MODELLING THE EFFECT OF URBANIZATION ON STORM FLOW IN THE BRAAMFONTEIN SPRUIT

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Modelling the effect of urbanization on storm flow in the Braamfontein Spruit

By P.KOLOVOPOULOS

A dissertation submitted in partial fulfilment of the requirements for the degree of Master of Science in the faculty of engineering of the University of the Witwatersrand. Johannesburg, June 1986.

DECLARATION.

I, PETROS KOLOVOPOULOS, declare that this is my own unaided work, and is being submitted for the degree of Master of Science in Engineering at the University of the Witwatersrand, Johannesburg. It has not been submitted before for any other degree or examination at any other university.

P.KOLOVOPOULOS

ACKNOWLEDGEMENTS.

I wish to thank my supervisor Prof. D.Stephenson for the unfailing support and interest shown in the project.

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DEDICATION.

This work is dedicated to my parents.

ABSTRACT

Urbanization of catchment areas can result in significant changes to the hydrologic response of the catchment. These effects usually manifest in an increase in the runoff volume, an increase in the peak runoff rate and a reduction in the catchment's response time.

This theoretical study provides guidance for those faced with the drainage problems which arise due to urban development taking place within rural or semi-urban areas. It describes the general problems posed by urbanization, identifying as the most significant physical changes brought about by urbanization, the increase in the proportion of impervious area and the alterations to the natural pattern of surface water drainage.

It also provides a guide for the calculation of the runoff from catchments that have been subjected to a degree of urban development, or for assessing the runoff in cases where future development is contemplated. For this purpose various computer simulation models are tested and compared. An example of drainage system designed with the traditional philosophy is analysed. The necessity for a dual drainage system design is demonstrated and a computer model based on the above concept is developed.

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1.0 INTRODUCTION

As our urban areas continue to expand and coalesce into metropolitan areas, the cost, damage and environmental effects of floods could become intolerable. Urban development eliminates most natural drainage systems and replace them by man-made stream-lined conduits. The hydrological cycle is thus affected. Storm water drains replace streams and the excess rain is no longer free to flow overland and meander along unlined channels. Instead, precipitation is on roofs or concrete or bitumen pavements. Urbanization reduces the average permeability of the ground by the construction of pavements and buildings. Because of the reduction of natural retention space of the flood plain, the flood wave is increased in amplitude and is accelerated. The concentration time is reduced due to increased runoff intensity, smoother surfaces and man-made channels. The design storm is therefore a shorter, more intense storm than that for no development. Natural basins or depressions may be levelled, thereby increasing excess runoff even further.

Before the era of environmental concern, the effects of urbanization were largely ignored in applying traditional drainage methods. Increased runoff as a result of urbanization was in fact conveyed by deepening and lining existing channels or enlarging pipes and culverts. This approach in time contributed to greater increases in runoff velocity and peak runoff. However, methods have changed significantly, with engineers now possessing modern techniques for the design of drainage systems. Recognition of dual drainage systems has led to a new design policy and a two-stage approach.

An important consideration in the design process of urban drainage facilities is an appreciation of the effects of future urbanization on runoff patterns. The engineer needs to fully understand the runoff process in addition to the factors that are pertinent in urbanization in order to plan stormwater drainage facilities correctly.

An important tool in the above analysis is the simulation model. Simulation programs are justified by the improvement they achieve with repetitive analysis. Not only do they enable discharges to be calculated with greater accuracy than other simplistic methods, but they also permit sensitivity studies. Major effects of urbanization as the increase of the impervious cover, canalization and reduced roughness can most readily be studied by modelling. Most of the available urban drainage models are deterministic single event models such as SWMM,WITWAT II, OTTHYMO etc. In this report use is made of three computer models:

1. SWMM (Storm Water Management Model, Version II), (Huber et al. 1982). It is possibly the best known hydrological simulation model in N.America. It has extensive data requirements and is divided into a number of blocks, or subroutines, each performing a separate task. The flow routing procedure is generally based on a numerical solution of the kinematic equations.

2. WITWAT II (Green 1984) is a micro-computer model, very friendly to use, developed by the Water System Research Program (WSRP). The model is also based on kinematic theory.Depth-discharge relationships are based on steady-flow discharge formulae such as that of Manning.

3. OTTHYMO model (Wisner 1980) is a lumped model (needs less detailed information than others). This model is the University of OTTAWA version of the HYMO model (J.Williams and R.Hann 1973). It employs unit hydrograph theory and for channel routing the Muskingum method is used.

All the above-mentioned models are used in this report. An attempt has been made to emphasise that models should not be treated as 'black boxes' giving answers, with little awareness of their limitations and constraints. The engineer should always be aware of the logic and the assumptions of the program and the range of applicability. This approach, in which one seeks to construct a model that responds to input in the same way as a physical system, is the only possible one particularly if the system is poorly understood.

The aim of this study is to understand the effects of urbanization and to develop tools and guidelines by which to assess them. The objectives are as follows:

1. To provide a general introduction to the problems posed by urban development within a semi-urban area

2. To indicate the impact such development may have on drainage in its immediate vicinity and downstream.

3. To provide information on the correct design alternatives which can be adopted to alleviate the problems caused by such development by illustrating the negative consequences of the traditional design and the necessity for dual analysis.

4. To develop a planning model for a more accurate prediction of the flood hydrographs in urbanized areas.

The methodology used to achieve the above-mentioned objectives is outlined below:

1. Urbanization is interpreted as causing changes in different factors such as imperviousness, canalization, roughness, infiltration etc. For studying these factors the WITWAT II model is used. The study area used was the Upper Braamfontein Spruit as it is a catchment gauged by the WSRP with available

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runoff and rainfall data. The Hillbrow catchment is a fully developed urban area comprising high-rise buildings and some high density housing development.

2. The drainage system of a large urban watershed, the Upper Braamfontein Spruit is analysed. The reason for the selection of this watershed is that this drainage system is a typical example of traditional design. It is overdimensioned and still overloaded. The necessity of stormwater drainage methods to include the 'dual' nature of every stormwater system is emphasised. The 'dual' character arises because stormwater drainage systems normally consist of a 'minor' or underground pipe system and a 'major' or surface overland flow (Binney, P D. 1981). This concept was ignored in the design of the drainage system of Braamfontein Spruit.

3. To design and evaluate the performance of dual drainage systems, runoff quantity should be accurately modelled. Since no existing model in the WSRP had this capability a new model was developed i.e. WITWAT III. The structure of the model has been based on the paper by Alley at al.(1980) and employs the friendly input and structure of the WITWAT II model.

4. The model is tested and compared with WITWAT II using a hypothetical catchment. Several runs are also performed in order to test the sensitivity of the new model to different parameters and the results are compared with SWMM and WITWAT II models. The Upper Braamfontein Spruit (Hillbrow) catchment is used again for the simulation and calibration of the model. The catchment is also analysed for severe events in order to simulate the response of the major system.

Much has been written on urban hydrology in general and on the effects of urbanization in particular. However, remarkably little quantitative information is available on the broad implications of the effects of urbanization on water resources management.

Many studies that have been conducted by Robey (1970), Davis (1974), Miles (1984) and others provide evidence on the effects of urban development. Some of the results of these studies are also described in the next chapters and a revision of their conclusions is attempted.

In order to assess the effects of urbanization, a direct method would be the comparison of identical catchments or a catchment to be monitored before and after urbanization. Such a program would take years to produce results in addition to being difficult and protracted. Although modelling is susceptible to errors, with the available data it was the only way to separate different urbanization parameters. It is believed that the present study provides for a better understanding of the effects of urbanization. It also produces a model capable of simulating the real runoff process more accurately than was previously possible.

2.0 EFFECTS OF URBANIZATION ON STORMWATER DRAINAGE

2.1 INTRODUCTION

The reasons for possible changes in runoff due to urbanization are many and varied. However the major effects of urbanization are:

1. Increase of the the impermeable cover
2. Canalization

The reduction of infiltration and initial abstractions are not considered as direct effect of urbanization. Rather they are result of the increase in impervious area which inhibits infiltration and leads to larger volumes of direct runoff. The connection of the impervious area directly to the drainage system eliminates the chance of seepage. The decrease in the roughness is due both to canalization and the increase of impervious surfaces. With the construction of roads, pavements and buildings the natural retardation of the surface runoff is eliminated and concentration time reduces. Storm water is not routed over the soil where the roughness ranges from 0,030 to 0,450 but through roads and pavements (roughness range: 0,012-0,018) and is collected from channels and small pipes with reduced roughness (roughness less than 0,014).

The actual measurement of the effects of urbanization on runoff is difficult. Any of the possible methods of assessing the effects takes years of measurements. At present such measurements are not available and the most convenient catchment being monitored by the WSRP for this study was the Upper Braamfontein Spruit catchment (Hillbrow). The effects of urbanization on runoff will be illustrated using this catchment.

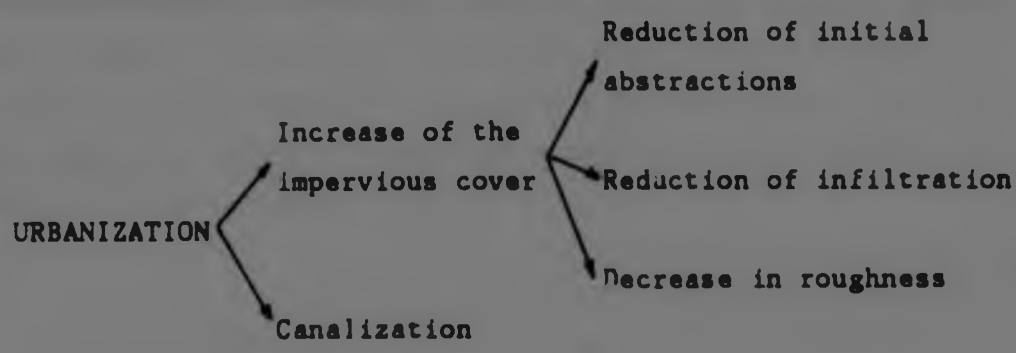

Figure 2.1 Effects of urbanization

2.2 METHOD OF APPROACH

The suburb Hillbrow, situated in the Upper Braamfontein catchment is a high density urban area and therefore yields high runoff. Rainfall and runoff data were available for only 4 events. Because of the small number of storms that were available the validity of the records could not be checked in order to discard the minor events. All rainfall data were considered reliable and were used for calibration of the model.

The runoff hydrographs at the outlet of the catchment were calculated by two methods: the SWMM model and the WITWAT II model. SWMM model is a main frame model popular in the USA, while WITWAT II is a micro-computer model developed by the WSRP (Green 1984). Exactly the same data were used for both the models. The reason for simulation of the catchment with two models was to gain the necessary experience in order to reach conclusions about the relative importance of the parameters and to test the performance of each model. Since the model was calibrated on the urbanized catchment using available storms, selected factors that represent urbanization are changed in order to break down the catchment to the pre-development situation. These changes are applied gradually in order to identify the significance of each factor. Even if both models compared favourably with the measurements, for the above analysis WITWAT II was used since it is more friendly to use and P.C. orientated.

2.3 MODELS STRUCTURE

Both models consider each subcatchment as a plane surface of spatially constant characteristics, such as infiltration rates, depth, ground slope and roughness coefficient. The overland flow hydrograph for the subcatchment is determined by a series of time-steps of average rainfall and outflow values. The excess flow depth (after subtraction of infiltration and depression storage) is assumed constant over the flow plane for a given time-step and is used to calculate the rate of overland flow per unit length (for WITWAT II) or width (for SWMM) of the subcatchment.

Some of the parameters of the models can be determined from physical data which can be measured, such as the roughness coefficient of a pipe. However, because the irregular shape of the watersheds is replaced by equivalent plane surfaces many factors have a high indeterminate component (Heeps et al. 1974).

The subcatchment's shape is a very important parameter because it can also alter the shape of the hydrograph, rather than just the runoff volume. The effect of increasing the runoff width in SWMM which is the same as decreasing overland flow length in WITWAT II for a constant area subbasin causes a quicker hydrologic response. This happens because increasing the width effectively provides a shorter flow path and greater cross-sectional area for outflow from the subcatchment, thus increasing the magnitude of the peak flow and decreasing the time to peak. Decreasing the width has the opposite effect, and the subcatchment surface acts more as a reservoir, reducing and delaying the peak. However for rainfall duration greater than the time of concentration the magnitude of the peak is not greatly affected.

It must be noted that the slope is not always a physical parameter because of the irregularity of an area. In the SWMM model the width, slope and roughness parameters are combined into one parameter. Thus, equivalent changes may be caused by appropriate alterations of any of the three parameters. The effects of decreasing Manning's roughness are nearly identical with the effects of increasing overland slope.

2.4 DESCRIPTION OF THE HILLBROW URBAN TEST CATCHMENT

2.4.1 CATCHMENT TOPOGRAPHY AND CHARACTERISTICS

The catchment is very densely developed with high rise buildings. The total area is 66,83 ha of which 51,75 ha (77,44%) is directly connected impervious surface, comprising roads, sidewalks, car parks, office blocks, and shopping complexes. The remaining area comprises lawns, unpaved parking areas and some small buildings that discharge on to pervious areas. The catchment slopes from the east corner towards the drainage outfall located in the west corner. The average catchment slope is 3,5%, but local slopes vary from 2% to 8%. Approximate ground level contours are shown in Figure 2.2

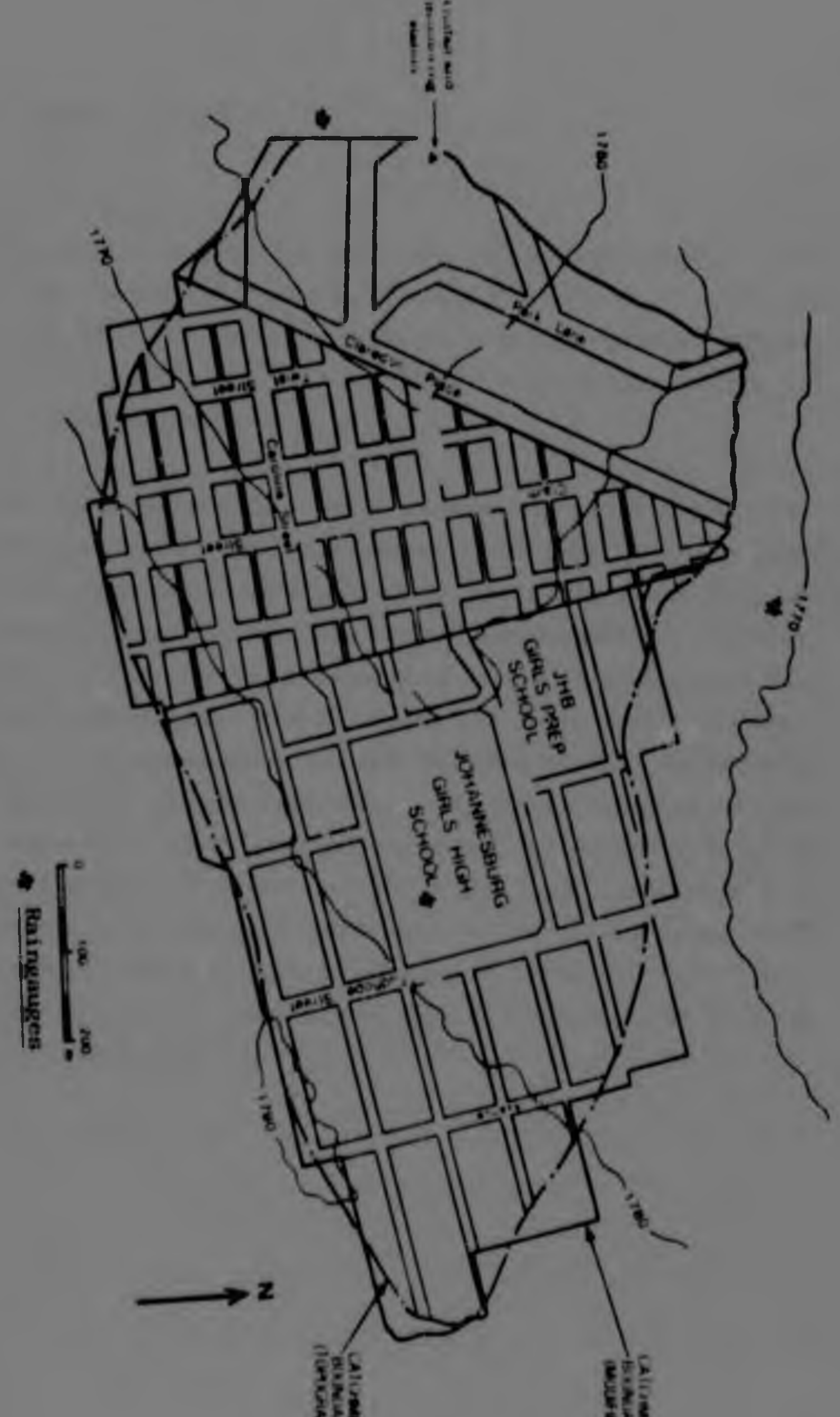

Figure 2.2 Hillbrow catchment

2.4.2 SEWER SYSTEM

The drainage system consists of concrete pipes and channels. Flow times were estimated assuming Manning (n) values of 0,012 for pipes and 0,014 for concrete channels. Sizes, lengths, slopes, capacities of pipes and channels used in runoff simulations are listed in Table 2.1.

In order to avoid possible sewer surcharging which would occur in model simulations but not necessarily in real life, the diameters of some pipes were increased along the route from the subcatchment inlet to the downstream subcatchment. The surcharging would be due to an assumption in runoff simulations that the total subcatchment runoff enters the sewer system through a single inlet located close to the centroid of the subcatchment area. At that point, the sewer pipe would not be designed to carry the total flow, since in the real sewer system water flows in through a number of inlets and only the very downstream pipe section needs a capacity sufficient to carry the total flow. Consequently, the diameters of the sewer pipes no. 1 to 9 were increased from 0,30m to 0,45m and pipe no. 10 from 0,45 to 0,60m diameter respectively.

Conduit no.	Drains to node	Pipe (1) or Channel (2)	Pipe diameter Width	Height	Roughness (n)	Slope (m/m)	Length
1	2	1	0,45*	0,00	0,012	0,027	187
3	2	1	0,45	0,00	0,012	0,010	108
2	4	2	0,70	0,70	0,014	0,019	183
4	5	2	0,70	0,70	0,014	0,019	108
6	7	1	0,45	0,00	0,012	0,060	75
7	8	1	0,45	0,00	0,012	0,036	70
8	10	1	0,45	0,00	0,012	0,027	225
9	10	1	0,45*	0,00	0,012	0,050	95
10	12	1	0,60*	0,00	0,012	0,022	93
5	11	2	0,70	0,70	0,014	0,027	130
11	12	2	0,70	0,70	0,014	0,029	105
13	12	1	0,45	0,00	0,012	0,021	95
12	14	2	2,00	1,50	0,014	0,029	158
14	15	2	2,00	1,50	0,014	0,024	125
15	16	2	2,00	1,50	0,014	0,029	138
16	99	2	2,00	1,50	0,014	0,029	140

* the real diameters of these pipes were 150mm less

Table 2.1 Conduit data

2.5 RAINFALL AND RUNOFF DATA

2.5.1 INSTRUMENTATION AND DATA COLLECTION

Rainfall records were obtained from three tipping bucket gauges recording every 2 minutes. Two were located within the catchment and one adjacent to the catchment as shown in Figure 2.2. Runoff was monitored by means of a stage-recorder located at the catchment outlet. Computer runs were made for all the available storms using the SWMM and the WITWAT II models. However, because the WITWAT II model does not accept rainfall data from more than one rain-gauge, weighting factors were specified for each gauge to determine basin-wide average rainfall in the watershed. These factors were determined by the Thiessen method. With the SWMM model there is not such a problem because data for up to 6 rain-gauges may be entered.

2.5.2 RAINFALL AND RUNOFF FLOWS

Because of the small number of storms that were available the validity of the records could not be checked. The storms could not be divided for validation and they were all used for calibration Basic characteristics of these 4 storms events are listed in Table 2.2.

Storm No.	Date	Duration (min)	Total Rainfall(mm)	Total Runoff(m3)	Total Runoff / Total Rainfall
1	16/12/83	60	16,42	8276	0,75
2	22/12/83	42	11,00	2421	0,33
3	30/12/83	30	5,68	2690	0,71
4	01/01/84	116	21,70	7272	0,50

Table 2.2 Characteristics of the observed storms

For urban rainfall/runoff records, the ratio of total runoff to total rainfall is considered to be a good validity check. Such ratios were calculated for the 4 storms and are shown in Table 2.2. However no conclusions can be extracted because the total runoff of storm 2 and 4 is unrealistic since Hillbrow catchment is a highly developed urban area. So from the beginning there are serious doubts concerning the validity of the records, especially for the storm on 22/12/83.

2.6 RUNOFF SIMULATIONS

2.6.1 CATCHMENT DISCRETIZATION

For running the WITWAT II model the catchment was discretized into 27 subcatchments as shown in Figure 2.3. The most important factor that was taken into account in deciding sub-catchment boundaries was the location of pipes, channels and areas having similar topographic and/or land-use characteristics.

Figure 2.3 Discretization of the Hillbrow catchment

Nine of the subcatchments cascade into other subcatchments. Since the SWMM model requires that each subcatchment discharge into either a pipe or a channel, this type of discretization was not possible. Hence, the number of subcatchments was reduced to 18, at the same time trying to maintain all the parameters and factors exactly the same as in the WITWAT II model.

2.6.2 CALIBRATION OF THE MODELS

An adjustment of the parameters in the optimum manner was performed. This was done by trial and error and not by computerized optimization technique. However, even if the models compare favourably with measurements, because of the limited number of events, this may just be a result of the calibration process and it is not necessarily true that similar results will be obtained from other input data. Also even if an acceptable goodness-of-fit is achieved,any other measured samples are not available to compare with the computed output in order to validate the model.

The infiltration rate is a function of soil type, surface cover and the antecedent moisture conditions of the soil (AMC) prevailing at the time of a particular storm. The U.S. Soil Conservation Service describes four hydrologic soil groups, from type A (high infiltration rates) to type D (very slow infiltration rates). The antecedent moisture conditions are also ranked as four groups from completely dry (type 1) to saturated (type 4) (Watson 1981). For computation of losses a soil type B (moderate infiltration rates) and a 3mm depression storage was assumed. Since a complete record of antecedent rainfall was not available an AMC of 3 (rather wet) was assumed for all events. For the paved area an average depression storage of 1mm was assumed. A time interval of 2 min was used for all computations.

The procedure that was generally used was (Jewell et al. 1978):

1. Adjust Manning's roughness (n) because this modifies flow retardance and therefore the peak flow and time to peak.

2. Adjust the volume of total runoff by modifying the detention storage coefficient.

After the calibration procedure the values of the Manning's coefficient was found to be 0,015 for impervious area and 0,250 for pervious area.

2.6.3 RESULTS OF SIMULATIONS

The results of runoff simulations are summarized in Table 2.3 and 2.4 for SWMM model and WITWAT II model respectively. The Tables contain comparisons of observed and simulated volumes, peak flowrates and mean flowrates. Also they contain 4 statistical measures in order to evaluate the accuracy of the hydrographs computed by the models when compared with the entire recorded hydrographs. These are the sum of squared residuals, sum of absolute residuals, coefficient of persistence, and coefficient of efficiency (Green 1985). However the most important and the most representative for the measure of agreement is the coefficient of efficiency. This has the property that the closer the value is to 1, the better is the agreement between the observed and the estimated values. Observed hyetographs, runoff hydrographs and simulated hydrographs are shown in Figures 2.4 to 2.7.

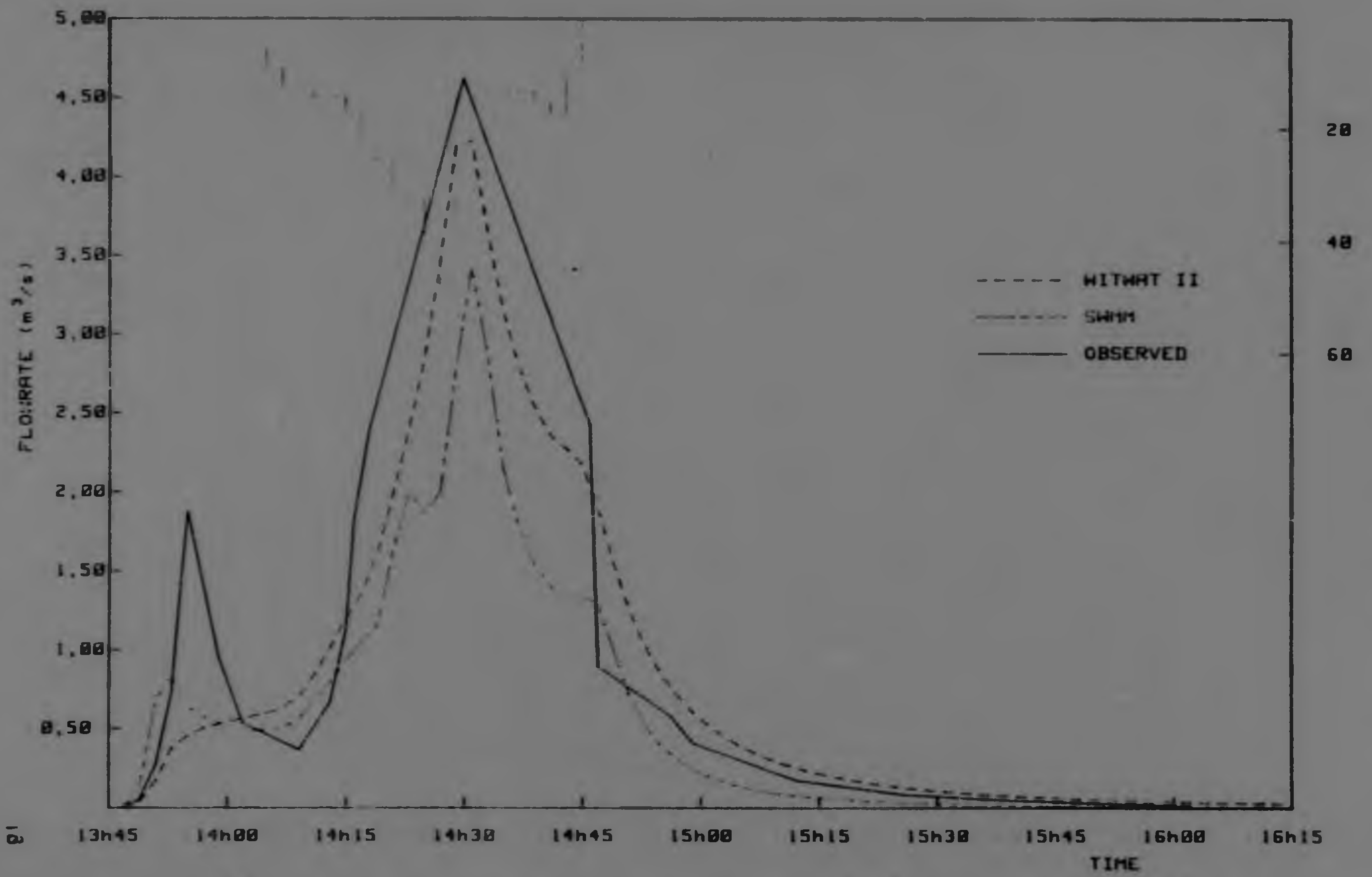

Figure 2.4 Comparison of observed and simulated hydrographs from Hillbrow catchment for storm on 16/12/83

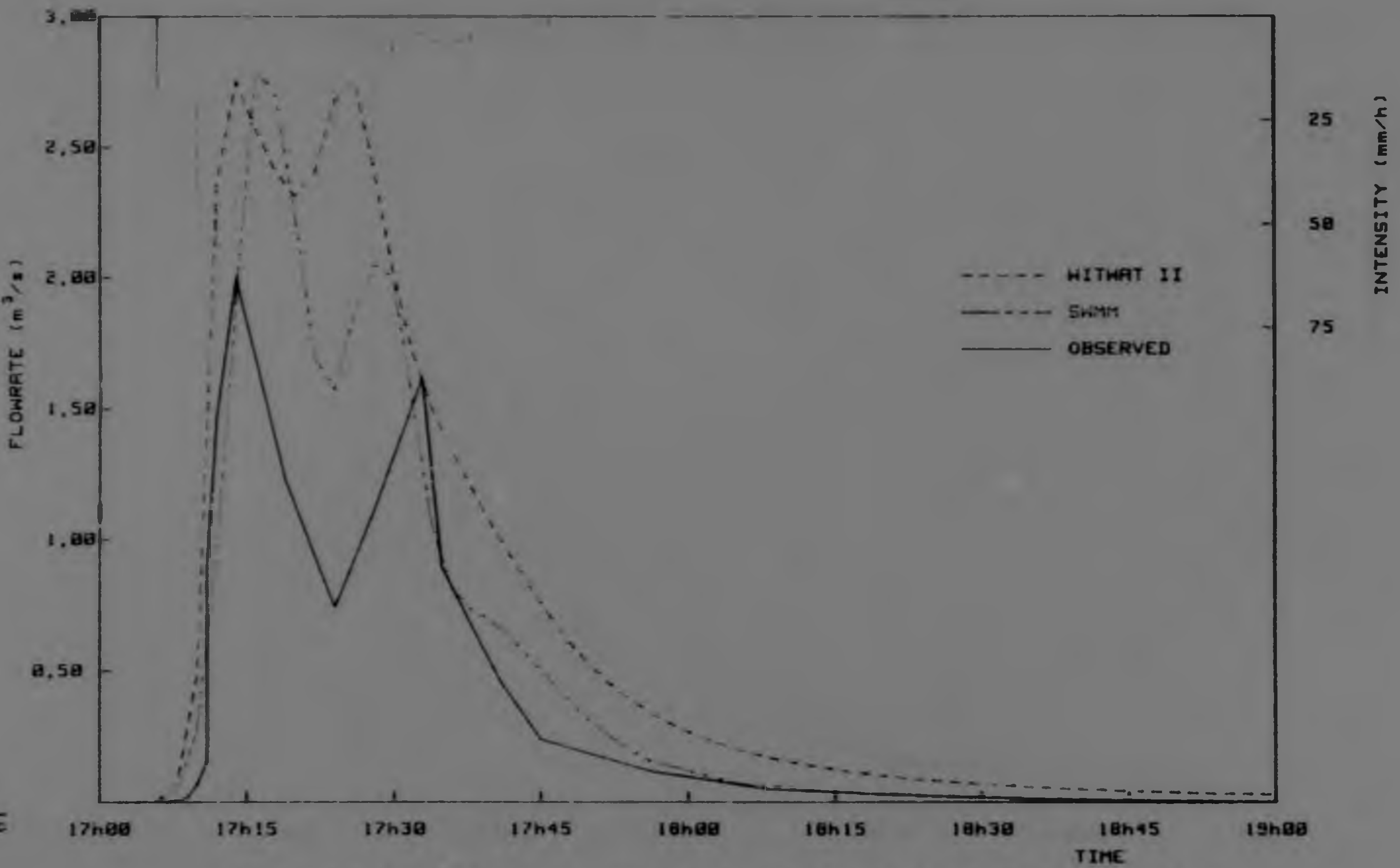

Figure 2.5 Comparison of observed and simulated hydrographs from Hillbrow catchment for storm on 22/12/83

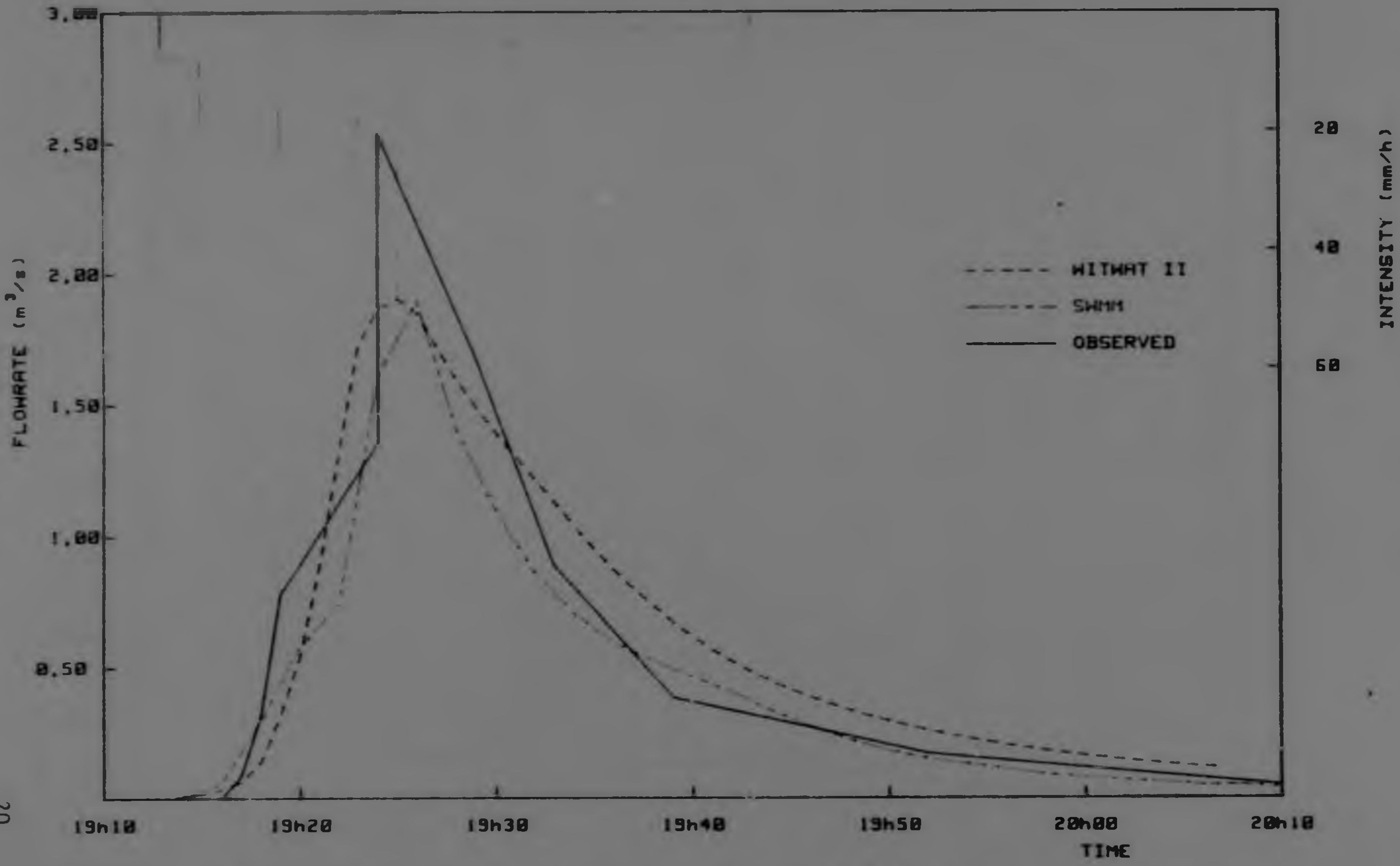

Figure 2.6 Comparison of observed and simulated hydrographs from Hillbrow catchment for storm on 30/12/83

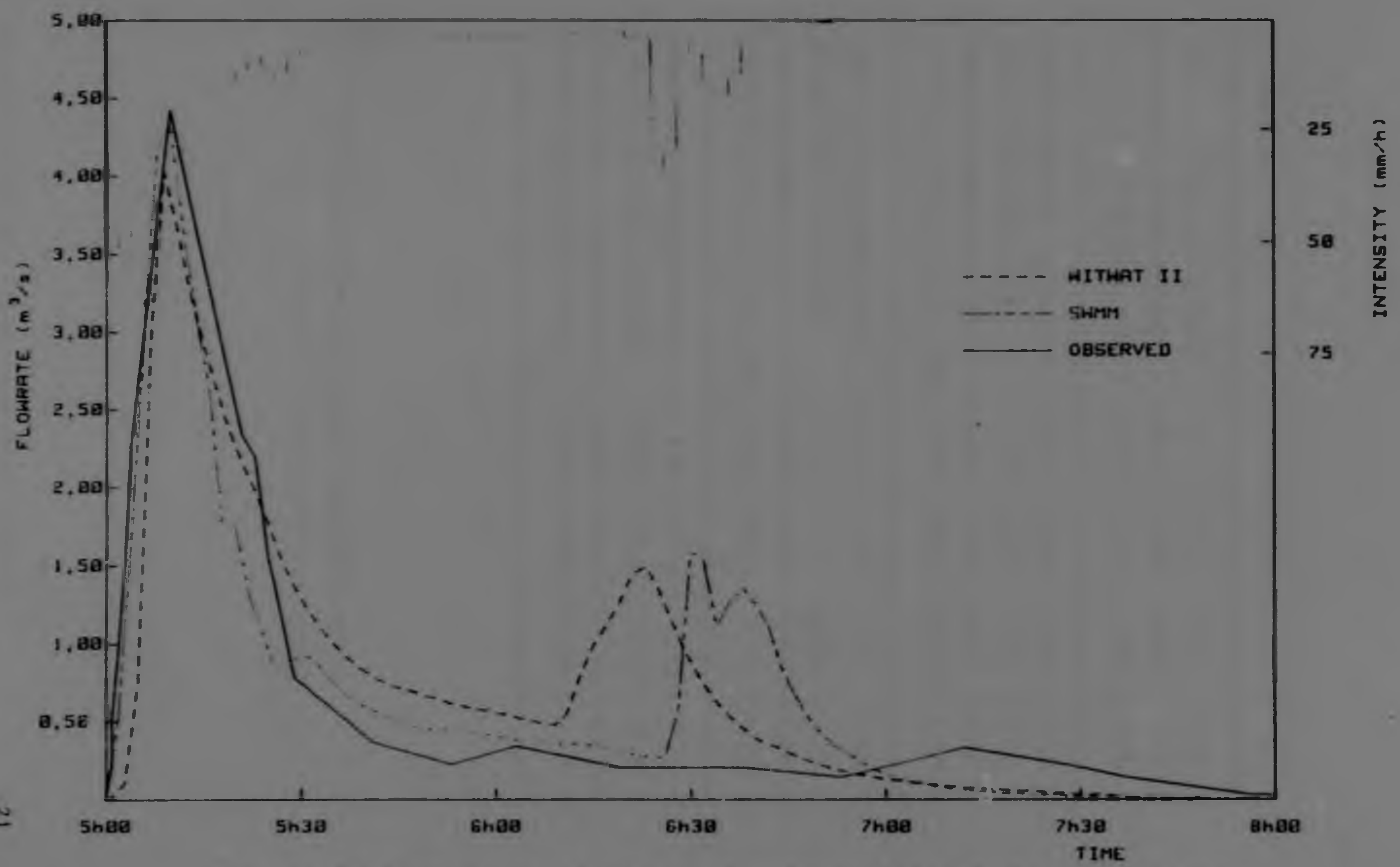

Figure 2.7 Comparison of observed and simulated hydrographs from Hillbrow catchment for storm on 01/01/84

Storm no.	1	2	3	4
Volume of flow(simulated)	5090	3422	1521	5062
Volume of flow(observed)	8276	2373	1690	5272
Ratio of Volumes(sim./obs.)	0,62	1,44	0,90	0,96
Peak flowrate(simulated)	3,43	2,80	1,90	4,33
Peak flowrate(observed)	4,62	2,00	2,54	4,43
Ratio of peaks(sim./obs.)	0,74	1,40	0,75	0,98
Percent.error in simul. peak	-25,8%	40,0%	-25,3%	-2,10%
Mean flowrate(simulated)	0,71	0,83	0,55	1,10
Mean flowrate(observed)	1,15	0,57	0,61	1,14
Ratio of mean	0,62	1,44	0,90	0,96
Percent.error in simul. mean	-38,5%	44,3%	-9,70%	-3,70%
Sum of squared residuals	40,01	12,00	1,59	6,70
Sum of absolute residuals	30,24	13,26	3,99	12,58
Coefficient of persistence	51,71	40,41	17,10	34,26
Coefficient of efficiency	0,661	-0,025	0,820	0,901

Table 2.3 Summary of runoff simulations with SWMM model on the Hillbrow catchment

Storm no.	1	2	3	4
Volume of flow(simulated)	7377	4764	2033	5136
Volume of flow(observed)	8276	2373	1690	5272
Ratio of Volumes(sim./obs.)	0,89	1,97	1,20	1,02
Peak flowrate(simulated)	4,23	2,76	1,75	4,02
Peak flowrate(observed)	4,62	2,00	2,54	4,43
Ratio of peaks(sim./obs.)	0,91	1,38	0,69	0,91
Percent.error in simul. peak	-8,60%	37,9%	-31,3%	-9,10%
Mean flowrate(simulated)	0,98	0,80	0,67	1,39
Mean flowrate(observed)	1,10	0,41	0,55	1,36
Ratio of mean	0,89	1,97	1,21	1,02
Percent.error in simul. mean	-10,80	97,0%	21,1%	1,80%
Sum of squared residuals	13,70	18,68	1,90	10,98
Sum of absolute residuals	19,89	19,58	6,02	14,83
Coefficient of persistence	36,68	81,61	27,74	36,98
Coefficient of efficiency	0,893	-0,253	0,801	0,807

Table 2.4 Summary of runoff simulations with WITWAT II model on the Hillbrow catchment

2.7 DISCUSSION OF SIMULATION RESULTS

2.7.1 STORM NO.1

The storm no.1 of 16/12/83 produced a two-peak hydrograph. However the computed hydrograph from the WITWAT II model did not follow the first peak. This can be explained by the way the model computes depression storage. In the model depression storage is considered as an initial abstraction, runoff only commencing once the volume of rainfall exceeds the volume available for depression storage. In the SWMM model the outflow to gutters or pipes is computed by subtracting at each step part of the depression storage from total depth. However, the first peak that SWMM produced is much smaller than the observed. Both models underestimated runoff volumes and peak discharges, but computed and observed hydrographs are similar in shape.

The WITWAT II model performed very well while the average ratio of estimated to observed peak discharge is 0,91 and the ratio of estimated to observed volume is 0,89. The coefficient of efficiency is very good (0,886). The agreement between measured and computed values was not as good for SWMM as for WITWAT II but nevertheless satisfactory and the coefficient of efficiency is 0,66 which is fair.

The underestimation of peak could be caused by various reasons. It was found that because the SWMM model eliminates the effect of the length of travel on the overland flow depth, the detention storage is too large and some delay may occur in the time of concentration. It follows that the peak flowrate may be underestimated and the error increases with the length of the plane. In practical applications, where the slope is not always a physical parameter because of the irregularity of an area, one may account for this effect by modifying the value of the slope. However the values of slope and roughness coefficient were not altered in order to use the same parameters as the WITWAT II model.

