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By D Stephenson

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Water Systems Research Group
University of the Witwatersrand
Johannesburg, 2000, South Africa

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Report No. 11
1992

Report to the Water Research Commission on the Project
"Effects on Urbanization on Catchment Water Balance"

Head of Department
and Project Leader : Professor D. Stephenson

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WATER RESEARCH COMMISSION

EFFECTS OF URBANIZATION ON CATCHMENT WATER BALANCE

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The rapid development of physically-based hydrological information and modelling systems has necessitated enhanced data entry and display systems. A 'mapping tool' is developed for the manipulation and display of spatial information, which is a cost-efficient, self-contained utility system that is suitable for use on micro-computers. It has the ability to be integrated as part of any modelling or information system. Diverse applications using the 'mapping tool' are briefly described including resource management systems for planners, mass balance studies in urban catchments and data entry systems for physically based models.

A vector and raster graphic mapping tool on micro-computers

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INTRODUCTION

The point source concept of measurement has been with us for centuries and still dominates data gathering techniques. Most geophysical processes are however, spatially variable. A practical solution is therefore to produce a 'grid' of point source measurement. Coupled with problems related to the spatial nature of the process being modelled is the need for development of efficient, user-friendly methods for the entry and display of the information.

The mapping of distinct features (e.g., roads, rivers, and other cartographic items) has, prior to automation, been annotated on maps using manual techniques. Development of automated mapping procedures originated through the direct translation of the manual methods. The present trend, however, is towards original systems which will directly produce the required output. In order to extend the abilities of these mapping systems, programmers have added data base systems capable of storing vast amounts of geographical information, hence the concept of a Geographical Information System (GIS) was born. The ability to store information derived from satellite imagery constitutes the most recent development stage.

Wiltshire et al. (1986), used digital capture techniques to speed up collection of catchment basin characteristics. These characteristics were used in multiple regression equations for the estimation of mean annual floods using the UK Flood studies report (Natural Environment Research Council, 1975). These techniques were not incorporated into one integrated system. Instead the various functions were executed on different hardware configurations.

Research that is being undertaken in several areas prompted the development of a computer graphics display utility. These research topics range from water resource planning (on a basin scale) to studies of the effects of urbanisation on water balance (on a small catchment scale). In this paper, we describe a tool that can be used as an effective method in spatial data manipulation and display.

GEOGRAPHIC INFORMATION SYSTEMS

Computer Aided Design (CAD) packages have been found to be restrictive when used for database linked mapping applications. This led to the development of the Geographic Information System (GIS).

The major benefits of a GIS are the ability to establish a large database of spatially (geographically) referenced data and the recall and manipulation of data as required. Many commercial Geographic Information Systems (GIS) exist, but there is a dearth of systems that can easily be incorporated into 'user designed' analytical structures and that are also cost effective for small organisations.

Some of the more prominent GIS systems include Arc/INFO, SICAD and INTERGRAPH (REGIS). These systems all exhibit very powerful GIS features and suffer from various problems. For example Arc/INFO does not have a continuous map ability and lacks one-to-many and many-to-one relationships with feature information. The INTERGRAPH system does not allow for redundancy-free modelling of complex topologies and the SICAD GIS has a complex interface which requires customising for the specific application.

All the GIS systems suffer from the following weaknesses (Clark et al., 1987):

- interface to non-proprietary engineering programs difficult
- unfriendly user interface
- directly accessible engineering programs limited and weak
- cost to small organisations and developing countries prohibitive
- extensive training required and customising is necessary.

In the development of the mapping utility described in this paper the object was to overcome the problems detailed above. Specific emphasis was given to the integration with existing, well tested engineering programs, and the cost and training required for developing regions to employ the technology within the restriction of a limited budget.

DEVELOPMENT

It is important to consider the nature of the data as this will affect both the method of data capture and the storage of coordinates. Beran (1982) suggested that geophysical data could be classified into various categories. In this paper we consider three classes of data.

1. Continuously varying data which is usually expressed as isolines (e.g., altitude and rainfall).
2. Continuous feature information (e.g., river channels, roads and railways).
3. Constant value information which is expressed on maps as a patchwork (e.g., forests, vegetation cover and soil types).

Figure 1 shows the three types of graphical information to be stored. Classes 1 and 2 were captured and stored as vector information whereas class 3 was stored as raster datasets.

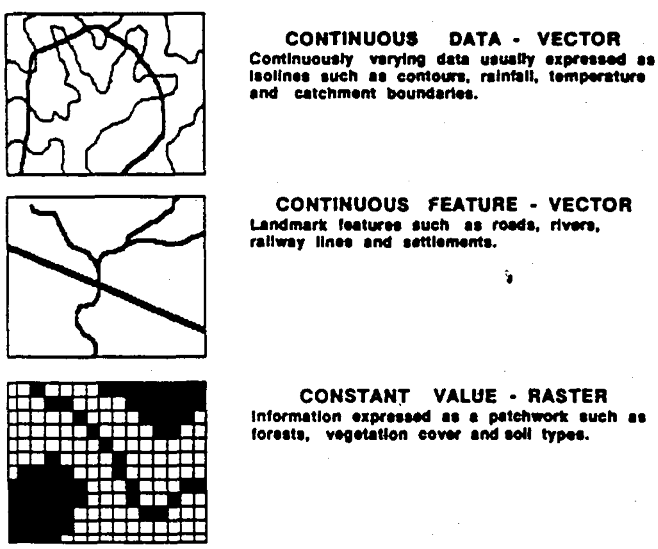

Figure 1. Classes of graphical information captured.

The scale of the map originally digitised will have a pronounced effect on the level of sensitivity of localised anomalies. This is apparent when maps of different scales are digitised into the same filing system. The digitised information is, of course only as good as the source maps. Brown and Fuller (1985) found that source maps compiled by different countries had different levels of detail especially with reference to river channels. They used two different map scales to overcome local features and thus produced two different map databases.

The mapper utility described in this paper has the ability to establish separate databases for each of five default scales. These scales were chosen to cover the full spectrum of possibilities and for the ease of map availability.

APPLICATIONS

(a) *Resource displays*

The planning and analysis of water resource systems has in the past been carried out on an *ad hoc* basis, each project being evaluated as a stand alone system. No facility existed whereby the planner could obtain a clear overview of all facets of the problem at hand. Due to the increased awareness of the limits of our natural resources in relation to the growth of the population, it has become essential that a more comprehensive methodology be developed to assist the planning professions.

A stand-alone utility (tool) was therefore required which could, graphically, represent the resources available, on a map of the planning region. In order to display social, economic and demographic conditions and needs, this tool would have to be independent of units and have the ability to interpret a variety of data types.

The first step in the process of studying a particular region is the capturing of the physical map. This follows the standard procedure for any digitising process with the required detail being captured on layers defined by the operator.

Typical layers which are used for water resource analysis problems are:

1. Catchment boundaries
2. Roads
3. Railway lines
4. Settlements and their names
5. Dams and lakes
6. Main rivers
7. Tributaries
   a. Underground lakes
   b. Underground streams, if available
   c. Dykes and aquifers
8. Contours
9. Overlay grids ($\frac{1}{4}$ or $\frac{1}{2}$ degree intervals).

The ability to specify which of the information items and/or layers should be displayed at any one time is possible using the map utility described. It is feasible therefore to display the population densities and overlay the available water supply data. This immediately indicates the areas requiring increased investment in water supply orientated projects. The analysis section is totally interactive. It is therefore possible to erase the population densities and superimpose the ground water potential in order to assess the possibility of using the aquifers as a potential water supply source to satisfy any shortfalls.

As an example, the Mbashe river drainage basin, situated in the Republic of Transkei, was used to demonstrate resource mapping with the graphics utility. The map was digitised in at a scale of 1:500,000. *Figure 2* shows bar graphs at the bottom of each sub-catchment superimposed on the basic map of the Mbashe basin.

Note that not all layers are 'switched' on for the benefit of clarity on a monochrome printout. The graphs represent various categories of river flow, for example average monthly and cumulative monthly. On the screen each bar is represented in a different colour for each location.

It is easy to see the benefits derived through this graphical representation. Persons of various professions and educational background are all equally able to interpret and understand the size of the relative bars and therefore the relationship between the flows from the various sub-catchments. The data can also be used as input to more complex analysis procedures.

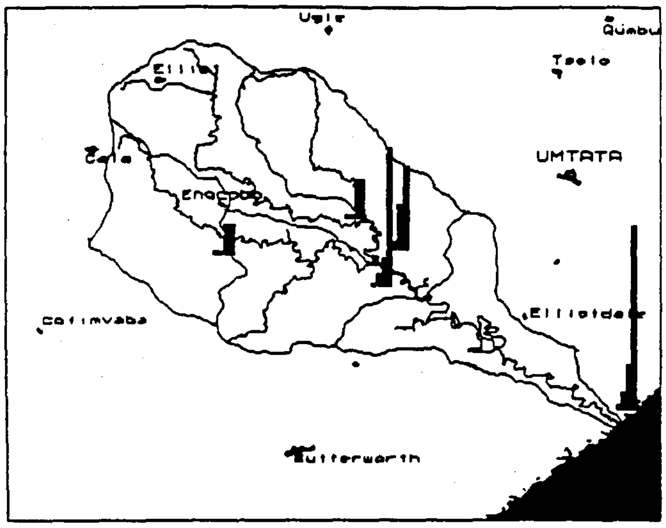

Figure 2. Resource mapping application - River flows on the Mbashe Basin, Republic of Transkei.

(b) *Mass balance*
The determination of a water balance within a catchment shows the overall performance of that catchment and the interaction of the individual mass fluxes. Particular attention has been applied to the determination of the effect of urbanisation and the resulting changes in the water balance equation. The demand for housing, coupled with the increased migration of the population to the towns and cities (particularly in South Africa and other developing countries), has prompted the study of urban catchment water balances.

Most water balance studies in the past (e.g., Grimmond and Oke, 1986; L'vovich and Chernogayeva, 1977; Aston, 1977) assumed a lumped catchment, whereas the spatial nature of the water balance should also be considered. A small ($1 km^2$) urbanised catchment is used as an illustration. This catchment is situated in Sunninghill Park, a suburb on the northern outskirts of Johannesburg, South Africa. The sizes of each individual property are approximately $1,000 m^2$. The catchment is predominantly low density residential.

The following information was digitised at a scale of 1:10,000 into the computer and stored on disk using the graphics tool:

- Contour levels (to produce a digital elevation map - DEM)
- Roads
- Stand or property boundaries
- Land use (houses, swimming pools, and vegetation cover)
- Soil types
- Defined stream channels
- Catchment boundary.

A water balance module was developed incorporating the graphics system to analyse the monthly water balance. Inputs to the water balance contained the following monthly time series:

- Rainfall totals - these were collated from five sites in the catchment and a digital rainfall map produced.
- Runoff totals at the outlet of the catchment was distributed thoughout the total catchment area.
- Domestic clean water supply to each stand/property.
- Sewer discharge at the outlet of the catchment was distributed throughout the total catchment area.
- Temperature and other meteorological parameters.

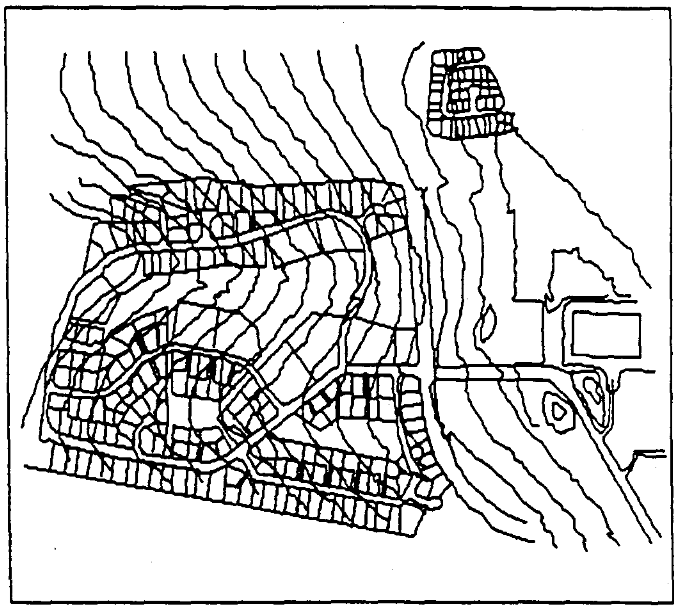

Figure 3. Water balance application - Road and stand feature for the Sunninghill Park Catchment, Republic of South Africa.

Other input data comprised parameters of infiltration and evaporation models which were related to the type of vegetation and soils. *Figure 3* shows road and stand features for the Sunninghill Park catchment.

Simple water balance calculations were performed for each pixel element of the catchment area using both the graphics information and the input data. A simplification which is deemed justified in view of the monthly time scale involved is the absence of routing in the model. A composite map of the water balance variation (stored as layers) for each month was also incorporated as a system call to the graphics routines.

(c) *Others - e.g., stream lengths, area of catchment*
With the development of hydrological simulation models becoming more physically-based (i.e., the parameters to the model can be determined by physical measurement), certain parameters may be determined from maps. Items such as slope, the length of streams, and the area of catchments are perhaps the most obvious.

Models such as TOPMODEL (Beven and Kirkby, 1979) and unit hydrograph analysis involve the entry of numerous topographic data values that can be derived from a map. The graphics sub-system can be used to enter the relevant map's information and an application can then be added comprising the simulation model and a series of routines to interpret the map information. The graphics utility will therefore act as an intelligent front end for the capture of data for simulation programs.

DISCUSSION
The package has been designed to fulfil a planning need within developing regions, possessing limited technical man

power and skills. The system is user-friendly and requires a negligible training period when compared to conventional CAD systems.

It is the intention of the authors that the graphics utility developed in this paper be incorporated into a variety of applications packages. The tool provides the operator with the ability to rapidly interpret data which would otherwise require a detailed analysis of tabular data. The utility is compiled in a stand-alone format and *Figure 4* gives an indication of a typical integration configuration with an independent modelling package.

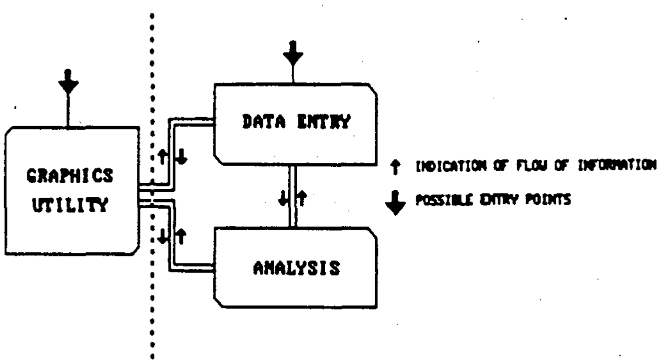

Figure 4. Illustration of a typical configuration with an independent modelling package.

A variety of applications have been identified. The descriptions of its use in the resource mapper and the mass balance problems, illustrates the emphasis on spatially variable data. As the simulation models being developed become more complicated in their analysis techniques, it is important that the data used matches the new levels of complexity. It is, at present, still the norm to use point data as input to simulation models. This should no longer be considered acceptable.

The availability of accurate and up to date maps allows any user to access and input spatially variable data into his model, if the model can handle it. This would normally be carried out using digitising techniques, such as those incorporated in the graphics utility. For the more sophisticated user the ability to download and analyse satellite imagery is already a viable option.

The tools described in this paper makes use of a stylus operated digitising platen. Gross (1989) describes the implementation of a monochromatic video camera and image grabber board. This system is able to 'digitise' black line images. In the context of this paper, the ability to view and enter different types of information using colour images is paramount. Image processing software would be needed which would allow adjustment of the different painting shades of map technology. Colour video cameras and grabber boards are available together with image processors, but in developing regions these devices are considered too costly at present.

The level of data density and accuracy achieved by these methods is far superior to the conventional point estimation methods. In addition, availability and the ability to interpret, display and manipulate spatial data (using the graphics utility), will, we hope, lead directly to the development of simulation models which have a greater ability to utilise spatial data.

The graphics utility developed appears to satisfy the major requirements of a utility of this nature. These include:

- Versatility – the tool has been shown to be highly adaptable. It has been applied to resource mapping in a basin wide context and was equally well suited to a mass balance analysis on a small catchment.
- Integration – it is possible to integrate the tool with any package as it is a stand alone utility.
- Ease of use – the entire utility is very user-friendly with menu-driven options run through the keyboard, a digitiser or a mouse.
- Compatibility – the utility was designed in order to be compatible with the most common hardware configurations. It was also specifically designed to run on medium speed, low memory micro-computers and peripherals which can be used for other tasks.

The graphics utility developed here will, no doubt, be found to be lacking in certain areas by specific users. It was never the intention to develop a tool which would satisfy all needs and it is not considered to be a replacement for any of the CAD packages available. We hope merely to have provided the engineer, planner or any other decision maker, with the means to quickly and simply, be able to capture and display spatially variable data, using the hardware configuration which is already at his disposal.

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SELECTION OF STORMWATER MODEL PARAMETERS

By D. Stephenson,[1] Member, ASCE

ABSTRACT: Stormwater models, which are used extensively for stormwater drainage design and management, require calibration. The parameters usually estimated by calibration include infiltration and roughness. Parameters that can be determined by measurement include catchment area, slope, and length of flow path. The dependency of parameter values on the level of model discretization was investigated. Experiments were performed on a small urban catchment, and results of different levels of discretization were compared to observed hydrograph volume and peak runoff. The only factor found to require adjustment based on the level of discretization and the size of subcatchment was the overland flow time. Small-scale discretization required a longer flow path or a higher Manning roughness coefficient than coarse discretization to predict lag times correctly.

INTRODUCTION

Computer models have become the most suitable method of estimating stormwater flows in urban areas. The accuracy of hydrologic estimates is improved by models that can accommodate a large number of elements and variables. Other reasons computer models are used include the ease of changing parameter values and performing sensitivity analyses, plus the simplicity of comparing alternative designs (Alley et al. 1980; Cousens and Burney 1976; Hughes and Beaten 1987).

The accuracy of computer simulation models is dependent on suitable calibration of the model (Lane and Woolhiser 1977; Pilgrim 1975; Zaghoul 1981). Some easily measurable parameters, such as catchment surface area and rainfall intensities, are not obtained by calibration. For example, the length of overland flow is often measured directly from a map by scaling the length of the longest water course. Other parameters are often obtained indirectly. Thus, infiltration rate is often not based on physical measurements but on calibration of the model. That is, different infiltration rates or percentage impervious cover are used until the computed volume of runnoff equals that observed for a selected storm event. Similarly, the surface roughness coefficient is adjusted until the shape and time to peak of the hydrograph agree with those observed.

The length of flow path, the roughness of the conduit or the overland flow surfaces, and land slope are usually interdependent; adjustment of only one of them for any particular conduit or plane will probably suffice for the model to reproduce the observed hydrograph with sufficient accuracy.

It is possible to adopt a fairly coarse discretization by averaging over a large area and adjusting parameters to optimize the model results. In these so-called lumped parameter models it is often not necessary to distinguish the impervious from semi-pervious surfaces, nor, in particular, to separate directly connected impervious surfaces from those surfaces that discharge

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onto permeable surfaces. Often a balanced or sufficiently accurate estimate of surface abstraction can be achieved by varying the infiltration rate of an assumed homogeneous underlying soil or aquifer. The fact that some water may infiltrate at one point and emerge down the plane is often not correctly accounted for. There are various approaches to simplify modeling this phenomenon. For instance, the stormwater drainage engineer is inclined to separate surface flow from subsurface flow and to consider only the direct runoff. The modeler of large rural catchments will indirectly account for the so-called interflow by adopting different calibration parameters.

Although the hydrologic type of model is often envisaged as a black box or empirical or analogy model, such as the Cells model (Diskin 1978), there are in fact many similarities between hydrologic models and hydrologic or physical models. Sophisticated hydrologic models may include unit hydrographs or simulated unit hydrographs based on Fourier series, such as the Hymo Model (Wisner 1980). These models are usually 'lumped variable' models, i.e., the catchments or subcatchments used are relatively large, and in some cases they include various types of soil cover and varying topography.

On the other hand, the smaller the subcatchments become, the more the model of choice tends to become a hydraulic-type model. Hydrologists may call a hydraulic model a 'distributed parameter' model because it is able to accommodate a number of different values for the same parameter in successive subcatchments. The smaller the size of the subcatchments relative to the whole, the greater is said to be the level of discretization (James and Robinson 1981). This is not to be confused with the degree of 'lumping' or averaging of parameters, however.

The hypothesis tested in this paper is that the smaller the subcatchment area, the closer one can approximate a physical model where all the parameters can be measured directly. That is, it is postulated that a sufficiently fine level of discretization is theoretically possible wherein the parameters used are those for which values could be physically measured in the field or obtained in the laboratory.

Differences between Hydrologic and Hydraulic Parameters

The values of parameters quoted for stormwater models, either in user's manuals (Green 1984) or in calibration studies, are often significantly different from those one would expect from hydraulic-type approaches or laboratory measurements. Infiltration tests in the field or laboratory invariably indicate infiltration rates higher than are normally used in stormwater models. Even with fairly accurate soil-physics-based models such as that of Green and Ampt (1911), which is still a simplified model, there are a number of factors to be estimated before the model can be calibrated. The permeability, initial moisture content, and soil suction at least are required. The Horton model (1933), a more empirical infiltration model, requires an initial infiltration rate and a final infiltration rate; it also assumes the field infiltration rate will decrease exponentially from one rate to the other. The antecedent moisture condition is often difficult to account for with this process, however, and the empirical model is more a surface water abstraction model than one that would also indicate the increase in water in the underlying aquifer.

Average infiltration rates over the area are often used for each subcatchment in a model so that such averaging may also include the effect of directly or indirectly connected impermeable areas. These areas could range from paved and roofed areas to unsurfaced roadways. Directly connected impervious areas, however, add considerably to the runoff even if the proportion of impermeable area is small. For example, if the infiltration rate into the natural soil were 90% of the precipitation rate, then if 10% of the surface were covered by directly connected impermeable pavement, the runoff rate would effectively double. This fact may be a reason why the infiltration rate used in models which have been calibrated is invariably lower than would be expected from laboratory and field infiltrometer tests.

If one were to account for infiltration actually occurring on a permeable surface downstream of an impermeable surface, then a large proportion of the runoff from the impermeable surface would be lost. In fact, with scientific estimation, very little of the runoff from unconnected impermeable surfaces would be expected to appear as direct runoff during an average storm on watershed with moderate slopes. In estimating the area of impermeable surface on a lumped catchment, then, there should be an important distinction made between directly connected and indirectly connected permeable surface.

Another parameter often assigned considerably different values in hydrologic models in comparison to figures quoted in hydraulic literature is the roughness coefficient. The Manning equation for velocity, which is written in terms of a friction factor, seems to be the one used most frequently; and the following sections will refer to the Manning roughness coefficient. Whereas values quoted in hydraulic literature for artificial surfaces can vary from 0.015 to 0.035, values used in stormwater models may range from 0.05 to 0.5 for planes. These values are higher than those that would normally be conceived in hydraulic research, and therefore they must account for more than just hydraulic factors. Some of the reasons that higher Manning's roughness numbers are required in hydrologic models are indicated below.

Overland flow is generally shallower than channel flow and therefore its Reynolds number is lower. Whereas the Manning roughness coefficient is generally assumed independent of Reynolds number, the Darcy Weisbach friction equation indicates an increasing roughness with reducing Reynolds number. For example, in a channel the Reynolds number may be $R = vy/\nu \cong 3 \times 1/10^{-6} = 3 \times 10^{6}$. In an overland flow situation R may be closer to $0.5 \times 0.01/10^{-6} = 5 \times 10^{3}$. The Darcy Weisbach friction factor for the two situations would be 0.01 for the channel and 0.04 for the overland flow situation (assuming identical Nikuradse roughness, i.e. 0.02 mm).

The factor of 4 difference between the friction factors may not be sufficient to account for the difference in predicted flow for some cases, but there are other factors that might. The effect of raindrops on overland flow, for example, is to increase turbulence and therefore create higher headloss (Overton and Meadows 1976).

Another factor that could require higher roughness factors in the hydrologic-type model is the fact that runoff is often not direct. Overland flow lengths are invariably scaled from a map. On the ground, however, water will follow a considerably longer path than that measured on a map. Water flowing over a roof will turn through 90° a number of times, thus increasing the flow length by 40% (the difference between two sides and the hypotenuse

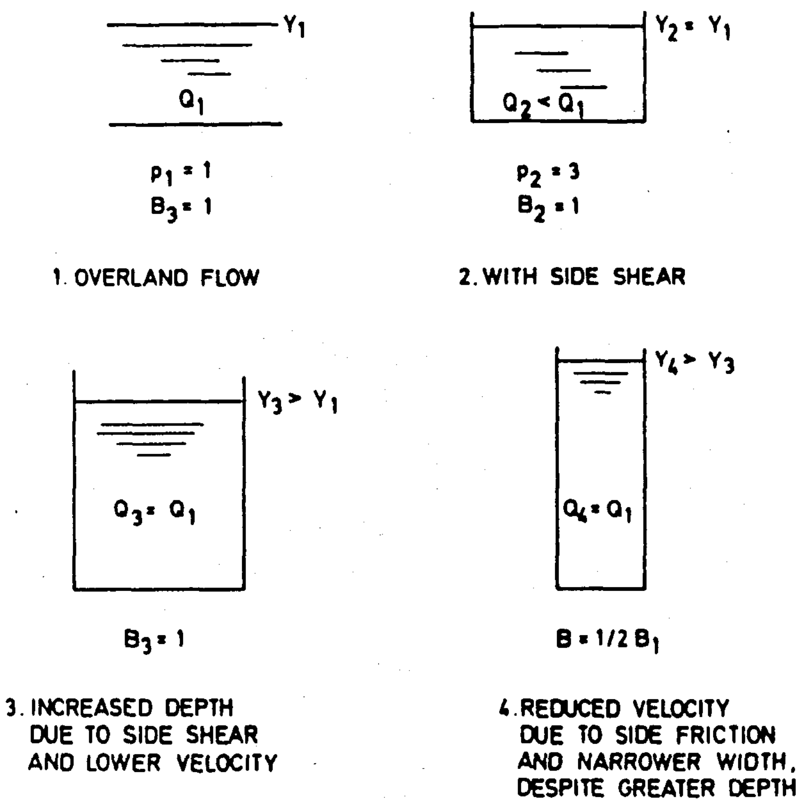

FIG. 1. Effect of Channelization and Rills on Flow Depth and Velocity

of an equilateral right triangle). In addition, downpipes and other vertical flow paths are not measured from maps. There may also be storage due to ridges and obstructions reducing the flow velocity. Then over natural ground there may be obstructions such as lumps of ground, stones, and vegetation, so that the actual flow path could easily exceed twice the length measured with a scale ruler. The length of flow path is usually used together with the Manning roughness to estimate travel time. One could therefore either increase the overland flow length or the Manning roughness coefficient to correct the lag time. A longer overland flow path also means that the longitudinal gradient is less for the same drop in elevation from one end of the catchment to the other, thus also adding to the travel time.

A third factor that could increase travel time in some instances and reduce it in others is the fact that water flowing over land does not flow as a sheet in most cases but forms rivulets in rills. Trickles of water wind their way between obstructions such as stones, bricks, and vegetation. There will be preferential flow paths, so that the flow will not be over the entire width of the plane. There exist two compensating factors in the flow-time calculation, however. One is the fact that the reduced effective width of the catchment would mean greater flow depth and therefore higher flow velocities. The concentration time is related to the velocity as indicated by the kinematic concentration time equation, namely

$$c = m\alpha y^{m-1} \quad (1)$$

where c = the wave speed; α = the conveyance; y = the flow depth; and m = an exponent that is 5/3 in the Manning equation. Also flow velocity $v = \alpha y^{m-1}$.

On the other hand, the fact that the water flows in rills rather than in sheet form means there is additional drag on the water due to the sides of the channels or rivulets.

Fig. 1 shows the effect of chanelization. Employing Manning's equation

$$Q = \frac{\sqrt{s}}{n} A \left(\frac{A}{p}\right)^{2/3} \quad (2)$$

the velocity for case 2 (with sides) and $y_1 = y_2$ is less than for case 1 (overland), so for case 3 ($Q_3 = Q_1$) the depth increases compared to case 1. By reducing the effective width (case 4), the depth increases further but the velocity reduces compared to case 1. The net effect is that the velocity reduces if the water flows in rills rather than in sheets, despite the increased flow depth in the rills.

Model Study

Kerst (1987) modeled an urban catchment (Sunninghill) north of Johannesburg. He looked at four levels of discretization, referred to as coarse, medium, fine and very fine. The corresponding average flow path lengths were 115 m, 47 m, 15 m, and 7 m. Maps for the coarse and very fine studies are given in Figs. 2 and 3.

The model used was WITWAT (Green 1984). This model uses kinematic theory for overland flow and time lag for conduits (Stephenson and Meadows 1986). It can be run in design mode with local regional hydrologic parameters, or analysis mode with applied hyetographs. It was in the latter mode that the program was run for the results presented here.

Description of Field Observations

The catchment studied is a portion of a 2 km^2 area monitored under a research contract to investigate the effects of urbanization on catchment water balance. A network of five autographic rain gauges existed, and detailed maps were available. The area isolated for this study was 0.74 ha, comprising four residential properties with single story single family houses.

The surface flow from the study catchment is discharged into a road gutter and thence into subsurface stormwater drain pipes. Measured water depths in the gutters and an assumption of uniform flow (the Manning equation) were used to estimate flow rate. Flow depths were measured at five-minute intervals for the observed flow events for which the observer was able to reach the catchment in time. The maximum flow rate event that occurred on December 20, 1986 was chosen for analysis and simulations. The storm duration was 3,000 secs, with a depth of 262 mm and peak intensity of 120 mm/h. Rain was measured by a tipping bucket rain gauge with a tip of 0.2

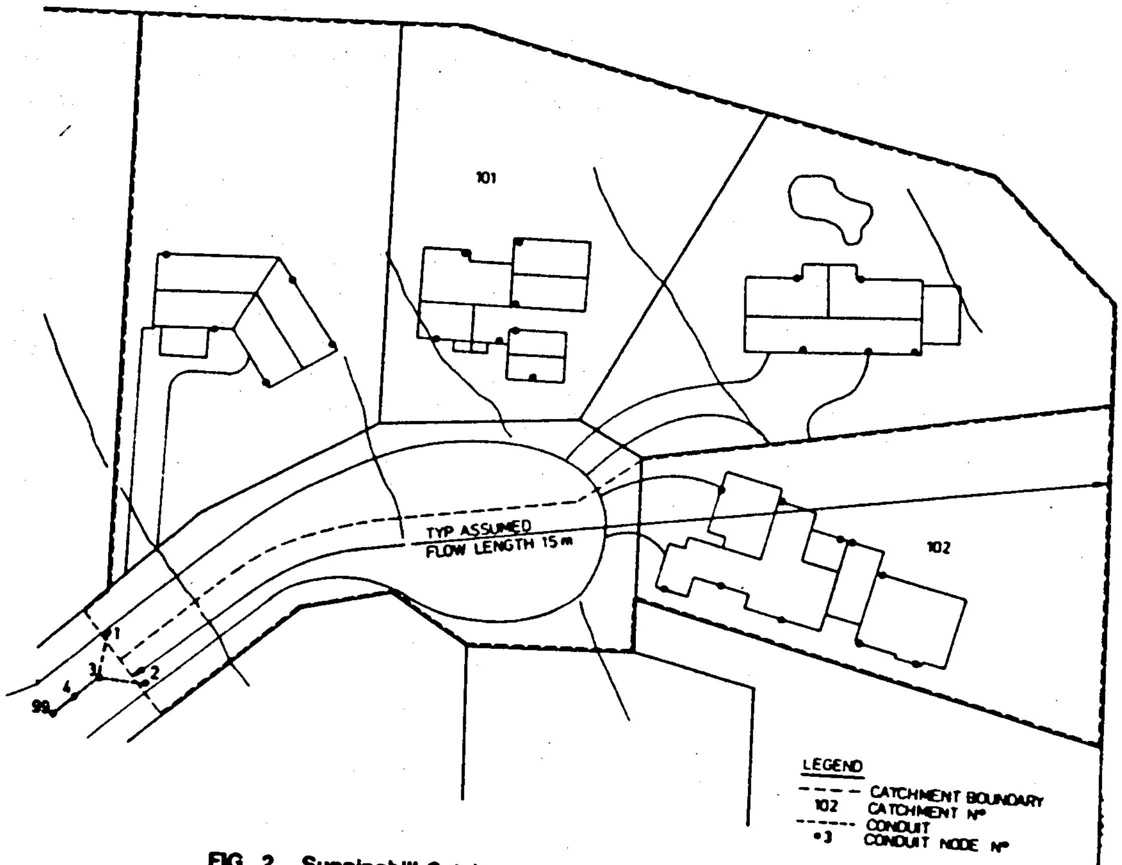

FIG. 2. Sunninghill Catchment—Course Level of Discretization

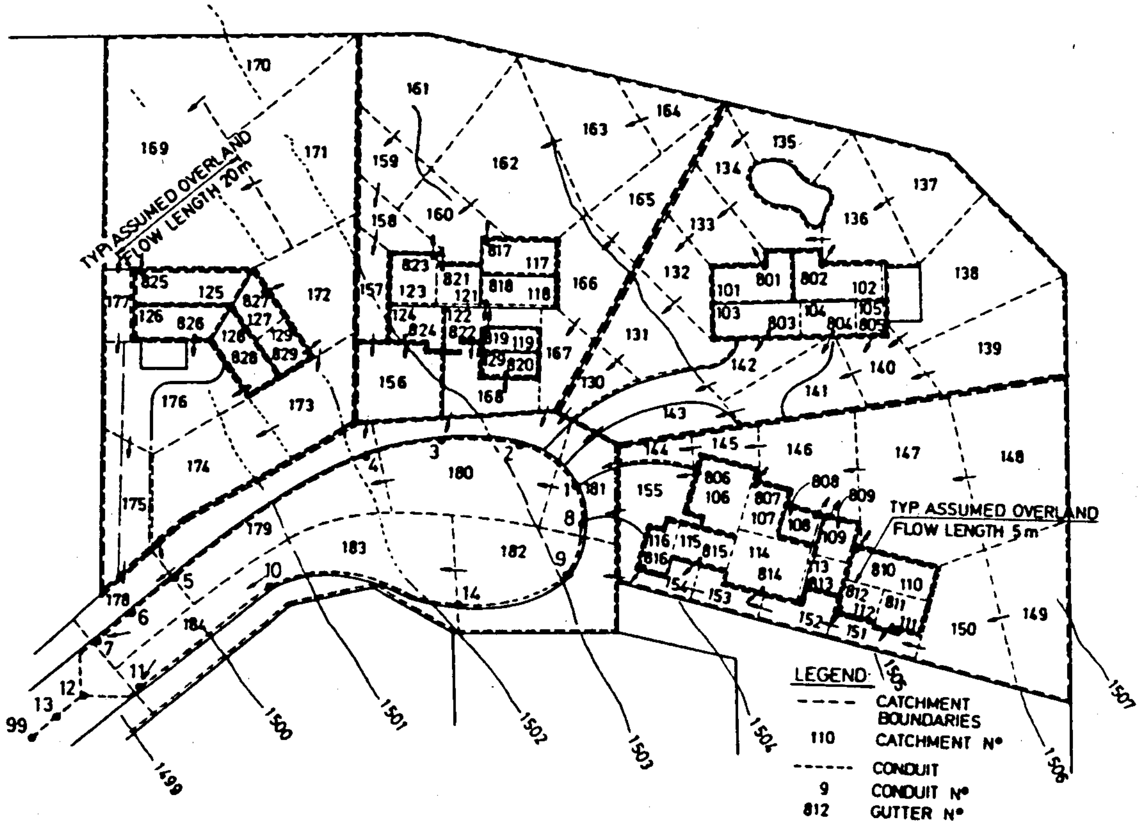

FIG. 3. Sunninghill Catchment—Very Fine Level of Discretization

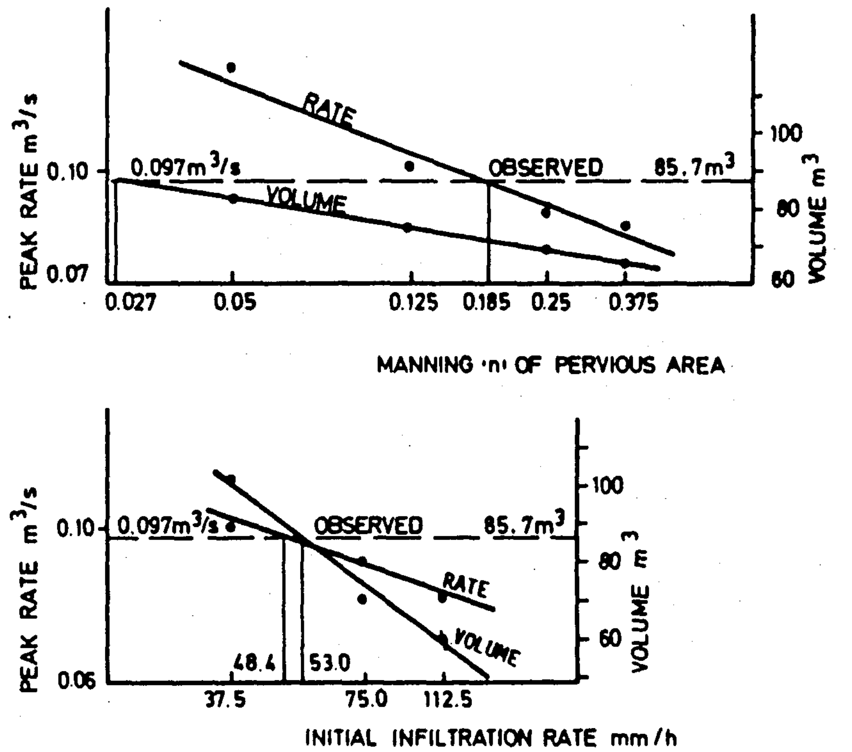

FIG. 4. Method of Calibrating Coarse Level Model for Manning Roughness Coefficient and Initial Infiltration Rate

mm. The data were collected on EPROMs (erasible programmable read-only memories) and processed by personal computer. The model was fitted by calibration against the observed storm with values of Manning roughness for impervious area (0.015) and pervious area (0.25), initial infiltration rate (75 m/h), ultimate infiltration rate (10 m/h), and depression storage for impervious area (1 mm) and pervious area (3 mm) averaged over the four levels of discretization. A computational time step of 60 s was found to be the most efficient. Each parameter was then varied methodically and the computed values of peak flow and volume of runoff were compared to measured values. Fig. 4 shows how the fits were obtained for the Coarse discretization case.

Fig. 5 presents the results for coarse, medium, fine and very fine discretization. It shows that the results are most sensitive to infiltration rate. A 50% variation in infiltration rate affects the peak runoff by up to 20% (for medium discretization) and affects volume of runoff by up to 30% (for fine discretization). The next most sensitive parameter was roughness of pervious area. A 50% variation alters runoff peak by up to 20% (for medium discretization), but volume is affected by only up to 7% (all levels of discretization). Other factors being equal, the more finely discretized model reproduced runoff values better than coarser models, but the coarse models

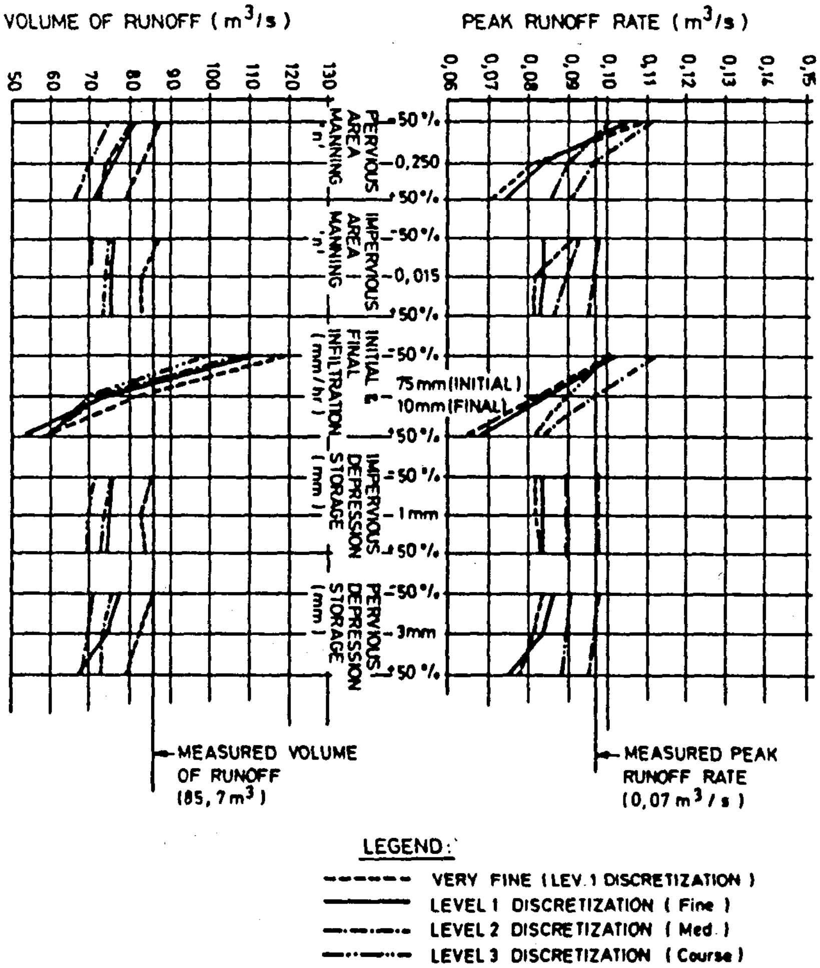

FIG. 5. Comparison of Peak Runoffs and Volumes of Runoff for Various Levels of Discretization and Time Steps

reproduced peak runoff rate better. The resulting two values of roughness for peak rate and volume prediction were closest for fine level discretization.

Fig. 6 and Table 1 show the effect of level of discretization on best-fit parameters as obtained by calibration. The values were obtained from Fig. 4 and similar graphs for other levels of discretization. The Manning roughness coefficient must be increased slightly with finer discretization, or the length factor must be increased, for correct prediction of peak runoff rate. Length factor is a number by which overland flow path length as measured on a map must be multiplied to obtain representative model results. Volume is less sensitive to variation in *n*.

Effective infiltration rate appears insensitive to the level of discretization, with slightly better volume prediction for finer levels of discretization.

