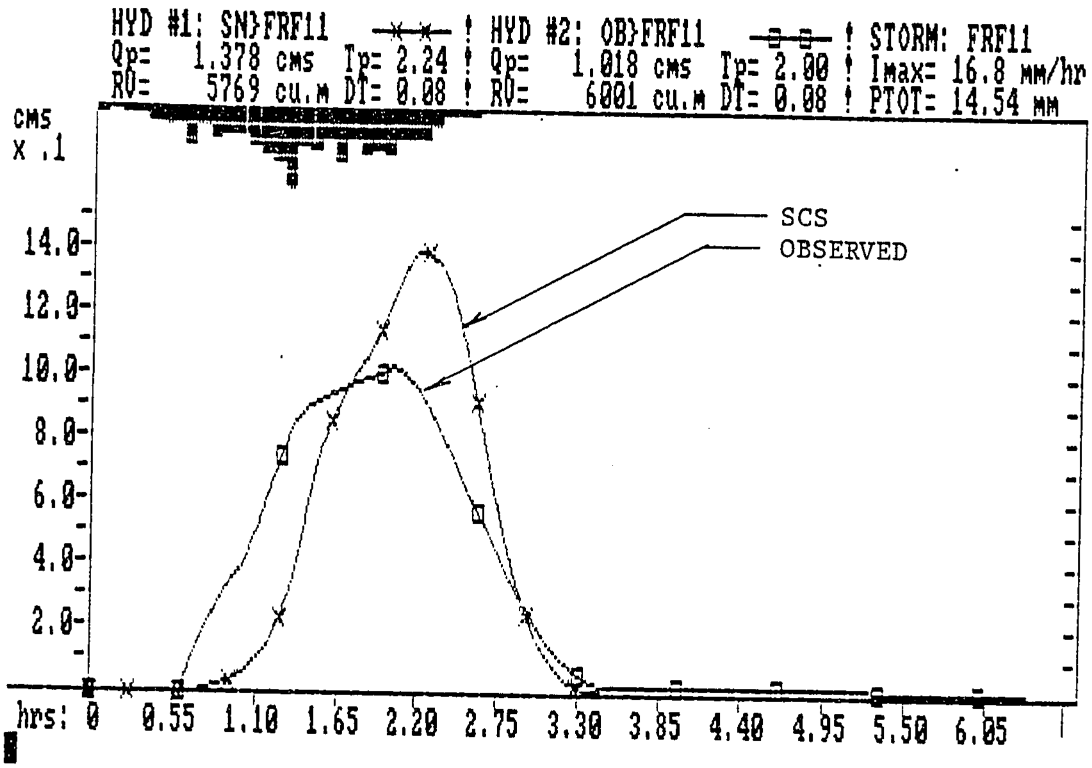


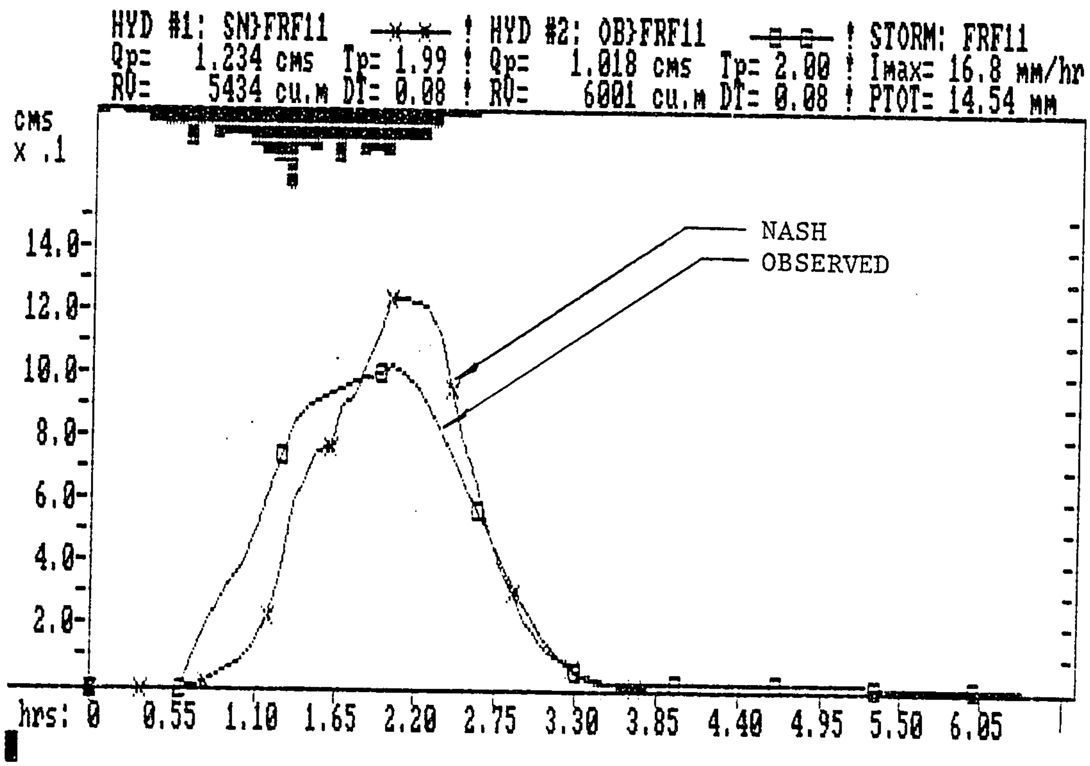
FRANCE EVENT 4



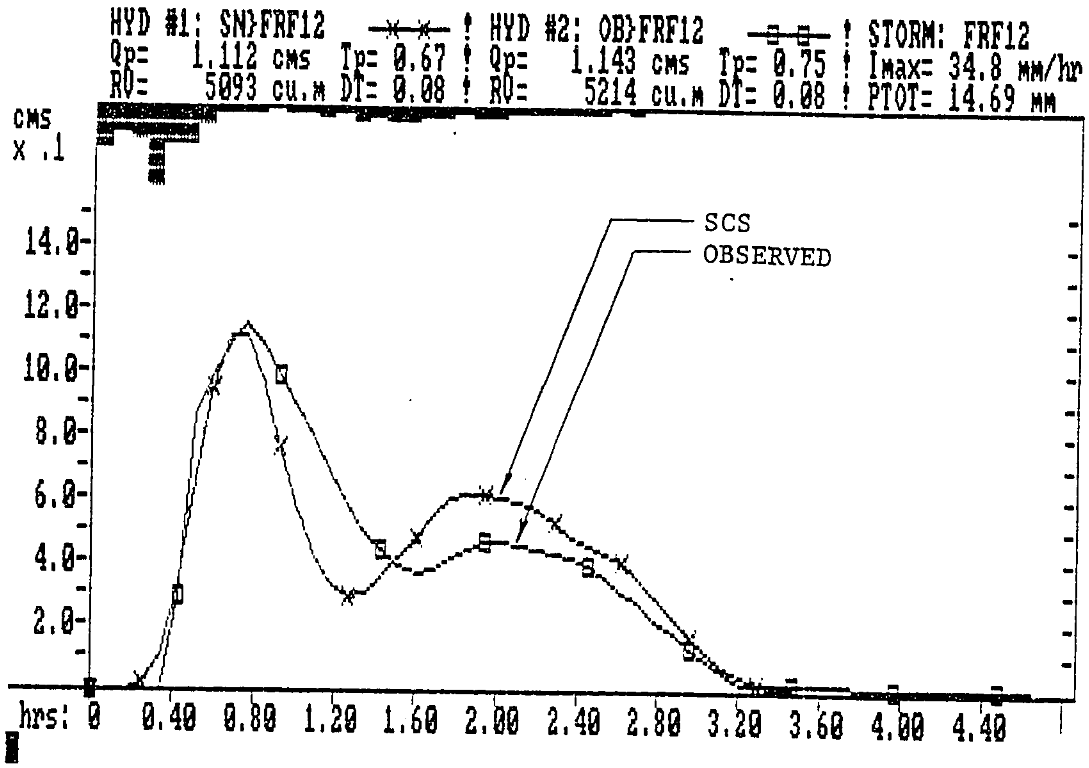


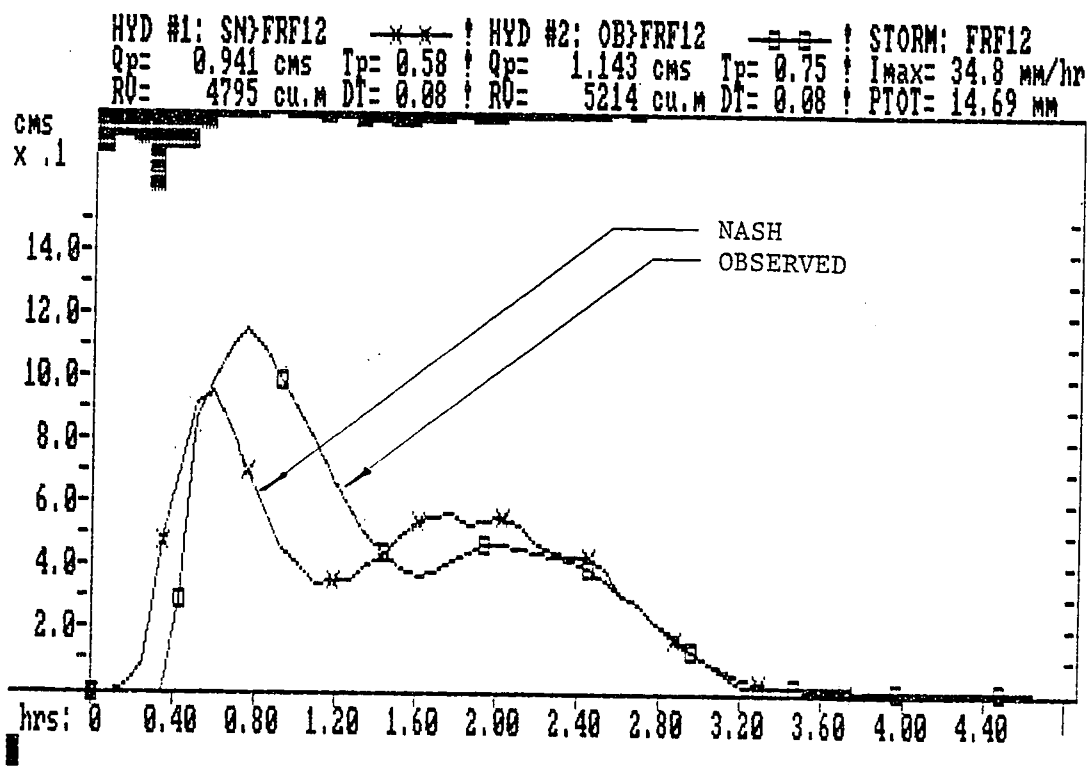
FRANCE EVENT 11



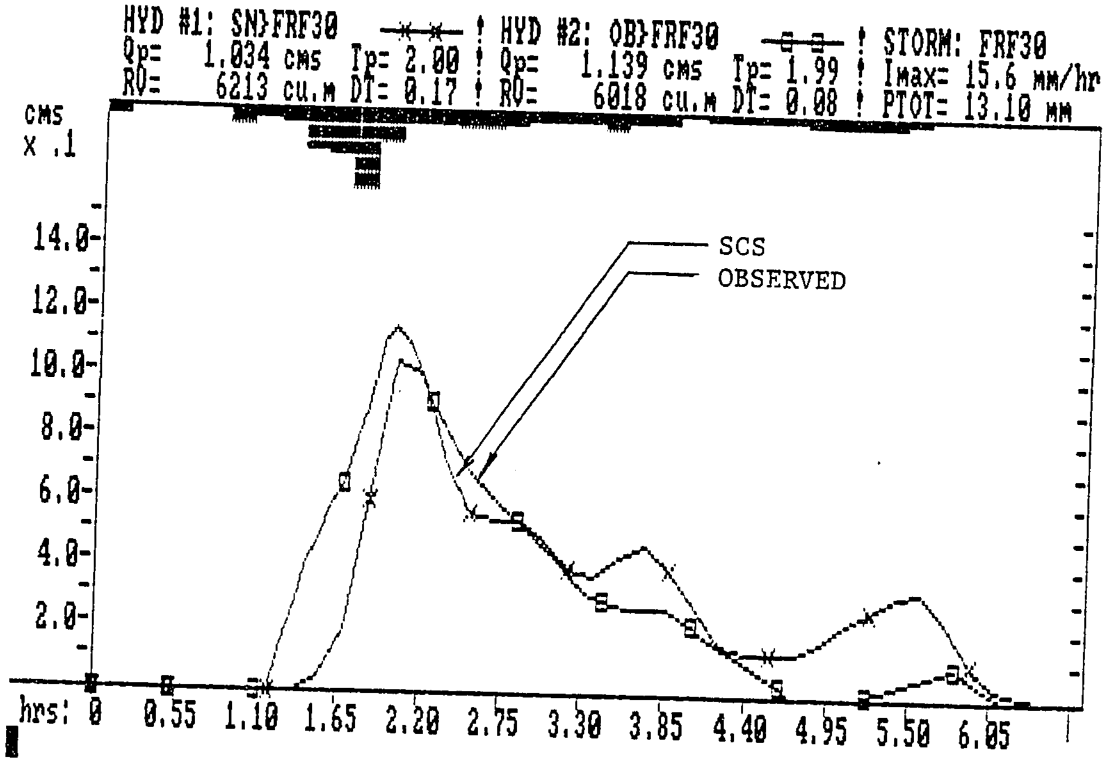


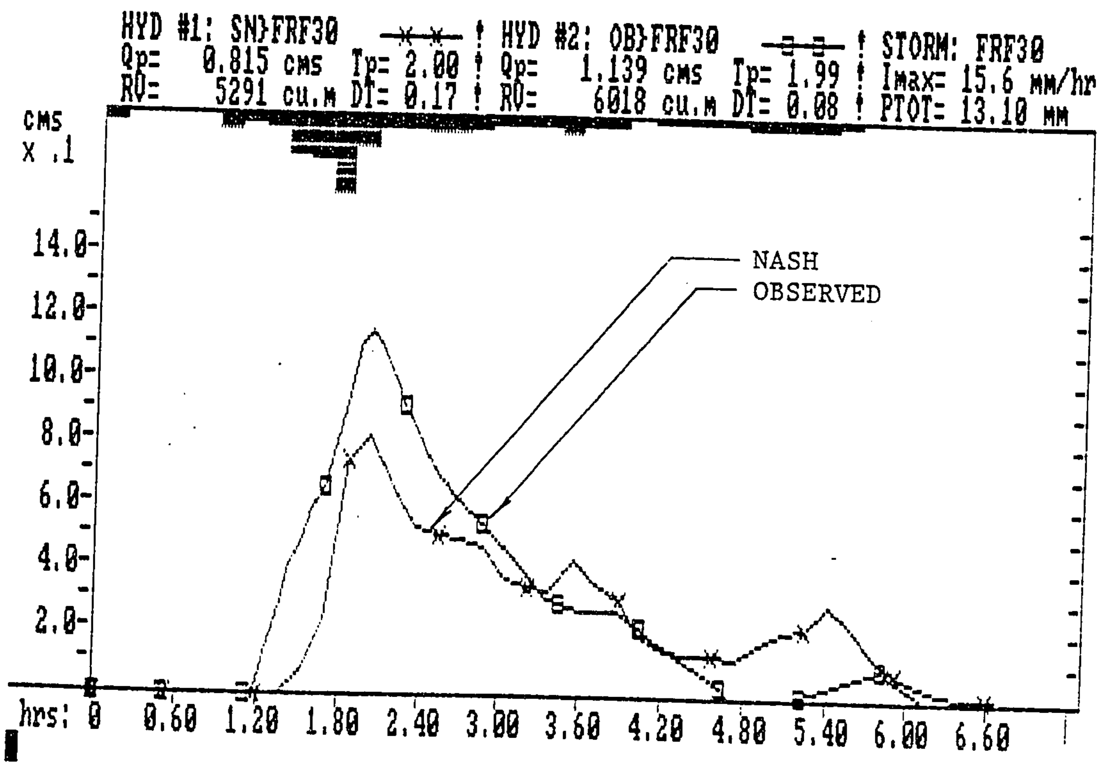
FRANCE EVENT 12



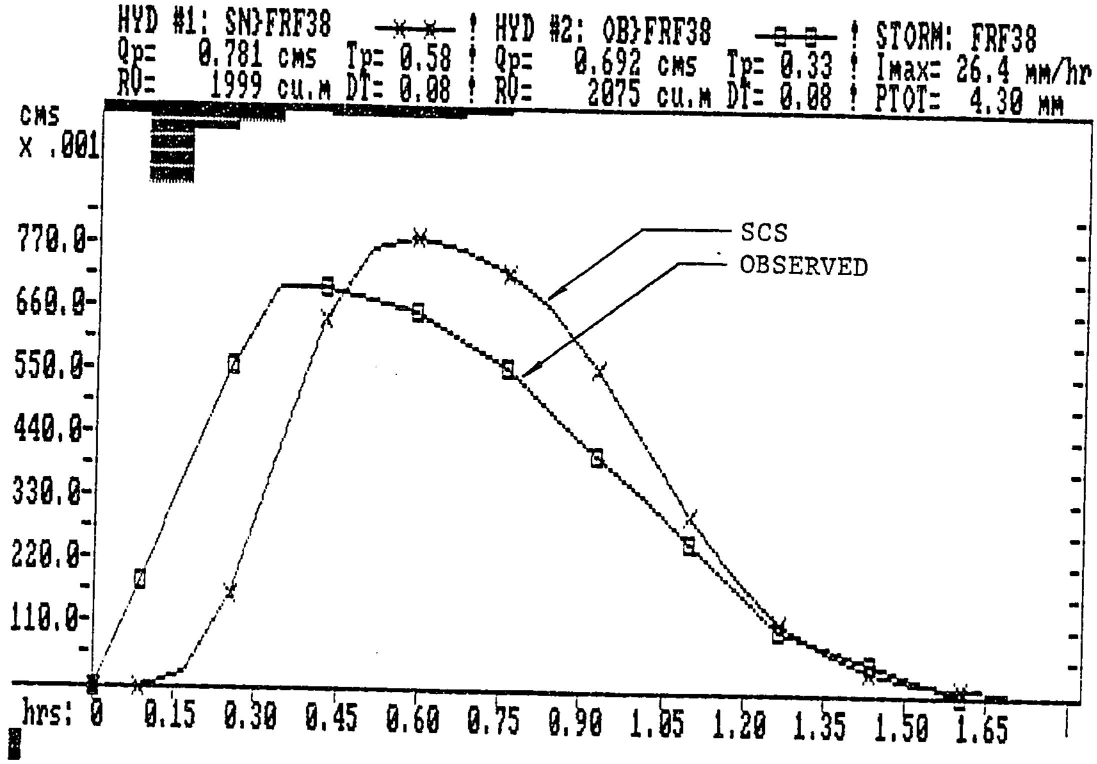