2.7.2 STORM NO.2

Storm no.2 of 22/12/83 was a medium intensity storm which produced a double peak flood hydrograph. The discrepancy for the event of 22/12/83 cannot be explained in this fashion. The volume is overestimated by both models and this could be due to rainfall data errors. From the beginning of the simulation there were serious doubts about the validity of the runoff records of this storm since the ratio of total runoff to total rainfall is unrealistic for a highly developed area such as Hillbrow.

The results are not as good as for the other storms. In fact, the computed hydrographs differ markedly from the observed. The SWMM simulation is better and produced also a double peak hydrograph with a greatly suppressed second peak in relation to the first peak. By visual comparison SWMM produced a better fit than WITWAT II.

2.7.3 STORM NO.3

Storm no.3 of 30/12/83 was a medium intensity short duration storm which produced single peak hydrograph. For storm no.3 both models performed satisfactory, although SWMM the model appears to be somewhat better. The coefficients of efficiency are very good (0,82 for SWMM and 0,801 for WITWAT).

2.7.4 STORM NO.4

Storm of 01/01/84 was a high intensity storm. It can be seen that both models introduced a second peak into the estimation and it was not possible to remove it from the analysis. The second peak is explained very logically since the storm after an hour of the first peak increases its intensity for a short period. The catchment for the first three storms was very sensitiva to the changes of the storm intensity and it was expected to present a second peak.

The only logical explanation is that the stage-recorder at the catchment outlet was stuck during that certain period. So for the validation of the model the period from 5h00 to 6h15 only is considered (just before the second peak begins).

2.7.5 ANALYSIS OF RESULTS

The catchment proved to be very sensitive to the changes of storm intensity. This is due to the high percentage of imperviousness of the catchment and the high slopes (up to 8%). The difference in performance between the two models for this test area is considered to be comparatively small from a practical point of view although the overall performance of the SWMM model as indicated in Table 2.3 appears to be somewhat better.

Accurate simulation of runoff volumes is of basic importance for good reproduction of observed hydrographs. The total runoff volumes were reproduced fairly accurately. The mean value of the ratio of measured to simulated runoff (except storm no.2) was 0,83 for SWMM model and the percentage error was 17,5% for SWMM model and 21,5% for the WITWAT II model.

The mean value of the ratios of the simulated runoff volumes indicated that virtually all the simulated runoff originated from the impervious area. The ability of a runoff model to simulate runoff peaks accurately is fairly important. From Tables 2.3-2.4 it can be inferred that the observed runoff peaks were underestimated more by SWMM model and less by the WITWAT II model. The percentage error of simulated peak was -17,70% for the SWMM model and 17,20% for the WITWAT II model.

The time to peak of the hydrograph is, from the practical viewpoint, the least important parameter of the hydrograph. A comparison of observed and simulated times to peak in Hillbrow catchment revealed that, on average, the simulated peaks occurred almost simultaneously with the observed ones.

Even if the models compare favourably with the measurements of the three storms because of the limited number of events firm conclusions about the validity of the models cannot be drawn. More measured samples to compare with the computed are necessary, otherwise the goodness-of-fit may just be the result of the calibration process.

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2.8 STUDY OF THE EFFECTS OF URBANIZATION IN HILLBROW CATCHMENT

2.8.1 RESULTS

As referred to earlier the two major factors which urbanization affects are imperviousness and canalization. Hillbrow catchment is a highly urbanized catchment, thus the values of the initial losses for the pervious part can not be established easily by calibration. For this reason, for each case a sensitivity analysis was performed for different infiltration rates and initial abstraction (Figure 2.8). The results of the sensitivity runs for the initial losses in urban and semi-urban conditions are shown in Figures 2.9 and 2.10 respectively.

EXISTING
IMPERVIOUSNESS

50% reduction ______ Reduction of canalization ______ (A)

80% reduction ______ Only a major natural channel ______ (A)

100% pervious ______ no drainage system ______ (A)

where (A) is the sensitivity analysis for different infiltration rates and initial abstractions

Figure 2.8 Approach to the analysis of the effects of urbanization in Hillbrow

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The imperviouness is reduced to total pervious conditions with intermediate stages 50% and 80% less than the existing imperviousness. The results of the comparison are shown in Figure 2.12. While the imperviousness was reduced, canalization was also reduced by reducing the number of pipes. For 50% of the existing imperviousness three cases of canalization were compared:

1. The existing drainage system.

2. Only a major channel from the centroid of the catchment to the outlet.

3. No drainage system.

The results of the three runs are shown in Figure 2.11. The overall comparison of urban and pre-urban conditions are shown in Figure 2.13. The cases that were compared were:

1. Urban conditions

2. Semi-urban conditions, i.e.
   a) 50% less imperviousness
   b) a major channel from the centroid of the catchment to the outlet.
   c) Initial losses: Infiltration fo=75 mm/hr, f =10 mm/hr
      Initial abstraction =4mm

3. Pre-urban conditions, i.e.
   a) 80% less imperviousness
   b) A major natural channel (n=0,050)
   c) Initial losses: Infiltration fo=125 mm/hr, f =15 mm/hr
      Initial abstraction =5mm

A synopsis of the performed runs is given in Table 2.5.

Run no.	Percent Imperviousness	Canalization	Infiltr. rates	Initial abstr.
R10	calibrated	existing drainage	25-5	3
R11	values	system	75-10	4
R12	"	"	125-15	5
R20	50% less imper.	"	25-5	3
R21	"	"	75-10	4
R22	"	"	125-15	5
R23	"	only a major	75-10	4
R24	"	channel	125-15	5
R25	"	no drainage system	75-10	4
R30	80% less imper.	"	75-10	4
R32	"	a natural channel	125-15	5
R41	100% pervious	"	125-15	5

Table 2.5 Summary of runoff simulations with WITWAT II model on the Hillbrow catchment

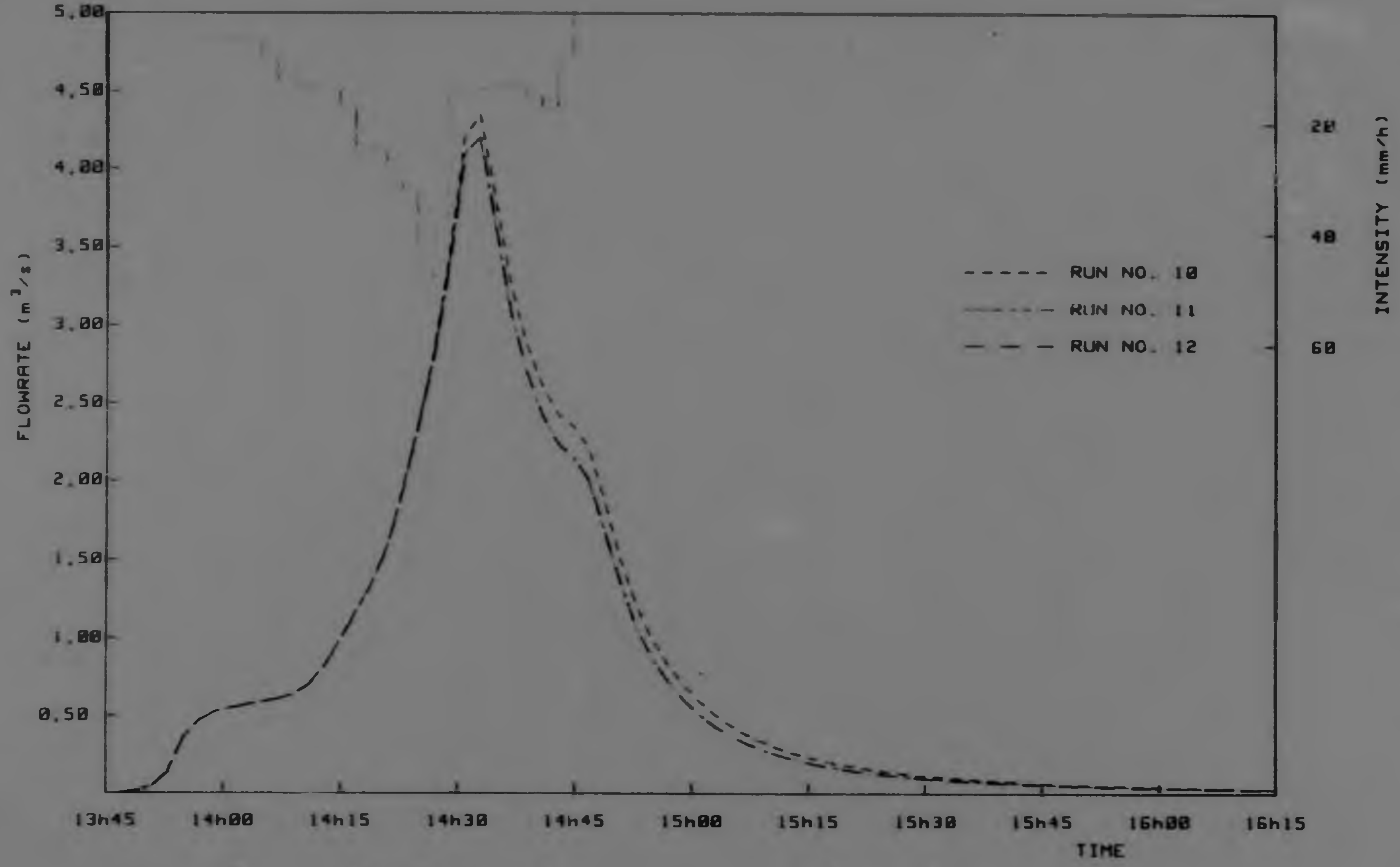

Figure 2.9 Sensitivity of changes of initial losses in Hillbrow catchment (urban conditions)

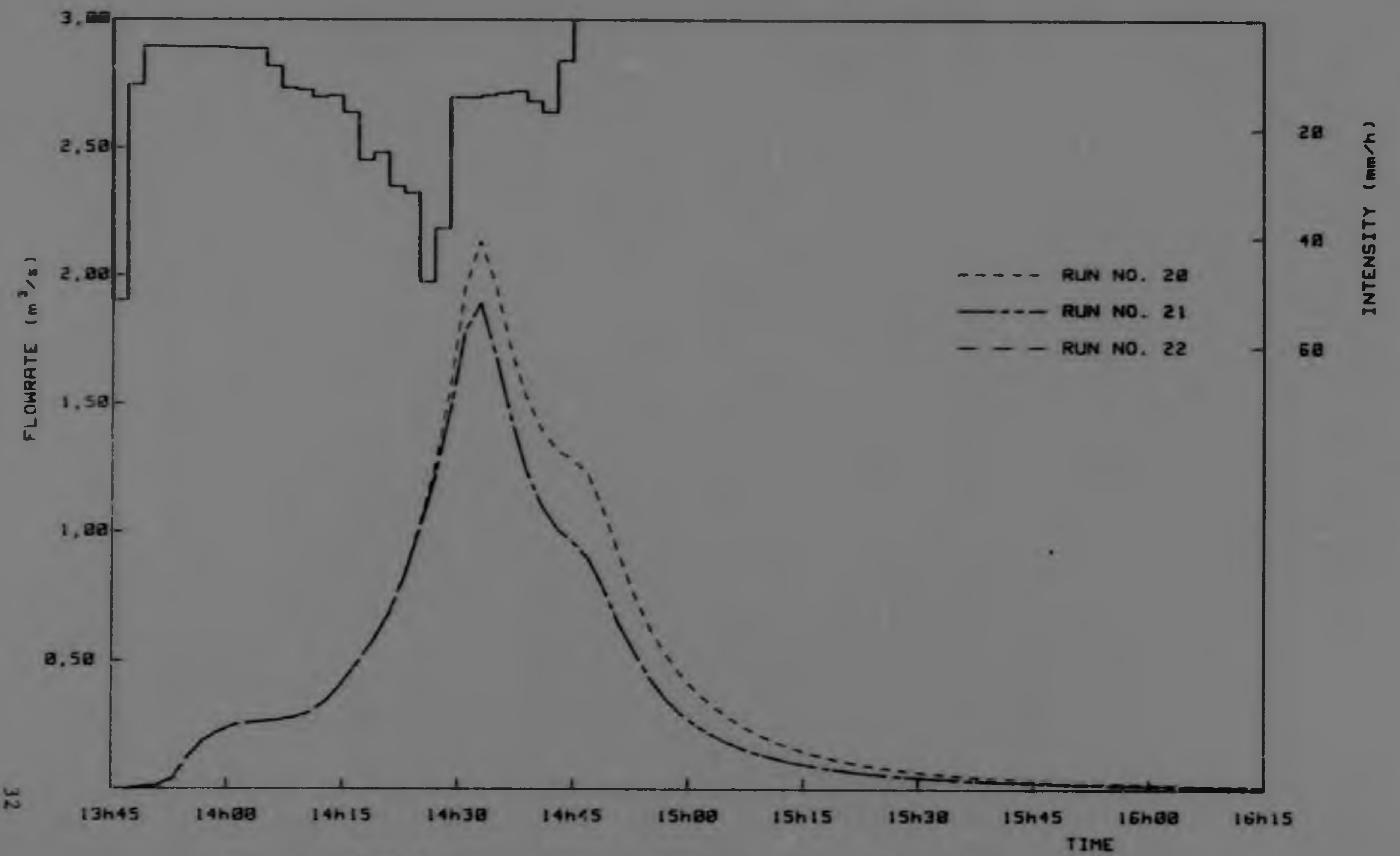

Figure 2.10 Sensitivity of changes of initial losses in Hillbrow catchment (semi-urban conditions)

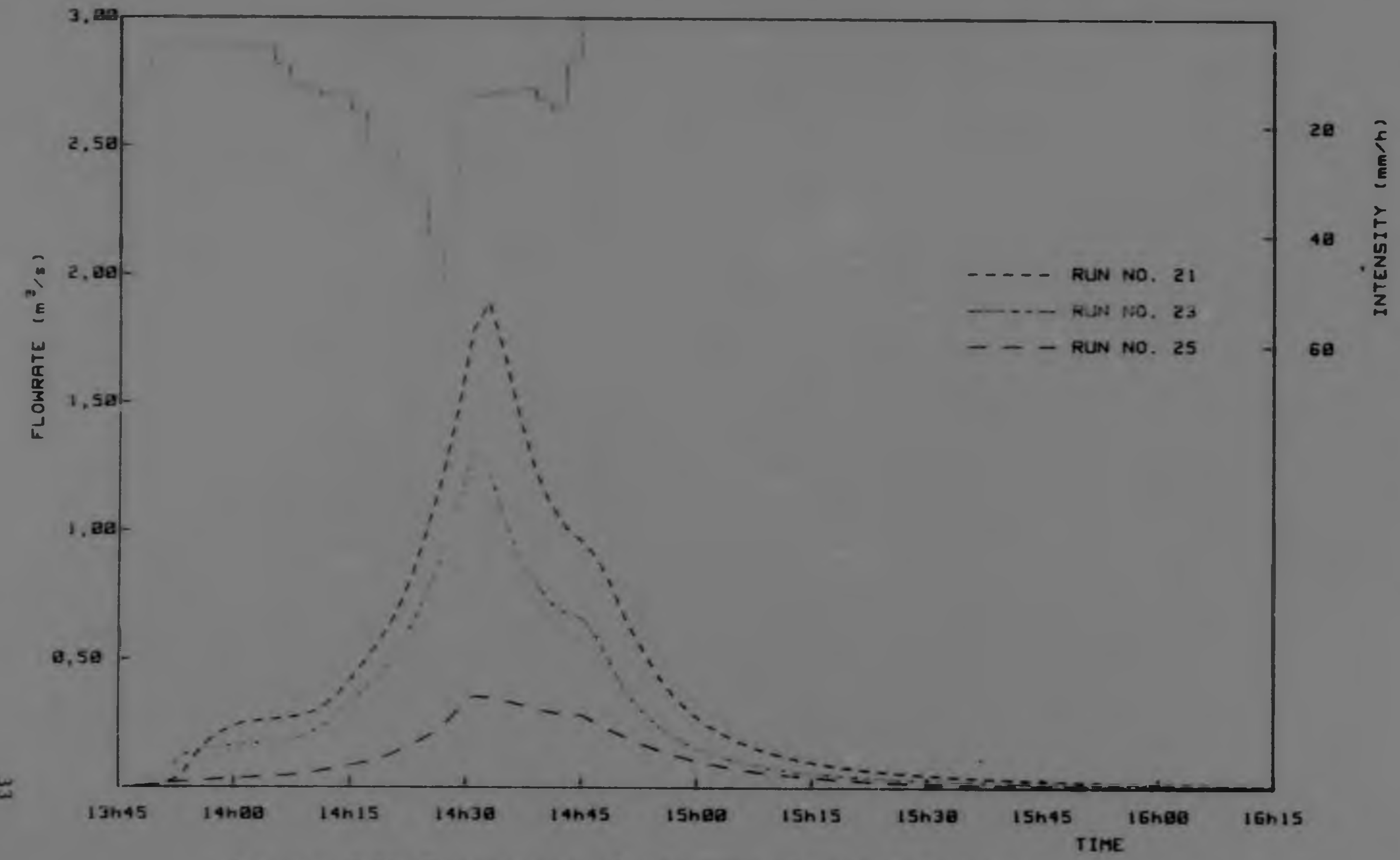

Figure 2.11 Comparison of changes of canalisation in Hillbrow catchment (semi-urban conditions)

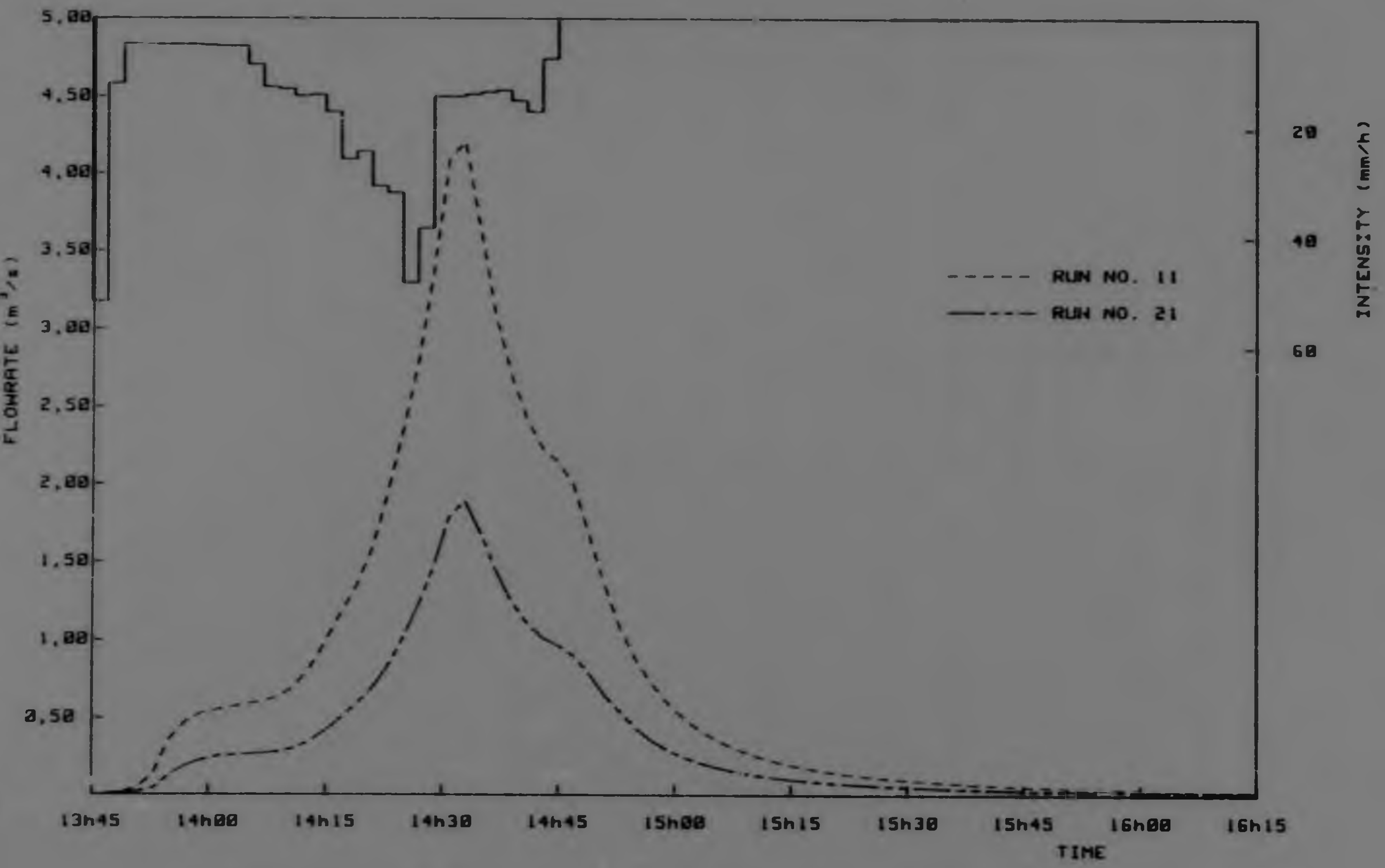

Figure 2.12 Comparisons of changes of imperviousness in Hillbrow catchment

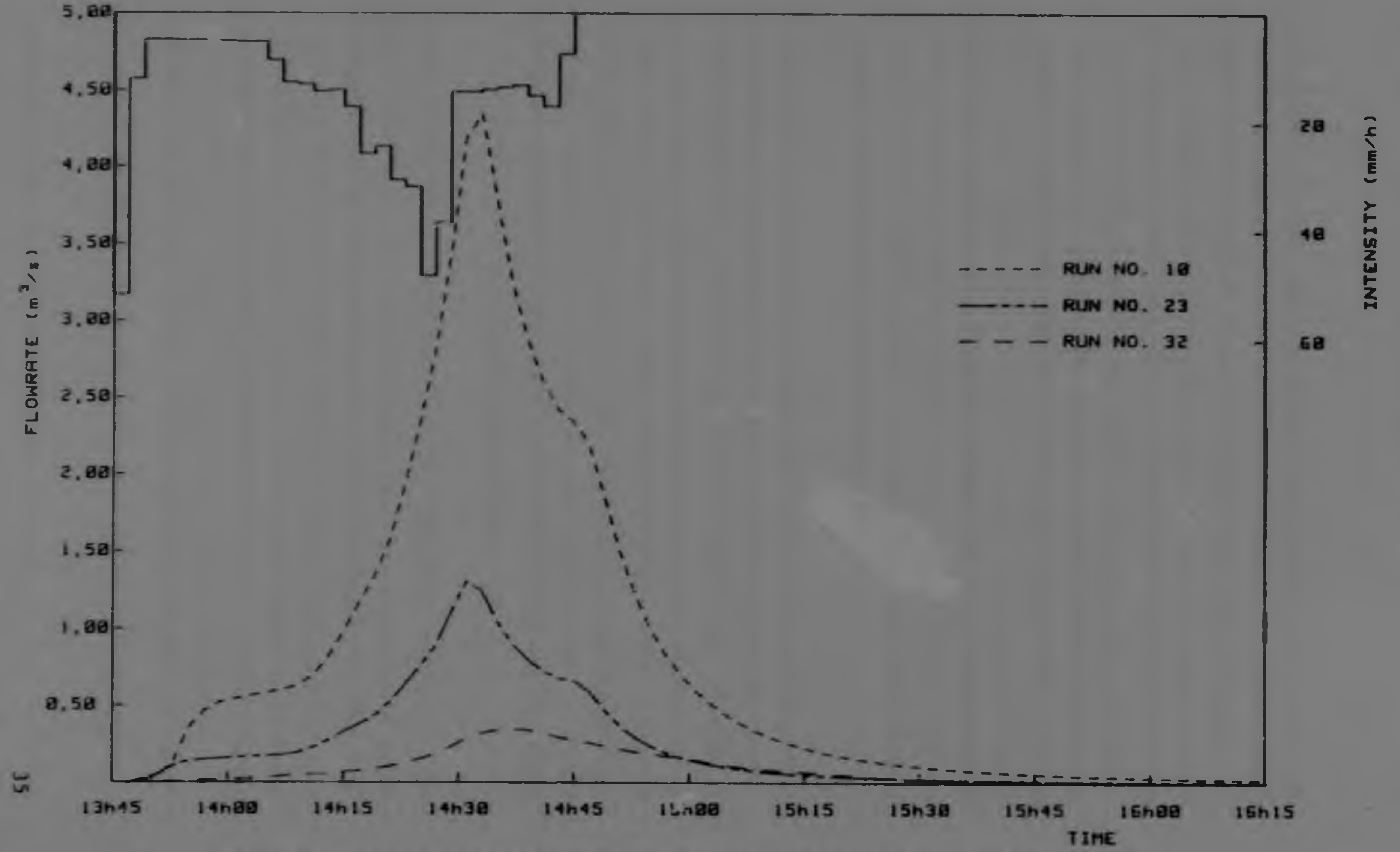

Figure 2.13 Comparison of urban and pre-urban conditions in Hillbrow catchment

2.8.2 ANALYSIS OF THE RESULTS

The WITWAT II model uses the Horton equation for infiltration and simplistic assumptions for the initial losses. In this regard, such simulation results lead to error if the imperviousness of the catchment is low. Therefore, for rural areas, a model should preferably be based on the SCS, or a more refined, method. Models based on the Horton method may result (especially for low intensity storms), in underestimating the runoff, as the latter is generated only if the rainfall intensity is higher than the infiltration capacity rate. Usually the critical storms for rural conditions are the long duration storms. Thus, the Horton based model did not generate any runoff, in the case of a short duration storm on a 100% pervious area, but it appeared that the rainfall totally infiltrated in the soil (Run no.41).

Under urban conditions, however, the critical storms have high intensities and short durations such as the Chicago-type storms. Here the Horton procedure is to be preferred because firstly it is sensitive to the storm intensity and secondly it results in higher peak flows than the CN procedure (Wisner 1980). Figures 2.11 and 2.12 illustrate that the runoff is extremely sensitive to the changes of imperviousness and canalization. In the Hillbrow catchment a 50% reduction in the imperviousness led to a reduction of the peak by more than 120%.

Under semi-urban conditions in the Hillbrow catchment the assimulation of the existing drainage system to only a major channel reduced the peak-flow by 47% and the volume of runoff by more than 40%. The original impervious cover accounted for less infiltration and greater runoff, whilst the smoother surfaces together with pipe-channel construction lead to reduced concentration time and therefore larger peak flows (Stephenson, D. 1980). The provision of concrete-lined channels in major watercources reduces the flow area with a similar reduction in the available in-channel storage of these watercources.

The direct effects of the increase of the impervious cover are the reduction of the initial abstractions and the infiltration. From Figure 2.9 it is evident that, for pervious area, the increase of the detention depth from 3mm to 5mm and of the initial infiltration rate from 25mm/hr to 125mm/hr have a minor effect on the outflow hydrograph. This result is due to the high impervious percentage of the Hillbrow catchment and to the fact that the simulated runoff from the pervious fraction is essentially zero. Figure 2.10 shows that as the pervious cover is increased, infiltration becomes the dominating factor that controls the runoff. With the construction of roads, pavements and buildings the natural retardation of the surface runoff is eliminated and the concentration time reduces. The infilling of obstructions of the natural flood plain reduces the available storage capacity of the catchment, leading to increased peak flowrates at its downstream (Beard et al. 1979).

The decrease of roughness is an indirect effect of urbanization and results from the increase of the impervious cover and canalization. Natural retardation of the system is then not possible and the water is conveyed in a very short time. The result is that shorter, sharper hydrographs occur. This in turn shortens the response time of urbanized catchments, making them more sensitive to shorter duration, higher intensity rainfall events.

Figure 2.13 depicts an overall comparison of the urban conditions with the hypothetical semi-urban and rural conditions in Hillbrow catchment. The results are remarkable even if the simulation for the rural area is only a vague approximation. If Hillbrow was rural, it would generate a minimum runoff, or none at all, for these storms. If semi-urban conditions were considered the runoff would be five time less than the actual urban runoff.

Similar studies have also indicated that as the relative magnitudes of flood peaks increase, the ratio of urban peak rate to rural rate declines, the effect of urbanization being more pronounced for the more frequent occurrences. According to Robey (1970), the lag time for an urban area with a stormwater drainage system is between 12% and 20% that of a comparable natural system. Davis (1974) noted that for an increase in imperviousness from 1% (rural area) to 35% (developed urban area), the peak flowrate increased by a factor of nine for the two year recurrence interval rainfall event and by a factor of five for the fifty years recurrence interval rainfall event. Miles (1984) claims that the total volume of runoff from an urbanized area can be up to twice that from a comparable natural area. All the above indicate that the most dramatic hydrologic impact of urban development, is that on peak flows in storm drainage.

The response time of a small catchment is partly determined by its drainage density. (i.e. the length of drainage path per unit area)(Hall, M.J. 1980). In an urbanized catchment, the drainage is considerably higher than that of the original rural river basin because of the widespread use of pipe and gutter drainage system associated with the expanded impervious area. The time which elapses before the water enters the sewer network is therefore greatly reduced. As the size of the catchment area increases the effect of the drainage channel improvements becomes more significant.

The effect of urban development on water yield increases also the volume of discharge considerably. It prevents soil moisture recharge, leading to a lowering of the local water-table and a reduction in dry weather stream flow (Robey, D.L. 1970). These effects are related to some extent to the pattern of development. The location of development within the catchment affects the relative timing of flood peaks from the urbanized and rural parts for any particular rainfall event. However, any design rainfall event is hypothetical, and to predict coincidence or separation of individual flood peaks is contentious, as observed storms are neither spatially nor temporally uniform. Only when analysing a large catchment with a known flood history should any account be taken of these 'phasing effects' (Beard et al. 1979).

Urbanization has less effect upon catchments which already have a high runoff coefficient. Similarly, a smaller difference in the runoff is found between urbanized and rural catchments when the latter are already saturated. Moreover, the runoff from urbanizing catchments is generally regarded as being less affected by antecedent moisture conditions. This lack of sensitivity, coupled with the shorter response time, makes the urbanized area more susceptible to intense rainfall from thunderstorms than to prolonged rainfall.

In conclusion, the following are noted:

1. The response time of a small watershed is partly determined by its drainage density (i.e. the length of drainage path per unit area).

2. The major effects on the runoff are caused by the increase in impervious area and by the improvements to the surface water drainage system.

3. The magnitude of the increase in percentage runoff as a result of urbanization is dependent upon the original rural percentage runoff and the antecedent rainfall conditions.

   The comparisons of Figures 2.9 to 2.13 showed that the evaluation and the assessment of the effects of urbanization on the hydrological system are characterised by a substantial complexity and are difficult to be quantified. Even though the changes in urban development cannot be studied more accurately at the present stage because of lack of data, the effects of urbanization were identified. The following chapter examines the effectiveness of the traditional solutions to the problem of urbanization and investigates a proposed revised approach with respect to urban drainage analysis and design.

3.0 ANALYSIS OF BRAAMFONTEIN SPRUIT SYSTEM

3.1 INTRODUCTION

One of the objectives of this study is an analysis of an overloaded drainage system due to urbanization. For this reason the drainage system of Braamfontein Spruit watershed was selected. The major concern will not be a detailed analysis with respect to individual flows and volumes, but the behaviour of the total watershed and the master drainage system. Because no runoff data were available, design storms of different durations and recurrence intervals were used. The following analysis will show the design drawbacks of the existing drainage system of the Braamfontein Spruit watershed.

3.2 SELECTION OF MODEL

An important decision in modelling is the selection of the level of discretization of a watershed. The most frequently used models namely SWMM, ILLUDAS and WITWAT II require a fine discretization in order to relate directly to the physical parameters of the watershed.

However, in watersheds such as Braamfontein Spruit, with very large areas and with non-homogenious characteristics, the choice of the level of discretization becomes very difficult. Detailed discretization of the subcatchments results in a large number of small watersheds and significant computational time. So it becomes obvious that in this watershed it is necessary to work with a model that could produce highly accurate results with larger and lumped areas and less detailed information. OTTHYMO is a model for the design of MASTER drainage plans and so for watersheds like Braamfontein Spruit with a catchment area of this size it appears the most appropriate.

3.3 DESCRIPTION OF OTTHYMO MODEL

OTTHYMO is a deterministic single-event model that uses different conceptual models to simulate the hydrographs from both the urban and rural watersheds. It is a computer package made up of a series of subroutines each of which corresponds to a specific hydrologic command (e.g. COMPUTE HYD - compute hydrograph). The basic subroutine used, URBHYD, calculates hydrographs using 'instantaneous unit hydrographs' (IUH). The hydrographs from the impervious and pervious portion of the watershed are simulated using two linear reservoirs in parallel. URBHYD is therefore a 'two parallel reservoir' model. Routing in channel reaches is done by the Muskingum method.

The model simulates the hydrograph for each subwatershed, routes the hydrograph through the channel or a pipe and then adds this routed hydrograph to a hydrograph from another subwatershed downstream. This process starts upstream and continues down the drainage network to the outlet. There is also the capability of routing through reservoirs for stormwater management purposes.

3.4 DESCRIPTION OF BRAAMFONTEIN SPRUIT WATERSHED

The Braamfontein Spruit watershed, situated to the north of Johannesburg has an area of 21,58 km^2 The ground slopes are moderately steep (average 4%) varying from 3% to 8%. For the purpose of this simulation the watershed was discretized in 15 subcatchments with varying characteristics (Figure 3.1). The only highly impervious subcatchment is Hillbrow (101). In all the other subcatchments the imperviousness ranges from 10% to 20% with varying l .d-uses, mainly residential and comprising roads, sidewalks unpaved parking areas, office blocks, lawns and other small buildings that discharge onto pervious areas.

Most of the water is collected by an artificial channel with a rectangular cross-section and a width varying from 3m upstream to 6m at the outlet where the only portion that is natural (Figure 3.2). If the simulation was performed by other models the length factor would become very important. The inlet of each subcatchment should be situated upstream of the catchment centroid to account for the large number of small pipes which collect the water all over the catchment. In these small pipes the water travels faster so it is important to account for the shorter collection time.

OTTHYMO model, however, since is a lumped model, simulates the hydrograph at the outlet of each subcatchment and the length used in the program is related only to the area of the subcatchment. (from comparisons with measurements it was found that L can be approximated by 1,5 L^2= A where A is the area of the watershed.) The pervious area depression storage was assumed to be 5mm and the soils were classified as type B. A Manning's roughness coefficient (n) of 0,013 was used for all the reaches except at the outlet where the channel is natural and a roughness of 0,050 was assumed. The impervious area depression storage was reduced to 1mm.

Sub. Basin	Area (ha)	XIMP (%)	TIMP (%)	SLI	SLP	MNI	MNP	Length
101	75,2	0,820	0,820	0,033	0,033	0,018	0,150	708
102	125,9	0,095	0,095	0,086	0,086	0,018	0,150	916
103	105,0	0,087	0,087	0,081	0,081	0,018	0,150	837
104	220,4	0,105	0,105	0,049	0,049	0,018	0,150	1212
105	189,4	0,125	0,125	0,067	0,067	0,018	0,150	1224
106	119,2	0,182	0,182	0,080	0,080	0,018	0,150	891
107	170,1	0,089	0,089	0,057	0,057	0,018	0,150	1065
108	130,5	0,177	0,177	0,067	0,067	0,018	0,150	933
109	104,1	0,154	0,154	0,095	0,095	0,018	0,150	833
110	54,0	0,010	0,010	0,074	0,074	0,018	0,150	600
111	173,2	0,085	0,085	0,080	0,080	0,018	0,150	1075
112	226,6	0,087	0,087	0,072	0,072	0,018	0,150	1229
113	284,0	0,125	0,125	0,065	0,065	0,018	0,150	1376
114	92,3	0,170	0,170	0,053	0,053	0,018	0,150	785
115	87,9	0,140	0,140	0,079	0,079	0,018	0,150	766

where

XIMP is the directly connected impervious area

TIMP is the total impervious area

SLI(SLP) is the slope of the impervious (pervious) area

MNI(MNP) is the Manning's impervious (pervious) coefficient

Length is approximated from $1,5L^2 = A$ where

A is the area of the subcatchment

Table 3.1 Subcatchment data

Figure 3.1 Discretization of Braamfontein Spruit watershed

Figure 3.2 Drainage System of Braamfontein Spruit watershed

Conduit or upstr. node	Drains to node	Dimensions Width	Height	Roughness	Slope	Length
1	2	2,50	3,00	0,014	0,030	1000
2	3	2,50	3,00	0,013	0,031	1300
3	6	3,00	3,00	0,014	0,035	1150
8	6	2,50	2,00	0,013	0,036	500
6	11	3,00	3,00	0,013	0,028	1900
11	13	3,00	3,00	0,013	0,019	800
13	15	3,00	3,00	0,013	0,019	450
16	17	2,50	3,00	0,013	0,031	1800
17	15	2,50	3,00	0,013	0,027	850
15	19	6,00	3,00	0,050	0,015	650
19	22	6,00	3,00	0,050	0,015	1200

Table 3.2 Conduit Data

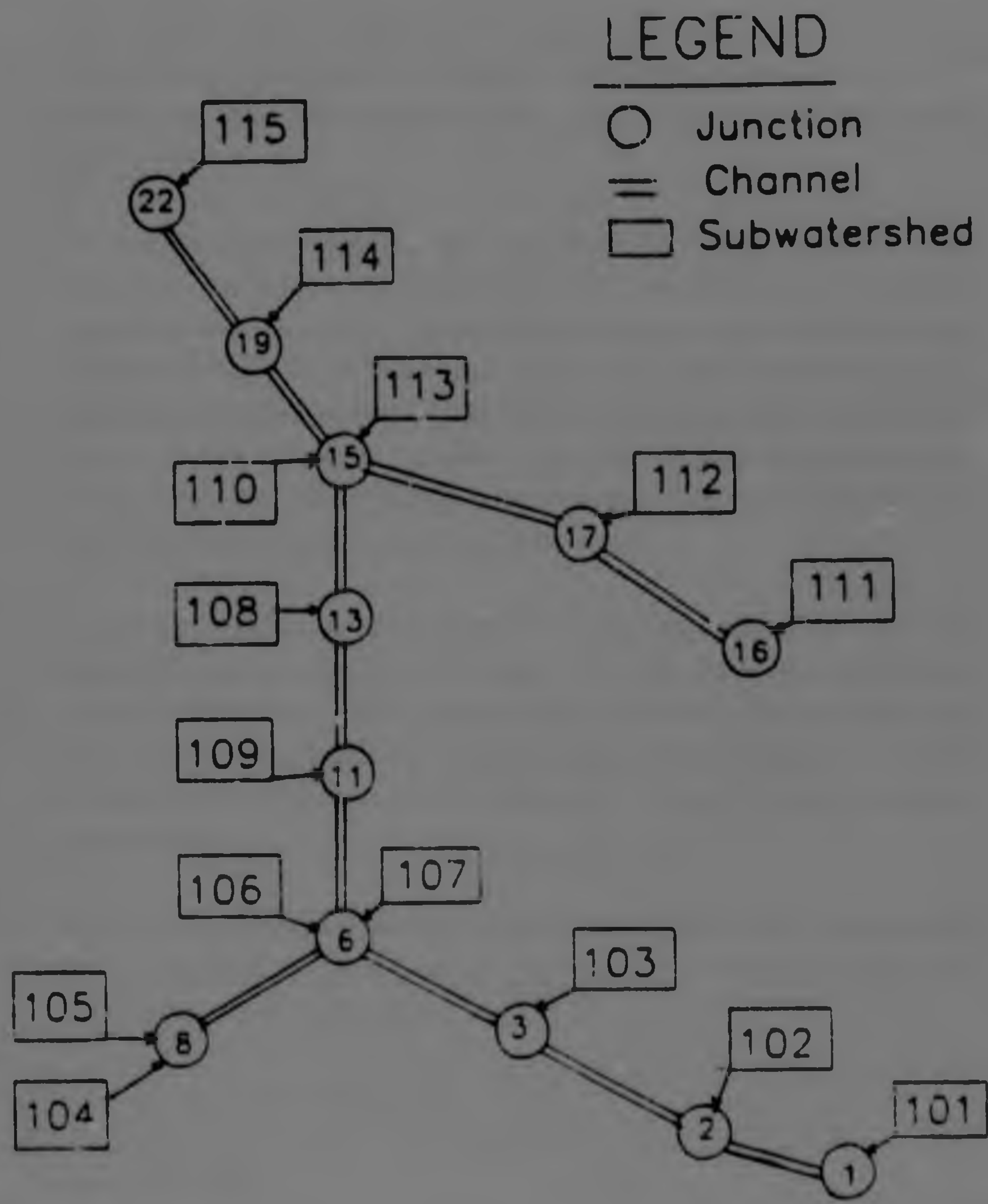

Figure 3.3 Flow diagram from Braamfontein Spruit catchment using OTTHYMO model

3.5 DESIGN STORMS

For a watershed of this size it is general advisable to assume a non-uniform distribution of the design storm. Generally a storm travelling down the catchment will give a worse peak than when moving upstream.

For purposes of this study, a relatively short duration HUFF distribution, such as 3 hours, with a maximum intensity of 90 mm/h was used (Watson 1981). However rainfall records for the area of Johannesburg shows that the storms are generally of high intensity and short duration. Also the first results showed that the watershed responded very fast and had the peak discharge only 1 hour after the start of the storm (Figure 3.6). So a simulation was also conducted for a 1 hour HUFF storm.

For a watershed of this size the recurrence interval that is normally used is more than 50 years. In order to demonstrate the importance of recurrence interval and to study the response of the watershed in short intensity storms, simulations were conducted also for 10 years HUFF storms for 1 and 3 hours duration respectively.

The maximum average intensity was deduced from I-D-F charts and an areal reduction factor of 0,80 was used to account for the very large area of the watershed.

3.6 RESULTS

The results of the simulation are shown in Figures 3.5-3.8 and are very interesting because they illustrate the response of the watershed. From Table 3.3 it is evident that if a storm has 1 hour duration the increase in the peak will be between 66% and 72% in comparison with a 3 hours duration storm.

Hence, for shorter duration and more intense storms the model indicated a considerable increase in the peak. The explanation for this rapid response is that the concentration time along the system is very low namely 30 minutes for 1 hour duration storm duration (both 10 and 50 years). The time of concentration for a three hour duration storm is 1 hour (for 10 and 50 years). The varying ratio of total runoff to total rainfall is also interesting. For short duration and intense storms the ratio is high, namely 0,48. Normally with the low imperviousness of the larger part of the watershed (10-20%) the ratio should be much smaller. However because of the short duration of the storm and the intensity , the ground rapidly approaches saturation thus doesn't have time to absorb the rain, and the runoff proportion of precipitation increases.