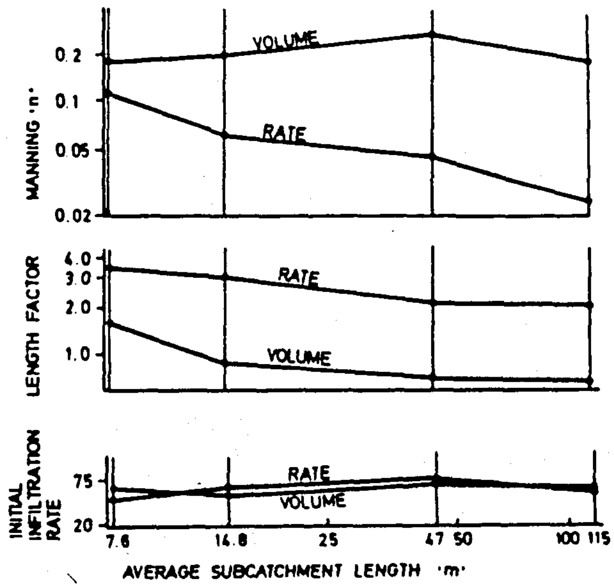

FIG. 6. Average Subcatchment Length versus Best Fit Manning *n*, or Length Factor, and Initial Infiltration Rate

TABLE 1. Best Values of Parameters at Different Levels of Discretization

MANNING *n* PERVIOUS: Initial Infiltration = Standard Value Length Factor = 1.0		INITIAL INFILTRATION RATE *f* (mm/hr): Manning *n*-Pervious = Standard Value		LENGTH FACTOR: Initial Infiltration Rate and Manning *n*-Pervious = Standard Values		
Rate (1)	Volume (2)	Rate (3)	Volume (4)	Rate (5)	Volume (6)	Level of discretization (7)
0.170	0.110	45.2	64.1	3.5	1.6	Very fine
0.186	0.061	59.7	47.9	3.0	0.9	Fine
0.260	0.048	73.0	61.6	2.1	0.7	Medium
0.185	0.027	48.4	53.0	2.0	0.65	Coarse

Conclusions

Stormwater parameters that cannot be measured directly are infiltration rates and subcatchment roughness. The runoff volume is most sensitive to the infiltration rate assumed, and the peak flow rate is most sensitive to roughness assumed.

In general, the most finely discretized model predicted runoff volumes more accurately than coarser models, whatever the values of the parameters. Peak flow rates were better predicted by coarser models. As the level of discretization is increased (subcatchments become smaller), the effective infiltration rate must be increased slightly to assure the correct runoff volume. This necessity could be because better account is made in finer discretization of directly connected impermeable areas; i.e., there is less loss from directly connected areas, but this may be lost in a "lumped" catchment.

Contrary to expectations, the smaller the subcatchment size, the bigger the Manning *n* that was necessary, or else the effective overland flow length had to be increased. Thus, the effect of longer total flow paths is in finer discretization masked by the fact that flow contributions are made at intermediate points along catchments, instead of at the top end of longer subcatchments. That is, the center of gravity of the storm contributions from roofs or other impermeable surfaces in effect is moved nearer the outlet from the catchment.

The hypothesis that finer discretization results in more acceptable (hydraulically) parameters is thus true for infiltration but not for roughness. Effective catchment length or Mannings *n* must be increased the finer the level of discretization, which may be accounted for by lower Reynolds number however. It thus appears a fine level of discretization is not always justified and may not improve accuracy in stormwater modeling.

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Determination of runoff frequencies for ungauged urban catchments

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Abstract

Engineers often intuitively use the recurrence interval of a design storm for runoff recurrence interval. It is suggested that this assumption is not soundly based, as the antecedent moisture content (AMC) or catchment wetness has a significant effect on the storm runoff recurrence interval.

A method is described for generating flood frequency information for a catchment for a design situation. The technique comprises a combination of deterministic and stochastic components. This involves deterministically modelling the response of the catchment together with the use of a stochastic element to derive the conditional probability vector of the outlet hydrograph peaks. The advantage of the method is that the infiltration is modelled using a range of values that is described statistically.

Introduction

The recurrence interval, or the risk of exceedence, of storm runoff requires assessment in order to avoid over or under-design of drainage facilities. Design flood estimation on small catchments, for which streamflow data are generally not available, is commonly based upon the use of design rainfall data together with a hydrological simulation model. There are various methods available to the design engineer for deriving the runoff from an applied design storm. These methods in order of simplicity range from the Rational method (Kuichling, 1889) to the distributed models (e.g. SHE model, Abbott *et al.*, 1986) and in hydraulic accuracy from Time-area to Kinematic methods.

The design engineer is usually interested in the recurrence interval of the peak runoff rate of the storm event as this indicates the risk of failure of the structure being designed. In the case of a smooth impermeable catchment the recurrence interval of the storm and of the runoff can be equated without significant loss of accuracy, provided the storm is defined by more than its rainfall depth. However, when a permeable catchment is considered, then large differences can exist between the recurrence intervals of the rainfall and the runoff and the engineer has no way of allowing for these variations in the design storm. A 10-year design storm on any particular permeable catchment will not necessarily produce the 10-year runoff hydrograph peak because the antecedent soil moisture condition can affect the runoff. Several researchers are in agreement with this conclusion (e.g. Dunsmore *et al.*, 1986; Hughes, 1986; Cordery, 1971; Hope, 1980).

The usual method of hydrograph synthesis for small catchments involves firstly selecting an appropriate design storm duration. The storm of required magnitude for a given frequency of occurrence and duration is distributed in time. Various storm durations are investigated before identifying the critical storm duration which produces the highest peak flow. The storm duration that produces the peak flow will not necessarily produce the greatest volume of runoff. The shape of the intensity distribution has a variable effect on the storm hydrograph peak and distribution (Akan and Yen, 1984, Lambourne and Stephenson, 1987).

Hope (1980) grouped the concepts of runoff generation into two major categories, namely those based on the infiltration and overland flow theory of runoff developed by Horton (1933), and those based on the unit source area theories, which include both the variable source and partial area approaches. The unit source area theories assume that the production of stormflow in the catchment is non-uniform. Either the stormflow is not necessarily surface flow but may be derived from subsurface flow (may be a combination of both), or stormflow occurs from certain areas of the catchment. In engineering practice a combination of Hortonian overland flow and subsurface flow is usually modelled. In urban catchments, subsurface flow is usually ignored (Green and Stephenson, 1986).

Runoff peak frequency methods

Methods of derivation of the frequency of storm runoff can be subdivided into either deterministic or probabilistic formulations.

Deterministic approach

Packman and Kidd (1980) used deterministic methods to assess the moisture content and storm shape for design applications to derive the design peak runoff with the same recurrence interval as rainfall input.

Constantinides (1982) proposed a concept involving excess storm depth – duration – frequency (D-D-F) curves. These curves are developed for different soil types based on the assumption that Hortonian theory dominates the streamflow process. Excess D-D-F curves are obtained by subtracting losses from actual storm hyetograph data and approximating the excess storms using a triangular shape to derive the duration. An assumption in the technique is that no further losses exist, which is not completely correct as infiltration occurs after the end of the storm, as long as there is water on the surface of the catchment. This assumption will not affect the peak discharge but will result in higher runoff volumes.

The basic problem with these methods is that runoff and rainfall data have to be available for the catchment being studied. While some autographic rainfall data may be available there will, in all probability, be inadequate data to define a reasonable range of D-D-F curves.

Statistical approach

When only peak flow records are required, then the best technique involves fitting parameters of a theoretical distribution to a

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flow record using statistical methods. In situations where flow records are not available, other techniques are adopted, based upon regionalised parameters or relationships with physical characteristics of the catchment (Benson and Matalas, 1967; NERC, 1975).

Where storage is required in a design problem, then the peak flow will be inadequate on its own. Hiemstra and Francis (1981) showed that more than one combination of flood peak and associated volume can have the same joint exceedence probability. They developed the 'runhydrograph' method to account for this using gauged runoff data. This is a technique which involves treating peaks and volumes as the two variables of a bivariate log-Normal probability distribution. This method allows for the synthesis of a whole family of design hydrographs for each derived return period.

Hughes (1986) investigated the bivariate probability distribution of the daily rainfall depth and an antecedent moisture index. He found that it was possible to make use of the bivariate normal distribution to represent the probability of one (or two) day rainfall and their associated indices of antecedent precipitation (API), given that suitable transformations were made to the data. The application of the method is a subject of further research.

Combined stochastic – deterministic approach

Most hydrological systems have both stochastic and deterministic components. The stochastic component is primarily defined through the application of probability distributions, whilst the deterministic component can be modelled mathematically without considering the probabilistic nature of the system. If any of the inputs to a hydrological system are stochastic, then the output will also be stochastic.

Most of the applications of combined hydrologic systems have been with storage behavioural patterns in reservoirs and water supply problems. Laurenson (1974) suggested that the marrying of stochastic and deterministic techniques can lead to great advances in many areas of hydrological practice, and detailed a combined approach to systems modelling.

Model formulation

A computer model called WHISPER was developed to derive flood frequency information for small urban catchments. The model is composed of a deterministic element, an AMC probability generator, and a combined deterministic-stochastic element.

Deterministic element

There are two relevant aspects in the deterministic modelling of the rainfall runoff process of concern, namely subcatchment overland flow and routing through channels. The components of the model WITWAT (Green, 1984) were used to describe these processes. WITWAT employs a kinematic routing for the overland flow contribution and time shift routing in channels. WITWAT has been tested (Green and Stephenson, 1985), both on South African and American catchments and has been shown to yield reasonable results.

In the runoff frequency model (described in this paper) excess rainfall hyetographs are calculated, subtracting losses using the Horton's equation. The depth of rainfall for a particular recurrence interval is estimated from D-D-F equations for the inland and coastal regions of South Africa as proposed by Op Ten Noort and Stephenson (1982). For a selected duration, the rainfall depth can be distributed over time using either a square topped or triangular shape (which accounts for the influence of the hyetograph shape on the hydrograph peak and volume). Constantinides (1982) derived a method for calculating excess storm hyetographs assuming a triangular storm hyetograph. With a square topped storm a triangular excess storm can be calculated with the maximum intensity at a time equal to the storm duration. The excess rainfall is then routed across the catchment to derive the runoff.

The infiltration parameters are selected on the basis of soil groups (A, B, C and D) and the AMC class (1, 2, 3 and 4). The loss parameters used in the Horton infiltration model were proposed by Constantinides (1982) and shown in Table 1. The deterministic element of the model produced excess storms for all the moisture condition classes, the probability of a particular moisture class being defined later.

TABLE 1
INFILTRATION VALUES FOR HORTON'S MODEL (AS RECOMMENDED BY CONSTANTINIDES, 1982)

	Hydrological soil group		A	B	C	D
Hortons	AMC					
Parameter	Bone Dry	(1)	83	67	42	25
Fo	Rather Dry	(2)	54	43	26	15
mm/h	Rather Wet	(3)	28	22	12	3
	Saturated	(4)	14	11	4	2
Fc mm/h			13	8	3	2

Fo = Initial infiltration rate
Fc = Final infiltration rate

Antecedent moisture condition probabilities

In determining the flood frequencies of runoff from a subcatchment, the probability of the antecedent moisture condition of the soil is also used. Gray *et al.* (1982) used simple probability methods for determining the average probability of each AMC class (1, 2, 3 and 4) assuming growing seasons for Indiana, Kentucky and Tennessee in the United States of America. In design practice, the seasonal variation of AMC class probability would be required.

In the current model attention was given to evaluating the probabilities of four AMC classes. This is a very simple approach to the concept and only requires the rainfall depth on the preceding five days to determine the AMC class. A sufficient length of daily rainfall record is required to adequately determine the probabilities for a particular catchment. Where record lengths do not allow the derivation of seasonal AMC probabilities, then an appropriate stochastic daily rainfall generator can be used. In South Africa such a model has been developed by Zucchini and Adamson (1984) using model parameters estimated for all regions of the Republic.

The four class AMC probability approach is very coarse and simplistic, and other methods of deriving the moisture status of the soil profile (e.g. use of ACRU model developed by Schulze, 1984; unsaturated component of the SHE model, Abbott *et al.*, 1986) would provide a more realistic approach.

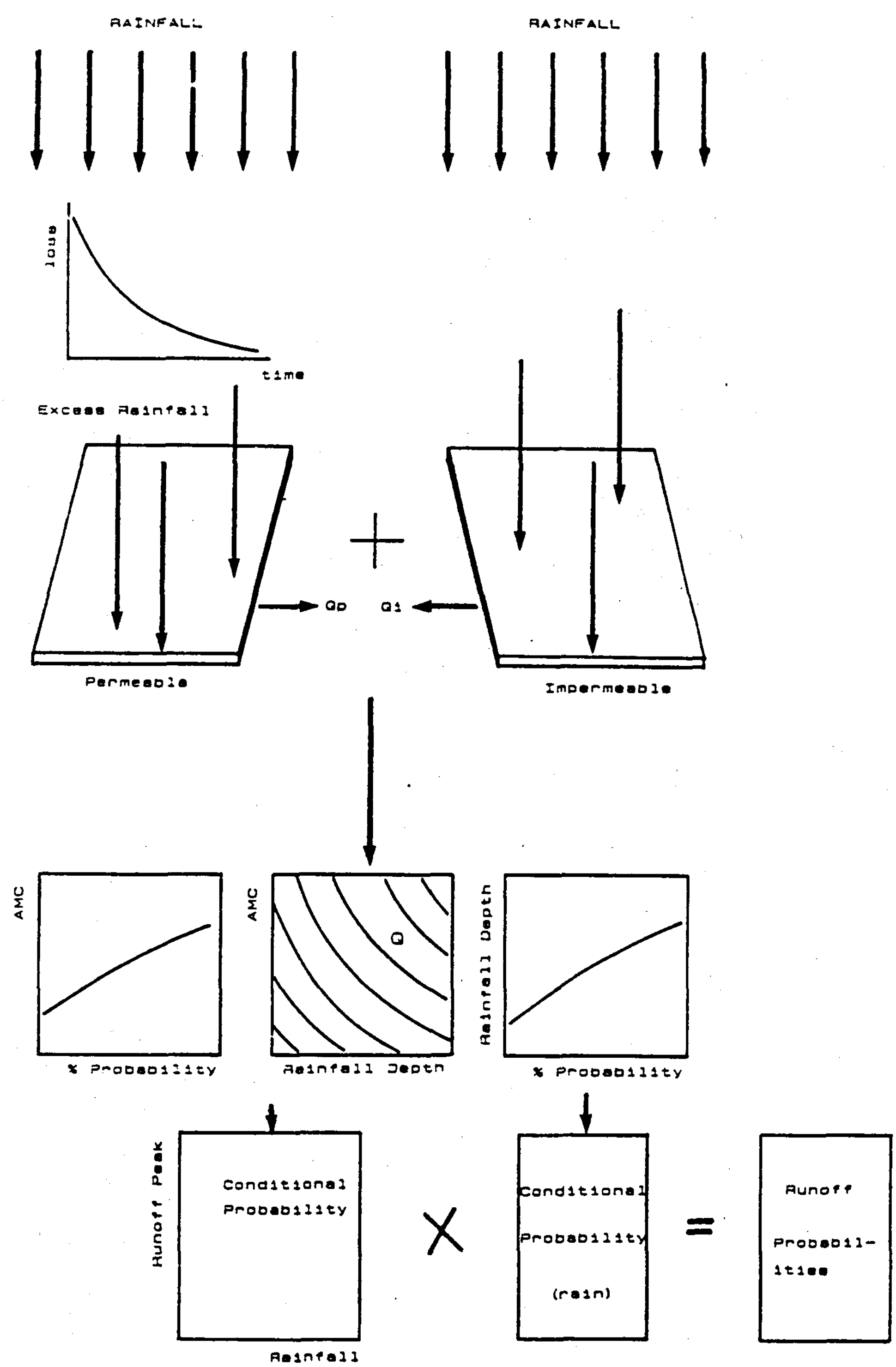

Figure 1
Subcatchment flood frequency procedure

Stochastic – deterministic element

The objective of this element of the model is to determine the probability distribution $\{q_i\}$ of the output y from a known probability distribution $\{p_i\}$ of the input x, the known probability distribution $\{r_i\}$ of a parameter z which may or may not be correlated with x, and a known deterministic matrix transformation T. This objective can be described in matrix notation by

$$Q = A.P \qquad (1)$$

were Q is the column matrix of output probabilities (order m), A is the matrix of transition probabilities (order mxn) and P is the column matrix of input probabilities (order n).

Applied to the rainfall-runoff approach, the probability distribution of flood peaks is derived from a known probability distribution of rainfall of duration D, known probability distribution of AMC and a known deterministic transformation to relate rainfall to runoff.

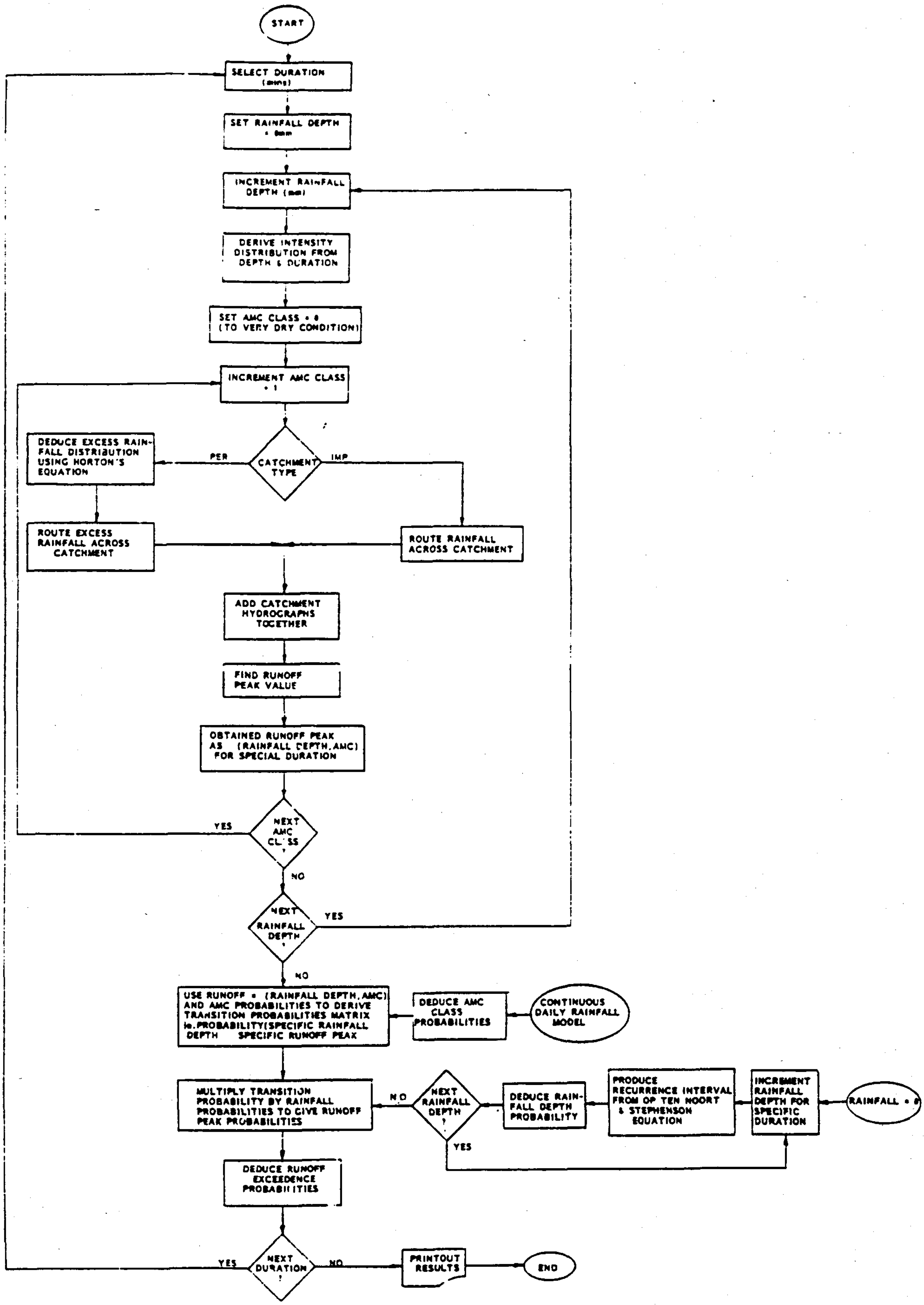

Figure 2
Flow-chart of flood frequency procedure

Let us first consider a catchment draining into an outlet. Goyen (1983) suggested that this was a 'concentrative' process with one independent parameter. The independent parameter is the AMC class. Based on the work of Beran and Sutcliffe (1972), it is assumed that the antecedent moisture content of the soil and the following rainfall distribution are independent events. For any particular value of the independent parameter (AMC), there is a one to one transformation from the input (rainfall) to the output (runoff). The conditional distribution of runoff, given a particular rainfall, consists of unity and a set of zeros (a 'concentrative' process as defined by Laurenson, 1974). However, there is a conditional distribution of runoff, not only for each rainfall but also for each possible value of the AMC. Each value of AMC has attached a certain probability m_k. So each element of the transition matrix (Equation 1) can be represented as

$$a_{ji} = \Sigma b_{jik} r_k \qquad (2)$$

where r_k is the probability of an AMC class and b_{jik} is the conditional probability of an output Y_j, given an input X_i and a parameter Z_k.

$$b_{jik} = Pr(y = Y_j \mid x = X_i, z = Z_k) \qquad (3)$$

A diagrammatic representation of the logic presented above and a flow-chart is given in Figs. 1 and 2 respectively. The transition probability from one rainfall depth (state i) to that of the runoff peak (state j) is calculated for each of the states based on the AMC probabilities. This probability transition matrix is then multiplied by the conditional rainfall probability array to give the peak runoff conditional probability vector. The corresponding exceedence probabilities are then computed and graphically displayed. In the present application the rainfall depth probabilities were determined from the use of D-D-F relationships.

Model application on a local catchment

The model was used to derive the flood frequency curve for a catchment in Montgomery Park, Johannesburg (Lat 26° 9,5'S Long 27° 59'E). Unfortunatly there is only a 3-year record of peak flows from the catchment during a time of drought in South Africa. The catchment is 1 036 ha in extent and is at a mean altitude of 1 695 m above mean sea level. The predominant land use is the natural soil land vegetation, although building programmes are underway in most sectors of the catchment which will increase the urbanised tracts of land and associated service roads and paved areas. The topography is fairly hilly with surface slopes ranging from 0,02 to 0,15 m/m. The highest elevation on the boundary of the catchment is 1 800 m above sea level and the outlet is about 1 600 m. The main drainage system of the catchment comprises both natural and artificial channel sections.

The Montgomery Park catchment information was applied to the model together with rainfall parameters, to determine the AMC probabilities. The peak runoff conditional probability vector was computed using the technique described in the previous section. The runoff peak frequency curve for the Montgomery Park catchment is presented (Fig. 3) as a function of either return period or percentage probability of exceedence and runoff peak discharge. The average of the observed peak flows in the catchment was 1,5 m^3/s and flows of 6,6 and 8,7 m^3/s were observed once in the record.

In order to test or verify a model of this type a considerable length of rainfall and runoff data would be needed (at least 50 years would provide adequate verification). This data however, is not readily available. The deterministic component of the model is based on the model WITWAT (Green, 1984) which has been tested satisfactorily on several catchments. The subcatchment

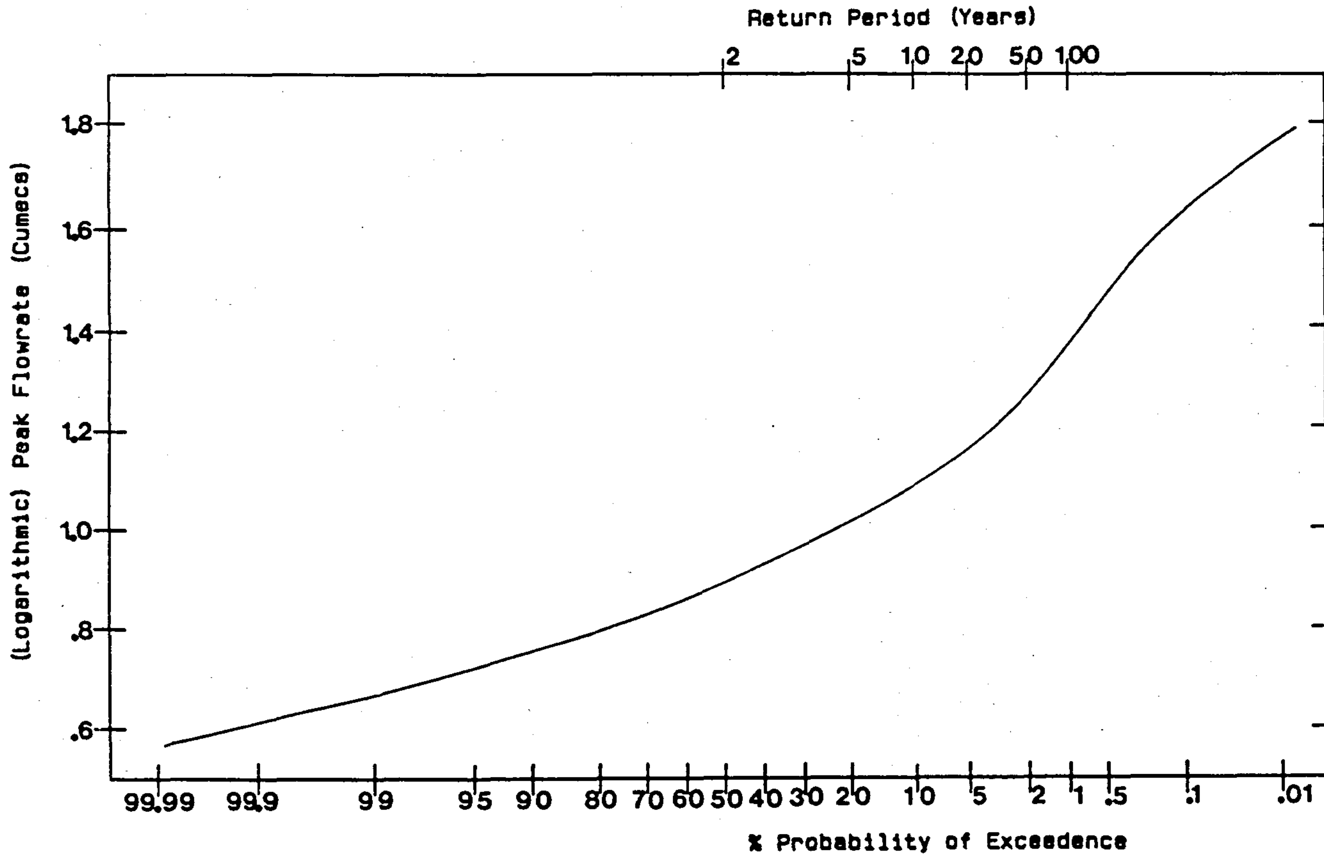

Figure 3
Flood frequency information for Montgomery Park catchment.

overland flow simulated by WITWAT was tested prior to its inclusion in the WITWAT model (Green, 1987). Another factor is that WITWAT performs to the same capability as SWMM (Huber *et al.*, 1982) which has been extensively tested. Laurenson (1974) tested the stochastic approach to deriving flood frequency information downstream of a confluence and found good agreement with that which was observed.

Discussion

A combined stochastic - deterministic procedure to overcome the shortcomings of both the excess D-D-F curves and excess storm recurrence interval methods was devised. It also overcomes the problems of transposition of parameters from gauged to ungauged catchments, which occur with statistical approaches to flood frequency estimation (e.g. the runhydrograph technique). This involves a catchment and routing procedure to derive the conditional probabilities of the outlet hydrograph peak. The deterministic component of the model is built around a tested simulation model. The method has the advantage of a traditional simulation model, of a modular construction for simulating land use effects and an output parameter (exceedence probability or return period) which is physically defined. Risk analysis can be easily applied to the result of this procedural technique. A program was written to perform the extensive calculations required with this type of technique.

The use of a range of infiltration values for different moisture conditions (the probabilities of occurrence being determined within the procedure) overcomes a major problem faced by engineers, of the correct selection of one infiltration value for a single event model.

The application of the method in its present form does use the Horton equation for derivation of losses, with no allowance for interflow mechanisms. Water movement within the soil matrix (an interflow component) could be easily added to the subcatchment procedure, provided a sufficient time-delay was incorporated. A further development of the method would be the replacement of both the Horton equation and the moisture condition classification by a procedure that would more adequately describe the infiltration process and moisture budget (as a function of moisture content and not classes). The rainfall input to the model is assumed to be spatially uniform and therefore the introduction of spatially varied rainfall would enable the method to be used for larger catchment areas.

The stochastic - deterministic technique presented in this paper goes some of the way to overcoming some of the problems identified with regard to obtaining flood frequency information on an ungauged catchment.

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Criteria for comparison of single event models

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ABSTRACT Methods of assessing the goodness-of-fit of hydrological simulation models are discussed. The methods generally compare the goodness-of-fit of a simulated hydrograph with an observed one and invariably have a bias towards one or another aspect of the hydrograph. It appears that no single parameter is sufficient to assess adequately the overall measure of fit between a computed and an observed hydrograph, particularly in view of the many objectives behind hydrological modelling. Previous research has concentrated on assessing the goodness-of-fit of long duration hydrographs resulting from continuous simulation. This paper discusses a number of criteria for hydrograph comparison in single event modelling studies and applies them to a specific stormwater modelling exercise. It is shown that particular criteria highlight particular aspects.

Critères de comparaison pour modèles caractérisant des événements isolés
RESUME On présente des méthodes pour évaluer la précision de l'ajustement des modèles hydrologiques. D'une façon générale, ces méthodes comparent le graphique résultant d'une simulation hydrologique avec le graphique réel associé et elles sont invariablement axées sur un aspect ou un autre de la courbe. Il apparaît qu'aucun paramètre unique n'est suffisant pour établir, de façon adéquate, la concordance entre la courbe simulée et la courbe réelle eu égard, notamment, aux nombreux objectifs inhérents à la modélisation hydrologique. De précédentes recherches étaient orientées vers l'évaluation de la valeur de l'ajustement d'hydrogrammes de longue durée résultant d'une simulation continue. Cet article envisage un nombre de critères de comparaison pour les modèles à événements isolés et les utilise, à titre d'exemple, dans un exercice spécifique où le modèle traite des écoulements dans les systèmes de drainage. Le fait que des critères particuliers mettent en évidence des aspects également particuliers est souligné.

INTRODUCTION

While it is generally accepted that no single hydrological simulation model output will be identical in all respects to the physical phenomenon it purports to represent, it is nevertheless required that this output be sufficiently close to its physical counterpart for the model to be considered acceptable.

The degree to which a model output conforms to the corresponding observed data can be measured by a variety of goodness-of-fit techniques. Such techniques may range from subjective, visual methods to purely objective techniques where the goodness-of-fit is measured by means of a statistical function of the differences between model and observed values.

Hydrological models have become popular for assessing water resources and flows in rivers, aquifers and man-made conduits. Modelling has become an accessible and accurate means of obtaining hydrograph data. Water resources are generally assessed by continuous modelling whereas single (storm) event models are often adequate for stormwater and flood studies. It is the latter type of hydrograph that is considered here.

Before acceptance, models need to be assessed with regard to accuracy by comparing their results for specific events against observed data or against other established models. The method of assessing a model will depend on the objective. For instance, if the modeller is interested only in peak flows there is little point in investigating low flows or even the hydrograph shape. On the other hand, if routing effects are important, the rising and falling limbs of hydrographs are important, and if storage is contemplated, the hydrograph volume will be important.

GRAPHICAL COMPARISON

Visual comparison of simulated and observed hydrographs provides a quick and often comprehensive means of assessing the accuracy of model output. Some disagreements, for instance in peak flow rates or in total flow volumes, are immediately evident, and a qualitative assessment can soon be made. Visual comparison, however, tends to be subjective, especially when a number of similar, but not identical, model outputs are compared to observed data and the "best" fit is sought. To overcome this difficulty, as well as to highlight certain model particularities, one or more of the statistical goodness-of-fit procedures discussed in the following sections can be employed.

Although the use of certain statistical goodness-of-fit techniques may be essential, the value of graphical comparison of simulated and observed hydrographs should not be overlooked. A graphical plot provides a "feel" for the model capabilities and can possibly mean as much, if not more, than the results of a numerical analysis of differences between simulated and observed values since it imparts more practical information than a statistical function. If one considers that many decisions are made on a subjective basis - even the choice of form of a statistical fitting technique is a subjective decision (Johnston & Pilgrim, 1976) - then the importance of subjective impressions will be appreciated. A graphical comparison between simulated and observed hydrographs should always be undertaken in any study involving computed and simulated hydrograph comparisons.

STATISTICAL GOODNESS-OF-FIT ASSESSMENT PROCEDURES

Statistical fitting procedures have been regularly employed in parameter calibration, one of the first such reported being in 1965 when Dawdy & O'Donnell (1965) demonstrated the usefulness of automatic optimization for parameter calibration for a hydrological model.

Since 1965 a number of goodness-of-fit criteria for assessing the accuracy of model output have been proposed. Any one particular criterion, however, may give more weight to certain aspects of disagreements between simulated output and observed data than another. This fact was recognized by Gorgens (1983) who stated that the choice and role of objective functions are aspects which "offer serious difficulties" to the modeller. Diskin & Simon (1977) demonstrated that there should be a definite link between the objective function and the application for which the model is used. They also demonstrated the need for considering more than one objective function for a given application.

Ibbitt & O'Donnell (1971) express similar sentiments with regard to statistical fitting procedures, namely that the form of the objective function chosen will affect the values of the fitted parameters because each criterion of fit places a different emphasis on the differences between simulated and observed runoff values. Thus the criterion of fit for flood prediction should be chosen to favour flood peaks. By the same token the objective function for fitting low flow sequences would probably include transformed flow rate components (e.g. logarithms of flow rates) to introduce a bias in the direction of low flows. Where more than one flow characteristic is of interest, and also to overcome the problem of bias, a multi-tier criterion can be adopted. This was demonstrated by Lichty *et al.* (1968) who proposed a two-tier criterion incorporating both peak flow rate and runoff volume.

Most criteria have been proposed with a view to application in continuous modelling studies. The objective of continuous modelling is the simulation of the water budget at regular intervals over a long duration, usually of the order of months or even years, and the desired output is generally a monthly or annual flow volume. Under these circumstances, protracted low flow sequences are of importance and any goodness-of-fit criterion chosen to assess the performance of such a model should be able to take this aspect of the hydrograph into account. Single event modelling on the other hand is usually employed to determine the peak flow rate, runoff volume and possibly hydrograph shape resulting from an isolated storm rainfall event. These factors can thereafter be used in the design or analysis of stormwater systems.

A distinction between types of errors generated by continuous and single event models can also be made. The errors in model prediction may be either random or systematic. Random errors occur when the model output shows no tendency to over- or under-estimate the observed data for a number of successive time intervals. On the other hand, systematic errors occur when the sign of the error persists over several successive time intervals. The importance of systematic errors was recognized by Aitken (1973) who pointed out that the criteria commonly used in testing hydrological models do not account for systematic errors. Criteria for the detection of systematic errors have been proposed by Aitken (1973) and Wallis & Todini (1975), and their use demonstrated by Pitman (1978) and Hughes (1982).

In continuous modelling studies, such as synthetic flow generation over days, months, or even years, systematic errors present in the model output could be of importance. Continuous over-estimation of low sequences on the one hand and reasonable agreement with the remainder of the flow record on the other would indicate a flaw in the model that could lead to serious repercussions if it were used for reservoir sizing. In a single event modelling context, the concept of random and systematic errors is not so clear. The relatively smooth shapes of hydrographs ensure that runs of errors of the same sign must occur and it does not seem reasonable to classify this situation as one of systematic error. It is therefore doubted whether systematic errors in single event modelling are of any real significance.

The foregoing presupposes that the ordinates are all equally spaced in time, since the detection of systematic errors with ordinates and resulting errors at irregular time intervals would be difficult. Since single event models operate at a user-defined time step, the resulting output and corresponding errors will all be equally spaced in time.

In most numerical techniques available for comparing the goodness-of-fit of two sets of data, the comparison is based on an analysis, in some form or another, of the errors or residuals, where:

(residual) = (observed value) - (computed value)

This type of analysis will generally be purely objective although semi-subjective analyses of residuals have been proposed. An example of this is a scatter diagram where every observation is represented by a point whose ordinate is the residual and whose abscissa is the computed value (Anscombe & Tukey, 1963).

According to Diskin & Simon (1977), the most commonly used objective function in assessing the goodness-of-fit of hydrological simulation models is the sum of squared residuals, defined by equation (1) in Table 1. Equation (1) can be considered to be a special case of a more general form given by equation (2) in Table 1.

Johnston & Pilgrim (1976) investigated the effects of variations in the values assigned to exponents b and c on the objective function G in equation (2). They concluded that changing the value of the exponent c in the objective function merely changes the vertical scaling of the response surface without altering the position of the minimum point. They further concluded that reproduction by the model of large and small events is independent of the value of the exponent c. According to Gorgens (1983), "this finding disproves the traditional assertion that changing the value of c varies the relative weighting given to small and large flow events." They did, however, recommend a value of $c = 2$ as this results in a parabolic response surface, which was considered to be the most favourable shape. Values of $c = 0.5$ and $c = 1$ give rise to a relatively flat response surface, making the location of the minimum point numerically difficult. With regard to the exponent b, however, it was found that the optimum values of parameters in hydrograph fitting depended to a large extent on the value assigned to b. For $b = 2$ large flow events were favoured while a value of $b = 0.5$ favoured low flow events. Power transformation of flows can be replaced by logarithmic flow transformations such as adopted by Lichty *et al.* (1968), which removes the bias from only large-magnitude flow events (Fleming, 1975).

Clarke (1973) criticizes the use of a sum-of-squares criterion, such as given in equation (1), or equation (2) with $c = 2$, unless consideration is given to the statistical properties of the residuals themselves. Selection of a sum-of-squares criterion implies that the joint probability distribution of the errors over the available length of record is normal with a mean of zero. Sorooshian & Dracup (1980) discuss the issue of heteroscedastic errors (i.e. with a changing variance) and maintain that the most commonly practised method of handling this case is through the application of the weighted least squares criterion. They point out that the computation of the correct weights is difficult and proposed an objective function in which the subjectivity involved in selecting a transformation in an attempt to stabilize the variance, or in selecting a weighted scheme, is eliminated. The fact that erroneous conclusions may be drawn from a sum-of-squares criterion when the underlying assumptions are violated was demonstrated by Kuczera (1983). A further disadvantage of $b = 2$ in equation (2) is that it exaggerates outlying points, which may in fact be in error and should then receive less attention.

The sum-of-squares criterion, in spite of its alleged shortcomings, has been used by many workers in the fields of either model comparison studies or model calibration and parameter optimization, notably Dawdy & O'Donnell (1965), Nash & Sutcliffe (1970), Johnston & Pilgrim (1976) and Pickup (1977).

Instead of a sum-of-squares criterion, Stephenson (1979) adopted the sum of absolute values of residuals as a goodness-of-fit criterion in an optimization study (equation (3)).

A drawback of the objective functions described by equations (1), (2) and (3) is the fact that they are dimensional. Furthermore, the value of the objective function thus obtained will depend on the number of ordinates used in the data sets. For a meaningful comparison of different models using a number of data sets differing not only in magnitude but also in number of records contained in the data sets, it is desirable to reduce the basis of comparison to a dimensionless form.

Nash & Sutcliffe (1970) proposed a dimensionless coefficient of model efficiency, given by equation (4) in Table 1. They termed F^2 the "index of disagreement" and F_o^2 the "initial variance" of the observed flows. The coefficient of efficiency has gained wide acceptance and seems a reasonable choice for a dimensionless measure of fit which is preferred to a dimensional measure for general studies. An appealing aspect of this coefficient lies in its simplicity, its value increasing toward unity as the fit of the simulated hydrograph progressively improves. Hughes (1983) made exclusive use of this coefficient for single event model calibration.

Table 1 List or goodness-of-fit criteria

No.	Criterion	Equation	Remarks	Reference
1.	Sum of squared residuals	$G = \sum_{i=1}^{n} [q_o(t) - q_s(t)]_i^2$	Most common	Diskin & Simon (1977)
2.	Sum of m-powered residuals	$G = \sum_{i=1}^{n} [(q_o(t))^b - (q_s(t))^b]_i^c$	Changes vertical scaling	Johnston & Pilgrim (1976)
3.	Sum of absolute errors	$G = \sum_{i=1}^{n} \mid q_o(t) - q_s(t) \mid_i$	Suited to optimization	Stephenson (1979)
4.	Model efficiency	$R^2 = \dfrac{F_o{}^2 - F^2}{F_o{}^2}$ with $F^2 = \sum_{i=1}^{n} [q_o(t) - q_s(t)]_i^2$ and $F_o{}^2 = \sum_{i=1}^{n} [q_o(t) - \bar{q}]_i^2$	Dimensionless	Nash & Sutcliffe (1970)
5.	Normalized objective function	$P = \dfrac{1}{\bar{q}}\left(\dfrac{F^2}{n}\right)^{1/2}$	Coefficient of variance	Ibbitt & O'Donnell (1971)
6.	Root mean square error	$RMSE = \left(\dfrac{1}{n} \sum_{i=1}^{n} (q_o(t) - q_s(t))_i^2\right)^{1/2}$	$P = \dfrac{RMSE}{\bar{q}}$	Patry & Marino (1983)
7.	Automatic parameter optimization	$G = \sum_{i=1}^{n} \left[\dfrac{q_o(t) - q_s(t)}{(q_o(t))^b}\right]^c$	b, c are control parameters	Wood (1974)
8.	Reduced error estimate	$REE = \left[\dfrac{\sum_{i=1}^{n} (q_o(t) - q_s(t))_i^2}{\sum_{i=1}^{n} (q_o(t) - \bar{q})_i^2}\right]^{1/2}$	Biased to larger flows	Manley (1978)

9.	Proportional error of estimate	$PEE = \left[\sum_{i=1}^{n} \left(\frac{q_o(t) - q_s(t)}{q_o(t)} \right)_i^2 \right]^{1/2}$	Equal weight to equal proportional errors	Manley (1978)
10.	Standard error of estimate	$SEE = \left(\sum_{i=1}^{n} \frac{(q_o(t) - q_s(t))_i^2}{(n - 2)} \right)^{1/2}$	Dimensional and independent of number of points	Jewell *et al.* (1978)
11.	Coefficient of persistence	$CP = \sum_{i=1}^{n} \left(\frac{a^2}{F^2} \right)_k$	Dimensional	Wallis & Todini (1975)
12.	Percent error in peak	$PEP = \frac{q_{ps} - q_{po}}{q_{po}} \times 100$		
13.	Percent error in volume	$PEV = \frac{V_s - V_o}{V_o}$		
14.	Percent error in mean	$PEM = \frac{\bar{q}_s - \bar{q}_o}{\bar{q}_o}$		
15.	Total overall sum of squared residuals	$TSSR = \sum_{j=1}^{m} \left\{ \sum_{i=1}^{n} [q_o(t) - q_s(t)]_i^2 \right\}_j$		
16.	Total overall sum of absolute residuals	$TSAR = \sum_{j=1}^{m} \left\{ \sum_{i=1}^{n} \lvert q_o(t) - q_s(t) \rvert_i \right\}_j$		
17.	Sum of absolute areas of divergence	$A = \sum_{i=1}^{n} \left\lvert \frac{(residual)_i + (residual)_{i+1}}{2} \Delta t \right\rvert_i$		

Table 1 *(continued)*

18.	Total sum of absolute areas of divergence	$TSAA = \sum_{j=1}^{m} A_j$
19.	Variance	$S^2 = \frac{1}{n} \sum_{i=1}^{n} [q_o(t) - q_s(t)]_i^2$
20.	Mean deviation	$MD = \frac{1}{n} \sum_{i=1}^{n} [q_o(t) - q_s(t)]_i$
21.	Absolute area/ordinate divergence	$\frac{A}{n}$

a area of an individual segment of deviation
k number of runs of successive over- or under-predictions
m number of events
n number of pairs of ordinates compared in a single event
$q_o(t)$ observed flow rate at time t
$q_s(t)$ simulated flow rate at time t
q, q_o mean of observed flow rate
q_s mean of simulated flow rate
q_{pc} observed peak flow rate
q_{ps} simulated peak flow rate
V_o observed volume
V_s simulated volume
Δt time step, here assumed uniform for an event

This particular criterion has been criticized by Garrick *et al.* (1978) on the grounds that it is insensitive. They claimed that poor models produce relatively high values of R^2 while the best models produce values that are not that much higher. They were, however, assessing the performance or sensitivity of the coefficient of efficiency using yearly river flow data in a highly seasonal regime and it is doubted whether their criticism is of any significance when R^2 is applied to a single event rainfall-runoff simulation.