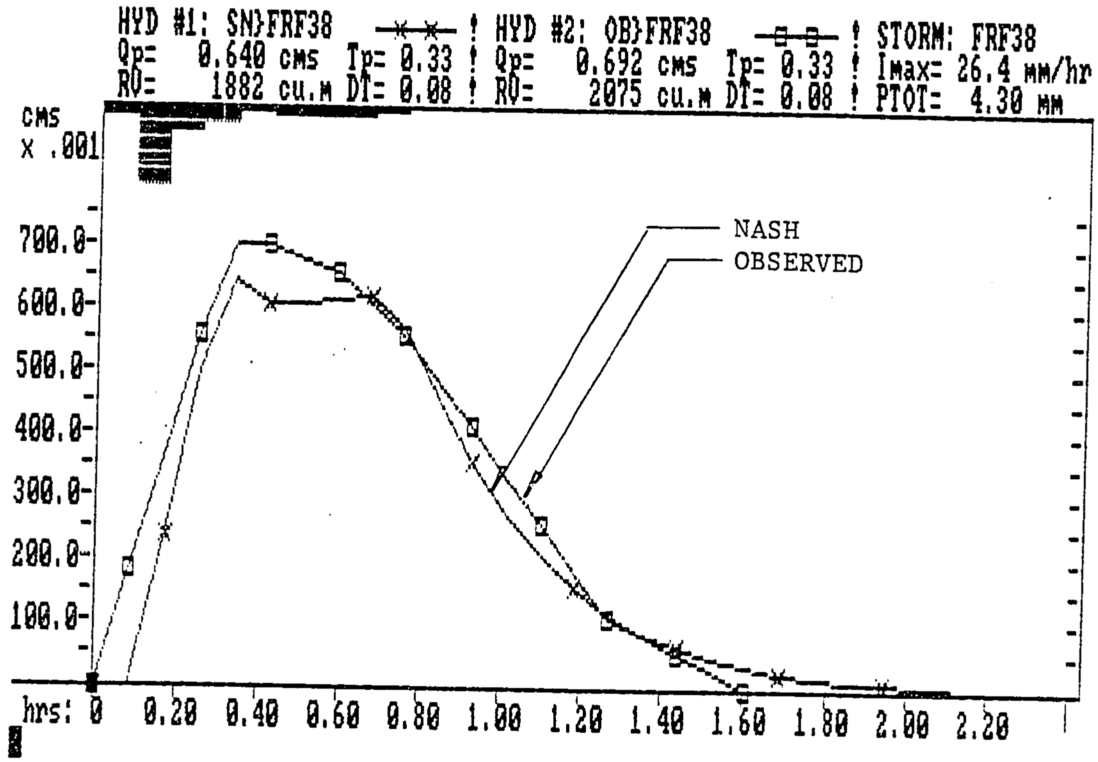
FRANCE EVENT 30



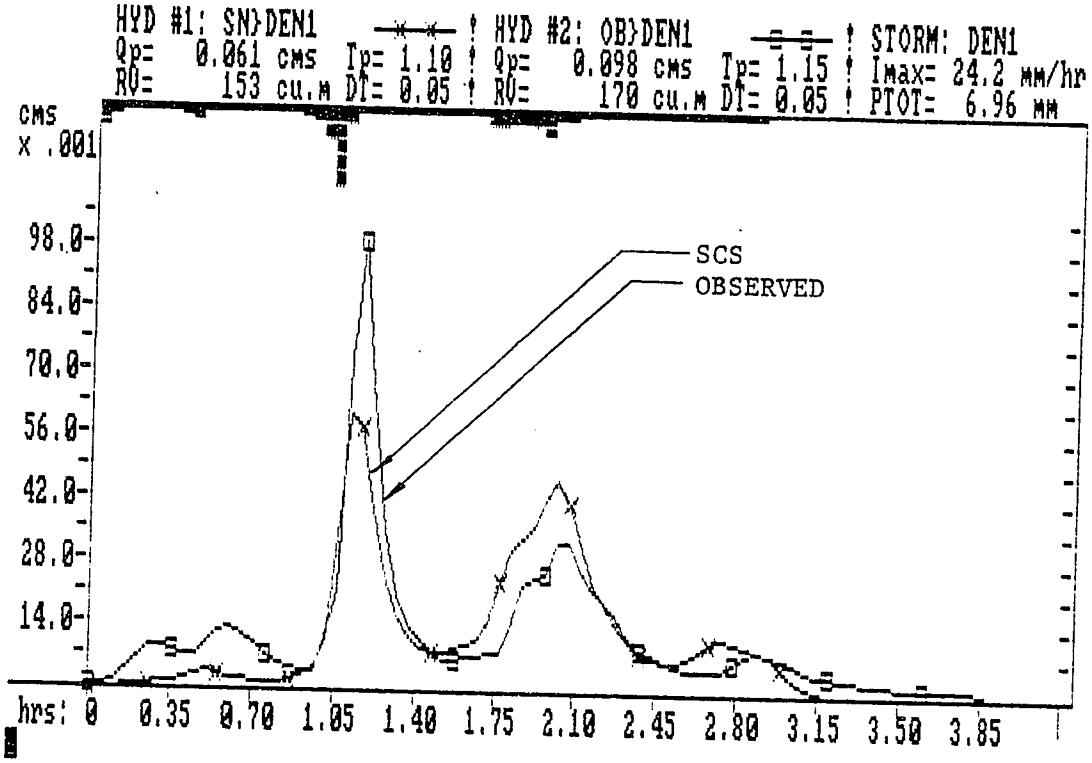


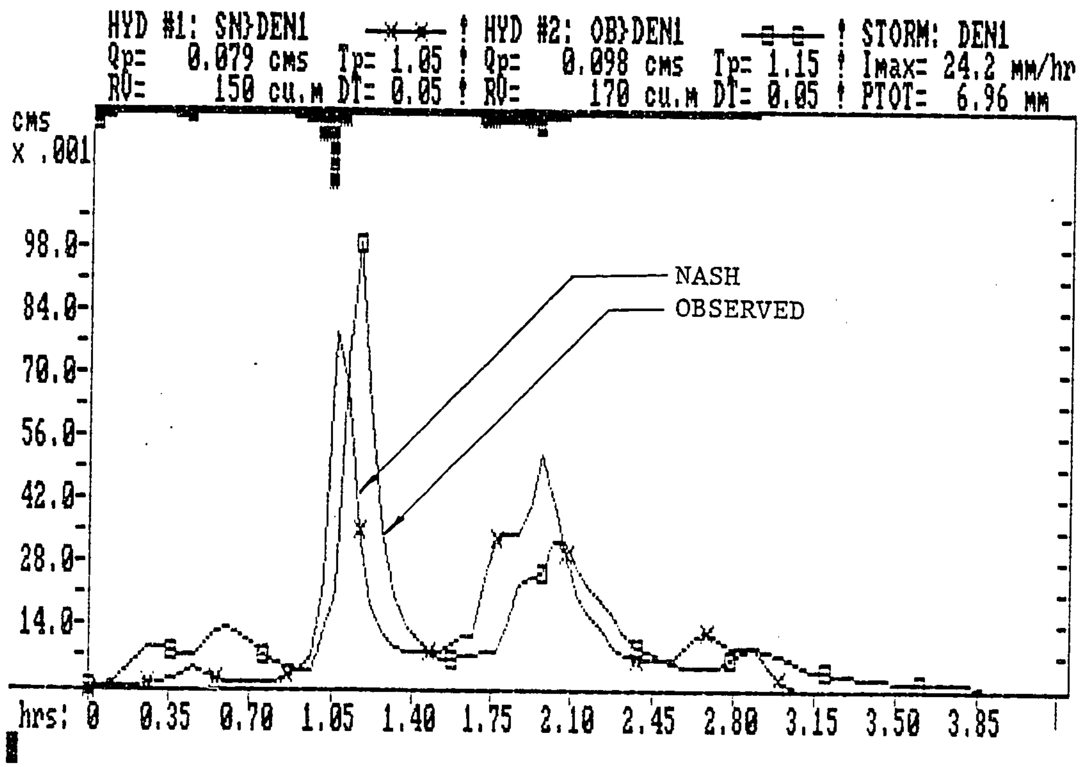
FRANCE EVENT 38



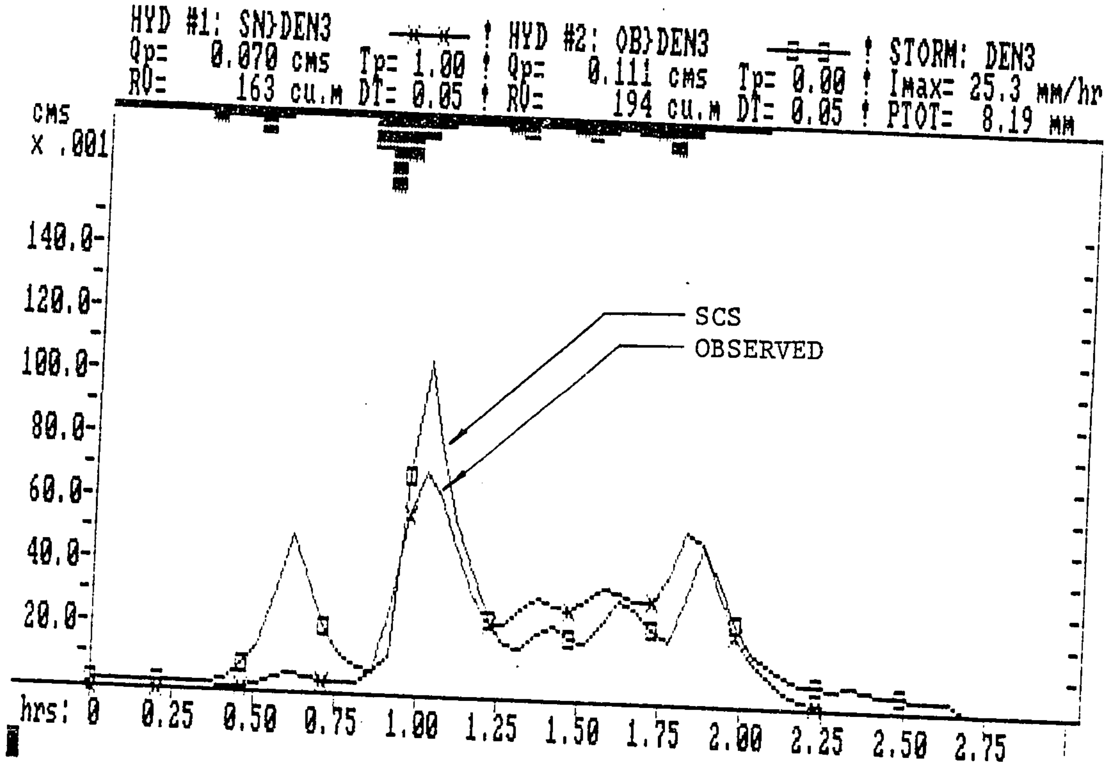


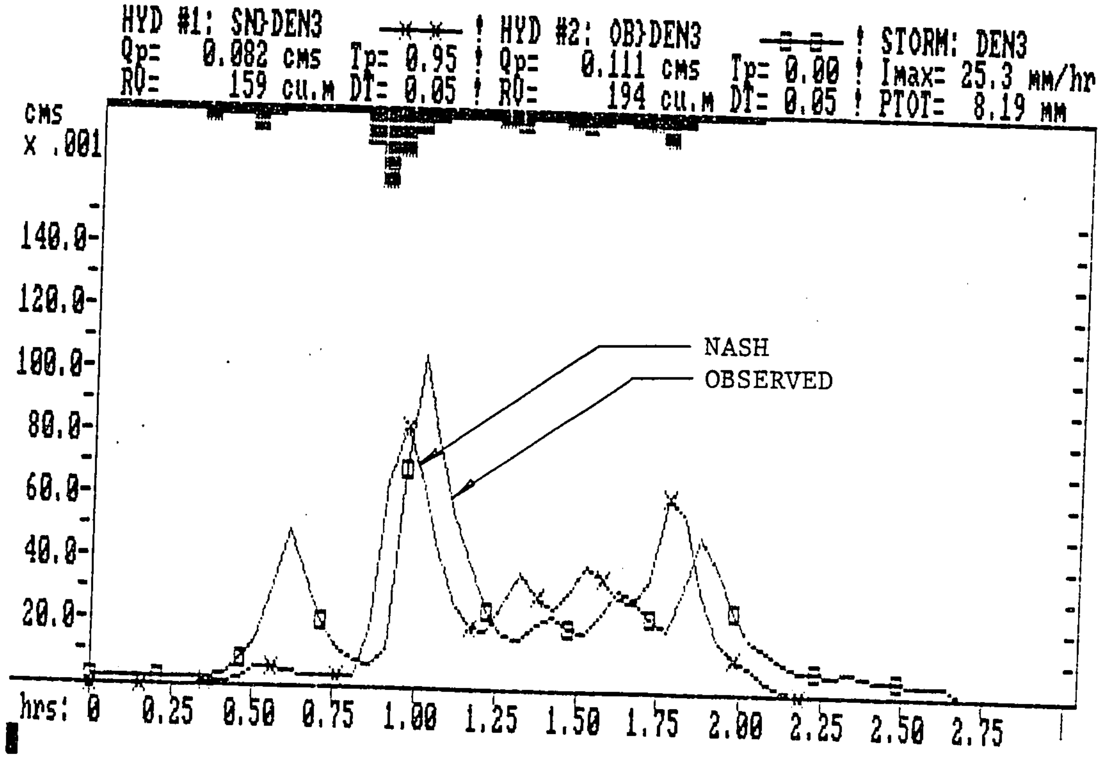
DENMARK EVENT 1



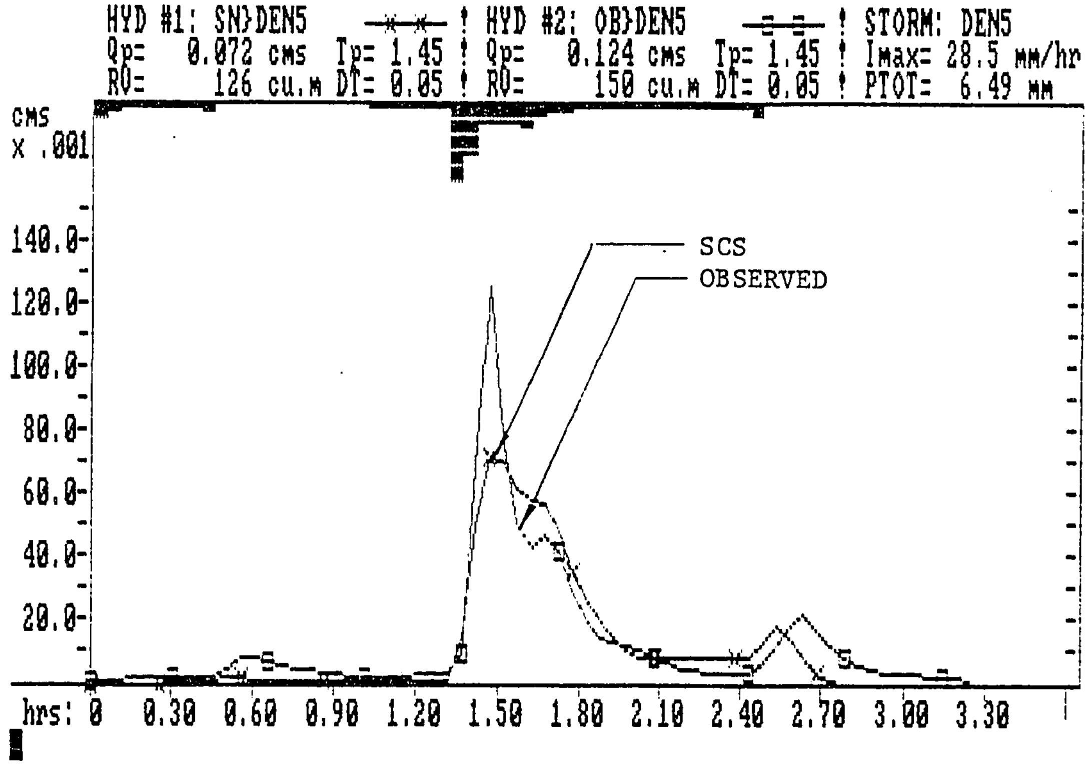