The simulation results for 50 years recurrence interval and three hours duration storm and 10 years and 1 hour duration are almost the same. The peak and the ratio of total rainfall to total runoff are approximately the same.

	50 years		10 years	
	1 hour	3 hours	1 hour	3 hours
Peak flowrate (m3/sec)	336,41	95,61	94,38	32,04
Runoff Volume (m3)	764148	415200	246875	150197
Rainfall Vol. (m3)	1605552	1813000	949520	1100600
total runoff / total rainfall	0,48	0,26	0,23	0,14

Table 3.3 Simulation Results

	1 and three hours		10 and 50 years	
	50 years	10 years	1 hour	3 hours
Difference in simulated peak	71,60%	66,10%	71,90%	66,50%
Difference in simulated Vol.	46%	40%	68%	63.8%

Table 3.4 Statistical Comparisons

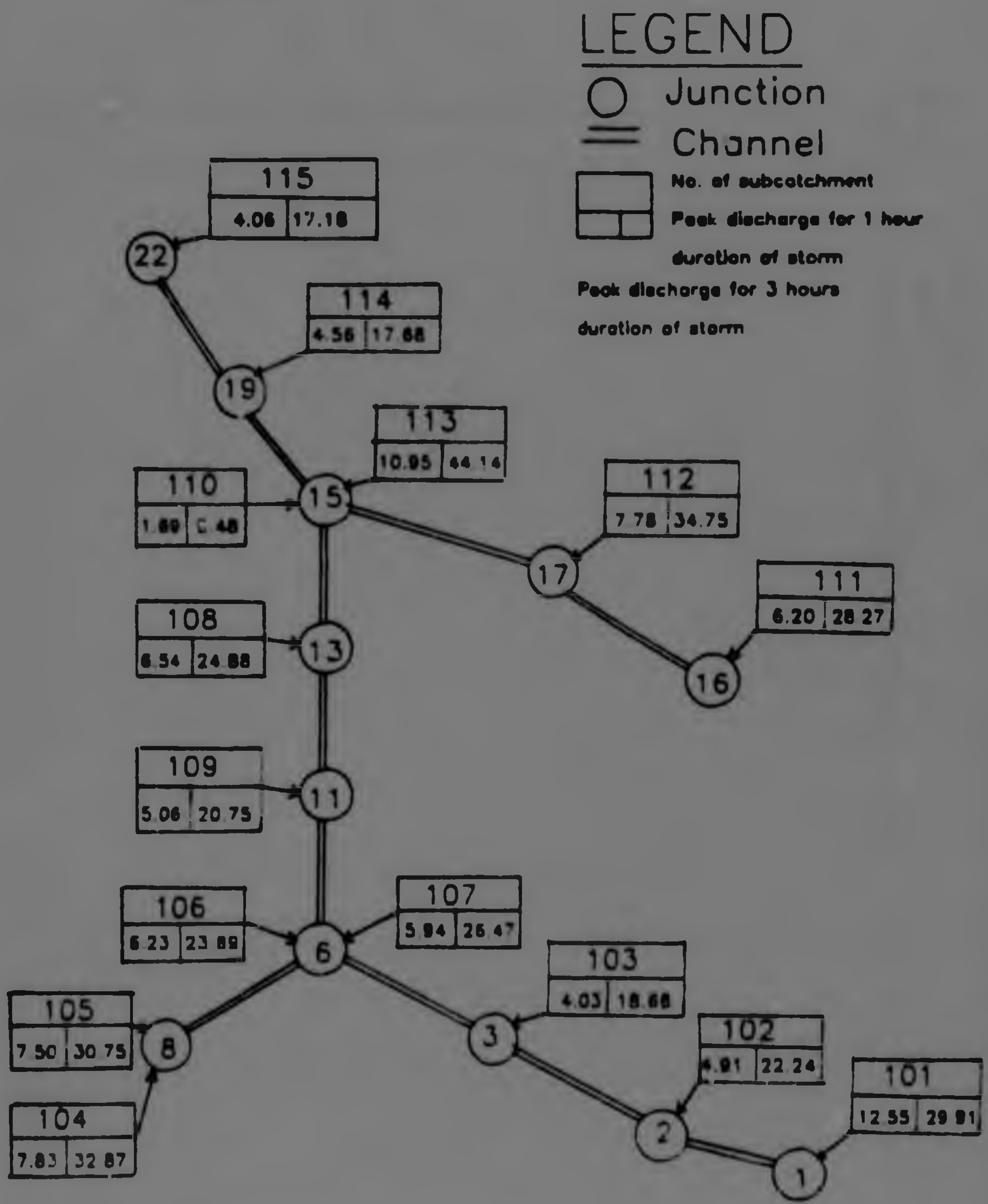

Figure 3.4 Comparison of peak discharges
for 1 and 3 hours duration of storm
(HUFF - 50 years)

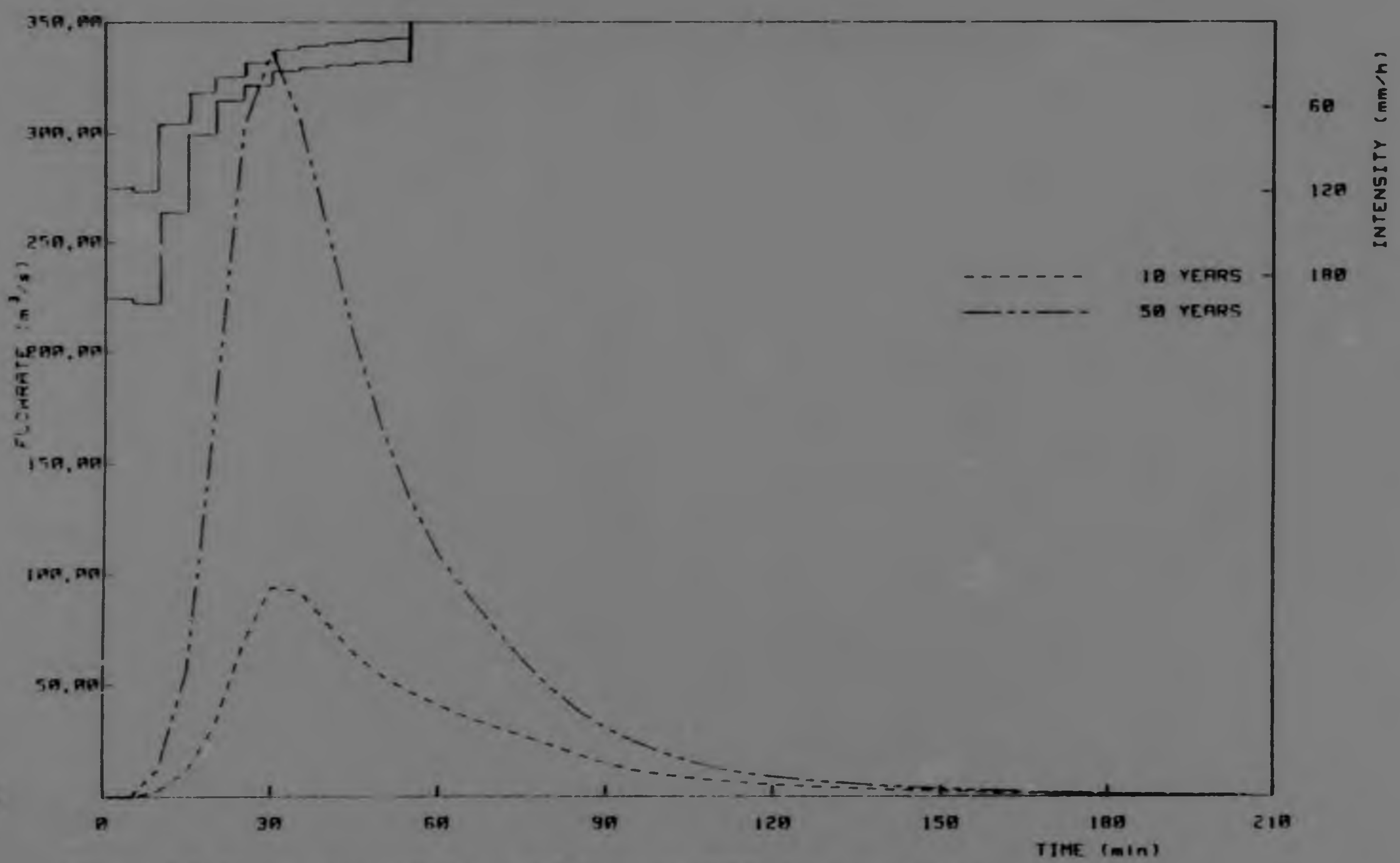

Figure 3.5 Comparison of 10 and 50 years HUFF design storms (1 hour duration)

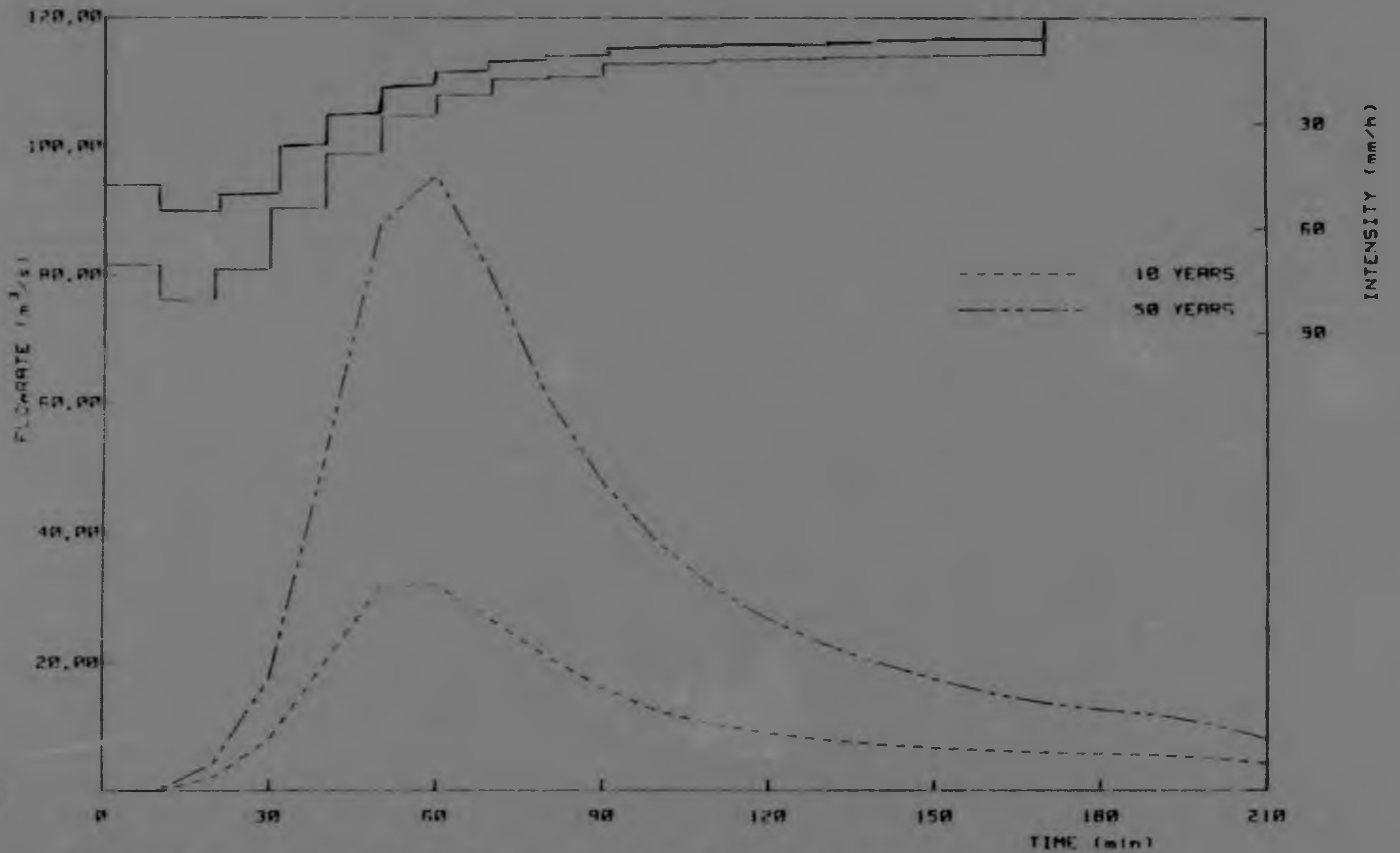

Figure 3.6 Comparison of hydrographs for 10 and 50 years HUFF design storms(3 hours duration)

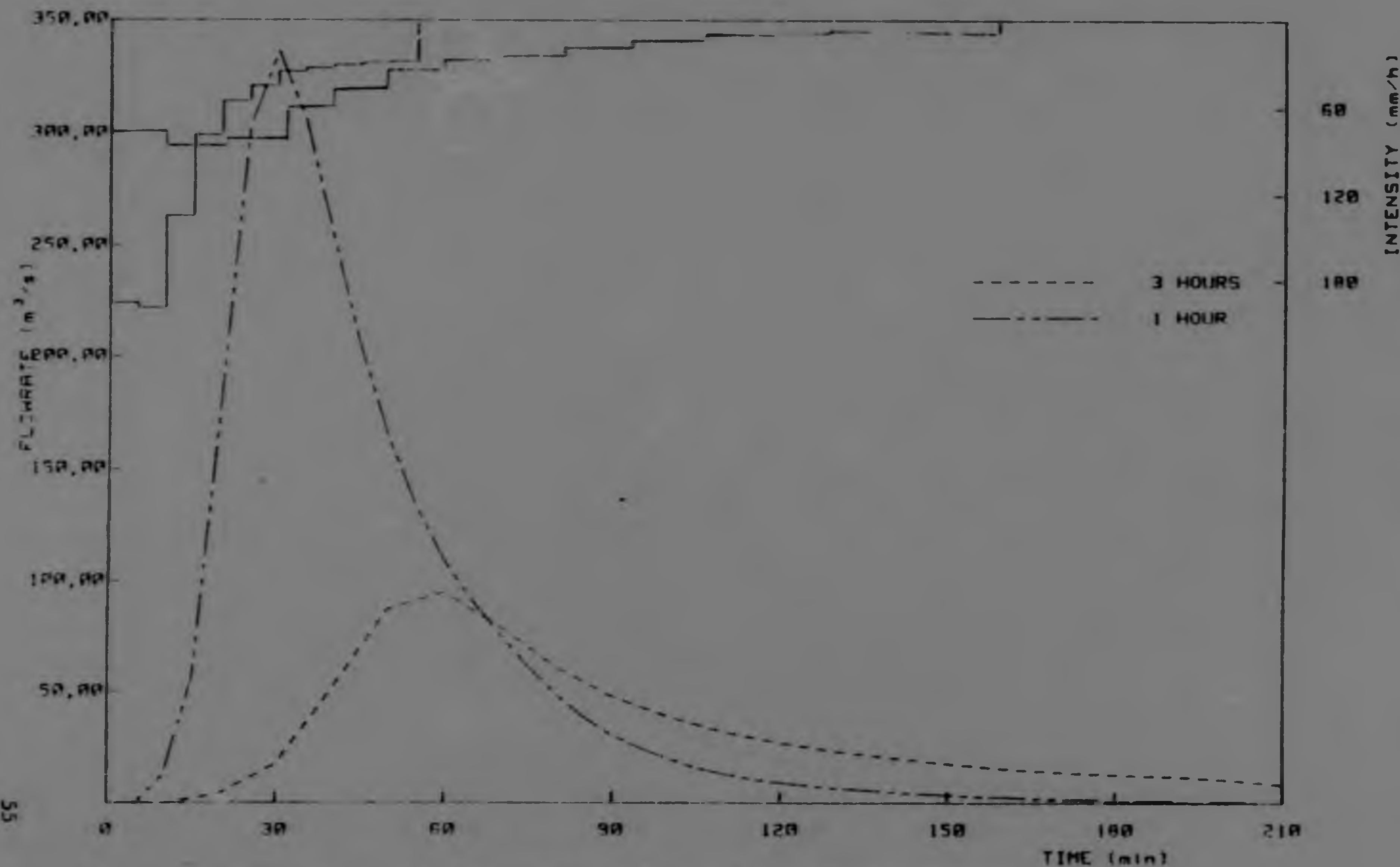

Figure 3.7 Comparison of hydrographs for one and three hours duration of storms (50 years HUFF distribution)

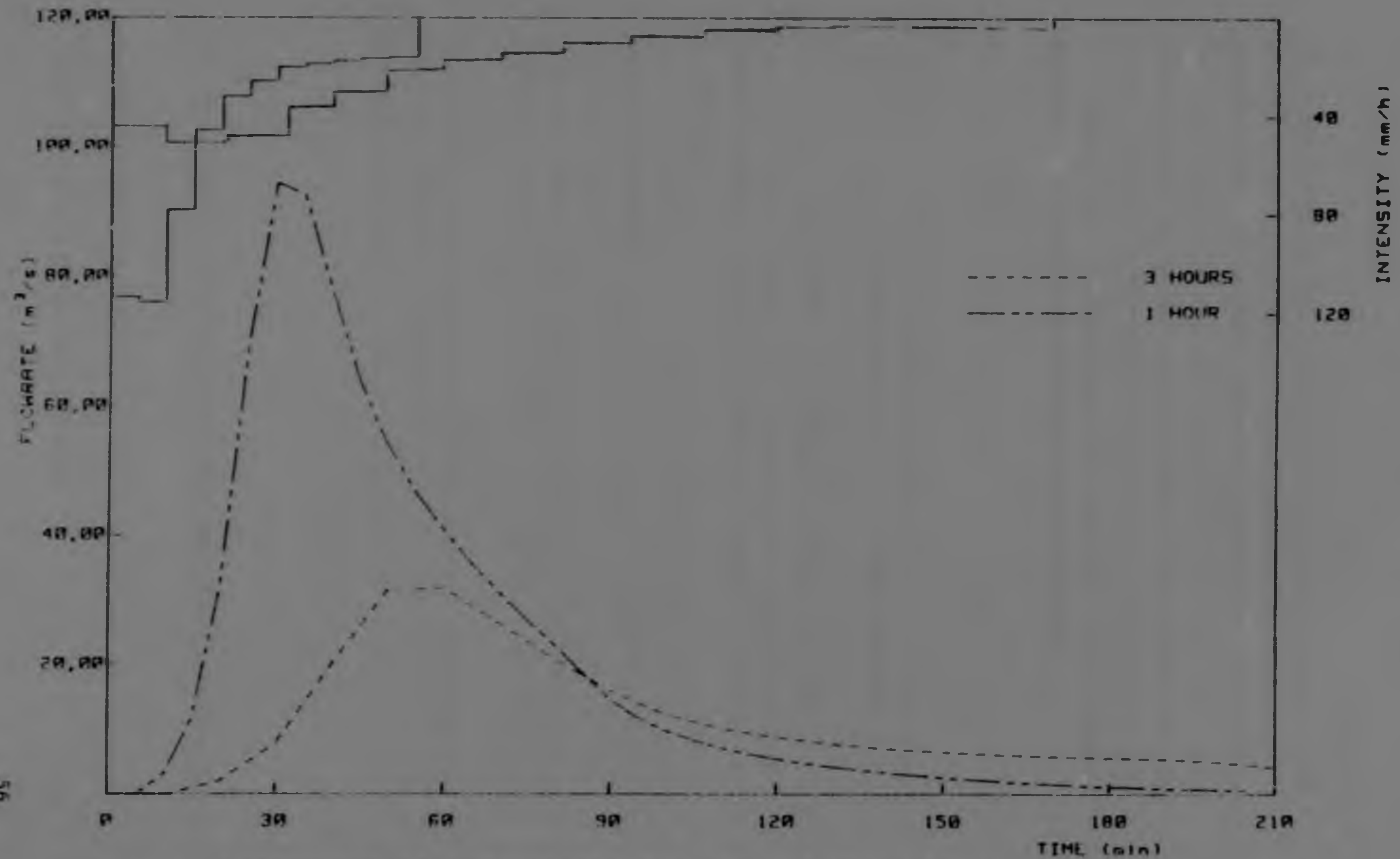

Figure 3.8 Comparison of hydrographs for one and three hours duration of storm (10 years HUFF distribution)

3.7 MAJOR-MINOR SYSTEM

It is not known to what extent street ponding may occur in the case of a major storm. The urbanized hydrographs computed by the parallel reservoir routine, URPHYD, can be separated into minor and major systems hydrographs by the DUHYD routine. The minor system peak flow is the product of the inlet capacity and the number of inlets in the minor drainage system. The major system is simply the mathematical subtraction of the urbanized hydrograph and the minor drainage system hydrograph.

	1 hour duration		3 hours duration	
	MINOR	TOTAL	MINOR	TOTAL
Peak flowrate (m3/sec)	254,20	336,41	69,01	95,61
Perc. difference in simul. peak	24,40%		27,80%	
Volume of flow (m3)	577650	745710	305652	390435
Perc. difference in simul. volume	22,50%		21,70%	

Table 3.5 Comparison of Minor and Total hydrographs

Without doing a detailed analysis it was possible with the use of DUHYD routine to simulate an extreme situation in which flow excess in the main collector is limited to a peak corresponding to a 10 years storm. It was assumed that there is one inlet for every hectare. The simulated hydrographs from the watershed in this case are trapezoidal as shown in Figures 3.9-3.10.

From Table 3.5 it is evident that even in the extreme case where the main collector is limited to a peak corresponding to a 10 year storm and the overland flow will not reach the channel the flow reduction is only 22% of the total volume. The difference in the simulated peak ranges from 24% to 28% for 1 hour and 3 hours duration of storms respectively. However the differences in peaks and volumes for one and three hours is too small for any conclusion to be drawn that could relate the duration of the storm to the response of the minor system.

For shorter duration storms and higher intensity normally it is expected that the capacity of the major system is much bigger than that for storms of three hours duration. This indicates that the minor system - even in this extreme case - responds very fast and collects the larger part of excess rain. By flowing into the drainage system through small pipes and culverts very high velocities result and the whole system responds very rapidly.

It is evident from the results that no provision was made for different design frequencies for the minor and the major systems. Normally the minor system should be designed to accommodate the runoff from the more frequent storms up to the design frequency of the system (e.g. up to once in 5 years).

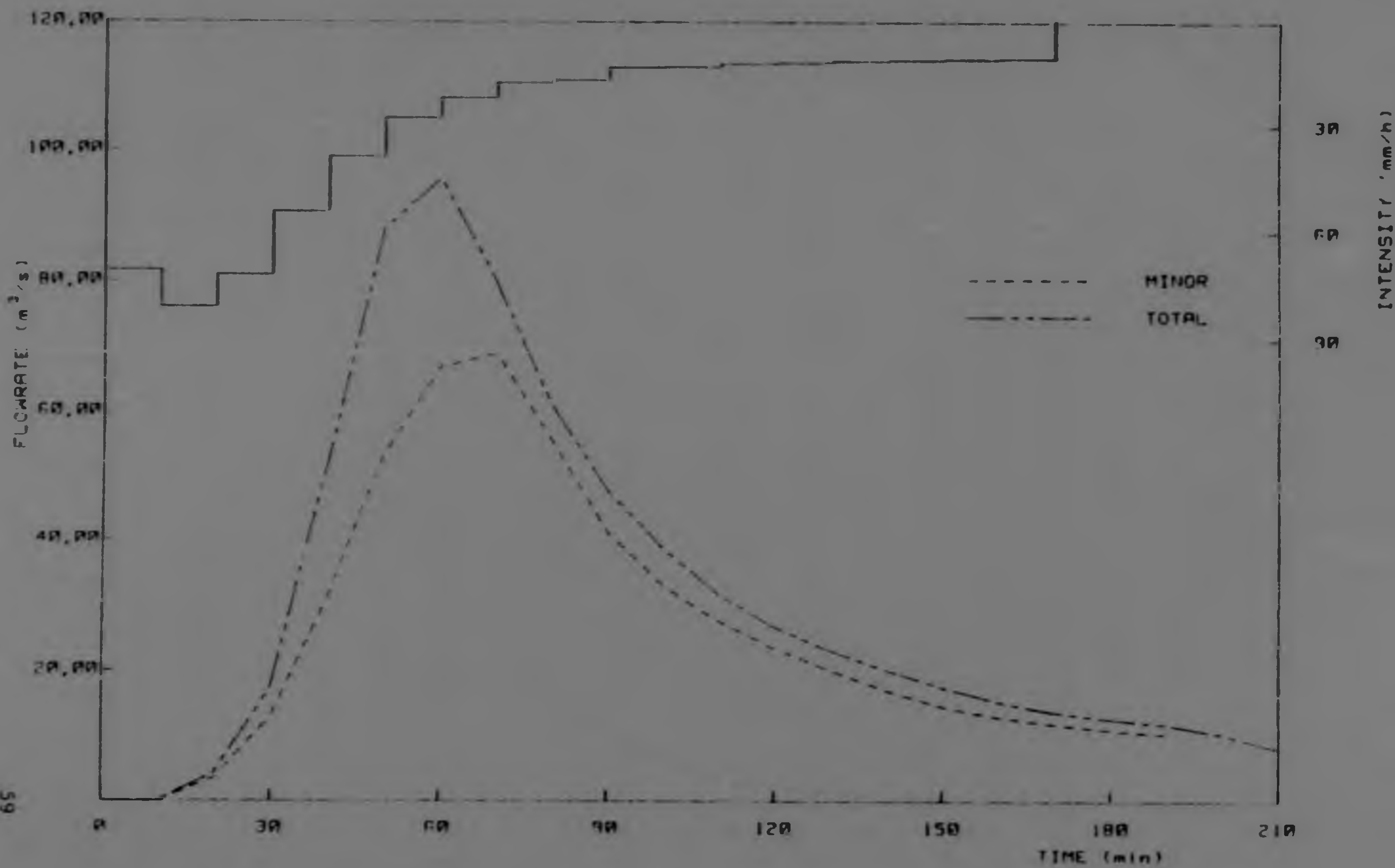

Figure 3.9 Comparison of minor and total hydrographs for 50 years HUFF design storm (3 hours duration)

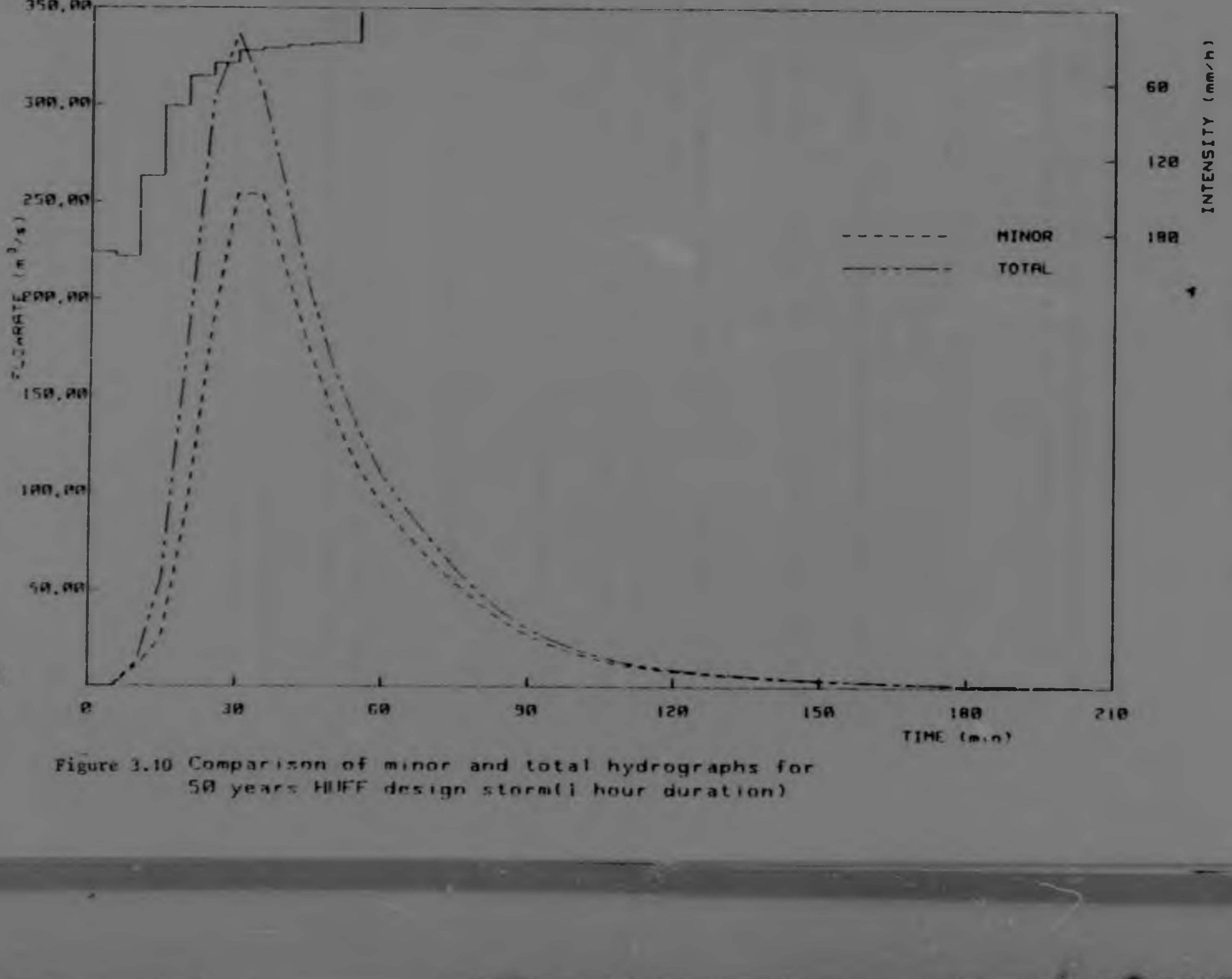

Figure 3.10 Comparison of minor and total hydrographs for 50 years HUFF design storm(1 hour duration)

3.8 REPLACEMENT OF ARTIFICIAL CHANNEL BY NATURAL CHANNEL

In order to illustrate the effect of change from a natural channel to an artificial concrete channel a simulation with trapezoidal cross-section and various roughness was conducted.

R.I.	50 years				10 years			
Roughness	0,014	0,040	0,060	0,080	0,014	0,040	0,060	0,080
Peak flowrate (m3/sec)	95,61	84,70	77,54	70,95	32,04	25,26	21,15	18,28
Perc. difference in simul. peak		12,9%	8,5%	8 3%		21,2%	16,3%	13,6%
Time to peak (min)	60	70	70	80	60	70	80	90

TABLE 3.6 Comparison for Different Roughness

The artificial channel was replaced from node 3 upstream to joint 15. From node 15 to node 22 the channel was already natural (Figure 3.4). The dimensions were increased in order to reduce the mean velocity to a range of 1,5 m/sec - 2,0 m/sec. through the whole main channel. So, the width at the physical channel ranges from 6 meters upstream to 10 meters at the outlet.

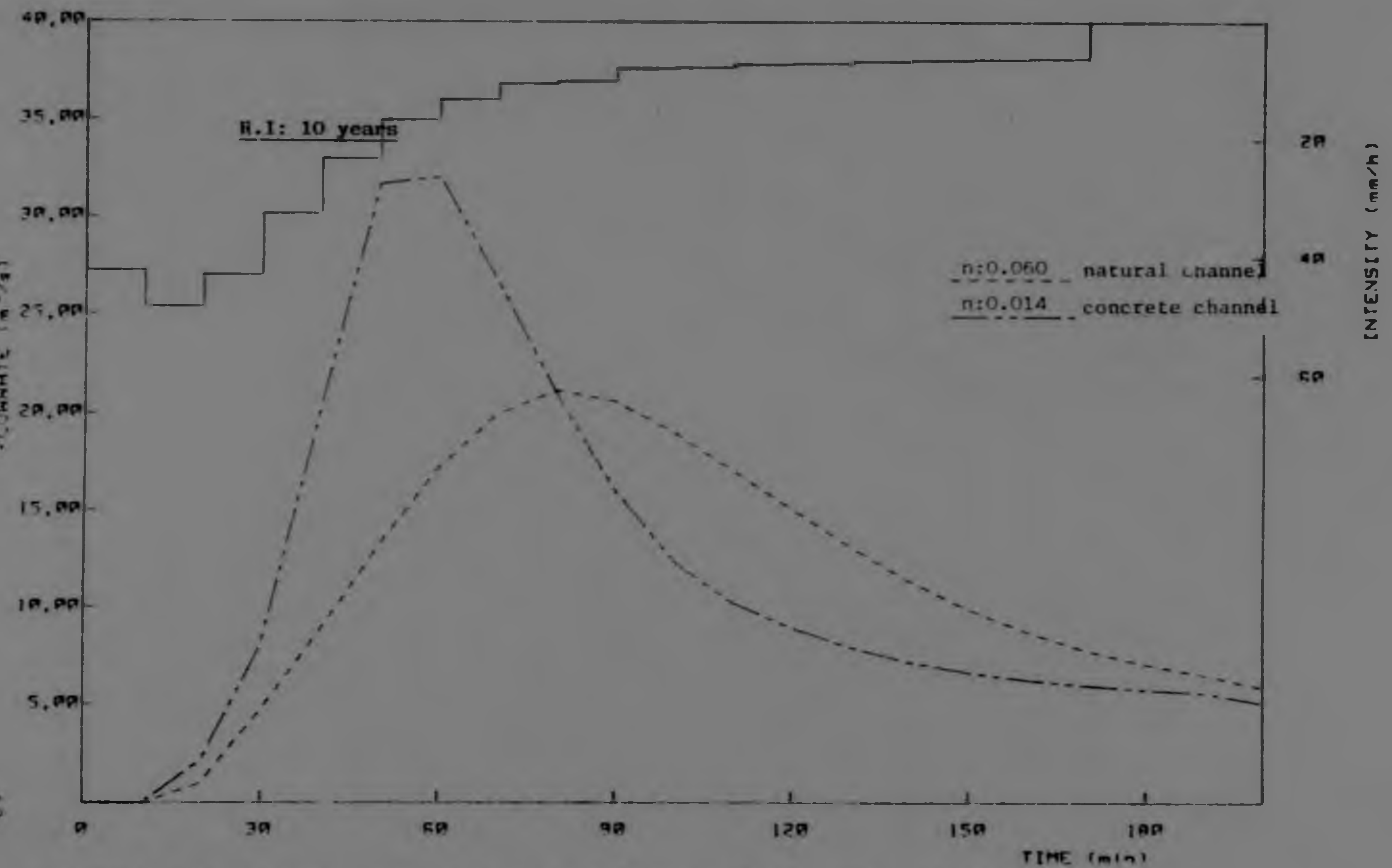

Figure 3.11 Comparison of hydrographs for physical and concrete chanbls from Braamfontein Spruit watershed

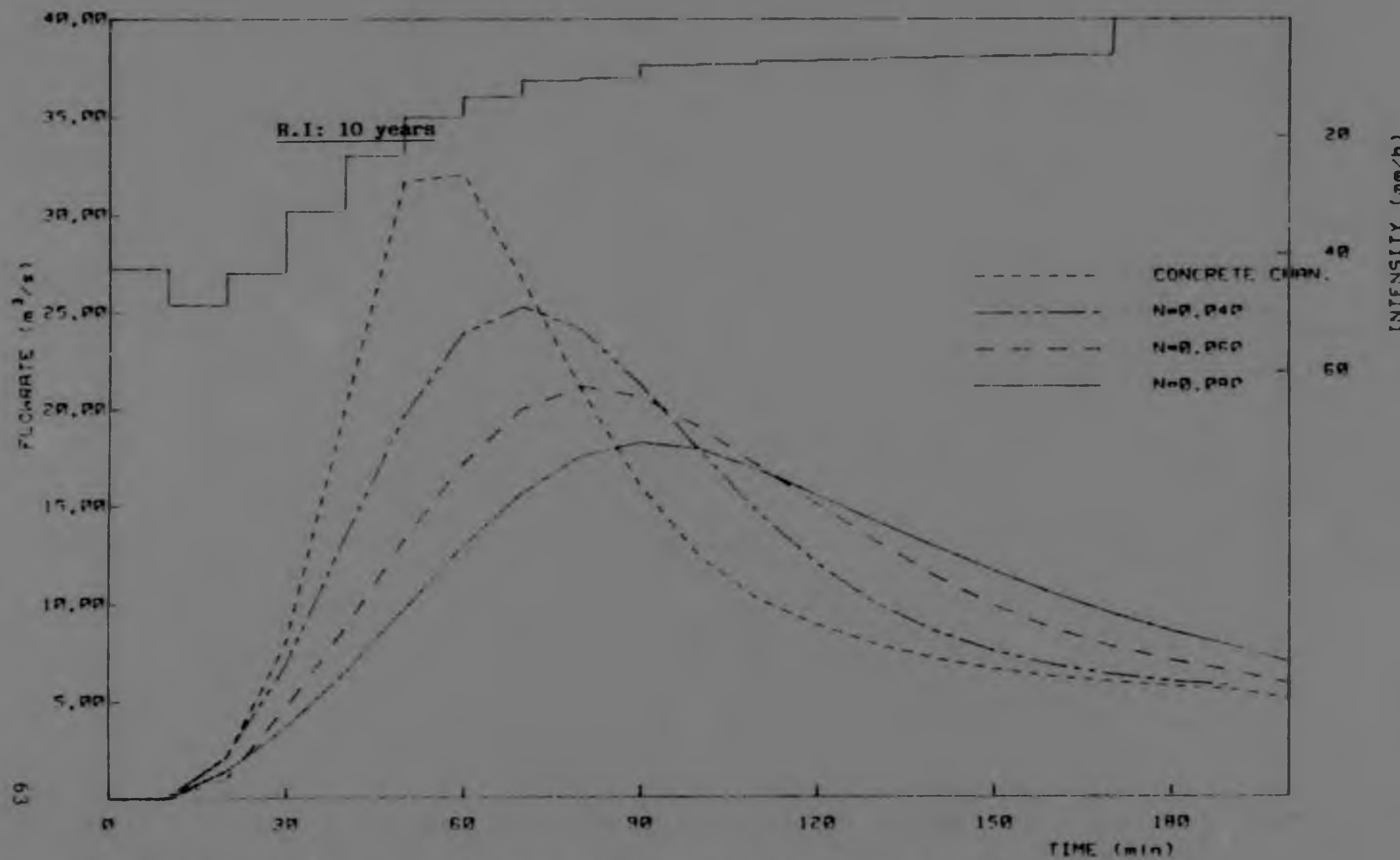

Figure 3.12 Sensitivity analysis of Manning's roughness coefficient for physical channalization in Braamfont. watershed

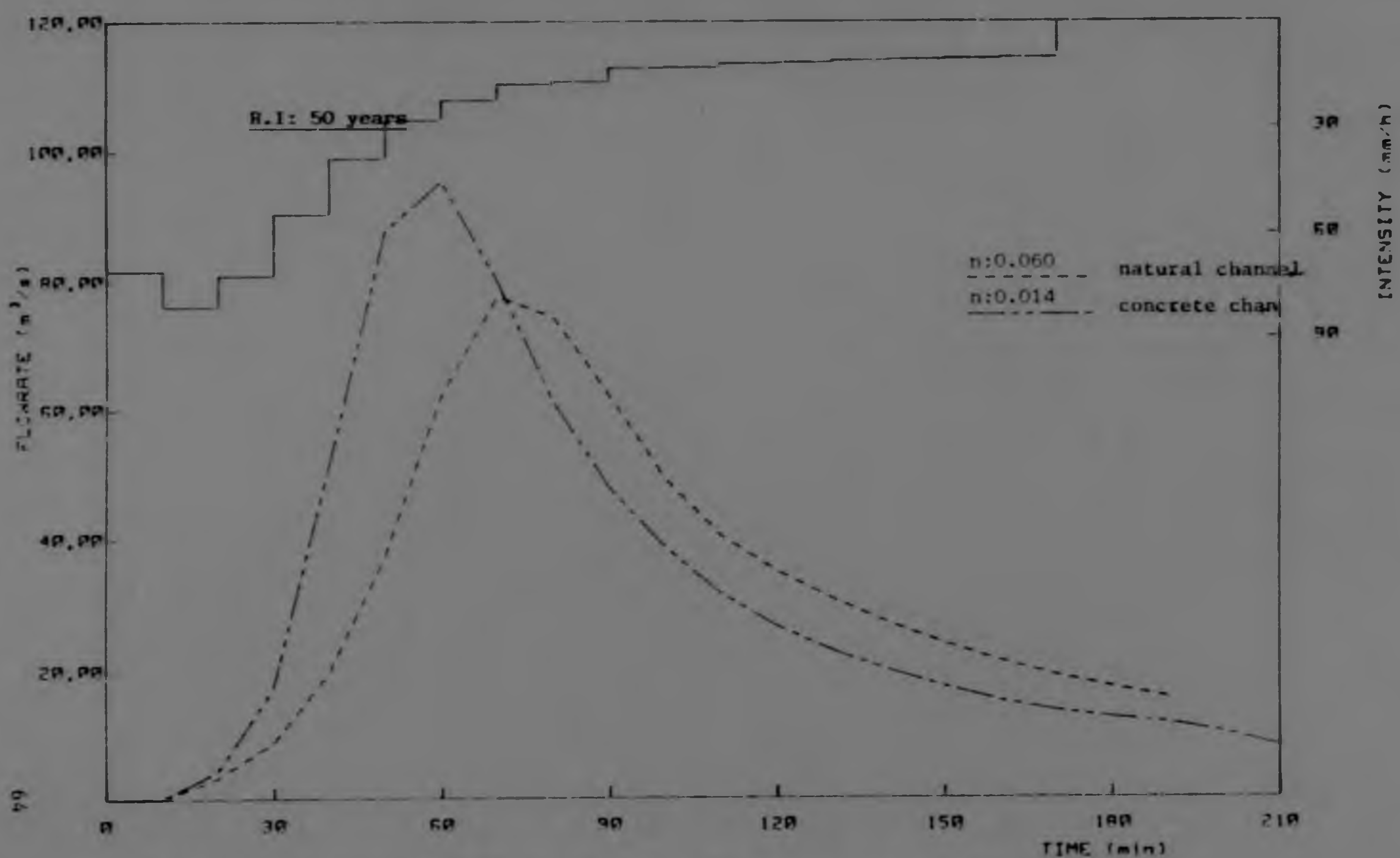

Figure 3.13 Comparison of hydrographs for physical and concrete chanals from Braamfontein Spruit watershed

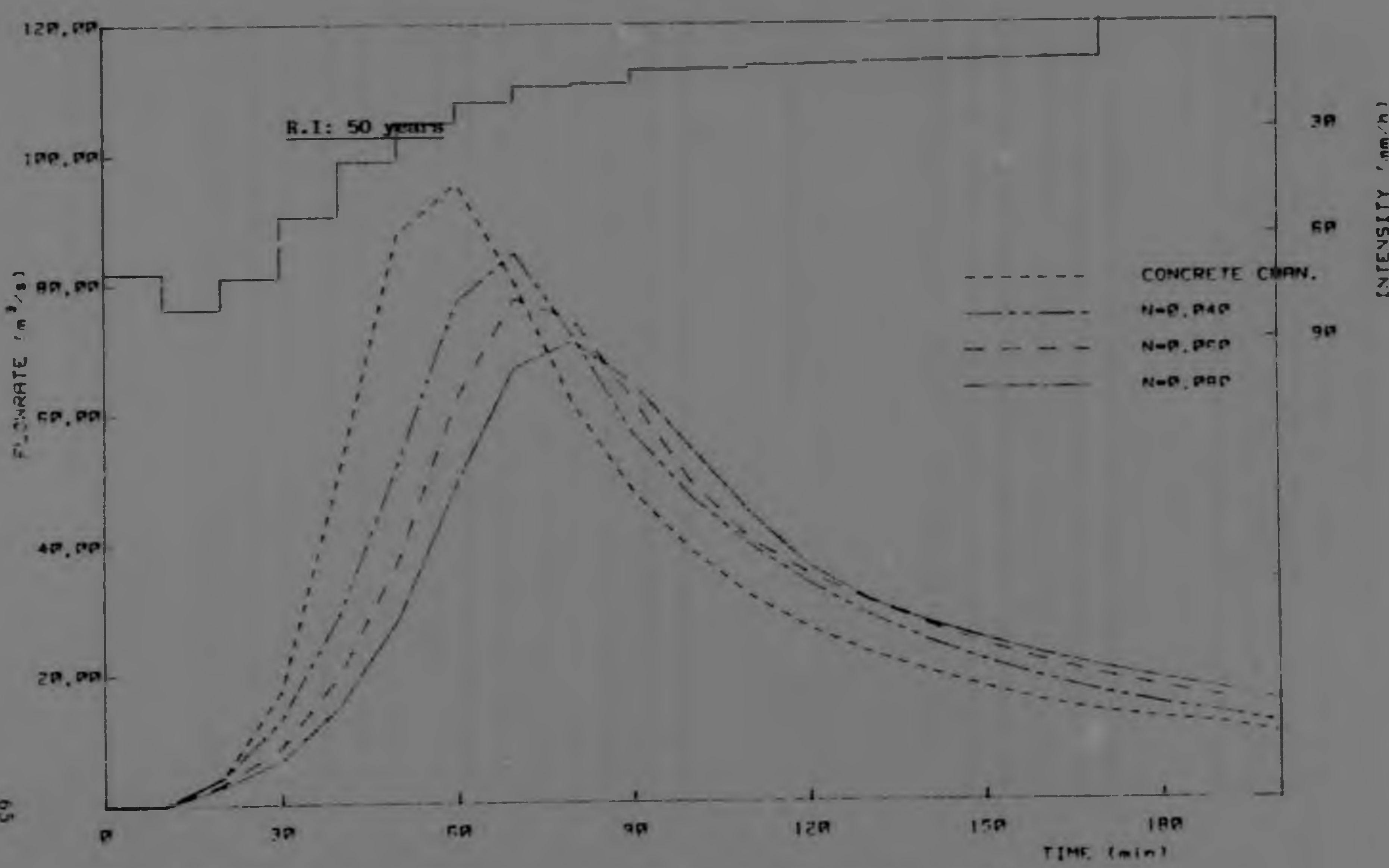

Figure 3.14 Sensitivity analysis of Manning's roughness coefficient
for physical channalization in Braamfont. watershed

Even if such a width is not applicable in practise the results of the natural channel are evident. Table 3.6 and Figures 3.8 to 3.12 indicate a reduction of peak flowrate of 26% for the 50 years HUFF storm and 40% for a 10 years HUFF storm (3 hours duration). The time of the peak is increased in both cases by more than 20 minutes. So, the effects of artificial channels with such a large width (6 meters) are to reduce the concentration time and increase the peak.