Ibbitt & O'Donnell (1971) proposed a "normalized" objective function in the form of the coefficient of variation (equation (5)).

Patry & Marino (1983) in assessing the performance of a nonlinear functional runoff model adopted the root-mean-square error (equation (6)) as a criterion for comparison of hydrographs. It can be seen that the root-mean-square error is dimensional, having dimensions of flow rate. The Natural Environment Research Council (NERC), in their *Flood Studies Report* (NERC, 1975) endorsed the views of Nash & Sutcliffe (1970) and Ibbitt & O'Donnell (1971) by suggesting that equations (4) and (5) be used as model fitting criteria.

Another criterion for hydrograph fitting was proposed by Wood (1974) in demonstrating an automatic parameter optimization technique. This is given by equation (7) in which a trial and error approach at determining values of b and c for different strategies was adopted by Wood. By varying the parameter b, a range of weightings between equal absolute errors and equal proportional errors can be achieved.

Two novel objective functions were considered by Manley (1978). One objective function, termed the reduced error estimate and given by equation (8), was found to be insensitive to low flow sequences. He therefore proposed the proportional error of estimate (equation (9)) in which equal wieght is given to equal proportional errors. Thus the residual error estimate is biased towards large flow events, while the proportional error or estimate will be effected more evenly over the whole range of flows.

Jewell *et al.* (1978) proposed a methodology for calibrating hydrological models in which the measure of fit was obtained from the standard error of estimate, or SEE, defined by equation (10). The standard error of estimate is dimensional, although the effect of record length, or number of ordinates, has been removed.

SELECTION OF GOODNESS-OF-FIT CRITERIA FOR SINGLE EVENT MODELLING

Flood studies are frequently single event studies, unless antecedent moisture changes are to be modelled. Urban drainage studies depend to an even less extent on preceding conditions and can generally be regarded as single event studies.

Peak flow rate, volume and mean flow rate

Peak runoff rates are often the most important outputs of single event modelling. A comparison of peaks can take the form of a ratio of simulated to observed peak flow rates or else it can be expressed as a percentage error in the simulated peak (equation (12)).

A similar comment applies to volume of runoff and mean flow rates, resulting in equations (13) and (14) respectively.

The volume criterion suggested (equation (13)) becomes irrelevant in the case of staged optimization, as advocated for example by Pilgrim (1975). Single event models generally comprise two distinct parts: a loss component that separates the losses (e.g. infiltration) from the effective rainfall, and a routing component that converts this excess rainfall into a flood hydrograph. If a model is calibrated by staged optimization on a single event, the loss parameters are usually adjusted to give the correct volume thus rendering the volume criterion irrelevant. Thereafter the parameters relating to the routing component are calibrated using one or more of the fitting criteria mentioned. If, on the other hand, it is desired to assess the performance of the now calibrated model on an event not used in the calibration process, then the volume criterion becomes relevant.

Dimensional, ordinate dependent shape factors

For optimizing the shape of the simulated hydrograph, a one-to-one correspondence of ordinates over the duration under consideration is generally required and the sum of squares or sum of absolute residuals criteria are considered adequate in this instance. However, to study the performance of a number of different models taken over several different events, it seems reasonable to compare the total sum of individual sums of squared or absolute residuals for each event, providing that the same number of ordinates is used within each individual event for each model, and that the total sum is taken over the same events for each model. Equations (15) and (16) result.

The percentage error in simulated volume, given by equation (13), may indicate good agreement between simulated and observed total volumes, whereas the shapes of the respective hydrographs could be considerably different. This difference can be highlighted by considering the sum of absolute areas of divergence between the two hydrographs, given by equation (17). For a comparison over several different events, a total sum of absolute areas of divergence, given by equation (18), can be employed.

Dimensional shape factors independent of the number of ordinates

A measure of goodness-of-fit whose value is affected by the number of ordinates used in any comparison is obviously limited in its application to either single events or to a number of events having equal numbers of ordinates within each event. Considering the sum of squared residuals criterion, and dividing the sum obtained by the number of ordinates in the particular event, one obtains the variance for that event, defined by equation (19).

In similar fashion, the mean deviation can be obtained by dividing the sum of absolute residuals for an event by the number of ordinates in that event resulting in equation (20).

With regard to the sum of absolute areas of divergence (i.e. the area contained between the simulated and observed hydrographs), the effect of the number of ordinates can similarly be removed by dividing the sum obtained by the number of ordinates (equation (21)).

While the effect of the number of ordinates has been removed in equations (19), (20) and (21), the dimensional character indicates that the effect of magnitude is still present. This effect is not seen as a serious drawback providing that when average values are taken for comparing the merits of different models, they are done so using the same events for each model.

Synchronization errors

In attempting to reproduce an observed hydrograph by simulation, a time-shift error is often apparent and should be accounted for in some way. This type of error is generally the result of asynchronous operation of rainfall and streamflow recording equipment, the time base of the simulated hydrograph being equated to that of the rainfall recording equipment. Another possible reason could be that the particular single event model under consideration requires one or even several time steps to initialize computations, thereby introducing a time-shift error.

Different approaches to dealing with synchronization errors have been adopted by various researchers. Marsalek (1979) and Watson (1981) detected the presence of timing anomalies in single event modelling studies but neglected them in the comparisons that followed. Haan (1975) and Constantinides (1982) on the other hand recognized the presence of synchronization errors and shifted the simulated hydrographs in time until a best fit was obtained with that observed.

It is considered that in studies of this nature, timing errors should be corrected by eye as it does not seem justifiable to allow an obvious error in synchronization to affect adversely a comparison between observed and simulated hydrographs.

CASE STUDY

The criteria selected for comparison of simulated and observed hydrographs and discussed in this section were employed by Green (1985) to assess the relative performance of several single event models. The models used in this particular study were SWMM (Huber *et al.*, 1982) employed in single event model, ILLUDAS (Terstriep & Stall, 1974) and WITWAT, an urban drainage model developed by the Water Systems Research Programme of the University of the Witwatersrand. WITWAT is a microcomputer-compatible program designed for interactive use in either analysis or design mode. It uses kinematic routing overland and (in this example) time shift routing down sewers.

Figures 1 and 2 illustrate graphical comparisons of the resulting simulated hydrographs with those observed for two recorded events. The relevant statistics corresponding to these events are listed in Tables 2 and 3. The input parameters for the models used were optimum parameters based on a trial and error calibration for the particular catchment used and were held constant for both events. The relatively smooth shapes of the hydrographs computed by both SWMM and WITWAT resulted in much higher coefficients of persistence (an indicator of systematic error given by equation (11)) than did

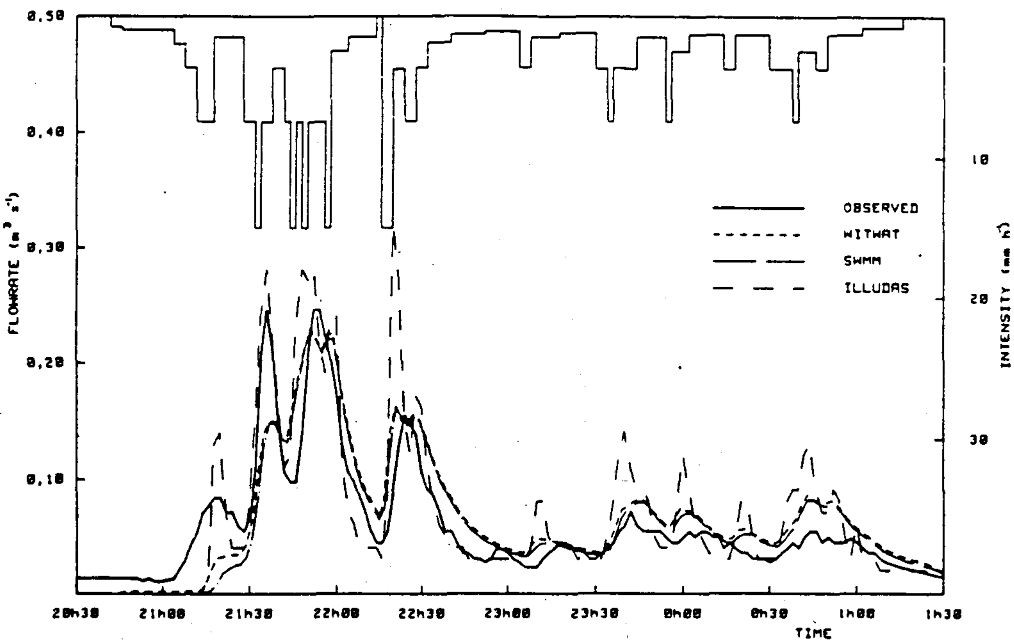

Fig. 1 Comparison of observed and simulated hydrographs from Pinetown catchment for storm on 4 May 1979.

Table 2 Results of goodness-of-fit tests from Pinetown catchment for storm on 14 May 1979

Parameter	WITWAT	SWMM	ILLUDAS
Peak flow rate ($m^3\ s^{-1}$)	0.23	0.23	0.32
Ratio of peaks (sim/obs)	0.94	0.93	1.31
% error in simulated peak	−6.5	−7.3	30.6
Mean flow rate ($m^3\ s^{-1}$)	0.07	0.07	0.08
Ratio of means (sim/obs)	1.15	1.09	1.19
% error in simulated mean	14.7	8.6	19.3
Volume of flow (m^3)	1153	1092	1202
Ratio of volumes (sim/obs)	1.15	1.09	1.19
% error in simulated volume	14.7	8.6	19.4
Sum of squared residuals $(m^3\ s^{-1})^2$	0.10	0.11	0.23
Sum of absolute residuals ($m^3\ s^{-1}$)	2.93	2.79	3.45
Sum of absolute areas of divergence (m^3)	342	325	358
Coefficient of efficiency	0.68	0.67	0.33
Proportional error of estimate	0.48	0.46	0.66
Coefficient of persistence $(min)^2$	72.35	64.07	15.79

Number of ordinates used in comparison = 130.

Observed hydrograph data
Peak flow rate = 0.25 $m^3\ s^{-1}$
Mean flow rate = 0.06 $m^3\ s^{-1}$
Volume = 1005 m^3

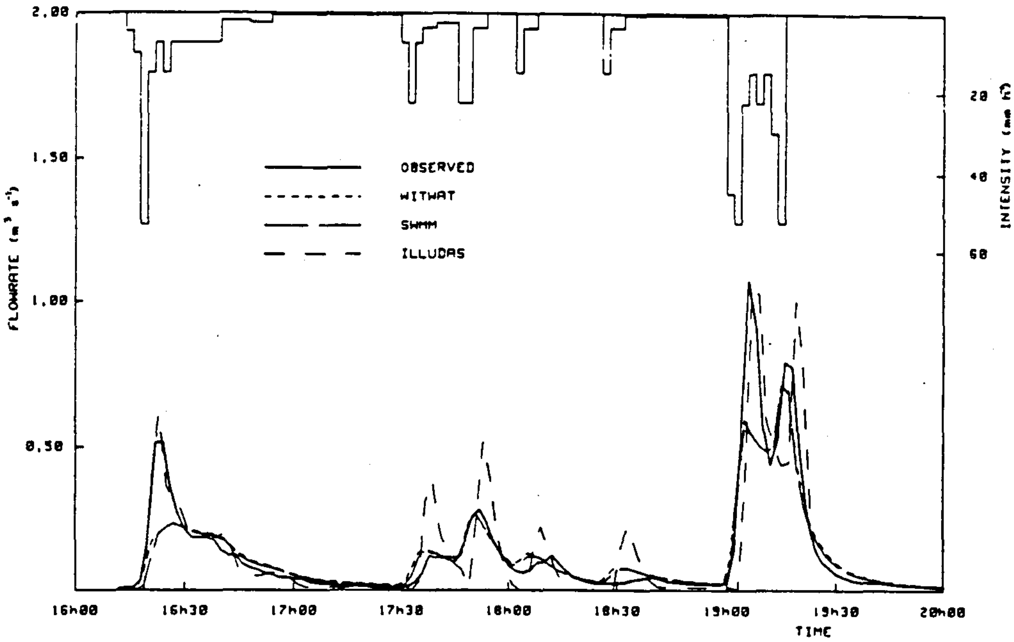

Fig. 2 Comparison of observed and simulated hydrographs from Pinetown catchment for storm on 22 May 1979.

Table 3 Results of goodness-of-fit tests from Pinetown catchment for storm on 22 May 1979

Parameter	WITWAT	SWMM	ILLUDAS
Peak flow rate (m^3 s^{-1})	0.70	0.72	1.05
Ratio of peaks (sim/obs)	0.65	0.67	0.98
% error in simulated peak	--34.7	−33.2	−1.9
Mean flow rate (m^3 s^{-1})	0.14	0.13	0.15
Ratio of means (sim/obs)	0.95	0.92	1.06
% error in simulated mean	−4.7	−7.6	5.6
Volume of flow (m^3)	1497	1452	1634
Ratio of volumes (sim/obs)	0.94	0.91	1.03
% error in simulated volume	--5.8	−8.6	2.9
Sum of squared residuals (m^3 s^{-1})	0.70	0.70	1.74
Sum of absolute residuals (m^3 s^{-1})	3.88	3.77	6.63
Sum of absolute areas of divergence (m^3)	381	368	571
Coefficient of efficiency	0.82	0.82	0.55
Proportional error of estimate	0.65	0.51	1.10
Coefficent of persistence $(min)^2$	11.85	11.58	4.60

Number of ordinates used in comparison = 93.

Observed hydrograph data
Peak flow rate = 1.07 m^3 s^{-1}
Mean flow rate = 0.14 m^3 s^{-1}
Volume = 1589 m^3

the hydrographs computed by ILLUDAS. The overall fits of the first two models were, however, better than those of ILLUDAS, evidenced by the sum of squared and absolute residuals, the sum of absolute areas of divergence and the coefficient of efficiency, thereby illustrating that systematic error can be misleading in a single event context.

With regard to the event illustrated in Fig.2, the statistics of which are given in Table 2, it will be seen that ILLUDAS achieves the best volumetric fit as well as the best estimate of peak flow rate in spite of the fact that the overall one-to-one fit of ordinates is poor compared with that of SWMM and WITWAT. This apparent contradiction is clearly demonstrated by the oscillatory nature of the hydrograph computed by ILLUDAS, and highlights the need for considering more than one criterion when comparing model performance.

Scale effects are also noticeable when comparing the sum of squared and absolute residuals for the two events. The mean flow rate for the event depicted in Fig.1 is 0.06 m^3s^{-1} while that for the event depicted in Fig.2 is 0.14 m^3s^{-1}. In the first case the sums of absolute residuals corresponding to the different models are between seventeen and thirty times greater than the respective sums of squared residuals, while for the second event with higher flow rates the sums of absolute residuals are only about five times greater than the sums of squared residuals. This illustrates the bias given to higher flow rates by the sum of squares criterion.

It also appears that this bias is present in the coefficient of efficiency. In the first event (Fig.1; Table 2) the sums of absolute areas of divergence for all three models are very similar, while the coefficient of efficiency with respect to ILLUDAS is about half that of SWMM and WITWAT. In the second event, however, (Fig.2; Table 2) the sum of absolute areas of divergence for ILLUDAS is much higher than for the other two models whereas the coefficient of efficiency with respect to ILLUDAS is improved relative to the other two models. This is because the divergence of the output compared by ILLUDAS over the second event is more evenly spread over the range of flows, while in the first event there are some rather significant discrepancies between the ILLUDAS output and the observed hydrograph. Since the coefficient of efficiency involves the squares of residuals, this statistic will be biased by more extreme variations.

CONCLUSIONS

An important point which emerges is that no single statistical goodness-of-fit criterion is sufficient to assess adequately for all purposes the fit between a computed and an observed hydrograph. As different goodness-of-fit criteria are weighed in favour of different hydrograph components (e.g. volumes, peak flow rates), the criteria ultimately chosen should depend on the objective of the modelling exercise.

The objectives of single event modelling are generally the determination of peak flow rates, runoff volumes and/or hydrograph shape. The criteria selected to assess the goodness-of-fit of the computed hydrographs with regard to these components are:

- for peak flow rates, a direct comparison of simulated and observed peaks as given by equation (12) is sufficient;
- for volumetric assessments, a direct comparison of simulated and observed volumes as given by equation (13) is adequate;
- for assessing the overall goodness-of-fit or shape of a simulated hydrograph for a single event, a sum of squares or sum of absolute residuals may be used;
- for assessing the goodness-of-fit of several simulated hydrographs from a number of events, the total sum of squared residuals or a total sum of absolute residuals should be used (equations (15) and (16)). A sum of absolute areas of divergence (equation (18)) can also be used for this purpose, providing the simulations are taken over the same duration for each event, and the sum taken over the same events.

If it is required to assess the performance of a model over a number of different events, a more general dimensionless ordinate-independent measure of fit is required. In this case the coefficient of efficiency given by equation (4) seems a reasonable choice. The proportional error of estimate (equation (9)) could also be used. Average values of these measures may then be used for an overall assessment.

No comparison would be complete without a plot of observed and simulated hydrographs. Modern computer hardware with enhanced graphics facilities can produce graphical plots in a very short time, and can thus provide the modeller with an immediate subjective assessment of a model's performance.

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A modular model for simulating continuous or event runoff

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ABSTRACT A review of stormwater model arrangements is made. Computer programs able to assemble loosely-connected elements are simplest to use and understand. Arrangement of elements and assembly in parallel or series enables all possible types of models to be accommodated. Hydraulic elements are used for surface runoff visualization and groundwater aquifers are simulated in parallel for continuous or long term simulations.

INTRODUCTION

The accuracy of runoff models can be improved at the expense of more and more data. There are many models available with differing levels of sophistication for such studies. The author contends however the biggest cost of modelling is often in learning the ins and outs of a model, and its principles and limitations. On the lower levels of the learning curve many users may wish to dabble with a model, and at the upper end there are many technical aspects to remember. Time away from the model causes users to forget aspects, and it is the ease of initial or re-access which can inspire confidence in the user and enable the model to be used to its fullest. Various types of models are discussed bearing these points in mind.

Sub catchment arrangement

The interconnection of one sub-catchment or element with another can be done in various ways (Fig. 1):

i) Finite difference grids In the case of a homogeneous type catchment a rectangular grid can be superimposed. Thus flows and water depths are computed at grid point. Either one or two directional flow can be assumed. In general two flow vectors must be assumed. An exception occurs if the flow is in one direction parallel to one of the axes. For most undular topography two-dimensional analysis is necessary.

ii) Finite element The computations can be reduced and size and shape of element varied to suite the topography if a finite element approach is used. In general a two-direction flow pattern must be assumed although if the boundaries of elements are perpendicular to flow, one-directional flow can be assumed.

iii) Modular The simplest and most versatile model is one made up of modules which can be linked up at the ends. Generally the flow is one directional along the axis but two dimensional catchments can be made up of modules in parallel and series, i.e. the orientation of the module is ignored because the directional momentum of the water is not considered. It is the latter configuration on which the model describes here is based.

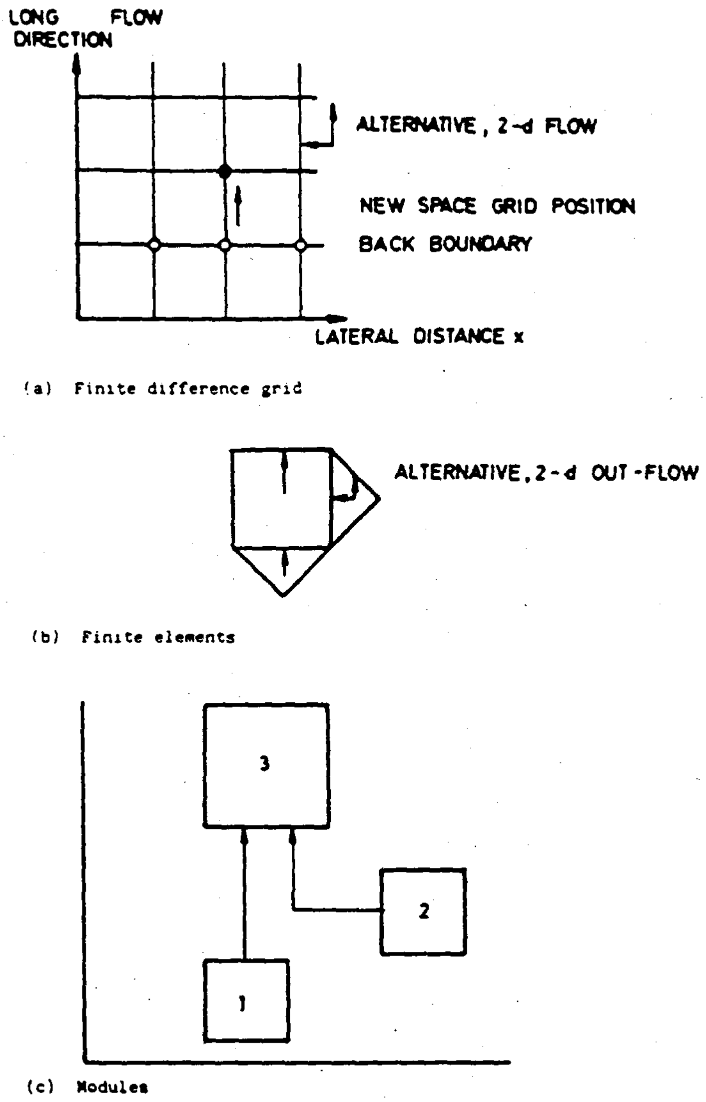

Fig. 1 Alternative grids.

THEORETICAL BASIS

Hydrological models range from statistical to conceptual, embracing probabilistic, curve fitting, black boxes, analogous e.g. cell type (Diskin et al 1984), through to the more hydrodynamically correct. Even the latter range from simplistic e.g. time area (Watson, 1981) though first approximation kinematic type, diffusion equations and hydrodynamic equations (SWMM, Huber et al, 1982). The latter are only necessary for surface runoff simulation and even then are not always warranted. For runoff determination accelerations and backwater effects are not significant. On the other hand the time-area approach which derived from the rational method, does not accommodate the effect of water depth on concentration time. Mono time axes and linear rainfall-runoff relationships have been taken to their extreme in unit hydrograph theory, and as a result the hydraulic basis is often overlooked in sophisticated models e.g. OTTHYMO (Wisner, 1980).

It is into the more hydraulically based models that the majority of research is now directed. By suitable selection of module arrangement, one-dimensional flow can be assumed. i.e. the module axis is taken in the general flow direction. Lateral flow time is neglected, (which could introduce error in flood plane type modules). Transverse i.e. lateral and vertical (for horizontal flow direction) accelerations are also neglected but this is quite satisfactory for all runoff modelling (see Fig. 2).

Thus the hydrodynamic equations are narrowed down to the St. Venant equations and their derivatives. Accelerations are not of importance in overland or long river studies so these terms are omitted, and backwater effects are only of importance in some channel situations, so most modules are limited to kinematic type equations.

Flow is assumed uniform down the reach. That is, the water depth is constant down the reach. There may be local backup which does affect system storage however. Since inflow is thus spread over the full length of a reach each time step, the routing effect can be unrealistic unless the time step is sufficiently large. Methods of minimizing numerical routing (or using it to approximate time routing) have been investigated (Holden and Stephenson, 1988).

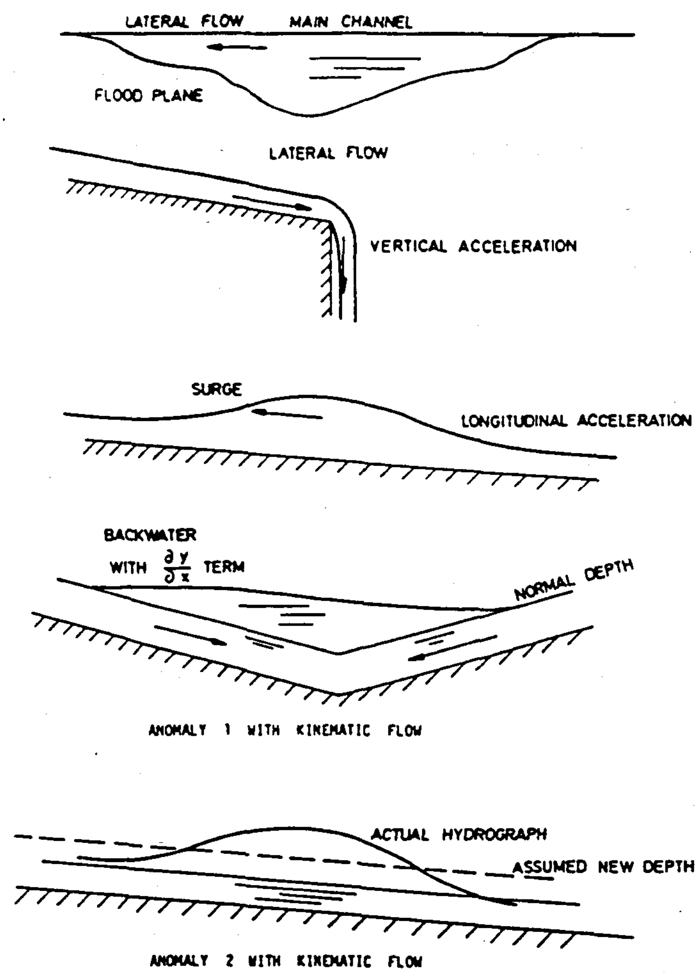

Fig. 2 Problems with specific numerical methods.

It should be noted the kinematic assumption of energy gradient parallel to conduit bed can cause complications at changes in slope. Depressions do not correctly store inflow, and separate storage modules are necessary.

CONTINUOUS SIMULATION

Groundwater flow capability with aquifer modules makes possible long term simulation of catchment yield. Groundwater contributions lag surface runoff by hours or even months. Recession limbs of stormwater hydrographs can be due to contributions from perched water tables or interflow. Longer term yields are from deeper aquifers.

Recharge of surface layers is however important from the point of view of antecedent moisture and permeability for forthcoming storms. The continuous simulation capability therefore improves estimation of surface storm runoff. Surface layer moisture is also important for estimating evaporation and losses.

The time scale of flow from deeper aquifers may be much longer than from the higher water tables, and a greater time step could be used once surface runoff is reduced.

The problem then arises as to future storm problems and their infiltration. However from the total yield point of view it is not critical if storm patterns are assumed.

ROUTING PROCESS

Kinematic waves are theoretically not subject to diffusion i.e. spreading and attenuation, as no dynamic effects are included in the equation. There may be changes in wave shape since dx/dt is a function of depth, but there can be no change in peak flow unless there is an inflow. The advantage of taking large distance increases with the kinematic method therefore results in a sacrifice in accuracy. Holden and Stephenson (1988) proposed a method for minimizing the numerical error and getting the best approximation to hydrodynamic diffusion.

The wave diffusion can be accounted for using the slightly more accurate equations, namely the diffusion equations, or the full dynamic equations. However in some cases wave diffusion can be reproduced numerically. From the mathematical point of view, numerical diffusion can be controlled or minimised. Explicit solution of the kinematic equations is often employed in preference to implicit solution as the friction equation is non-linear, and explicit schemes such as the backward centred, or semi explicit such as the 4-point scheme of Brakensiek (1967) are reasonably accurate and fast. Explicit schemes can be subject to numerical instability unless the time increment is small enough, i.e. $\Delta t < \Delta x/(dx/dt)$, (the Courant criterion) where $dx/dt = \alpha m y^{m-1}$. On the other hand the smaller Δt the greater the numerical diffusion as the numerical effect travels at a speed $\Delta x/\Delta t$. The optimum compromise is for $\Delta x/\Delta t = dx/dt$. This is not always possible in an equispaced grid as dx/dt varies. Ponce (1986) attempted to reproduce actual diffusion in kinematic equations by writing the finite difference equations for flow in a way similar to the Muskingum-Cunge routing equation.

Adopting a more practical approach the kinematic diffusion process can be explained as follows. The routing process which occurs with kinematic modelling is similar to reservoir routing where discharge depends only on the stage at the outlet. A unique stage discharge relation is assumed i.e. no allowance is made for accelerations or water surface gradient. A compromise could be made by setting discharge a function of stage at more than one point e.g. average of upstream and downstream stages.

The resulting effect is similar to that employed in the Muskingum method and in addition allows for non-linearity in the stage-discharge relationship. It also has the advantage that the parameters in the equations are physically measurable and not empirical. To overcome the non-linear relationships the kinematic equations can be solved in two steps, namely the continuity equation to determine change in water depth, and discharge is obtained from stage using the selected discharge equation.

The discharge equation is not limited to a channel type equation such as that of Manning. Thus using a general discharge equation of the form

$$Q = Kh^m$$

if h is stage at discharge point and m = 5/3, one has the Manning equation, if m = 5/2 one has a triangular weir, m = 3/2 is a rectangular weir, m = 1/2 is an orifice and m = 1 is a deep rectangular channel. If h is the difference between upstream and downstream stages, then if m = 1/2 one has turbulent pipe flow, and if m = 1 one has laminar flow in a closed conduit or contained aquifer.

MANAGEMENT CAPABILITY

A drawcard of a model prepared on the above lines WITSKM (WITS STORM KINEMATIC MODULAR MANAGEMENT MODEL) is its versatility when it comes to redirecting flows and attenuating hydrographs. The facility of readily being able to redirect flows along different routes means channel storage or open versus closed conduit conveyance can be explored. The re-routing of flows along circuitous routes may increase channel storage. This in turn increases concentration time and could reduce design peak flows. New townships layouts could be varied until a suitable stormwater drainage pattern emerged.

The overflow facility also enables dual drainage to be used to maximum advantage. Excess flow could be led to shallow channels (or roadways) which will provide retardation or lead to channels which are only used in emergencies. the overflow level can readily be varied to permit difference risk storms to be accommodated in the minor (underground conduit) system.

The aquifer option is also of use in urban catchment management studies. Aquifers can be recharged by direct infiltration or with water led to them from less pervious areas. In either case the absorption of the aquifer is only limited by the depth-discharge characteristics and initial moisture conditions.

A useful module for hydrograph attenuation is the storage module. Reservoir surface area, dead storage and overspill crest level can be varied to achieve an optimum balance between maximum water depth and dam cost. The ability to vary the outlet discharge characteristic is however the most versatile facility of the storage module. By means of a general discharge equation of the form

$$Q = (WA)y^m$$

any form of outlet control can be used. For example an orifice is represented if m = 1/2 and WA = Ca $\sqrt{(2g)}$, where C is a discharge coefficient, a is the orifice cross-sectional area and g is gravity.

For detention attenuation which has a decreasing effect with inflow, m should be high and for high detention at all depths m should be small. Again by trial, an optimum compromise between dam cost and cost of conveying away the discharge can be achieved.

MODULES

The versatility of the computer programme is enhanced by the possibility of fitting in various types of hydrological units or modules into a system. Catchments, aquifers, conduits and storage basins can be linked in any order. The various modules which can be built-in are as follows (Fig. 3).

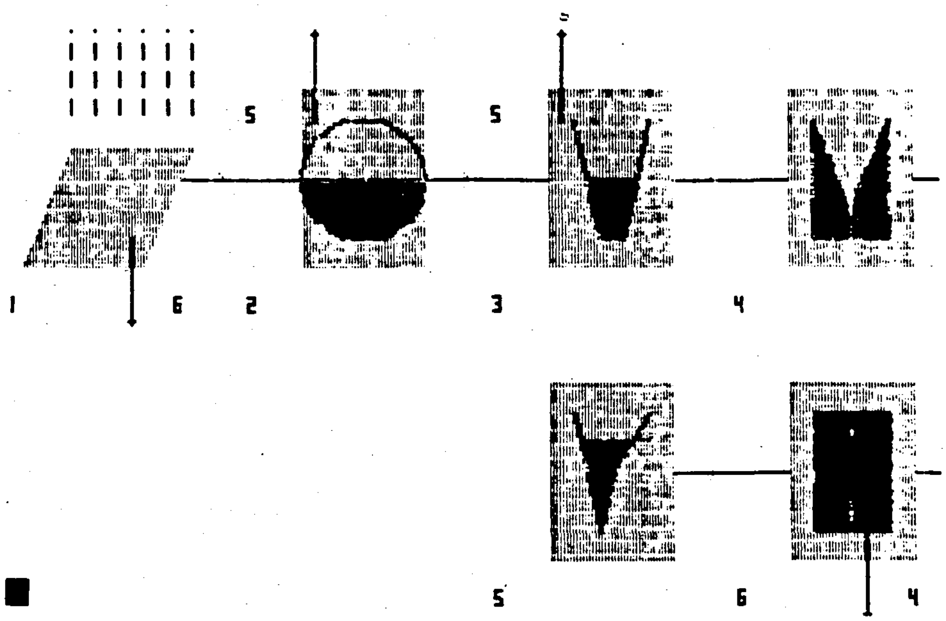

Fig. 3 Graphs output for connectivity check.

Catchments

A basic catchment is a rectangular shape sloping in one direction. The module reference number, its downstream module, initial water depth, length, width and discharge coefficient (ratio of discharge to depth to the power of 5/3) e.g. $\sqrt{(S/n)}$ where S is gradient and n is Manning roughness, are required as input data. In addition the surface permeability, suction at the ground wetting front, initial moisture content and aquifer module number are required. An infiltration process based on the soil physics model of Green and Ampt (1911) is assumed.

Catchments can be linked in cascades (in series) for example changing slopes or disconnected impervious surface, or in parallel, for instance if portion has directly connected impermeable cover.

Circular conduits (pipes)

Urbanized catchments are normally sewered with underground pipes, which run part full for most of the time. When they surcharge, the excess flow continues down roads and may be directed to channels. Such a system ("major/minor" system) is common at high flows whether intentional or not and provides roads free of ponding for all but exceptional storms. The capability of modelling such systems is therefore important.

Data required for this type of module are module reference number, downstream module, initial depth, length, diameter, conveyance ($\sqrt{(S/n)}$), and overflow module number.

Trapezoidal channels

Open channels are the most common conduits, be they roadways, gutters, ditches or canals. Where the channel is a simple trapezoid, the data requirements are limited to module reference number, downstream module, initial flow depth, length, base width, conveyance, size slopes, maximum depth and overflow module.

Compound channels

Natural channels may be defined using an arbitrary number of co-ordinates across a section. The stream between any two neighbouring points is treated as an independent section so that velocity varies depending on flow depth and roughness. Flood planes are thus accommodated with slow moving storage on the banks and a more rapid stream between banks.

Data are module reference number, downstream module, initial depth, length, slope, points, co-ordinates and roughness of each section.

This facility can be used to calculate normal depth in compound channels. An impermeable catchment upstream with an area of 3600m x 100m is fed with Rmm of catchment rain (where R is normal flow in m^3/s) and after a period of time the depth in the downstream channel stabilizes at normal depth.

Storage basins

Where detention of retention is required, on- or off- channel storage may be of use.

Data required are module reference number, downstream module, initial water depth, length, width, conveyance α, discharge depth coefficient $m(Q = W\alpha y^m)$, side slope of basin, dead storage before discharge, and crest level of dam wall. By experimenting with the outlet e.g. crest or orifice spillway, a best design may be achieved.

Aquifers

Water may infiltrate to aquifers from catchments or be discharged directly into them from any conduit or overflow. The aquifer acts as a conduit albeit with a much slower flow rate. The aquifer will also have a maximum depth and may leak to a lower aquifer. Stacking or cascading of aquifers is possible. The kinematic equations are entirely adequate for this type of flow as dynamic effects are absent.

Data include aquifer reference number, downstream module number, initial flow depth, length, width, conveyance defined as kS where k is permeability and S is gradient, porosity, aquifer depth and underneath aquifer number.

OTHER FACILITIES

A frequent source of error in stormwater programs arise when downstream catchment number is changed or forgotten. A facility exists for displaying graphically on a PC colour screen the entire network once it is entered on the computer. Each module is drawn according to the type e.g. pipe, catchment, channel, and is connected upstream and downstream as indicated in the data. Overflow routes are also indicated. In general the model is designed for easy understanding and input and cross checking. It is especially useful for stormwater management studies. The groundwater modules enable continuous simulation to be performed, which is useful for establishing antecedant moisture conditions for storms, and dry weather flows (Fig. 4).

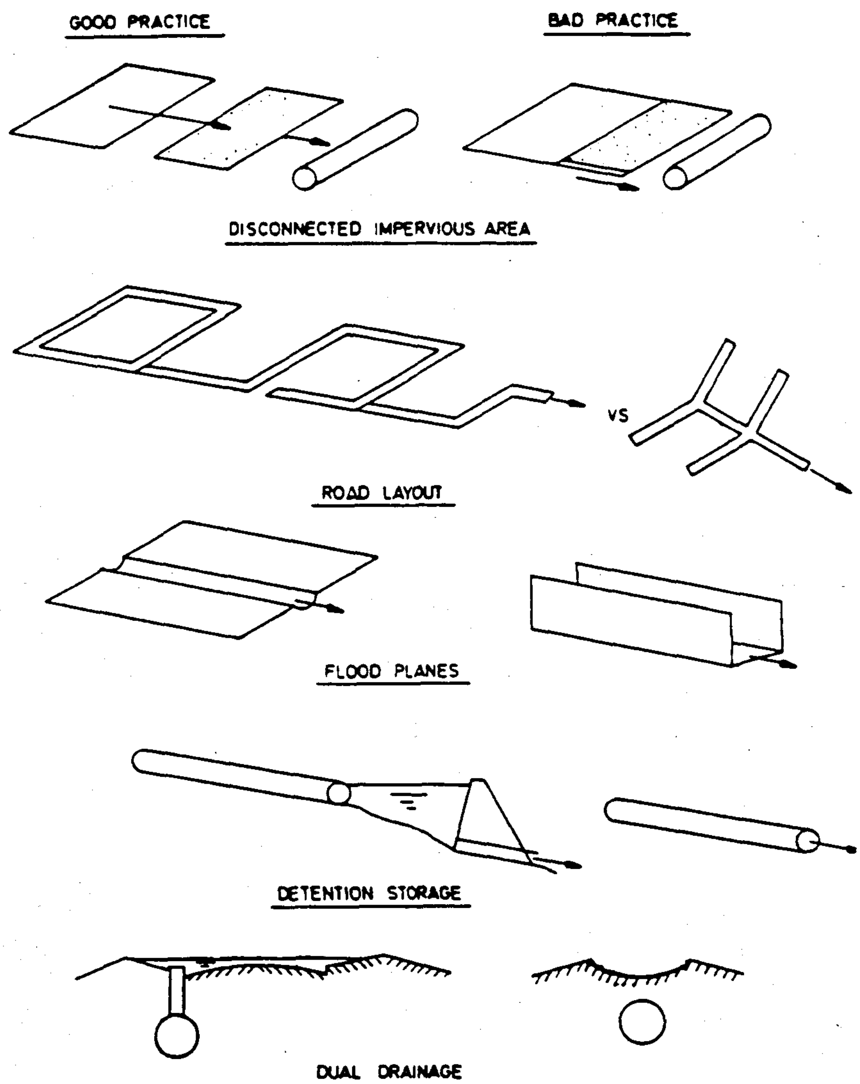

Fig. 4 Management methods investigated with model.

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Improved four-point solution of the kinematic equations

Résolution des équations des ondes cinématiques par un schéma numérique amélioré à quatre points

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SUMMARY
In finite difference solutions of the kinematic equations for overland flow, uncontrolled numerical diffusion can be introduced into a system by the finite difference scheme. A formulation for overland flow based on the Muskingum-Cunge routing procedure [4] incorporating a four-point numerical scheme is described and the results of numerical experiments are presented, comparing the method with other kinematic routing methods. The experiments verify the dependence of kinematic routing accuracy on grid spacing and on the position of the space derivative, and show that the Muskingum-Cunge results are independent of grid spacing. By comparing the conventional kinematic routing formulation with the Muskingum-Cunge formulation, it is shown that a centred scheme is the most appropriate for overland flow routing if a fixed grid formulation of the kinematic equations is employed.

RÉSUMÉ
Dans la résolution des équations des ondes cinématiques pour les ruissellements de surface, un schéma numérique aux différences finies peut introduire un coefficient de diffusion numérique non contrôlable. L'article décrit une résolution basée sur la méthode Muskingum-Cunge avec un schéma numérique à 4 points et donne les résultats d'essais de calcul comparés avec ceux d'autres méthodes de résolution. On vérifie ainsi la dépendance de la précision de la propagation des ondes cinématiques en fonction du pas d'espace de la grille de calcul ainsi que de la position de la dérivée spatiale, et on montre que les résultats de la méthode Muskingum-Cunge sont indépendants du pas d'espace. En comparant la formulation classique des ondes cinématiques à celle de la méthode Muskingum-Cunge, on constate qu'un schéma centré convient le mieux pour le calcul des ruissellements de surface si on utilise une formalation avec maillage fixe.

1 Introduction

One reason for the popularity of the kinematic equations for overland flow and stream flow simulation is the ease of solution of the equations. Both analytical and numerical solutions are much simpler than for the Saint Venant equations or even the diffusion equations. The kinematic equations are also often sufficiently accurate for stream flow, where backwater effects and flow accelerations are not important. The kinematic equations comprise the continuity equation

$$\partial q/\partial x + \partial y/\partial t = i_e \tag{1}$$

and a steady flow head gradient equation of the form

$$q = \alpha y^m, \tag{2}$$

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where q is flow per unit width, y is flow depth, x is longitudinal distance in the flow direction, t is time, i_e is excess rain, and α and m are constants depending on the flow resistance equation used. For Manning's equation, α is $(\sqrt{s})/n$ where s is the bed slope and n is Manning's roughness coefficient, and m is 5/3 if the flow is turbulent.