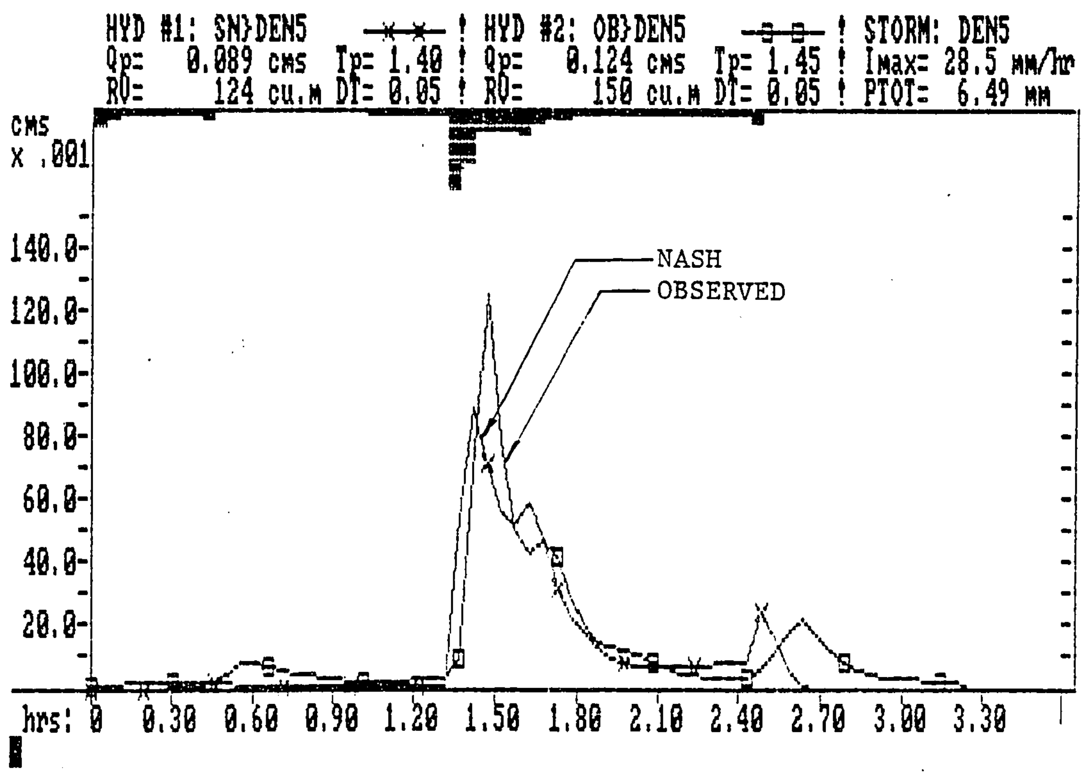
DENMARK EVENT 3



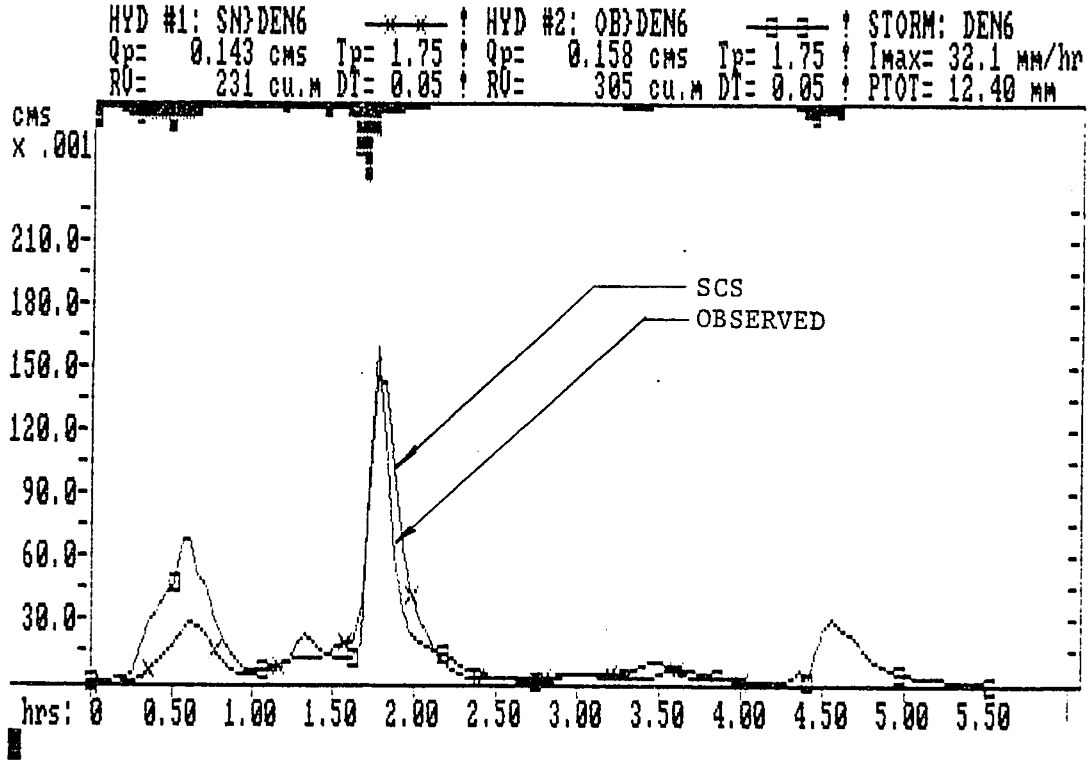


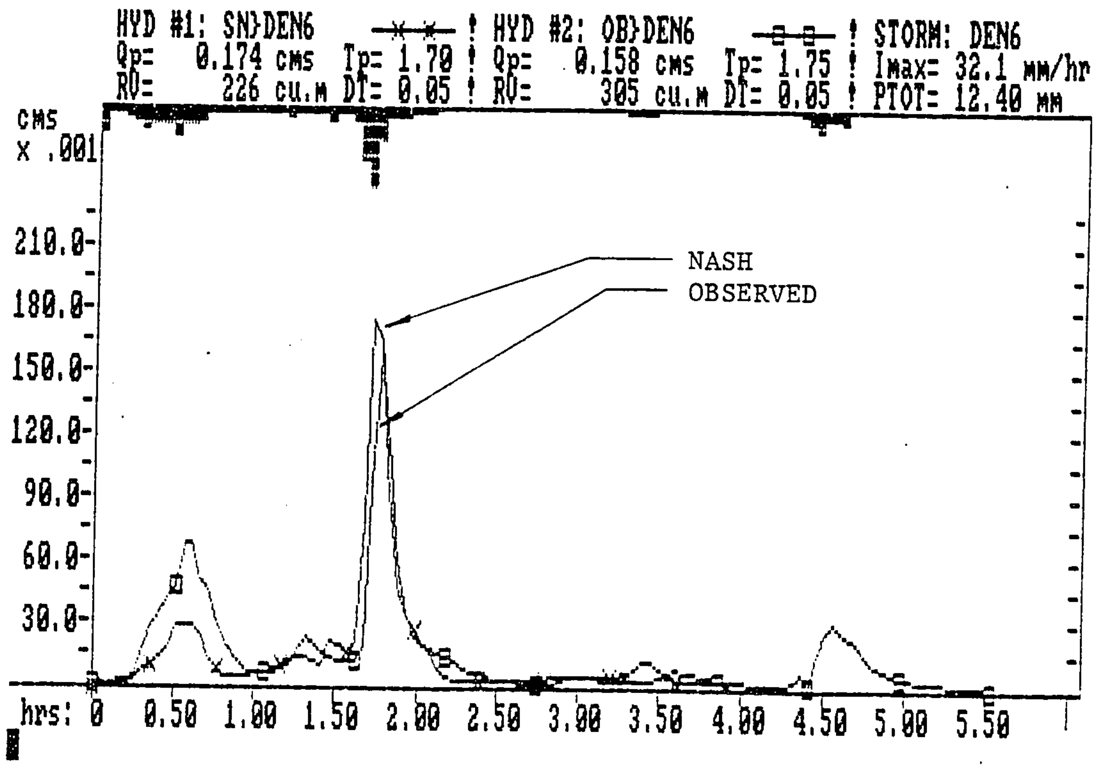
DENMARK EVENT 5



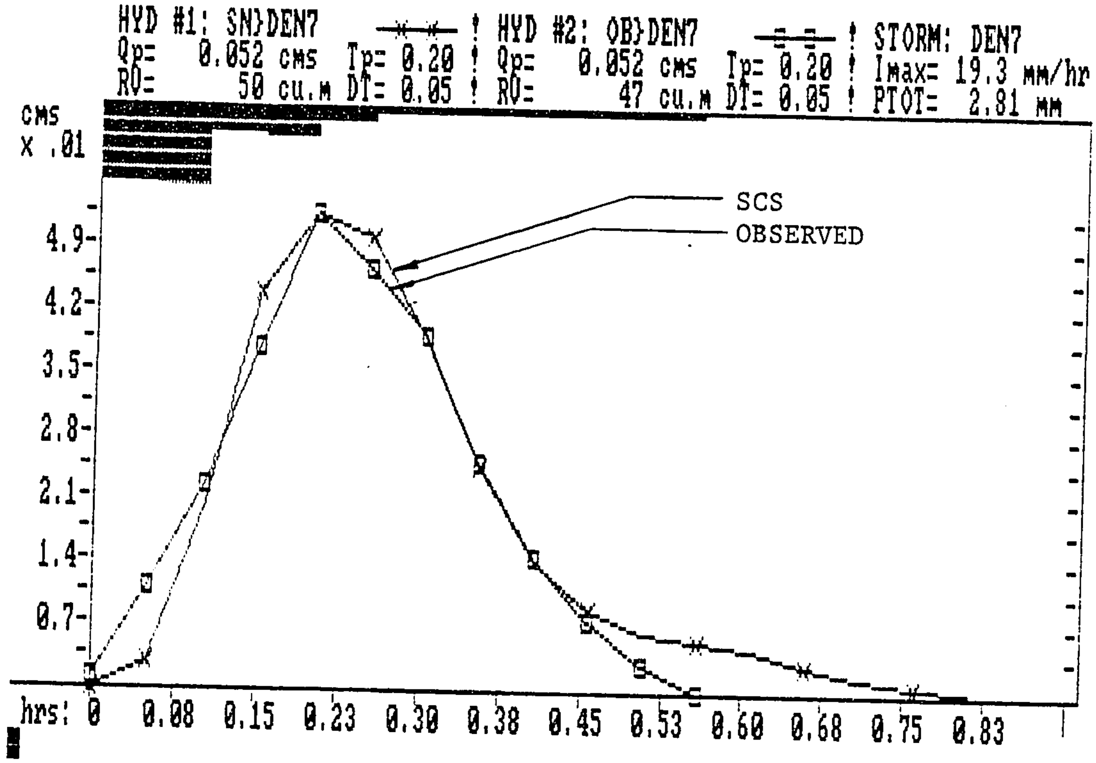


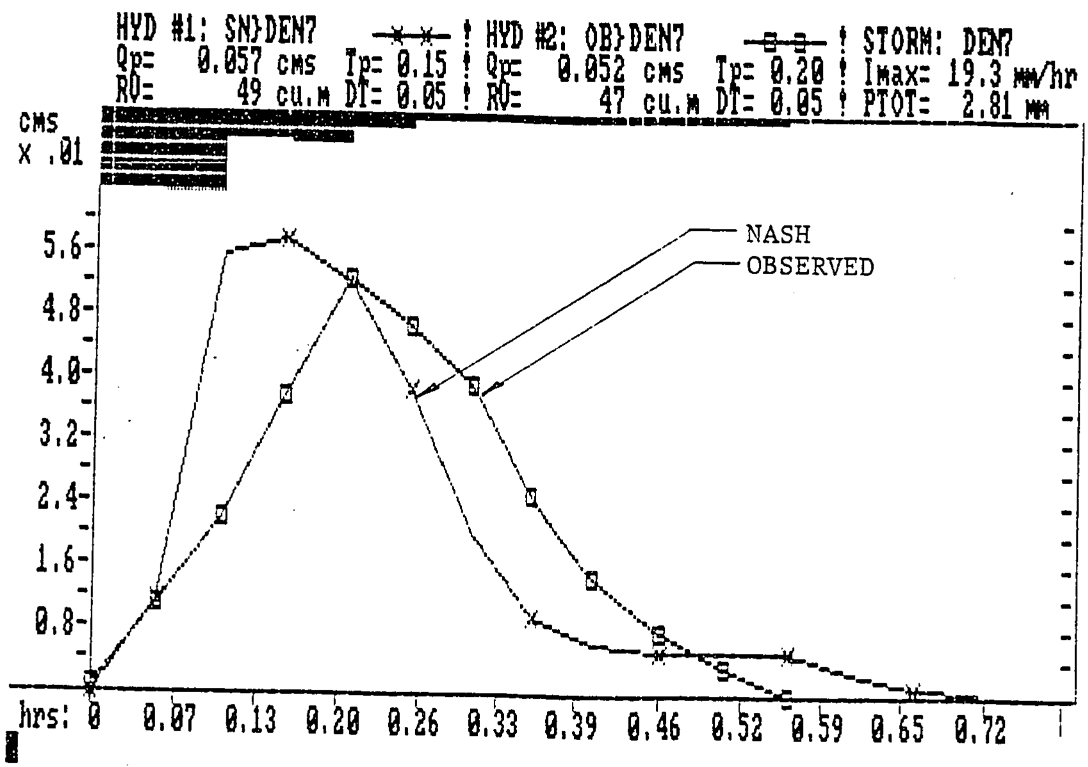
DENMARK EVENT 6



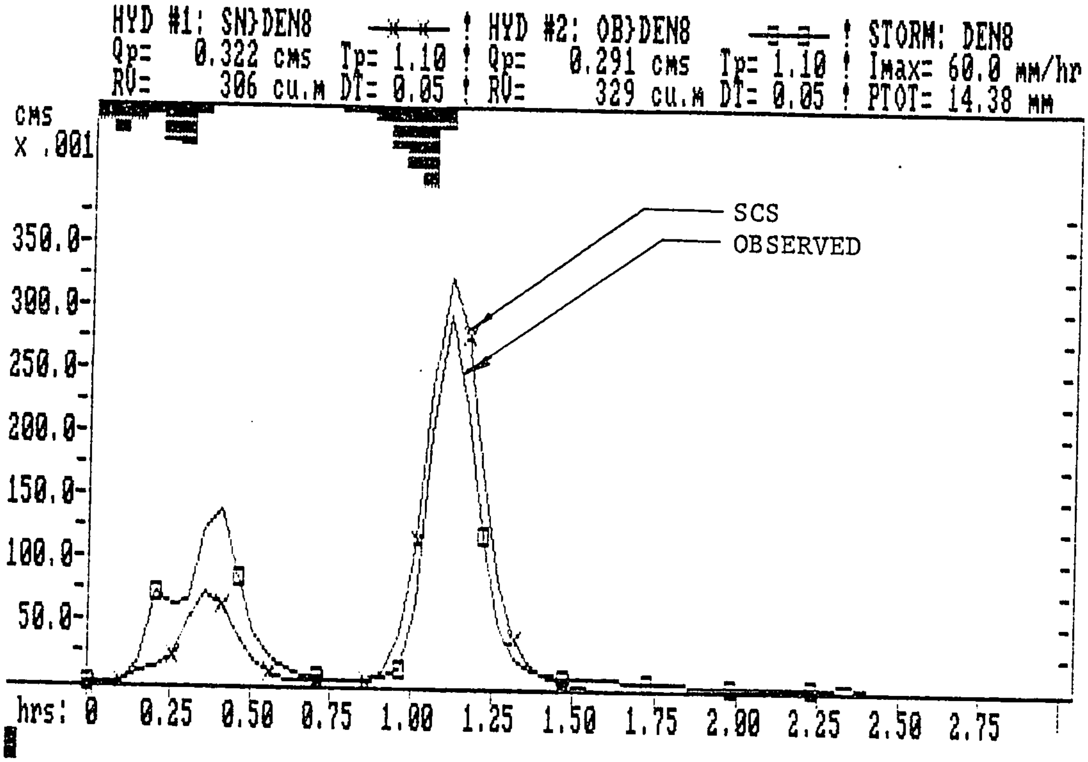


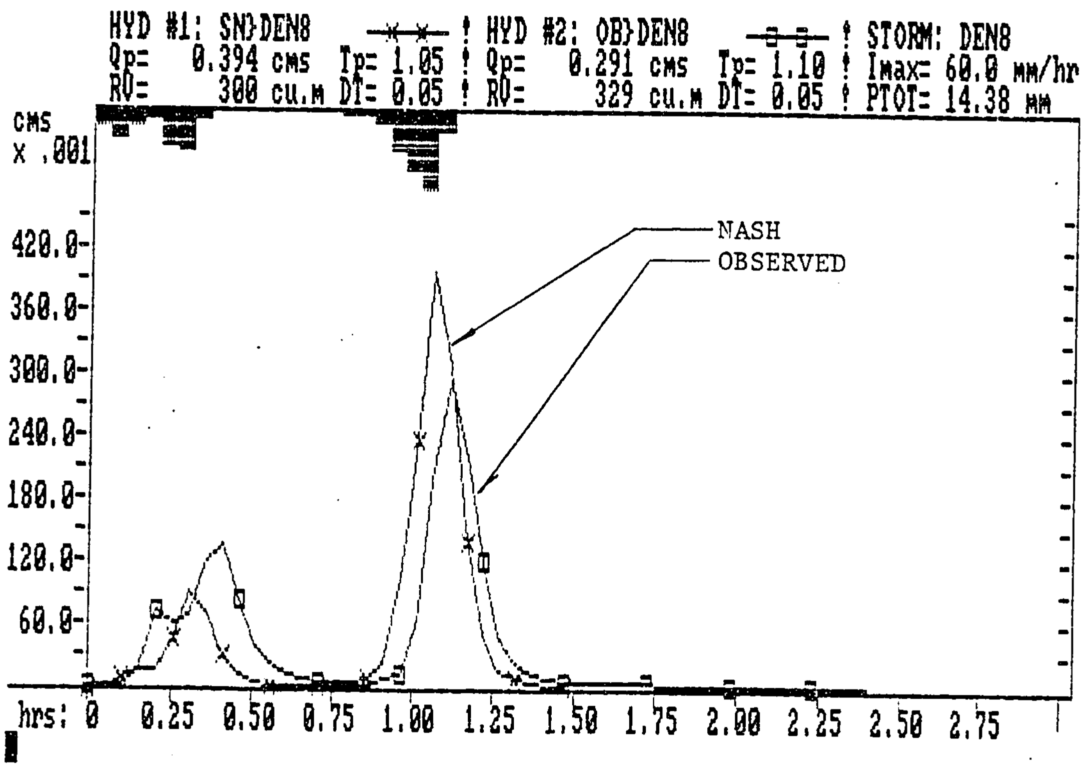