The concentration time is reduced, the system response is faster and as a result shorter, sharper showers are the worst from the point of view of runoff peak. In the conduits the water travels faster so the correct retardation is important. Increase in the width of channels has the same effect as an increase in roughness which then increases storage due to the decrease of velocity.

3.9 DISCUSSION OF RESULTS

The drainage system of Braamfontein Spruit watershed was designed using traditional philosophy: to collect the runoff and carry it away as fast as possible out of the boundaries of the watershed.

The negative consequences of this philosophy were evident through-out this analysis. For the 50 years HUFF storm the peak discharge at the outlet was more than 300 m3/sec and the time of concentration for such a large catchment only 30 minutes.

The analysis of the minor and major systems proved that even in extreme cases the larger part of the water is collected by the minor system and so it develops very high velocities thus the whole system responds very fast. This indicates that in the design of the watershed no separation in the study was made for major and minor systems and both were designed for the same design frequency.

On the contrary, the minor drainage system should have been designed for shorter design frequency (e.g. 1:5 years) in each subbasin and in addition it should also have been designed to capture no more than this amount of runoff.

The keystone to good watershed drainage is the appropriate design of the major system, since it should accommodate the runoff from very infrequent storms such as once in a hundred years.

In the previous section it was proved that even if one part of the main channel is replaced by natural waterway, or the concrete channel had not been constructed, the reduction of the peak would be more than 30% and an important attenuation of the hydrograph would occur. Even if the size of the natural channel is unrealistic it is evident that the incorporation in the design of natural waterways, valleys, and man-made swales is very important. Most natural channels meander back and forth across their flood plain and consequently have relatively small slopes, lower velocities, and longer paths. In the design of the channels it is possible to replicate natural conditions by having relatively straight floodways and again can be particularly attractive when incorporated into a continuous greenway system.

4.0 DEVELOPMENT OF A PROGRAM FOR ATTENUATING URBAN FLOODS

4.1 INTRODUCTION

At the commencement of this study the existing drainage system of the Braamfontein Spruit watershed was analysed. The objective was not a detailed pipe analysis with respect to individual flows and volumes but an analysis of the behaviour of the total watershed, thus a lumped model like OTTHYMO was used.

However even if OTTHYMO model helped in the identification of the problem it is a model based on the controversial unit hydrograph theory and the semi-empirical Muskingum method for routing. In order to study the effects of urbanization on stormwater drainage a more accurate model,for instance based on kinematic theory, should be used.

Urban drainage models currently available are: SWMM, ILLUDAS AND WITWAT II . SWMM is a main-frame orientated model and not very friendly to use. Several algorithms in this model are somewhat outdated and analysis of dual systems is not possible. ILLUDAS is a model based on the wrong concept of the time-area method (time of concentration is unique for each catchment and equated to the storm duration). WITWAT is a model based on kinematic theory and has already been tested in several catchments and has been proved fairly accurate and the most friendly to use (Green 1985). However, as it will be shown in the next chapter, WITWAT is a model, very sufficient for practical applications but with simplifications (mainly in routing procedures and storage) and in its present form not appropriate for this particular scientific research. Also the routing is limited to pipes and rectangular channels so it is difficult to use it in rural areas or for the study of the effects of change of canalization on various types of catchments.

WITWAT was therefore modified in order to account for all the above-mentioned cases. A new model, WITWAT III, was developed which is capable of simulating the runoff in urban or rural areas more efficiently. The structure of the model has been based on the paper by Alley et al., (1980), which describes the model currently used by the U.S.Geological Survey and at Stanford University. Because it is based on kinematic wave theory this model cannot indicate backwater effects, and is therefore inappropriate for very flat catchments, or for stormwater flows in which the kinetic energy forms a large proportion of the upstream potential energy (Brakensiek, D.L. 1967).

4.2 ROUTING IN CONDUITS, CONNECTIVITY AND STORAGE IN WITWAT II

4.2.1 ROUTING IN CONDUITS

The overland flow contribution to runoff can be routed by the model through pipes or rectangular channels networks to the outlet of the system. The routing technique for pipes is the time-shift method whereas for channels the time-shift or the kinematic method may be used.

According to Constantinides(1982), time-shift routing is the simplest and most commonly used method. He points out that since a numerical solution of the kinematic equations is tedious, use of these equations may not always be justified for routing hydrographs through conduits. He compared the effects of employing either time-shift routing or kinematic routing for circular and trapezoidal conduits, producing graphs indicating when time-shift routing could be used with a loss of accuracy of less than 10% compared with kinematic routing.

Time-shift method is based on the assumption that the hydrograph ordinates are preserved after being lagged or shifted in time. Storage considerations within the conduit are therefore not accounted for. The velocity of the flow in the conduit, which is used for compute the lag time, is usually taken to be the uniform flow velocity when the conduit is almost full (Constantinides ,1982) although the velocity of the inflow hydrograph peak in the conduit under steady conditions has also been used (Watson, 1981). In general, it is more likely that time-shift routing may be acceptable when short pipes with small diameters and steep gradients are involved, and that kinematic routing should be used when dealing with large diameter pipes of considerable lengths and flatter gradients.

WITWAT II uses for routing through channels, as an option, an identical kinematic routine to that used for the catchments. As one neglects the frictional effects of the sides of the channel on the flow (setting the hydraulic radius equal to the flow depth), for shallow flows only and wide channels, this is a valid approximation. However, as flow depths increase, friction on the channel sides becomes relatively more important and setting the hydraulic radius equal to the flow depth is no longer a valid assumption (Green 1984). This routing procedure also presents problems with numerical instability and it may be found that time-consuming smaller time steps have to be used. The channels are not sub-divided in smaller lengths and the whole procedure may be looked upon rather as a storage routing or mass balance technique which assumes uniform low over the length of the channel (Green 1985).

4.2.2 CONNECTIVITY AND STORAGE

WITWAT II is a very flexible model with regard to the numbering scheme for the area and the conduit network. A connectivity routine sets up a connectivity matrix and in this way additional areas or conduits can be added or deleted at will without necessarily affecting the numbering scheme imposed by the user. However, a very simplistic storage routine has been incorporated into the connectivity routine. In the analysis mode, when pipes or channels are undersized for the particular flowrate encountered, the excess flowrates are stored at the nodes respectively. So once the flowrate exceeds the conduit capacity, the flowrate is set equal to the conduit capacity and the excess flowrate is

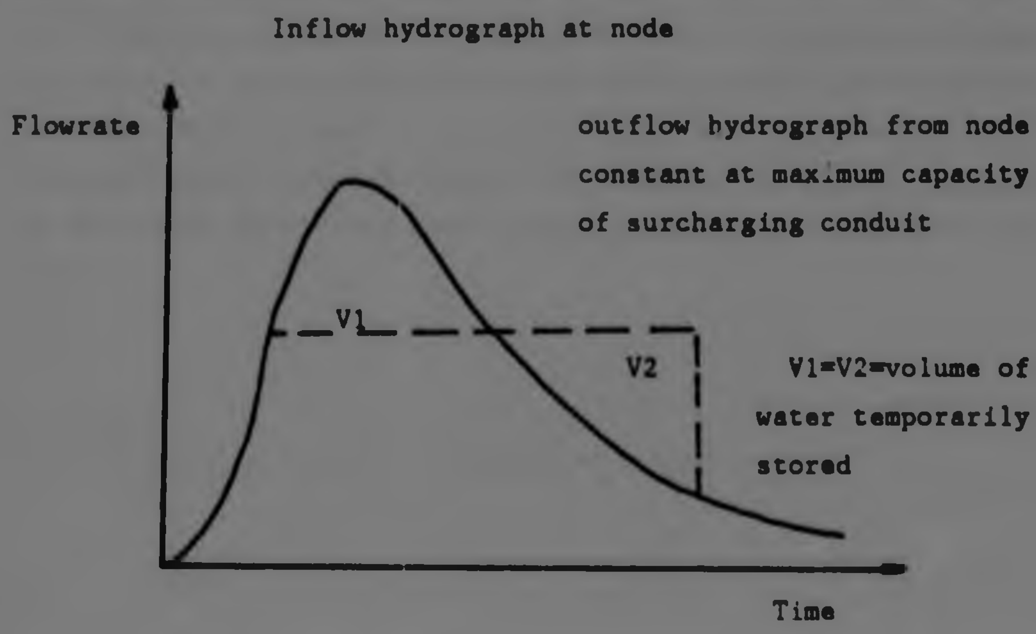

Figure 4.1 Schematic representation of inflow and outflow hydrographs for a surcharging conduit (Green 1984)

stored at the node according to the continuity equation,

$$\Delta S = I\Delta t - O\Delta t \tag{4.1}$$

where ΔS is the change in storage
I is the inflow rate
O is the outflow rate(here equal to conduit capacity)
Δt is the time-step

A schematic representation of this process is given in Figure 4.1.

With a knowledge of the flowrate at the node, whether it is surcharged or not the current and a number of previous time-steps as well as the lag time to the node draining it, the lagged flowrate is computed for the current time-step. The flowrate from the following node in the connectivity matrix is similarly lagged to all its successive downstream nodes until the outlet is reached. This process is repeated until all the contributions from all nodes have been lagged and added to the already existing flowrates at downstream nodes before proceeding to the next time step.

4.3 INTRODUCTION TO KINEMATIC WAVE THEORY

4.3.1 EQUATIONS FOR ONE-DIMENSIONAL FLOW

One-dimensional flow implies flow along one axis only, while two-dimensional flow refers to the components of flow in two directions perpendicular to each other.

The one-dimensional kinematic equations may be derived by simplifying the dynamic equations for the one-dimensional case. The latter equations are commonly known as the St.Venant equations. The St.Venant equations are in turn a simplified version of the more general equations of motion known as the Navier-Stokes equations (Cooley et al. 1976). They can be derived in two ways: firstly by reducing the general equations of flow by suitable assumptions and secondly by making similar assumptions and then considering the [illegible]les o[illegible] continuity and momentum balance. The first meth[illegible]e rigorous while the second method may be found in most relevant literature, e.g. Chow (1959)

In both methods the following assumptions are made

1. The fluid is homogeneous and incompressible.

2. Flow is one-dimensional i.e. fluid acceleration in all directions other than the direction of the flow is negligible.

3. Flow must be gradually varied i.e. no rapid change in the flow cross-sectional must exist.

4. The friction slope is approximated by the energy slope.

5. Velocity is constant across any section.

6. Pressure distribution across any section is hydrostatic.

7. Lateral inflow into or out of the main flow carries no considerable momentum.

8. The bed gradient is small so that $\theta \simeq \sin\theta \simeq \tan\theta$

These assumptions require that there is no rapid change in flow cross-sections. In particular, there should be no flow separation and the flow should not be highly curvilinear i.e. the St. Venant equations are inaccurate when the Froude number of the flow is close to unity, and they are unreliable in highly curvilinear flows, curved or highly prismatic channels, e.g. under sluice gates, around entrances, drop outlets, junctions, weirs and other similar structures. It must also be realised that the St. Venant equations cannot describe accurately discontinuities in flow such as a surge or a hydraulic jump. The continuity equation may be derived by balancing flow across the boundaries of an element as shown in Figure 4.2.

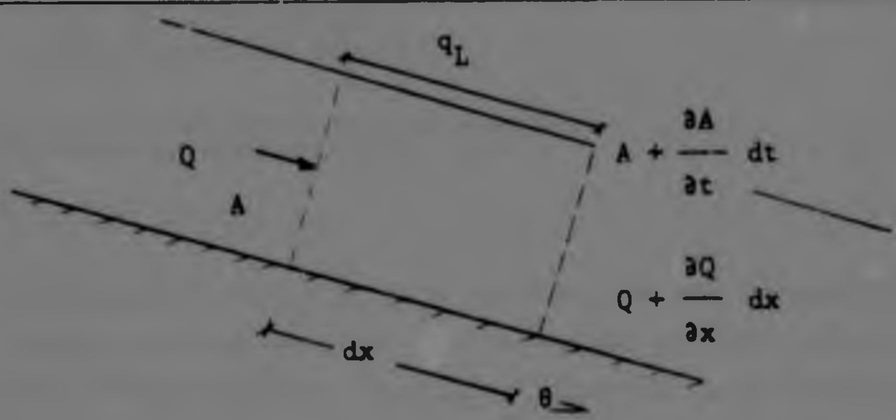

Figure 4.2. Continuity of flow

Equating the difference between inflow and outflow to the rate of increase in storage results directly in the following equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q_l \qquad (4.2)$$

where Q is the flow rate,
A is the cross-sectional area,
q_L is the lateral inflow per unit length along the x axis
and t is time

In order to compare the effect of steady with unsteady flow and uniform with non-uniform flow , the St.Venant dynamic equation can be rearranged as:

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} - \frac{q_1 v}{Ag} \qquad (4.3)$$

Steady uniform flow

Steady non-uniform flow

Unsteady non-uniform flow

The kinematic equations are derived from the dynamic ones. The analysis can be found in many books and it will not be repeated here. The kinematic continuity equation is the same as the dynamic continuity equation. For overland flow where a constant width is considered the continuity equation becomes

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i_e \qquad (4.4)$$

where q is the average discharge across a section per unit width
and i_e is the excess rainfall rate.

Kinematic theory furthermore relies on the assumption that the discharge at any point is a function of the water depth only, i.e.

$$Q = ay^m \qquad (4.5)$$

This is a form of relationship which most equations describing hydraulic resistance to uniform flow obey. Therefore a and m are parameters (assumed constant) depending on the uniform flow friction equation used, with a being a function of the acceleration due to gravity and bed slope. (Since the friction slope is assumed to be equal to the bed slope in kinematic theory). Equations 4.4 and 4.5 are therefore the kinematic equations used to describe one-dimensional overland flow. It is useful to bear in mind that the use of the kinematic equations is limited by the assumptions made in reducing the dynamic equations to the kinematic form as well as by the assumptions used in deriving the dynamic equations (Yen, B.C. 1973).

4.3.2 ANALYTICAL SOLUTION OF THE KINEMATIC EQUATIONS

The most widely used method in the past to obtain analytical solutions of the kinematic equations is the method of characteristics. It describes unsteady flow in the form of a wave or a disturbance travelling along the flow at a specific velocity, dx/dt. The family of curves described by dx/dt in the x-t plane is then called the characteristic. The flow properties, depth and discharge along each characteristic are described by relationships obtained from the unsteady flow equations using the relationship of dx/dt. In other words the relationships derived describe the flow properties as seen by an observer travelling along the flow at a velocity defined by the characteristics.

In the case of the kinematic equations in the overland flow problem a disturbance will be caused by a discharge gradient occurring anywhere on the catchment. If one travels together with this disturbance at a velocity dx/dt along the flow one will not observe a discharge gradient but the water level appears to rise linearly with excess rainfall intensity. This observation implies that one can define along the characteristic curves

$$\frac{dy}{dt} = i_e \tag{4.6}$$

Using equations 4.4, 4.5 and 4.6 one can derive an expression for the disturbance velocity, dx/dt. Equation 4.6 can be expanded in terms of partial differentials as follows

$$\frac{dy}{dt} = \frac{\partial y}{\partial t} + \frac{\partial y}{\partial x}\frac{dx}{dt} = i_e \tag{4.7}$$

Differentiating equation 4.5 with respect to x and substituting in equation 4.4 gives

$$\frac{\partial y}{\partial t} + \frac{\partial y}{\partial x} amy^{m-1} = i_e \tag{4.8}$$

Comparing equations 4.7 and 4.8 gives the disturbance velocity dx/dt, i.e.

$$\frac{dx}{dt} = amy^{m-1} \tag{4.9}$$

Using the above equations one can obtain expressions for the flow properties along any characteristic.

The concept of information propagation in time and space along a characteristic is illustrated in Figure 4.3.

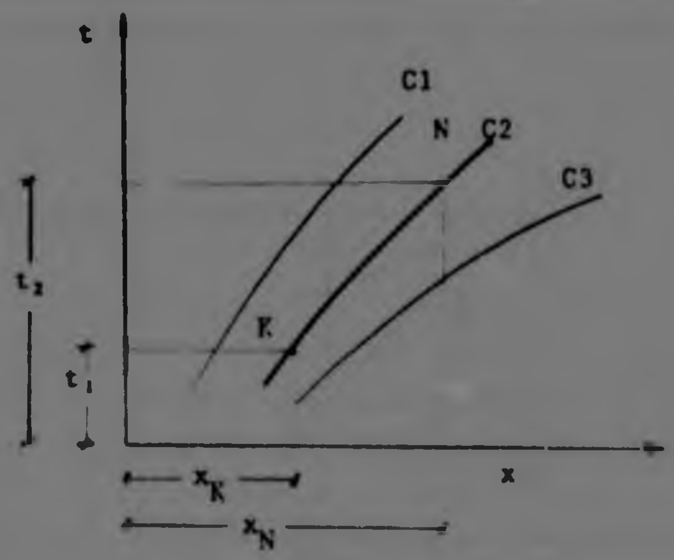

Figure 4.3 Propagation of information along characteristics of the kinematic equations

In Fig. 4.3, C1, C2, C3 are a set of characteristics described by equation 4.8. For each characteristic there is a corresponding set of trajectories. A trajectory is the path traced in the y-x plane by an observer moving with a velocity of dx/dt i.e. it represents the water depth as seen by the observer. The kinematic wave speed, i.e. the effective speed of propagation of flow is given by the expression

$$w=dQ/dA \tag{4.10}$$

If the kinematic wave speed is equal the ratio dx/dt then this wave speed will follow a trajectory that would pass exactly through points K and N (Abbot, M B. 1979).

Alley et al.(1980) proposed a method for solving equations 4.2 and 4.5 by using a finite difference approximation. Four points of a finite- difference mesh are illustrated in Figure 4.4.

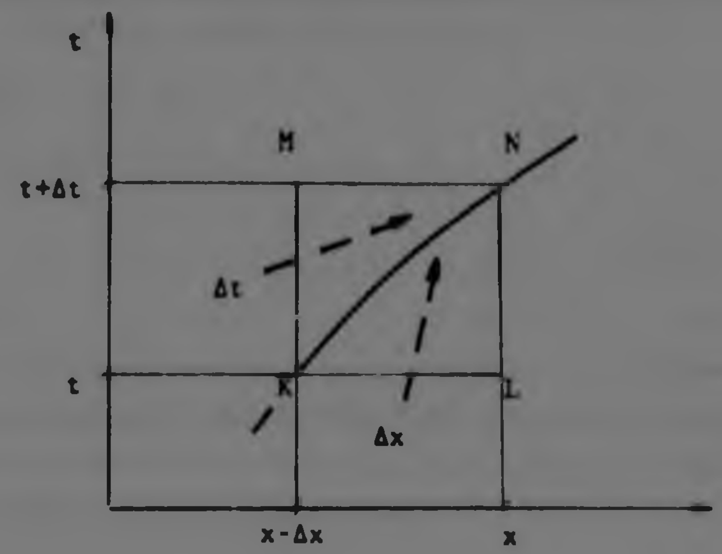

Figure 4.4 Four-point Finite-Difference Mesh

The finite-difference equations are solved for A and Q at point N. In order to obtain the flow properties of a point N which lies on the same characteristic as point K two finite difference-methods of solution based on equations 4.2 and 4.5 are used (Bradey D.K. 1984).

In order to avoid the convergence and stability problems that can occur with particular numerical grid spacings (the relative sizes of Δt and Δx) Alley et al. created a dimensionless parameter, theta, which is the ratio of the kinematic wave speed to the local gradient, thus:

$$\text{theta} = w/(dx/dt) \qquad (4.11)$$

Line KN represents the case which theta is equal 1. If theta exceeds 1 (the kinematic wave speed exceeds dx/dt) flow characteristics will be propagated toward point N along trajectories passing between points K and M. This involves only mesh points K,M and N. So the equation used will be

$$Q_N = Q_M + q\Delta x - (\Delta x/\Delta t)(A_M - A_K) \tag{4.12}$$

$$A_N = \left[\frac{Q_N}{a}\right]^{1/m} \tag{4.13}$$

When theta is less than unity (kinematic wave speed is relatively slow compared to dx/dt) flow characteristics will be propagated toward point N along trajectories passing between points K and L. Then the equations used are

$$A_N = A_L + q\Delta t + (\Delta t/\Delta x)\,(\,Q_K - Q_L\,) \tag{4.14}$$

$$Q_N = aA_N^{\,m}$$

The kinematic-wave solution is based on the assumption that the disturbances are allowed to propagate only in the downstream direction.

Therefore, the model does not account for backwater effects or flow reversal. In addition the capacity of a circular-pipe or a trapezoidal -conduit segment is limited to non pressurized-flow capacity.

It must also noted that in many real channels a simple real function relating A and Q does not exist or it is inaccurate. In such cases (as in this model) flow has to be deduced from flow area of the cross-section and vice versa.

4.4 MODEL STRUCTURE OF WITWAT III

4.4.1 INTRODUCTION

In order for the model to account for any kind of cross-section it was obvious that an exponential type relation between area of cross-section and flow rate would not suffice. Also at every time-step the value of theta is computed by obtaining first the value of the kinematic wave-speed via equation

$$w = dQ/dA \tag{4.15}$$

In order for the model to accommodate non-power-function channels, a less convenient method for deducing the local value of dQ/dA had to be used since equation 4.5 cannot be used.

It was decided that the model should account for 4 different types of cross-sections most normally encountered, viz:

1. Natural channels

2. Compound channels

3. Pipes with natural channels above

4. Pipes with compound channels above

The latter two cases would give the model the ability to account for dual systems and to represent the real runoff in urban areas.

4.4.2 MAIN STRUCTURE

Storm flows usually commence at very low depths with correspondingly slow kinematic wave velocities, so equation 4.14 dominates the early stages of computation. If values of dx are relatively large everywhere, and if the integration time-step is relatively small, the value of theta may always remain below unity and equations 4.12 and 4.13 may never be used.

During any integration time-step, each successive application of equations 4.12 and 4.14 to the next downstream subdivision of a flow segment involves finding antecedent values for either

If theta<1 A_L, Q_K, Q_L for equation 4.14

If theta>1 A_K for equation 4.12

Since it is known in advance which of these options will be selected, it is necessary to store old values of both A and Q for both the subdivisions under consideration and its immediate upstream neighbour. The latter information may then be discarded as soon as computations commence for the following downstream subdivision (i.e. without waiting till the end of the complete time-step).

By creating only four extra values (A1, A2, Q1, Q2) for temporary storage of the needed old information, and by gradually shifting the role of these variables downstream as appropriate, the need for duplicate arrays is eliminated.

4.4.3 CATCHMENT RUNOFF

In order to make the program more flexible and because the calculation of theta is of major importance for the whole process the separation of the time-step and the rainfall interval became necessary. In WITWAT II the only way to make the time-step shorter was to input the rainfall intensities more times, a tedious process.

The main process for the calculation of the runoff from the catchments was retained, but the infiltration routine, the calculation of the time to runoff after the subtraction of the detention storage and the Newton-Raphson routine were modified in order to be adjusted to the time-step instead to the rainfall interval.

4.4.3.1 Depression Storage

Depression storage is considered as an initial abstraction, runoff commencing once the volume of rainfall exceeds the volume available for depression storage. Even though this assumption may not be entirely correct, it was retained. The time to runoff from the pervious and the impervious catchment was recalculated according to the time-step.

4.4.3.2 Infiltration

The infiltration routine for the modified Horton equation was changed and now the incremental volume of water infiltrated is calculated over a single time-step and not over every rainfall interval. In this way the incremental volume may be calculated in smaller time-steps and thus more accurately

4.4.3.3 Overland flow routing

For the overland flow routing in WITWAT II the kinematic theory was adopted. Equation 4.2 was solved with respect to depth using the Newton-Raphson technique (Green 1984). The same routine is used in WITWAT II for channel routing when the kinematic option is specified by the user. In WITWAT III this option was suppressed, the routine was written again and was adjusted only for overland flow.

4.4.4 CONNECTIVITY

In WITWAT II connectivity and storage were incorporated into one routine which would have been impractical to separate. So a new connectivity routine was built up which was as flexible as that in WITWAT II.

The logic of the new routine is completely different and as in WITWAT II allows the user to input areas and conduits in any order. As shown in the logic diagram 4.6 the routine does not built up a connectivity matrix but is based on two arrays, UP(I) and J18(Ii). Conduits are classified in two categories.

1. Conduits that have only catchments upstream.

2. Conduits that have conduits upstream.

If a conduit J has a catchment upstream then UP(J)=0. The other array contains the order of routing. The program starts from a random conduit in the network and searches for an upstream channel. If it finds one, it transfers to the upstream conduit and searches once again for a channel. If one is not found, this means that catchments are upstream of the J conduit. Therefore this is the conduit that will be routed first. For this first conduit the values of the arrays is UP(J)=C and J18(1)=J. Thus all the conduits in the network are reordered according to the arrays J18(J). Those that have only catchments upstream are kept in memory with array UP(J). A third array, is used in order to remember the branches through which the program has already passed, thus avoiding duplication to pass again. Using this third array, the number of the upstream conduits concentrated on a node is unlimited. During routing if the program identify a conduit that has catchments upstream searches for these catchments and adds the runoff. The program therefore allows for more than one catchment contributing to the same node.

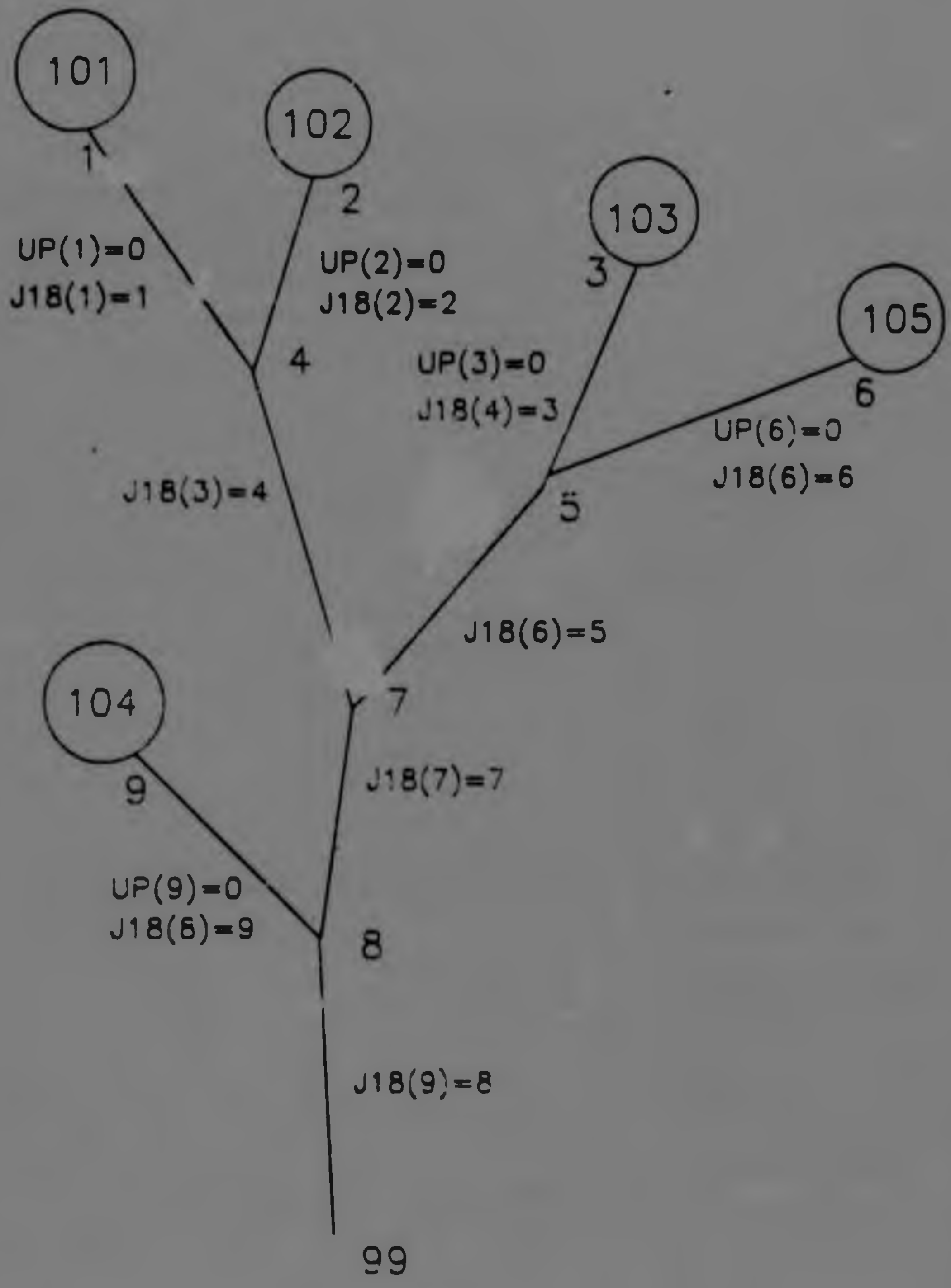

Figure 4.3 Example of Connectivity routine
Reordering the conduits
Building up arrays UP(J) and J18(Ii)

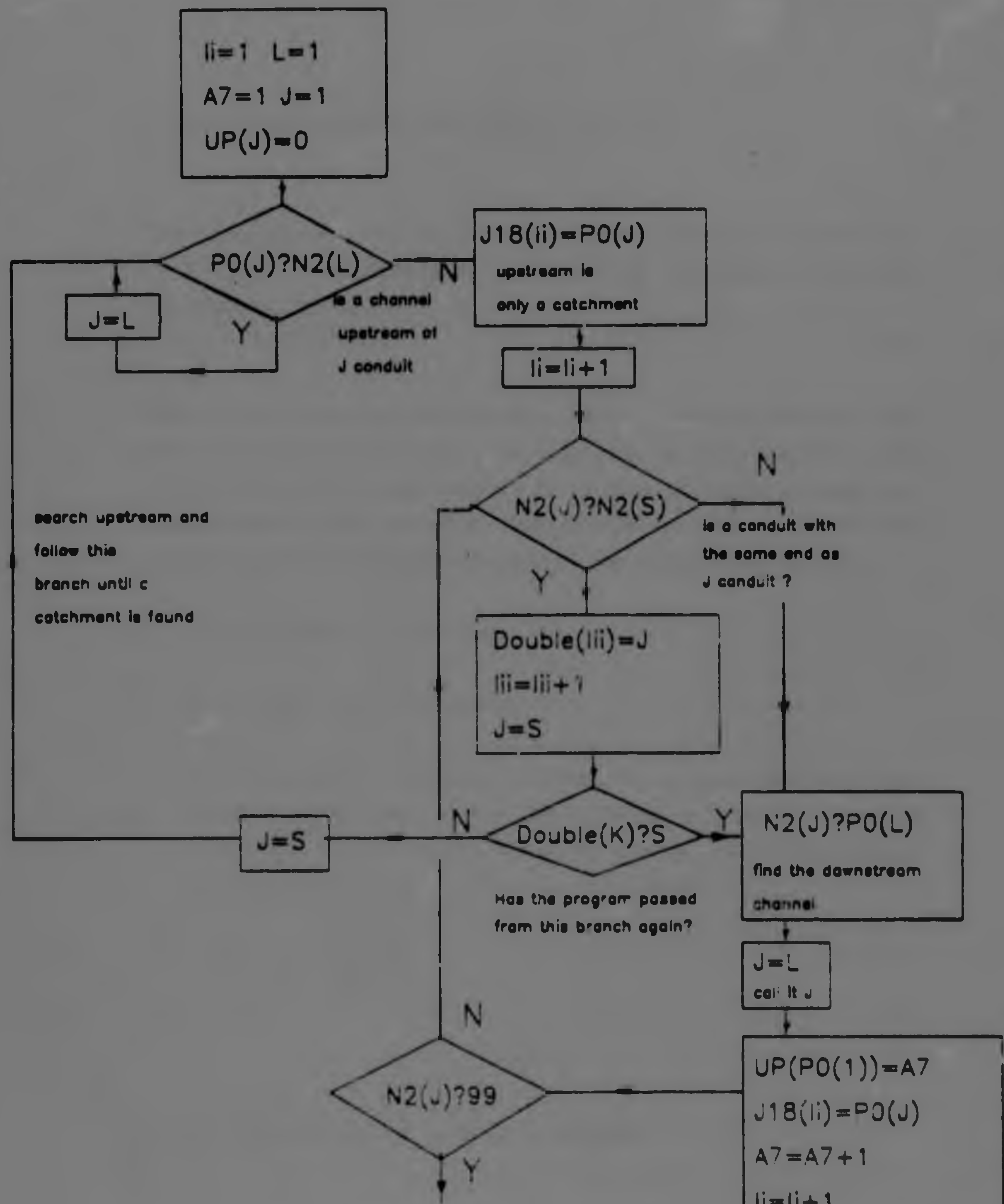

Figure 4.6 Flowchart for connectivity routine

4.4.5 RELATIONSHIP BETWEEN Q AND A

The model proposed by Alley et al. (1980) involves a simplification of the momentum equation (implying dominance of momentum by friction losses)

$$Q = aA^{m} \tag{4.16}$$

However for most cross-sections a direct relation of this type does not exist. Since the main objective in this study is the construction of a model capable of routing the water through natural channels and analysing dual systems the above relation cannot be applied. Therefore a routine was constructed so that:

1. Q is deduced if A is known

2. A is deduced if Q is known

3. The kinematic wave speed, dQ/dA, is estimated with A or Q known (By assuming a very small change in depth the ensuing changes in Q and a may be found and the ratio of these changes approximates dQ/dA.

$$w = \frac{Q1-Q2}{A1-A2} \tag{4.17}$$

4.4.6 DEDUCTION OF Q IF A IS KNOWN

The Newton-Raphson technique is applied for the deduction of Q if A is known. In order to apply the latter technique the derivative of an equation with respect to the depth of flow must be calculated. However the derivative can only be calculated when the water surface has been restricted between 4 points of the cross-section. In order to confine the water surface and derive the depth of flow the following procedure was applied:

Consider the left hand-side of the cross-section (Figure 4.7). Given the area of a cross-section, the routine establishes, as follows, where the water surface is situated (i.e. between which left and right points).

1. It begins by finding the lowest elevation point of the cross-section

2. For depth of flow up to the nearest point, from the latter mentioned point, the area is calculated (A_s)

3. The area which has just been calculated (A_d) and the given area (A_d) are compared. If the calculated area is greater than the given area, then it has been established between which two (left) points the water surface lies. Conversely the routine moves on to the next higher point (on the left) and the above-mentioned process (from step 2) is repeated. The process is shown in Figure 4.8.

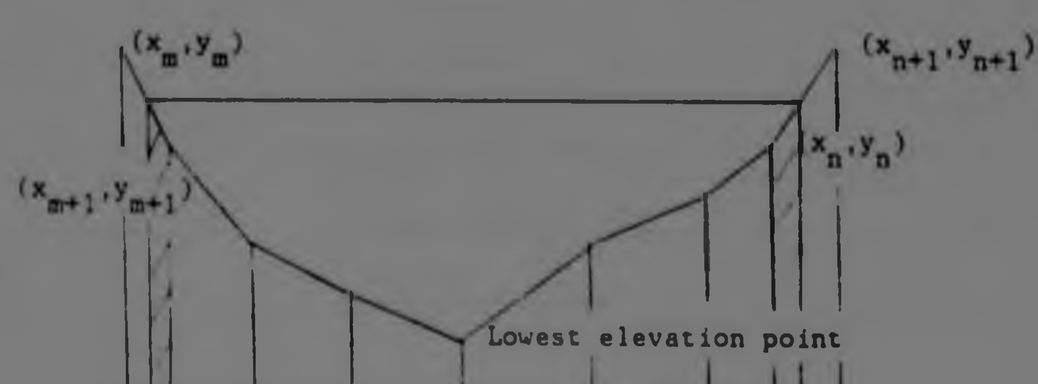

Figure 4.7 Calculation of the depth of flow for any kind of cross-section

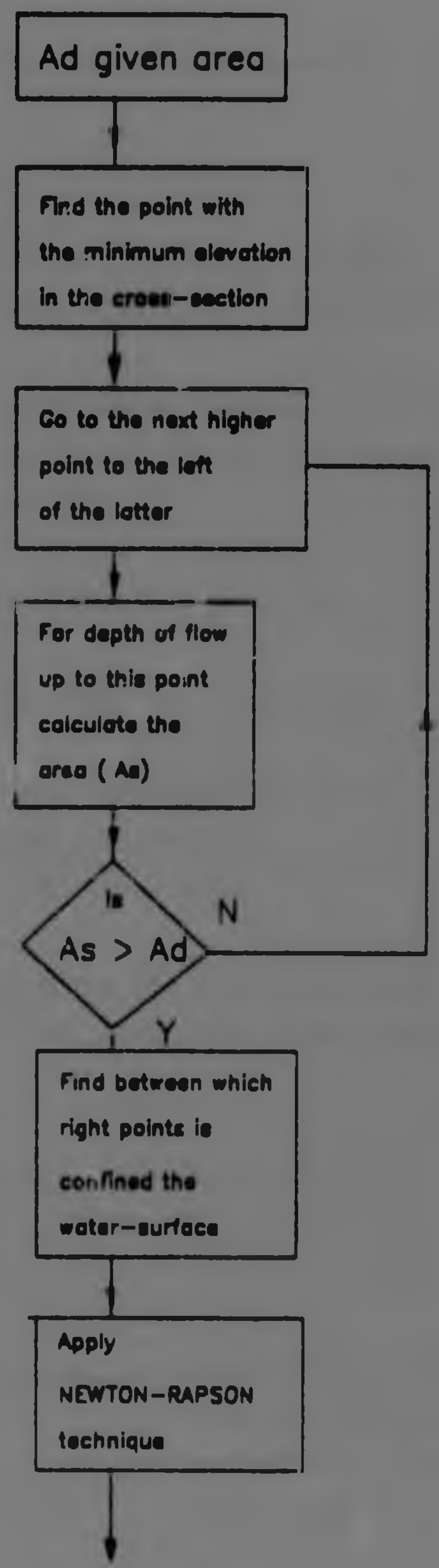

Figure 4.8 Flowchart for the calculation of the flowrate when the Area is known

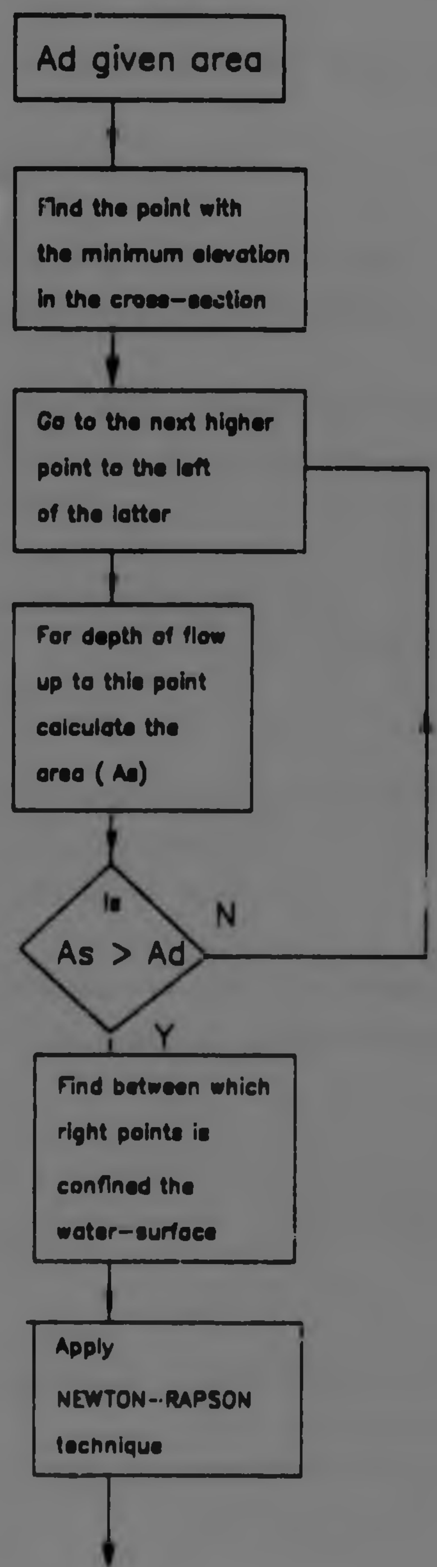

Figure 4.8 Flowchart for the calculation of the flowrate when the Area is known

For the calculation of the area a convenient equation is used

$$A_s=1/2\Sigma((x_{i+1}-x_i)(y_{i+1}+y_i)) \qquad (4.18)$$

where

x_i, y_i : are the coordinates of the points that confine the area

When it has been established between which two left and right points the area is confined, the Newton-Raphson technique is applied to deduce the depth.