Equations (1) and (2) describe translation of the flow but since the flow surface is assumed parallel to the bed, there can be no hydraulic dispersion or diffusion. Dispersion is a lateral spreading of a hydrograph as it travels down a catchment or a channel reach, and diffusion is attenuation or subsidence of the hydrograph crest. Henderson [5], Stephenson and Meadows [12] and others have demonstrated mathematically that the kinematic equations model a non-dispersive, non-diffusive wave. Woolhiser and Ligget [14] developed a criterion which indicates that this assumption is good for routing flows down steep slopes with low flow depths.

Although the kinematic equations are non-dispersive and non-diffusive, the numerical solution can introduce numerical diffusion (Ponce [9]). This numerical diffusion depends on the finite difference formulation used.

2 Finite difference formulations

The basic differential equation can be re-written in finite difference form as follows:

$$\Delta q/\Delta x + \Delta y/\Delta t = i_e \tag{3}$$

Fig. 1 shows a grid representing the two dimensions of space x and time t in which the solution to the kinematic equations is carried out. θ and ϕ are weighting parameters which allow flexibility in the finite difference formulation. The most general formulation is a four-point scheme in which the flow conditions (flow rate and flow depth) at the present time step are calculated as weighted functions of the conditions at the four corners of each computational cell. The continuity equation (3) can be expressed as follows:

$$[\phi(q_3 - q_1) + ((1-\phi)(q_4 - q_2)]/\Delta x + [\theta(y_2 - y_1) + (1-\theta)(y_4 - y_3)]/\Delta t = i_e \tag{4}$$

ϕ is a time position weight and θ is a longitudinal space weight. $\phi = 0$ results in an implicit scheme and $\phi = 1$ is an explicit scheme. $\theta = 0$ is a backward difference and $\theta = 1$ is a forward difference. This scheme was originally suggested by Preissmann [11] for numerical modelling of the full one-dimensional hydrodynamic equations.

Weinmann and Laurenson [13] considered the effects of different θ values when the temporal derivative is centred, i.e. $\phi = 0.5$. They showed that under these conditions θ must lie between 0 and 0.5 for a valid solution. They maintained that using $\theta = 0.5$ results in a pure kinematic wave without any attenuation (if the grid spacing is chosen correctly), and $\theta = 0$ results in numerical diffusion which causes an attenuation of a flood wave. Presumably, for values of θ between 0 and 0.5, varying degrees of numerical diffusion are introduced, although there is no indication in the literature as to how the numerical diffusion is affected by intermediate θ-values in this range.

Brakensiek [1] compared three numerical schĕmes, all using $\theta = 0.5$, namely explicit, implicit and a centred scheme, and deduced that the latter two were more stable than the explicit scheme. He further illustrated that the use of an implicit scheme does not require simultaneous solution of equations for q and y at the new point in time. Proceeding from the upstream catchment boundary, one can calculate y and q at successive grid points down the catchment, and repeat the process for each new time step. He did not investigate the effect of grid spacing on accuracy and

stability. Chaudhry [2] used a centred scheme ($\theta = 0.5$) for both an explicit and an implicit formulation, and found that it worked well for both formulations. He called the explicit centred scheme a diffusion scheme.
For the Saint Venant equations and diffusion equations it is essential that θ is greater than zero since disturbances downstream will be felt upstream unless the flow is supercritical, i.e. backwater effects must be modelled. In the case of the kinematic equations, only downstream translations of water depths are allowed to occur. Hydraulically therefore there is no need for forward differences. Numerically however there is an advantage in using θ values greater than zero, as illustrated later.
The numerical scheme to be described below is a refinement on most finite difference schemes for the kinematic equations, in that θ is varied over a range. ϕ is set as 0.5 since this results in the average flow conditions at the centre of the time increment being calculated. A formulation incorporating a varying weighting parameter θ was recommended by Ponce [9] because the numerical diffusion is matched to the physical diffusion by correct choice of θ. The central difference ($\theta = 0.5$) is found to reproduce hydrodynamic diffusion most accurately for the kinematic equations, as indicated by comparison with a Muskingum-Cunge solution with varying θ.

3 Grid geometry

Overton and Meadows [8] showed that for numerical stability, the grid spacing $\Delta x/\Delta t$ should equal or exceed a critical value given by the equation

$$\Delta x/\Delta t \geq dx/dt = dq/dy = amy^{m-1} \tag{5}$$

Constantinides [3] illustrated that for any Δx, the accuracy of a kinematic finite difference formulation decreases with increasing $\Delta x/\Delta t$, i.e. as Δt decreases. This was ascribed to numerical diffusion whereby the extremes of a wave extend at a rate $\Delta x/\Delta t$ irrespective of the true dx/dt. For any $\Delta x/\Delta t$ the accuracy also drops off as Δx increases since the grid becomes too coarse. This dependence of accuracy on grid spacing must be taken into account when comparing different numerical schemes.

4 Muskingum-Cunge routing

Cunge [4] proposed an explicit finite difference scheme which was reported by Weinmann and Laurenson [13] in their review of flood routing methods. Consider the continuity equation:

$$\partial Q/\partial x + \partial A/\partial t + q_i \tag{6}$$

where Q is the flow rate, A is the cross-sectional flow area, and q_i is the inflow term. This equation can be expressed in terms of Q only as follows:

$$\partial Q/\partial x + (dA/dQ)(\partial Q/\partial t) = q_i \tag{7}$$

It may be shown that dQ/dA is the celerity c of a flood wave. Hence this equation can be written:

$$\partial Q/\partial x + (1/c)(\partial Q/\partial t) = q_i \tag{8}$$

By centring the time derivative (i.e. putting $\phi = 0.5$) and keeping θ a variable, this can be expressed in finite differences as follows (referring to Fig. 1 for the positions of the points):

$$\langle 1/c\rangle[\theta(Q_2 - Q_1) + (1-\theta)(Q_4 - Q_3)]/\Delta t + [Q_3 + Q_4 - Q_1 - Q_2]/2\Delta x = q_i \tag{9}$$

where $\langle\,\rangle$ indicates an average value. Note that the *average* celerity in the computational cell is used, since it is difficult to isolate the celerity as applying to any one of the four corner points of the grid in Fig. 1. It is assumed that $1/\langle c\rangle$ is approximately equal to $\langle 1/c\rangle$. For overland flow the inflow q_l is equivalent to $i_e \Delta A$ where i_e is excess rain and ΔA is the surface area of the catchment between points x and $x - \Delta x$ (see Fig. 2). By solving this equation for Q_4 the following equation is obtained:

$$Q_4 = C_1 Q_1 + C_2 Q_2 + C_3 Q_3 + C_0 i_e \Delta A \tag{10}$$

where

$$C_0 = 2\Delta t/[\Delta t + 2K(1-\theta)] \tag{11a}$$

$$C_1 = [\Delta t + 2K\theta]/[\Delta t + 2K(1-\theta)] \tag{11b}$$

$$C_2 = [\Delta t - 2K\theta]/[\Delta t + 2K(1-\theta)] \tag{11c}$$

$$C_3 = [2K(1-\theta) - \Delta t]/[\Delta t + 2K(1-\theta)] \tag{11d}$$

$$C_1 + C_2 + C_3 = 1.0 \tag{11e}$$

$$K = \Delta x/\langle c\rangle \tag{12}$$

The unit of K is time and it is approximately equal to the travel time of a flood wave through a distance Δx. The form of equations (10) to (12) is identical to the equations for Muskingum routing of floods in rivers (Ponce and Yevjevich [10]).

Cunge [4] showed that equations (10) to (12) constitute a second order approximation to diffusion routing if the weighting coefficient θ is found from

$$\theta = \tfrac{1}{2}(1 - q/sc\,\Delta x), \qquad 0 \le \theta \le 0.5 \tag{13}$$

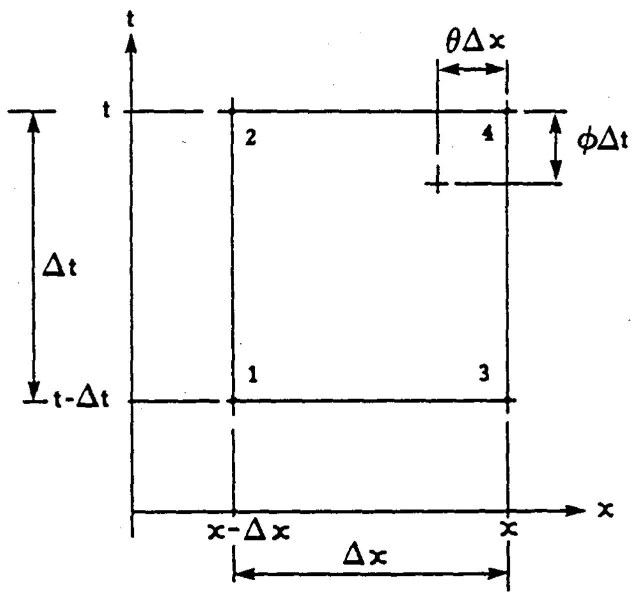

Fig. 1. Finite difference grid showing a computational cell.
Schéma de calcul aux différences finies.

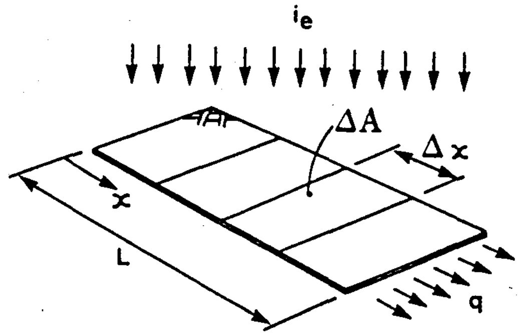

Fig. 2. Rectangular sloping plane subject to steady excess rainfall.
Surface plane rectangulaire et en pente soumise à des précipitations constantes.

where q is the flow per unit width, s is the bed slope and c the celerity. Two subsequent papers have laid a foundation for the potential use of the Muskingum-Cunge method in overland routing and channel routing. Ponce and Yevjevich [10] provided for K and θ to vary in time and space as the flow varies, instead of the conventional approach in Muskingum routing which is to define average values of K and θ for an entire flood event. Ponce [9] applied Muskingum-Cunge routing to overland flow and compared it with traditional kinematic routing. Ponce referred to this as a "matched diffusivity" approach since attenuation is artificially modelled by choosing θ for each computational step such that the numerical and physical diffusion are matched.

A formulation for MC routing necessitates the calculation of the average values of q and c for each computational cell for use in calculating K and θ. The arithmetic averages can be expressed as follows, where c_1, c_2, c_3 and c_4 represent the celerities at grid points 1, 2, 3 and 4 respectively, and q_1, q_2, q_3 and q_4 represent the unit width discharges at these grid points:

$$\text{Implicit:}\quad q = \tfrac{1}{4}(q_1 + q_2 + q_3 + q_4) \tag{14}$$

$$c = \tfrac{1}{4}(c_1 + c_2 + c_3 + c_4) \tag{15}$$

$$\text{Explicit:}\quad q = \tfrac{1}{4}(q_1 + q_2 + 2q_3) \tag{16}$$

$$c = \tfrac{1}{4}(c_1 + c_2 + 2c_3) \tag{17}$$

Equations (14) and (15) imply an implicit iterative solution since flow conditions at grid point 4 are unknown. In equations (16) and (17) a weighted average in favour of grid point 3 has been used to compensate for the missing flow and celerity q_4 and c_4 at the downstream end of the segment. q can be expressed as the flow per unit width of the overland flow plane:

$$q_i = Q_i/B \qquad (i = 1, 2, 3 \text{ or } 4) \tag{18}$$

The celerity is found by differentiating equation (2) with respect to flow depth y:

$$c = \mathrm{d}q/\mathrm{d}y = \alpha m y^{m-1} \tag{19}$$

Substituting (19) into equations (15) and (17) yields:

$$\text{Implicit:}\quad c = \tfrac{1}{4}\alpha m(y_1^{m-1} + y_2^{m-1} + y_3^{m-1} + y_4^{m-1}) \tag{20}$$

$$\text{Explicit: } c = \tfrac{1}{4}am(y_1^{m-1} + y_2^{m-1} + 2y_3^{m-1}) \tag{21}$$

Using equation (2), similar expressions can be obtained for the average unit width discharge q. Substituting the expressions for q and c into equation (13) yields:

$$\theta = \tfrac{1}{2}[1 - (1/m)(\langle y^m \rangle / \langle y^{m-1} \rangle)(1/s\,\Delta x)]; \qquad 0 \le \theta \le 0.5 \tag{22}$$

where $\langle y^m \rangle$ and $\langle y^{m-1} \rangle$ are given by:

$$\text{Implicit: } \langle y^m \rangle = \tfrac{1}{4}(y_1^m + y_2^m + y_3^m + y_4^m) \tag{23}$$

$$\langle y^{m-1} \rangle = \tfrac{1}{4}(y_1^{m-1} + y_2^{m-1} + y_3^{m-1} + y_4^{m-1}) \tag{24}$$

$$\text{Explicit: } \langle y^m \rangle = \tfrac{1}{4}(y_1^m + y_2^m + 2y_3^m) \tag{25}$$

$$\langle y^{m-1} \rangle = \tfrac{1}{4}(y_1^{m-1} + y_2^{m-1} + 2y_3^{m-1}) \tag{26}$$

Similarly equation (10) for K becomes:

$$K = \Delta x / am\langle y^{m-1} \rangle \tag{27}$$

where $\langle y^{m-1} \rangle$ is given by equation (24) or (26).

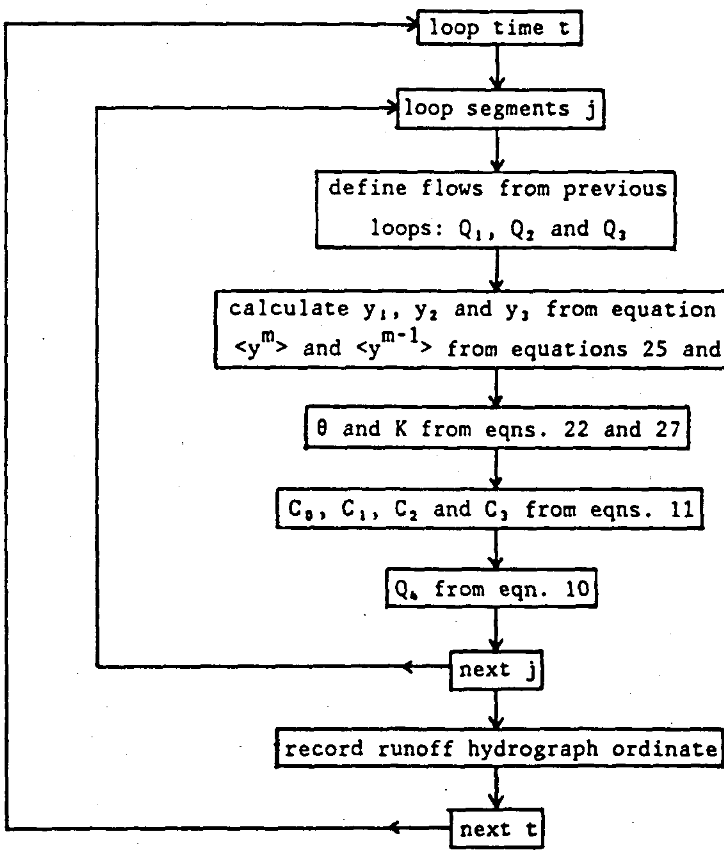

Fig. 3. Flow diagram for explicit non-iterative MC routing algorithm.
Organigramme de l'algorithme par la méthode explicite non itérative MC.

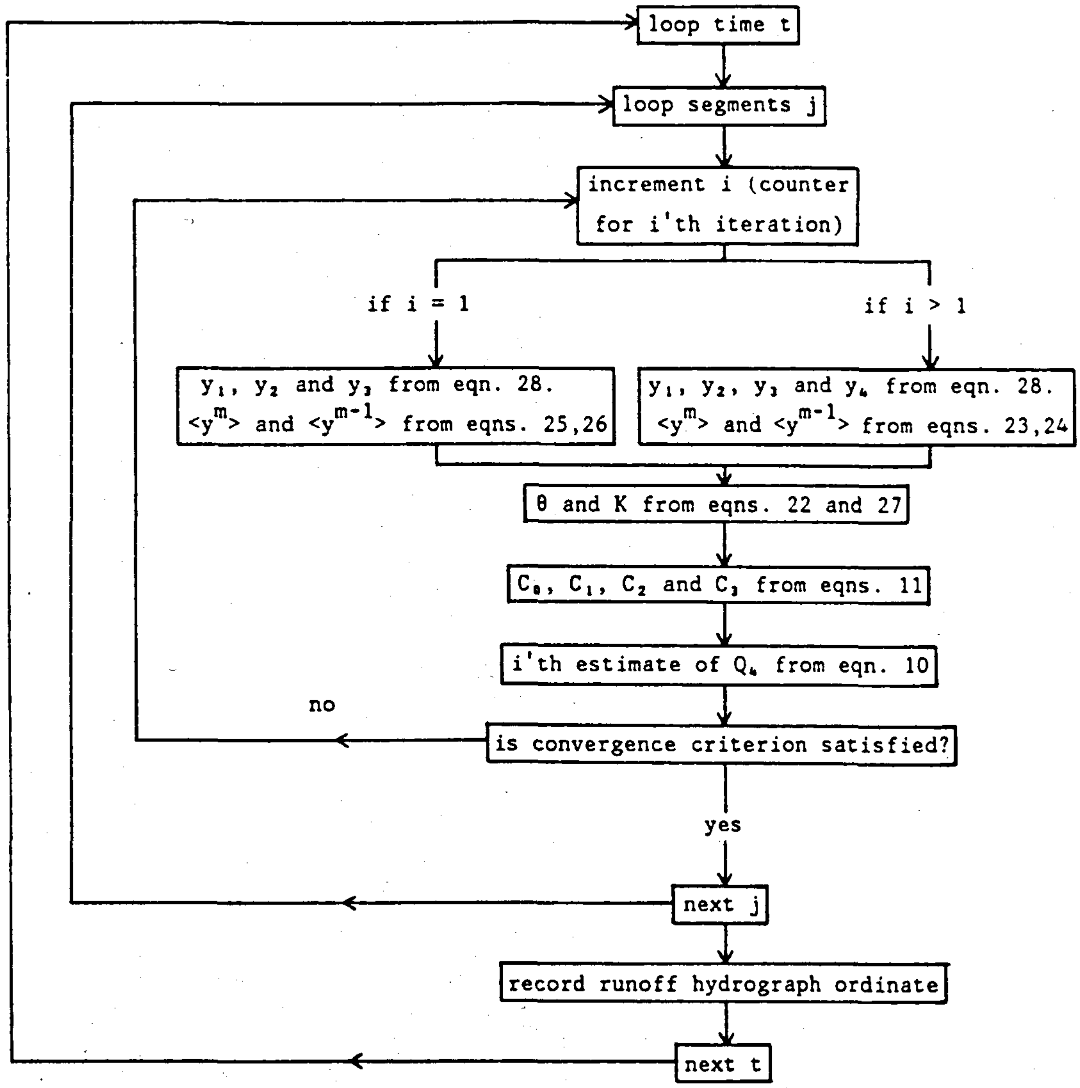

Fig. 4. Flow diagram for implicit iterative MC routing algorithm.
Organigramme de l'algorithme de la méthode implicite itérative MC.

The flow depth y can be found from equation (2) which can be written as:

$$y_i = (q_i/\alpha)^{1/m} = (Q_i/B\alpha)^{1/m} \qquad (i = 1, 2, 3, 4) \tag{28}$$

where B is the width of the overland flow plane. Equations (22)-(28) comprise a complete description of K and θ for either an explicit or an implicit scheme. K and θ are calculated for each computational cell and then used in conjunction with equations (10) and (11) for Muskingum-Cunge routing of overland flow. The algorithms for the iterative implicit and non-iterative explicit methods are illustrated by the flow diagrams in Figs. 3 and 4 respectively.

In order to compare results for different numerical approaches, these algorithms were used in routing runoff from the uniform rectangular plane depicted in Fig. 2. For the first time step the flow depths y_1 and y_3 were zero because the plain was assumed initially dry. In addition at $x = 0$, y_2 is also zero. Hence for the explicit solution, equations (25) and (26) would yield zero which would prevent computation of θ and K (equations (22) and (27)) because of division by zero. For this reason the non-iterative routing utilises the iterative routing algorithm for the first time step and thereafter reverts to the non-iterative explicit algorithm.

The Muskingum-Cunge equations can be seen as a special case of the kinematic equations (Weinmann and Laurenson [13]), but this special case, together with a variable θ, permits an accurate numerical solution. In order to distinguish between conventional kinematic modelling with fixed θ, and the special case of varying θ, the latter is referred to here as Muskingum-Cunge (MC) routing.

A comparison of MC routing and conventional routing is made below using a uniform precipitation over the plane. A formulation for conventional kinematic routing was obtained from equation (4) by using $\phi = 0.5$, keeping θ a variable, and solving for y_4:

$$y_4 = i_e \Delta t/(1-\theta) + y_3 + [\theta/(1-\theta)](y_1 - y_2) + \tfrac{1}{2}(\Delta t/\Delta x)(q_1 + q_2 - q_3 - q_4)/(1-\theta) \quad (29)$$

where

$$q_i = \alpha y_i^m \qquad (i = 1,2,3,4) \quad (30)$$

Since q_4 is a function of y_4 this equation is implicit and must be solved iteratively for y_4.

5 Numerical experiments

Two extreme overland conditions were investigated: a steep, smooth plane with slope $s = 0.1$ and Manning's $n = 0.01$, and a gentle, rough plane with $s = 0.01$ and $n = 0.1$. The former exhibits a quick response to rainfall input and the latter a slow response. These two conditions span a range of overland flow conditions. For all experiments a constant excess rainfall of 50 mm/h was used. Plane dimensions were set at length $L = 100$ m and breadth $B = 20$ m. The lowest value of $\Delta x/\Delta t$ for each experiment was chosen as close to the critical value as possible. (Critical values for numerical stability were found by trial). The Δx-increment was set at 20 m corresponding to fine discretisation. Δx was held constant and the grid spacing $\Delta x/\Delta t$ made progressively larger by decreasing Δt. For each value of $\Delta x/\Delta t$ the runoff hydrograph was generated using each of the three routing methods described above: conventional kinematic formulation (equation (29)), explicit MC routing and implicit MC routing. For the MC routing θ was varied in accordance with equation (13). The peak flows, which coincided with the end of the storm event, are plotted in Fig. 5 as functions of the grid spacing.

Examining Figs. 5(a) and (b), it can be seen that the results of the conventional kinematic routing with constant θ show marked dependence on grid spacing, whereas those of the MC routing with varying θ are virtually independent of grid spacing. It is also evident that the lower the value of θ in kinematic routing, the lower is the peak runoff, indicating that the closer θ is to zero, the more attenuation is introduced in the form of numerical diffusion.

It was found that the values of θ for the MC routing were very close to 0.5 for all flow conditions, θ ranging from about 0.47 to 0.5. (The writers' experience has been that when applying MC routing to river channels, θ drops significantly lower than this because the more gentle bed slopes result in greater attenuation of a flood hydrograph). This explains why the results for the conventional kinematic routing using θ fixed at 0.5 are close to those for the MC routing in Fig. 5. It can

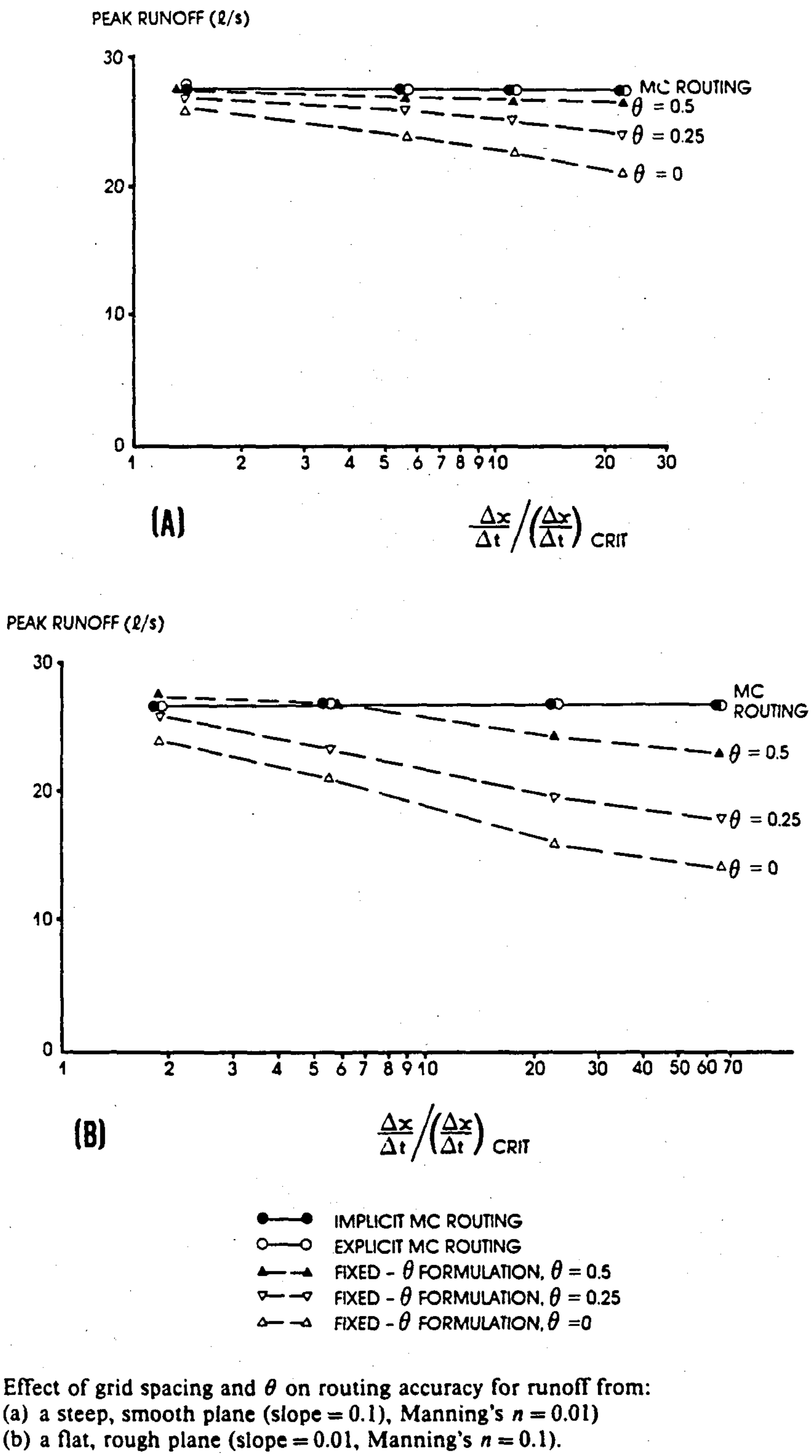

Fig. 5. Effect of grid spacing and θ on routing accuracy for runoff from:
(a) a steep, smooth plane (slope = 0.1), Manning's $n = 0.01$)
(b) a flat, rough plane (slope = 0.01, Manning's $n = 0.1$).
Effet de la variation du pas d'espace et du coefficient θ sur la précision du calcul de la propagation:
(a) sur une surface lisse en pente raide, (pente = 0,1, coefficient de Manning $n = 0{,}01$)
(b) sur une surface rugueuse en pente douce, (pente = 0,01, coefficient de Manning $n = 0{,}1$).

be concluded that for overland flow θ should be close to 0.5 and this value can be adopted in schemes that do not allow for a variable θ.

The explicit and implicit MC routing algorithms show almost identical results. Consequently it is safe to use the explicit routing, which saves on computation time.

It can be noted that the peak flow decreases more for smaller $\alpha(=(\sqrt{s})/n)$ as is to be expected since there will be more channel storage. The accuracy of the routing could be gauged by comparing with a full hydrodynamic solution. Criteria for deciding the accuracy of the kinematic solution were presented by Stephenson and Meadows [12].

6 Conclusions

1. It is confirmed that for traditional kinematic routing with constant θ, the accuracy of the solution is dependent on the grid spacing, and uncontrolled numerical diffusion is introduced into the system by the choice of the parameter θ in the finite difference formulation. This numerical diffusion increases as θ approaches zero and as grid spacing $\Delta x/\Delta t$ increases.
2. A formulation for Muskingum-Cunge routing [4] adapted to overland flow is described in detail and termed "MC routing". As pointed out by Ponce [9], it has the advantage that attenuation is modelled by matching the numerical diffusion to the physical attenuation with the correct choice of the parameter θ. This formulation was shown in this paper to be independent of grid spacing. There does not appear to be any advantage in the implicit formulation of the MC equations and the explicit formulation has the advantage of economy in computer time.
3. Comparison of the conventional type of finite difference formulation for the kinematic equations using constant θ, with the Muskingum-Cunge formulation, shows that a central difference ($\theta = 0.5$) is most accurate for overland flow, although the accuracy declines for large $\Delta x/\Delta t$. Alternatively the MC formulation described above can be used as an accurate method for overland routing for all grid spacings greater than the critical grid spacing for numerical stability.

Notations

A	cross-sectional area of flow
B	width of overland flow plane
c	wave celerity
c_1, c_2, c_3, c_4	celerities at grid points 1, 2, 3 and 4 respectively
$\langle c \rangle$	average celerity in a computational cell
C_0, C_1, C_2, C_3	Muskingum coefficients
j	counter for overland flow segment
i_e	excess rain intensity
K	constant in Muskingum routing, $=\Delta x/\langle c \rangle$
L	length of overland flow plane
m	exponent in uniform flow equation
n	Manning's roughness coefficient
Q	flow rate
Q_1, Q_2, Q_3, Q_4	flows at grid points 1, 2, 3 and 4 respectively
q	flow rate per unit width of overland flow plain
q_1, q_2, q_3, q_4	flows per unit width at grid points 1, 2, 3 and 4 respectively

q_i	inflow term
s	bed slope
t	time
Δt	time increment
x	longitudinal distance in direction of flow
Δx	space increment
y	flow depth
y_1, y_2, y_3, y_4	flow depths at grid points 1, 2, 3 and 4 respectively
α	coefficient in uniform flow equation, $=\sqrt{s}/n$ in Manning's equation
θ	space weighting in finite difference grid
ϕ	temporal weighting in finite difference grid

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Model study of the effect of temporal storm distributions on peak discharges and volumes

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ABSTRACT A series of synthetic five-year return period storms having rectangular, triangular and bimodal (triangular) temporal profiles was compiled from depth-duration-frequency (D-D-F) relationships based on (a) standard sorted clock time depths and (b) complete storms only. These were used as inputs to a catchment runoff model to simulate the peaks and volumes of runoff from an actual and a hypothetical catchment respectively. The hypothetical catchment was used to prove the adequacy of the simplified hyetographs and the real catchment to demonstrate the difference in runoff hydrographs for the various hyetographs. The results were compared to establish the effect of storm profiles on peak and volume of runoff as well as to indicate the shortcomings of D-D-F relationships derived by conventional methods. In general, on the assumption that the model correctly converts storm input to runoff, triangular, and in particular bimodal, profiles were shown to reproduce adequately runoff peaks and volumes.

Etude par un modèle, des effets de différents types d'averses périodiques sur les débits de pointe et les volumes d'écoulement
RESUME Une série hypothétique d'averses de période de retour égale à cinq ans, et ayant des hyétogrammes rectangulaires, triangulaires et triangulaires à deux modes ont été compilés à partir des relations existantes entre hauteurs, durée et fréquence (relations D-D-F en anglais). Ces relations sont basées sur (a) les hauteurs de précipitations pour des intervalles de temps standards et (b) pour des averses entières. Les suites d'averses ainsi constituées ont été utilisées comme données dans un programme mettant en modèle le débit d'écoulement provenant d'un bassin hydrographique. Ceci afin de simuler les débits de pointe et les volumes d'écoulement issus d'un bassin versant réel et théorique respectivement. Le bassin versant théorique servait à établir la validité des simplifications admises pour les graphiques donnant, pour la durée d'un orage, l'intensité de la pluie, sur le bassin de réception, en fonction du temps (hyétogrammes). Le bassin versant réel servait à mettre

Open for discussion until 1 December 1987.

en évidence les variations dans les graphiques donnant le débit d'écoulement, à l'exutoire du bassin, en fonction du temps pour divers hyétogrammes. Les résultats obtenus ont été comparés entre eux d'une part, afin d'établir l'effet des divers types de hyétogrammes d'averses sur les débits de pointe et volumes d'écoulement et d'autre part, pour mettre en évidence les insuffisances des relations entre chute de pluie, durée et fréquence obtenues par les méthodes conventionnelles. En général, avec l'hypothèse que le modèle utilisé représente correctement l'écoulement pour un averse donnée, il est démontré que les profils triangulaires, en particulier ceux à deux modes, reproduisent de manière efficace les débits de pointe et volumes d'écoulement.

INTRODUCTION

Designers require reasonably accurate estimates of the characteristics of runoff hydrographs. For conduit sizing one is generally interested in the peak discharge, but runoff volume is also important for stormwater management. Correct selection of both hyetograph and "losses" is important in the derivation of the runoff hydrograph. In this paper, hydrographs of runoff from a catchment in South Africa are generated by the use of the WITWAT input-output model to compare the effects of different time distributions of rainfall input on the hydrograph peak and volume.

THE DESIGN PROBLEM

Given the catchment characteristics and historical information on rainfall inputs, the peak rate and volume of runoff from an ungauged catchment can be estimated by means of a suitable model describing the rainfall-runoff process. A design storm can be described in terms of its total duration, total depth of rainfall and a temporal or hyetograph shape parameter or series of parameters. Depth-duration-frequency (D-D-F) relationships coupled with shape provide a quick means of constructing a design storm. The designer selects the desired frequency of occurrence and critical storm duration (the latter either by trial or by an analytical technique) and reads the total depth from the D-D-F curves. The resulting hydrograph, however, can vary significantly depending upon the shape of the rainfall profile, as has been demonstrated by Akan & Yen (1984)

DEPTH-DURATION-FREQUENCY RELATIONSHIPS

D-D-F curves, e.g. those of Midgley & Pitman (1978) which are in fairly common use, were derived using an accepted technique. Rainfall data were discretized in short time intervals (usually five minutes). The data were then analysed for arbitrarily defined durations (e.g. 5, 10, 15, ..., 120 minutes). For each defined storm duration, the maximum rainfall recorded within that duration is abstracted (Fig.1). It should be noted, however, that the

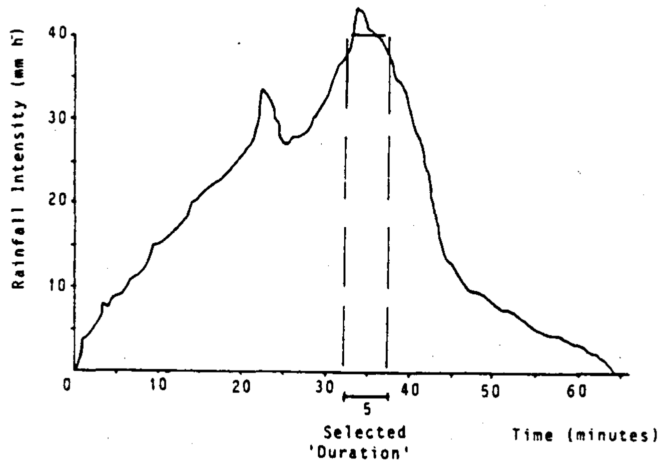

Fig. 1 Conventional method of D-D-F curve derivation.

prescribed durations bear no relation to those of the actual storm and may be embedded in events of longer durations or may bracket shorter ones. Furthermore the method takes no account of the time profile of the event. The preceding and succeeding rain can well have a significant effect on the soil moisture state and therefore on the consequent runoff.

To overcome the deficiencies, D-D-F curves were revised by taking into account the natural storm durations as follows.

(a) A matrix of storm duration classes was established, with storm depths within each class ranked in order of decreasing magnitude, and from it the D-D-F curves were produced. The technique ensures independence of samples.

(b) Where the data set was small a slightly different method had to be employed. This involves fitting a storm profile to each natural event to define the storm duration. Synthetic hyetographs could then be generated for a range of durations (Lambourne, 1985).

Four D-D-F relationships were selected for this study, namely those derived by:

(a) the conventional method described above, using set 15 minute intervals;

(b) the conventional method using digitized autographic data, processed with an arbitrary storm start time;

(c) a triangular profile approximated to storm events; and

(d) a bimodal profile (triangular shapes) approximated to storm events.

INTENSITY PROFILES

Three storm profiles or shapes were used to represent a range of hyetograph patterns. These embrace the extremes in rainfall distribution of those encountered and are:

square-topped;
triangular; and
bimodal (triangular shapes).

They are depicted in Fig.2. Distribution A was used with D-D-F relationships 1 and 2, B with 1 and 3, and C with 4. Thus, five storms resulted for each duration.

The triangular shape was first suggested by Yen & Chow (1980) as a suitable profile of rainfall intensity. For a specific duration the peak intensity is twice that derived from the square-topped profile. Positioning of the peak in relation to the duration of the storm event is based on the results of statistical analysis of recorded storm data. An average position for South African conditions is 0.39 times the duration measured from the commencement of the storm (Lambourne, 1985). Both Constantinides (1982) and Sutherland (1983) adopted the triangular profile for use in South African design practice.

A storm event is generally composed of several cells passing across a catchment (rare occurrences of a uni-cellular storm have been recorded). At a point, these cells produce a series of marked periods of high intensity rainfall which can be modelled by a series of triangles. A bimodal profile (two triangular shapes) was found to fit South African data (Lambourne, 1985). The shape parameters, including time to peak and durations of first and second triangles, were established from statistical analysis of historical records.

TEST OF ADEQUACY OF SYNTHETIC STORM PROFILES

A comparative study was undertaken to test the adequacy with which actual storm hyetographs could be approximated by synthetic triangular and bimodal profiles. A 10-year record of storm rainfall (with each storm greater than 10 mm depth) was available from the autographic raingauge nearest to the test catchment i.e. situated at Potchefstroom (90 km WSW of Johannesburg), and this was used in the analysis. From the data set, 125 events, i.e. periods of continuous significant rainfall, were approximated by the triangular and the bimodal shapes respectively using the moment method detailed by Lambourne (1985).

With these as input to the WITWAT model (Green, 1984), runoff hydrographs were generated and compared with those generated from the actual hyetographs. If the WITWAT model could be assumed to generate the flood correctly in the hydraulic sense, then the analysis would show which synthetic hyetograph could best reproduce the actual hydrograph.

Two different catchment surfaces were modelled, one impermeable and the other permeable. The former was modelled as V-shaped rectangular planes with 1 mm depression storage but impervious. The second was plane, with various values of infiltration rate and depression storage.

The ratios of hydrograph peak from synthetic hyetographs to hydrograph peak from actual hyetographs were calculated for the 125 events, and the mean and standard deviation of these ratios evaluated. The ratios of the volumes were treated in the same way. A dimensionless efficiency of fit parameter, based on the Nash & Sutcliffe (1970) model, was also evaluated i.e.

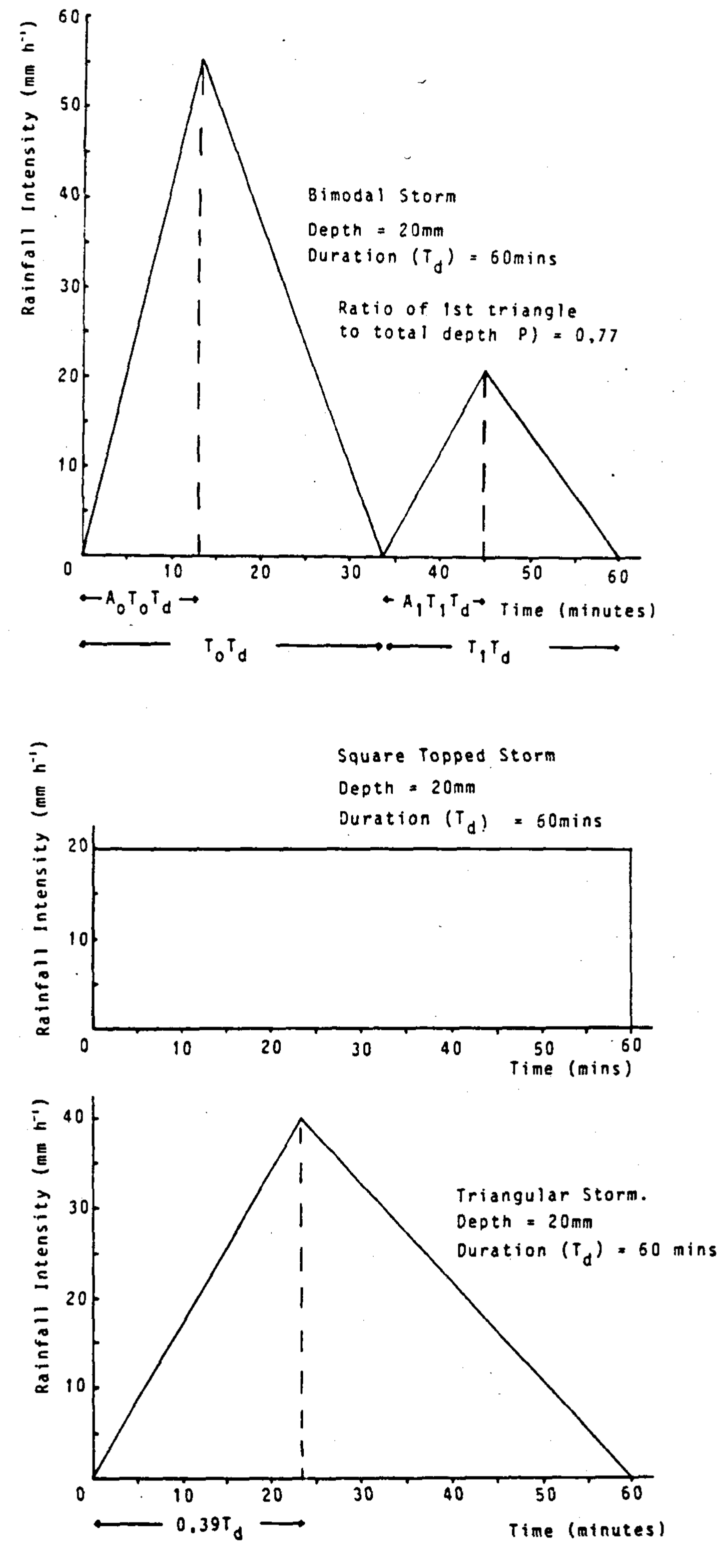

Fig. 2 Distribution of total rainfall during a storm event.

$$R^2 = \frac{F_o^{\ 2} - F^2}{F_o^{\ 2}} \tag{1}$$

where

$$F^2 = \Sigma(q_s(t) - q_o(t))^2 \tag{2}$$

$$F_o^{\ 2} = \Sigma(q_o(t) - \bar{q}_o)^2 \tag{3}$$

in which $q_o(t)$ is the measured discharge from an observed storm, $q_s(t)$ the discharge generated from the synthetic profile, and $\bar{q}_o$ the mean observed discharge.