DENMARK EVENT 7



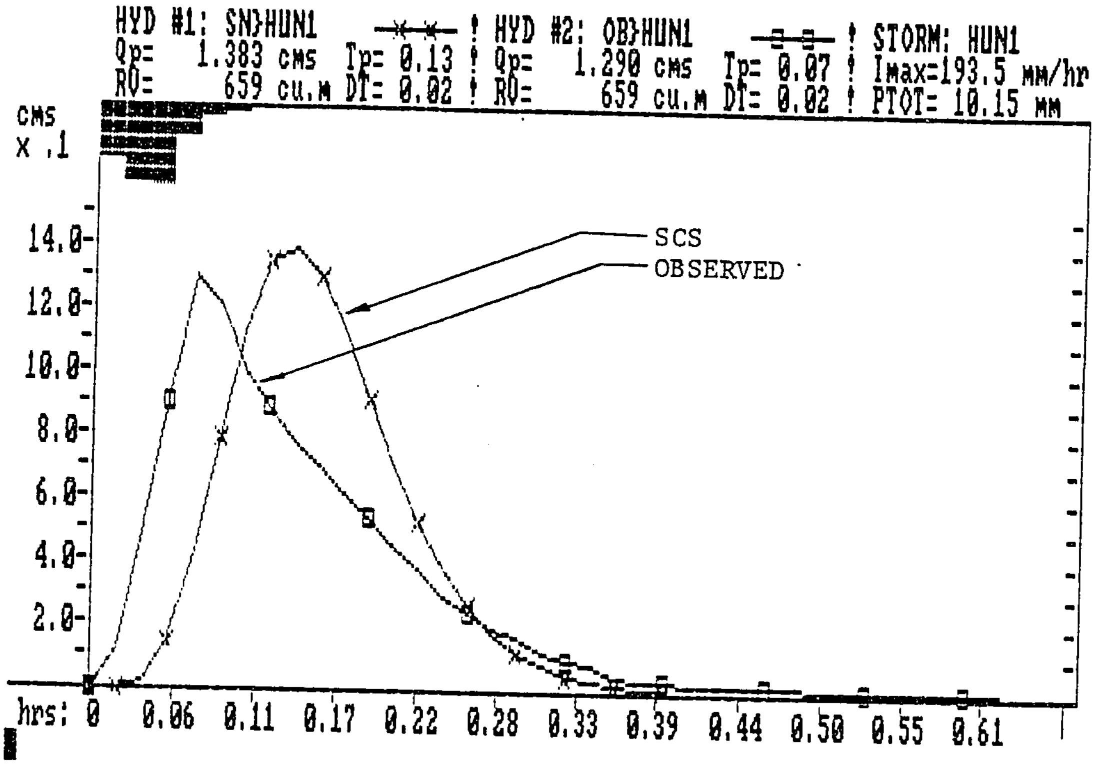


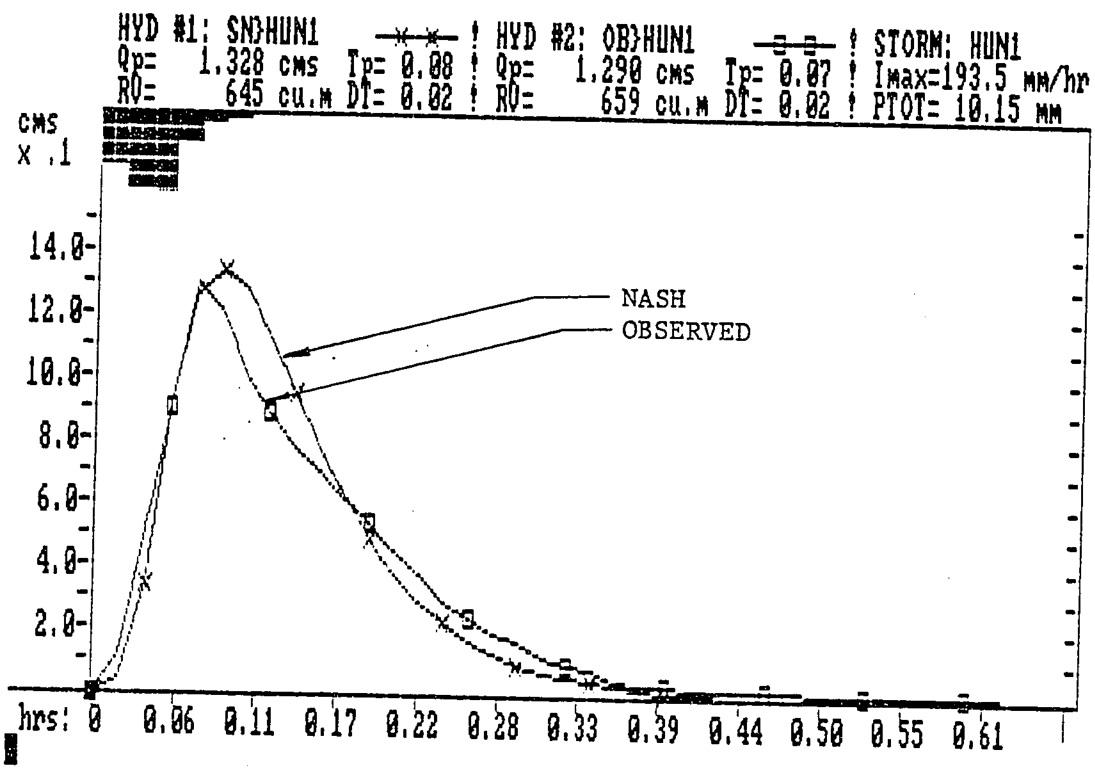
DENMARK EVENT 8



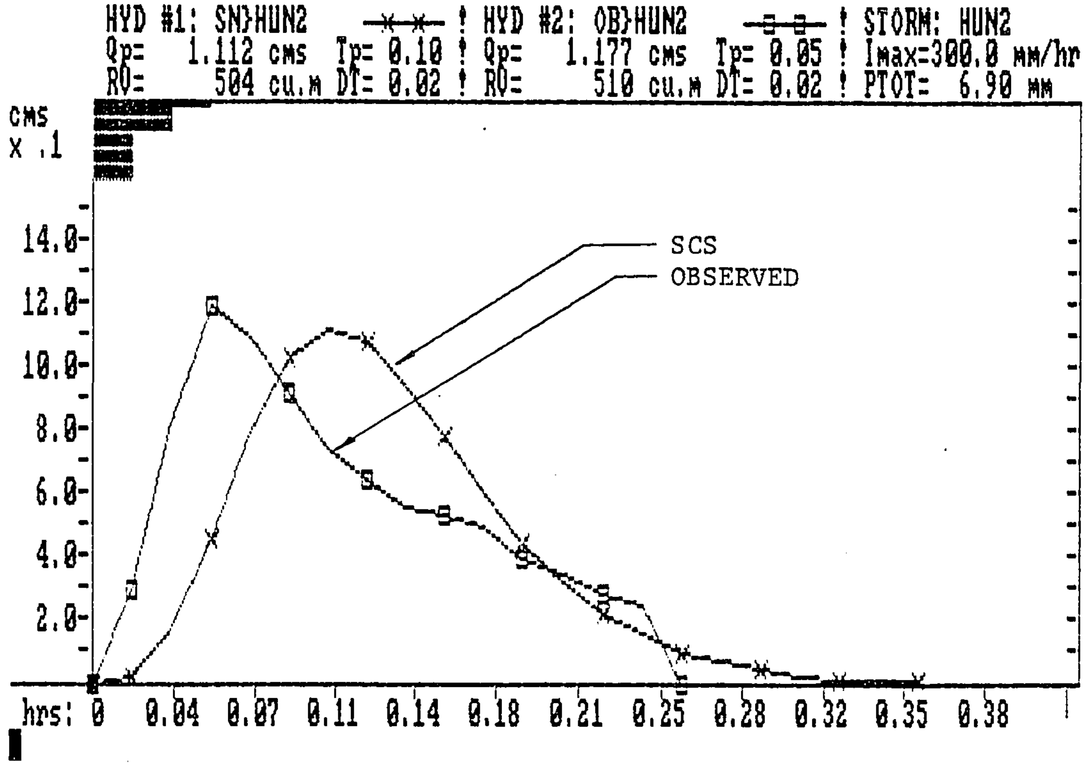


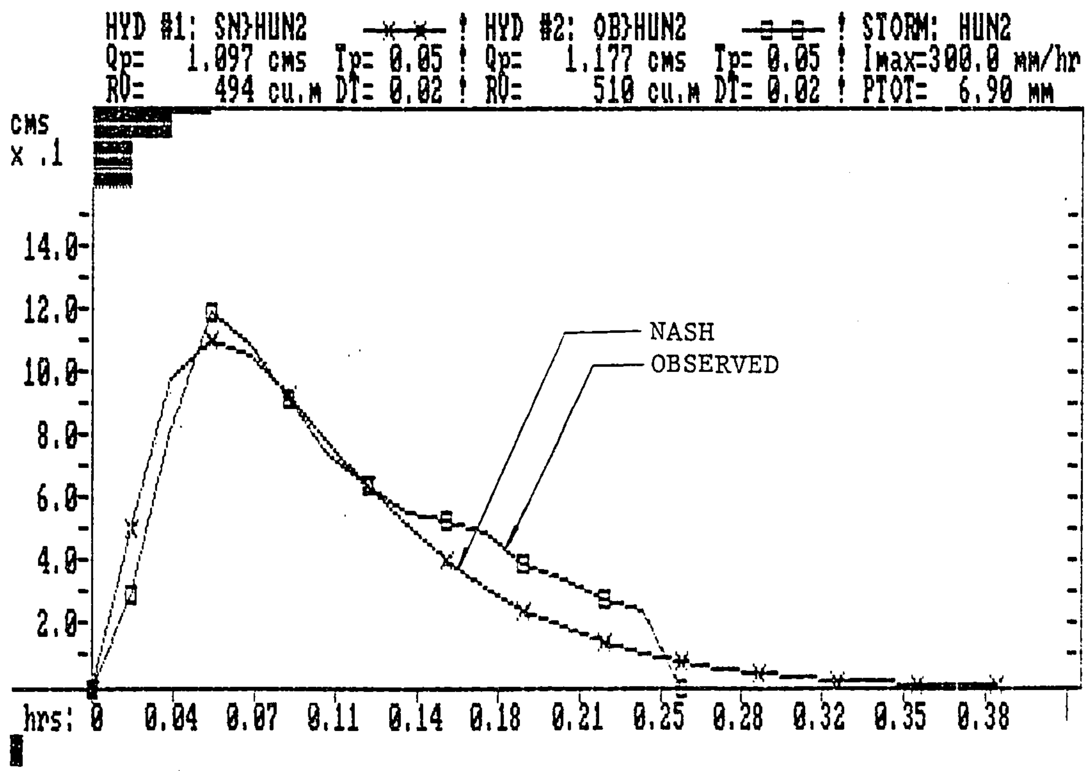
HUNGARY EVENT 1



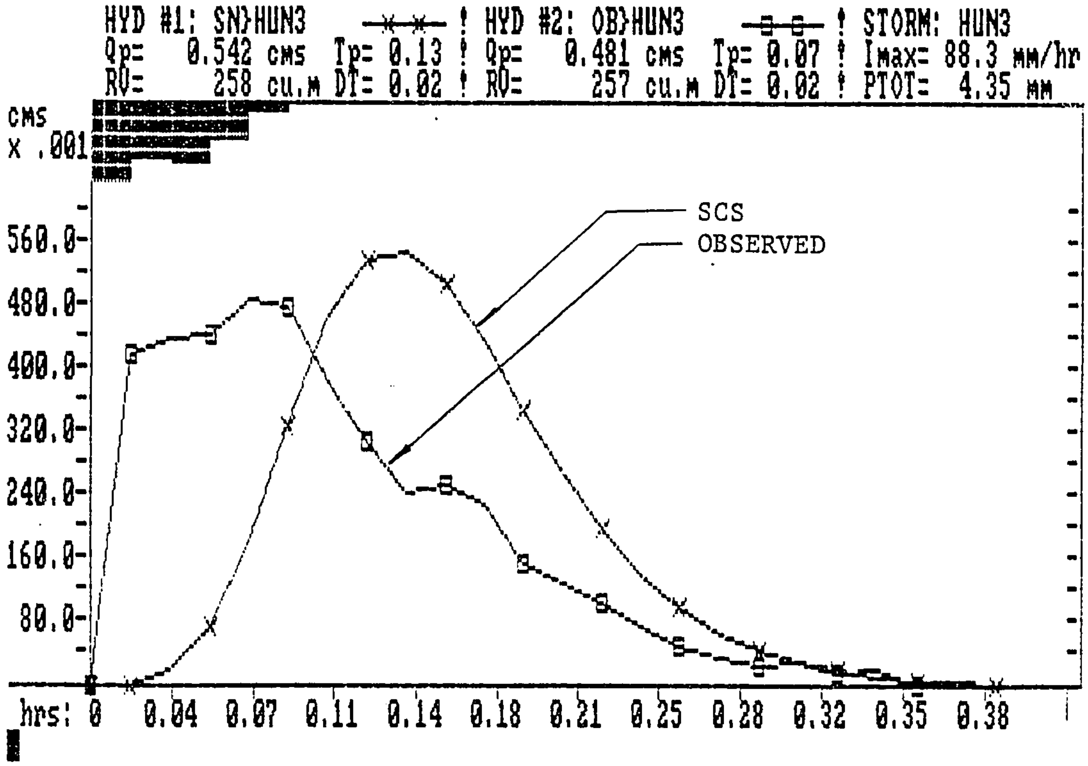


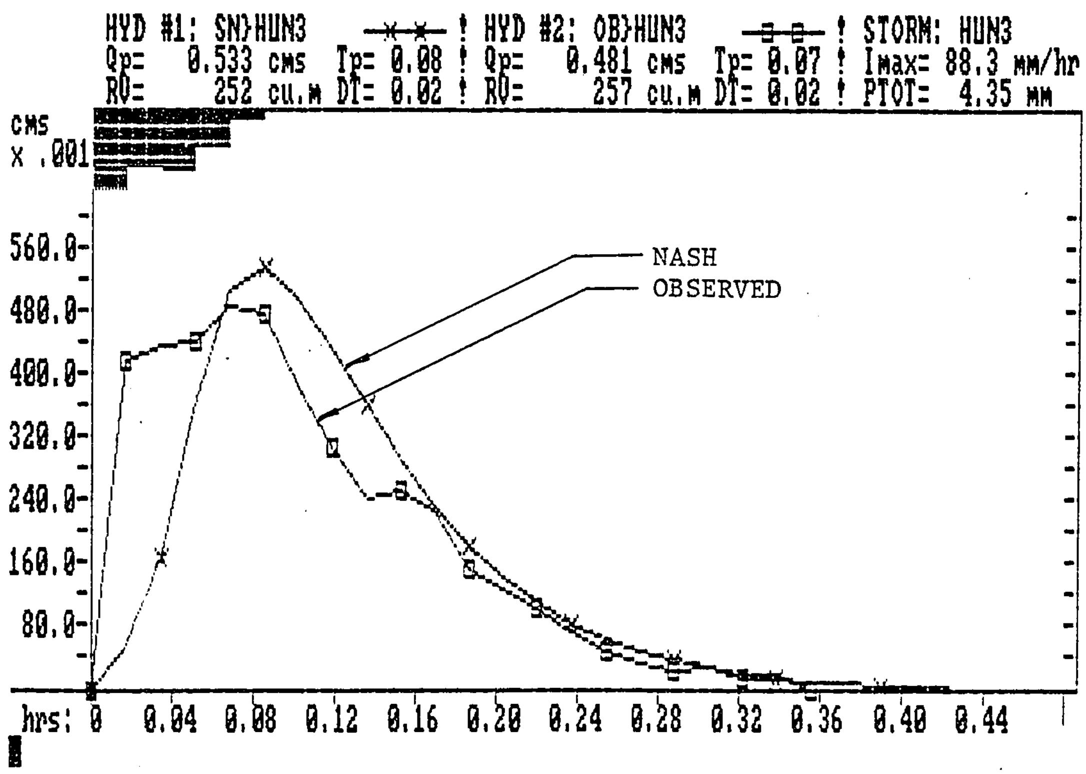