The equation 4.18 is written in the form

$$f(x) = A_s(y) - A_D \qquad (4.18a)$$

where

$A_s(y)$ = is the approximation of the area with respect to depth y

A_D = is the given area

Since the points confining the area are known, the only unknown in equation 4.18 is the depth (y). Rewriting equation 4.18 becomes:

$$\begin{aligned} As(y) = {} & [(y_{m+1}-k(y-y_{m+1})) - (x_n+\lambda(y-y_n))\ 2y\ ] - \\ & -[(x_{m+1}-y_{m+1}(1+k)+ky)(y_{m+1}+y)] - [(x_n+\lambda(y-x_n))] \end{aligned} \qquad (4.19)$$

where

(x_n,y_n), (x_{n+1},y_{n+1}) : coordinates of the left points that confine the water surface

(x_m,y_m), (x_{m+1},y_{m+1}) : coordinates of the right points that confine the water surface

$$k = \frac{x_{m+1} - x_m}{y_{m+1} - y_m} \quad \text{and} \quad \lambda = \frac{x_{n+1} - x_n}{y_{n+1} - y_n}$$

Differentiating equation 4.19 with respect to depth y gives:

$$A_s'(y) = 2(\lambda+k)y + (2x_n - x_{m+1} - y_{m+1} - 2\lambda y_n - 2ky_{m+1}) \qquad (4.20)$$

The Newton-Raphson technique is based on the equation:

$$Y_{i+1} = Y_i - \frac{f(Y_i)}{f'(Y_i)} \qquad (4.21)$$

Applying equations 4.18a and 4.20 to 4.21 give:

$$Y_{i+1} = Y_i - \frac{2(A_s(Y_i) - A_D)}{A'_{s(Y_i)}} \qquad (4.22)$$

The Newton-Raphson method starts with the depth as half the depth between the two left points.

$$Y_1 = \frac{y_m + y_{m+1}}{2} \qquad (4.23)$$

When the desired level of accuracy is reached the area A_s and the flow Q_s are calculated. Also the area A_{s1} and the flow Q_{s1} calculated for a finite difference of depth ($Y_1 = Y_1 + 10^{-10}$)

With these values at each step the new theta is calculated according to the equation

Theta = ratio of kinematic wave speed to (dx/dt)

$$\text{Theta} = \frac{W}{\frac{dx}{dt}} = \frac{Q_1 - Q_{s1}}{A_1 - A_{s1}} \frac{dt}{dx} \qquad (4.24)$$

4.4.7 DEDUCTION OF A IF Q IS KNOWN

The Newton-Raphson technique could not be applied for the deduction of A because the differentiation of an equation similar to equation 4.19 with respect to Depth is not possible. So a less convenient method for the deduction of A was used.

The method used for the deduction of the area is called 'Binary Search'. Its approximation in this method is called 'argument'. With a binary search, the first comparison is made against the 'argument' (in our case depth) in the middle of the two left points that confine the water surface. Then either the top half or the bottom half is searched, depending upon the relationship of the search argument to that mid-point argument. If the search argument is less than the middle argument, the lower half must be checked. Conversely, if the search argument is greater than the middle argument, the upper half becomes the search area.

Then the search argument is compared with the argument of the middle in the selected half to determine their relationship. Depending upon the result of that comparison, again it is split in half and the middle entry of that portion is checked. This halving process is repeated until an argument is found with the desired approximation.Then the same method as before is used for the calculation of theta.

The whole process is shown in Figure 4.9 The steps are as follows:

1. The lower limit, called LO.LIM in the flowchart, is set equal to the elevation of the lower left point that confines the water surface

2. The higher limit, called HI.LIM in the flowchart, is set equal to the elevation of the higher left point that confines the water surface

3. The midpoint is calculated by adding LO.LIM and HI.LIM, and dividing the sum by two. This midpoint is called MID.

4. The flowrate for depth of flow equal to MID is calculated and is compared with the given flowrate Q_D If the search argument is higher than Q_D, the MID value is moved to the LO.LIM value; if lower, MID is moved to HI.LIM. In both cases, control returns to the midpoint calculation, step 3, above.

When the search argument reach the desired approximation then the process is terminated.

Manning's formula was used for the calculation of flow. The reason is that this formulae is the most widely used and best known among engineers and the roughness values used are tabulated in many publications.

The Manning's equation is:

$$Q = \frac{1}{n} \left(\frac{A}{P}\right)^{2/3} A \, S_f^{1/2} \qquad (4.25)$$

where
Q = flow-rate
n = Manning friction factor of the segment
S_f = friction slope
A = cross-sectional area
P = wetted perimeter

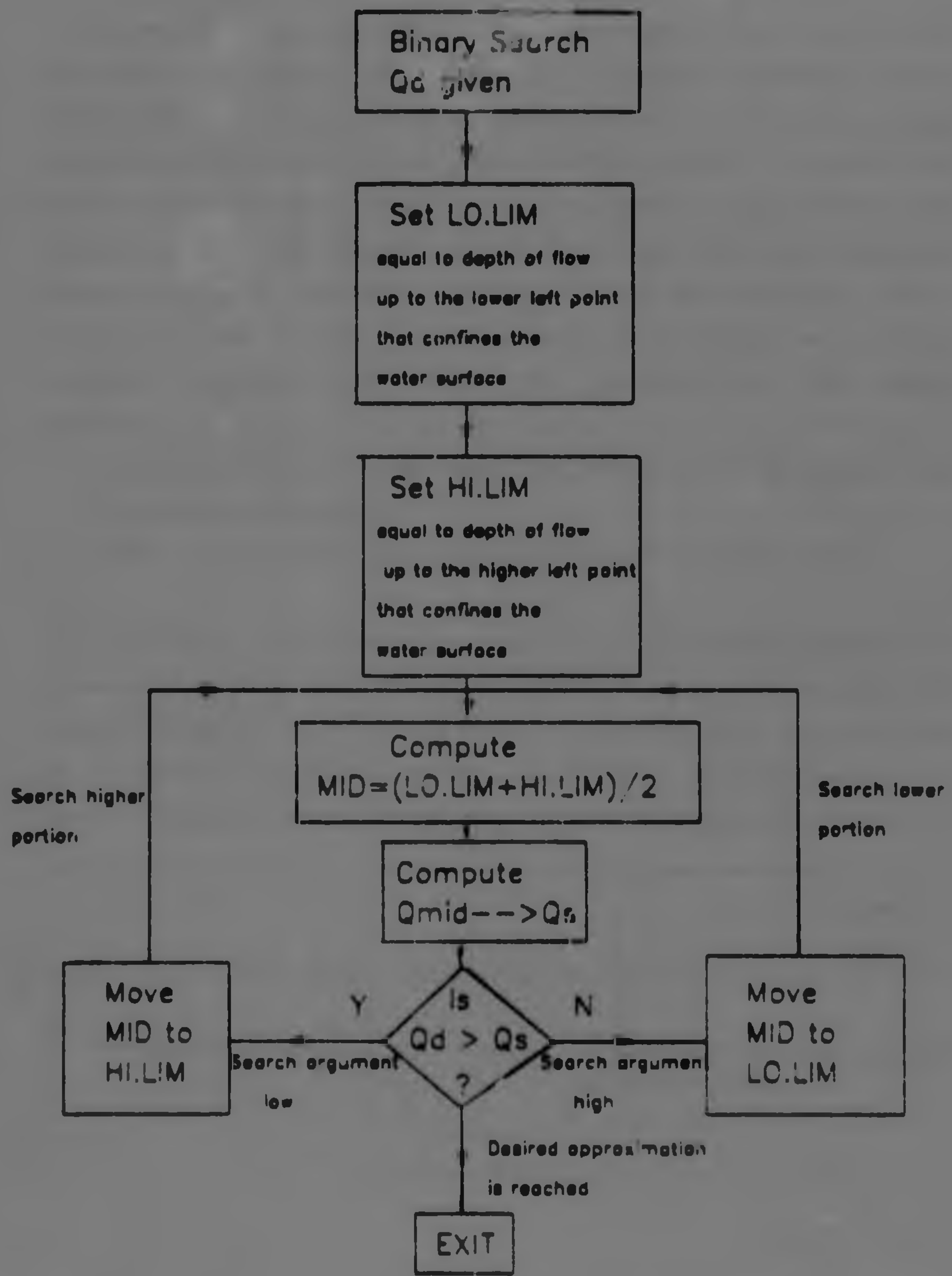

Figure 4.9 Flowchart for the deduction of A if Q is known (Binary search logic)

4.4.8 CALCULATION OF FLOW FOR COMPOUND CHANNELS

Cross-sections may be subdivided into segments to take account of such features as variations of boundary roughness in the transverse direction. In order to minimise the resulting demand on data-handling facilities, the allowable number of segments has been limited to three. Once a cross-section has been divided into segments, certain general assumptions must be made regarding proportioning of the total discharge among the segments. An advantage of the program is the distinction between two types of compound channels with respect to variation of flow depth, namely:

1. Compound channels with small variations of flow depth in the transverse direction.
2. Compound channels with large variations of flow depth

In the first case the flow depths in the minor segments are greater than about half the depth in the main segment at bank full stage (Figure 4.10). It may then be assumed that water surface is horizontal in the transverse direction , although cases have been recordered where deviations from this assumption occur.

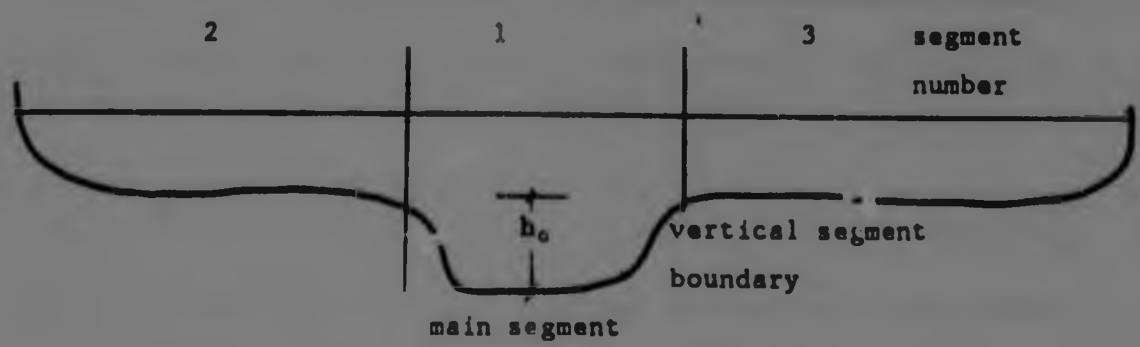

Figure 4.10 Compound channel with small variation of flow depth in transverse direction

A method often employed for compound channels of the type illustrated in Figure 4.10, usually referred to as the separate-channel method, requires the calculation of discharge in each segment separately. The vertical fluid boundaries between the segments are considered not to add to the wetted perimeter; in other words, these boundaries are taken to be planes of zero shear.

Discharge, Q, in each segment is calculated from the Manning equation, viz.

$$Q = \frac{1}{n}\left(\frac{A}{P}\right)^{2/3} A \, S_f^{1/2} \qquad (4.26)$$

The conveyance, $C = \frac{1}{n}\left(\frac{A}{P}\right)^{2/3} A$, can be isolated as a unique function of the segment geometry. The total conveyance, Ct, is the sum of all segment conveyances, viz.

$$C_t = \Sigma C \qquad (4.27)$$

It is assumed that the friction slopes in the longitudinal direction are the same for all segments, then

$$Q_t = \Sigma Q = S_f^{1/2} \, \Sigma C \qquad (3.28)$$

In the second case of flow situation a relatively deep channel is flanked by a flood plain with flow depths less than about half the channel depth at bank full stage, h_s. The steep velocity gradients between segments create separation zones near the segment boundaries, across which there may be appreciable transfer of momentum. The result is that flow in the channel may be retarded whereas flow on the flood plain may in fact be propelled. Moreover, the energy losses in the separation zone may be substantial. Cases have been recordered where bank full discharge was greater before than after overflow on to the flood plain. Clearly the separate channel method must be modified to account for such situations.

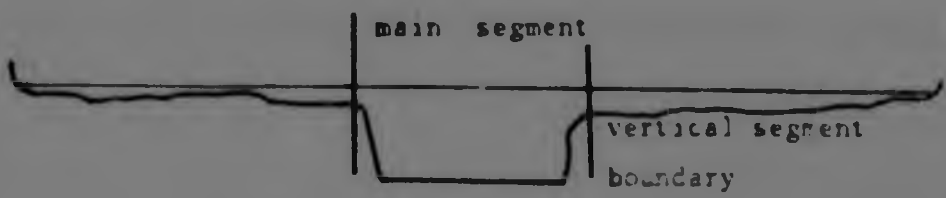

Figure 4.11 Compound channel section with shallow flood plain flow

Posey(1967) has tested a cross-section containing a rectangular channel and a flood plain sloping towards the channel at 1:10. His conclusion was that for depths at the segment boundary of less than 0,30 times the channel depth at bank full stage the separate channel method should be modified. Good agreement between calculated and measured values for the total discharge were achieved if imaginary vertical segment boundaries exerting resistance to the flow in the channel were introduced as elongations of the vertical channel walls up to the water surface. Their roughness was assumed equal to the mean roughness of the channel. The main segment discharge was thereby reduced for a particular depth whereas the flood plain discharge remained unaltered.

The ordinary separate-channel method gave good results for flood plain depths larger than 0,30 times the channel depth at bank full stage. Good results were also achieved for these larger depths by simply treating the entire cross-section as one single channel. The mathematical model described in this Manual is based on the separate-channel principle as discussed in the previous paragraph. Provision is made, however, for introduction of the vertical fluid boundary between segments or parts thereof as an additional solid boundary, as suggested by Posey (1967) in his modified separate-channel method.

4.4.9 CALCULATING OF FLOW FOR PIPES RUNNING PARTLY FULL

If a pipe runs partly full a simple power function can be used to express the relationship between wetted perimeter and sectional area. The fundamental properties of part full pipe flows may be represented in terms of ratios of part full to full pipe conditions for both area and flowrate at various depths.

Plotting this information to show the direct relation between flowrate and area, it is apparent that a linear relationship

$$Q = (Qf/Af)\,A \tag{4.29}$$

may be regarded as a fair approximation, representing empirically observed conditions even better than theoretical conditions based on Manning's equation. If equation 4.5 is considered satisfactory (particularly for depths near mid-diameter), it follows from Manning's equation that

$$a = (\,Qf/Af\,) \tag{4.30}$$

$$= (1/n)S_f^{1/2}\,(D/4)^{2/3}$$

$$m = 1 \tag{4.31}$$

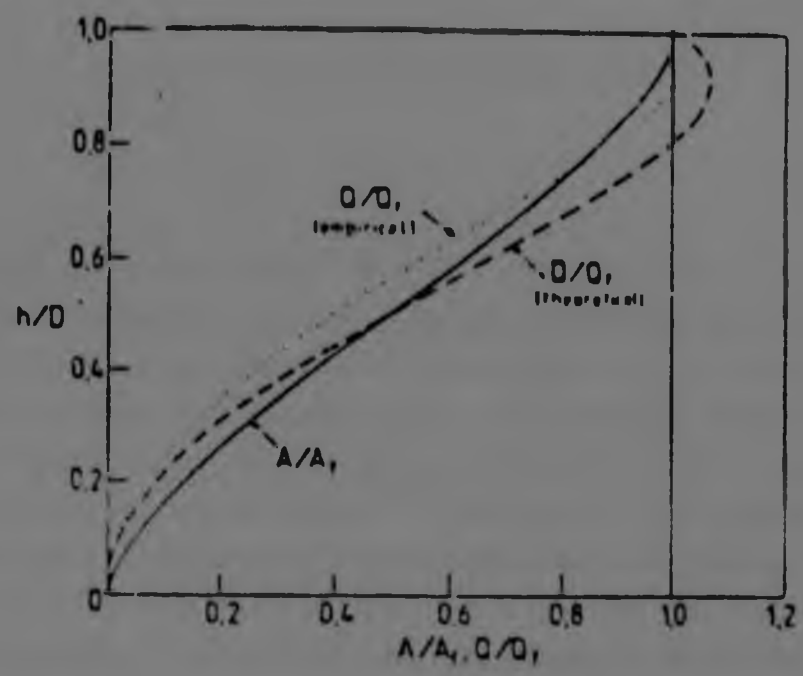

Figure 4.12 Characteristics of Part full Flow in pipes (A.S.C.E. 1960)

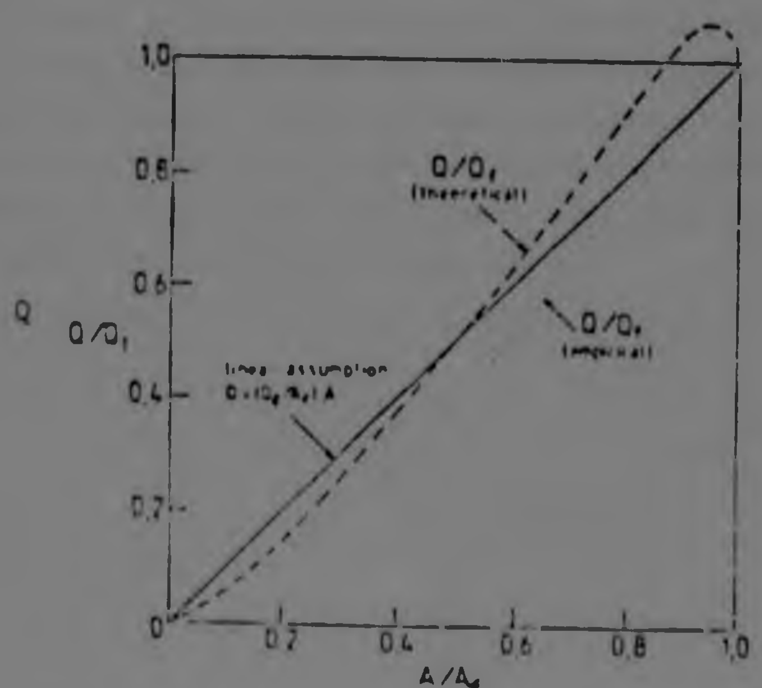

Figure 4.13 Part full flow in pipes, Q/Qf versus A/Af

4.4.10 CALCULATION OF FLOW FOR COMPOSITE CROSS-SECTIONS

A major simplification in WITWAT II was the storage routine. As it was programmed, once the flowrate exceeds the conduit capacity the flowrate is set equal to the conduit capacity and the excess flow is stored at the node. In practical terms, this means that since there is no further storage volume available once a pipe is completely full, a change in discharge at the upstream end is immediately transmitted to the downstream end. However this description is simplistic and does not represent the real behaviour of the system. In reality, when the flowrate exceeds the conduit capacity, the excess flowrate will be routed down streets and the roads.

The traditional urban drainage programs actually considered only the minor system. However WITWAT III has the capability of considering any kind of cross-section, including pipes with compound channels above (Figure 4.14). So when the pipe is running full, the excess water instead of being transmitted to the downstream node, is routed through the above the pipe cross-section. Thus flow in pipes can pass alternately from free surface to pressurised conditions and back again.

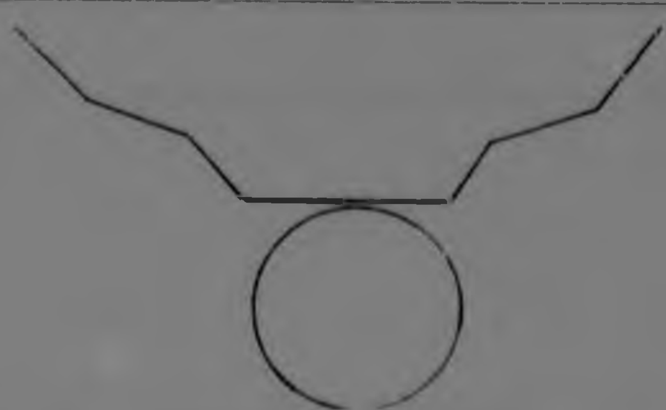

Figure 4.14 Pipe with channel above

4.5 WORKED EXAMPLE

A comparison between WITWAT II and WITWAT III was conducted with the same worked out example as that which appears in the manual of WITWAT II (Green 1985). The specimen catchment is illustrated in Figure 4.15. The discretization was done in such a way as to illustrate various conditions of connectivity. The catchment will be examined only in the analysis mode and the data that was used was exactly the same as that used in WITWAT II.

Subcatchment number	Drains to	Area (ha)	Percent Imperv.	Overland flow length (m)	Slope (m/m)
101	1	1,2	12	140	0,08
102	3	1,0	15	120	0,08
103	2	1,6	5	85	0,03
104	4	0,8	10	120	0,05
105	5	0,3	2	90	0,04
106	5	0,8	30	110	0,04
107	108	0,4	40	40	0,02
108	7	0,4	5	40	0,04

Table 4.4 Topographical features of specimen catchment

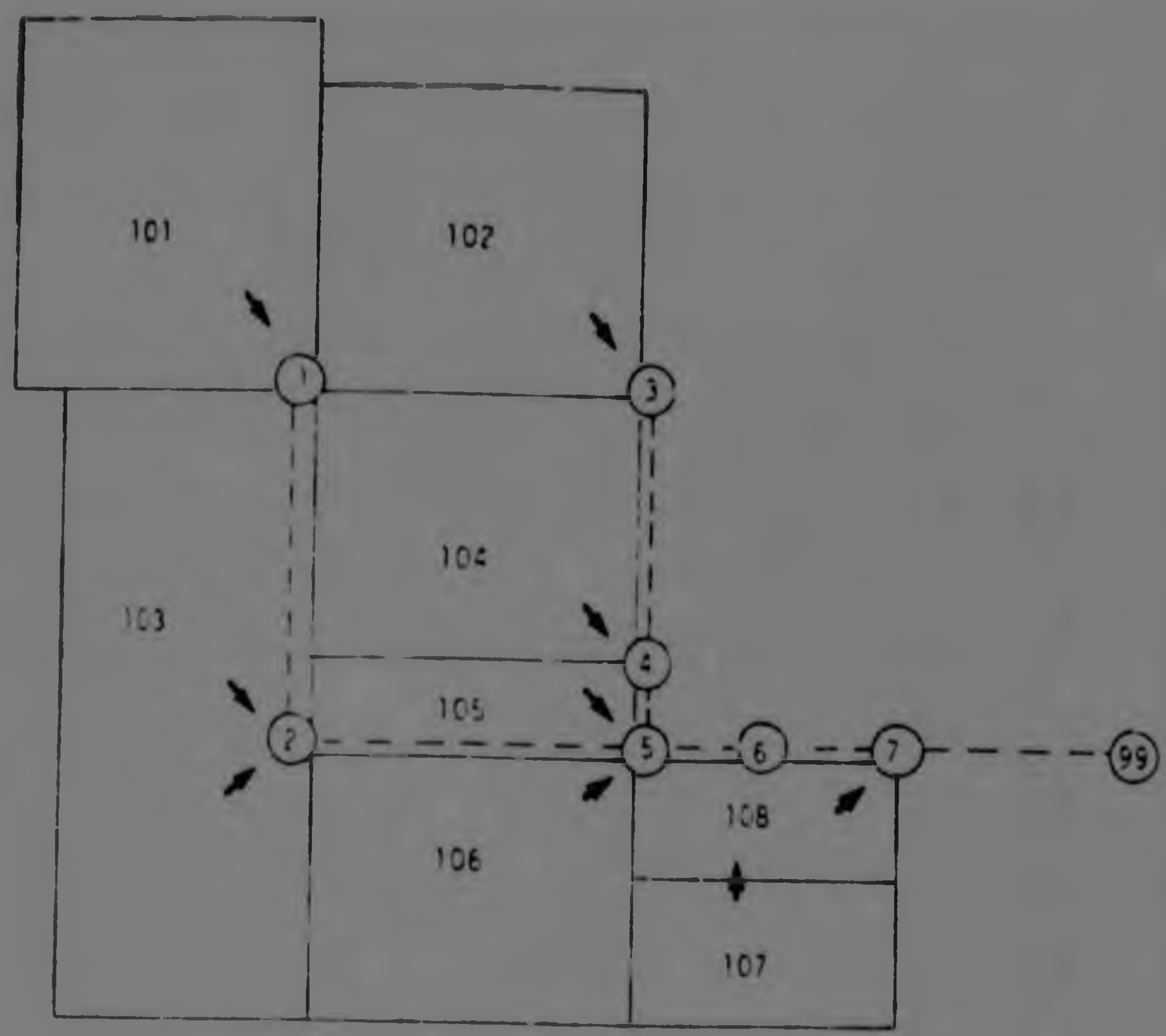

Figure 4.15 Specimen catchment for worked example

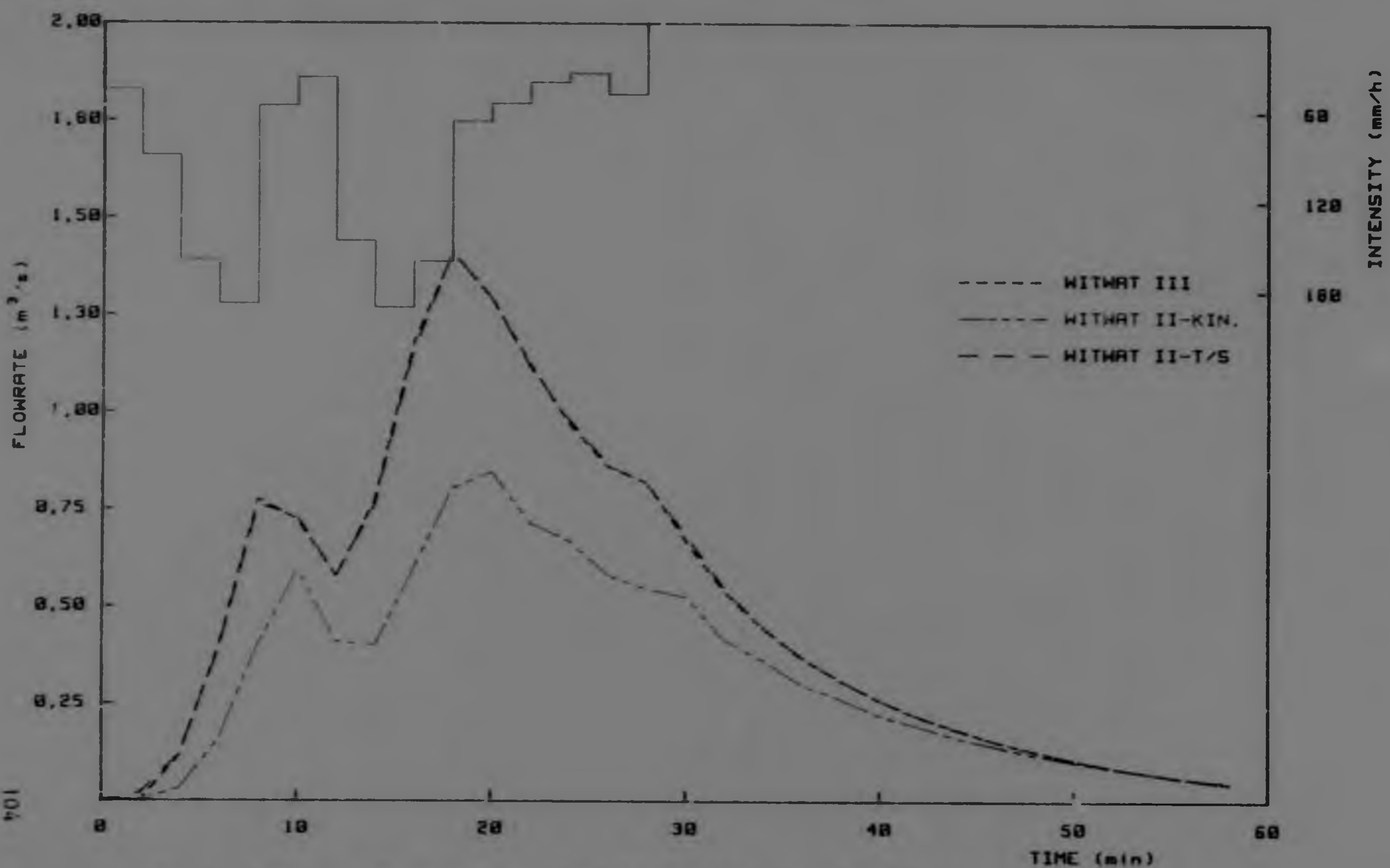

Figure 4.16 Comparison of WITWAT II and WITWAT III (Specimen catchment -Worked example)

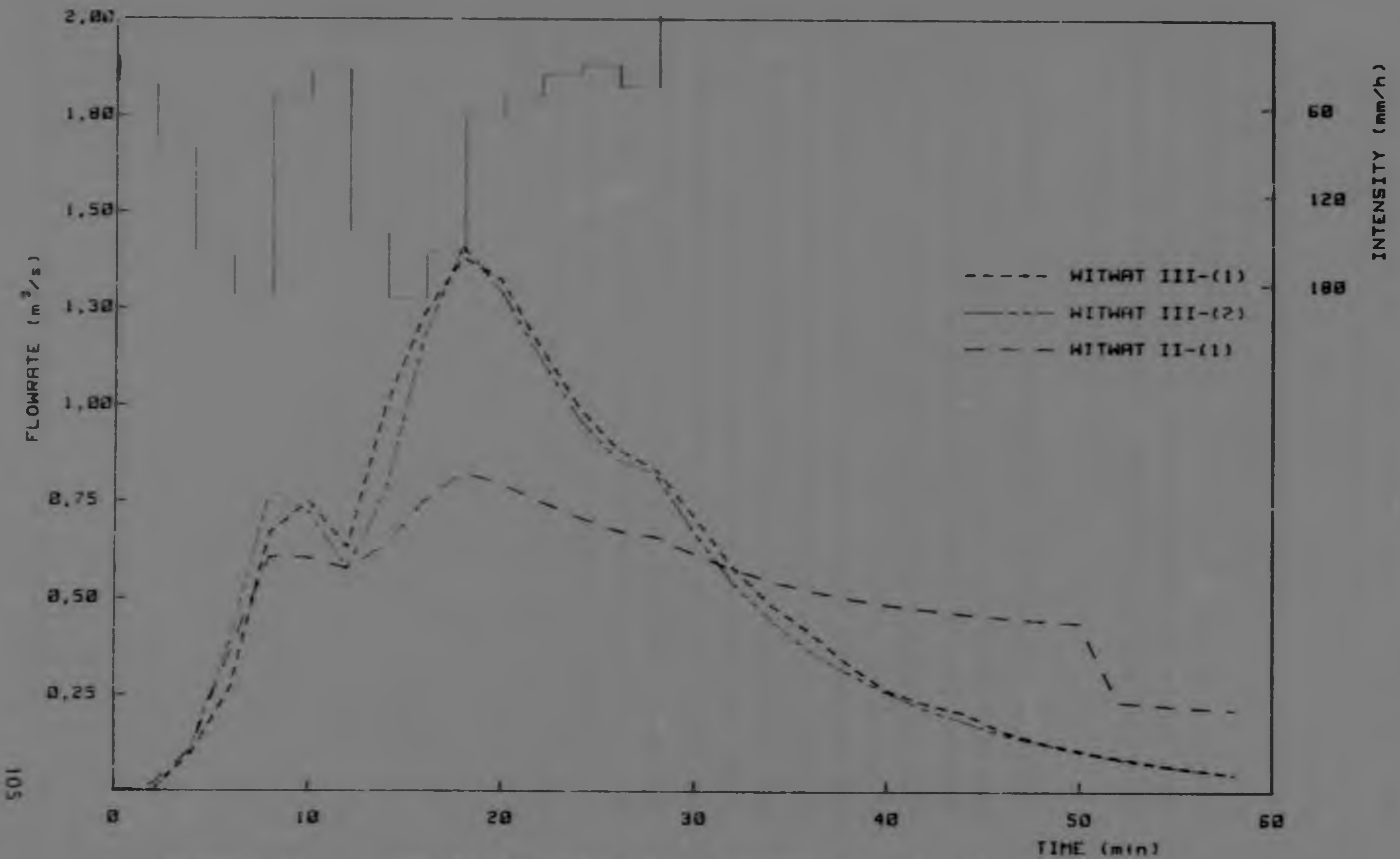

Figure 4.17 Comparison of WITWAT II and WITWAT III
(1) Conduits surcharge (2) Conduits unsurcharge

The pipe sizes during the first set of runs of the models were kept to such dimensions as not to surcharge the conduits. The results showed that WITWAT III gave almost the same flowrates as WITWAT II, when the time-shift method was used. WITWAT II was also tested using the kinematic option for routing through channels. All the flowrates were underestimated by more than 30%. There was a considerable difference in the calculated total volume of runoff. Since the kinematic option of WITWAT II is considered inaccurate all the following comparisons with WITWAT II model made used only the time-shift method.

In the second set of runs the dimensions for all the conduits were undersized so that the conduits surcharged. The diameter of most of the pipes was reduced to 300 mm. WITWAT II in this case does not actually produce any meaningful results, since when the conduits surcharges the excess volume of water is shifted to downstream nodes. In WITWAT III the difference between the two cases , namely that when the conduits were surcharged and when the conduits were unsurcharged was not considerable. This was caused by the fact that the length of the conduits was small. Thus the attenuation of the hydrograph and the fall in the two peaks is not significant since the water is routed through the streets for only a small distance. However as it will be shown in the next chapter, using a real catchment, the effect of the major system on the attenuation of the hydrographs may be significant.

4.6 SENSITIVITY OF THE MODEL FOR DIFFERENT VALUES OF THETA

In order to test the reliability of the results of the program and the effect of different values of theta on the accuracy of the results, a set of runs was conducted for a single catchment and a conduit at the outlet. The conduit was a pipe, which had a diameter of 2 meters in order to avoid surcharge.

The results were compared mainly with SWMM, since WITWAT II uses for routing the time-shift method, where water is actually not routed but transferred downstream. The only reason why the results of WITWAT II are exhibited here (Table 4.5) is to show how illogical they become when the length of the conduits are increased.

In all the cases the total load from the catchments (volume of water) was the same. SWMM model uses the kinematic method for routing and 1 segment is always internally assumed. It is very interesting to notice the effects of the different values of theta on the peak flowrates. Increasing the number of segments of the conduits does not result in a significant reduction on the peak flowrates. Even using 10 segments per conduit instead of one (theta is increased) there is not much effect. However the smaller time-step had a very serious effect on the estimation of the peak flowrates. For a 10 seconds time-step the peak flow rates are almost the same with SWMM. Since SWMM model is one of the most reliable and long used models the results of WITWAT III as a first approximation are considered accurate enough.

Theta has a very serious effect on the accuracy of the results. Even if the computetional time increases considerably the time-step must always be kept as low as possible in order for theta to be maintained lower than unity.

```
 ______
|      | 1000m   1000m   1000m   1000m
|      |=======|=======|=======|=======|
|______|1      2       3       4       99
```

In all the cases TOTAL LOADS = 2950 m3

node no.	PEAK FLOWS AT NODES 1,2,3,4				
	WITWAT III			S W M M	WITWAT II
	time-step=120 sec.		10 sec		
	1 segm.	10 segm.	1 segm.		
1	5,083	5,083	5,083	5,055	5,083
2	3,985	3,540	3,270	3,364	4,270
3	3,357	3,043	2,656	2,567	4,921
4	2,906	2,791	2,208	2,103	4,432
theta	5,817	>10,00	0,485	-------	------

Table 7 Comparison of WITWAT III with SWMM and WITWAT II for different values of theta

5.0 APPLICATION OF MODEL TO THE UPPER BRAAMFONTEIN SPRUIT

5.1 INTRODUCTION

As WITWAT II, WITWAT III is a 'deterministic' model, thus the theoretical structure of the model is based on physical laws. The accuracy and reliability of the model depend mainly to the accuracy and reliability of the model input.

In order to verify the model further, the highly urbanized catchment, Hillbrow, was again used. The reason that this catchment was selected, is that it has already been tested with available models (SWMM, WITWAT II).

As it was shown in chapter 2, out of the 4 recorded rainfall events only the 3 were reliable and thus only these were used. For running with the WITWAT III model the catchment was discretized into the maximum number of subcatchments that the program can accommodate, namely 25.

Both models reproduced favourably the observed hydrographs. What is mainly of concern is not the goodness-of-fit of the two models with the observed hydrographs, but the relevant behaviour of the models since WITWAT II has already been tested in many catchments. In all the simulations exactly the same subcatchment and conduit data for both the models were used. These values can be found in the Appendices, where the output of WITWAT III is reproduced.