It will be seen from Table 1 that both synthesized profiles (triangular and bimodal) perform well for the two synthetic catchments. However, in all cases the bimodal profile has smaller standard deviations and gives a slightly high (conservative) estimate.

Table 1 Applicablity of design storm shapes (Potchefstroom data)

	Comparison for impermeable catchment:					Comparison for permeable catchment:			
	Triangular		Bimodal			Triangular		Bimodal	
Statistic	Mean	SD	Mean	SD	Statistic	Mean	SD	Mean	SD
Peak ratios	1.02	0.159	1.02	0.157	Peak ratios	1.03	0.239	1.01	0.236
Volume ratios	0.91	0.119	1.04	0.083	Volume ratios	1.02	0.233	1.07	0.173
Efficiency R^2	0.91	0.114	0.91	0.085	Efficiency R^2	0.87	0.130	0.87	0.116

When considering the results, it must be borne in mind that the runoff hydrograph is a function of storm and catchment variables, and for an actual catchment the results may be different.

DESIGN APPLICATION OF SYNTHETIC HYETOGRAPHS

The runoff characteristics derived from the four selected D-D-F relationships were compared to assess the effect of the rainfall profile on volume and peak discharge. The representative urbanized catchment chosen for this purpose is situated in Vanderbijlpark, some 50 km southwest of Johannesburg, and covers an area of 142 ha reflecting a range of land uses from light residential to townhouse development and from light commercial to medium industrial. Figure 3 is a plan of the catchment and major stormwater drainage system. Being a "small" catchment, critical storm duration for maximum peak discharge would be likely to be the same as concentration time (Stephenson & Meadows, 1986). Average percentage imperviousness is estimated to be 25% and ground slopes vary from 0.005 to 0.02 m m^{-1}. A pipe and channel system drains the impervious areas to a small detention pond in the southeast corner of the catchment.

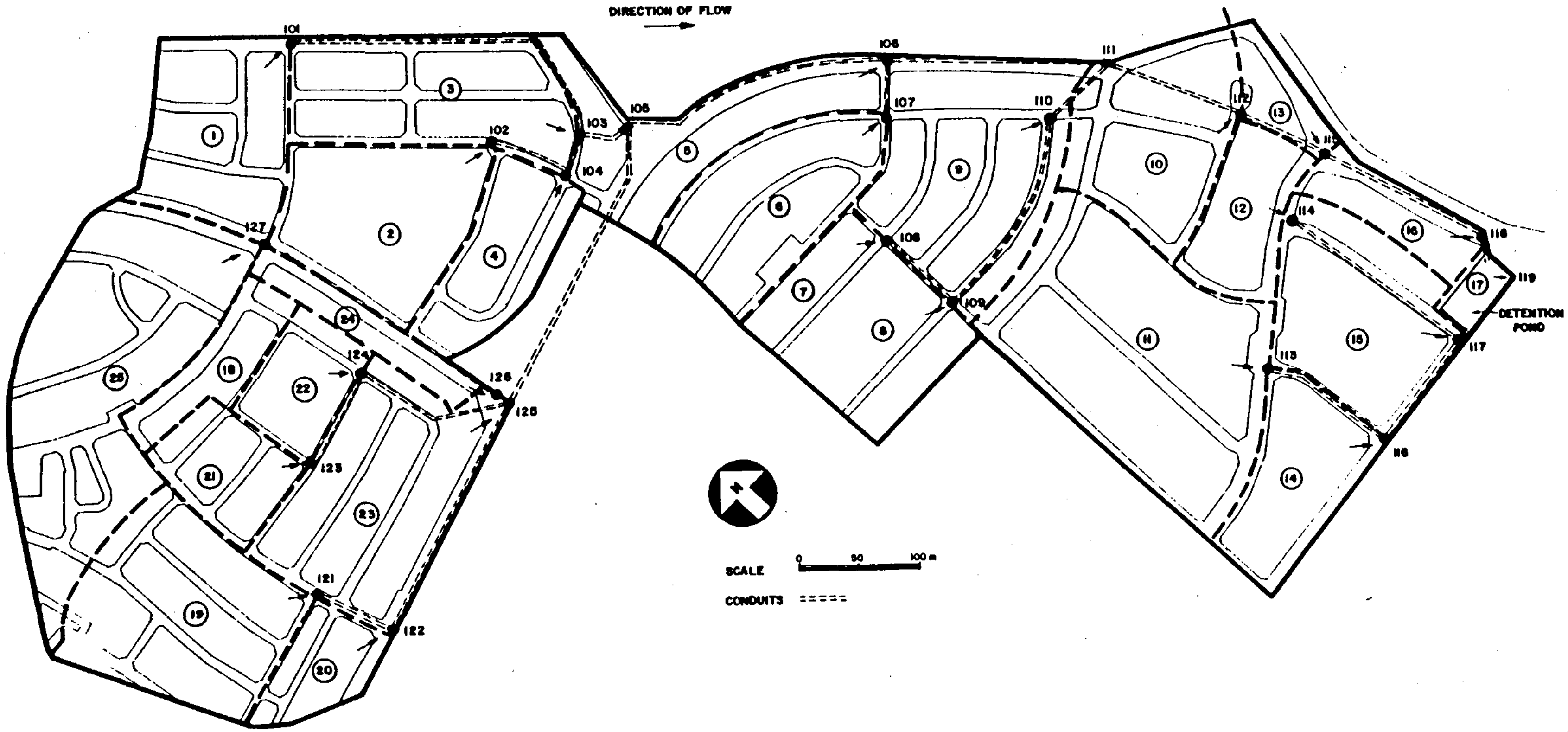

Fig. 3 Plan of Vanderbijlpark catchment.

The catchment was divided into 25 sub-catchments according to land use and overland flow drainage pattern. The simulation program WITWAT does not allow for dual drainage systems and so the five-year return period storm was adopted to ensure that the stormwater would be accommodated by the existing pipe and gutter network. WITWAT uses kinematic routing for overland flows and time-shift for closed conduits. A routine computes the design hyetograph on the basis of equations developed by Op ten Noort & Stephenson (1982) for describing the D-D-F relationships of Midgley & Pitman (1978). For the comparison, design storms having both square-topped and triangular profiles were computed.

The adapted infiltration function was the Horton equation modified by Green (1984) to account for the interruption of the abstraction decay during intensities lower than the capacity rate.

Five different rainfall patterns having various durations were simulated. Hyetographs for each specific duration were derived in two steps:

(1) The total depths for various durations were taken from the D-D-F relationships of Midgley & Pitman (1978) and Lambourne (1985) respectively and these are presented in Table 2.

(2) The peak of the triangular profile was taken as twice the height of the square-topped hyetograph and the time-to-peak 0.39 times the total duration (Lambourne, 1985). Shape parameters of the bimodal storm (Fig.2) are given in Table 3.

The corresponding runoff hydrographs were generated by WITWAT and the resulting peaks and volumes of runoff plotted in Fig.4. The

Table 2 Total depths from D-D-F curves (examples) MAP = 685 mm (derived for Vanderbijlpark region)

Duration (minutes)	Midgley/Pitman (mm)	Updated M/P* (mm)	Triangular (mm)	Bimodal (mm)
15	23.54	27.00	27.04	20.40
30	32.62	32.75	37.29	30.57
45	37.76	37.09	43.36	37.99
60	41.21	40.70	46.62	43.67
90	45.72	46.65	50.35	51.09
120	48.69	52.26	52.68	55.90
150	50.87	56.21	59.92	64.42

*Depths calculated by the authors using the conventional method adopted by Midgley & Pitman.

Table 3 Shape parameters for the bimodal distribution

Symbol	Ratio	Description
T_0	0.56	Duration of 1st triangle to total duration
T_1	0.44	Duration of 2nd triangle to total duration
P	0.77	Ratio of first triangle rainfall to total rainfall volumes
A_0	0.39	Time to peak of 1st triangle to 1st triangle duration
A_1	0.43	Time to peak of 2nd triangle to 2nd triangle duration

Symbols are illustrated in Fig. 2.

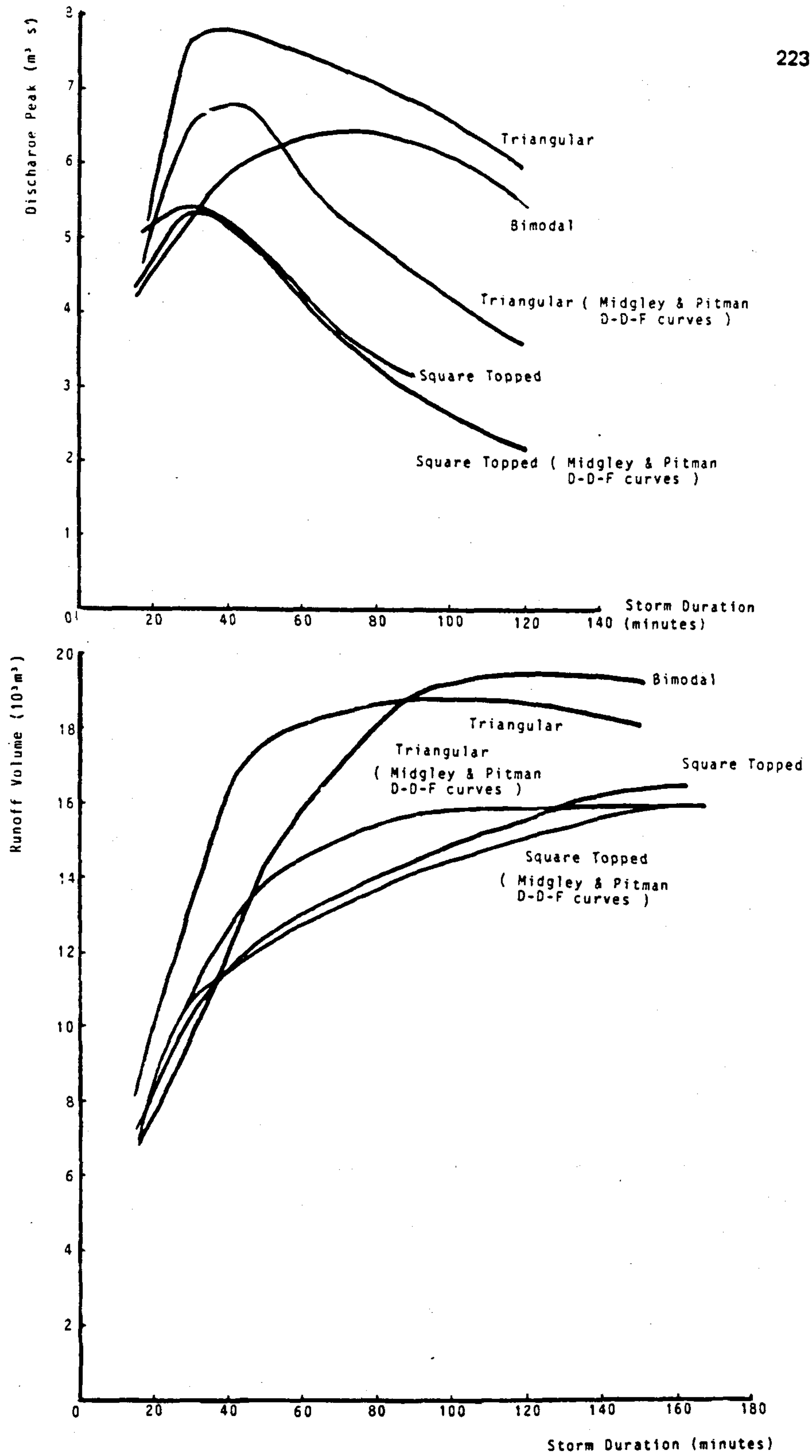

Fig. 4 Graph of peak flows and volumes of runoff for different durations and temporal distributions.

influence of profile shape on runoff is summarized in Table 4.

Table 4 Effects of design rainfall on runoff (Vanderbijlpark)

D-D-F Formulation	Rainfall shape	Peak ($m^3 s^{-1}$)	Volume (m^3)
Conventional	Square-topped	5.5	16 500
Triangular	Triangular	7.9	18 800
Bimodal triangular	B.T.D.	6.4	19 500
Midgley & Pitman	Square-topped	5.4	16 000
Midgley & Pitman	Triangular	6.75	16 000

DISCUSSION OF RESULTS

The aim of the study was to demonstrate the effect on peak and volume of runoff of varying the time distribution of rainfall input. Actual rainfall data were used to synthesize the design storms but the corresponding runoff data were not available and one had to rely on a rainfall-runoff model to generate the flood responses. Figure 4 indicates that the different storm profiles resulted in a wide range both of peaks and of volumes of runoff.

Design storm durations also differ according to the assumed profile shape. Inputs taken from conventional D-D-F relationships (e.g. Midgley & Pitman, 1978) yielded peaks and volumes of runoff lower than those generated by inputs taken from Lambourne's D-D-F relationships. The latter (last three columns of Table 2) indicates total rainfall depths greater than the former. A possible reason for this is that the former are based on abstractions at clock quarter-hour intervals on the rainfall charts (e.g. 1400 h to 1415 h) whereas the Lambourne relationships were derived from breakpoint digitized autographic data. Another reason is that data for the last two columns in Table 2 were taken from D-D-F curves derived from the statistics of entire observed events and not arbitrarily defined periods which could have been embedded in longer events or could have bracketed shorter ones.

The triangular profile yields highest peak discharge and the square-topped profile lowest peak as is to be expected because peak rainfall intensity for the triangular profile is twice that for the square-topped profile. This was confirmed by Stephenson (1984) using kinematic routing of excess rainfall across simple plane catchments. Since over-design is safer than under-design, adoption of the triangular profile would be conservative. Substantial over-design, however, would mean unnecessary additional construction cost.

The simulations for Vanderbijlpark showed that the critical duration bimodal profile storm generated greater runoff volumes, however, than did those of other profiles. A possible explanation is to be found in the excess rainfall distribution.

CONCLUSIONS

The paper offers aids to the correct choice of design storm for the type of catchment studied. If it can be accepted from the Potchefstroom analysis that the single and the bimodal triangular distribution correctly predict peak discharges, then it can be deduced from the Vanderbijlpark simulations that the rectangular profile under-predicts the peaks by up to 30%. Thus the rectangular storm profile under-predicts both volume and peak discharge from an urbanized catchment and users of this form of design hyetograph therefore err on the dangerous side. The triangular profile for the critical duration storm, on the other hand, over-predicts peak discharge for the average antecedent moisture conditions of the catchment. Advocates of the triangular storm are therefore somewhat conservative in this respect.

The Potchefstroom model indicates that the bimodal profile predicts runoff volume better than does the triangular profile while the Vanderbijlpark model shows that the square-topped profile under-predicts volumes by up to 18%. Use of the triangular or bimodal storm is therefore recommended for estimating peak discharges. Similarly the bimodal distribution is recommended for estimating volume of runoff.

In addition, the study shows that D-D-F curves generated from breakpoint data, taking into account the duration of the original storm, are preferable for design purposes to those derived by the conventional method since they produce significantly greater volumes of runoff.

Regionalized statistics of triangular and bimodal storms could be compiled for design purposes with little more effort than that required for storms assumed to be of uniform intensity. The availability of digitized data and micro-computers now facilitates such tasks (e.g. Lambourne, in preparation).

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TECHNICAL PAPER

Application of a stormwater management program

By D STEPHENSON

Synopsis

Use of a microcomputer catchment modelling program designed for stormwater management studies is described. The program is a kinematic type model, ie hydrodynamics are based on uniform flow conditions and continuity in conduits. The advantages of modular over finite elements and finite difference models are discussed. The modules in the program include catchments with infiltration, aquifers, conduits with circular, regular or compound cross-sections and storage reservoirs. The modular nature of the program enables alternative combinations of hydrological modules to be assembled ranging from large catchments to urban drainage systems. The program is therefore of use not only for a variety of catchment configurations but also for management studies, ie alternative detention or diversion systems to reduce peak flows and increase catchment recharge. The kinematic equations limit the hydraulic capability, ie backwater computations are not possible, but the resulting equations are suitable for rapid analysis on microcomputers. Routing is achieved by outlet control, but care is necessary to avoid numerical diffusion. An example application demonstrates the versatility in comparing alternative stormwater management strategies in an urban catchment in Sandton. Detention storage and road layout appear to have more influence than disconnected impervious areas, dual drainage or floodplain storage.

Samevatting

Gebruik van 'n mikrorekenaarprogram vir modelering van stormafloop van opvanggebiede word beskryf. Dit is 'n kinematiese model, waarin die hidrodinamika op eenvormige vloeitoestande en kontinuiteit in kanale gebaseer word. Modules in die program sluit in opvanggebiede met infiltrasie, ondergrondse vloei, kanale met ronde, eenvormige of komplekse deursnitte en opgaarreservoirs. Groot opvanggebiede of stedelike dreinering kan geanaliseer word. Opvangsbestuur soos detensie of wegleiding van water om spitsvloeie te verminder kan gedoen word. Terugstorting van watervlaktes is nie moontlik nie, maar die analise is ideaal geskik vir mikrorekenaars. Roetering is gedoen deur uitlaatbeheer, maar daar moet opgepas word dat numeriese verspreiding nie ernstig is nie. 'n Toepassing in Sandton wys hoe dit moontlik is om stormwaterbeheermetodes te vergelyk. Opgaring en paduitlegging het meer van 'n invloed as ongekoppelde ondeurdringbare oppervlaktes, dubbeldreinering of vloedpleinopgaring.

Introduction

It is recognized by many hydrological engineers that modelling is the most versatile and accurate method of estimating floods. There are many such computer models available[1]. This paper is concerned primarily with a model for estimating stormwater runoff from urbanized catchments (although the principles have been applied to rural catchments[2]). Programs used in South Africa for stormwater computations include:

1. ILLUDAS or its variants[3], a program based on time area methods, ie a development of the rational method with unique concentration or travel times for each subcatchment.
2. WITWAT[4], which employs the kinematic method for overland flow and was fitted with SA rainfall data. It is a simple to use model for design or analysis of stormwater networks.
3. SWMM[5], a mainframe program recently adapted for use on microcomputers. This model has a number of 'blocks' ranging from a kinematic runoff block to hydrodynamic analysis of large drains, and includes storage and empirical water quality models. The model is relatively sophisticated and requires background knowledge. Limited management facilities are available with the program.

Other models, eg OTTHYMO[6], only use computers for speed of calculation and are not based on numerical solution of the runoff equations. They are based on manual-orientated methods, eg unit hydrograph or conceptual models[7].

Subcatchment arrangement

The interconnection of one subcatchment or element with another in the models can be done in various ways:

1. *Finite difference grids.* In the case of a homogeneous type catchment a rectangular grid can be superimposed. Thus flows and water depths are computed at grid points. Either one- or two-directional flow can be assumed. In general two flow vectors must be assumed. An exception occurs if the flow is in one direction parallel to one of the axes. For most undular topography two-dimensional analysis is necessary.

2. *Finite element.* The computations can be reduced and size and shape of element varied to suit the topography if a finite element approach is used. In general a two-directional flow pattern must be assumed, although if the boundaries of elements are perpendicular to flow, one-directional flow can be assumed.

3. *Modular.* The simplest and most versatile model is one made up of modules that can be linked up at the ends. Generally the flow is one-directional along the axis but two-dimensional catchments can be made up of modules in parallel and series, ie the orientation of the module is ignored because the directional momentum of the water is not considered. It is the latter configuration on which the model described here is based.

What is not recognized in many of these models is man's influence on runoff and, even more so, man's ability to reduce runoff. Catchment management is now recognized as an important aspect of water resource planning[8]. That is, from a long-term point of view catchment yield is influenced by vegetation cover and land usage.

For single events, eg floods, stormwater management is now also recognized as being important[9]. Drainage engineers are now aware of methods of attenuating floods, eg with detention storage basins or dual drainage systems[10]. Such techniques should therefore be accommodated in models.

The necessity of groundwater recharge in order to maintain an adequate water table also supported the idea of stormwater soakaways

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and retention storage. Facilities for studying groundwater fluctuations would therefore also be desirable in models. To go a step further, a temporary perched water table frequently occurs near the ground surface during a storm. This water can re-appear as surface flow further down the catchment and this 'interflow' is recognized as a contributing component, particularly during recession of a hydrograph.

Computer models are the only practical tools available to hydrological engineers for studying such complex phenomena. The drainage engineer could use a catchment model with the correct facilities to optimize the design of a detention storage dam or to balance dual flow between subsurface conduits and surface channels or even roadways. The effectiveness of various trial designs can be compared using a model and alternatives, then costed in order to optimize a drainage system.

In addition to stormwater designs for new townships, improvements to existing services can also be made using modelling. Designs may originally have been based on low-density township development and flow rates increased when more intense development occurred, eg subdivision of stands, extension of city limits, increased cost of flooding. At that stage it may be too late to construct larger drains and the only resorts available are temporary storage or overflow into roadways or parkways. In such cases, peak flows (as obtained by the 'rational' method for example) are insufficient for design and a complete hydrograph, and even antecedent conditions, are required to estimate volumes of runoff.

It is with these requirements in mind that a stormwater management model was developed. WITSKM (Wits stormwater kinematic management model) was designed to provide accessible facilities to study alternative stormwater management methods.

General comments

It is towards the more hydraulically based models that the majority of research is now directed. By suitable selection of module arrangement, one-dimensional flow can be assumed, ie the module axis is taken in the general flow direction. Lateral flow time is neglected (which could introduce error in floodplain-type modules). Transverse, ie lateral and vertical (for horizontal flow direction), accelerations are also neglected, but this is quite satisfactory for all runoff modelling.

Thus the hydrodynamic equations are narrowed down to the St Venant equations and their derivatives. Accelerations are not of importance in overland or long river studies, so these terms are omitted, and backwater effects are only of importance in some channel situations, so most modules are limited to kinematic-type equations.

Groundwater flow capability with aquifer modules makes possible long-term simulation of catchment yield. Groundwater contributions lag surface runoff by hours or even months. Recession limbs of stormwater hydrographs can be due to contributions from perched water tables or interflow. Longer-term yields are from deeper aquifers.

Recharge of surface layers is, however, important from the point of view of antecedent moisture and permeability for forthcoming storms. The continuous simulation capability therefore improves estimation of surface storm runoff. Surface layer moisture is also important for estimating evaporation and losses.

The time scale of flow from deeper aquifers may be much longer than from the higher water tables, and a greater time step could be used once surface runoff is reduced.

Theory

The terms in the hydrodynamic equations that control catchment runoff are employed. These are the continuity or mass balance equation

$$B\frac{\delta y}{\delta t} + \frac{\delta Q}{\delta x} = q_i$$

and a flow depth/discharge relationship, eg

$$Q = A\alpha y^m$$

where Q is discharge rate, y is water depth, B is surface width, q_i is inflow per unit length, x is longitudinal distance, α is a catchment constant, A is cross-sectional area and m is an exponent. Using the Manning equation for overland flow, $\alpha = \sqrt{(S)}/n$ and $m = 5/3$ where S is longitudinal gradient and n is Manning's roughness.

The modular structure of the input data makes it possible to interactively change drainage features, ie type or even connectivity. Random numbering or ordering of data makes it easy to add or remove modules.

The kinematic equations exclude backwater effects and flow accelerations in time and space. Whereas the latter are rarely significant in river flow, backwater profiles can be of importance at obstructions such as weirs and bridges, since the water depth is increased. However, the scale of a backwater necessitates much smaller distance intervals than are necessary for kinematic models, so backwater analysis is handled separately at specific locations where required. Backwater can generally be analysed assuming steady flow conditions, which greatly simplifies analysis and enables a rapidly converging numerical method to be used, ie water depths are calculated assuming flow rates are known. Using a kinematic program it is the flows that are difficult to compute. This problem becomes even more severe if the full hydrodynamic equations (actually the one-dimensional hydraulic equations of St Venant) are employed. The steady flow case is a subroutine and can be used to initiate water depths. For two-dimensional problems a simpler cell-type model is preferred.

Program description

The program uses a two-step backward explicit method to solve the non-linear discharge equation. By solving the continuity equation for change in water depth at each point first, any combination of inflows and outflows to a reach is accounted for.

The program is in BASIC language suitable for PC-DOS-based microcomputers and is one of a suite of programs, WITWATER, under devel-

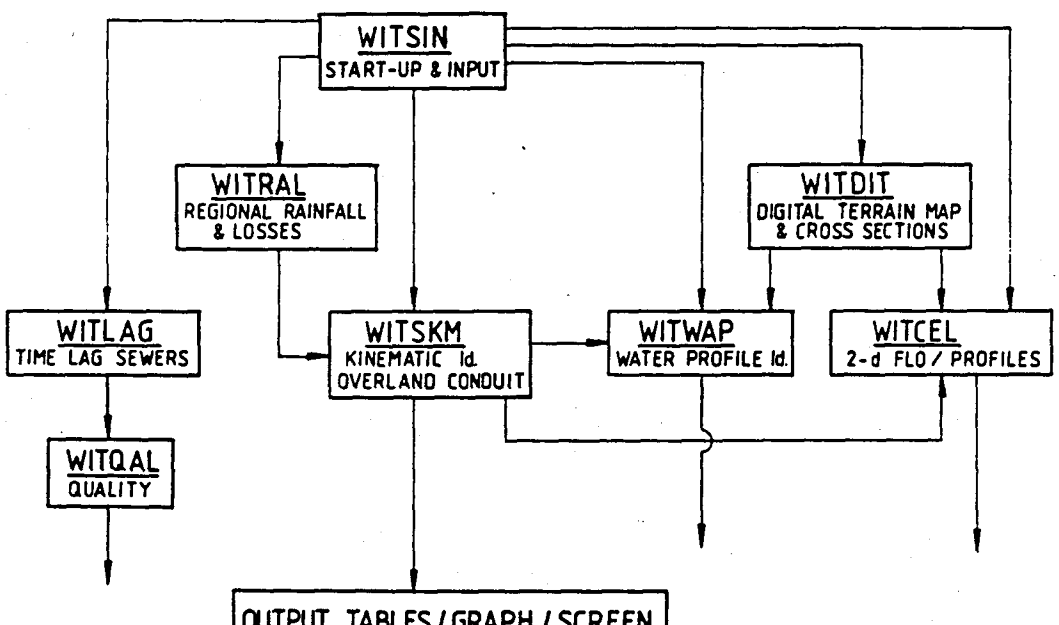

Fig 1: WITWATER suite

opment by the Water Systems Research Group at the University of the Witwatersrand (Fig 1).

A great advantage over hydrological-type models, which include time area methods, is that the data represent physically measurable parameters. Thus catchment and channel shapes, slopes, roughnesses and rainfall all affect concentration times. Very few, if any, empirical factors are required, and little calibration is necessary except where groundwater flow is involved. Here it is generally more economical and expedient to obtain representative parameters from observed input-output events, especially where complex aquifers are involved. The values of roughness and infiltration rates to use with such models are often not the ones normally used in hydraulic calculations. To account for small obstructions and circuitous routes, the effective macro-scale roughness is generally larger than would be used for a channel. The effective infiltration rates are also generally less than would be measured in a laboratory. This is because connected impermeable areas lumped into an averaged catchment can contribute a high proportion of runoff.

The infiltration routine is also handled using soil science rather than empirical factors such as the rational 'C' or Horton or SCS factors.

Routing process

Kinematic waves are theoretically not subject to diffusion, ie spreading and attenuation, as no dynamic effects are included in the equation. There may be changes in wave shape since dx/dt is a function of depth, but there can be no change in peak flow unless there is an inflow. The advantage of taking large distance increments with the kinematic method therefore results in a sacrifice in accuracy. A method of minimizing the numerical error and getting the best approximation to hydrodynamic diffusion was proposed[11].

The wave diffusion can be accounted for by using the slightly more accurate equations, namely the diffusion equation, or the full dynamic equations. However, in some cases wave diffusion can be reproduced numerically. From the mathematical point of view, numerical diffusion can be controlled or minimized. Explicit solution of the kinematic equations is often employed in preference to implicit solution as the friction equation is non-linear, and explicit schemes such as the backward centred, or semi explicit such as the four-point scheme[12], are reasonably accurate and fast.

Explicit schemes can be subject to numerical instability unless the time increment is small enough, ie $\Delta t < \Delta x/(dx/dt)$ (the Courant criterion), where $dx/dt = \alpha m y^{m-1}$ On the other hand the smaller t the greater the numerical diffusion as the numerical effect travels at a speed $\Delta x/\Delta t$. The optimum compromise is $\Delta x/\Delta t = dx/dt$. This is not always possible in an equispaced grid as dx/dt varies. Ponce[13] attempted to reproduce actual diffusion in kinematic equations by writing the finite difference equations for flow in a manner similar to the Muskingum-Cunge routing equation.

Adopting a more practical approach the kinematic diffusion process can be explained as follows: The routing process that occurs with kinematic modelling is similar to reservoir routing where discharge depends only on the stage at the outlet. A unique stage-discharge relation is assumed, ie no allowance is made for accelerations or water surface gradient. A compromise could be made by setting discharge a function of stage at more than one point, eg average of upstream and downstream stages.

The resultant effect is similar to that employed in the Muskingum method and in addition allows for non-linearity in the stage-discharge relationship. It also has the advantage that the parameters in the equations are physically measurable and not empirical. To overcome the non-linear relationships the kinematic equations can be solved in two steps, namely the continuity equation to determine change in water depth, and discharge is obtained from stage using the selected discharge equation.

The discharge equation is not limited to a channel-type equation such as that of Manning. Thus using a general discharge equation of the form

$$Q = Kh^m$$

K is a constant.

If h is stage at discharge point and $m = 5/3$, one has the Manning equation. If $m = 5/2$ one has a triangular weir, $m = 3/2$ is a rectangular weir, $m = 1/2$ is an orifice and $m = 1$ is a deep rectangular channel. If h is the difference between upstream and downstream stages then if $m = 1/2$ one has turbulent pipe flow and if $m = 1$ one has laminar flow in a closed circuit or confined aquifer.

Management capability

A drawcard of the model prepared along the above lines (WITSKM) is its versatility when it comes to redirecting flows and attenuating hydrographs. The facility of readily being able to redirect flows along different routes means channel storage or open versus closed conduit conveyance can be explored. The re-routing of flows along circuitous routes may increase channel storage. This in turn increases concentration time and could reduce design peak flows. New township layout could be varied until a suitable stormwater drainage pattern emerges.

The overflow facility also enables dual drainage to be used to maximum advantage. Excess flow could be led to shallow channels (or roadways) which will provide retardation or lead to channels that are only used in emergencies. The overflow level can readily be varied to permit different risk storms to be accommodated in the minor (underground conduit) system.

The aquifer option is also of use in urban catchment management studies. Aquifers can be recharged by direct infiltration or with water led to them from less pervious areas. In either case the absorption of the aquifer is only limited by the depth-discharge characteristics and initial moisture conditions.

A useful module for hydrograph attenuation is the storage module. Reservoir surface area, dead storage and crest level can be varied to achieve an optimum balance between maximum water depth and dam cost. The ability to vary the outlet discharge characteristics is, however, the most versatile facility of the storage module attained by means of a general discharge equation of the form

$$Q = WAy^m$$

where W is catchment of channel bed width.

For detention attenuation that has a decreasing effect with inflow m should be high and for high detention at all depths m should be small. Again by trial, an optimum compromise between dam cost and cost of conveying the discharge can be achieved.

Modules

The versatility of the computer program is enhanced by the possibility of fitting various types of hydrological units or modules into a system. Catchments, aquifers, conduits and storage basins can be linked in any order. The various modules that can be built in are as follows (Fig 2):

Catchments

A basic catchment is a rectangular shape sloping in one direction. The module reference number, its downstream module, initial water depth, length, width and discharge coefficient (ratio of discharge to depth to the power of 5/3, eg $\sqrt{(S)}/n$ where S is gradient and n is Manning roughness) are required as input data. In addition the surface permeability, suction at the ground wetting front, initial moisture content and aquifer module number are required. An infiltration process based on the soil physics model of Green and Ampt[14] is assumed.

Catchments can be linked in cascades (in series), eg changing slopes or disconnected impervious surface, or in parallel, for instance if a portion has directly connected impermeable cover.

Circular conduits (pipes)

Urbanized catchments are normally sewered with underground pipes, which run partly full most of the time. When they surcharge, the excess flow continues down roads and may be directed to channels. Such a system ('major/minor' system) is common at high flows whether intentional or not and provides roads free of ponding for all but exceptional storms. The capability to model such systems is therefore important.

Data required for this type of module are module reference number, downstream module, initial depth, length, diameter, conveyance $\sqrt{(S)}/n$, and overflow module number.

Trapezoidal channels

Open channels are the most common conduits, be they roadways, gutters, ditches or canals. Where the channel is a simple trapezoid, the data requirements are limited to module reference number, downstream module, initial flow depth, length, base width, conveyance, side slopes, maximum depth and overflow module.

Compound channels

Natural channels may be defined using an arbitrary number of co-

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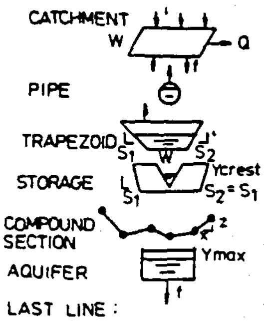

	UNIT N°	LEADS TO	TYPE	y_0	L	W	$\alpha = \sqrt{S_0}/n$ ($Q = W\alpha y$)				
CATCHMENT			1					k,mm/h	sucn. m	m c frac	Intil to
PIPE			2					ovflo to			
TRAPEZOID			3					S1	S2	Y max m	O flo to
STORAGE			4					Exp. m	S1	Y dead m	Y crest
COMPOUND SECTION			5			N° pts.	Slope	x	z	n	etc each W
AQUIFER			6			W	$\alpha = k_L S_0$ m/s	k_v mm/h	POR FRAC	Y max, m	Intil to
LAST LINE :	0										

Fig 2: Data input for WITSKM

ordinates across a section. The stream between any two neighbouring points is treated as an independent section so that velocity varies depending on flow depth and roughness. Flood plains are thus accommodated with slow-moving storage on the banks and a more rapid stream between banks.

Data are module reference number, downstream module, initial depth, length, slope, points, co-ordinates and roughness of each section.

This facility can be used to calculate normal depth in compound channels. An impermeable catchment upstream with an area of 3 600 m x 1 000 m is fed with *R* mm of catchment rain (where *R* is normal flow in m^3/s) and after a period of time the depth in the downstream channel stabilizes at normal depths.

Storage basins

Where detention or retention is required, on- or off-channel storage may be of use. Data required are module reference number, downstream module, initial water depth, length, width, conveyance α, discharge depth exponent $m (Q = W\alpha y^m)$, side slope of basin, dead storage before discharge, and crest level of dam wall. By experimenting with the outlet, eg crest or orifice spillway, the best design may be achieved.

Aquifers

Water may infiltrate to aquifers from catchments or be discharged directly into them from any conduit or overflow. The aquifer acts as a conduit, albeit with a much slower flow rate. The aquifer will also have a maximum depth and may leak to a lower aquifer. Stacking or cascading of aquifers is possible. The kinematic equations are entirely adequate for this type of flow, as dynamic effects are absent.

Data include aquifer reference number, downstream module number, initial flow depth, length, width, conveyance defined as kS where k is permeability and S is gradient, porosity, aquifer depth and underlying aquifer number.

Other facilities

A frequent source of error in stormwater programs arises when the downstream catchment number is changed or forgotten. A facility exists for displaying graphically on a PC colour screen the entire network once it is entered on the computer (Fig 3). Each module is drawn according to the type, eg pipe, catchment or channel, and is connected upstream and downstream as indicated in the data. Overflow routes are also indicated. In general the model is designed for easy understanding and input and cross-checking. It is especially useful for stormwater management studies. The groundwater modules enable continuous simulation to be performed, which is useful for establishing antecedent moisture conditions for storms, and dry weather flows.

Guide for hydrological and hydraulic parameters

Until experience is gained in estimating various parameters the following may assist.

Permeability is the saturated value and can be obtained from laboratory tests, eg as high as 1 000 mm/hr for sand down to as low as 1 mm/hr for clay. 5 mm/hr to 10 mm/hr is most typical of highveld catchments.

Effective soil suction due to capillary attraction is in metres of water head, eg 0,7 m for clay, 0,1 m for sand. Moisture content is fraction by volume (ratio of water to dry soil), eg 0,1 for dry soil or 0,3 (or porosity) saturated.

Manning roughness can be normal hydraulic values for conduits, eg 0,013 for smooth pipe, 0,03 for rough channel, but increases to 0,1 or even 0,2 for overland flow with obstructions. For all conduits except storage modules the Manning equation is assumed to hold, so $m = 5/3$ in the discharge equation

$$Q = \frac{A}{n} \left(\frac{A}{P}\right)^{m-1} S^{1/2}$$

where *A* is cross-sectional area of flow, *P* is wetted perimeter and *S* is bed slope in the flow direction.

For overland flow per unit width the equation becomes

$$q = \frac{1}{n} y^m S^{1/2} \quad \text{or } q = \alpha y^m \text{ where } \alpha = \sqrt{(S)}/n$$

For storage basins with outlet control, m may be 1/2 for an orifice or 3/2 for a rectangular weir. In the case of a weir of width w, $\alpha \approx 1{,}8w/W$ where W is module width, so $Q = 1{,}8wy^{3/2}$, and in the case of an orifice $\alpha \approx 0.6A \sqrt{(2g)}/W$ so $Q = 0{,}6A \sqrt{[2g(y-y_d)]}$ where y_d is depth of dead storage and A is the orifice area[15].

Application to Sunninghill catchment

The program was used to study alternative ways of attenuating floods in a gauged catchment in Sandton. The Sunninghill catchment is monitored for a project funded by the Water Research Commission and therefore calibration data are available. Four autographic raingauges and a streamgauge exist. The catchment is generally residential with a predominance of single storey dwellings, some townhouses and a small commercial complex. Fig 4 is a contour map of the catchment showing the street layout.

A number of alternative stormwater drainage arrangements were studied by a group of postgraduate students at the University of the Wit-

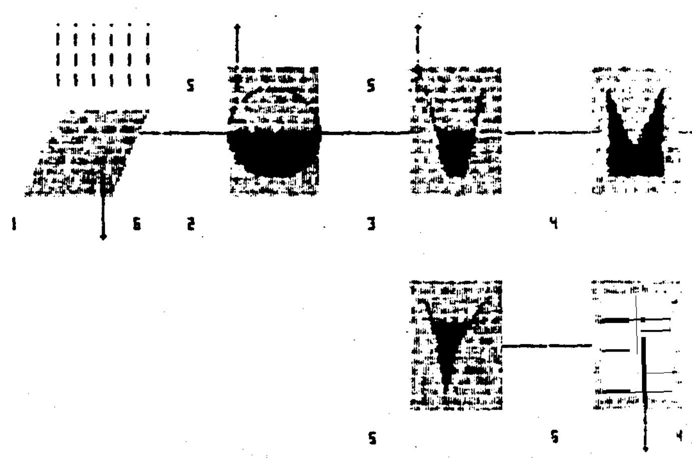

Fig 3: Screen graphics for connectivity check in WITSKM

watersrand. The existing arrangement of stormwater pipes was used as a standard and also to calibrate the model. Then five alternative arrangements were studied. These were (Fig 5):

1. Disconnecting most impervious areas
2. Re-arranging road plan
3. Adding floodplain storage on channel
4. Detention storage
5. Dual drainage

Each alternative was open to further alternatives, and engineering judgement was used, together with limited trials, to obtain the best arrangement for the alternative. Thus roads were not lengthened unnecessarily in changing layout. Dual drainage was only a variation of what already exists, ie overflow into the streets is already possible for a 10- to 20-year storm or greater. Disconnection of impervious areas required diverting flows into a park in some places. Detention storage was limited in capacity to the available space and an implicit economic balance was attempted.

The results are therefore subjective and obviously site specific. They indicate only the order of magnitude possible in flood peak reduction with some limited changes in plans. Each alternative would be more costly than the existing plan and no attempt is made to evaluate cost implications here. It should also be noted that projected storms (the 10-year storm) were used to see the effect of more extreme storms than the calibration storm, and those projected storms were based on regional Weather Bureau data, and each alternative may have used a different design storm duration as being appropriate to the alternative. Table 1 summarizes the results of the studies. The effectiveness of each management method is ranked in Table 2 in order of effectiveness for a two-year storm and a 10-year storm.