HUNGARY EVENT 2



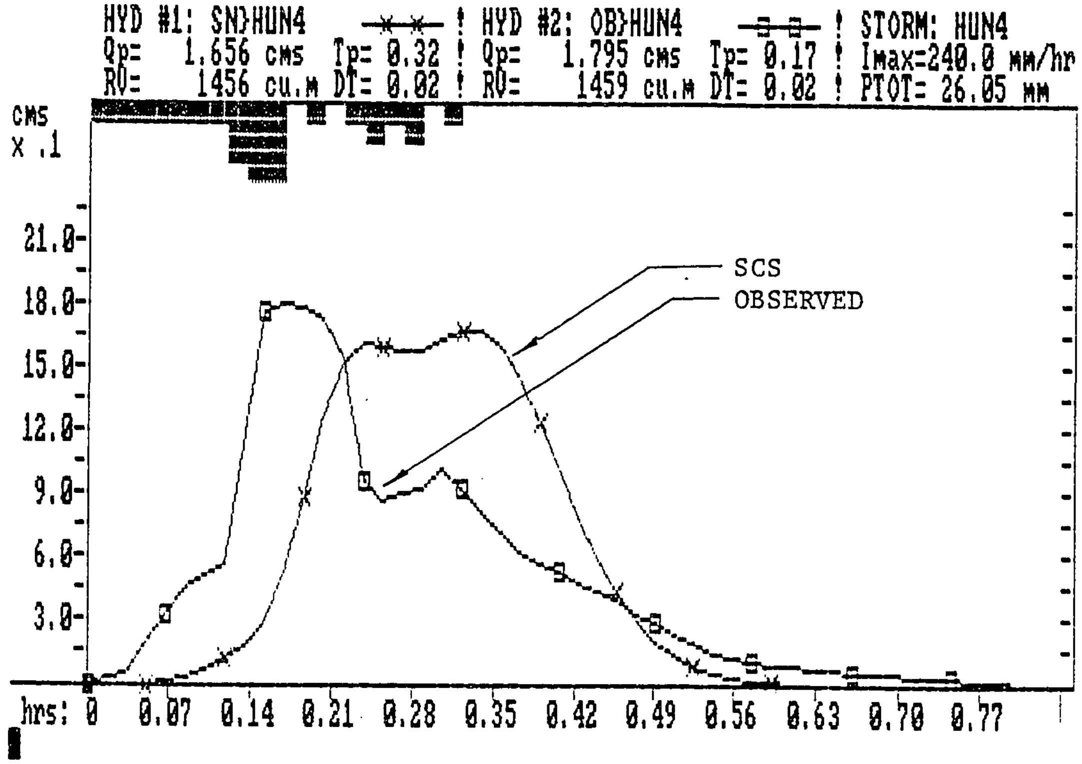


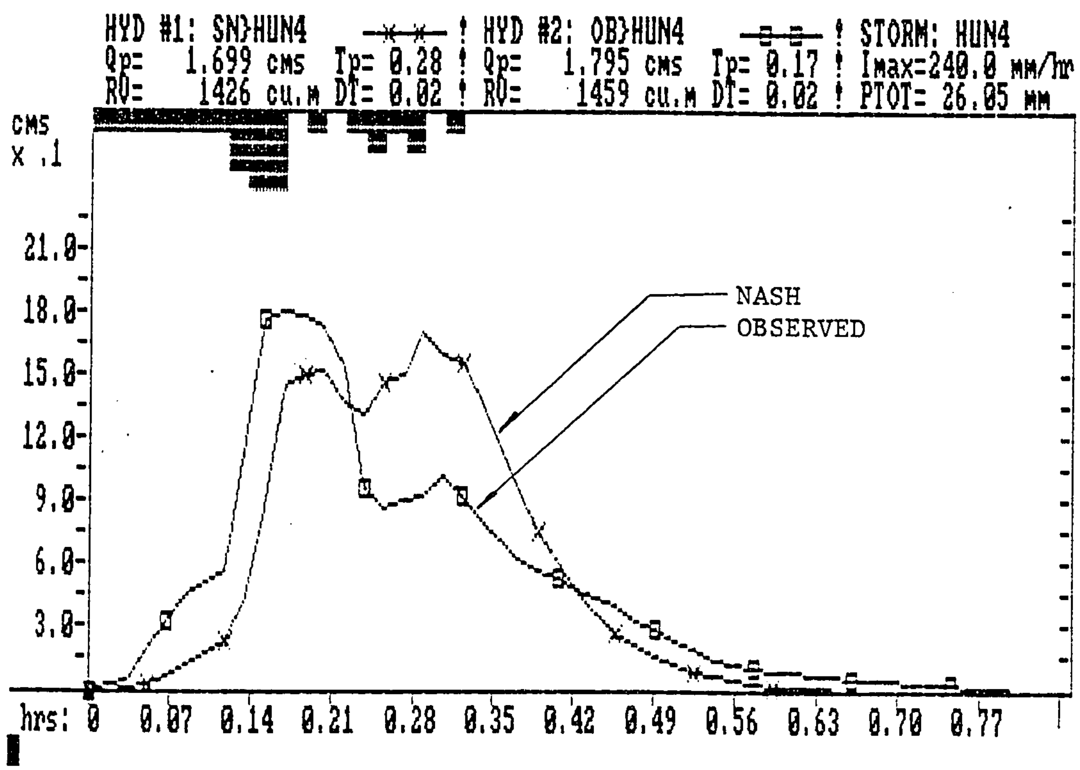
HUNGARY EVENT 3



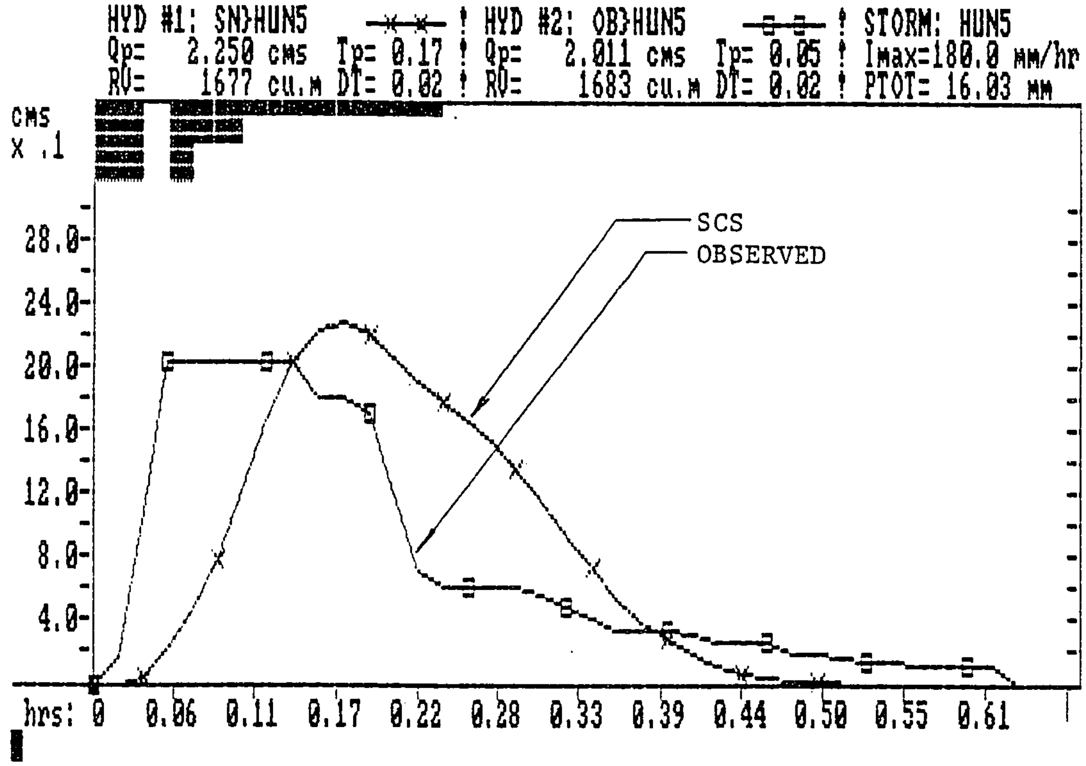


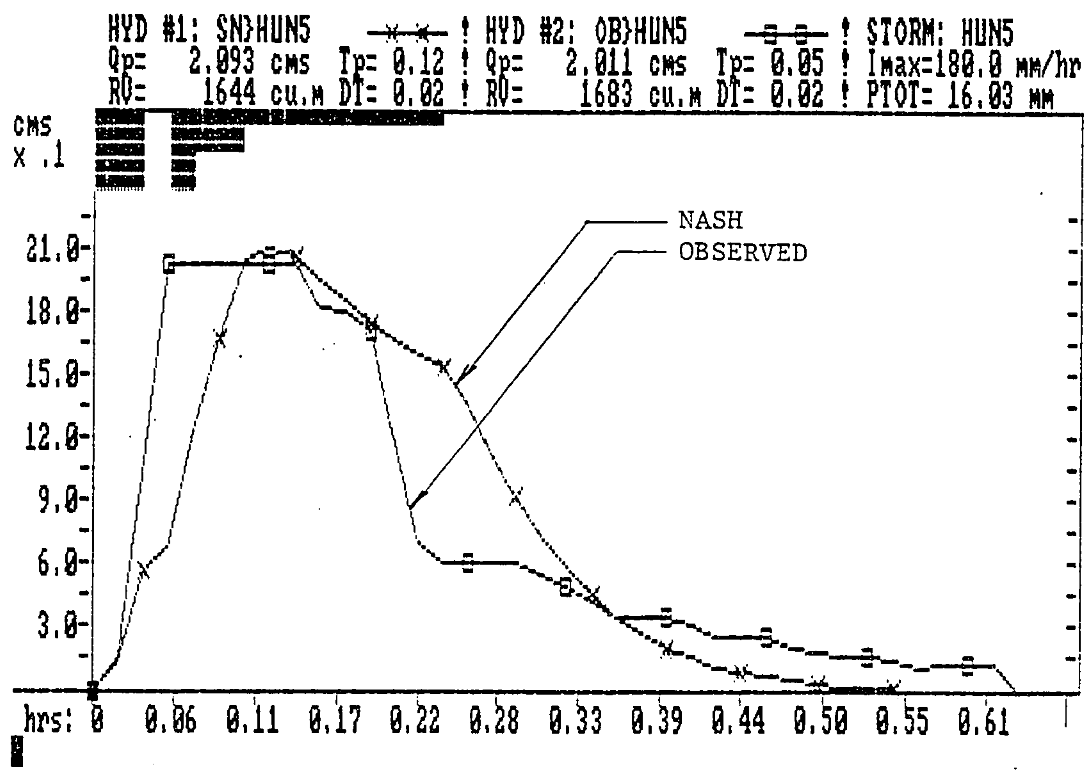
HUNGARY EVENT 4



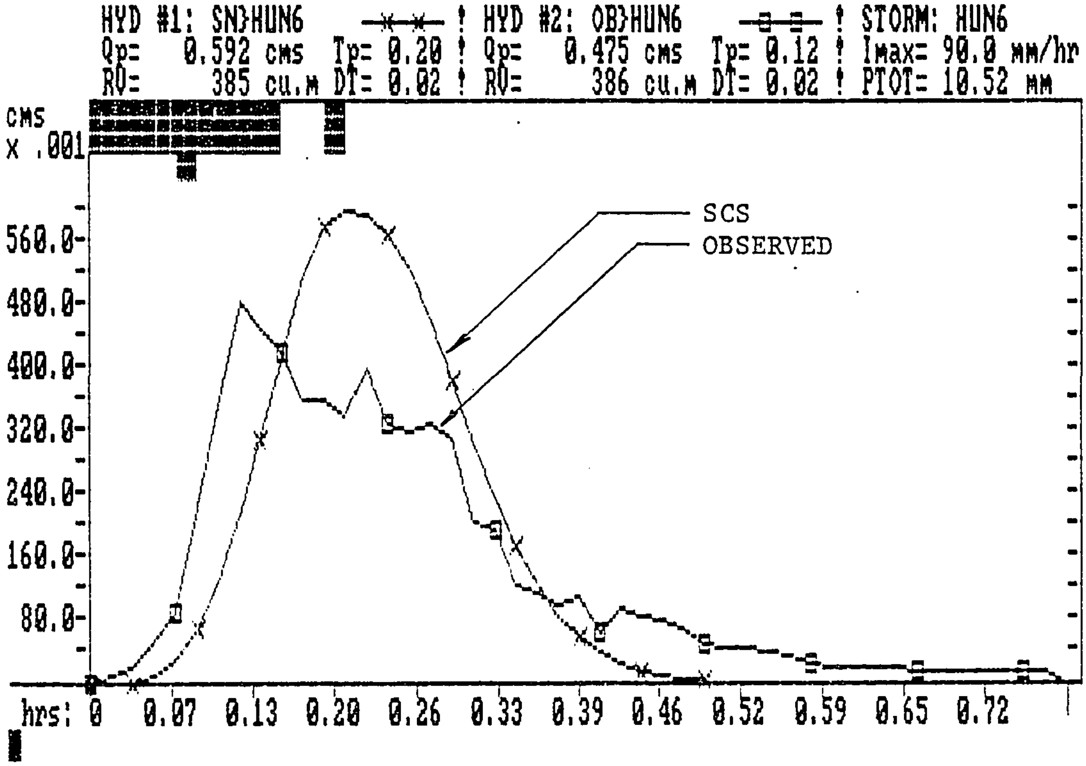


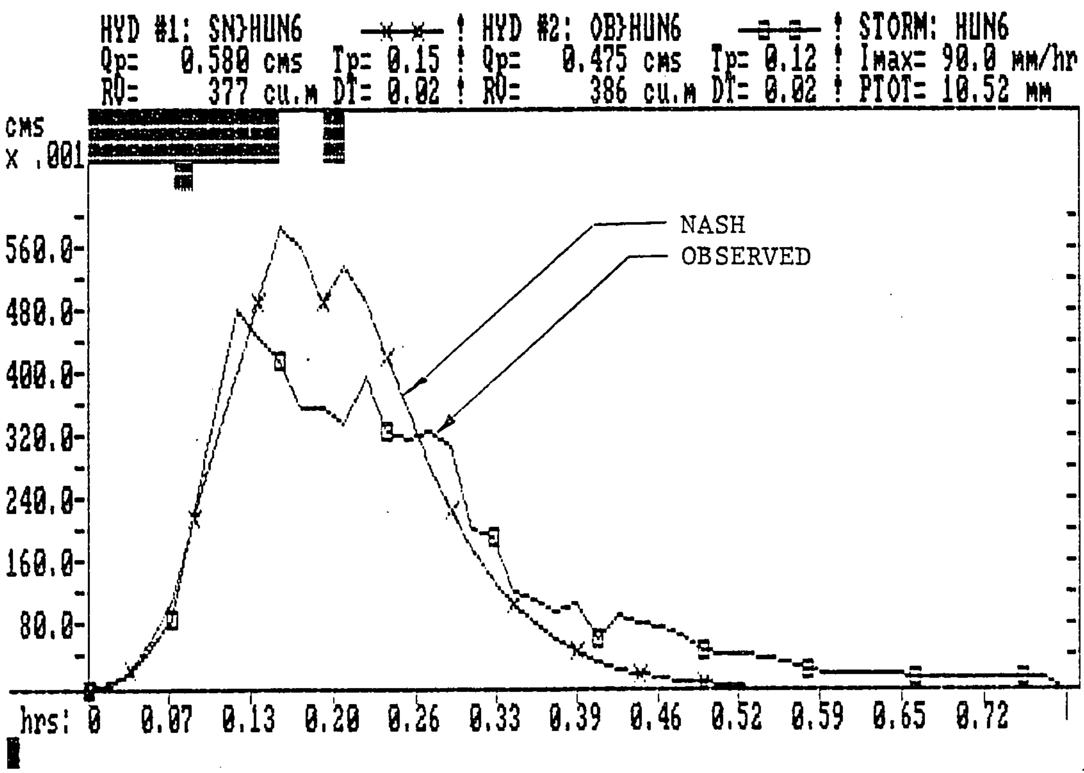
HUNGARY EVENT 5