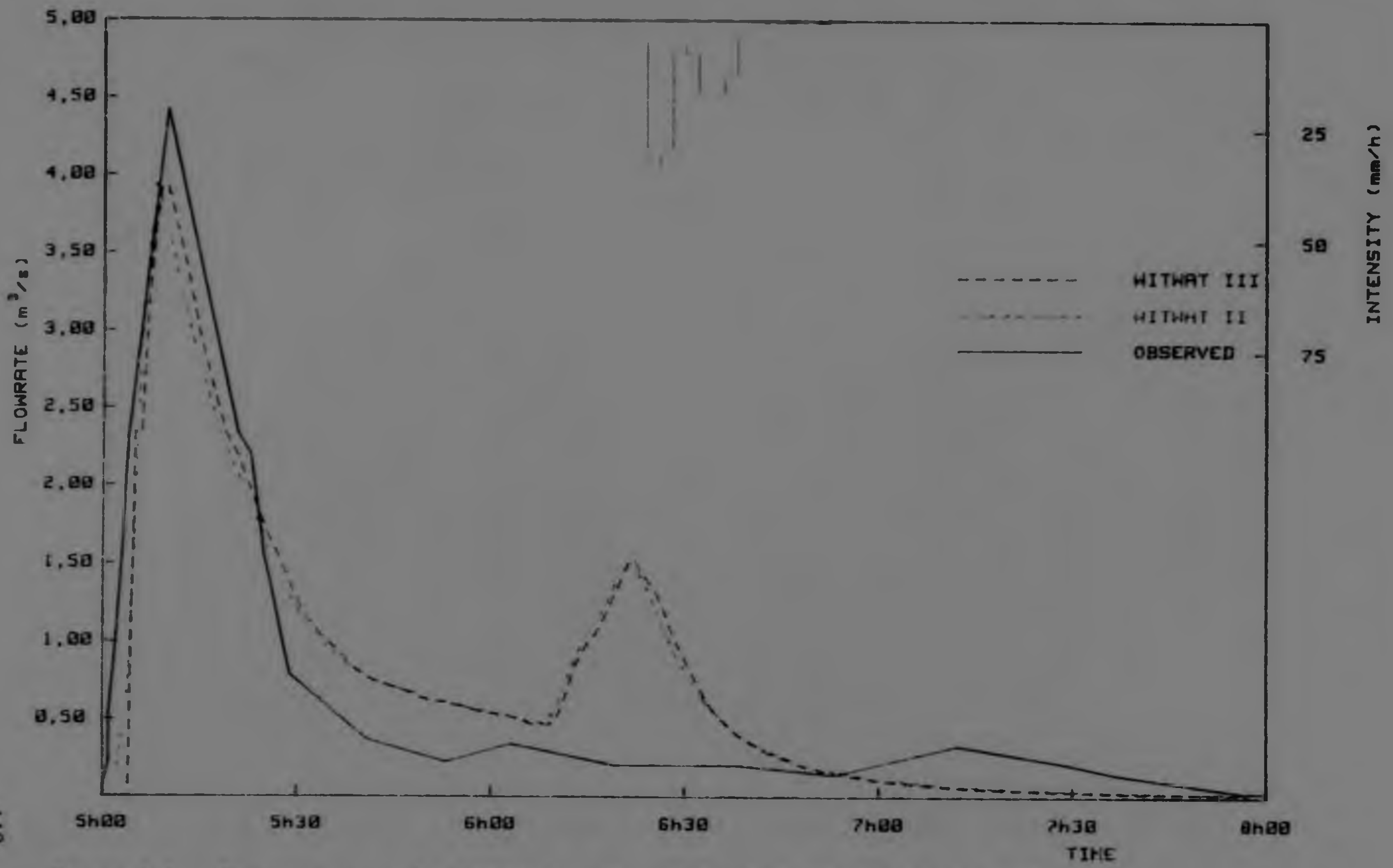

Figure 5.1 Comparison of observed and simulated hydrographs from Hillbrow catchment for storm on 01/01/84

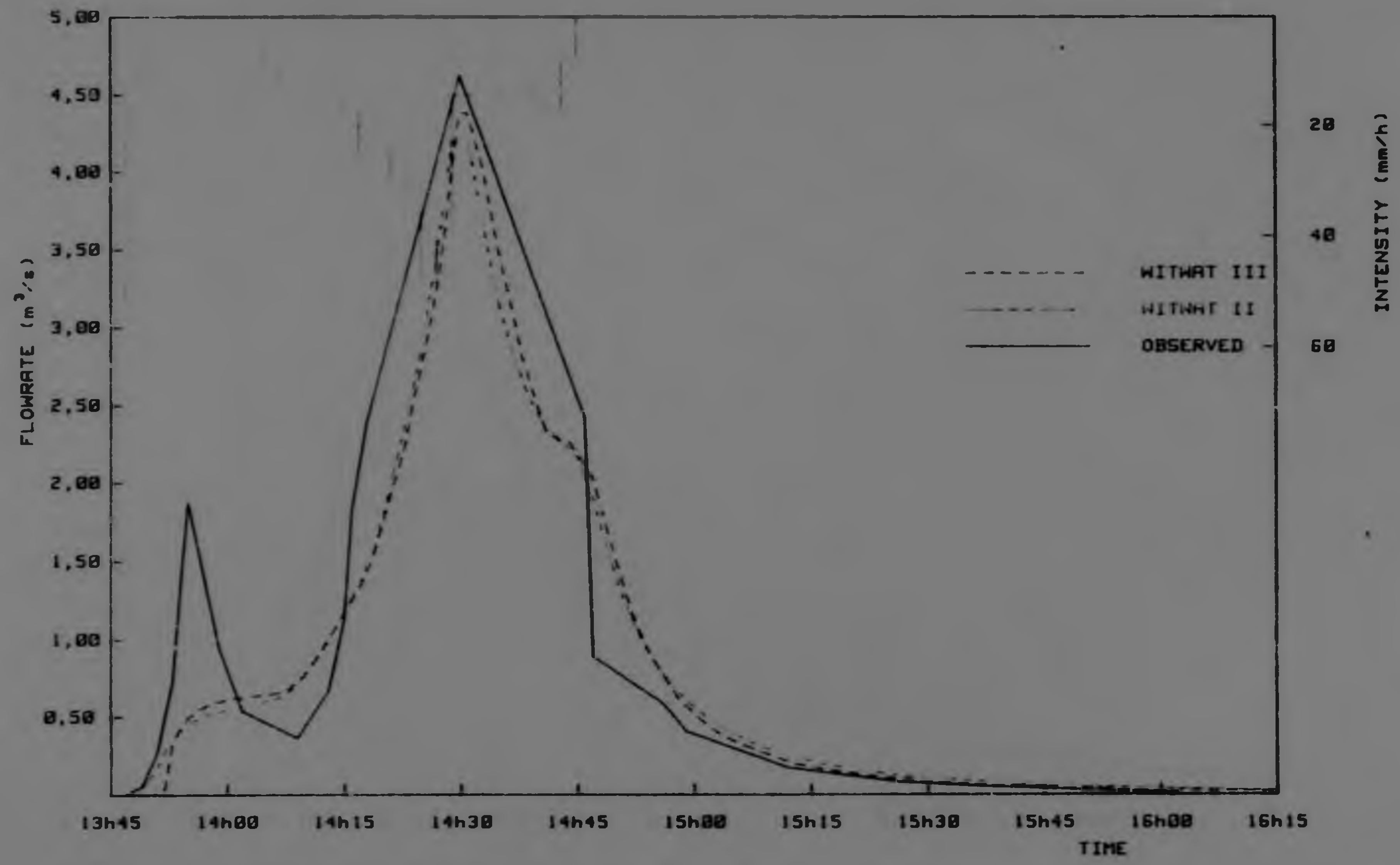

Figure 5.2 Comparison of observed and simulated hydrographs from Hillbrow catchment for storm on 16/12/83

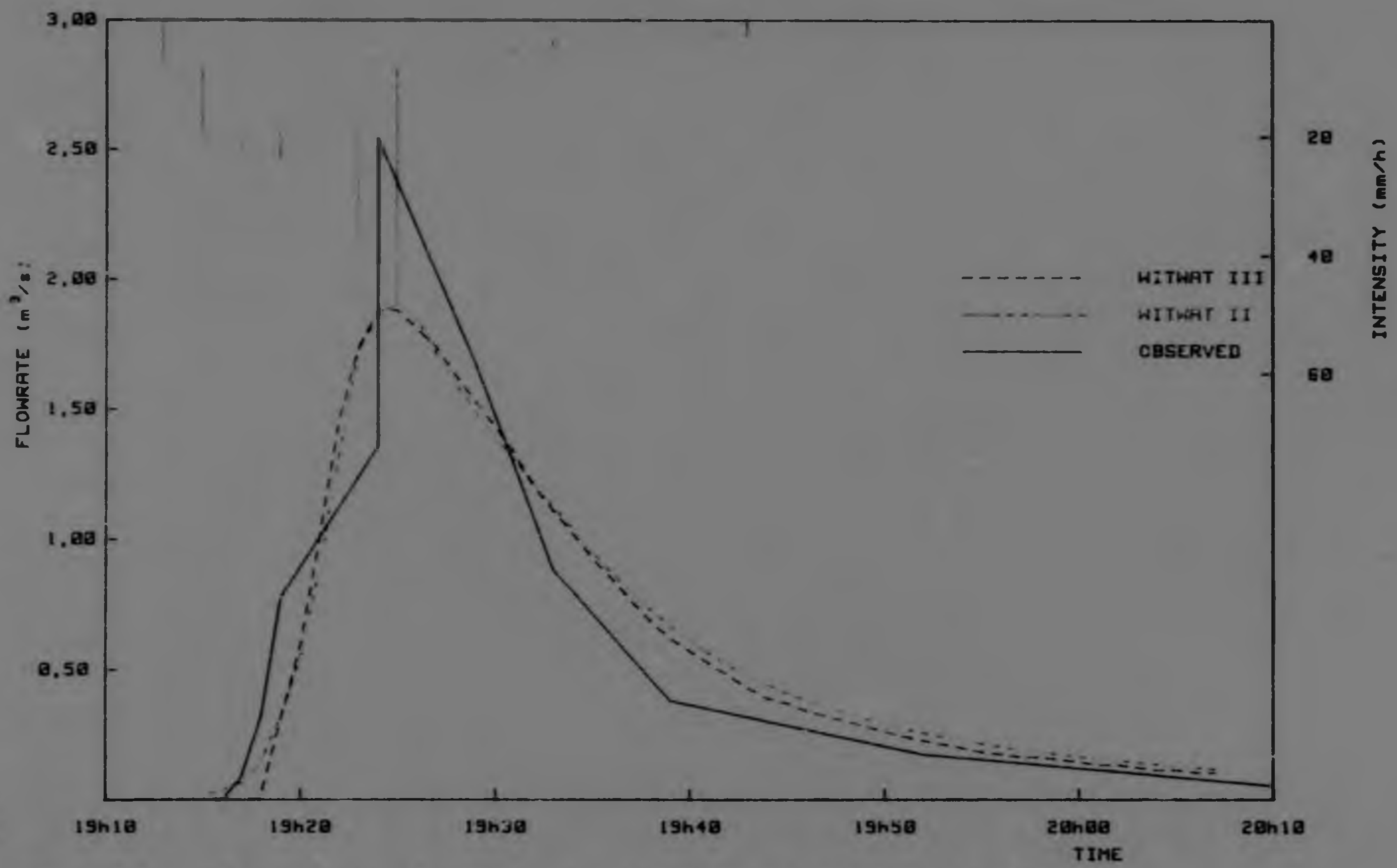

Figure 5.3 Comparison of observed and simulated hydrographs from Hillbrow catchment for storm on 30/12/83

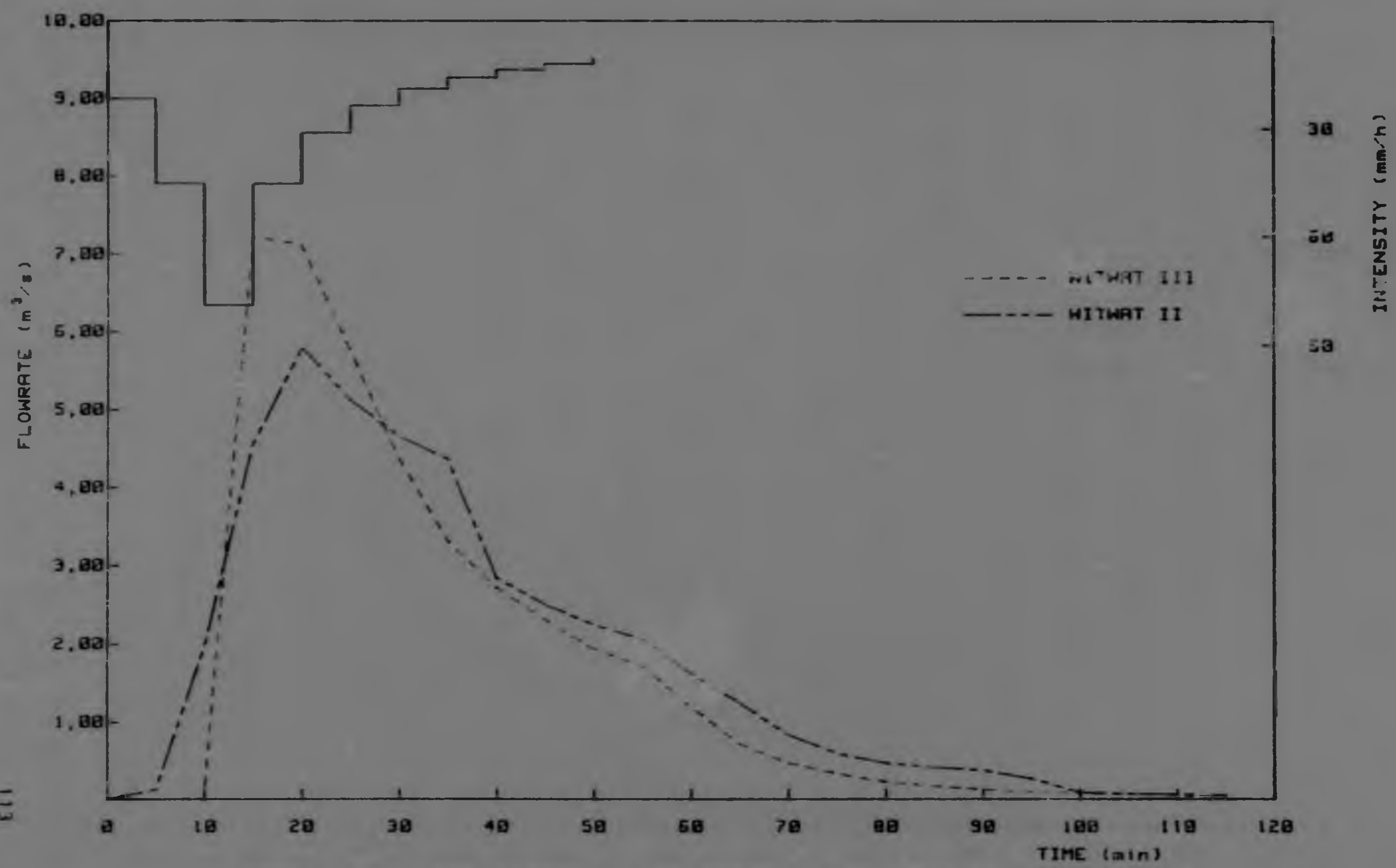

Figure 5.4 Comparison of hydrographs from Hillbrow catchment for 5 years CHICAGO design storm

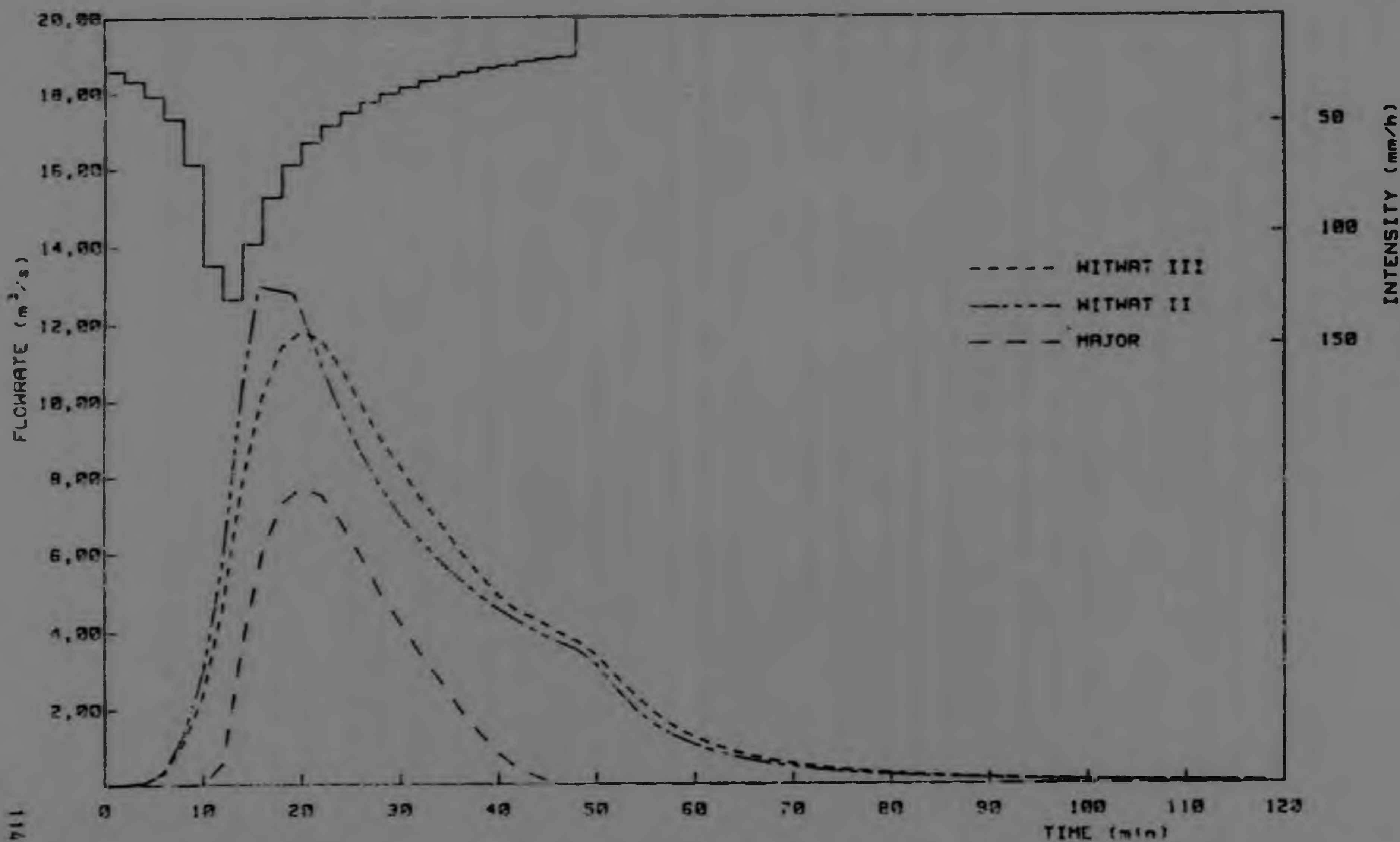

Figure 5.5 Comparison of hydrographs from Hillbrow catchment for 20 years CHICAGO design storm

5.2 RESULTS

The results of the simulation for the three recorded rainfall events are shown in Figures 5.1 to 5.3. In the same graphs are plotted for comparison the results of the simulation of the area with WITWAT II. The two hydrographs are almost identical. The peak is only higher for the storm on 01/01/84.

As mentioned before the catchment responds very fast. The reason for this is the steep slope of the area and the extended and overdimensioned drainage network. The channel at the outlet is very big and does not surcharge in any of the three rainfall events. Only two upstream conduits in the network surcharge but it is of very little significance. Generally most of the pipes are running partly full and for this magnitude of storm the major system is not used. However as the rainfall intensities increase the computed hydrographs of the two models diverge.

In Figure 5.4 a synthetic Chicago design storm with five years recurrence interval was used resulting in most of the upstream conduits surcharging. The outlet channel, because of its large dimensions (rectangular channel with dimensions 2m x 1,5m) does not surcharge. However the computed hydrograph of WITWAT II has an irregular shape and the peak is underestimated by approximately 30%. This is because of the surcharge routine. As mentioned before, the excess water in the conduits that surcharge, builds up in the upstream nodes. The excess water is released only when the downstream pipe begins to run partly full. For this reason after the first peak the hydrograph has an irregular shape because of the volume of water that is successively released from each node. The real runoff is better represented by the hydrograph of WITWAT III, where the excess water is not retained by the upstream nodes but is routed through the streets. In order to compare the performance of the two models the capacity of the conduits in WITWAT II was increased until no surcharge occured. This procedure has been justified by Marsalek (1979). However since the dimensions of the whole drainage system were doubled the catchment responded very fast (time of concentration less than 15min).

Figure 5.5 gives even a better idea of the difference in the representation of the runoff of the two models. Now the event is very severe and the synthetic Chicago storm that is used has a recurrence interval of 20 years. The computed hydrograph of WITWAT II normally indicates that the conduit at the outlet is surcharged. In order to illustrate the difference even better the channel at the outlet in WITWAT III model is replaced by a pipe of one meter diameter. In this way the model can analyse the behaviour of the total and the major systems. In order for WITWAT II to print a more realistic hydrograph it is necessary that the conduit at the outlet be overdimensioned. Figure 5.5 shows the difference between the major the total hydrograph. Now the amount of excess water running in the streets is severe and the flood plains at the outlet of the catchment are surcharged. The difference in the performance of the two models is now noticeable. Peak flow is retarded more than 10% and the whole hydrograph is attenuated.

5.3 DISCUSSION OF RESULTS

The study of the Hillbrow catchment showed that for the specified rainfall events, the two models give essentially the same results. Even though WITWAT II has major simplifications compared with WITWAT III in this case the computed hydrographs were almost identical. The reason for this is that most of the conduits in the network of the Hillbrow catchment run partly full, so there is no surcharge and thus the routine was not used. So in cases where the conduits are capable of carrying the runoff the use of

WITWAT II is thoroughly justified since it is much faster than WITWAT III. (depending on the number of the conduits and if they are surcharged it can be more than 10 times faster).

WITWAT III should be used mainly in cases where the conduits in the network are surcharged and the excess runoff runs down the streets. In this case, as was proved using design rainfall patterns, the runoff is retarded and the two models have differences in the time to peak and the peak flow rates.

It must be remembered that the major system will exist in a community whether or not it has been planned or designed and whether or not developement has been wisely situated with respect to it. In other words, in the case of very serious rainfall events, where the drainage system of the area is not designed for such recurrence interval the 'dual' system will come into effect. Sometimes this is not undesirable since, in this way, the flow is retarded and at the outlet of the catchment the flood is avoided. In any case the identification of the flood-prone areas is very important for the restriction of developement.

Under surcharging conditions within the catchment WITWAT II model will print a hydrograph which is not representative of the real runnoff because the routing through the roads and the streets of the catchment will be totally ignored. On the contrary the excess runoff will be retarded. However in cases where the length of the conduits is small, this procedure does not have a serious effect on the accuracy of the results.

WITWAT III, as shown in Figure 5.3, simulated the behaviour of the area accurately and presented a better understanding of the response of the basin. In this way, basins can be designed to capture all of the flows up to the intensity of the design frequency (e.g. 1:5 years). But they should also be designed to capture no more than this amount of runoff. When higher intensity is experienced, some of the water will bypass the catchbasins and will flow down the streets.

For all the above-mentioned reasons WITWAT III can be most helpful to Engineers involved in the design of drainage networks and at the analysis stage of the response of the catchment basins. The only deficiency of the model is the longer running time but the analysis of dual systems,the friendly use and most important by running on P.C.'s compensate for the above.

Finally it must be remembered that since the model has been tested only on a few catchments the full abilities of the model and the best values for calibration are not fully known. This can be achieved only through repeated usage and experience by the user. The decision for the use of the model does not rely only on the accuracy of the model but on the accuracy of the input data and on experience in the setting up of data files.

6.0 CONCLUSIONS

From the analysis in the Upper Braamfontein Spruit catchment the following are concluded:

1. The effects of sewer installation, considering initially only a major natural channel, increased flood peak magnitudes more than 50%. This increase is dependent on the impervious development.

2. The increase of the impervious cover from semi-urban conditions up to the existing development of Hillbrow led to an increase of the volume and the peak flowrate by more than 120%.

3. The overall comparison in Hillbrow of urban, semi-urban and pre-urban conditions indicated a dramatic impact of the urban development on the hydrologic regime. The peak-flow rate and the total volume of runoff were increased by a factor of two to three compared with semi-urban conditions.

4. For severe events (Chicago-20 years recurrence interval) the flood plains at the outlet of the catchment are surcharged and excess water runs through the streets. Due to the major system, the reduction of the peak is more than 10% and the whole hydrograph is attenuated.

5. Due to the high percentage of imperviousness, the steep slopes and the extended drainage system, the catchment under the existing conditions has a very short concentration time and it is very sensitive to the changes of storm intensity. The very short concentration time can also account for the fact that the major system does not cause as big a retardation and reduction of the peak flowrate as one might expect. For

a flatter catchment with longer channel lengths for routing, the reduction will be much more evident. All the above-mentioned results change for larger floods and different recurrence intervals.

It was shown that urbanization results in discharges of greater runoff volumes and in shorter times. This inevitably leads to higher peak flowrates. The above effects vary between catchments, the greater difference being caused by the interaction between the effects of the increase in impervious area and the improvements to the surface water drainage system. The response time of a small watershed is partly determined by its drainage density (i.e. the length of drainage path per unit area.). Because of the close correlation between the increase of impervious area and drainage improvements, which take place simultaneously during urbanization, their relative influence is extremely difficult to quantify in small data sets. The dual nature of the urbanized drainage area makes the problem more complex, but even so no drainage analysis or further study of effects of urbanization can be performed if the dual factor is ignored

Only the understanding of the impacts of urbanization on the hydrological regime can lead to the right stormwater management policy. Channel retardation, pervious surfaces, storage. retention or detention facilities are some of the effective means of controlling peak runoff rates. Urbanization in Braamfontein Spruit overloaded the drainage system and a combination of all the above means must be used for the reduction of peak flows. Frequently a single measure is not enough, since the watershed must be examined as a whole. Every possible storage space, from the top of the catchment to the outlet must be utilised. Series of small dams and weirs in combination with other flood attenuation measures taken upstream can give a considerable reduction in peak flows. Flood storage schemes are particularly appropriate where flooding is already critical, and in any case substantial storage has to be provided to accommodate the effects of urbanization. Dual purpose schemes are frequently found to result in substantial economies.

There are many stormwater drainage systems, like Braamfontein Spruit, designed for a certain runoff and level of development, both of which have been exceeded in the course of time. In these watersheds stormwater management can only be applied effectively if the effects of urbanization are fully understood. The recognition of dual drainage systems helps to a better understanding of the response of urbanized watersheds. Township layouts must be designed in such a way as not to interfere with the major drainage system. This policy has also led to the restricting of developement in flood-prone areas.

Minor drainage systems should be designed for shorter design frequencies (e.g. 1:2 years) and in addition should also be designed to discharge no more than this amount of runoff. The keystone to good watershed drainage is the appropriate design of the major system, since it should accommodate the runoff from even the least frequent storm such as once in a hundred years. It must be understood that dual concept is by no means only an alternative management policy but it also reflects the nature of every stormwater drainage system. The major and the minor system exist in a community irrespective of whether they have been planned or designed for.

For the above-mentioned reasons stormwater programs should be capable of analysing dual drainage systems. Otherwise the importance of canalization as a factor of urbanization cannot be studied accurately and the response of the watershed cannot be analysed. The results of the application of WITWAT III in Hillbrow showed that even if the storm event was very serious and the conduits overdimensioned the reduction in peak was pronounced.

In improving the WITWAT III modelling accuracy, the calibration of dual drainage system response would be a reasonable first step. The ideal situation, of course, would be the calibration against reliable field data. This data is, however, seldom available Research into the behaviour of urbanizing catchment areas has been impeded by the lack of sufficient good quality, long term data to yield meaningful statistical relationships. However this study can help to a better assessment of the changes of runoff from urban catchments.

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APPENDIX B. WITWAT VERSION III USER'S MANUAL

B.1 GENERAL

This user's manual deals with the operation of WITWAT III model only on a HP-9816 computer. From the WITWAT II suite of programs the only program that isn't maintained, is the 'MAIN 30' which was used for design mode. Thus the new suite of programs can be used only for analysis mode. All the other functions of WITWAT II were maintained such as the duplication of files and the sensitivity routine.

B.2 DATA INPUT

B.2.1 RUN PARAMETERS

After creating a new data file, the program prompts the user for exactly the same data as WITWAT II, except that:

1. No alternative in the mode of operation is possible (The program is set in the analysis mode).

2. It prompts separately for time-step (in seconds) and for rainfall interval (in minutes). So in order to make theta less than 1 the user has to change only the time-step and thus the rainfall interval is not affected.

3. The program doesn't prompt for routing option for the channels since time-shift method is suppressed.

4. It prompts the user as to whether the major or the total hydrograph should be printed. It also prints a warning that if the DUAL system has to be analysed the hydrograph may be printed only at the outlet of the catchment (last node before 99).

B.2.2 AREA DATA

The required data for catchment areas are exactly the same as in WITWAT II. However, it must be underlined that the number of areas that the program can now accommodate has been reduced from 50 to 25 because of problems with memory.

B.2.3 CONDUIT DATA

The required conduit data have been changed. The number of conduits that the program can accommodate has also been reduced to [illegible] because of the larger amount of data required to describe the cross-sections of the conduits.

[illegible] the program can accommodate four different types of cross-sections. Each type has an identification number as follows:

1. Compound channel = 1

2. Natural channel = 2

3. Pipe with compound channel above = 3

4. Pipe with natural channel above = 4

When data are echoed to the screen for either updating or re-acceptance, the user can change the cross-section from the one type to the other by simply changing the identification number. Having been prompted for the conduit number the data are echoed as shown in Figures B.1 to B.2.

The data are echoed in the order indicated above the string. To edit the data the user need only move the cursor to the relevant positions on the string, overtype and enter.

As shown in Figures B.1 to B.2, in EDIT mode, the data of each conduit are echoed subsequently. First are echoed all the characteristics of the conduit, as upstream node, type of cross-section , number of nodes, etc. On pressing ENTER are echoed the coordinates of the cross-section. The number of the coordinates is specified at the first echo of the conduit in the next field to the type of cross-section. The program can accommodate for each conduit up to 10 points in order to describe the cross-section.

For pipes the description of the above cross-section is compulsory since it has to route the excess water in case the pipe surcharges.

For compound channels the points where the flood plains begin must be specified in order for the program to distinguish the segments of the cross-section. The cross[illegible] plain either on the left hand side or the right hand side of the main channel. In the case of the flood plain on the left hand-side (in EDIT mode), the field where program specifies the beginning of the right flood plain must be set equal to zero and vise-versa.

```
Conduit 14 data as follows:-
----------------------------

Conduit no                                      014
Downstream node                                 015
COMPOUND CHANNEL                             Type=1
The cross-section has                      8 points
Flood plains start from the        3th   6th points
Roughness of the main channel                  .014
```

```
Roughnesses of the flood plains          .018    .018
Number of sections                                  2
Slope                                      0.024 m/m
Length                                          125 m

> Overtype echo, QUIT or ENTER


014  015  1  8  3  6  0.014  0.018  0.018  2  0.024  125
```

```
Conduit  14 cross-section data as follows:-
-------------------------------------------

                      X          Y
 1 coordinate       0.000      2.000
 2 coordinate       0.000      1.000
 3 coordinate       3.000      1.000
 4 coordinate       3.000      0.000
 5 coordinate       4.000      0.000
 6 coordinate       4.000      1.000
 7 coordinate       7.000      1.000
 8 coordinate       7.000      2.000

>Overtype echo,QUIT or ENTER


  0.000   2.000   0.000   1.000   3.000   1.000   3.000   0.000   4.000   0.000
  4.000   1.000   7.000   1.000   7.000   2.000
```

Figure B.1 Specific Conduit data echo (compound channel) in either Update or Browse routine

```
Conduit 16 data as follows:-
----------------------------

Conduit no                                        016
Downstream node                                   099
PIPE                                           Type=3
Above the pipe is a compound channel with    8 points
Diameter of the pipe                          1.000 m
Roughness of the pipe                           0.014
Roughness of the above  main channel             .014
Roughnesses of the above flood plains   .018     .018
Flood plains start from the         3th and  6 points
Number of sections                                  2
Slope                                     0.029 m/m
Length                                          138 m

> Overtype echo, QUIT or ENTER

016  099  3   8   1.000  0.014  0.014  0.018  0.018   2  0.029  138  3   6



Conduit  16 cross-section data as follows:-
-------------------------------------------

                        X            Y
 1 coordinate         0.000        2.000
 2 coordinate         0.000        1.000
 3 coordinate         3.000        1.000
 4 coordinate         3.000        0.000
 5 coordinate         4.000        0.000
 6 coordinate         4.000        1.000
 7 coordinate         7.000        1.000
 8 coordinate         7.000        2.000

>Overtype echo,QUIT or ENTER


   0.000   2.000   0.000   1.000   3.000   1.000   3.000   0.000   4.000   0.000
   4.000   1.000   7.000   1.000   7.000   2.000
```

Figure B.2 Specific conduit data echo (pipe) in either Update or Browse routine

B.3 OUTPUT

The program prints the full data echo at the output, if the option is required. The coordinates of the cross-sections and the points where the flood plains begin are also printed separately. At its time-step the maximum theta and the conduit in which this occurred are printed. If the cross-section is of type 1 or 2, (compound or natural chann[illegible] depth and the velocity are also printed at each time-s[illegible]e required node. If the cross-section is of type 3 or 4 ([illegible] with compound or natural channel above) the depth and the velocity printed are of the cross-section above the pipe. So if the pipe is running partly full the depth and the channel velocity are set equal to zero.

B.4 IMPORTANT ASPECTS TO NOTE IN RUNNING THE PROGRAM

1. The program is fixed to print a message when the value of theta exceeds 1. This is to inform the user that during the specified time-step in a certain conduit of the network the wave speed exceeded the local (computational) gradient. The message is shown in Figure B.3.

```
  FOR ROUTING THROUGH  1  CHANNEL,AT THE  1 SEGMENT
THETA=  7.814[illegible]1 ====>WAVE SPEED [illegible] DX/DT


YOU HAVE TO REDUCE THETA BY DECRESING **DT**
OR USING FEWER SECTIONS IN THE CHANNEL
HOWEVER IF YOU WANT TO CONTINUE RUNNING THE PROGRAM AS IT IS
Type Yes or press  ENTER otherwise NO
```

Figure B.3 Message printed when value of theta exceeds 1

2. It is important for the user of WITWAT III to have in mind that the length used for the catchments in WITWAT II is different from the length used in WITWAT III. The difference is illustrated in Figure B.4.

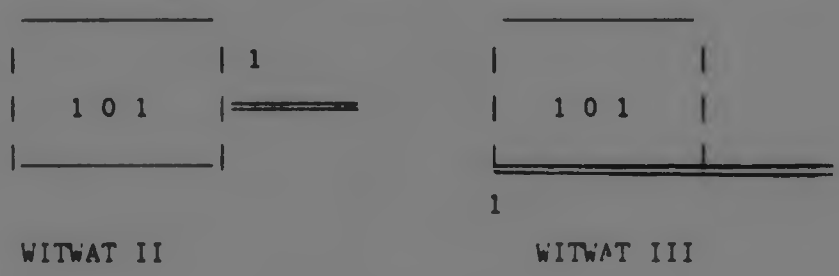

Figure B.4 Length used in the simulation by the two models

In WITWAT II at each time-step all the water is concentrated at the upstream node of the conduit which drains the catchment and then is routed downstream. In WITWAT III, the conduit that is connected to the catchment accepts the water lateral to its full length.

3. As mentioned before the program can print the major hydrograph if the option is requested by the user. However, the major hydrograph can be printed only for the outlet of the catchment (the last node before 99) and not for any other node. A relevant warning is printed on the screen as can be seen in Figure B.5.

```
Hydrographs will be output for the following  1 nodes:-
 16


If the major hydrograph is required the node must be only the
outlet of the catchment(upstream of 99)
```

Figure B.5 Typical display as warning in Brouse routine

4. All the other options as duplications of files, sensitivity analysis etc. ,may be used in exactly the same way as in WITWAT II.

APPENDIX C. EXAMPLE OF OUTPUT OF WITWAT III

W I T W A T   S T O R M W A T E R   D R A I N A G E   P R O G R A M

Version III.1/9016

Developed by: Water Systems Research Programme
University of the Witwatersrand
Johannesburg
South Africa

Data file name: HIL16

EXAMPLE OF OUTPUT OF WITWAT III
HILLBROW CATCHMENT - STORM ON 16/12/83

ANALYSIS option requested with KINEMATIC routing

HORTON infiltration routine with decay constant = 0.00056 /s

User input hyetograph

Ordinates are in mm/h at 2 min intervals

0.00	51.75	11.96	4.93	5.03	5.13
5.13	5.13	5.28	5.43	5.43	8.67
12.91	12.99	14.26	13.89	17.08	25.83
24.33	30.67	31.90	40.32	38.27	14.22
14.22	13.81	13.40	13.07	15.03	16.99
7.50					

Storm duration = 62 min

Rainfall interval= 2 min
Simulation time step = 120 sec, Simulation duration = 110 min

Subcatchment data echo

The data are set out in the following order for each subarea
Subcatchment number, Drains to node, Area of subcatchment (ha)
Percentage imperviousness (%), Overland flow length (m), Overland slope (m
Depression storage - pervious area, Depression storage - impervious area (r
Initial infiltration rate, Final infiltration rate (mm/h)
Roughness - pervious area, Roughness - impervious area (n)

101	103	2.74	90	90	0.032	3	1	66	13	0.250	0.01
102	104	2.43	85	80	0.040	3	1	66	13	0.250	0.01

103	001	2.95	82	100	0.050	3	1	66	13	0.250	0.015
104	001	3.33	90	100	0.038	3	1	66	13	0.250	0.015
105	003	2.04	90	100	0.023	3	1	66	13	0.250	0.015
106	006	3.46	80	200	0.070	3	1	66	13	0.250	0.015
107	006	2.38	40	100	0.080	3	1	66	13	0.250	0.015
109	004	4.64	35	70	0.040	3	1	66	13	0.250	0.015
110	002	2.92	85	100	0.020	3	1	66	13	0.250	0.015
111	009	4.64	90	200	0.070	3	1	66	13	0.015	0.015
112	011	4.30	90	80	0.045	3	1	66	13	0.250	0.015
113	005	2.44	50	60	0.032	3	1	66	13	0.250	0.015
114	115	1.41	60	120	0.040	3	1	66	13	0.250	0.015
115	013	2.36	90	180	0.060	3	1	66	13	0.250	0.015
116	011	.89	90	70	0.045	3	1	66	13	0.250	0.015
117	118	3.85	90	180	0.070	3	1	66	13	0.250	0.015
118	012	2.96	90	160	0.045	3	1	66	13	0.250	0.015
119	014	3.19	80	220	0.050	3	1	66	13	0.250	0.015
120	015	3.96	70	190	0.030	3	1	66	13	0.250	0.015
121	012	2.06	90	145	0.040	3	1	66	13	0.250	0.015
122	121	.80	70	60	0.050	3	1	66	13	0.250	0.015
123	124	1.40	25	30	0.050	3	1	66	13	0.250	0.015
124	125	1.71	35	50	0.050	3	1	66	13	0.250	0.015
125	014	1.88	70	110	0.050	3	1	66	13	0.250	0.015
108	015	2.09	62	100	0.045	3	1	66	13	0.250	0.015

Conduit data echo

The data are set out in the following order for each conduit
   Conduit number, Drains to node, Type of cross-section, where:
   Compound channel =1
   Natural channel =2
   Pipe with compound channel above =3
   Pipe with natural channel above =4
   Diameter & Roughness (n) of pipe or null for channel
   Roughness of cross-section & flood-plains (if they are)
   Number of sections, Slope (m/m), Length (m)

001	002	4	.300	0.012	0.015	0.000	0.000	1	0.027	107

003	002	4	.450	0.012	0.015	0.000	0.000	1	0.010	108
002	004	1	0.000	0.000	0.014	0.015	0.015	1	0.019	183
004	005	1	0.000	0.000	0.014	0.015	0.015	1	0.019	108
006	007	4	.450	0.012	0.015	0.000	0.000	1	0.060	75
007	008	4	.450	0.012	0.015	0.000	0.000	1	0.036	70
008	010	4	.450	0.012	0.015	0.000	0.000	1	0.027	225
009	010	4	.300	0.012	0.015	0.000	0.000	1	0.050	95
010	012	4	.450	0.012	0.015	0.000	0.000	1	0.022	93
005	011	1	0.000	0.000	0.014	0.015	0.015	1	0.027	130
011	012	1	0.000	0.000	0.014	0.015	0.015	1	0.029	105
013	012	4	.450	0.012	0.015	0.000	0.000	1	0.021	95
012	014	1	0.000	0.000	0.014	0.015	0.015	1	0.029	158
014	015	1	0.000	0.000	0.014	0.015	0.015	1	0.024	125
015	016	1	0.000	0.000	0.014	0.015	0.015	1	0.029	138
016	099	1	0.000	0.000	0.014	0.015	0.015	1	0.029	138

Description of the cross-section of each conduit
(for pipes is the above the pipe cross-section)
Data are set in the following order:
Conduit number,number of coordinates
at which nodes flood-plains begin(for natural channel=0)
COORDINATES of the cross-section (X,Y)

```
Conduit 001        004    003    006
-------- ------------------------------
1 Coordinate    0.00     1.00
2 Coordinate    0.00     0.00
3 Coordinate    4.00     0.00
4 Coordinate    4.00     1.00

Conduit 003        004    000    000
---------------------------------------
1 Coordinate    0.00     1.00
2 Coordinate    0.00     0.00
3 Coordinate    4.00     0.00
4 Coordinate    4.00     1.00

Conduit 002        008    003    006
---------------------------------------
1 Coordinate    0.00     1.00
2 Coordinate    0.00      .70
3 Coordinate    3.00      .70
```

```
4 Coordinate    3.00      0.00
5 Coordinate    3.70      0.00
6 Coordinate    3.70       .70
7 Coordinate    6.70       .70
8 Coordinate    6.70      1.00

Conduit 004       008   003   006
----------------------------------------
1 Coordinate    0.00      1.00
2 Coordinate    0.00       .70
3 Coordinate    3.00       .70
4 Coordinate    3.00      0.00
5 Coordinate    3.70      0.00
6 Coordinate    3.70       .70
7 Coordinate    6.70       .70
8 Coordinate    6.70      1.00

Conduit 006       004   000   000
----------------------------------------
1 Coordinate    0.00      1.00
2 Coordinate    0.00      0.00
3 Coordinate    4.00      0.00
4 Coordinate    4.00      1.00

Conduit 007       004   000   000
----------------------------------------
1 Coordinate    0.00      1.00
2 Coordinate    0.00      0.00
3 Coordinate    4.00      0.00
4 Coordinate    4.00      1.00

Conduit 008       004   000   000
----------------------------------------
1 Coordinate    0.00      1.00
2 Coordinate    0.00      0.00
3 Coordinate    4.00      0.00
4 Coordinate    4.00      1.00

Conduit 009       004   000   000
----------------------------------------
1 Coordinate    0.00      1.00
2 Coordinate    0.00      0.00
3 Coordinate    4.00      0.00
4 Coordinate    4.00      1.00

Conduit 010       004   000   000
----------------------------------------
1 Coordinate    0.00      1.00
2 Coordinate    0.00      0.00
3 Coordinate    4.00      0.00
4 Coordinate    4.00      1.00

Conduit 005       008   003   006
----------------------------------------
1 Coordinate    0.00      1.00
2 Coordinate    0.00       .70
```

```
3 Coordinate     3.00       .70
4 Coordinate     3.00      0.00
5 Coordinate     3.70      0.00
6 Coordinate     3.70       .70
7 Coordinate     6.70       .70
8 Coordinate     6.70      1.00

Conduit 011        009    003    006
----------------------------------------------
1 Coordinate     0.00      1.00
2 Coordinate     0.00       .70
3 Coordinate     3.00       .70
4 Coordinate     3.00      0.00
5 Coordinate     3.70      0.00
6 Coordinate     3.70       .70
7 Coordinate     6.70       .70
8 Coordinate     6.70      1.00

Conduit 013        004    002    002
----------------------------------------------
1 Coordinate     0.00      1.00
2 Coordinate     0.00      0.00
3 Coordinate     4.00      0.00
4 Coordinate     4.00      1.00

Conduit 012        008    003    006
----------------------------------------------
1 Coordinate     0.00      2.00
2 Coordinate     0.00      1.50
3 Coordinate     3.00      1.50
4 Coordinate     3.00      0.00
5 Coordinate     5.00      0.00
6 Coordinate     5.00      1.50
7 Coordinate     8.00      1.50
8 Coordinate     8.00      2.00

Conduit 014        00[illegible]    003    006
----------------------------------------------
1 Coordinate     0.00      2.00
2 Coordinate     0.00      1.50
3 Coordinate     3.00      1.50
4 Coordinate     3.00      0.00
5 Coordinate     5.00      0.00
6 Coordinate     5.00      1.50
7 Coordinate     8.00      1.50
8 Coordinate     8.00      2.00

Conduit 01[illegible]        00[illegible]    003    0[illegible]
----------------------------------------------
1 Coordinate     0.00      2.00
2 Coordinate     0.00      1.50
3 Coordinate     3.00      1.50
4 Coordinate     3.00      0.00
5 Coordinate     5.00      0.00
6 Coordinate     5.00      1.50
7 Coordinate     8.00      1.50
8 Coordinate     8.00      [illegible]
```