The conclusions cannot be regarded as general and the main benefits of the study are:

1. To show that there are many alternatives that could be considered in

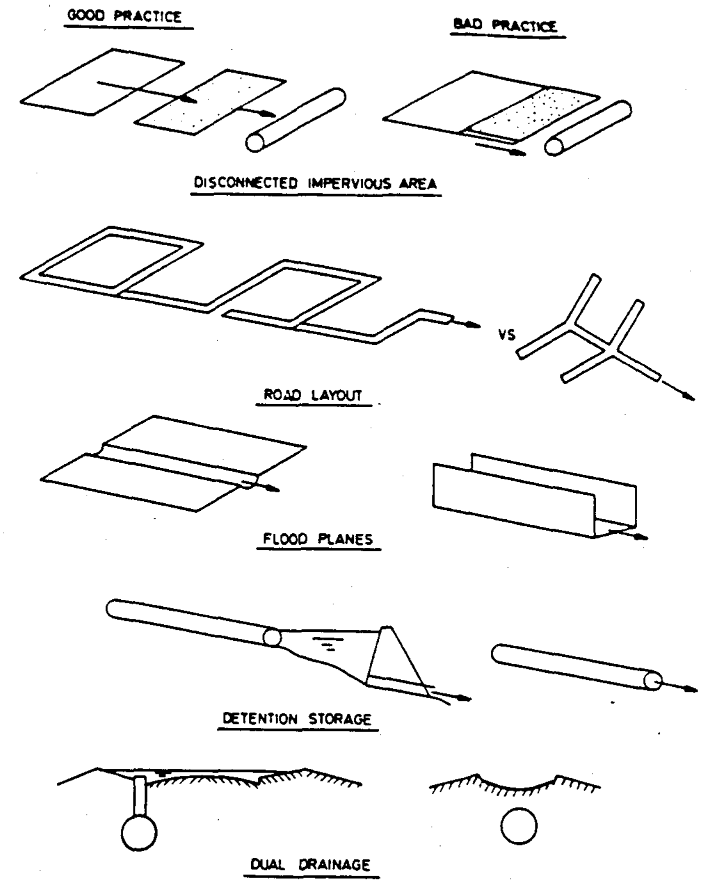

Fig 5: Summary of stormwater management devices

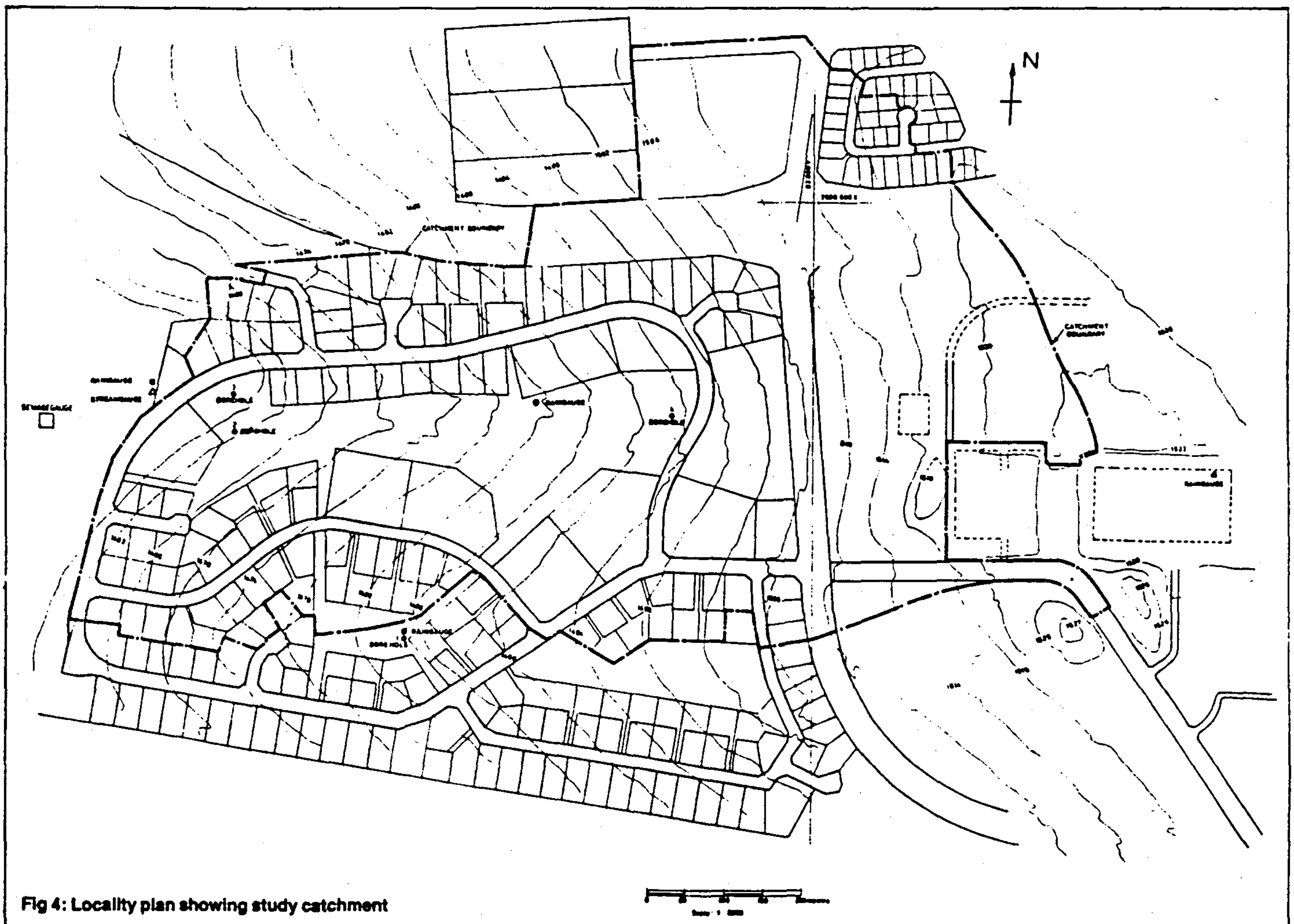

Fig 4: Locality plan showing study catchment

Table 1: Modelled effect of stormwater management methods in Sunninghill

Method	*Two-year peak* m^3/s	*10-year peak* m^3/s	*Cost implication*
1. No change to layout	2.2 (observed)	6.6	Nil
2. Disconnect impervious area	1.6	(5.0)	Low
3. Road layout	1.3	(3.9)	High
4. Floodplain and channel storage	(1.5)	5.1	Low here
5. Detention storage 1 000 m^3	1.2	4.4	Medium here
6. Dual drainage	1.9	(5.3)	Medium

Table 2: Ranked stormwater management methods

Two-year storm	*10-year storm*
1. Detention storage	Road layout
2. Road layout	Detention storage
3. Floodplains	Disconnect impervious
4. Disconnect impervious	Floodplains
5. Dual drainage	Dual drainage
6. Existing drains	Existing drains

stormwater drainage planning.

2. To demonstrate that such studies can easily be done by computer modelling.

The highly complex nature of catchments, the variability in topography from one catchment to another, management effects and temporal and spatial variability of storms can most readily be cited as reasons for using such models.

Acknowledgements

The research described above is sponsored by the Water Research Commission. The computer programs described have been enhanced by Drs I R A Green, W A J Paling and J J Lambourne and Mr T Coleman of the Water Systems Research Group at the University of the Witwatersrand.

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Effects of Momentum Transfer in Compound Channels

By D. Stephenson[1] and P. Kolovopoulos[2]

Abstract: Different methods of considering the shear stresses between a main channel and its flood plains are discussed. The methods considered include the simplistic method of considering the entire hydraulic cross section as one, dividing the cross section into independent sections, assuming limited shear stresses between main channel and the flood plain, trying to find an interface with zero shear stress, or allowing for the shear stresses from a momentum balance. Based on evaluation of alternative steady-state models, it is concluded that the area method i 'he most promising for computation of discharge and that the Prinos-Townsend equation gives accurate results for apparent shear stresses at the main channel–flood plain interface. The results of the theoretical study are used in an unsteady-flow model, prepared based on hydrodynamic principles. The model facilitates the investigation of the significance of the momentum-transfer phenomenon between the main channel and its floodplains. The analysis demonstrates that momentum interchange results in a shift in the loop rating curve to the right, a delay in the falling of water levels, an increase in flood plain flow, and a decrease in the main channel carrying capacity.

Introduction

Many flood-routing methods assume a single cross section for purposes of calculation of the stage discharge characteristics. These methods therefore ignore the transfer of momentum between the main channel and its floodplains.

Owing to simplistic models, calibration with one set of data does not necessarily ensure reliable results for other data, particularly if for one of the cases the floodplains are inundated. The reduced hydraulic radius of the floodplain and the often higher hydraulic roughness result in lower velocities on the floodplains than in the main channel. These differences result in a bank of vortices as demonstrated by Knight and Hamed (1984) (Fig. 1), referred to as the "turbulence phenomenon." There is therefore a lateral transfer of momentum that results in an apparent shear stress.

The momentum transfer was examined by Zheleznyakov (1966), who demonstrated that such flow characteristics are dominated by the momentum transfer. He indicated that the momentum transfer decreases the overall rate of discharge for floodplains immediately above the bank-full condition when compared with the assumption of a single channel. Wright and Carstens (1970) studied floodplains with depths exceeding 0.5 times the channel depth. They found that the separate channel method compared favorably with observations, although segment discharges were often up to 10% incorrect. Yen and Overton (1973) plotted the velocity distributions in compound channels and

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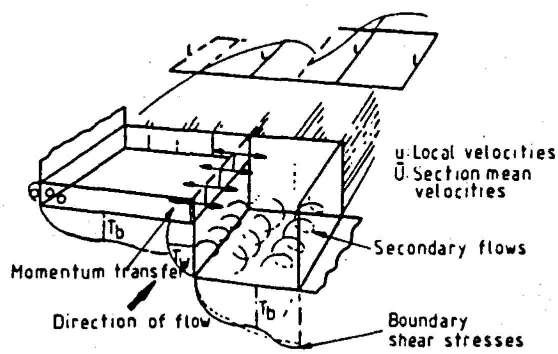

FIG. 1. Momentum Transfer in Compound Channels

found that the interaction distorted the velocity pattern near the boundaries. Myers (1973) showed a decrease of up to 22% in channel shear and an increase of up to 260% in floodplain shear stresses.

Recently, research into the understanding of the physical processes controlling flow in compound channels has been intensified. Wormleaton et al. (1982) used laboratory experiments to derive an empirical relationship between apparent shear stress at the interface and a number of parameters describing the channel geometry. The studies of Myers (1987) illustrated the following aspects: Where the channel is taken as a single unit with one area and one wetted perimeter, the discharge is underestimated owing to the reduced main-channel velocity. Where the sections are taken independently, the total flow rate is generally overestimated as the momentum transfer or shear between the main channel and the slower-flowing floodplains is ignored.

More recent research sponsored by the U.K. Science and Engineering Research Council and Hydraulic Research, Ltd., has been reported by Myers and Brennan (1990). The research investigated resistance coefficients of compound channels. Wormleaton and Merrett (1980) proposed an index to allow for floodplain flow.

Models for Apparent Shear Stress

The shear stress on the interface may be calculated from the following equation:

$$\tau_{ai} = \frac{1}{P_{ai}} (\gamma A_{ci} S_0 - \tau_c P_c) \qquad (1)$$

where τ_{ai} = apparent shear stress acting upon the assumed interface (i); γ = unit weight of water; τ_c = average boundary shear stresses for the main channel solid boundary (see Fig. 2); P_c = main channel wetted perimeter; A_{ci} = flow area of the main channel; P_{ai} = total length of the interface (i); and S_0 = bed slope of the channel.

Prinos and Townsend (1984) proposed the following equation for apparent shear stress, in which V is in m/s and τ_{ai} is in N/m^2:

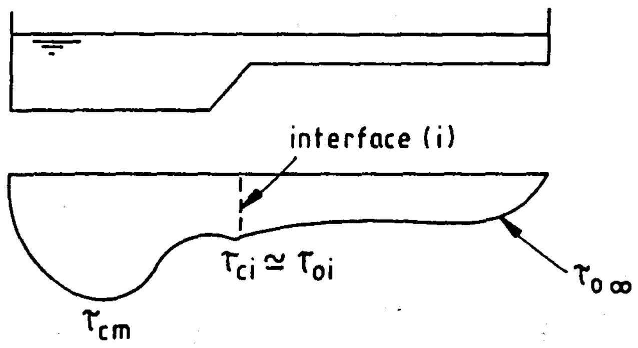

FIG. 2. Cross-Sectional Distribution of Longitudinal Shear Stress

$$\tau_{ai} = 0.874(\Delta V)^{0.92}\left(\frac{d}{D}\right)^{-1.129}\left(\frac{W_f}{W_c}\right)^{-0.514} \quad (2)$$

where d = water depth over flood plain; D = maximum depth in channel; Wf = floodplain width; W_c = channel width; and ΔV = difference between velocity on floodplain and in channel. Or dividing by the shear stress on the boundary $\tau_{0\infty} = \gamma d S_0$, in which density γ = 1,009 N/m^2; and d = depth in m give relative apparent shear stress

$$\tau_r = \left(\frac{1}{11{,}213 d S_0}\right)(\Delta V)^{0.92}\left(\frac{d}{D}\right)^{-1.129}\left(\frac{W_f}{W_c}\right)^{-0.514} \quad (3)$$

The Prinos-Townsend equation was found to apply to results of Wormleaton et al. (1982) and Knight et al. (1983). The equation was found to fit well up to values of apparent shear stress ratio up to 6. In addition, the equation accounts for varied cross sections and rough floodplains and therefore is the one adopted for an unsteady flood-routing model described in the following.

Because of the difficulty of obtaining real data for validating such equations, the majority of verifications have been based on laboratory data. Four existing steady-state computation methods were compared.

Separate Channel Method

By assuming that the flows of the main channel and the two adjoining floodplains were independent, we may compute the discharge in each using an empirical equation such as the Manning equation. It is generally found that the method overpredicts flow rate, owing to the neglect of the interfacial shear stresses on the main channel flow.

Inclined Interface or λ Method

Yen and Overton (1973) estimated that the lines of zero shear stress between the main channel and floodplains occurred on a line commencing from the bank at intersection inclined toward the center of the main-channel surface (Fig. 3). These lines are referred to as diagonal interfaces. Wormleaton et al. (1982) attempted to measure the shear stresses on this interface and concluded that they were negligible except for values of the apparent shear-stress ratio (ASSR) up to 0.4

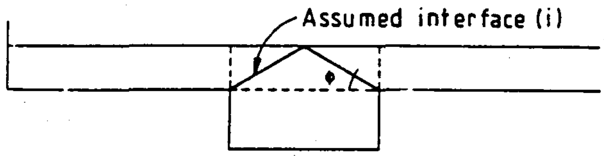

FIG. 3. Vertical, Horizontal, and Inclined Division Lines (Wormleaton et al. 1986)

$$ASSR = \frac{\tau_{ai}}{\tau_c} = \lambda \quad (4)$$

where τ_c = average shear stress in the main channel; and τ_{ai} = apparent shear stress on the diagonal, horizontal, or vertical interface. The interface shear stress can always be neglected in the case of the floodplain flow, since its boundary shear stresses are so great.

Area Method

Holden (1986) developed a method of accounting for shear stresses that assumed an arbitrary position for the interface, as in Fig. 4. He was able to calculate the additional area to be included in the floodplains or subtracted from the main channel flow employing momentum principles

$$A' = A_c - 2(\Delta A) \quad (5)$$

$$A' = A_p + 2(\Delta A) \quad (6)$$

$$\Delta A = \tau_r d^2 \quad (7)$$

where A_c' = modified area of main channel; and A_p' = modified area of floodplain. The area correction ΔA is derived theoretically (Holden 1986) from the equilibrium of forces in the floodplain region when a vertical interface divides the main channel from the flood plain

$$\Sigma F_p - \tau_{av} d = \gamma A_p S_0 \quad (8)$$

in which ΣF_p = shear force on the wetted perimeter of the floodplain P_f; and τ_{av} = shear stress on the vertical interface.

If an inclined interface is used such that there is zero shear on the interface

$$\Sigma F_p = \gamma(A_p + \Delta A)S_0 \quad (9)$$

combining these two equations yields

$$\gamma(A_p + \Delta A)S_0 - \tau_{av} d = \gamma A_p S_0 \quad (10)$$

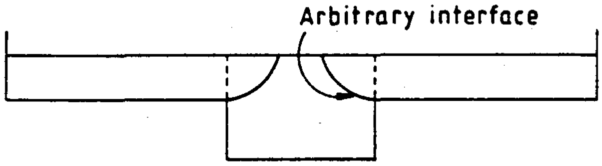

FIG. 4. Probable Shapes of Interfaces Used in Area Method (Holden 1986)

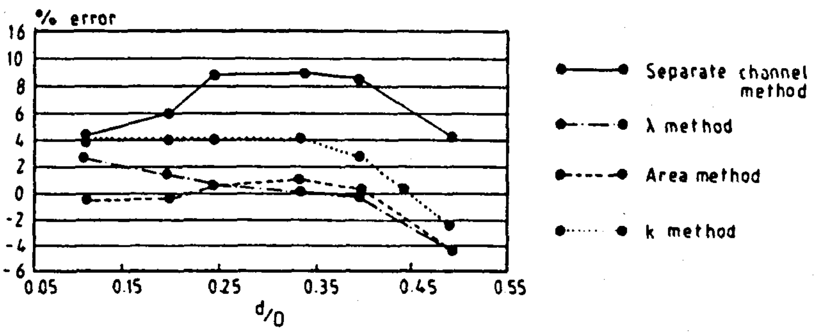

FIG. 5. Assessment of Discharge Computation Methods—Data of Knight et al. (1982)

and simplifying

$$\Delta A = \tau_r d^2 \quad (11)$$

K-**Method**

The *K*-method is an improvement on the vertical interface method. The wetted perimeter of the main channel is increased by $2K_c d$; therefore

$$P_c' = P_c + 2K_c d \quad (12)$$

where K_c caters for both the increase in resistance caused by the apparent shear stress on the interface and for the reduction in average channel boundary shear. K_c is always positive because the increase in the interfacial shear stress is always greater than the decrease in boundary shear stress. Similarly, the propelling effect of the interface and the distortion of the shear stresses on the bed of the floodplain are lumped into a single floodplain factor (K_p).

The various aformentioned methods were tested with the flume data of Wormleaton (1982) and Knight and Demetriou (1983). In general, the area method was found to be the most reliable (see Fig. 5).

Development of Flood-Routing Model Incorporating Momentum Transfer

To study the effects of momentum transfer on hydrograph routing, we incorporated the aforementioned results in a computer model of a channel. The model was based on the St. Venant equations (Kolovopoulos and Stephenson 1988) and momentum transfer was accounted for using the Prinos-Townsend equation for apparent shear stress at the interface. The area method was used for computation of total conveyance.

It was thus assumed that steady flow friction laws apply to unsteady flow. The numerical method and accuracy were proved prior to the study described herein.

For the model be widely applicable the Prinos-Townsend equation was corrected for values of apparent shear stress greater than 6. Since ΔV is a function of τ_r, an iterative procedure was adopted. For the first iteration, τ_r was calculated using the vertical interface method. When τ_r was greater than

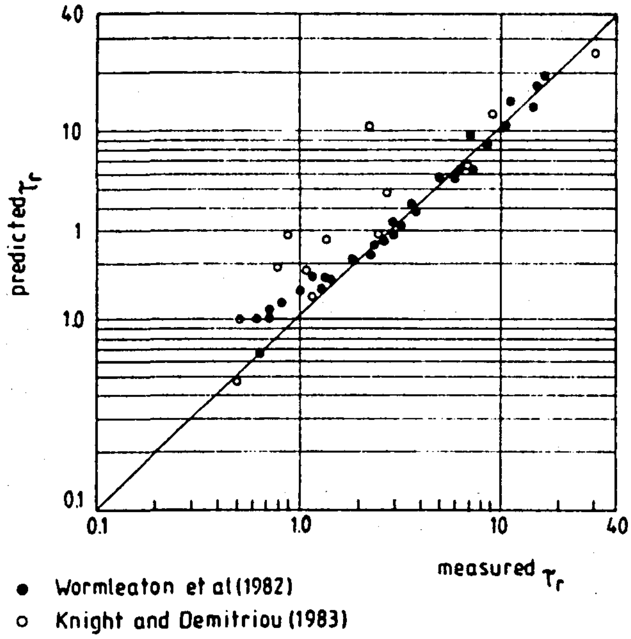

FIG. 6. Assessment of Prinos-Townsend Equation in Conjunction with Iterative Procedure (Area Method)

6 the area method was used for the subsequent iterations. The procedure was found to converge within five iterations. The results are shown in Fig. 6. The scatter for τ_r values larger than 6 was much smaller than for the results when no iterative procedure was adopted. The Prinos-Townsend equation tends to overestimate the apparent shear stress for values of τ_r less than 1. The error is not significant in the discharge computations. In general, Fig. 6 indicates an adequate overall performance by the Prinos-Townsend equation, although more testing is necessary.

The area method was found to be preferable to other methods for the following reasons.

1. It is computationally simple.
2. It has a theoretical basis.
3. It does not require previous knowledge of the shape or inclination of the zero shear stress interfaces.
4. It is not based on any limit or ratio in order to be applicable.
5. The error in the total discharge computation is in the same ranges or less than in other methods.

By incorporating these features as options in the unsteady-flow simulation model, the effect of the momentum-transfer mechanism in flood routing was analyzed. A range of hydrographs was routed through a compound channel with symmetrical floodplains and smooth boundaries. The length of the channel was 5 km and the slope S_0 = 0.0006. The other parameters were W_c = 3

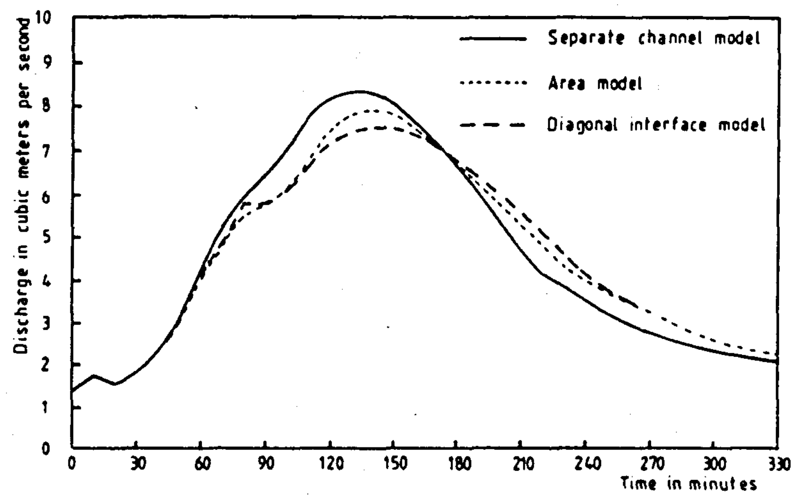

FIG. 7. Calculated Flood-Discharge Hydrographs by Three Floodplain Models—Low Flows

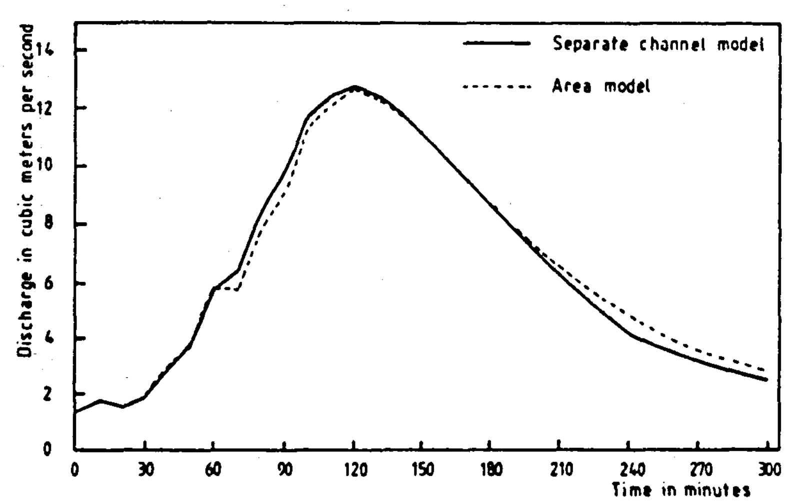

FIG. 8. Calculated Flood-Discharge Hydrographs by Two Floodplain Models—High Flows

m; $W_f = 5$ m; $n_c = 0.018$; and $n_f = 0.036$. The dimensions of the channel were kept in the same ratio, but to a larger scale than those from the flume experiments performed by Wormleaton et al. (1982).

The separate-channel and area models were used for the simulations. The upstream hydrographs had durations of 6 hr and peaks ranging from 15 m^3/s to 30 m^3/s. Figs. 7 and 8 depict the routed-flow hydrographs for low and high flows using different models. For low flows (Fig. 7), the momentum-transfer mechanism leads to attenuation of the flow-rate hydrograph. The

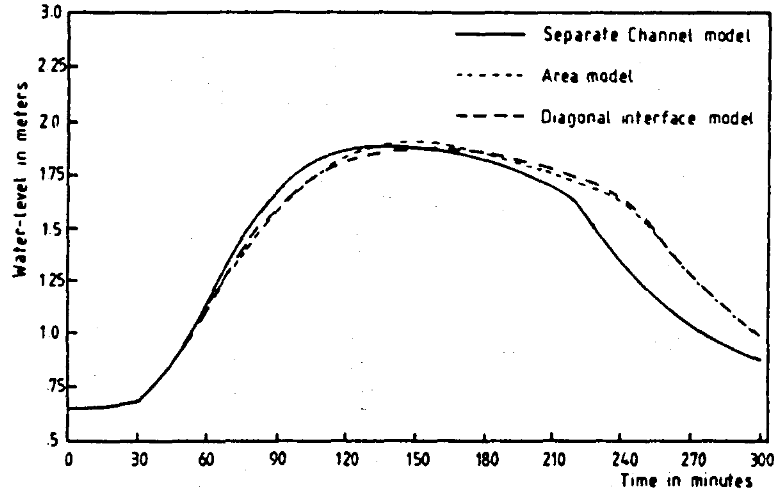

FIG. 9. Calculated Water-Level Hydrographs by Three Floodplain Models—High Flows

error in the calculated peak was 7% with the area method and more than 10% with the diagonal-interface method.

For higher flows (Fig. 8), the results of the separate channel and area models were identical. The effect of the momentum-transfer mechanism in the calculated total flood-discharge hydrograph was found to be minimal.

The effect of the main channel–floodplain interaction is more evident on the water-level hydrograph (Fig. 9). Even though the calculated peaks were approximately the same with all the methods, the momentum-exchange mechanism delayed the falling limb of the water-level hydrograph by more than half an hour. Turbulence reduced the floodplain velocities and the floodplains acted only as storage, conveying virtually no discharge.

All the results are more interestingly illustrated in the loop-rating curves plotted in Fig. 10. The momentum exchange creates a shift of the loop-rating curve downward and to the right, and a modification of the rising and falling branches of the rating curve. The loop closes somewhat for depths near the junction of the main channel–floodplains and opens at other depths. The effect of the momentum mechanism on the loop-rating curve is not the same as the effect of an increase in the roughness of the channel, although both cases result in a decrease in the conveyance. In the latter case the loop-rating curve shifts downward and to the left and the loop widens.

One of the most noticeable features of the results is depicted in Fig. 11. This figure shows the distribution of discharge in the main channel and the floodplains, for low and high flows, allowing for the transfer mechanism (area method) and ignoring it (separate-channel method). In spite of the accurate prediction of the total discharge for high flows, if the momentum-transfer mechanism is ignored, the calculated discharges in the main channel and floodplain subdivisions exhibit a large error. The overestimation of the main-channel capacity is compensated for by an underestimation of floodplain discharge. The error in each subdivision's flow rate is more than 10%.

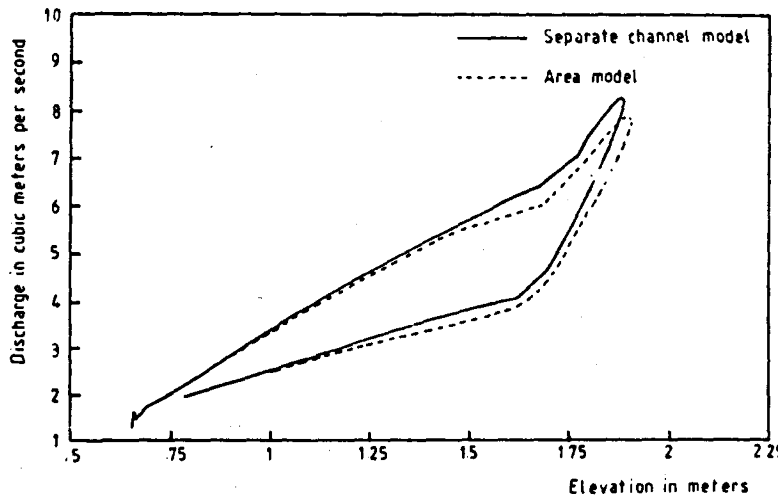

FIG. 10. Shift of Computer Loop Rating Curve Caused by Momentum-Transfer Mechanism

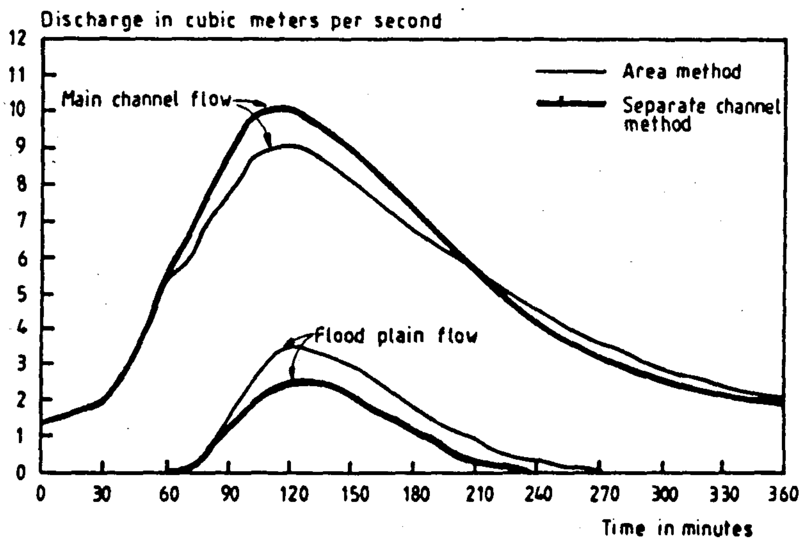

FIG. 11. Error in Flow Distribution in Main Channel and Floodplains—High Flows

The results are in agreement with the findings of Wormleaton and Hadjipanos (1985) in their experimental analysis of the flow distribution in compound channels. It is obvious that the separate-channel method does not model the proportions of flow in channel and floodplain accurately at the lower depths. The discrepancy is greatest at a depth of 0.40, where floodplain capacity is underestimated by around 27% by the conventional method.

CONCLUSIONS

The mechanism of the momentum transfer was initially analyzed under steady-state conditions. The existing models for the prediction of the apparent shear stress were evaluated with sets of flume data. The Prinos-Townsend equation was identified as the best model. Four existing steady-state discharge computation methods were assessed and compared based on published flume data, incorporating a fairly wide range of bed roughnesses and floodplain widths. From the evaluation, it was concluded that the most promising method is the area method, as it is conceptually sound; it is the most consistent method and does not need to comply with any special criteria before application.

The unsteady-flow model was modified to incorporate a modified form of the Prinos-Townsend equation for the calculation of the apparent shear stresses at the interfaces. The area method was then used for the computation of the total conveyance. By incorporating these features in the model, the effect of the momentum-transfer mechanism in flood routing was analyzed. A range of hydrographs was routed through a compound channel with symmetrical floodplains. The analysis showed that the momentum exchange results in the following.

1. An attenuation of the flow-rate hydrographs at low depths.
2. A delay in the falling of the water levels.
3. A shift in the loop-rating curve.
4. An increase in the flood plain flow and a decrease of the main channel carrying capacity.

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Appendix II. Notation

The following symbols are used in this paper:

A_c = flow area of main channel;
A_{ci} = flow area of main channel;
A_p = flow area of floodplains;
D = water depth in channel;
d = water depth on floodplain;
F_p = shear force on wetted perimeter of floodplain;
K_c = factor for increased shear stress at interface;
K_p = floodplain factor;
P_{ai} = total length of interface;
P_c = main channel wetted perimeter;
S_0 = bed slope of channel;
V = water velocity;
W_c = width of channel;
W_f = width of floodplain;
γ = unit weight of water;
λ = apparent shear stress ratio;
ΔV = difference between velocity in channel and velocity on floodplain;
τ_{ai} = apparent shear stress on interface;
τ_{av} = shear stress on vertical interface;
τ_c = average boundary shear stress for main channel;
τ_0 = floodplain shear stress;
$\tau_{0\infty}$ = shear stress on boundary; and
τ_r = relative apparent shear stress.

FLOODS IN PERSPECTIVE

CSIR CONFERENCE CENTRE PRETORIA 20-21 OCTOBER 1988
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PAPER/REFERAAT 3.5

THE DEVELOPMENT OF A MODELLING SYSTEM FOR THE ROUTING OF FLOODS

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ABSTRACT

A micro-computer compatible one-dimensional model for the simulation of flow in looped or branched open channels has been developed. The model is based on the solution of a box scheme of the Preismann type by an implicit looped algorithm. The suite of models is user-orientated, interactive and it has an operational data-handling structure. Three examples demonstrated the model's capability of simulating with minimal distortion, a wide range of unsteady flows, tidal flows, and regulated flows encountered in natural channels. The advantage of hydraulic routing over hydrological routing method is demonstrated.

OPSOMMING

'n Mikro-rekenaar gebaseer een-dimensionele model vir die simuleering van vloei in geslote of getakte oop kanale was ontwikkel. Die model is gebaseer op die oplossing van 'n boks skema van die Freismann tipe deur 'n ingewikelde algoritme. Die stel modelle is gebruiker georienteerd, onderling-aktief en het 'n bewerkings-data hanteering struktuur. Drie voorbeelde demonstreer die model se bekwaamdheid van voorgee met minimale verwringing, die wye reeks van onstabiele vloei, gety vloe, en gereguleerde vloei wat mens teekom in natuurlike kanale.

THE DEVELOPMENT OF A MODELLING SYSTEM FOR THE ROUTING OF FLOODS

1.0 INTRODUCTION

The recent floods in South Africa emphasised the need for a reliable model for flood routing. However, the wide variety of hydraulic methods of channel routing provide a bewildering choice for a potential user. These methods can be classified into three groups:

- (a) **Hydrological** or **storage** methods (i.e. Muskingum method).
- (b) **Approximation** methods or simplifications of the full Saint Venant equations (kinematic and diffusion models).
- (c) Methods using a numerical solution of the full Saint Venant equations for gradually varying flow in open channels.

Hydrological models have many drawbacks such as, stage and velocity cannot be obtained directly, the accuracy of the model is limited, and backwater conditions must be avoided. In most cases the hydrological models cannot account for a changing downstream boundary condition.

The kinematic and diffusion wave models have found wide application in engineering practise. In natural channels, however, a wider range of unsteady flows (e.g. gravity waves, bores, tidal waves, regulated flows etc.) are encountered, which can not be simulated accurately by the approximation models.

Because of the limitations of the first two groups of routing methods, hydraulic computations can be based on numerical solution of the full Saint Venant equations. Studies, for more than two decades, into the various aspects of unsteady open channel flow, have resulted in the development of many computer flow simulation models or modelling systems. Two very well known systems are: SOGREAH'S CARIMA system (Cunge et al., 1980) and the Danish Hydraulic Institute's SIVA System 21 (Abbot, 1979). The majority of the systems simulating unsteady flow are operating on main-frame computers. However, developments in the micro-computers have demonstrated that it is feasible to develop such a system on a micro-computer.

Amongst the Civil and Structural engineering firms in South Africa the availability of comprehensive software, on micro-computers, for simulation of a wide range of unsteady flows, was at the time of writing, very little, if any. It must be noted that steady state backwater programs such as HEC-2 are used for the computation of water-surface profiles. In cases where the full hydrograph must be routed, Muskingum flood routing methods or kinematic simplifications are employed (WITWAT model; Green, 1985). Advanced models like the one developed by Weiss et al. (1978), are large, monolithic, main-frame orientated and outdated in the computational procedures. Therefore, it becomes clear that a comprehensive, micro-computer orientated suite of programs (based on the full Saint Venant equations) can aid analysing the effects of periodic flooding of either urban or rural areas. It will enable the engineers to apply better planning and flood management, and to control flood plain development.

1.1 AN OPERATIONAL SYSTEM FOR RIVER FLOW SIMULATION

The suite of models, called **OSYRIS** (Operational SYstem for RIver flow Simulation) has been developed to provide both a comprehensive set of open channel flow computational procedures and a framework for practical analysis of South Africa rivers. OSYRIS-Version I is an interactive micro-computer orientated suite of programs for simulation of flow in channel networks. It is used for:

- **Steady-state calculations (backwater)**
- **Flood routing**
- **Unsteady flow simulation**

The suite of programs is able to simulate a river with its tributaries, inundated plains, and existing structures (e.g. bridges, canals, culverts etc). The development of 'OSYRIS' is based on the work and the experience on modelling systems and computational techniques of the most involved European and American Institutes as the Delft Institute, Grenoble school, U.S. Geological Survey etc. It also incorporates the research findings of the author on mathematical modelling of unsteady flow in the Water Systems Research Group of the University of the Witwatersrand. The developed mathematical models are designed in different levels of sophistication with respect to the range of application and the experience of the user. Emphasis is given on the simulation of a wide range (in terms of wave length) of shallow water waves. An experienced modeller with knowledge on computational hydraulics can apply 'OSYRIS' to his individual research needs. Although the suite of programs is capable of modelling very complicate situations, its application to simple problems requires both minimal input and user knowledge.

1.2 SOLUTION ALGORITHM

The suite of programs is intended to be used on any stream network as long as the flow can be regarded as one dimensional. The model is based on the solution of a box scheme of the Preismann type by an implicit algorithm. (Preismann, 1961; Preismann and Cunge, 1961). Implicit schemes involve the solution of systems of simultaneous equations and thus the inversion of large matrices. The large storage requirements coupled with the more complicated computational procedures resulted in the use of mainframe computer systems for all the industrial implicit models. Thus, the formulation of a general applicable one-dimensional flow model capable of running on any common micro ,i.e. such as the IBM AT, while maintaining computational efficiency becomes a difficult task. Different computational procedures were tested and methods for inversion of matrices were evaluated in terms of accuracy, computational speed, storage requirements and flexibility.

If a program is to be of general use, it must be capable of simulating looped networks, since interconnected channels is a common feature in the most rivers. Therefore an algorithm for the solution of looped networks was developed. The principle of the looped solution algorithms (Friazinov, 1970) was based on the development of a system of simultaneous equations, in which the only unknowns are the water-levels at the confluences of the branches. In this way the number of simultaneous linear algebraic equations, to be solved by matrix inversion, is limited to the number of unknowns at the nodes. Once the water-stages and flowrates are known at the nodes, the water-stages and flowrates, at intermediate points of each branch, can be found by either the double sweep method or any other technique. The difficulty of the method is the formulation of the initial system, with unknowns such as the water-stage and flowrates at each node, since it requires the development of equations relating the unknown water-levels and flowrates at the boundaries of each branch. This is obtained by formulating branch transformation equations which define the relationship between consecutive cross-sections (Schaffranek et al. 1981).

The developed techniques for the solution of the looped network involve complicated procedures. However, the algorithm is very efficient in terms of speed and storage requirements. The ability of the program to handle looped networks enables the model to simulate physical systems that were previously simulated by expensive and tedious use of two-dimensional programs.

1.3 MODEL APPLICATION

The ability of the model to simulate various river flow conditions is presented by three examples.

1. An example of unsteady flow simulation is given for an ocean estuary situated at St.Lucia lake, with predominantly tidal flow. St.Lucia lake is a shallow water body situated on the subtropical east coast of Northern Natal. It is connected to the sea by a narrow estuary. The St.Lucia lake and the estuary were studied by Hutchison I.P.G. (1976) in an effort to simulate the water and salt circulation in the system. In this example, the simulation was repeated for only a part of the estuary. The recorded water-levels at two gauges: namely, the Esengeni station upstream, and the Bridge station downstream were considered as boundary conditions. The model was calibrated against measured water-levels at an intermediate gauge. Water-levels were available for two periods (July 10-15, 1972; December 23-30, 1972) and for three recorders. Several calibration runs were performed in order to evaluate the optimum values of channel roughness. Figure 1 illustrates the comparisons between model and natural system

water-levels for the calibrated simulation. The model simulated accurately the prototype after very few simulations. It can be concluded that the model is capable of simulating the tidal propagation in a major watercourse as the St. Lawrence river. The accuracy is limited mainly by that of the available data used in the original calibration process and also in the specification of the boundary conditions.

2. The model was used to simulate a sudden turbine closure in a trapezoidal power canal. Similar numerical experiments were conducted at SOGREAH (Cunge et al., 1964), using a mathematical model of trapezoidal channel having dimensions typical to the power canals in River Rhone Valley (France). The channel was assumed to be 2000 meters in length, conveying a steady discharge of 2500 m^3/s. The boundary conditions were constant water-stage at the downstream boundary (reservoir) and variable discharge at the upstream boundary (power plant).

   Figure 2 shows the flowrate variation at the end of the channel and at 700 meters distance from the power plant. In this example the turbine closure time from 2500 m^3/s to 250 m^3/s was 1 min. It is seen that a wave reflecting from the tunnel head keeps the same sign, while a wave reflecting from the reservoir changes it sign.

3. One of the most significant practical problems is the wave propagation along a channel (i.e. the wave deformation or **wave subsidence**). In other words the flood wave becomes longer (dispersion) and lower (attenuation) as it moves downstream. In order to obtain a generalised picture of the damping of a flood wave in prismatic channels the subsidence of a range of waves was investigated using the unsteady flow simulation model. Different flood hydrographs were routed in a prismatic channel with a slope of 0,001, width of 40 m, and an initial depth of 2 m.

In practice, when a flood routing program is used for prediction purposes, or for the calculation of flood lines, the upstream hydrograph will probably be a synthetic flowrate hydrograph generated by an overland flow routing program such as WITWAT (Green, 1985); SWMM (Huber et al., 1982) etc. Synthetic flowrate hydrographs were routed over a distance of 100km. Different runs were performed depending on the distance in order to increase the accuracy. For routing over 2km, Δx was taken as 200m; over 20 km routing Δx was taken as 500m; and for 100km Δx=1km. Typical results of the modification of discharge hydrographs are presented in Figure 3. It can be seen that there can be a significant subsidence of the wave with distance.

The abovementioned examples exhibit some of the capabilities of the unsteady flow simulation model to simulate a wide range and different types of flow. The engineer using an operational system, such as OSYRIS, need only concern with the physical aspects of the problem, just as does the user of a scale model.

1.4 CONCLUSIONS

The developed suite of programs has all the capabilities of standard mainframe models, and at the same time is computationally efficient and accurate. It is applicable to any channel (branch) or system of channels (network of branches), interconnected (looped) or not, subject to backwater flow, unsteady flow, or both, whether caused by ocean tides or flood waves.

The 'OSYRIS' suite of programs can be used to determine flood-levels along urban water-courses which in turn will restrict development along streams. All the above serve as the basis for economic analysis and a more accurate appraisal of the consequences of flooding.