HUNGARY EVENT 6

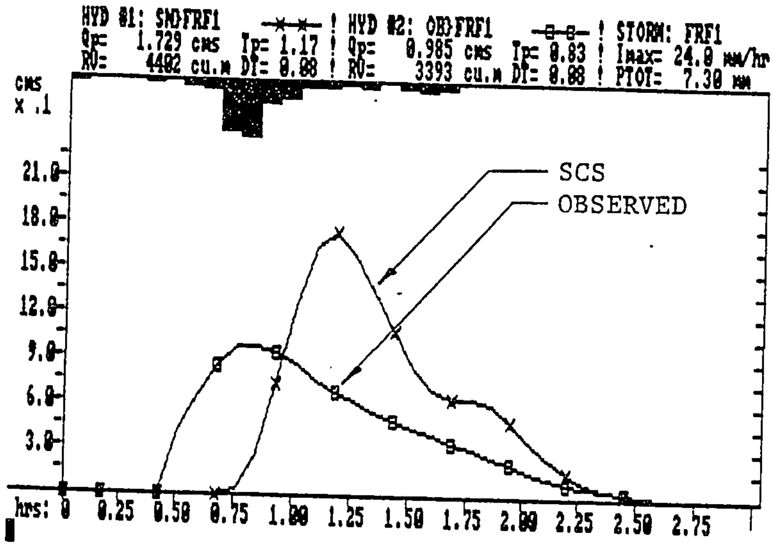


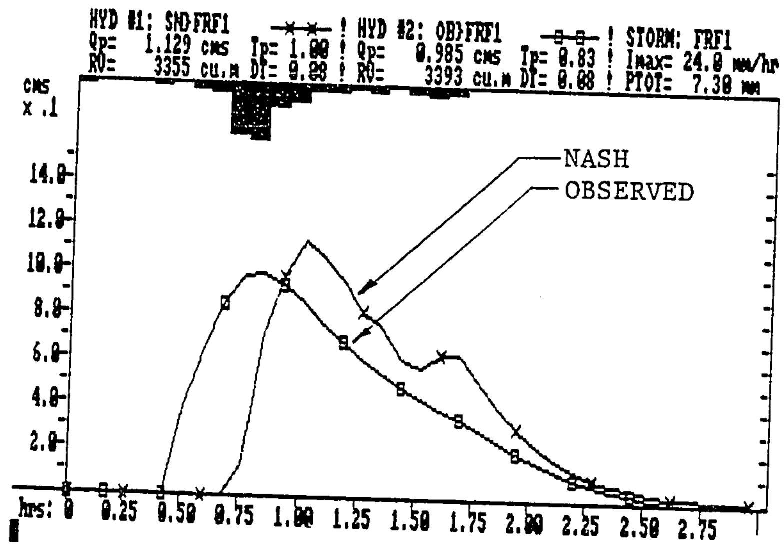


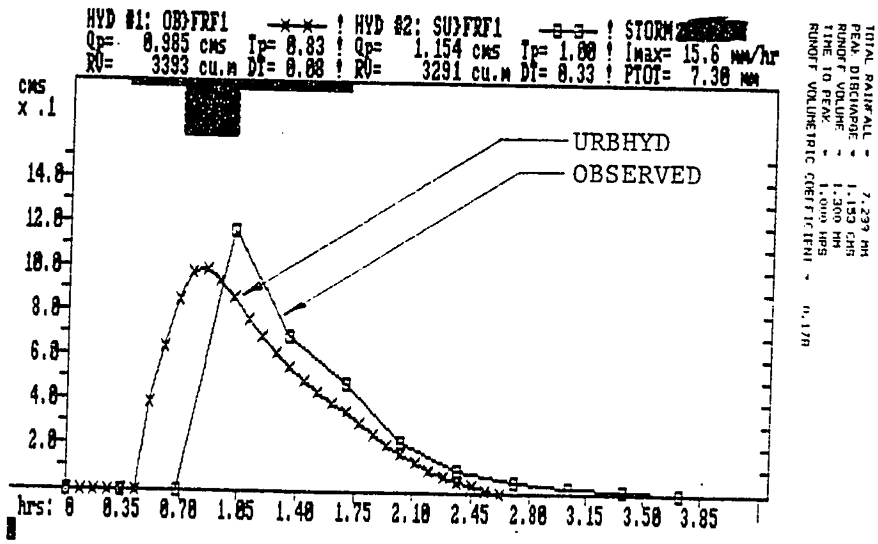
A P P E N D I X   B

URBHYD Simulation Results for the
St. Denis, Paris Watershed

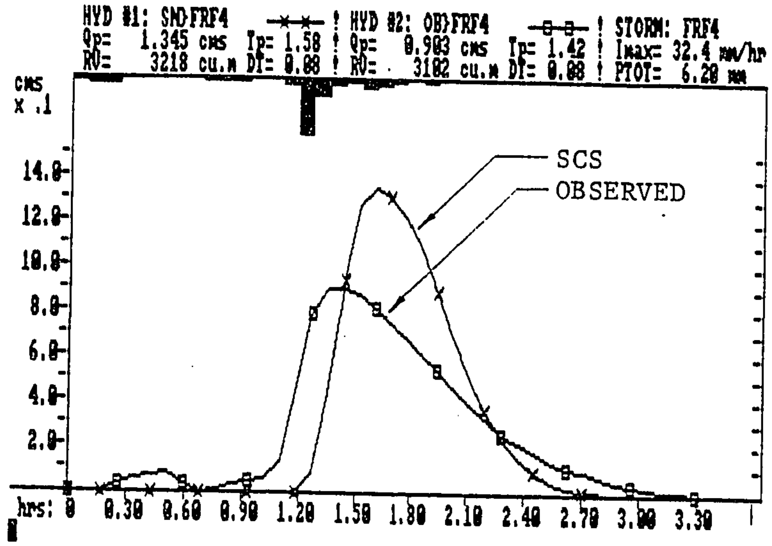
ST. DENIS - EVENT 1

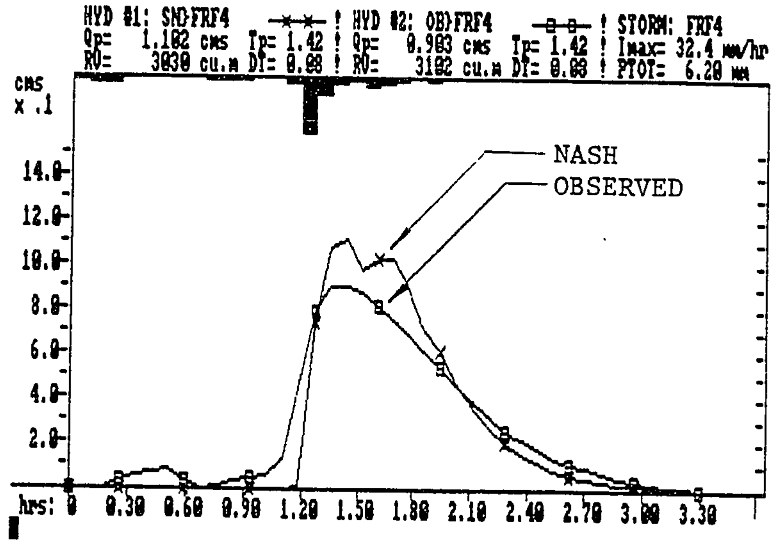