Conduit 016	008	003	006
1 Coordinate	0.00	2.00	
2 Coordinate	0.00	1.50	
3 Coordinate	3.00	1.50	
4 Coordinate	3.00	0.00	
5 Coordinate	5.00	0.00	
6 Coordinate	5.00	1.50	
7 Coordinate	8.00	1.50	
8 Coordinate	8.00	2.00	

Connectivity matrix

Conduit	Contributing areas
103	101
104	102
1	103,104
3	105
6	106,107
4	109
[illegible]	110
9	111
11	112,116
5	113
115	114
13	115
118	117
12	118,121
14	119,125
15	120,108
121	122
124	123
125	124

Conduit	Upstream conduits
[illegible]	1,3
4	[illegible]
5	4
7	6
8	[illegible]
10	8,9
12	10,11,13
13	5
14	12
15	14
16	15
99	16

TIME(min)	MAX. THETA	CONDUIT in which this occured
2.00	7.611	6
4.00	7.611	6
6.00	7.611	6
8.00	7.611	6
10.00	7.611	6
12.00	7.611	6
14.00	7.611	6
16.00	7.611	6
18.00	7.611	6
20.00	7.611	6
22.00	7.611	6
24.00	7.611	6
26.00	7.611	6
28.00	7.611	6
30.00	7.611	6
32.00	7.611	6
34.00	7.611	6
36.00	7.611	6
38.00	7.611	6
40.00	7.611	6
42.00	7.611	6
44.00	7.611	6
46.00	7.611	6
48.00	7.611	6
50.00	7.611	6
52.00	7.611	6
54.00	7.611	6
56.00	7.611	6
58.00	7.611	6
60.00	7.611	6
62.00	7.611	6
64.00	7.611	6
66.00	7.611	6
68.00	7.611	6
70.00	7.611	6
72.00	7.611	6
74.00	7.611	6
76.00	7.611	6
78.00	7.611	6
80.00	7.611	6
82.00	7.611	6
84.00	7.611	6
86.00	7.611	6
88.00	7.611	[illegible]
90.00	7.611	[illegible]
92.00	7.611	6
94.00	7.611	6
96.00	7.611	6
98.00	7.611	6
100.00	7.611	[illegible]
102.00	7.611	6
104.00	7.611	[illegible]
106.00	7.611	6
108.00	7.611	6
110.00	7.611	6

Hydrograph in conduit 16

Time(min)	Flowrate(m3/s)	Depth (m)	Velocity (m/s)
2.00	0.000	0.000	0.000
4.00	0.000	0.000	0.000
6.00	.029	.018	.813
8.00	.237	.064	1.862
10.00	.443	.094	2.364
12.00	.524	.104	2.518
14.00	.572	.110	2.603
16.00	.596	.113	2.644
18.00	.609	.114	2.665
20.00	.634	.117	2.705
22.00	.647	.119	2.725
24.00	.685	.123	2.785
26.00	.792	.135	2.939
28.00	.933	.149	3.121
30.00	1.111	.167	3.328
32.00	1.268	.182	3.490
34.00	1.465	.199	3.676
36.00	1.776	.226	3.936
38.00	2.142	.255	4.203
40.00	2.569	.287	4.474
42.00	2.976	.316	4.782
44.00	3.792	.372	5.096
46.00	4.380	.410	5.340
[illegible].00	4.288	.404	5.303
50.00	3.748	.369	5.076
52.00	3.227	.334	4.832
54.00	2.811	.305	4.613
56.00	2.490	.281	4.426
58.00	2.333	.270	4.329
60.00	2.218	.261	4.254
62.00	2.105	.252	4.177
64.00	1.811	.224	3.963
66.00	1.465	.199	3.676
68.00	1.173	.173	3.394
70.00	.933	.149	3.121
72.00	.751	.130	2.882
74.00	.621	.116	2.685
76.00	.512	.10[illegible]	2.497
78.00	.432	.092	2.34[illegible]
80.00	.368	.083	2.20[illegible]
82.00	.317	.076	2.081
84.00	.274	.070	1.967
86.00	.237	.064	1.862
88.00	.207	.059	1.741
90.00	.182	.054	1.682
92.00	.163	.051	1.609
94.00	.144	.047	1.534
96.00	.129	.044	1.472
98.00	.119	.042	1.424
100.00	.107	.039	1.368
102.00	.097	.037	1.318
104.00	.090	.035	1.276
106.00	.083	.033	1.232
108.00	.075	.031	1.185
110.00	.068	.030	1.143

TOTAL LOADS= 7410.9845m3

APPENDIX D. LISTING OF "WITWAT III"

APPENDIX D. LISTING OF PROGRAM "MAIN32"

```
10 ! RE-STORE "Main32"
20 ! LOAD "WITWAT",1
30 ! WITWAT SUITE .... Version III.2/9816
40 ! ANALYSIS MODE : LAST UPDATED 04/03/86
45 ! MODIFIED BY P.KOI[illegible]VOPOULOS
50 ! WATER SYSTEMS RESEARCH PROGRAMME,DEPT OF CIVIL ENG.
60 ! WITS UNIVERSITY,JOHANNESBURG
70 !
80 ! ASSIGNMENTS
90  !
100     ASSIGN @Crt TO 1
110     ASSIGN @Kbd TO 2
120 !
130     OUTPUT @Crt;FNClear$;
140     OUTPUT @Crt USING 150
150     IMAGE 3/,27X,"SIMULATION PROGRAM [illegible] D"
160     C1=3600*1000.
170     F5=.76
180     SET TIME 0
190     OPTION BASE 1
200     COM W$[6],T1$[80],T2$[80],M$[8],I$[7],U$[12],F$[10],Drive_def$[20],Drive_data$[20]
210     COM INTEGER A0(26),N1(25),[illegible](26),N2(25),C(25),X(10),Minor
220     COM INTEGER N3,N4,N5,E,R1[illegible]1,T4,T5,T6,T9,F,X1,X2,X3,C2,RI
230     COM A1(25),P1(25),L1(25),[illegible]5),Mannperv(25),Mannimp(25),Mannpipe(25)
235     COM S2(25),L2(25),I9(120),[illegible]3,R3,N(25,3),N10(25),Ns(25),F11(25)
240     COM C11(25,2),X11(25,10,2),X12(10),X22(10)
250     COM Deprperv(25),Deprimp(25),Infil_i(25),Infil_f(25),H(25),K4,Diam(25)
260     COM Hydr_user,N_user,Hydr_num,Q_user(120)
270     INTEGER I,J,K,L,M,K1,K2,Flag1,Number,Timestep,R,F10,Y,S,II,III,A14,S33
280     REAL Z1,Z2,Z3,Z4,A3,Ad,A6,A11,A21,A31,Dep1,Dep2,Vels,Qs1,As1,As,Q21,Q11
285     DIM W,W1,W2,W3,Y1,Y2,Y3,Y4,S3,S4,P3,A7,K7,S19
290     DIM Q1(25,2),Q2(25),Q6(25),Q9(25),Depth_prev(25,2),Ti[illegible]_run[illegible]ff(25,2),V1(25)
295     Vol_infil(25),G1(25),A(25,50),Q(25,50),Double(20),J18(25),As2(25,50),Up(20)
300     DIM Q7(25),Z(25),F6(25),F7(25),D9(25),Q3(25),Qplot(365),X15(12,2),X6(12,2)
305     Q13(100),H13(100),Q8(9,300),Qs2(25,50)
310     ALLOCATE Vel(10,300),Depth(10,300),Th(25,200)
320     DIM A$[80]
330     S19=0
340     PRINTER IS X3
350     GOSUB 11780           ! Subroutine "HEADING"
360     IF X1=1 THEN 370
370 ! ==========================================
380 ! PIPE CAPACITIES,LAG TIMES
390 ! AND INITIALISE
400 ! ==========================================
410     FOR J=1 TO N4               ! N4=number of conduits
420       FOR I=1 TO Ns(J)+2
430         Qs2(J,I)=0
440         As2(J,I)=0
450         A(J,I)=0
460         Q(J,I)=0
470         Double(J)=0
```

```
480          J18(25)=0
490        NEXT I
500       IF C(J)=3 OR C(J)=4 THEN
510        Q6(J)=.335282*Diam(J)^2.667*SQR(S2(J))/Mannpipe(J)
520       ELSE
530        Q6(J)=0
540       END IF
550        Q2(J)=0
560        Q3(J)=0
570        Q9(J)=0
580        V1(J)=0
590        Q7(J)=0
600        Z(J)=0
610        F6(J)=0
620        F7(J)=0
630     NEXT J
640     S33=0
650     FOR I=1 TO N3
660         G1(I)=Infil_I(I)
670         Vol_Infil(I)=0
680     NEXT I
690 ! ==========================================
700 !       INTENSITIES
710 ! ==========================================
720      A$=U$
730      IF FNUpc$(A$[1,1])="U" THEN 1210
740      A$=I$
750      IF FNUpc$(I$[1,1])="I" THEN 770
760      IF FNUpc$(I$[1,1])="C" THEN 820
770 ! INLAND PARAMETERS-----
780      A2=(7.5+.034*M1)*R1^.3
790      B2= .4
800      C4 .89
810      GOTO 860
820 ! COASTAL PARAMETERS------
830      A2=(3.4+.023*M1)*R1^.3
840      B2=.2
850      C4=.75
860 ! AREAL REDUCTION--------
870      A7=0
880      FOR I=1 TO N3
890         A7=A7+A1(I)/100
900      NEXT I
910      IF A7>1 THEN 940
920      R2=1
930      GOTO 950
940      R2=(1.04-.08*LOG(A7))*(T/60)^(.02*A7^.28)
950      A$=U$
960      IF FNUpc$(A$[1,1])="C" THEN 990
970      IF FNUpc$(A$[1,1])="R" THEN 1070
980      IF FNUpc$(A$[1,1])="T" THEN 1130
990 ! CHICAGO---------------
1000     FOR K1=R1 TO T STEP R1
1010         K=K1/R1
1020         IF K1<=R3*T THEN I8=A2*((1-C4)*(R3*T-K1)/60/R3+B2)/((R3*T-K1)/60/R3+B2)^(1+C4)
1030         IF K1>R3*T THEN I8=A2*((1-C4)*(K1-R3*T)/60/(1-R3)+B2)/((K1-R3*T)/60/(1-R3)+B2)^(1+C4)
1040         I9(K)=I8*R2/(C1)! M/S
```

```
1050   NEXT K1
1060   GOTO 1210
1070! RECTANGULAR-----------
1080   I8=A2/(B2+T/60)¬C4
1090   FOR K=1 TO T/RI
1100      I9(K)=I8*R2/(C1)! M/S
1110   NEXT K
1120   GOTO 1210
1130 ! TRIANGULAR-----------
1140   I3=2*(A2/(B2+T/60)¬C4)
1150   FOR K1=RI TO T STEP RI
1160      K=K1/RI
1170      IF K1<=R3*T THEN I8=I3*K1/(R3*T)
1180      IF K1>R3*T THEN I8=I3*(T-K1)/((1-R3)*T)
1190      I9(K)=I8*R2/(C1)! M/S
1200   NEXT K1
1210 !
1220 !----------------------------------------------------------------
1230 ! CALC TIMES TO RUNOFF
1240 !----------------------------------------------------------------
1250   FOR I=1 TO N3                            ! for all the subcatchments
1260      Time_runoff(I,1)=0
1270      Time_runoff(I,2)=0
1280      D1=0
1290      D2=0                                  ! T=storm duration
1300      FOR K=1 TO T/(T1/60)                  !Duration of storm/time-step
1310         IF (T1/60)*K/RI<=1 THEN            !(T1/60)*K=Current time
:320            K1=1                            !
1330         ELSE                               !  PERVIOUS AREA
1340            M=(T1/60)*K/RI                  ! a<?<b --> ?=b
1350            K1=M
1360                                            ! Runoff will only begin
1370         END IF                             ! when depth of rain will
1380         D1=D1+I9(K1)*T1                    ! be > than depression storage
1390         IF D1>=Deprperv(I) THEN 1420       ! of the catchment
1400      NEXT K                                !
1410      GOTO 1430
1420      Time_runoff(I,1)=(T1/60)*K-(D1-Deprperv(I))/I9(K1)/60
1430      FOR K=1 TO T/(T1/60)                  !
1440         IF (T1/50)*K/RI<=1 THEN            !
1450            K1=1                            !
1460         ELSE                               !  IMPERVIOUS AREA
1470            M=(T1/60)*K/RI
1480            K1=M
1490          END IF
1500         D2=D2+I9(K1)*T1                    !
1510         IF D2>=Deprimp(I) THEN 1540        !
1520      NEXT K                                !
1530      GOTO 1550                             !
1540      Time_runoff(I,2)=(T1/60)*K-(D2-Deprimp(I))/I9(K1)/60
1550   NEXT I
1560 ! ---------------INITIALISE D4,D5,D9,Q1--------------------
1570   D5=.0001
1580   FOR I=1 TO N3
1590      Q1(I,1)=0
1600      Q1(I,2)=0
1610      Depth_prev(I,1)=.0001
```

```
1620      Depth_prev(I,2)=.0001
1630    NEXT I
1640    FOR J=1 TO N4
1650      D9(J)=.001
1660    NEXT J
1670  ! ==============================================================
1680  !       CONNECTIVITY
1690  ! ==============================================================
1700    II=1                                    !J18(II) = order of Routing
1710    A7=1
1720    L=1
1730    J=L
1740    III=1
1750    FOR L=1 TO N4                           ! look for the channel that is
1760      IF PO(J)=N2(L) THEN 1730              ! upstream of J
1770    NEXT L                                  ! if it find it then L-->J
1780    FOR K=1 TO II                           !
1790     IF J18(K)=PO(J) THEN 1840              !
1800    NEXT K                                  !
1810    J18(II)=PO(J)                           !If not ,this means that upstream is
1820    II=II+1                                 !a catchment,so it routes through J
1830  ! ---------------------------------------START--------------------------
1840    FOR S=1 TO N4                           !                  :   It asks if there
1850      FOR K=1 TO III                        !                  :  is a conduit with
1860        IF Double(K)=S THEN 1970            ! --------       N2:    the same end
1870      NEXT K                                !         ========*7
1880      IF N2(J)=N2(S) AND J<>S THEN          ! --------  J    N2(J)
1890        FOR K=1 TO II
1900          IF J18(K)=N2(S) THEN 1990
1910        NEXT K
1920        Double(III)=J                       !
1930        III=III+1
1940        J=S                                 ! If there is one this becomes J
1950        GOTO 1750                           ! and goes back to follow this brantch
1960      END IF                                ! until it finds again a catchment
1970    NEXT S                                  !
1980  ! ----------------------------------------------------------------------
1990    FOR L=1 TO N4                           ! If there is no upstream channel
2000      IF N2(J)=PO(L) THEN                   ! then it finds the downstream channel
2010        J=L                                 ! it call it J
2020        FOR K=1 TO II
2030          IF J18(K)=PO(J) THEN 2110         !
2040        NEXT K
2050        Up(PO(J))=A7
2060        A7=A7+1
2070        J18(II)=PO(J)                       !
2080        II=II+1                             !
2090        GOTO 1830                           !
2100      END IF                                !
2110      IF N2(J)=99 THEN 2140                 !
2120    NEXT L                                  !
2130  ! ------------------------------
2140  ! ==============================================================
2150  ! MAIN TIME STEP LOOP
2160  ! --------------------------------------------------------------
2170    OUTPUT @Crt;FNClear$;
2180    OUTPUT @Crt USING 2190
```

```
2190   IMAGE 2/,29X,"WITWAT ANALYSIS MODE",/,29X,20("-")
2200   A$=U$
2210   IF FNUpc$(A$[1,1])="U" THEN 2250
2220   IF FNUpc$(A$[1,1])="C" THEN 2280
2230   IF FNUpc$(A$[1,1])="T" THEN 2310
2240   IF FNUpc$(A$[1,1])="R" THEN 2340
2250   OUTPUT @Crt USING 2260
2260   IMAGE 2/,20X,"Simulation with user supplied hyetograph"
2270   GOTO 2360
2280   OUTPUT @Crt USING 2290
2290   IMAGE 2/,16X,"Simulation using Chicago synthetic distribution"
2300   GOTO 2360
2310   OUTPUT @Crt USING 2320
2320   IMAGE 2/,15X,"Simulation using triangular synthetic distribution"
2330   GOTO 2360
2340   OUTPUT @Crt USING 2350
2350   IMAGE 2/,14X,"Simulation using rectangular synthetic distribution"
2360   OUTPUT @Crt USING 2370
2370   IMAGE 2/,14X,"Now computing time step no ",3X," at time = ",6X,"minutes"
2380 ! ==========================1st TIME-LOOP=========================
2390 ! ================================================================
2400 ! MAIN TIME STEP LOOP BASED ON DURATION OF SIMULATION
2410 ! MAIN TIME-STEP =RAINFALL INTERVAL(RI)
2420 ! -----------------------------------------------------------------------
2430   T3=T9/RI !NUMBER OF STEPS=Duration of simul./min. per rainfall interval
2440   FOR K=1 TO T3
2450 ! ========================2nd TIME LOOP===============================
2460 ! ================================================================
2470 ! TIME LOOP FOR WHICH RAINFALL IS STEADY
2480 ! TIME-STEP=T1(sec)
2490 ! ------------------------------------------------------------------
2500     FOR T7=T1 TO RI*60 STEP T1
2510       OUTPUT @Crt;FNClear$;
2520       OUTPUT @Crt USING 2530
2530       IMAGE 9/,14X,"Now computing time step no ",3X," at time = ",6X," minutes"
2540       IF K=1 THEN                   ! K=numb. of rainfall interval
2550         K1=T7                       !   that is running that moment(e.g.7th)
2560       ELSE                          ! K1=current time
2570         K1=T7+RI*60*(K-1)
2580       END IF
2590       K2=K1/T1
2600       CONTROL 1;42,10
2610       OUTPUT 1 USING 2620;K2
2620       IMAGE 3D
2630       K7=K1/60
2640       CONTROL 1;55,10
2650       OUTPUT 1 USING 2660;K7
2660       IMAGE 3D.2D
2670 ! ===================1st SPATIAL LOOP===============================
2680 ! FIRST SPATIAL LOOP CONSIDERS EACH SUBCATCMENT IN TURN
2690 ! RUNOFF FROM SUBCATCHMENTS-FINITE DIFFERENCE SCEME-NEWTON RAPHSON SOLUTION
2700 ! ================================================================
2710       FOR I=1 TO N3                 ! N3=number of subcatchments
2720         F9=0
2730         Q_cascade=0
2740         FOR L=1 TO N3
2750           IF A0(I)=N1(L) THEN Q_cascade=Q_cascade+Q1(L,1)
```

```
2760            NEXT L
2770 ! PERVIOUS AREA
2780            A7=SQR(S1(I))/(L1(I)*Mannperv(I))
2790            Area_temp=A1(I)*10000
2800            T2=Time_runoff(I,1)
2810            D4=Depth_prev(I,1)
2820            GOSUB 11560! RETURNS INFILTRATION PARAMETERS
2830            GOSUB 6140! RETURNS D6
2840            Depth_prev(I,1)=D6
2850            Q1(I,2)=A7*A1(I)*(1-P1(I)/100)*10000*D6^(5/3)
2860 ! IMPERVIOUS AREA
2870            A7=SQR(S1(I))/(L1(I)*Mannimp(I))
2880            Area_temp=A1(I)*10000
2890            T2=Time_runoff(I,2)
2900            F3=0
2910            F4=0
2920            D4=Depth_prev(I,2)
2930            GOSUB 6140! RETURNS D6
2940            Depth_prev(I,2)=D6
2950            Q1(I,2)=Q1(I,2)+A7*A1(I)*(P1(I)/100)*10000*D6^(5/3)
2960         NEXT I
2970 !-----------------RESET Q1()'s-------------------------------------------
2980         FOR I=1 TO N3           !N3=number of subcatchments
2990            Q1(I,1)=Q1(I,2)
3000 !-----------H,GRAPH AT THIS OUTLET?-----------------------------------
3010            FOR P=1 TO N5  !number of nodes where hydrograph will be plotted
3020               IF X(P)=A0(I) THEN Q8(P,K2)=Q1(I,2)
3030            NEXT P
3040         NEXT I
3050 !==================1st spatial loop is finished=======================
3060 ! RESET Q2(J,K2) ARRAY, INITIALISE Q2(),Q4() AND Q9()
3070         FOR J=1 TO N4        ! N4=number of conduits
3080            Th(J,K2)=0
3090            Q2(J)=0
3100            F7(J)=0
3110            IF Hydr_user=1 THEN              ! If a hydrograph is
3120               IF N_user=P0(J) THEN          ! supplied
3130                  Q2(J)=Q_user(K)            !
3140                  Q3(J)=Q_user(K)            ! Q3(J,K2)=user supplied hydrogr.
3150               END IF                        !
3160            END IF                           !
3170         NEXT J
3180 ! ===================================================================
3190 !       CONNECTIVITY
3200 ! ===================================================================
3210 ! FLOW INTO NODES DIRECTLY OFF SUBCATCHMENTS
3220 ! -------------------------------------------------------------------
3230         FOR I=1 TO N3                      ! Q1(I,2):flow from I subcatchment
3240            FOR J=1 TO N4                   ! Q2(J,K2)
 J conduit at K2 time
3250               IF N1(I)<>P0(J) THEN 3300 !      ___________
3260               S19=S19+Q1(I,2)            !     |           |
3270               Q2(J)=Q1(I,2)/L2(J)+Q2(J)  !     |      N1(I)|? P0(J)
3280               Q3(J)=Q2(J)                !     |           *  *=======
3290                                          !     |___________|
3300            NEXT J                        !
3310         NEXT I                           !
```

```
3320 ! -------------------------------------------------------------------------
3330 ! ====================2nd SPATIAL LOOP=============================
3340 ! ===============================================================
3350 ! SECOND SPATIAL LOGP CONSIDERS EACH PIPE OR CHANNEL IN TURN
3360 ! ===============================================================
3370        A14=1
3380        FOR J1=1 TO N4          ! for all the channels
3390           J=J18(J1)            !  J=upstream node of the conduit
3400 !
3410 !    Up(J)=0  ---->  upstream of J conduit  are ONLY  catcments
3420 !    Up(J)<>0 ---->  upstream of J conduit are conduits and perhaps catchments
3430 !
3440          FOR I=1 TO N4
3450           IF J=PO(I) THEN
3460            J=I
3470            GOTO 3550
3520           END IF
3530          NEXT I
3540                                 !
3550           IF Up(J18(J1))=0 THEN
3560            Q11=Q(J,1)
3570            Q(J,1)=0
3580           END IF
3590           A11=A(J,1)            !            _______________________
3600           A21=A(J,2)            !           |                       |
3610                                 !           |                       |
3620                                 !           |       Catchment       |
3630           Q21=Q(J,2)            !    A(J,1)|                       |
3640           Qd=Q(J,1)             !    Q(J,1)|===|===|===|===|====|=======
3650           F10=0                 !              Conduit with Up(J)=0
3660           I* Qd=0 THEN
3670             Ad=0
3680             GOTO 3720
3690           END IF
3700             GOSUB 6590    ! Qd-->Ad
3710           Q(J,1)=Qd
3720           A(J,1)=Ad             !
3730           X7=L2(J)/Ns(J)        ! X7=length of segment
3740           Y8=Ns(J)+1            !
3750 ! ====================3rd SPATIAL LOOP=============================
3760 ! THIRD SPATIAL LOOP CONSIDERS EACH SEGMENT OF CHANNEL IN TURN
3770 ! ===============================================================
3780           FOR Y=2 TO Y8      !for all the segments of the channel
3790 ! ---------------------------------------------------------------
3800 ! THETA=RATIO OF KINEMATIC WAVE SPEED TO (DX/DT)
3810 ! ---------------------------------------------------------------
3820              IF C(J)=3 OR C(J)=4 THEN                       !
3830                IF A(J,Y)<A6 OR Q(J,Y)<Q6(J) THEN
3840                  Th1=(T1/X7)*((S2(J)^.5/Mannpipe(J))/(4/Diam(J))^(2/3))
3850                  GOTO 3930
3860                END IF                                      !
3870              END IF
3880              IF As2(J,Y)=0 AND A(J,Y)=0 THEN
3890                Th1=0
3900                GOTO 3930
3910              *ND IF
3920              Th1=(Qs2(J,Y)-Q(J,Y))/(As2(J,Y)-A(J,Y))*(T1/X7)
```

```
3930              IF Th1>Th(J,K2) THEN
3940                 Th(J,K2)=Th1
3950              END IF
3960  !                                                  !  A11       :x-dx,t
3970                                                     !  Q(J,Y)    :x    ,t+dt
3980              IF Th1<=1 THEN 4161                    !  Q(J,Y-1):x-dx,t+dt
3990              IF Th1>1 AND S33<>1 THEN 5300          !  A(J,Y-1):x-dx,t+dt
4000              Q(J,Y)=Q(J,Y-1)+Q2(J)*X7-(X7/T1)*(A(J,Y-1)-A11)!
4010              IF Q(J,Y)<.000000001 THEN Q(J,Y)=0
4020              Qd=Q(J,Y)                              !
4030              F10=0                                  !===|=============|=====
4040              IF Qd=0 THEN                           !  x-dx            x
4050               Qs1=0
4060               As1=0
4070               Ad=0
4080               GOTO 4120
4090              END IF
4100                GOSUB 6590      ! Q(J,Y)-->A(J,Y)
4110  ! ----------------------------------------------------------- ----------
4120              Qs2(J,Y)=Qs1
4130              As2(J,Y)=As1
4140              Q(J,Y)=Qd
4150              A(J,Y)=Ad                               ! Q11:x-dx,t
4160              GOTO 4340                               ! A21:x    ,t (t=0->0)
4170              A(J,Y)=A21+Q2(J)*T1+(T1/X7)*(Q11-Q21)! Q21:x    ,t (t=0->0)
4180              IF A(J,Y)<.00001 THEN A(J,Y)=0
4190              F10=1                                   !A(J,Y):x    t+dt
4200              Ad=A(J,Y)                               !=|=============|====
4210                                                      !x-dx             x
4220              IF Ad=0 THEN
4230                Qs1=0
4240                As1=0
4250                Qd=0
4260                GOTO 4290
4270              END IF
4280                GOSUB 6590      ! A(J,Y)-->Q(J,Y)
4290              Qs2(J,Y)=Qs1
4300              As2(J,Y)=As1
4310              A(J,Y)=Ad
4320              Q(J,Y)=Qd
4330  ! ----------------------------------------------------------------------
4340              A11=A21                     !  Upstream          Downstream
4350              A21=A(J,Y+1)                ! Y-2    Y-1    Y    Y+1  Y+2
4360                                          !A(Y-2) A(Y-1) A(Y) ________\    t+dt
4370              Q11=Q21                     !Q(Y-2) Q(Y-1) Q(Y)         /     NEW
4380              Q21=Q(J,Y+1)                !       -------------
4390              IF Y<Y8 THEN 4470           ! lost |A11     A21|   A(Y+1)       t
4400              FOR P=1 TO N5               ! lost |Q11     Q21|   Q(Y+1)      OLD
4410                 IF X(P)=P0(J) THEN       !       -------------
4420                    Depth(P,K2)=Dep1      ! The dotted window is shifted
4430                    Vel(P,K2)=Vels        ! downstram after A(Y),Q(Y) are
4440                    Q8(P,K2)=Q(J,Y8)      ! computed
4450                 END IF                   !
4460              NEXT P                      !
4470           NEXT Y                         !
4480 ! =============3rd spatial loop is finished=======================
4490           IF J18(J1+1)=0 THEN 4700   ! If the next conduit is 99.....
```

```
4500                                     !
4510          IF Up(J18(J1+1))=A14 THEN  ! If the next conduit(J18(J1+1)) has
4520                                     ! upstream a channel then.....
4530            FOR I=1 TO N4
4540             IF J18(J1+1)=PO(I) THEN
4550              J=I
4560              GOTO 4580
4570             END IF
4580            NEXT I
4590            Q11=Q(J,1)
4600            Q(J,1)=0                 !
4610 !                                                N2(J2)? PO(J18(J1+1))
4620            FOR J2=1 TO N4           ! ============| |================|
4630             IF J18(J1+1)=N2(J2) THEN      !
4640               Q(J,1)=Q(J,1)+Q(J2,Y8)      !
4650             END IF
4660            NEXT J2
4670           A14=A14+1
4680          END IF
4690 ! ----------------------------------------
4700        NEXT J1                      !
4710 ! ================2nd spatial loop is finished=====================
4720     NEXT T7
4730 ! ================Second time-loop is finished=====================
4740    FOR P=1 TO N5
4750        IF Q7(P)<Q8(P,K2) THEN Q7(P)=Q8(P,K2)
4760    NEXT P
4770   NEXT K
4780 ! ================Main time-loop is finished=======================
4790 ! ================================================================
4800 !            PRINTOUT OF RESULTS
4810 ! ================================================================
4820   OUTPUT @Crt;FNClear$;
4830   OUTPUT @Crt USING 4840
4840   IMAGE 3/,29X,"Computations completed",2/,32X,"Printing results"
4841   S313=0
4860   A$=M$
4870   IF FNUpc$(A$[1,1])="D" THEN 5920
4880 ! ------------------------------------------------------------
4890   PRINT USING 4900
4900   IMAGE /,2X,"TIME(min)",6X,"MAX. THETA ",6X,"CONDUIT in which this occured",/,70("=")
4910   FOR K=1 TO (T9*60)/T1
4920         Th1=Th(1,K)
4930         R8=PO(1)
4940      FOR J=1 TO N4-1                     ! for all the conduits
4950         IF Th(J,K)<Th(J+1,K) THEN
4960            Th1=Th(J+1,K)
4970            R8=PO(J+1)
4980         END IF
4990      NEXT J
5000      PRINT USING 5010;(K*T1)/60,Th1,R8
5010      IMAGE 5X,DCDD,10X,3D.3D,20X,DD
5020   NEXT K
5030   FOR P=1 TO N5              ! N5=In how many nodes we want printout
5040      PRINT USING 5050;X(P)
5050      IMAGE /,4X,"Hydrograph in conduit ",DDD,/,4X,25("=")
5060      IF Minor=1 THEN
```

```
5070       PRINT USING 5080
5080       IMAGE /,4X,"ANALYSIS OF DUAL SYSTEM",/,4X,"MAJOP HYDROGRAPH"
5090      END IF
5100      IF C(P)=3 OR C(P)=4 THEN
5110        PRINT USING 5120
5120        IMAGE 3/,5X,"The Depth and the Velocity are of the above the pipe cross-section"
5130        PRINT USING 5140
5140        IMAGE /,5X,"If the pipe is running partly full the Depth and the Ve-",/,5X,"locity are set equal null"
5150      END IF
5160      PRINT USING 5170
5170      IMAGE /,5X,"Time(min)",4X,"Flowrate(m3/s)",4X,"Depth (m)",4X,"Velocity (m/s)",/,66("=")
5180      FOR K2=1 TO (T9*60)/T1
5181       S313=S313+Q8(P,K2)
5190       IF Minor=0 THEN
5200         PRINT USING 5240;(K2*T1)/60,Q8(P,K2),Depth(P,K2),Vel(P,K2)
5210       ELSE
5220         PRINT USING 5240;(K2*T1)/60,Q8(2,K2),Depth(P,K2),Vel(P,K2)
5230       END IF
5240         IMAGE 6X,3D.2D,10X,3D.DDD,7X,3D.DDD,10X,3D.DDD
5250      NEXT K2
5260      PRINT USING 5261;S313*T1
5261      IMAGE /,"TOTAL LOADS=",8D.4D,"m3",3/
5262      S313=0
5270   NEXT P
5271      FOR P=1 TO N5
5272       IF X1=2 THEN GOSUB 13930
5273      NEXT P
5280   GOTO 5610
5290 ! ------------------------------------------------------------------------------
5300      OUTPUT @Crt;FNClear$;
5310      S33=1
5320      OUTPUT @Crt USING 5330;J18(J1),Y-1
5330      IMAGE 3/,2X,"FOR ROUTING THROUGH ",2D,"  CHANNEL"  AT THE ",DD," SECTION"
5340      OUTPUT @Crt USING 5350;Th1
5350      IMAGE  "THETA=",3D.3D,">1 ====>","WAVE SPEED > DX/DT"
5360      OUTPUT @Crt USING 5370
5370      IMAGE 3/,"YOU HAVE TO REDUCE THETA BY DECIESING **DT**",/,"OR USING FEWER SECTIONS IN THE CHANNEL"
5380      OUTPUT @Crt USING 5390
5390      IMAGE "HOWEVER IF YOU WANT TO CONTINUE RUNNING THE PROGRAM AS IT IS",/,"Type Yes or press  ENTER otherwise NO"
5400      A$=""
5410      INPUT A$
5420      IF A$="" THEN
5430        PRINT FNClear$
5520        GOTO 4000
5530      END IF
5540      IF FNUpc$(A$[1,1])="N" THEN
5550         GOTO 5760
5560      ELSE
5570         PRINT FNClear$
5580         GOTO 3980
5590      END IF
5600 !------------------------------------------------------------------------------
5610      B=TIMEDATE MOD 86400
5620      PRINT USING 5630;INT(B/60),B MOD 60
5630      IMAGE 2/,"Run time = ",3D," min ",ZZ," secs",4/," "
5640      OUTPUT @Crt;FNClear$;
5650      OUTPUT @Crt USING 5660
```

```
5660      IMAGE //,"This hydrograph to be filed ???"
5670      A$=""
5680      INPUT A$
5690      IF FNUpc$(A$[1,1])="" THEN 5960
5700      IF FNUpc$(A$[1,1])="N" THEN 5960
5710      IF FNUpc$(A$[1,1])="Y" THEN
5720         OUTPUT @Crt;FNClear$;
5730         DISP "Filename...";
5740         INPUT File$
5750            FOR I=1 TO 300
5751             IF Minor=1 THEN
5752               Qplot(I)=Q8(2,I)
5753             ELSE
5760               Qplot(I)=Q8(1,I)
5761             END IF
5770            NEXT I
5780            FOR I=301 TO 365
5790               Qplot(I)=0
5800            NEXT I
5810         Flag1=1
5820         Number=T9/(T1/60)
5830         Timestep=T1/60
5840         MASS STORAGE IS Drive_data$
5850         CREATE BDAT File$,365,24
5860         ASSIGN @Data TO File$
5870         CONTROL @Data,7;365
5880         ON END @Data GOTO 5930
5890         OUTPUT @Data,1;Flag1
5900         OUTPUT @Data,2;Number,Timestep
5910         OUTPUT @Data,3
5920         OUTPUT @Data;Qplot(*)
5930         ASSIGN @Data TO *
5940         MASS STORAGE IS Drive_def$
5950      END IF
5960      OUTPUT @Crt;FNClear$;
5970      OUTPUT @Crt USING 5980
5980      IMAGE //,"Do you wish to return to the edit program ????",2/,"> Enter YES or NO"
5990      A$=""
6000      INPUT A$
6010      IF FNUpc$(A$[1,1])="Y" THEN 6040
6020      IF FNUpc$(A$[1,1])="N" THEN 6080
6030      GOTO 5640
6040      OUTPUT @Crt;FNClear$;
6050      OUTPUT @Crt USING 6060
6060      IMAGE 4/,34X,"INITIALISING",2/,34X,"PLEASE  WAIT"
6070      LOAD "WITWAT",1
6080      OUTPUT @Crt;FNClear$;
6090      OUTPUT @Crt USING 6100
6100      IMAGE ////,36X,"FINISHED"
6110      PRINTER IS 1
6120      BEEP 1000,.5
6130      GOTO 14640
6140 ! ==============================================================
6150 !    SUBROUTINE
6160 !    ** NEWTON RAPHSON **  ( catchments )
6170 ! ==============================================================
6180 ! IF F9=1 THEN 5370
```

```
6190      IF K1<=T2*60 THEN                         ! current time< time to runoff
6200         D6=.0001
6210         GOTO 6580
6220      END IF
6230 ! ---------------------------------
6240      IF K2=1 AND K1>T2*60 THEN                 ! time for runoff<first interval
6250         I7=(I9(K)-F3)*(K1-T2*60)/T1
6260         GOTO 6400
6270      END IF
6280 ! --------------------------------
6290      IF K2>1 AND K1-T1<T2*60 AND K1>T2*60 THEN
6300         I7=(I9(K)-F3)*(K1-T2*60)/T1
6310         GOTO 6400
6320      END IF
6330 ! --------------------------------
6340      IF K1<=T*60 THEN                    ! current time< storm duration
6350         I7=I9(K)-F3
6360         GOTO 6400
6370      END IF
6380 ! --------------------------------
6390      IF K1>T*60 THEN I7=-F4              ! after storm
6400      A$=F$
6410      IF K1>T*60 AND FNUpc$(A$[1,1])="S" THEN I7=-F4
6420      M=1
          F1=D4-D5+I7*T1+Q_cascade*T1/Area_temp-A7*T1*(D4/2+D5/2)^(5/3)
          F2=-1-5/6*A7*T1*(D4/2+D5/2)^(2/3)
          D6=D5-F1/F2
          IF ABS(D6-D5)/D6<.001 THEN 6560
          M=M+1
          IF M>20 THEN
             D5=D6
             GOTO 6430
          END IF
6520      PRINT USING 6530;A0(I),K1
6530      IMAGE //,"Excessive iterations",/,"Area no.",DDD,"at time",DDDD,"seconds"
6540      PRINT USING 6550
6550      IMAGE "Latest value used as default",/,"Execution continues"
6560      IF D6<=0 THEN D6=.0001
6570      F9=0! RESET CHANNEL FLAG
6580      RETURN
6590 !=================================================================
6600 !   SUBROUTINE
6610 !   FOR A GIVEN AREA IT ACCOUNTS BETWEEN WHICH
6620 !   LEFT NODES IS THE DEPTH OF FLOW
6630 ! -----------------------------------------------------
6640    IF C(J)=3 OR C(J)=4 THEN                  ! C(J)=3 OR 4 :PIPE
6650      IF F10=0 THEN 11360                     ! F10=0: Q-->A
6660      IF F10=1 THEN 11250                     ! F10=1: A-->Q
6670    END IF
6680 ! -----------------------------------------------------
6690 !   FIND THE MINIMUM ELEVATION OF THE CROSS SECTION
6700 ! -----------------------------------------------------
6710      I1=1
6720      I11=1
6730      FOR I=1 TO N10(J)-1                     ! N10=number of coordinates
6740         IF X11(J,I+1,2)>=X11(J,I,2) THEN 6760
6750         K5=I+1                               ! K5=number of coordinate with
```

```
6760      NEXT I                                     !    the minimum elevation
6770      II=1
6780 ! ------------------------------------------------------------------
6790 ! FIND BETWEEN WHICH NODES IS THE AREA
6800 ! ------------------------------------------------------------------
6810      FOR S=1 TO K5-1             ! for all the nodes before K5
6820         L=K5-S                   ! starting from the K5-1
6830                                  !
6840         FOR A7=K5+1 TO N10(J)          !                 X11(A7,2)*  |
6850            IF L=0 THEN 7290            !      X11(L,2)            /  | |
6851            IF X11(J,A7,2)<X11(J,L,2) THEN 6910!*-----------------* R | |
6860            Z1=X11(J,A7,2)-X11(J,A7-1,2)!      \              /      |Z2
6870            Z2=X11(J,L,2)-X11(J,A7-1,2 !        \            /      Z1|
6880            Z3=X11(J,A7,1)-X11(J,A7-1,1)!        \      *X(A7-1,2)|  |
6890            Z4=(Z2/Z1)*Z3               !   X11(K5,2)    * /
6900            GOTO 6920                   !                 <...Z3...>
6910         NEXT A7                        !
6920         FOR I=L TO A7-1             !
6930            FOR M=1 TO 2             !   CREATE AN ARRAY X(R,J)
6940               R=I+1-L               !   WITH ELEMENTS THE
6950               X15(R,M)=X11(J,I,M)   !   COORDINATES OF THE NODES
6960            NEXT M                   !  (FROM L--->A7-1 AND NODE "R")
6970         NEXT I                      !   THAT SURROUND THE AREA "A"
6980         X15(R+1,1)=X11(J,A7-1,1)+Z4!
6990         X15(R+1,2)=X11(J,L,2)       !
7000         P3=(X15(1,1)-X15(R+1,1))*(X15(1,2)+X15(R+1,2))
7010         FOR I=1 TO R                        !
7020            S3=X15(I+1,1)-X15(I,1)           !
7030            S4=X15(I,2)+X15(I+1,2)           !    CALCULATION OF AREA FOR A
7040            P3=P3+S3*S4                      !    GIVEN LEFT NODE (L)
7050         NEXT I                              !
7060         As=ABS(P3)/2                        !
7070         IF F10=0 THEN                        !
7080           IF C(J)=2 OR C(J)=4 THEN           ! for the given de
7090             IF II=1 THEN Dep1=X11(J,L,2)     ! go and find the
7100             GOTO 7880                        !
7110           ELSE                               !
7120             IF II=1 THEN Dep1=X11(J,L,2)     !
7130             GOTO 8590                        !
7140           END IF                             !
7150         END IF                               !
7160         IF As<Ad THEN 7200                !find the first left node with
7170                                           !greater area than Ad
7180         IF F10=0 THEN 7220                        !
7190         IF F10=1 THEN 7270
7200      NEXT S
7210 ! ----------------------------------------------------------------------
7220 !  SUBROUTINE
7230 !  **NEWTON RAPHSON**
7240 !  FOR  A GIVEN AREA AND KNOWING BETWEEN WHICH LEFT NODES
7250 !  IS THE WATER LEVEL IT ACCOUNTS THE DEPTH FLOW
7260 ! ----------------------------------------------------------------------
7270      Dep1=(X11(J,L,2)+X11(J,L+1,2))/2           !
7280      IF Dep1>X11(J,1,2) OR Dep1>X11(J,N10(J),2) THEN !
7290        PRINT FNClear$;
7300        BEEP
7310        PRINT USING 7320;J
```

```
7320        IMAGE 3/,10X,"EXISTING CONDUIT ",DO," IS INADEQUATE",
7325PRINT USING 7326
7326        IMAGE 4/,10X,"CHANGE THE CROSS SECTION OF THE CONDUIT"
7330        STOP
7340      END IF
7350 ! ------------------------------------------- !    TYPICAL CROSS-SECTION
7360      FOR I=K5+1 TO N10(J)         !It finds    !          X11(M+1,2)*
7370         IF X11(J,I,2)<Dep1 THEN 7400!between which! \                  /
7380         M=I-1                     !right nodes !X11(L,2)*-------------/
7390         GOTO 7430                 !is the depth !X15(1,1)------------/X(R+1,2
7400      NEXT I                       !   H1       !         \|||||||||*X11(M,2)
7410 !                                              X11(L+1,2) *|||||||/
7420 !                                                          \||||||/
7430      Y1=(X11(J,L+1,1)-X11(J,L,1))/(X11(J,L,2)-X11(J,L+1,2))
7440      Y2=(X11(J,M+1,1)-X11(J,M,1))/(X11(J,M+1,2)-X11(J,M,2))
7450      Y3=2*(Y2+Y1)*Dep1+2*X11(J,M,1)-X11(J,L+1,1)-X11(J,L+1,2)-2*X11(J,M,2)*Y2-2*X11(J,L+1,2)*Y1
7460      FOR I=L+1 TO M             !
7470         FOR A7=1 TO 2 STEP 1    !
7480            R=I-L+1              !
7490            X15(R,A7)=X11(J,I,A7)                ! CREATE AN ARRAY X15(R+1,A7)
7500         NEXT A7                                 ! WITH ELEMENTS THE
7510      NEXT I                                     ! COORDINATES OF THE NODES
7520      X15(1,1)=Y1*(X11(J,L,2)-Dep1)+X11(J,L,1)   ! THAT SURROUND THE AREA
7530      X15(1,2)=Dep1                              !
7540      X15(R+1,1)=X11(J,M+1,1)-Y2*(X11(J,M+1,2)-Dep1)!
7550      X15(R+1,2)=Dep1                            !
7560      P3=(X15(1,1)-X15(R+1,1))*(X15(1,2)+X15(R+1,2))!
7570      FOR I=1 TO R                                !    CALCULATE THE AREA
7580         S3=X15(I+1,1)-X15(I,1)                   !
7590         S4=X15(I,2)+X15(I+1,2)                   !
7600         P3=P3+S3*S4                              !
7610      NEXT I                                      !
7620      As=ABS(P3)/2
7630      IF F10=1 AND III=0 THEN 7830
7640      IF F10=1 THEN 7670
7650      IF F10=0 AND C(J)=2 OR C(J)=4 THEN 7880      ! Q-->A  natural chan.
7660      IF F10=0 AND C(J)=1 OR C(J)=3 THEN 8590      ! Q-->A  compound chan.
7670      Y4=2*(As-Ad)                                 !
7671      Y3=ABS(Y3)
7680      Dep2=Dep1-(Y4/Y3)                           !       NEWTON-RAPHSON
7690      IF ABS(As-Ad)/As<.001 THEN                   !
7700         IF III<>0 THEN
7710         Dep2=Dep1+.000000001
7720         As1=As
7730         III=1
7740         GOTO 7830
7750        END IF
7760         III=0
7770         GOTO 7830
7780      ELSE
7790         III=III+1
7800         IF C(J)=1 OR C(J)=3 THEN 8460
7810         GOTO 7880
7820      END IF
7830      IF C(J)=1 OR C(J)=3 THEN 8460    ! Find the areas for compound channel
7840 ! ------------------------------------------------------------------------
7850 ! CALCULATION OF WETTED PERIMETER AND FLOW
```