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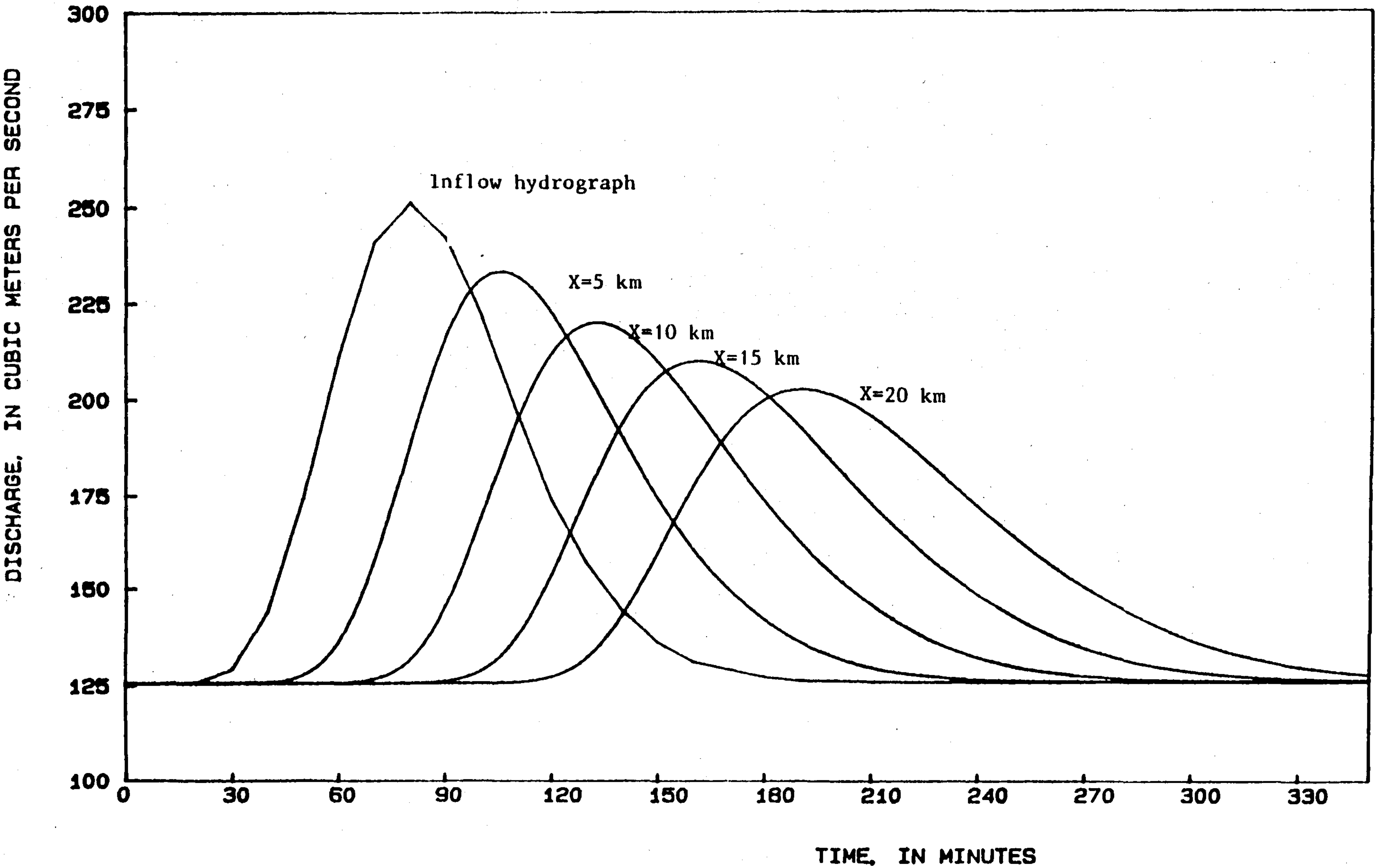


Figure Modification of discharge hydrograph with distance

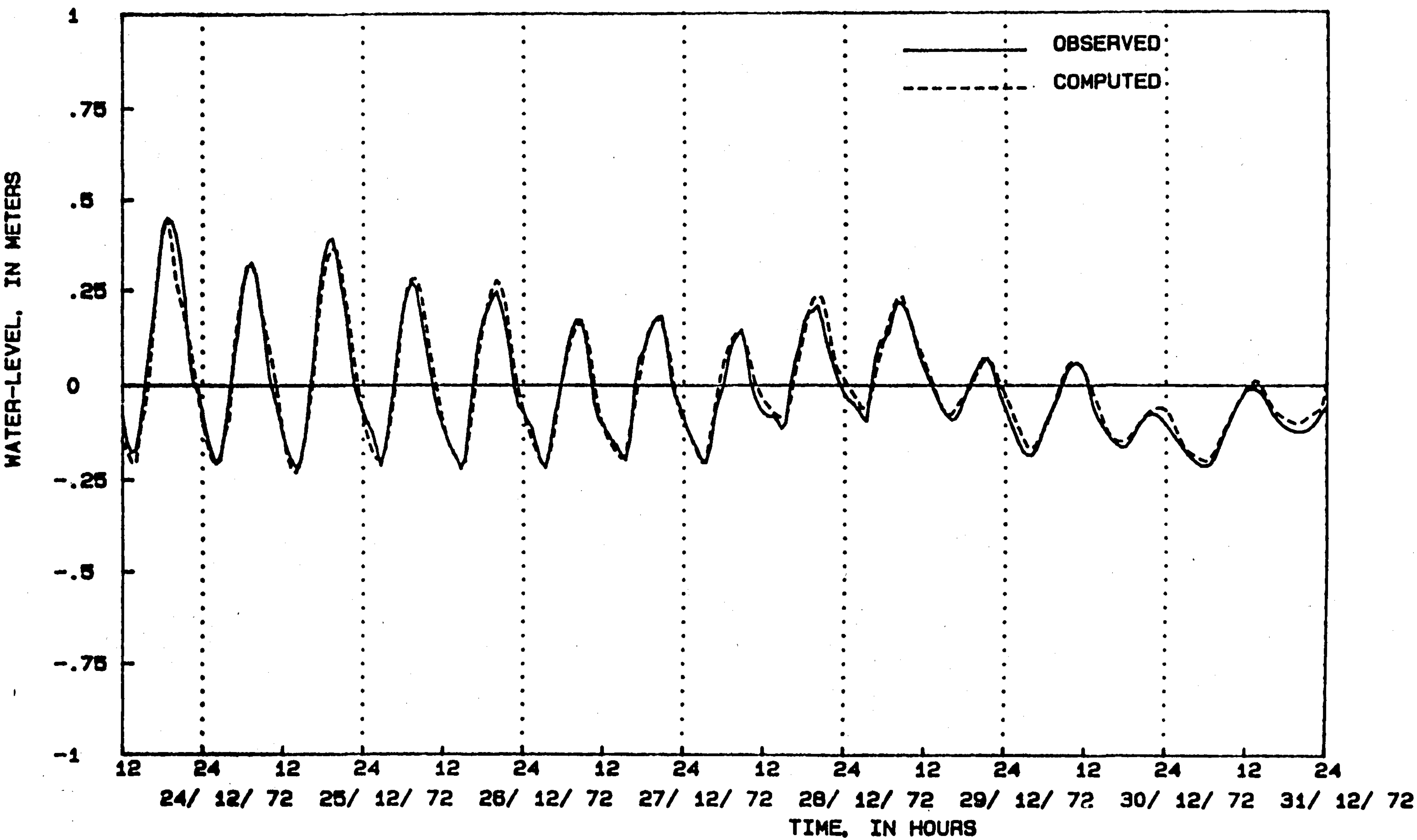


Figure Observed and simulated water-levels from the St.-Lucia lake estuary - Narrows station - December 23-30, 1972.

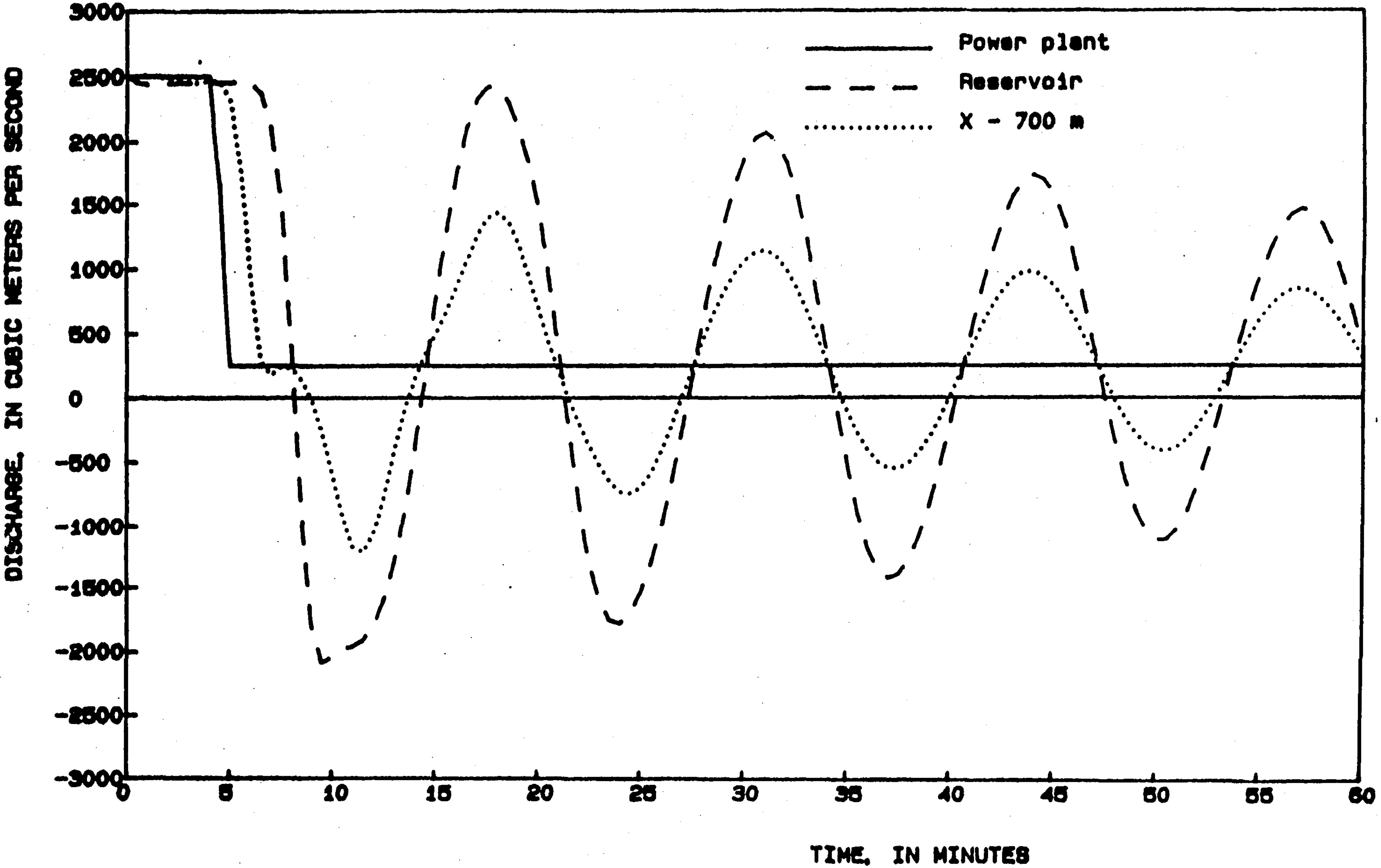
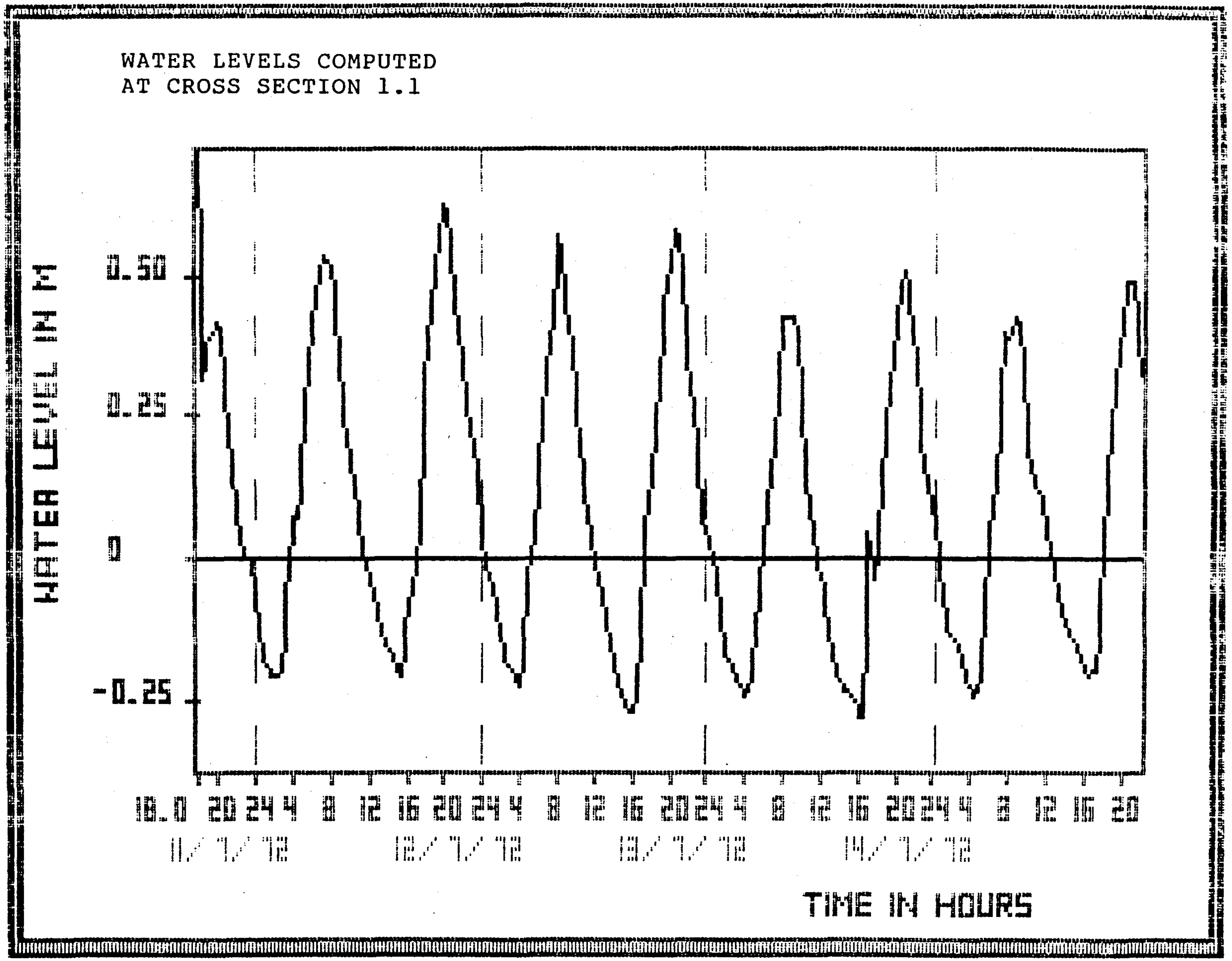


Figure Simulated flowrates - turbine closure from 2500 m3/s to 250 m3/s in 1 min



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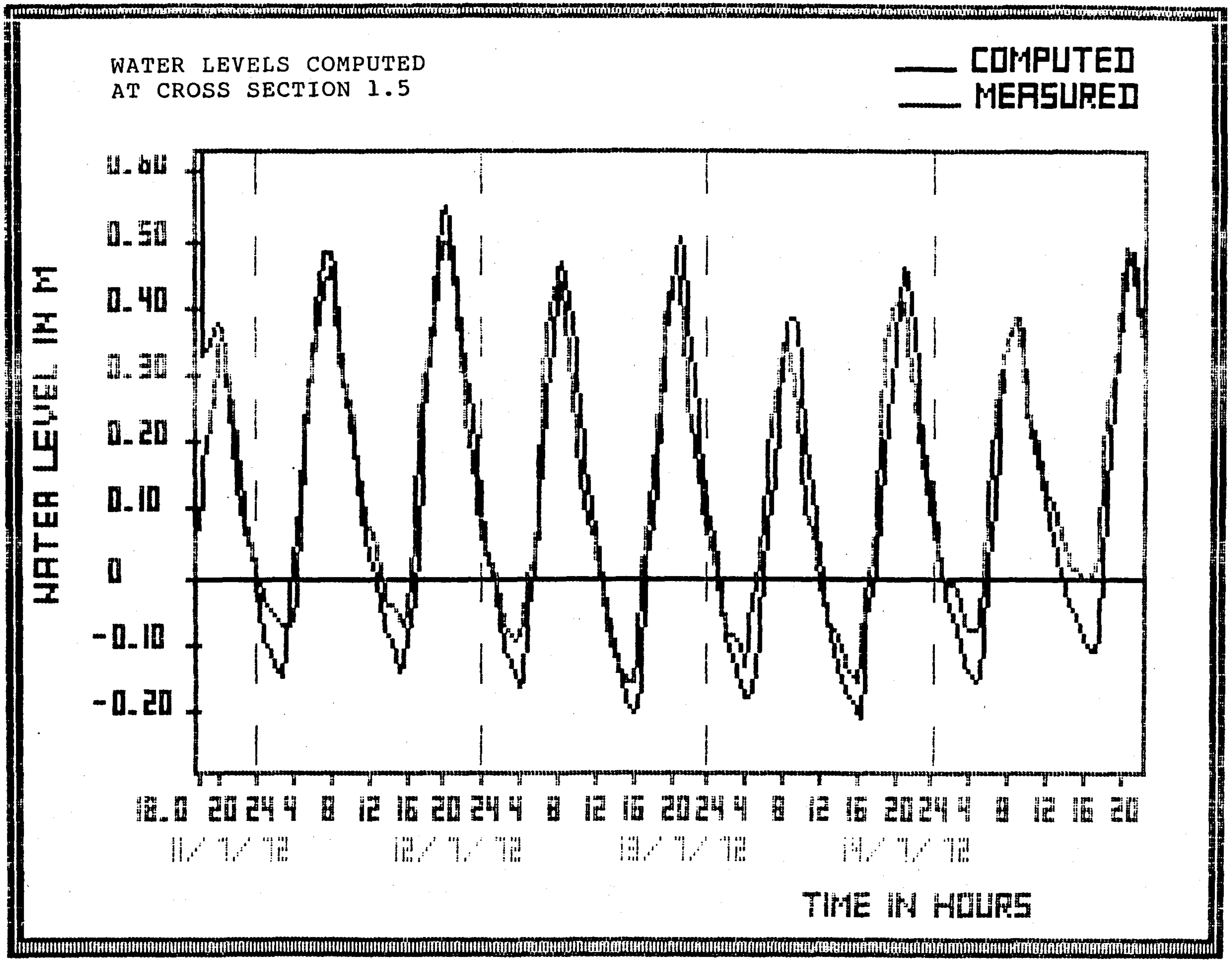

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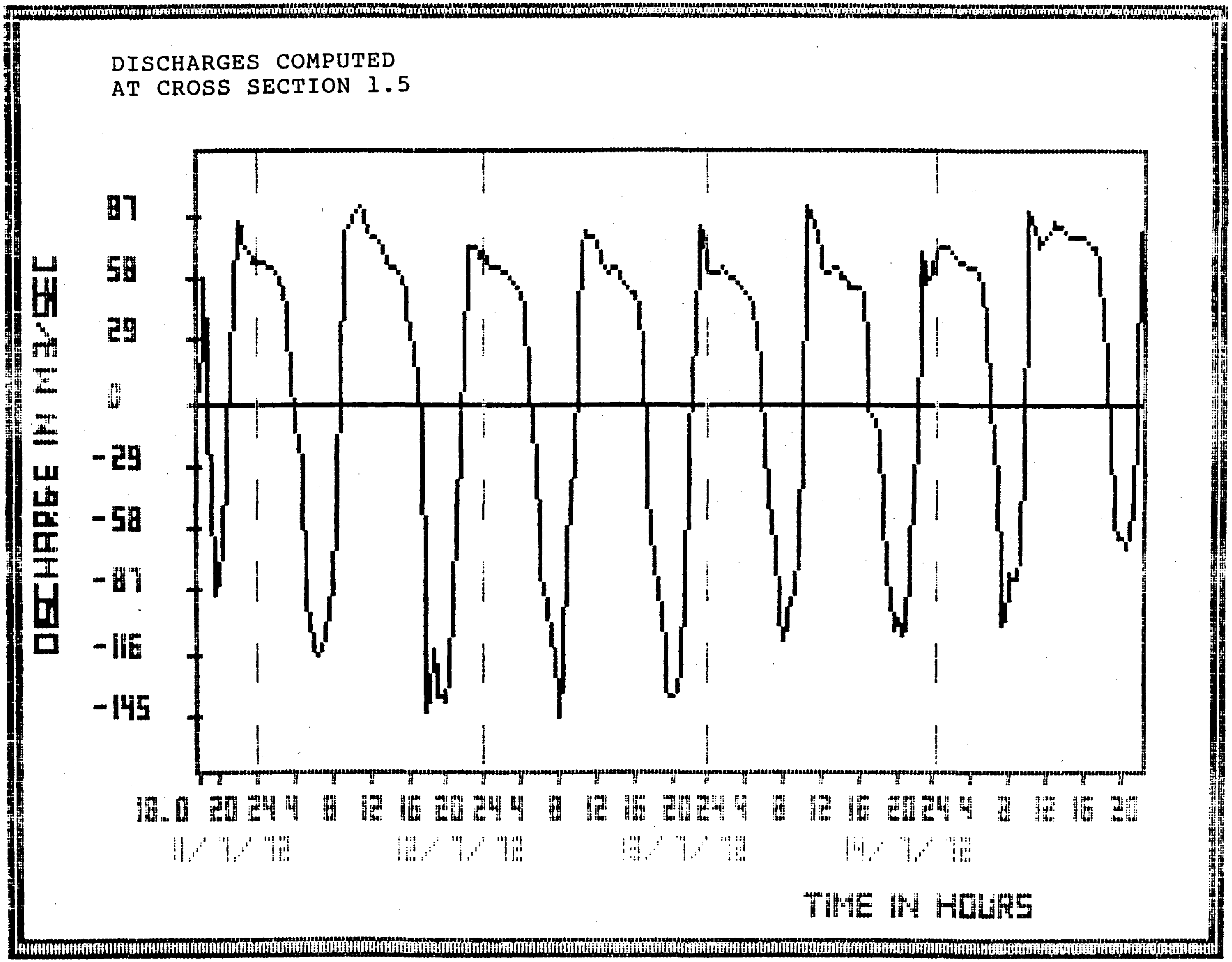

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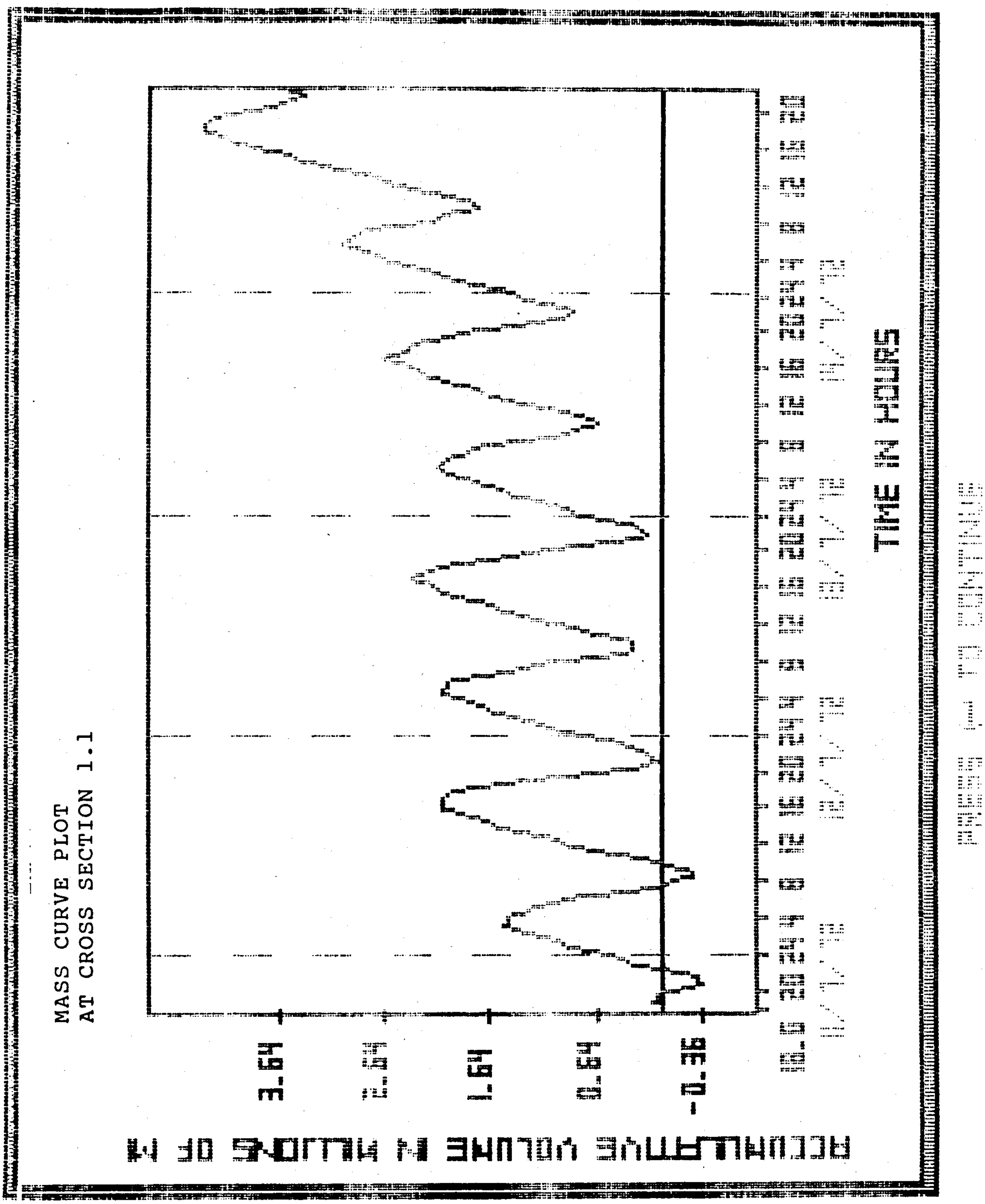
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AT CROSS SECTION 1.1
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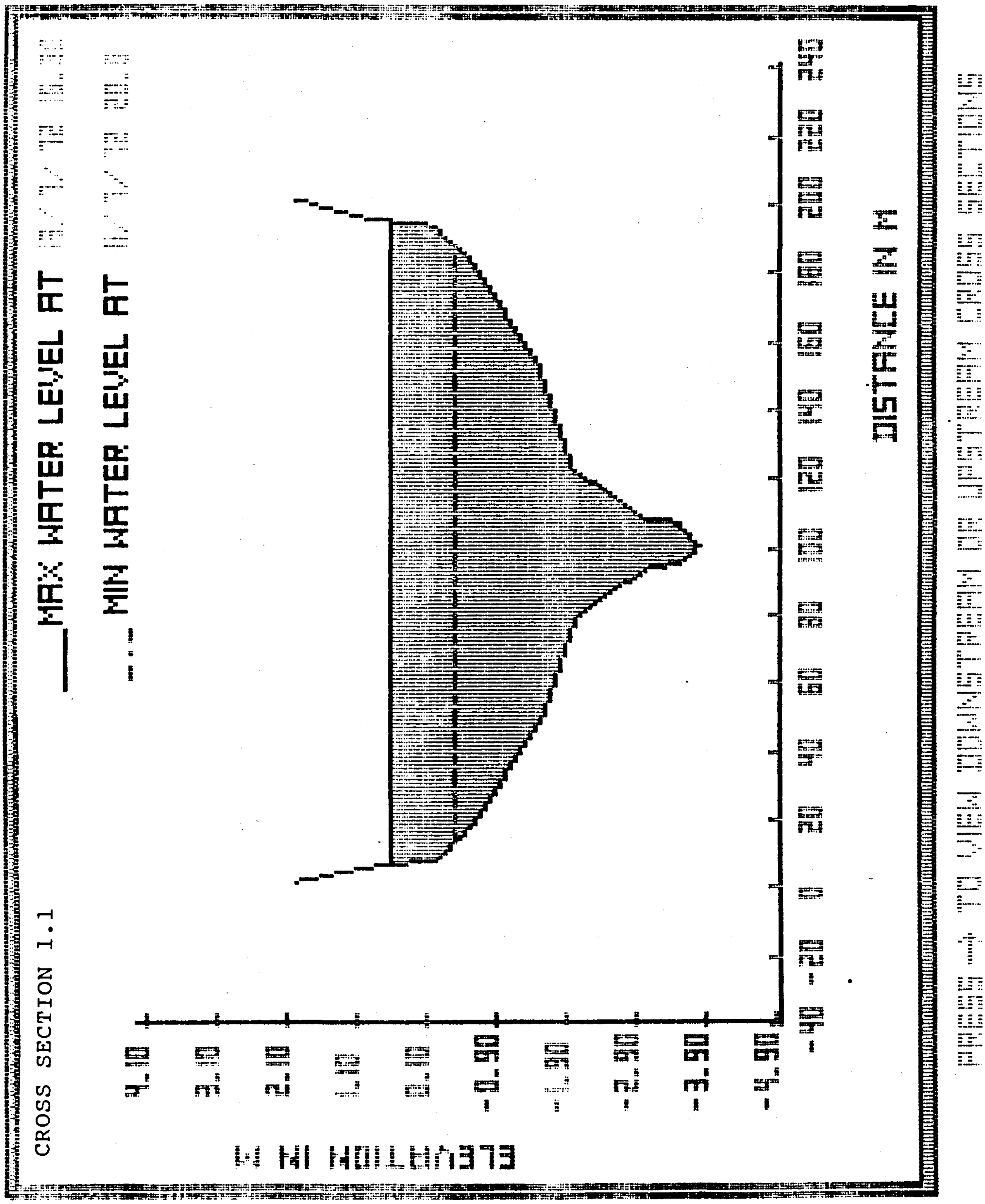
1.64
0.64
-0.36
TIME IN HOURS


CROSS SECTION 1.1
MAX WATER LEVEL AT
MIN WATER LEVEL AT
ELEVATION IN M
4.10
3.10
2.10
1.10
0.10
-0.90
-1.90
-2.90
-3.90
-4.90
-40 -20 0 20 40 60 80 100 120 140 160 180 200 220 245
DISTANCE IN M
PRESS → TO VIEW DOWNSTREAM OR UPSTREAM CROSS SECTIONS

INTERNATIONAL CONFERENCE, URBAN STORM DRAINAGE, OSAKA, 1990

SPACIAL VARIATION OF RAINFALL INTENSITIES OVER A SMALL PERI-URBAN CATCHMENT

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ABSTRACT: The variable nature of convective storms over a small area was investigated using rainfall data from a 10.4km^2 peri-urban catchment near Johannesburg. Several numerical surface fitting techniques were compared and the most applicable, the inverse squared distance method, was used to develop a contouring model. The model graphically simulated rainfall intensities over the study catchment. Analysis showed that convective storms are erratic and that rainfall intensity varies greatly within a small area. This suggests that a spatial description of rainfall at discrete time steps should be included when studying the rainfall-runoff process.

KEYWORDS: Numerical Surface Fitting, Contouring, Spatial Variation in Rainfall Intensity.

INTRODUCTION

Rainfall data for estimating runoff or infiltration is usually obtained from sparse raingauge networks with a single gauge often representing a large surface area. Modern data loggers enable accurate temporal recording of rainfall, but do not improve the spatial description of rainfall. Previous research emphasis has been on the temporal distribution of rainfall and temporal variations of hyetograph shapes for the same storm event for different gauges within a study catchment are well known.

From the literature and studies by the authors, it was discovered that storm events are composed of several cells that are the main rainfall producing components. The lifetime of these cells is short (between 10 and 40 minutes), they have large changes in rainfall intensities over short distances and their movement over an area is very erratic (Berndtsson and Niemczynowicz, 1986, Shaw, 1983 and Maaren and Brawn, 1984). The authors suggest the lack of coincidence of hyetograph shape can be attributed to the cellular structure of a storm event - an aspect that can be studied by analysing spatial distribution of rainfall.

MATHEMATICAL SURFACE FITTING TECHNIQUES AND CONTOURING

Storms can be studied using radar tracking and cross-correlation techniques. However, three major factors suggested the adoption of a numerical surface fitting approach; the study catchment covered a small area, a graphical representation of rainfall patterns was desired and high ground wind speeds are often associated with convective storms which made surface measurement of rainfall preferable. From the surface fitting techniques available, four were compared for their applicability to this study. These were the inverse squared distance, multi-quadratic, polynomial surfaces and distance weighted least-squares techniques. Krigging was not compared because sensitivity of variogram models would have to be determined first.

Artificial data sets were generated to represent a "true" surface shape against which generated surfaces could be compared. Random combinations and numbers of points from the artificial data were selected to represent raingauges, to which the surface fitting techniques were applied in an attempt to interpolate the "true" surface. Their ability to do this was tested by means of eight test statistics (e.g. coefficient of variation, efficiency, correlation coefficient) and by visual comparisons of contour maps. The inverse squared distance technique was chosen as it was the most consistent in magnitude of errors, was the fastest method (or second fastest for more than 40 data points/gauges) and did not require the solution of any matrices. For other comparisons of the methods see Shaw and Lynn, 1972, Cliff, 1975, Mclain, 1974, Heymann and Markham, 1982, Patrick, 1989 and Maaren and Brown, 1984.

A roving square technique was developed to contour the data once it had been gridded by the surface fitting technique. The roving square was applied to each set of four data points on a regular 10 by 10 grid system generated by the numerical method. The four corner heights of the square were defined by interpolated rainfall values and a fifth value at the intersection of the diagonals by averaging the four corner values.

Thus four three-dimensional triangles were postulated through which the path of a contour could be traced and plotted on a screen. A local coordinate system was used for the roving square and a global coordinate system was used for the positioning of the contours over the catchment. In this way a completely general algorithm for one triangle could be used four times by taking into account the orientation of the four positions of a triangle within the roving square. Linear scaling was used to convert the local to the global coordinate system. Thus a fast contouring method was developed to produce contour pictures of rainfall intensities over the study catchment.

For other applications such as rainfall-runoff modelling, the centre of gravity of module areas could be used as interpolation points, and rainfall distributed spatially as hyetograph input for the model.

APPLICATION TO A STUDY CATCHMENT

A peri-urban, 10.4km^2 catchment in the nort-west of Johannesburg at an altitude of 1500m above MSL was used for the study. The catchment is served by 5 raingauges two of which are of the syphon type - the other three being of the tipping bucket type with .2mm/tip resolution (see figure 1). The drop from highest to lowest point within the catchment is 200m. All 5 gauges are logged by means of clockwork chart recorders that are checked weekly.

Strip chart data for 21 storm events where all 5 gauges were active was digitised, converted into five minute rainfall intensity values and synchronised into one file. Contour maps of each five minute time interval were produced and dumped via a printer for the durations of the 21 storms.

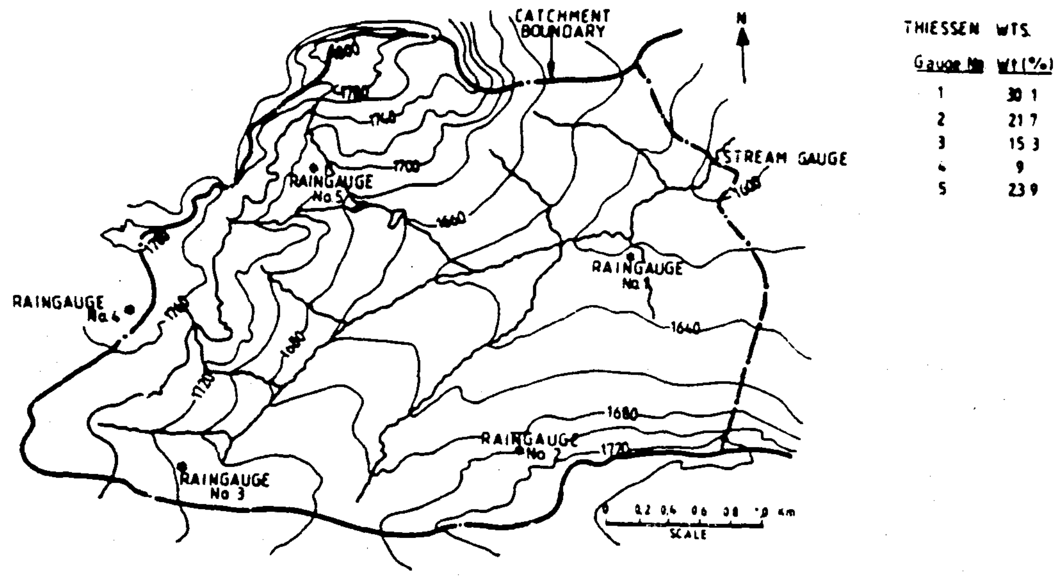

Fig. 1 CONTOUR MAP OF STUDY CATCHMENT, MONTGOMERY PARK

The spatial description of rainfall highlighted growth and decay of rainfall "pockets" over a very small area (less than 2km radius) that last for short periods of time (10 to 40 minutes). Several such "pockets" can appear for the duration of a storm event, either singularly or together. It is unknown if the peaks of such "pockets" passed over the gauges within the study catchment but observed variations from 60mm/hr to insignificant values in less than 2km (see figure 2).

CONCLUSIONS AND DISCUSSION

From the study it was found that severe variations of rainfall intensity over a small catchment are normal. Areas affected by rainfall change radically in intensity and location for the duration of a storm. The spatial and temporal characteristics of convective storms are more complicated than was realised and spatial averaging of rainfall will smooth out erratic factors that may have had an otherwise significant effect. Further studies as to the cellular make-up of storms and their erratic behaviour is necessary.

Several related points arose as a result of this study: Thiessen weights can apportion rainfall with an incorrect emphasis. The applicability of numerical surface fitting methods to large catchments is unknown but it is reasonable to expect increases in accuracy of modelling if the changing

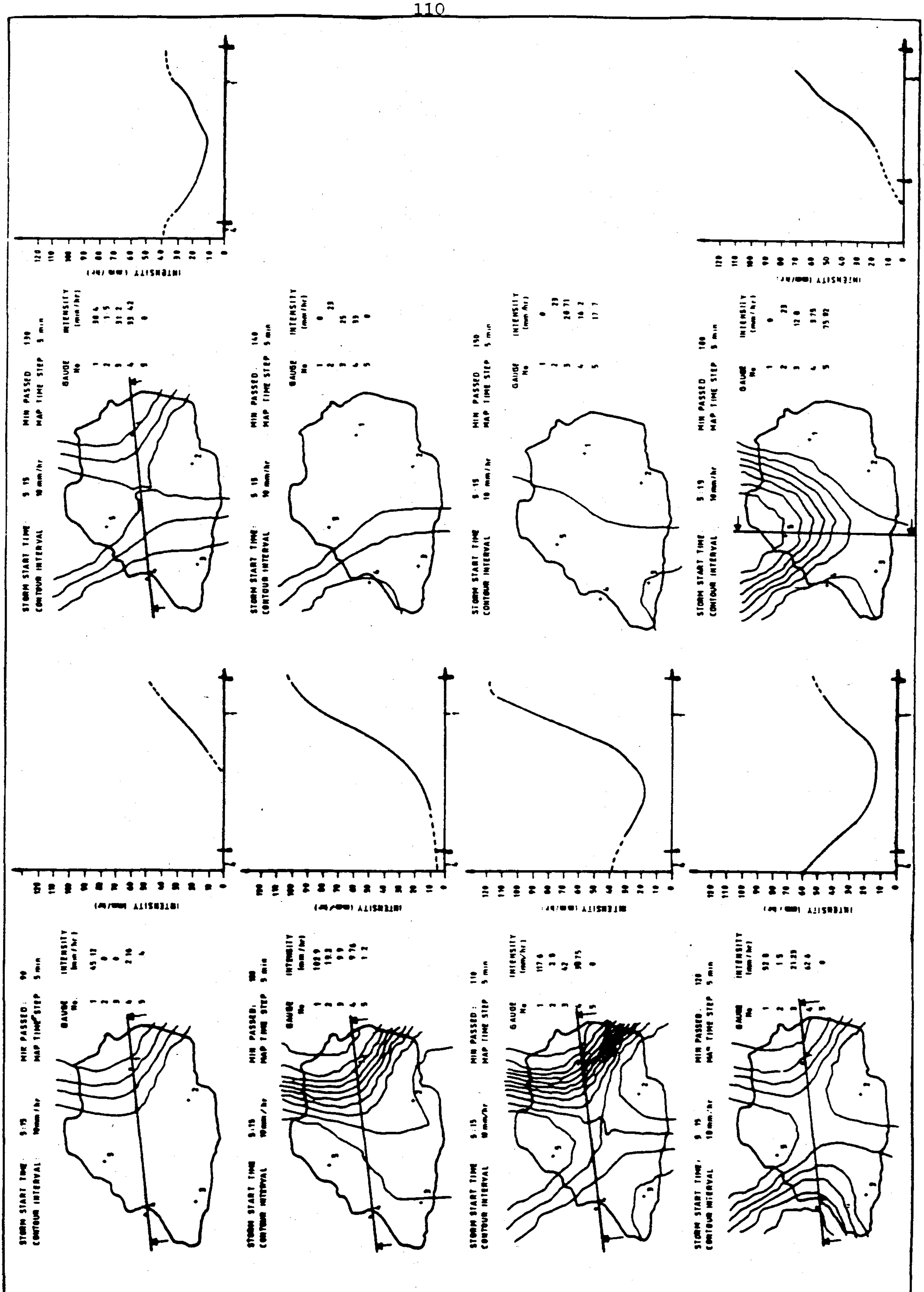

Fig. 2 RAINFALL CONTOUR VARIATIONS

patterns of storm events is included. Location of raingauges is seldom optimal for hydrological studies and densities are mostly inadequate to reflect observed spacial variations in rainfall over small catchments. Rainfall-runoff process models are describing the physical characteristics of catchments with increaseing degrees of discretisation and accuracy. Gains in this direction may be negated if the rainfall input is not similarly distributed, instead of being lumped together as a single input hyetograph as is commonly done. Incorporation of spacial variation of rainfall into runoff models should be investigated.

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INTERNATIONAL CONFERENCE ON URBAN STORM DRAINAGE, OSAKA, 1990

COMPARISON OF URBANIZED AND UNDEVELOPED CATCHMENTS

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ABSTRACT: Two adjacent similar 75ha catchments, one urbanized, the other undeveloped, were monitored. Rainfall, runoff, groundwater, water supply and sewage flows were measured. Runoff from the developed catchment was largest. Groundwater levels in the urbanized catchment appear to be dropping.

KEYWORDS: Catchment, Comparison, Groundwater, Mass Balance, Runoff, Urbanization, Water Supply.

1. CATCHMENT DESCRIPTION

Two adjacent 75 hectare catchments in Johannesburg, South Africa, were gauged in 1986 as part of a research contract (Water Research Commission of South Africa, 1989). Both are on gently sloping, granitic base topography. One catchment (Waterval) is undeveloped and is covered with grassland and some trees. The other catchment (Sunninghill) is developed for upper-middle class, residential bungalow-type occupancy, with some townhouses, one block of shops and a clubhouse.

The catchments slope down 50m over approximately 800m. There is no clearly defined watercourse in Waterval, so a cutoff trench was constructed to lead surface runoff to a measuring weir. A stone pitched channel runs down the centre of a parkway in Sunninghill, and all roofs, roadways and underground stormwater drains lead to the waterway.

A weir was constructed across the stormdrain and the separate sanitary sewer is also gauged. Water supply is monitored to individual consumers (unfortunately only on a tri-monthly basis) and a leak detection analysis indicated negligible leakage.

Geophysical surveys and boreholes were used to assess underlying geology. The depth of sandy loam overlying the granite varied from 2m to 15m, the latter depth being composed more of weathered granite particles. Dykes and fissures crossing the catchments rendered the accurate assessment of groundwater flow and aquifer storage very difficult. Water seeps to the surface at the bottom of Sunninghill, due to dyke crossing the catchment.

The Sunninghill catchment was developed about 6 years ago and is now 95% developed. There are 120 house sites plus eleven townhouse complexes. The impervious area is about 40% of the total. The mean annual rainfall is about 650m, and annual evaporation potential, 900mm.

Rainfall is measured by means of 8 tipping bucket raingauges and recorded by electronic battery operated loggers. EPROM chips are used to store the for transmission to a P.C. at monthly intervals. Borehole levels and water levels at weirs were logged from pressure sensors. Sewage flow was measured with an industrial ultrasonic gauge in a manhole and a solar cell was used for power pack charging as the current required here was greater than at the other loggers.