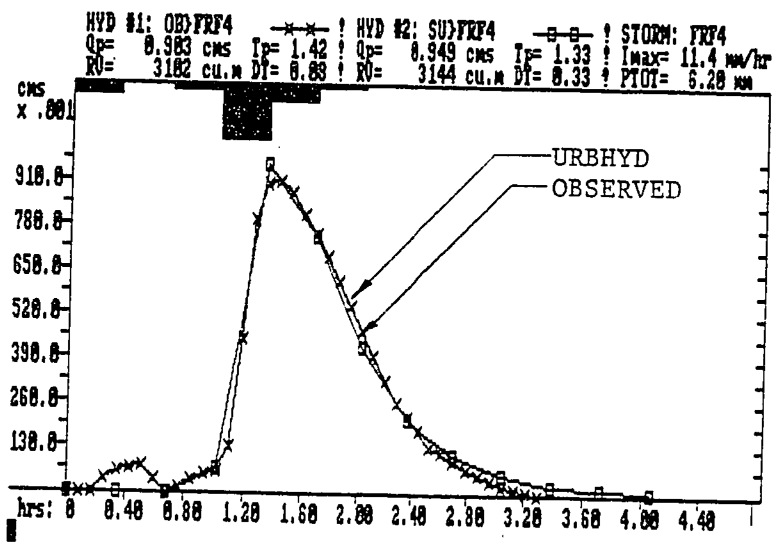




ST. DENIS - EVENT 4

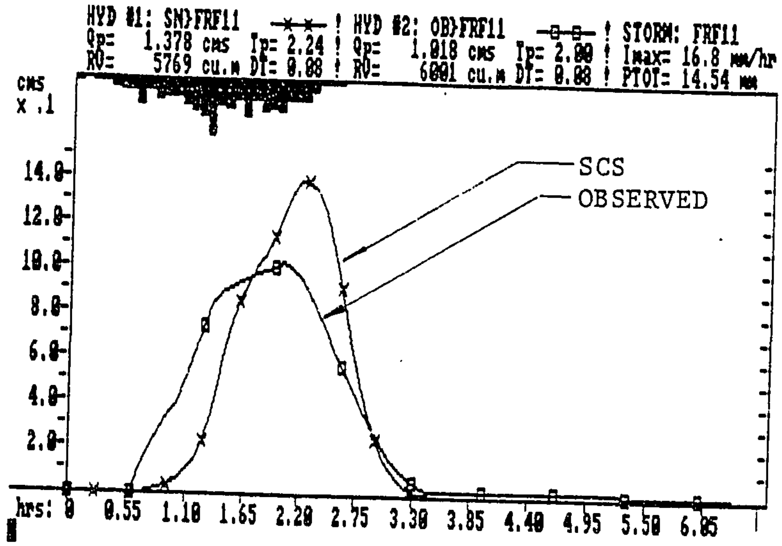


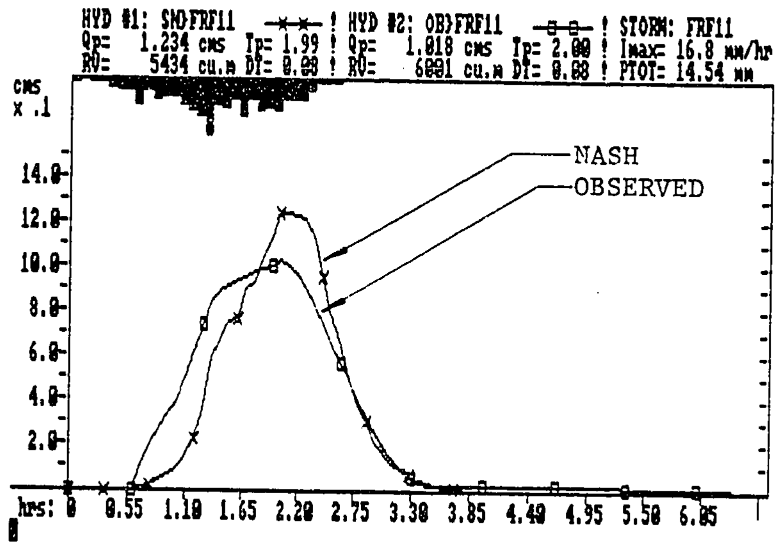


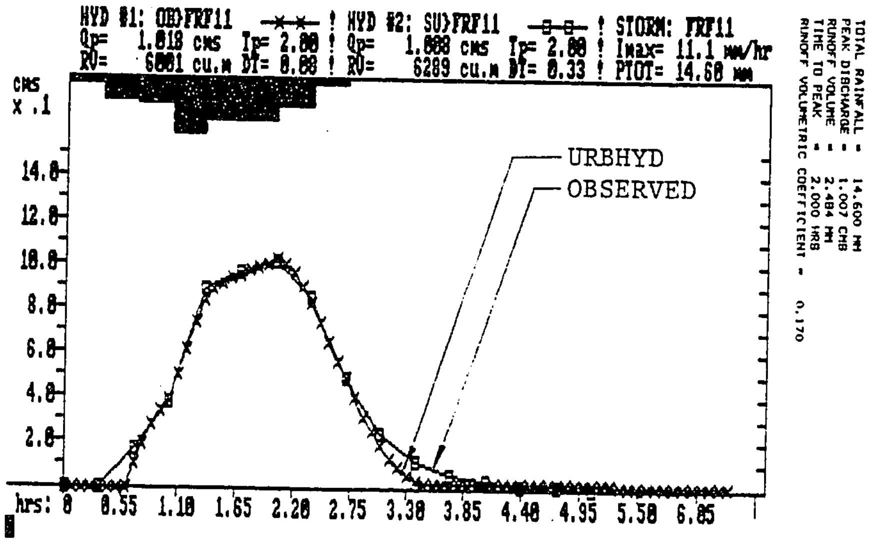


TOTAL RAINFALL = 6.199 MM
PEAK DISCHARGE = 0.949 CMS
RUNOFF VOLUME = 1.242 MM
TIME TO PEAK = 1.333 HRS
RUNOFF VOLUMETRIC COEFFICIENT = 0.200