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7860 ! ACCORDING  TO MANNING'S FORMULA FOR NATURAL CHANNEL --------------------
7870 ! ----------------------------------------------------------------------------
7880      W=0                                                                   |
7890      FOR I=1 TO R                          !   I*                          |
7900         S3=(X15(I+1,1)-X15(I,1))^2         !     \       \         X15(I+1,2)-X15(I,2)
7910         S4=(X15(I+1,2)-X15(I,2))^2         !                \              !
7920         W=W+(S3+S4)^.5                     !                 I+1*          -
7930      NEXT I                                !                      \
7940 ! *************************************!  ________________________
7950      Vels=(S2(J)^.5*(As/W)^.6667)/N(J,2)!  X15(I+1,1)-X15(I,1)
7960      Qs=Vels*As                            !
7970 ! **************************************
7980 ! -----------------------CONTROL-----------------------------------------
7990      IF L=1 AND Qs<Qd AND II=1 THEN
8000       IF Dep1>X11(J,1,2) OR Dep1>X11(J,N10(J),2) THEN 7290
8010      END IF
8020      IF F10=1 AND III=1 THEN                       ! last time
8030        Qs1=Qs
8040        Dep1=Dep2
8050        III=0
8060        GOTO 7360
8070      END IF
8080      IF F10=1 AND III=0 THEN                       ! go to the main  program
8090         Ad=As
8100         Qd=Qs
8110         IF C(J)=3 OR C(J)=4 THEN 11450
8120         GOTO 11550
8130      END IF
8140      IF F10=1 THEN                                 ! one more time
8150         Dep1=Dep2
8160         GOTO 7360
8170      END IF
8180 ! --------------------------------------------------------
8190      IF ABS(Qs-Qd)/Qd<.01 THEN
8200       IF F10=0 AND III<>0 THEN
8210         As1=As
8220         Qs1=Qs
8230         Dep1=Dep1+.0000000001
8240         III=0
8250         GOTO 7360
8260       END IF
8270       IF F10=0 AND III=0 THEN
8280         Ad=As
8290         Qd=Qs
8300         IF C(J)=3 OR C(J)=4 THEN 11450
8310         GOTO 11550
8320       END IF
8330      END IF
8340 ! --------------------------------------------------------
8350      III=III+1
8360      IF F10=0 AND II>2 THEN 10830
8370      IF F10=0 AND II=2 THEN 11090     !
8380      IF F10=0 AND Qs>Qd THEN          !        only for II=1
8390         GOTO 10790                    !        and when it finds
8400      ELSE                             !        a left node L with
8410         L=L-1                         !        Qs>Qd then it goes
8420         IF L=0 THEN L=1
```

```
8430        GOTO 6840                        !      to 7270
8440     END IF                              !
8450 !=====================================================================
8460 ! SUBROUTINE FOR THE
8470 ! CALCULATION OF AREAS AND WETTED PERIMETERS
8480 ! FOR COMPOUND CHANNEL
8490 ! -----------------------------------------------------------------------
8500 !          L*
8510      !      \!______________________________________/R+1
8520      !       \          |                   |      /
8530      !        *L+1      |                   |     /
8540      !         \___A11__|                   |___A31__/
8550      !         C11(J,1)-L\      A21         /C11(J,2)-L
8560      !                    \________________/
8570      !
8590 !------------------SPESIAL CASE----------------------------------------
8591   IF C11(J,1)=0 THEN 8610
8592   IF C11(J,2)=0 AND Dep1<=X11(J,C11(J,1),2) THEN 7880
8594   IF C11(J,2)=0 AND Dep1>=X11(J,C11(J,1),2) THEN 8960
8600   IF Dep1<=X11(J,C11(J,1),2) THEN                  ! IF A11=0
8610        IF Dep1<=X11(J,C11(J,2),2) THEN 7880    ! IF A11=0 AND A31=0
8620        A11=0
8630        W1=0                                      ! IF A11=0 AND A31>0
8640 !                                            CALCULATE AREA A21
8650      FOR I=L+1 TO C11(J,2)               !  ________\-------------|---------/
8660         X6(I-(L+1)+2,1)=X11(J,I,1)    !           \  A21        |__A31___/
8670         X6(I-(L+1)+2,2)=X11(J,I,2)    !            \            /
8680      NEXT I                             !             \__________/
8690      X6(1,1)=X15(1,1)                   !
8700      X6(1,2)=Dep1                       !
8710      I=C11(J,2)-(L+1)+3
8720      X6(I,1)=X11(J,C11(J,2),1)
8730      X6(I,2)=Dep1
8740      A21=(X6(1,1)-X6(I,1))*(X6(1,2)+X6(I,2))
8750      FOR A7=1 TO I-1                   !
8760         S3=X6(A7+1,1)-X6(A7,1)         !      CALCULATE  AREA "A21"
8770         S4=X6(A7,2)+X6(A7+1,2)         !
8780         A21=-A21-S3*S4                 !
8790      NEXT A7                           !
8800      A21=ABS(A21)/2
8810      IF (Dep1-X11(J,C11(J,2),2))/(X11(J,C11(J,2),2)-X11(J,K5,2))<.30 THEN
8820        M=I-1
8830      ELSE
8840        M=I-2
8850      END IF
8860      W2=0                                !
8870      FOR A7=1 TO M                       !
8880         S3=(X6(A7+1,1)-X6(A7,1))^2       !
8890         S4=(X6(A7+1,2)-X6(A7,2))^2       !   CALCULATE WETTED PERIMETER "W2"
8900         W2=W2+(S3+S4)^.5                 !
8910      NEXT A7                             !
8920      W2=(A21/W2)^.6667/N(J,2)            !
8930        GOTO 9870                         !
8940   END IF
8950 ! -------------------------------------NORMAL**--------------------
8960 !
8970      FOR I=L+1 TO C11(J,1)               !
```

```
8980                                                   !
8990         X6(I-(L+1)+2,1)=X15(I,1)                  !
9000         X6(I-(L+1)+2,2)=X15(I,2)                  !
9010      NEXT I                                       !
9020      I=C11(J,1)-(L+1)+3
9030      X6(1,1)=X15(1,1)
9040      X6(1,2)=Dep1
9050      X6(I,1)=X15(C11(J,1),1)                       !
9060      X6(I,2)=Dep1                                  !
9070      P3=(X6(1,1)-X6(I,1))*(X6(1,2)+X6(I,2))
9080      FOR A7=1 TO I-1               !
9090          S3=X6(A7+1,1)-X6(A7,1)    !           CALCULATE AREA "A1"
9100          S4=X6(A7,2)+X6(A7+1,2)    !
9110          P3=P3+S3*S4               !
9120      NEXT A7                       !
9130      A11=ABS(P3)/2
9140      W1=0                              !
9150      FOR A7=1 TO I-2                   !
9160          S3=(X6(A7+1,1)-X6(A7,1))^2!
9170          S4=(X6(A7+1,2)-X6(A7,2))^2!       CALCULATE WETTED PERIMETER "W1"
9180          W1=W1+(S3+S4)^.5              !
9190      NEXT A7                           !
9191      IF A11=0 THEN
9192        W1=0
9193        GOTO 9203
9194      END IF
9200      W1=(A11/W1)^.6667/N(J,1)      !
9203      IF C11(J,2)<>0 THEN 9560
9210 !------------------ -----------SPECIAL CASE-------------------------------------
9220    IF Dep1>=X11(J,C11(J,1),2) THEN                 ! A31=0
9230      A31=0
9240      W3=0                                           \                    /
9250      FOR I=C11(J,1) TO (L-1)+R         !             \___A11__|           /
9260          X6(I-C11(J,1)+2,1)=X11(J,I,1)!                       \   A21   /
9270          X6(I-C11(J,1)+2,2)=X11(J,I,2)!                        \_______/
9280      NEXT I                            !
9290      X6(1,1)=X11(J,C11(J,1),1)         !
9300      X6(1,2)=Dep1                      !
9310      I=(L-1)+R-C11(J,1)+3                         CALCULATE AREA "A2"
9320      X6(I,1)=X15(R+1,1)                !
9330      X6(I,2)=Dep1                      !
9340       P3=(X6(1,1)-X6(I,1))*(X6(1,2)+X6(I,2))
9350      FOR A7=2 TO I-1                   !
9360          S3=X6(A7+1,1)-X6(A7,1)        !
9370          S4=X6(A7,2)+X6(A7+1,2)        !
9380          P3=P3+S3*S4                   !
9390      NEXT A7                           !
9400      A21=ABS(P3)/2
9410      W2=0                                  !
9420      IF (Dep1-X11(J,C11(J,1),2))/(X11(J,C11(J,1),2)-X11(J,K5,2))<.30 THEN
9430        M=1
9440      ELSE
9450        M=2
9460      END IF
9470      FOR A7=M TO I-1                       !
9480          S3=(X6(A7+1,1)-X6(A7,1))^2        !
9490          S4=(X6(A7+1,2)-X6(A7,2))^2        !  CALCULATE WETTED PERIMETER "W2"
```

```
9500        W2=W2+(S3+S4)^.5                            !
9510     NEXT A7                                        !
9520     W2=(A21/W2)^.6667/N(J,2)                       !
9530     GOTO 10130
9540   END IF                                           ------------------------------NORMAL**--------------
9550 ! ---------------------------------------------------------------------------------
9560     FOR I=C11(J,1) TO C11(J,2)               !
9570        X6(I-C11(J,1)+2,1)=X11(J,I,1)          !
9580        X6(I-C11(J,1)+2,2)=X11(J,I,2)          !
9590     NEXT I                                   !
9600     X6(1,1)=X11(J,C11(J,1),1)                !
9610     X6(1,2)=Dep1                             !
9620     I=C11(J,2)-C11(J,1)+3                            CALCULATE AREA "A2"
9630     X6(I,1)=X11(J,C11(J,2),1)                !
9640     X6(I,2)=Dep1                             !
9650      P3=(X6(1,1)-X6(I,1))*(X6(1,2)+X6(I,2))
9660     FOR A7=2 TO I-2                          !
9670        S3=X6(A7+1,1)-X6(A7,1)                !
9680        S4=X6(A7,2)+X6(A7+1,2)                !
9690         P3=P3+S3*S4                          !
9700     NEXT A7                                  !
9710     A21=ABS(P3)/2
9720     W2=0                                      !
9730     IF (Dep1-X11(J,C11(J,2),2))/(X11(J,C11(J,2),2)-X11(J,K5,2))<.30 THEN !
9740       P3=1                                    !    |______    | d/D<.30 |   _d________|
9750       M=I-1                                   !               |________|    D
9760     ELSE                                      !
9770       P3=2                                    !
9780       M=I-2                                   !    |______     d/D>.30     _d________|
9790     END IF                                    !               |_________|   D
9800     FOR A7=P3 TO M                            !
9810        S3=(X6(A7+1,1)-X6(A7,1))^2             !   CALCULATE WETTED PERIMETER "W2"
9820        S4=(X6(A7+1,2)-X6(A7,2))^2             !
9830        W2=W2+(S3+S4)^.5                       !
9840     NEXT A7                                   !
9850     W2=(A21/W2)^.6667/N(J,2)                  !  ---------------------------NORMAL**------------
9860 ! ---------------------------------------------------------------------------------
9870   FOR I=C11(J,2) TO (L-1)+R                   !
9880      X6(I-C11(J,2)+2,1)=X11(J,I,1)            !      CALCULATE AREA "A3"
9890      X6(I-C11(J,2)+2,2)=X11(J,I,2)            !
9900   NEXT I                                      !
9910   X6(1,1)=X11(J,C11(J,2),1)                   !
9920   X6(1,2)=Dep1
9930   I=(L-1)+R-C11(J,2)+3
9940   X6(I,1)=X15(R+1,1)
9950   X6(I,2)=Dep1
9960    P3=(X6(1,1)-X6(I,1))*(X6(1,2)+X6(I,2))
9970   FOR A7=2 TO I-1                             !
9980          S3=X6(A7+1,1)-X6(A7,1)               !
9990          S4=X6(A7,2)+X6(A7+1,2)               !
10000          P3=P3+S3*S4                         !
10010  NEXT A7
10020     A31=ABS(P3)/2                                 !
10030     W3=0                                          !
10040     FOR A7=2 TO I-1                               !
10050        S3=(X6(A7+1,1)-X6(A7,1))^2                 !CALCULATE WETTED PERIMETER "W3"
10060        S4=(X6(A7+1,2)-X6(A7,2))^2
```

```
10070       W3=W3+(S3+S4)^.5                     !
10080     NEXT A7                                !
10082     IF A31=0 THEN
10083       W3=0
10084       GOTO 10100
10085     END IF
10090     W3=(A31/W3)^.6667/N(J,3)               !
10100! ======================================================================
10110! CALCULATION OF FLOW IN COMPOUND CHANNEL ACCORDING
10120! TO MANNING'S FORMULA
10130! ----------------------------------------------------------------------
10140     W1=W1*A11
10150     W2=W2*A21
10160     W3=W3*A31
10170     As=A11+A21+A31
10180     Qs=S2(J)^.5*(W1+W2+W3)
10190     Vels=Qs/As
10200! ------------ CONTROL----------------------
10210     IF L=1 AND Qs<Qd AND II=1 THEN
10220       IF Dep1>X11(J,1,2) OR Dep1>X11(J,N10(J),2) THEN 7290
10230     END IF
10240     IF F10=1 AND III=1 THEN         ! one last time
10250        Qs1=Qs
10260        Dep1=Dep2
10270        III=0
10280        GOTO 7360
10290     END IF
10300     IF F10=1 AND III=0 THEN         ! go to main program
10310        Ad=As
10320        Qd=Qs
10330        IF C(J)=3 OR C(J)=4 THEN 11450
10340        GOTO 11550
10350     END IF
10360     IF F10=1 THEN                   ! one more time
10370        Dep1=Dep2
10380        GOTO 7360
10390     END IF
10400! -----------------------------------------
10410     IF ABS(Qs-Qd)/Qd<.01 THEN
10420       IF F10=0 AND III<>0 THEN
10430        As1=As
10440        Qs1=Qs
10450        Dep1=Dep1+.00000000001
10460        III=0
10470        GOTO 7360
10480       END IF
10490       IF F10=0 AND III=0 THEN
10500        Ad=As
10510        Qd=Qs
10520        IF C(J)=3 OR C(J)=4 THEN 11450    !only for pipes
10530        GOTO 11550
10540       END IF
10550     END IF
10560! -----------------------------------------
10570     III=III+1
10580     IF F10=0 AND II>2 THEN 10830
10590     IF F10=0 AND II=2 THEN 11090     ! only if II=2
```

```
10600      IF F10=0 AND Qs>Qd THEN               ! only if Ii=1
10610         GOTO 10790                         ! and when it finds
10620      ELSE                                  ! a left node L with Qs>Qd
10630         L=L-1                              ! then it goes to 7270
10640         GOTO 6840                          !
10650      END IF                                !
10660 ! ---------------------------------------------------------------------
10670 ! SUBROUTINE FOR THE
10680 ! APPROXIMATION TO Qd
10690 ! (FOR GIVEN Qd FINDS THE Ad)
10700 ! ---------------------------------------------------------------------
10710 ! First specify between which nodes is Qd
10720 ! (Dep1=L-1-->Qs?Qd, Dep1=L-->Qs?Qd, Dep1=L+1-->Qs?Qd.......)
10730 ! (..until Qs>Qd  .This node is L,Qs->Q13(1),Ii=1,H6=H(L),H5=H(L+1)  )
10740 ! Then for Ii=2 and H13(2)=(H5+H6)/2 I find :Q13(2)
10750 ! If Q13(2)<Qd-->H13(3)=(H13(2)+H6)/2
10760 ! If Q13(2)>Qd-->H13(3)=(H13(2)+H5)/2
10770 ! And so on. Confine Qd each time more and more..........
10780 ! ---------------------------------------------------------------------
10790       H5=X11(J,L+1,2)
10800       H6=X11(J,L,2)
10810       Ii=2                                       !            L*H6
10820       IF Ii=2 THEN 7270                          !             |
10830       Q13(Ii)=Qs                                 !             |
10840       IF (Q13(Ii)-Qd)*(Q13(Ii-1)-Qd)>0 THEN 11090 !            |
10850         IF H13(Ii)<H13(Ii-1) THEN                !     Qd??    |
10860          H5=H13(Ii)                              !             |Q13(2)
10870          H6=H13(Ii-1)                            !             |
10880         ELSE                                     !             |
10890          H5=H13(Ii-1)                            !             |
10900          H6=H13(Ii)                              !          L+1*H5
10910         END IF                                   !
11050       H13(Ii+1)=(H5+H6)/2
11060       Dep1=H13(Ii+1)                             !
11070       Ii=Ii+1                                    !
11080       GOTO 7360
11090       Q13(Ii)=Qs                       !Q13(2) is the flow for H=(L)+(L+1)/2
11100       H13(Ii)=Dep1                     !
11110       IF Q13(Ii)<Qd THEN               !
11120          H13(Ii+1)=(H13(Ii)+H6)/2      !
11130       END IF                           !
11140       IF Q13(Ii)>Qd THEN               !
11150          H13(Ii+1)=(H13(Ii)+H5)/2      !
11160       END IF                           !
11170       Dep1=H13(Ii+1)                   !
11180       Ii=Ii+1                          !
11190       GOTO 7360                        !
11200 ! =====================================================================
11210 ! SUBROUTINE                         =======C(J)=3 or 4======
11220 ! FOR A GIVEN DIAMETER OF PIPE AND OVERLAND CROSS-SECTION
11230 ! AND GIVEN AREA IT ACCOUNTS THE FLOW  (Ad--->Qd), AND THE OPPOSITE
11240 ! =====================================================================
11250       A6=(3.1416*Diam(J)^2)/4
11260       IF Ad<A6 THEN
11270          Qd=((S2(J)^.5/Mannpipe(J))/((4/Diam(J))^(2/3)))*Ad
11280          Th1=(T1/X7)*((S2(J)^.5/Mannpipe(J))/(4/Diam(J))^(2/3))
11290          GOTO 11550
```

```
11300      ELSE
11310         Ad=Ad-A6
11320         GOTO 6690          ! Ach---->Qch     (F10=1)
11330      END IF
11340      GOTO 11450
11350! =====================Qd--->Ad======================
11360      IF Qd<Q6(J) THEN
11370         Ad=Qd/((S2(J)^.5/Mannpipe(J))/((4/Diam(J))^(2/3)))
11380         Th1=(T1/X7)*((S2(J)^.5/Mannpipe(J))/(4/Diam(J))^(2/3))
11390         GOTO 11550
11400      ELSE
11410         Qd=Qd-Q6(J)
11420         GOTO 6690          ! Qch--->Ach     (F10=0)
11430      END IF
11440! ---------------------CONTROL--------------------------------
11450      A6=(3.1416*Diam(J)^2)/4
11460      IF Minor=1 AND N5=1 THEN               ! Minor=1 -->DUAL
11470        IF Y=Y8 AND X(1)=PO(J) THEN          ! N5=1    -->one node specified
11480          Q8(2,K2)=Qd                        ! X(1)=PO(J)-->
11490        END IF
11500      END IF
11510      As1=As1+A6
11520      Qs1=Qs1+Q6(J)
11530      Qd=Qd+Q6(J)
11540      Ad=Ad+A6
11550      RETURN
11560! ================================================================
11570!                     SUBROUTINE  INFILTRATION
11580! ================================================================
11590      A$=F$
11600      IF FNUpc$(A$[1,1])="H" THEN 11640
11610      F3=Infil_i(I)
11620      F4=Infil_f(I)
11630      GOTO 11770
11640! HORTON INFILTRATION
11650      IF K1>T*60 THEN I9(K)=0     !K1=current time>duration of storm(sec)
11660      IF I9(K)+D4/T1>=G1(I) THEN 11680
11670      IF I9(K)+D4/T1<G1(I) THEN 11720
11680! V2=Infil_f(I)*T1+1/K4*(Infil_i(I)-Infil_f(I))*(1-EXP(-(K4*T1)))*EXP(-(K4*K1))
11690! V2=Infil_f(I)*T1+1/K4*(G1(I)-Infil_f(I))*(1-EXP(-(K4*T1)))*EXP(-(K4*K1))
11700      V2=G1(I)*T1
11710      GOTO 11730
11720      V2=I9(K)*T1+D4
11730      Vol_infil(I)=Vol_infil(I)+V2
11740      G1(I)=Infil_i(I)-Vol_infil(I)*K4+Infil_f(I)*K1*K4
11750      F3=G1(I)
11760      F4=G1(I)
11770      RETURN
11780! ================================================================
11790!                SUBROUTINE **HEADING**
11800! ================================================================
11810      OUTPUT @Crt USING 11820
11820      IMAGE 3/,30X,"Printing control data"
11830      PRINT CHR$(27);CHR$(71)
11840      PRINT USING 11850
11850      IMAGE 4X,72("="),/,7X,"W I T W A T   S T O R M W A T E R   D R A I N A G E   P R O G R A M",/,4X,72("=")
11860 !
```

```
11870 !
11880     PRINT USING 11890                    ! Program
11890     IMAGE 30X,"Version III.1/9816"       ! version
11900 !
11910 !
11920     PRINT USING 11930
11930     IMAGE /,16X,"Developed by: Water Systems Research Programme"
11940     PRINT USING 11950
11950     IMAGE 30X,"University of the Witwatersrand",/,30X,"Johannesburg",/,30X,"South Africa"
11960     PRINT CHR$(27);CHR$(72)
11970     PRINT USING 11980;W$
11980     IMAGE 5/,"Data file name: ",K,/,22("-"),/
11990     PRINT USING 12000;T1$,T2$
12000     IMAGE /,K,/,K
12010     PRINT USING 12020
12020     IMAGE /,"ANALYSIS option requested with KINEMATIC routing"
12030     A$=F$
12040     IF FNUpc$(A$[1,1])="H" THEN
12050        PRINT USING 12060;K4
12060        IMAGE /,"HORTON infiltration routine with decay constant = ",Z.5D," /s"
12070     ELSE
12080        PRINT USING 12090
12090        IMAGE /,"SIMPLIFIED infiltration routine"
12100     END IF
12110     A$=U$
12120     IF FNUpc$(A$[1,1])="U" THEN 12380
12130     IF FNUpc$(A$[1,1])="C" THEN 12160
12140     IF FNUpc$(A$[1,1])="T" THEN 12250
12150     IF FNUpc$(A$[1,1])="R" THEN 12310
12160     PRINT USING 12170
12170     IMAGE /,"Regionalised CHICAGO synthetic distribution parameters:-"
12180     PRINT USING 12190;I$,M1,R1
12190     IMAGE 6X,"Region:",33X,K,/,6X,"MAP:",36X,4D," mm",/,6X,"Recurrence interval:",18X,3D," years"
12200     PRINT USING 12210;T1,R3
12210     IMAGE 6X,"Simulation time step:",19X,3D," sec",/,6X,"Time to peak ratio:",24X,Z.2D
12220     PRINT USING 12230;T,T9
12230     IMAGE 6X,"Storm duration:",25X,3D," min",/,6X,"Simulation duration:",20X,3D," min"
12240     GOTO 12730
12250     PRINT USING 12260
12260     IMAGE /,"Regionalised TRIANGULAR synthetic distribution parameters:-"
12270     PRINT USING 12190;I$,M1,R1
12280     PRINT USING 12210;T1,R3
12290     PRINT USING 12230;T,T9
12300     GOTO 12730
12310     PRINT USING 12320
12320     IMAGE /,"Regionalised RECTANGULAR synthetic distribution parameters:-"
12330     PRINT USING 12190;I$,M1,R1
 2340     PRINT USING 12350;T1
12350     IMAGE 6X,"Simulation time step:",19X,3D," min"
12360     PRINT USING 12230;T,T9
12370     GOTO 12730
12380     PRINT USING 12390
12390     IMAGE /, "User input hyetograph",/,21("-")
12400     PRINT USING 12410;R1
12410     IMAGE "Ordinates are in mm/h at ",DDD," min intervals",/," "
12420     FOR K=1 TO 1/R1
12430        I9(K)=I9(K)*C1
```

```
12440        U=K MOD 6
12450        U=DROUND((U/6),3)
12460        U1=DROUND((1/6),3)
12470        U2=DROUND((2/6),3)
12480        U3=DROUND((3/6),3)
12490        U4=DROUND((4/6),3)
12500        U5=DROUND((5/6),3)
12510        IF U=0 THEN PRINT USING 12570;I9(K-5),I9(K-4),I9(K-3),I9(K-2),I9(K-1),I9(K)
12520        IF RI*K=T AND U=U5 THEN PRINT USING 12580;I9(K-4),I9(K-3),I9(K-2),I9(K-1),I9(K)
12530        IF RI*K=T AND U=U4 THEN PRINT USING 12590;I9(K-3),I9(K-2),I9(K-1),I9(K)
12540        IF RI*K=T AND U=U3 THEN PRINT USING 12600;I9(K-2),I9(K-1),I9(K)
12550        IF RI*K=T AND U=U2 THEN PRINT USING 12610;I9(K-1),I9(K)
12560        IF RI*K=T AND U=U1 THEN PRINT USING 12620;I9(K)
12570        IMAGE 6X,6(3D.DD,3X)
12580        IMAGE 6X,5(3D.DD,3X)
12590        IMAGE 6X,4(3D.DD,3X)
12600        IMAGE 6X,3(3D.DD,3X)
12610        IMAGE 6X,2(3D.DD,3X)
12620        IMAGE 6X,3D.DD,3X
12630     NEXT K
12640     FOR K=1 TO T/RI
12650        I9(K)=I9(K)/C1
12660     NEXT K
12670     PRINT USING 12680;T
12680     IMAGE /,"Storm duration = ",DDD," min"
12690     PRINT USING 12700;RI
12700     IMAGE /,"Rainfall interval= ",DDD," min"
12710     PRINT USING 12720;T1,T9
12720     IMAGE "Simulation time step = ",DDD," sec,   Simulation duration = ",DDD," min"
12730     IF Hydr_user=0 THEN 12910
12740     PRINT USING 12750;N_user
12750     IMAGE /,"User supplied hydrograph at node ",3Z,/,36("-"),
12755     PRINT USING 12756;
12756     IMAGE /,"Hydrograph ordinates are in m^3/s",/," "
12760     U1=DROUND((1/6),3)
12770     U2=DROUND((2/6),3)
12780     U3=DROUND((3/6),3)
12790     U4=DROUND((4/6),3)
12800     U5=DROUND((5/6),3)
12810     FOR K=1 TO Hydr_num
12820        U=K MOD 6
12830        U=DROUND((U/6),3)
12840        IF U=0 THEN
12845           PRINT USING 12570;Q_user(K-5),Q_user(K-4),Q_user(K-3),Q_user(K-2),Q_user(K-1),Q_user(K)
12846        END IF
12850        IF U=U5 AND K=Hydr_num THEN
12855           PRINT USING 12580;Q_user(K-4),Q_user(K-3),Q_user(K-2),Q_user(K-1),Q_user(K)
12856        END IF
12860        IF U=U4 AND K=Hydr_num THEN
12865           PRINT USING 12590;Q_user(K-3),Q_user(K-2),Q_user(K-1),Q_user(K)
12866        END IF
12870        IF U=U3 AND K=Hydr_num THEN
12875           PRINT USING 12600;Q_user(K-2),Q_user(K-1),Q_user(K)
12876        END IF
12880        IF U=U2 AND K=Hydr_num THEN PRINT USING 12610;Q_user(K-1),Q_user(K)
12890        IF U=U1 AND K=Hydr_num THEN PRINT USING 12620;Q_user(K)
12900     NEXT K
```

```
12910     ON E GOTO 13920,12920
12920     PRINT USING 12930
12930     IMAGE 4/,"Subcatchment data echo",/,22("-"),/
12940     PRINT USING 12950
12950     IMAGE "The data are set out in the following order for each subarea"
12960     PRINT USING 12970
12970     IMAGE 3X,"Subcatchment number, Drains to node, Area of subcatchment (ha)"
12980     PRINT USING 12990
12990     IMAGE 3X,"Percentage imperviousness (%), Overland flow length (m), Overland slope (m/m)"
13000     PRINT USING 13010
13010     IMAGE 3X,"Depression storage - pervious area, Depression storage - impervious area (mm)"
13020     PRINT USING 13030
13030     IMAGE 3X,"Initial infiltration rate, Final infiltration rate (mm/h)"
13040     PRINT USING 13050
13050     IMAGE 3X,"Roughness - pervious area, Roughness - impervious area (n)"
13060     FOR I=1 TO N3
13070        PRINT USING 13080;A0(I),N1(I),A1(I),P1(I),L1(I),S1(I),
13075        PRINT USING 13080;Deprperv(I)*1000,Deprimp(I)*1000,Infil_i(I)*C1,Infil_f(I)*C1,Mannperv(I),Mannimp(I)
13080        IMAGE /,3Z,3X,3Z,3X,4D.2D,3X,3D,3X,4D,3X,Z.3D,3X,2D,3X,2D,3X,3D,3X,3D,3X,Z.3D,3X,Z.3D
13090     NEXT I
13100!-------     -------CONDUIT-DATA-PRINTOUT------------------------------------
13110     PRINT USING 13120
13120     IMAGE /,"Conduit data echo",/,17("-"),/
13130     PRINT USING 13140
13140     IMAGE "The data are set out in the following order for each conduit"
13150     PRINT USING 13160
13160     IMAGE 3X,"Conduit number, Drains to node,","Type of cross-section,where:"
13161     PRINT USING 13162
13162     IMAGE 3X,"Compound channel                  =1",/,3X,"Natural channel                   =2"
13163     PRINT USING 13165
13165     IMAGE 3X,"Pipe with compound channel above =3",/,3X,"Pipe with natural channel above  =4"
13190     PRINT USING 13200
13200     IMAGE 3X,"Diameter & Roughness (n) of pipe or null for channel"
13210     PRINT USING 13220
13220     IMAGE 3X,"Roughness of cross-section & flood-plains(if they are)"
13230     PRINT USING 13240
13240     IMAGE 3X,"Number of sections, Slope (m/m), Length (m)",/
13250     FOR J=1 TO N4
13260        PRINT USING 13270;P0(J),N2(J),C(J),Diam(J),Mannpipe(J),N(J,2),N(J,1),N(J,3),Ns(J),S2(J),L2(J)
13270        IMAGE /,3Z,3X,3Z,3X,D,3X,2D.3D,3X,Z.3D,3X,Z.3D,3X,Z.3D,3X,Z.3D,3X,2D,3X,Z.3D,3X,4D
13280     NEXT J
13290     PRINT USING 13300
13300     IMAGE  4/,3X,"Description of the cross-section of each conduit"
13310     PRINT USING 13320
13320     IMAGE 3X,"(for pipes is the above the pipe cross-section)"
13330     PRINT USING 13340
13340     IMAGE 3X,"Data are set in the following order: "
13350     PRINT USING 13360
13360     IMAGE 3X,"Conduit number,number of coordinates "
13370     PRINT USING 13380
13380     IMAGE 3X,"at which nodes flood-plains begin(for natural channel=0)"
13390     PRINT USING 13400
13400     IMAGE 3X,"COORDINATES of the cross-section (X,Y)",/
13410     FOR J=1 TO N4
13420        PRINT USING 13430;P0(J),N10(J),C11(J,1),C11(J,2)
13430        IMAGE 2/,4X,"Conduit ",3Z,6X,3Z,3X,3Z,3X,3Z,/,4X,40("-")
13440        FOR I=1 TO N10(J)
```

```
13450           X12(I)=X11(J,I,1)
13460           X22(I)=X11(J,I,2)
13470           PRINT USING 13480;I,X12(I),X22(I)
13480           IMAGE 3X,DD," Coordinate",2X,3D.2D,3X,3D.2D
13490        NEXT I
13500     NEXT J
13510!------------------------------------------------------------------------
13520! CONNECTIVITY MATRIX
13530!------------------------------------------------------------------------
13540     PRINT USING 13550
13550     IMAGE //,"Connectivity matrix",/,19("-")
13560     B$=","
13570     PRINT USING 13580
13580     IMAGE /,15X,"Conduit",3X,"Contributing areas"
13590     PRINT USING 13600
13600     IMAGE 15X,7("-"),3X,18("-")
13610     FOR I=1 TO N3
13620        A$=VAL$(A0(I))
13630        FOR I1=I+1 TO N3
13640           IF I=N3 THEN 13690
13650           IF N1(I)<>N1(I1) THEN 13670
13660           A$=A$&B$&VAL$(A0(I1))
13670        NEXT I1
13680        IF I=1 THEN 13720
13690        FOR I2=1 TO I-1
13700           IF N1(I2)=N1(I) THEN 13740
13710        NEXT I2
13720        PRINT USING 13730;N1(I),A$
13730        IMAGE 16X,DDD,6X,K
13740     NEXT I
13750     PRINT USING 13760
13760     IMAGE /,15X,"Conduit".3X,"Upstream conduits"
13770     PRINT USING 13780
13780     IMAGE 15X,7("-"),3X,17("-")
13790     FOR J=1 TO N4
13800        A$=VAL$(P0(J))
13810        FOR J1=J+1 TO N4
13820           IF J=N4 THEN 13860
13830           IF N2(J)<>N2(J1) THEN 13850
13840           A$=A$&B$&VAL$(P0(J1))
13850        NEXT J1
13860        FOR J2=1 TO J-1
13870           IF N2(J2)=N2(J) THEN 13910
13880        NEXT J2
13890        PRINT USING 13900;N2(J),A$
13900        IMAGE 16X,DDD,6X,K
13910     NEXT J
13920     RETURN
13930!------------------------------------------------------------------------
13940!----------------SUBROUTINE PLOT------------------------------------------
13950     IF P=1 THEN
13951       DEALLOCATE Vel(*),Depth(*),Th(*)
13952     END IF
13960      IF Q7(P)<=.5 THEN
13970         Z1=.5
13980         GOTO 14220
13990      END IF
```

```
14000     IF Q7(P)<=1 THEN
14010        Z1=1
14020        GOTO 14220
14030     END IF
14040     IF Q7(P)<=5 THEN
14050        Z1=5
14060        GOTO 14220
14070     END IF
14080     IF Q7(P)<=10 THEN
14090        Z1=10
14100        GOTO 14220
14110     END IF
14120     IF Q7(P)<=50 THEN
14130        Z1=50
14140        GOTO 14220
14150     END IF
14160     IF Q7(P)<=100 THEN
14170        Z1=100
14180        GOTO 14220
14190     END IF
14200     Z1=Q7(P)
14210 !
14220     PRINT FNClear$;
14230     GRAPHICS ON
14240     GINIT
14250     GCLEAR
14260     WINDOW -(.15*T9),1.02*T9,-(.2*Z1),1.02*Z1
14270     CLIP 0,1.02*T9,0,1.02*Z1
14280     AXES .2*T9,.2*Z1,0,0
14290     CLIP OFF
14300     LORG 5
14310     FOR J=1 TO 5
14320        Z2=Z1*J/5
14330        T4=T9*J/5
14340        MOVE -(.11*T9),Z2
14350        IF Q7(P)<=100 THEN
14360           LABEL VAL$(Z2)
14370        ELSE
14380           LABEL VAL$(INT(Z2))
14390        END IF
14400        IF T9<100 THEN
14410           MOVE T4,-(.1*Z1)
14420        ELSE
14430           MOVE T4,-(.1*Z1)
14440        END IF
14450        LABEL VAL$(INT(T4))
14460     NEXT J
14470     MOVE -(.125*T9),.65*Z1
14480     LABEL "Q"
14490     MOVE .68*T9,-(.15*Z1)
14500     LABEL "T"
14510     X$=VAL$(X(P))
14520     MOVE .35*T9,.9*Z1
14530     LABEL USING 14540;X$
14540     IMAGE "Hydrograph in conduit ",K
14550     MOVE 0,0
14560     FOR K1=(T1/60) TO T9 STEP (T1/60)
```

```
14570        K=K1/(T1/60)
14571        IF Minor=1 THEN
14572          DRAW K1,Q8(2,K)
14573        ELSE
14580          DRAW K1,Q8(P,K)
14581        END IF
14590     NEXT K1
14600     CALL Dump_graphics(X2,X3)
14610     GCLEAR
14620     GRAPHICS OFF
14630     RETURN
14640     END
14650!
14660! UPPER CASE FUNCTION
14670!
14680 DEF FNUpc$(A$)
14690     INTEGER Ij
14700     ALLOCATE R$[LEN(A$)+1]
14710     R$=A$
14720     FOR Ij=1 TO LEN(A$)
14730        IF (A$[Ij;1]>="a") AND (A$[Ij;1]<="z") THEN R$[Ij;1]=CHR$(NUM(A$[Ij;1])-32)
14740     NEXT Ij
14750     A$=R$
14760     RETURN A$
14770 FNEND
14780!
14790 DEF FNClear$
14800     OUTPUT 2;"K";
14810     RETURN Dummy$
14820 FNEND
14830 SUB Dump_graphics(INTEGER X2,X3)                ! Arrays start at 1
14840     OPTION BASE 1
14850     IF X2=1 THEN
14860        DUMP DEVICE IS X3
14870        DUMP GRAPHICS
14880        GOTO 15280
14890     END IF
14900     IF X2=0 THEN
14910        CALL Graphics_dump(X3)
14920        GOTO 15280
14930     END IF
14940     INTEGER X_pixels,Y_pixels,Words_per_row,Row,Column,Index! Speed it up...
14950     DIM Pad$[45]                                ! Padding
14960!  IF NPAR=1 THEN                                ! Is output device specified?
14970     Dev_selector=X3                             ! If so, use it
14980!  ELSE                                          ! Otherwise,
14990!    Dev_selector=701                            ! Default to 701
15000!  END IF
15010     IF ABS(RATIO-1.31362467866)<1.E-9 THEN ! 512x390 pixels?
15020        X_pixels=512
15030        Y_pixels=390
15040     ELSE
15050        X_pixels=400
15060        Y_pixels=300
15070     END IF
15080     Words_per_row=X_pixels/16                   ! How many integers per row?
15090     ALLOCATE Hi$[Y_pixels],Lo$[Y_pixels]        ! High- and low-order bytes
```

```
15100      ALLOCATE INTEGER Screen(0:Y_pixels*Words_per_row-1)! Screen array
15110      Pad$=CHR$(0)
15120      FOR Ii=1 TO 44
15130         Pad$=Pad$&CHR$(0)
15140      NEXT Ii
15150      GSTORE Screen(*)                          ! Store the picture
15160      Esc$=CHR$(27)&"K"&CHR$((Y_pixels+45) MOD 256)&CHR$((Y_pixels+45) DIV 256)
15170      OUTPUT Dev_selector USING "K";CHR$(27)&"A"&CHR$(8)
15180      FOR Column=0 TO Words_per_row-1
15190         FOR Row=Y_pixels-1 TO 0 STEP -1
15200            Index=Column+Row*Words_per_row
15210            Hi$[Y_pixels-Row]=CHR$(INT(Screen(Index)/256))
15220            Lo$[Y_pixels-Row]=CHR$(Screen(Index) MOD 256)
15230         NEXT Row
15240         OUTPUT Dev_selector USING "K";Esc$&Pad$&Hi$
15250         OUTPUT Dev_selector USING "K";Esc$&Pad$&Lo$
15260      NEXT Column
15270      PRINT CHR$(27);"2";
15280   SUBEND
15290   SUB Graphics_dump(INTEGER Device_selector)
153001   (FILE "82905DUMP")  Dumps Graphics to HP82905B printer
```