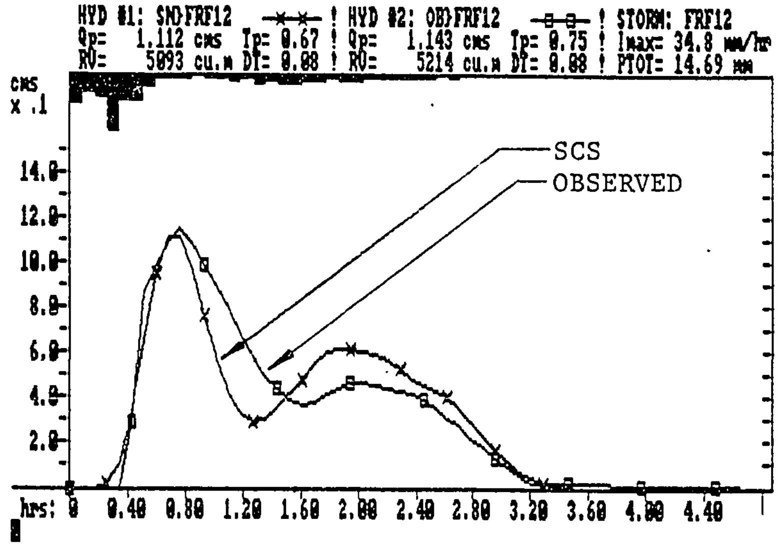
ST. DENIS - EVENT 11

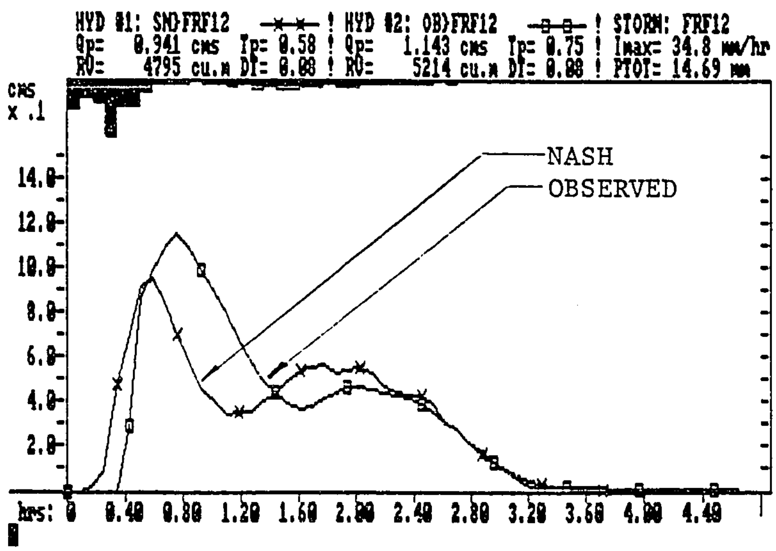


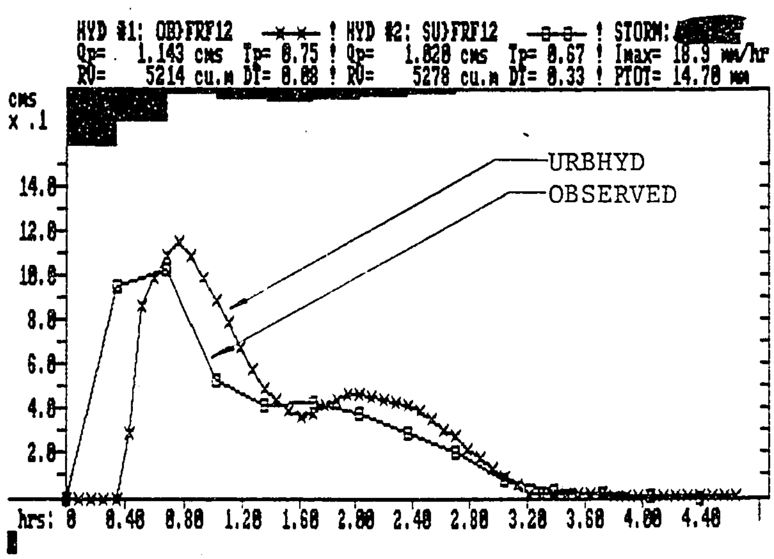




ST. DENIS - EVENT 12

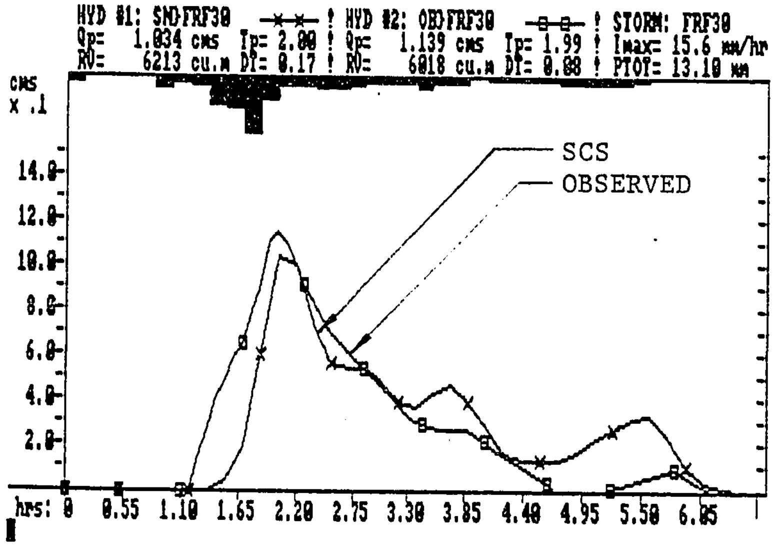


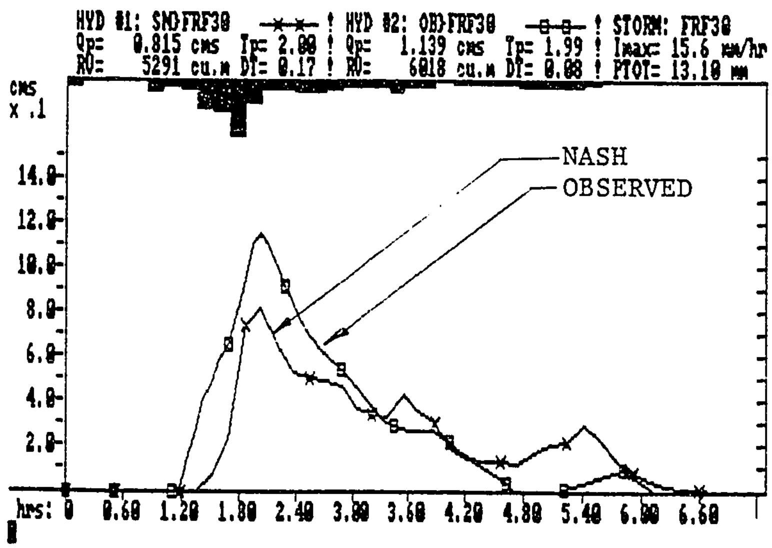


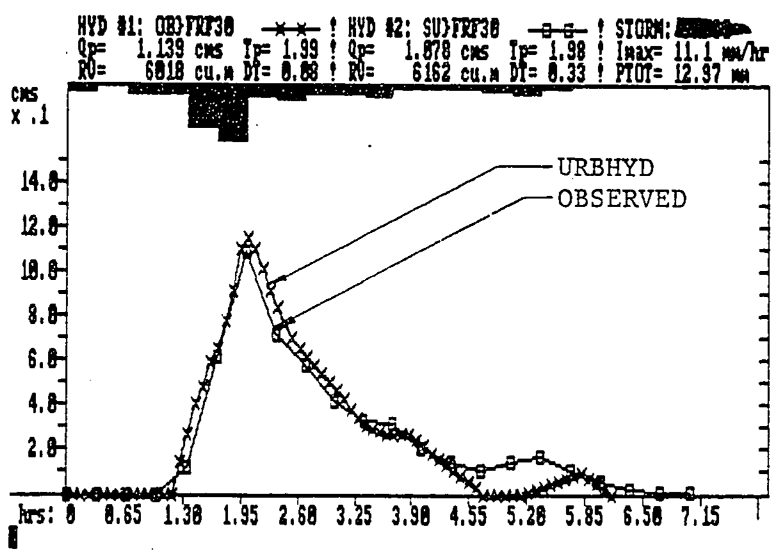


TOTAL RAINFALL = 14.699 MM
PEAK DISCHARGE = 1.019 CMS
RUNOFF VOLUME = 2.085 MM
TIME TO PEAK = 0.667 HRS
RUNOFF VOLUMETRIC COEFFICIENT = 0.142

ST. DENIS - EVENT 30





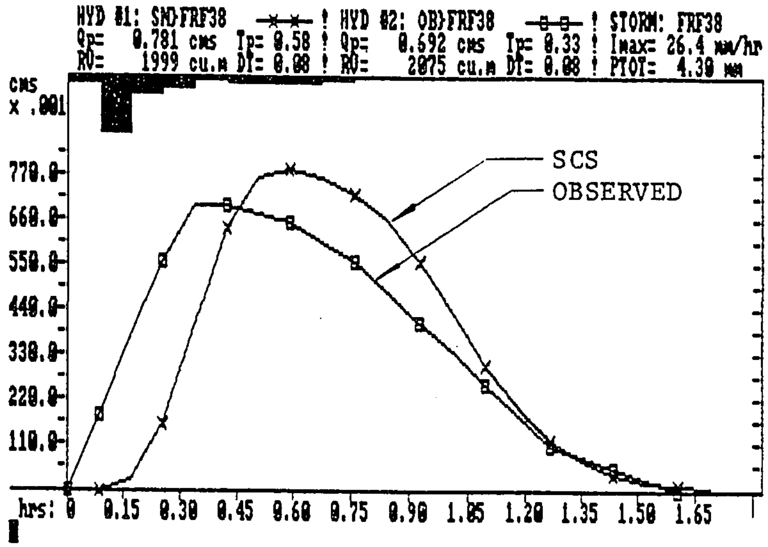


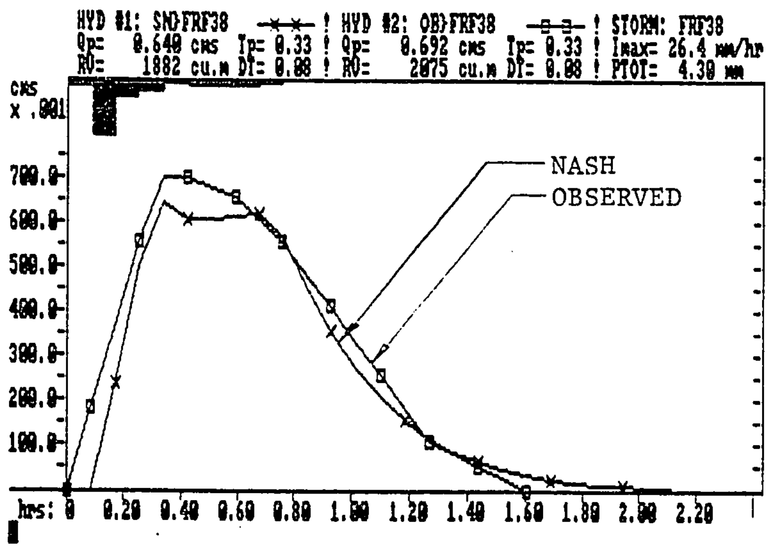
TOTAL RAINFALL = 12.969 MM
PEAK DISCHARGE = 1.078 CMS
RUNOFF VOLUME = 2.434 MM
TIME TO PEAK = 1.980 HRS
RUNOFF VOLUMETRIC COEFFICIENT = 0.188

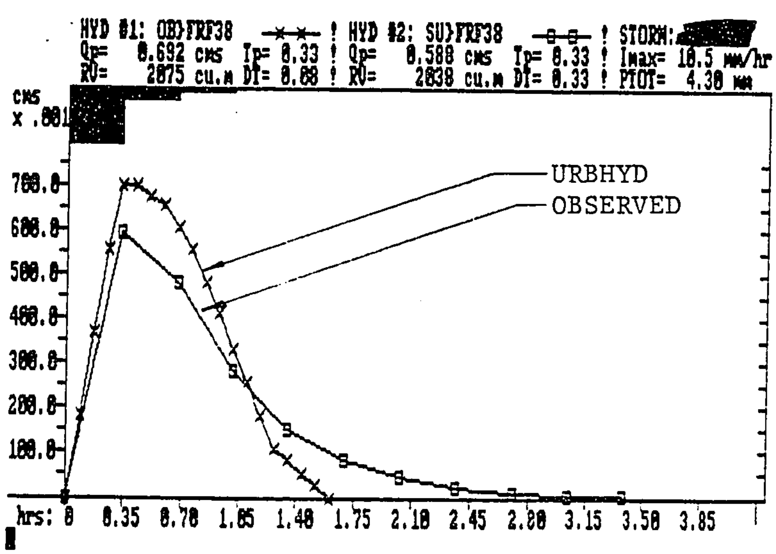
TABLE 4 - SCS AND NASH RESULTS FOR ALL EVENTS

CATCHMENT	EVENT NO.	OBSERVED Q_p (cms)	OBSERVED t_p (hr)	SCS (velocity method) Q_p	SCS (velocity method) t_p	OPIMIZED NASH Q_p	OPIMIZED NASH t_p
Malvern, Burlington	2	0.711	1.98	0.77	2.00	0.51	2.13
	8	0.135	1.88	0.18	2.17	0.16	2.26
	12	0.901	0.52	1.28	0.50	0.91	0.65
	15	0.140	0.77	0.18	0.84	0.15	0.92
	16	0.770	0.15	0.60	0.17	0.40	1.88
	21	0.428	0.15	0.36	0.57	0.28	0.69
	22	0.188	0.25	0.20	0.52	0.17	0.64
	23	0.201	0.18	0.15	0.52	0.12	0.62
	24	0.249	0.83	0.23	0.87	0.18	1.00
King's Creek, Florida	1	0.895	0.28	0.90	0.33	0.94	0.33
	2	0.753	1.25	0.72	1.16	0.72	1.25
	3	0.493	0.47	0.66	0.42	0.67	0.42
	4	0.441	2.97	0.51	2.74	0.50	2.74
	5	0.391	0.13	0.25	0.17	0.24	0.25
	6	0.451	0.88	0.40	0.42	0.37	0.50
Gray Haven, Baltimore	1	2.261	1.58	2.51	0.60	2.33	0.70
	2	2.212	1.58	2.28	0.65	2.07	0.70
	3	0.864	0.38	0.87	0.70	0.71	0.75
	4	0.828	0.38	0.77	0.70	0.68	0.75
	5	0.763	0.48	0.70	0.50	0.63	0.60
	6	2.466	0.22	1.63	0.20	1.53	0.35
Munkerisparken Denmark	1	0.103	1.17	0.079	1.05	0.063	1.10
	3	0.111	1.03	0.082	0.95	0.070	1.00
	5	0.129	1.47	0.089	1.40	0.072	1.45
	6	0.162	1.78	0.174	1.70	0.143	1.75
	7	0.063	0.22	0.057	0.15	0.052	0.20
	8	0.307	1.13	0.394	1.05	0.322	1.10
St. Denis, Paris	1	0.985	0.83	1.13	1.30	1.28	1.17
	4	0.903	1.42	1.10	1.40	1.29	1.58
	11	1.018	2.08	1.23	1.99	1.25	2.49
	12	1.143	0.83	0.94	0.58	1.11	0.67
	30	1.139	2.00	0.94	1.90	1.00	2.00
	38	0.692	0.42	0.64	0.30	0.81	0.58
Miskolc, Hungary	1	1.290	0.07	1.33	0.08	1.38	0.13
	2	1.177	0.05	1.10	0.05	1.11	0.10
	3	0.481	0.07	0.53	0.08	0.54	0.13
	4	1.795	0.18	1.70	0.28	1.66	0.32
	5	2.011	0.05	2.09	0.12	2.25	0.17
	6	0.475	0.12	0.58	0.15	0.59	0.20

ST. DENIS - EVENT 38







TOTAL RAINFALL = 4.300 MM
PEAK DISCHARGE = 0.588 CMS
RUNOFF VOLUME = 0.805 MM
TIME TO PEAK = 0.333 HRS
RUNOFF VOLUMETRIC COEFFICIENT = 0.187