The borehole water level observations fluctuated and were difficult to interpret. Water levels appeared to rise shortly after storms and this was initially interpreted as due to a perched water table, but later to surface seepage into the boreholes. Even disregarding these results, the compartmentalization of the Waterval catchment by dykes made groundwater flow estimates difficult and the results presented here are largely by deduction. In both catchments, the estimate of groundwater outflow is difficult to determine, although the exfiltration of water in the Sunninghill catchment is measured at the weir.

Evapo-transpiration is also difficult to assess directly without recourse to a lysimeter which was cost-prohibitive. A weather station measuring barometric pressure, temperature, relative humidity, solar radiation, wind speed and direction was installed in the Waterval catchment. An evaporation pan installed at the same site was not considered reliable for estimation of potential evporation. Instead, equations were sought to derive the potential evaporation from the measured meteorological parameters.

Morton's (1983) potential evapo-transpiration system of equations were used together with a simple moisture budgeting calculation to derive actual evapo-transpiration (related to the amount of rainfall). The Penman-Monteith equation (Thom, 1975) is a more realistic approach to estimation of potential evaporation using meteorological parameters (De Jager et al. 1987). However, De Jager indicates that the calculation of the actual evapo-transpiration needs to include the growing cycle and vegetation type distribution of the catchment. This direction results in the water balance having to be entirely modelled which is beyond the scope of this paper.

The logged data was processed using a data management program WITDMS (Lambourne, 1988), which can summarize data in a tabular or graphical form. The open-ended architecture of the system allows data to be easily abstracted from the database for use in data-intensive programs.

The analysis of results is based on a mass balance of rainfall, water supply, stormrunoff, groundwater flow, sewage outflow, evapotranspiration and change in aquifer storage (Fig. 3). A continuous trickle of 4-6ℓ/s flows over the Sunninghill weir even in the dry season (April to October). This is attributed to seepage from two artesian boreholes near the weir. It appears that the dyke crossing the catchment in the vicinity of the weir causes the water to surface. At 4ℓ/s, this low flow averages 10 000m^3/month, which is about the groundwater flow estimated from the mass balance for Sunninghill.

In some cases the data was not available for all the months that the water balance was considered. This was mainly because of problems experienced with some of the data logging equipment (Green et al, 1987). Generation of the missing data was undertaken using the cyclic pattern of the observed data. This was especially the case with the sewage measurement record, but the variation of sewage outflow was minimal, so interpolation will not effect the results to too great a degree.

2. WATER BALANCE OBSERVATIONS

An attempt is made in Table 1 to summarize inflows and outflows to the two catchments over the limited period for which data is available. The variations from month to month are given in Figs. 4 and 5. It will be noted that rainfall is the largest input, and evapotranspiration the largest outflow to both catchments. (However bear in mind that the actual evapotranspiration is estimated using a simple moisture budgeting model which consolidated the whole catchment).

Storm runoff as a percentage of the total rainfall remains low (1%) even after urbanization. This could be because most residences have gardens for catching roof runoff. Also it is only for intense storms that surface runoff occurs from residences, as the ground is relatively absorbent.

Net infiltration into the ground remains a large component for both catchments, and again is only assessed by deduction. Attempts to model the aquifer with permeabilities and storage coefficients were highly speculative as the results were very sensitive to input assumptions. This is not entirely surprising owing to the nature of the geological structure in the area.

There appears to be a slight decrease in contribution to the groundwater over the 3-year period but this may be because of decreasing rainfall. Results over a longer period will therefore be of value.

3. DISCUSSION

It is generally accepted that urbanization leads to increased runoff, particularly stormwater flow. Impervious cover reduces infiltration, therefore primarily leading to recession of the water table, and drying of the upper soils.

The observations at Sunninghill indicate that urbanization in the form of residential housing results in limited change in groundwater levels. Despite the large proportion of water supply draining off through sewers (70%), a large proportion is retained, presumably for garden usage, which could affect groundwater levels, albeit indirectly by reducing evapotranspiration from sub-terranean aquifers.

Surface stormwater runoff remained a small proportion (1%) of total rainfall from the urbanized catchment, (0.2%). Subsurface seepage appears a large outflow from both catchments. Whereas surface stormwater in the Sunninghill catchment is derived from the total area, only the bottom (more clayey) area of the Waterval catchment produces surface runoff.

In order to achieve a more accurate comparison of the two catchments; firstly a greater length of record is needed, and secondly a more rigorous method of calculating actual evapo-transpiration has to be used. In the latter case the different vegetation and land use should be considered. Groundwater flow will then be obtained by subtraction.

Table 1 : Summary of Average Flows in m³/Month, 1986-88

CATCHMENT	SUNNINGHILL (Devel.) Average	1986-88	WATERVAL (Undevel.) Average	1986-88
Inflows				
Water Supply	5000	(4000-6000)	-	-
Rainfall	35000	(80000-20000)	35000	(80000-20000)
TOTAL	40000		35000	
Outflows				
Sewerage	3500	(3000-4000)	-	
Storm Runoff	3000	(4000-2000)	600	(1000-200)
Evapotranspiration	25000	(50000-10000)	25000	(40000-10000)
Groundwater Flow	10000		10000	
TOTAL	41500		35600	
Net Decrease in Groundwater Storage	1500		500	
Average Decrease in Depth of Groundwater	60mm p.a.			

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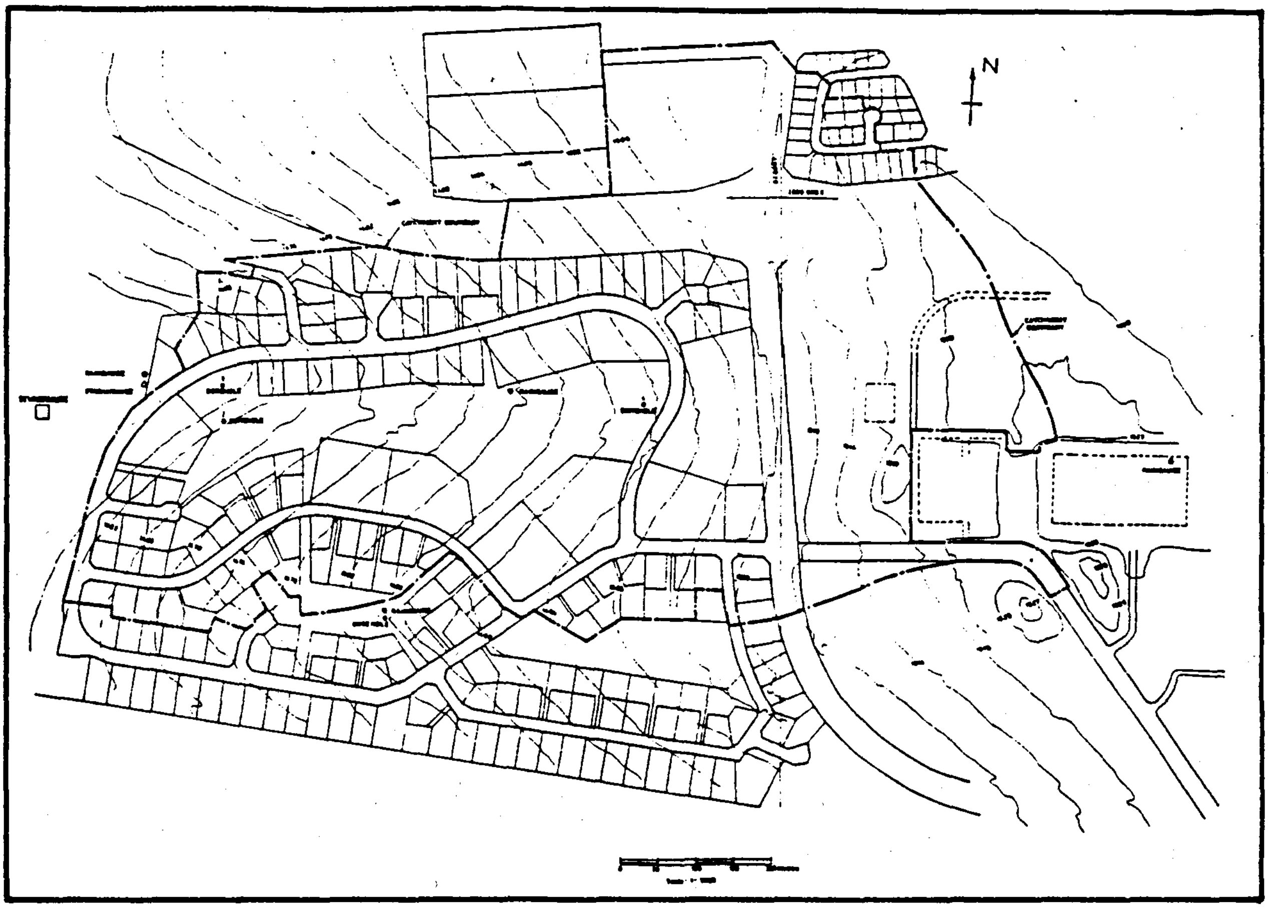

Fig. 1 Map of Urban Catchment

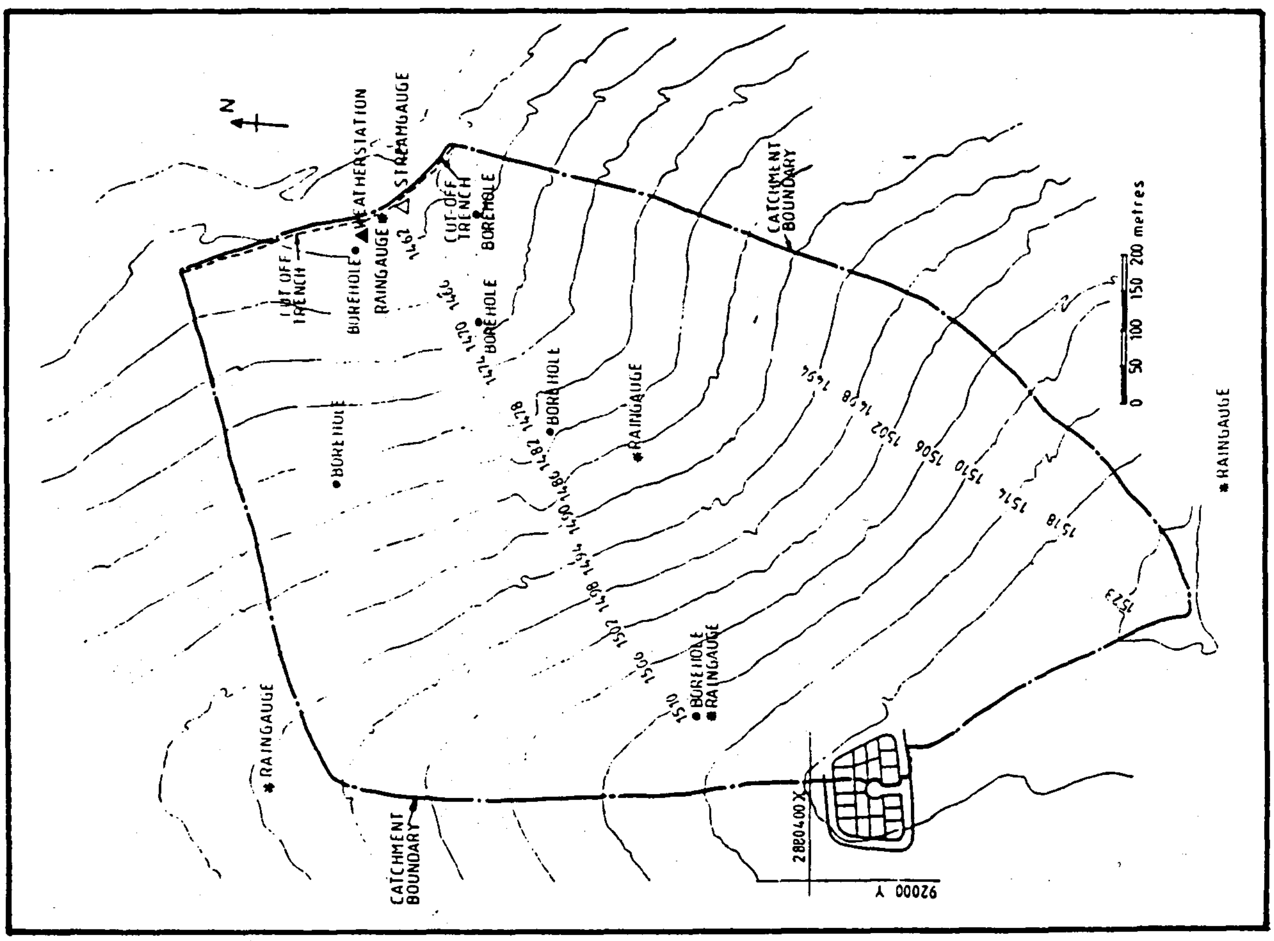

Fig. 2 Map of Undeveloped Catchment

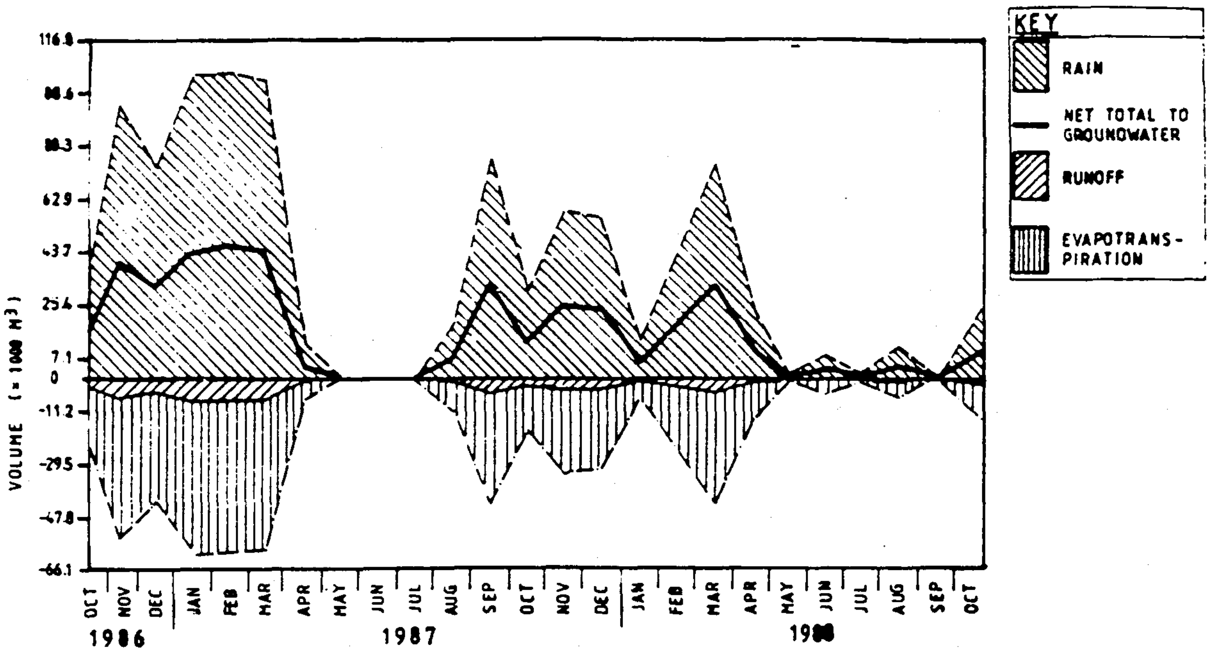

Fig. 3 MONTHLY FLOWS, WATERVAL (UNDEVELOPED CATCHMENT)

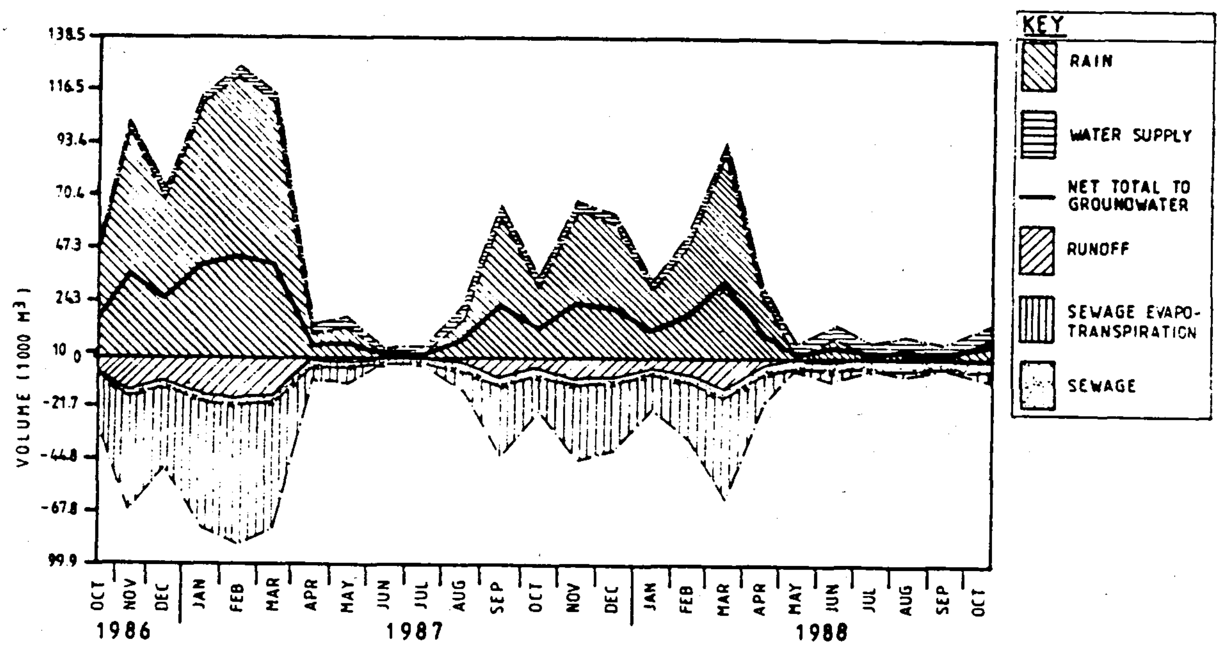

Fig. 4 MONTHLY FLOWS, SUNNINGHILL (URBANIZED)

INTERNATIONAL SYMPOSIUM ON STORM WATER MANAGEMENT, KUALA LUMPUR, 1990

EVALUATION OF DIFFERENT STORMWATER MANAGEMENT POLICIES FOR A PROPOSED HOUSING DEVELOPMENT

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ABSTRACT

In this paper, the stormwater management computer model WITSKM is described as well as the application of the programme to the 75 ha Waterval catchment outside of Johannesburg, South Africa. This catchment is at present undeveloped and the rainfall and runoff from the catchment has been monitored since 1987. A proposal is in the process of being submitted for developing the catchment as a residential area and WITSKM has been used to examine the effect that the different possible stormwater management options for the proposed township could have on the runoff from the catchment. Included in the possible options are such management strategies as retention storage dams, dual drainage, and use of flood plains. By using the recorded rainfall events, the hydrographs resulting from the simulation runs for the different management policies can be compared to the recorded hydrographs. In this way not only can the effects of urbanization on the runoff be determined but the capabilities of the different management options in attenuating floods can be compared and evaluated.

INTRODUCTION

The Water Systems Research Group(WSRG) has a contract to investigate the effects of urbanization on the water balance of catchments. One of the aspects of this study is to examine the effectiveness of different drainage system design approaches in limiting the increase in runoff peaks and volumes caused by urbanization. Rainfall and runoff events have been recorded on the Waterval catchment which is at present in its natural state. This catchment has an area of 75 ha and is situated outside of Johannesburg, South Africa. Plans are in the process of being approved for the development of this catchment as a residential area. The only method that can be used to determine what the effects of such a development could be on the runoff from the catchment is that of computer simulation. The simulation program WITSKM(Witwatersrand Stormwater Kinematic Model) (Stephenson,1989) has been used with recorded rainfall and runoff events, to examine the capabilities of different stormwater drainage system designs in coping with the increased runoff due to the urbanization of the catchment. Such stormwater management strategies as the use of detention storage facilities, flood plains, and dual drainage have been incorporated into the drainage system designs considered.

DESCRIPTION OF MODEL

The modular approach as used in SWMM(Huber et al,1982) was adopted in WITSKM because of the flexibility that can be achieved in modelling a catchment and, for the analysis of stormwater management strategies, the different hydrological units can easily be linked together to model the required stormwater policy. The modules that are available are overland flow planes(permeable or impermeable), aquifers, pipes, channels, and storage basins. With the inclusion of aquifer modules in WITSKM the process of interflow and subsurface flow, which can be the dominant runoff process, particularly in a rural type catchment, can be modelled. This increases the area of application of the program not only to urban catchments but to rural catchments as well. The Green and Ampt(Mein and Larson,1973) infiltration model has been used in the program as this is based on physically measurable parameters as opposed to the Horton type approach which is purely empirical.

In WITSKM the kinematic routing approach is used for the routing of flow over permeable and impermeable catchments as well as through pipes and channels. The method of solution of the kinematic equations adopted in the program is a form of Muskingum-Cunge routing procedure suggested for channel routing by Cunge(1969). This approach of matching the physical diffusion and the numerical diffusion has been used for overland flow routing by Ponce and Yevjevich(1978) and compared by Ponce(1986) to the more traditional kinematic routing methods. The method used in WITSKM is described by Holden and Stephenson(1988).

STORM EVENTS USED IN ANALYSIS

For the analysis of the different stormwater management options and for the comparison of the WITSKM output to the recorded output for the catchment in its natural state, three storm events that resulted in runoff were selected from the catchment data. Event 1 occurred on the 3 February 1987, event 2 on the 21 March 1987, and event 3 on the 26 September 1987. The rainfall for events 1 and 3 had a triangular distribution while the distribution for event 2 was approximately uniform. Using the equations describing the Intensity-Duration-Frequency curves for the inland region of South Africa, the recurrence intervals for the rainfall events were estimated and these together with other pertinent data are presented in Table 1.

TABLE 1. Rainfall and Runoff Information for Recorded Storm Events

Rainfall					
Event	Peak Intensity (mm/h)	Duration (mins)	Volume (m^3)	Time to peak (mins)	Rec Int
1	142.0	60.0	36306	20	30
2	21.4	120.0	15926	-	1.4
3	67.0	50.0	16962	25	2
Runoff					
Event	Peak Runoff (m^3/s)	Duration (mins)	Volume (m^3)	Time to Peak (mins)	
1	0.23	90.0	615	30	
2	0.00	-	0.0	-	
3	1.80	110.0	3723	15	

The Waterval catchment falls in the summer rainfall region of South Africa. Event 3 was the first rainfall event of the rainy season and occurred on a bare catchment. It is also suspected that a relatively impermeable crust is formed on the soil surface in the lower reaches of the catchment during the winters. This would explain the higher runoff volumes and peaks generated by event 3 when compared to the runoff for the higher recurrence interval event 1.

GENERAL DESCRIPTION OF TOWN LAYOUT

The urbanized Sunninghill Park catchment which has the same catchment area and is adjacent to the Waterval catchment is also monitored by the WSRG. The proposed development at Waterval will be of a similar nature to that of Sunninghill Park. Of the plots in the proposed town 70% will be developed with houses and 30% of the plots will be made available for the development of town house and flat complexes. The sizes of the plots designated for houses, range from 1000 m^2 to 1500 m^2 with the percentage imperviousness in the order of 25%. The degree of imperviousness for the town house and flat complexes will be higher in the order of 50%. The infiltration and roughness parameters for the various land use types that were obtained when calibrating WITSKM for the Sunninghill Park catchment were used on the Waterval dvelopment. The layout of the town and the details of the discretization, pipe and road networks are shown in Fig 1. The level of the discretization used to model the catchment is coarse with the exact positions of the plots and access roads into the various land use areas not being detailed.

In order to accommodate the possible drainage options, a park area that can be used as a flood plain has been included in the township layout. This area runs down the centre of the town and discharges into the natural outlet of the catchment. A natural or lined channel can be included in the park area for the simulation of the dual drainage option. The pipe network can also drain into this area at various points along its length. No development has been planned at the outlet of the catchment to make provision for the inclusion of a detention storage pond and areas that can act as temporary storage for excess water.

DESCRIPTION OF DRAINAGE SYSTEMS

Introduction

Six possible drainage systems were considered for the catchment. They proceed from a fully linked system of large conduits which will give the worst case as far as the production of runoff is concerned through the flood plain and dual drainage options to the use of detention storage facilities which could be in the form of a dam or temporary storage of runoff on a sports field. For all the simulation runs for the drainage systems considered the same antecedent moisture content was used.

System 1 : Fully Linked Drainage System

This option represents the worst in storm drainage design from the cost and runoff point of view. The pipe drainage network is designed to take all the flows from the roads and developed areas. The pipes are then linked directly to a lined channel running down the centre of the flood plain which discharges at the outlet of the catchment. The channel has also been designed so as not to surcharge onto the surrounding flood plain. The linking and roughness of the conduit network will cause the catchment to respond quickly to rainfall and the catchment will be susceptible to the shorter duration higher intensity storms. This type of drainage system will result in large pipe networks and expensive lined channels. As most of the runoff is confined to the pipe and channel network, the residents will not be frequently inconvenienced by flooded roads and park areas. The question however arises as to what size storm the drainage system should be designed for.

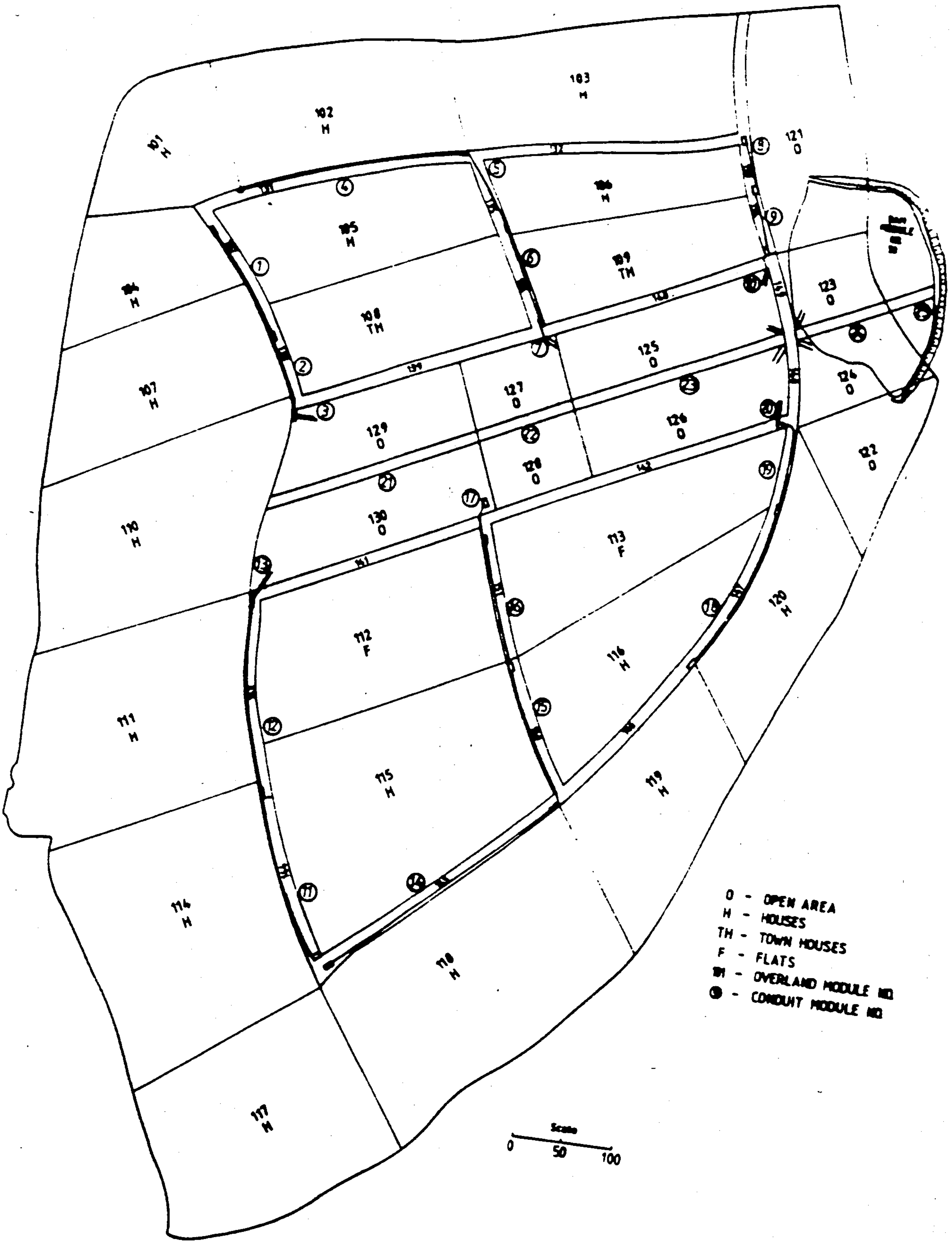


Figure 1. Proposed Town Layout

System 2 : Flood Plain System

For this system the pipe and channel networks of system 1 were maintained, however the pipes were disconnected from the channel network and allowed to discharge onto the flood plains. In this way infiltration is encouraged and the runoff is spread over a larger and rougher surface which results in smaller flow depths and hence greater retardation and temporary storage of runoff. To model the flood plain cum channel system, the park area (see Fig 1.) is subdivided into overland flow modules and channels. The runoff into a channel is from the upstream channel and from the overland flow modules immediately upstream. Referring to the township layout shown in Fig. 1, the flow into channel 23 for example is from channel 22 and overland flow modules 127 and 128.

System 3 : Dual Drainage System

A dual drainage system consists of a minor and a major drainage system. The minor drainage system consists of the pipe and channel network, which is designed to cope with lower recurrence interval events. The major drainage system, in this case the roads and flood plain, is designed to cater for the rarer higher recurrence interval events. In this way the size of the pipe and channel system can be reduced thereby lowering the costs of the drainage system. The question arises in designing these systems as to which event the minor system should be sized to cope with so as not to inconvenience the users too frequently. Event 3 is estimated to be a 2 year event and was used to size the minor drainage system. The pipe network was, as in the case of system 2, disconnected from the channel network. Unlined rougher channels were used and were sized so as not to surcharge for this event. For event 1, the pipes were made to surcharge onto the roads and the channels onto the adjacent flood plain modules.

System 4 : No Pipe Network System

A further system considered was to remove the pipe reticulation network entirely and use only the roads to remove the stormwater to the catchment outlet. The water was kept on the roads until road sections 148 and 149 from where the water was routed into channel 24. This system will be a low cost system but inconvenient for the the inhabitants of the town.

System 5 : Temporary Storage System

The drainage system of system 3 was altered so that the water surcharging from channel 24 could be stored temporarily on an open area such as a sports field. For this purpose overland flow modules 123 and 124 were given a flatter slope and a greater roughness to act as temporary storage for the surcharged water. Being constructed at the bottom of the catchment, this option holds little advantage for the residents of the town. This option will however reduce the flood peaks that could be expected downstream which would be beneficial for the downstream communities.

System 6 : Detention Storage System

As in system 5 the dual drainage option of system 3 was adapted to route the flows into a detention storage pond at the bottom of the catchment. The type of detention facility used had a 0,5m wide and 0,5m high culvert outlet and a spillway. The site at the bottom of the catchment could support a 7 m high dam wall giving a storage capacity of 62000 m^3.

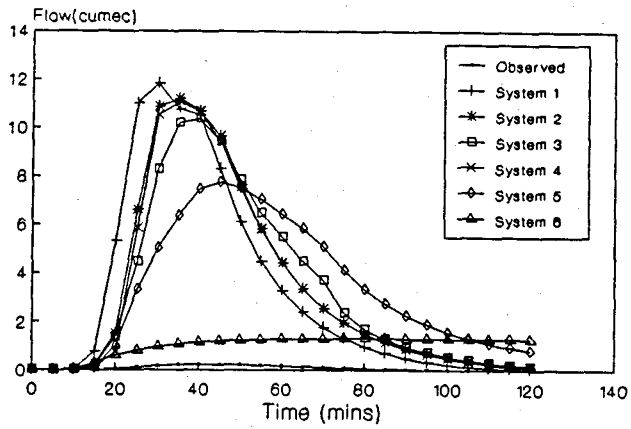
DISCUSSION OF RESULTS

A summary of the results of the simulations are presented in Table 2 and shown plotted in Fig. 2. In Table 3 the maximum depths on the flood plain and roads are presented for the different drainage systems and events. The results show that developing the catchment will increase the runoff volumes and peaks with the ratio sim/obs varying from 52 to 0.5 for the peaks and from 36 to 1.2 for the volumes. The simulations show that by making use of rougher and wider overland flow planes instead of confining the runoff to a conduit

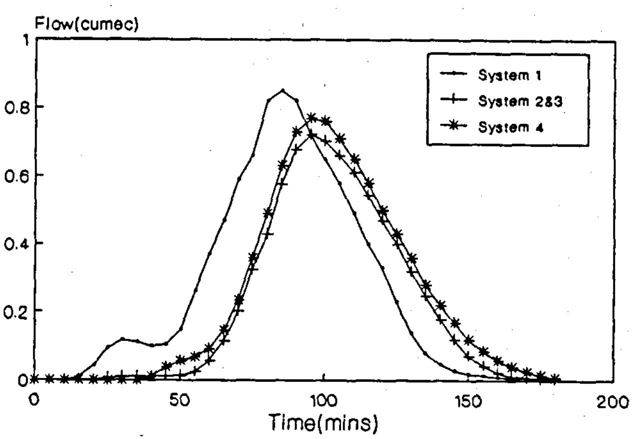
TABLE 2. Results of Simulations

Event 1					
System	Peak (m^3/s)	Ratio of Peaks Sim/obs	Time to Peak (mins)	Volume (m^3)	Ratio of Vols Sim/obs
1	11.9	52	30.0	22037	36
2	11.2	49	35.0	21594	35
3	10.4	45	40.0	21538	35
4	11.0	48	35.0	21892	36
5	7.8	34	45.0	21083	34
6	1,3	6	70.0	21538	35
observed	0,23	-	45.0	615	-
Event 2					
System	Peak (m^3/s)	Ratio of Peaks Sim/obs	Time to Peak (mins)	Volume (m^3)	Ratio of Vols Sim/obs
1	0.85	-	85.0	1258	-
2 & 3	0.72	-	95.0	469	-
4	0.78	-	95.0	998	-
observed	0	-	-	-	-
Event 3					
System	Peak (m^3/s)	Ratio of Peaks Sim/obs	Time to Peak (mins)	Volume (m^3)	Ratio of Vols Sim/obs
1	3.9	2.2	40.0	5294	1.4
2 & 3	3.3	1.8	45.0	4634	1.2
4	3.5	1.9	40.0	5133	1.4
6	0.89	0.5	65.0	4634	1.2
observed	1.80	-	30.0	3723	-

Event 1
3 February 1987



Event 2
21 March 1987



Event 3
26 September 1987

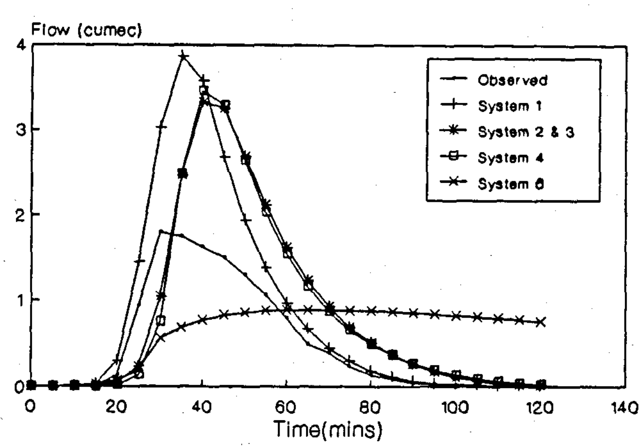


Figure 2. Plots of Observed and Simulated Hydrographs

TABLE 3. Maximum Flow Depths(m) on Roads and Flood Plains

Event 1						
System	1	2	3	4	5	6
Roads	0.014	0.014	0.073	0.114	0.073	0.073
Flood Plain	0.026	0.095	0.222	0.147	0.246	0.026
Event 2						
System	1	2 & 3	4			
Roads	0.002	0.002	0.022			
Flood Plain	0.002	0.024	0.036			
Event 3						
System	1	2	3	4	5	6
Roads	0.009	0.009	0.009	0.059	0.009	0.009
Flood Plain	0.008	0.043	0.043	0.079	0.080	0.008

network, the runoff peaks and volumes can be reduced. The greatest reduction being achieved for system 3 where surcharges from the conduit system are spread over the roads and central flood plain. A reduction of some 10% being achieved for event 1. However the systems utilizing the storage option in addition to the dual drainage of system 3, caused the largest reductions in peak and volume with the detention storage facility coming the closest to limiting the flow to the recorded runoff for event 1.

Although the use of roads and flood plains as conveyors of excess runoff result in cheaper drainage systems and have been shown to result in lower runoff volumes and peaks, these types of drainage systems can be inconvenient for the immediate community served by the system. This is shown in Table 3 for event 1 where the flow depths on the roads reach uncomfortable levels for drainage systems 3, 4, 5, and 6. In particular system 4, where a pipe system draining the roads has not been used. The flood plains are also inundated for event 1 especially at the catchment outlet for drainage system 5 where the water is stored temporarily on an open area.

CONCLUSIONS

The flexibility and capabilities of the stormwater simulation model WITSKM have been demonstrated in its application to the analysis of various drainage systems for a proposed development. The results of the analysis have benn used to assess the abilities of various drainage systems in limiting the runoff due to urbanization and to assess the extent of these increases when compared to the catchment in its natural state. The increases in peaks and volumes of runoff for event 1 are somewhat dramatic while the results for event 3 are more in line with those reported in the literature (Whipple et al,1983). More light will hopefully be shed on the reasons for the high runoff for event 3 with the capture of more rainfall events before the development of the town. The accuracy of the computer simulation techniques in predicting the runoff for the development will be able to be assessed once the town has been established.

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FOURTH INTERNATIONAL CONFERENCE ON URBAN STORM DRAINAGE

THE IMPORTANCE OF DUAL DRAINAGE

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1.0 SUMMARY

It is demonstrated that dual drainage systems can considerably reduce flood risks when compared with single conduit urban storm drains. Underground pipes should only convey minor events as they flow fast when full. Shallow roadways or flood plains will take the balance at lower velocities and therefore reduce peaks. Surface conveyance systems can also take extreme events with little increase in flow depth.

1.1 INTRODUCTION

Urbanization has a number of effects on storm runoff and catchment water balance. The most immediate factor which springs to mind is the reduction in impermeability resulting in greater runoff. However, the connection of impervious areas to water-courses is of even greater significance, since a large proportion of runoff from impervious areas can be infiltrated on reaching pervious ground.

1.2 HYDRAULICS OF RUNOFF

An important consideration in the design process of urban drainage systems is an appreciation of the effects of future urbanization on runoff patterns. Urbanization may cause changes in different factors such as imperviouness, channalization, roughness, infiltration etc. The increase of the channalization and of the imperviousness in an urbanized catchment takes place simultaneously and results in discharges of greater runoff volumes in shorter times. The reduction of infiltration and initial abstractions are the result of the increase in impervious area which inhibits infiltration and leads to larger volumes of direct runoff. The reduction in roughness caused by paving and roofing can have an indirect effect in increasing design flows to a greater proportion than would be expected (see Table 1). Likewise the construction of efficient channels reduces concentration times and thereby increase design storm intensity if it is assumed the catchment reaches equilibrium for the critical storm. This is not the necessarily the case for many catchments however

(Stephenson and Meadows, 1986), and what the increased hydraulic conductivity does for large catchment is in fact to increase the contributing area at peak design flow for any particular risk.

The overland flow and the underground pipes consist the major and the minor system in most stormwater drainage systems. These systems invariably exist in an urbanized catchment irrespective of whether they have been planned or designed for. The dual nature of the drainage system makes the problem of analysing the effects of urbanization more complex, but even so drainage analysis or design cannot be performed effectively if the dual factor is ignored.

In traditional design of drainage systems the dual nature generally was not taken into account and the increased runoff was conveyed by deepening and lining existing channels or enlarging pipes and culverts. This resulted in drainage systems overdimensioned and overloaded. On the contrary, the effect of channalization or conduits can be reduced by utilising overflow systems which convey the water at slower velocities. Such drains which would be designed to take major events would act as flood plains by retarding runoff due to the small hydraulic radius (depth) and absorbing the volume (a form of channel detention storage). Such conveyance systems can be incorporated in roadways at design stage, since major events e.g. greater than 2-year recurrence interval can have only minor disruption if flooding is over a brief period and shallow. Economic developments may thus be designed on this basis, whereas more affluent areas may prefer to pay for less inconvenience.

1.3 SIMULATION OF DUAL DRAINAGE SYSTEMS

To design and evaluate the performance of dual drainage systems, runoff quantity should be accurately modelled. Since most of the available urban drainage models (SWMM, ILLUDAS) have not the capability of simulating dual systems directly, an existing kinematic model, WITWAT, was modified to incorporate routing through compound channels with flood plains. The structure of the developed model was based on a paper by Alley et al. (1980) which utilizes a four point finite- difference mesh.

Generally the kinematic method incorporates the assumption that the discharge at any point is a function of the water depth or the area only, i.e Q=aA . However, for most cross-sections a direct relation of this type is inaccurate and thus in the developed model the flowrate is deduced from the flow area of the cross-section and vise-versa, using a Newton-Raphson technique. In this way the WITWAT model has the capability of considering any kind of cross-section, including pipes with compound channels above, and therefore simulating dual systems. When the pipe runs full, the excess water instead of being transmitted to the downstream pipe, as in most models is routed through the section above. Thus flow in pipes can pass alternately from free surface to pressurised conditions and back again.

Using the WITWAT model, on an HP 200 series micro, the dual systems of two catchments were analyzed. The first study area was the Upper Braamfontein Spruit in Johannesburg catchment which is a fully developed urban area comprising high-rise buildings and high density housing development. For minor events most of the pipes run partly full and the major system does not function. However, under a severe event (Chicago storm with a recurrence interval of 20 years) the conduits in the drainage network are surcharged and the excess runoff runs down the streets. In order to illustrate the effect of the major system the capacity of the conduits was increased until no surcharged occured. In Figure 1 are plotted the simulated hydrographs for the catchment when the excess water is routed through the streets and when the overdimensioned drainage system is used. The major system reduces the peak by approximately 10%. 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. The very short concentration time accounts for the fact that the major system does not cause as big as retardation and reduction of the peak flowrate as one might expect.

The effect of the major system becomes much more evident analysing a larger catchment along the Braamfontein Spruit. This is a flatter catchment with longer channel lengths for routing. Braamfontein Spruit is a watershed designed using the 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 the design become clear through the simulation analysis (Figure 2). For 50-year HUFF design storm (three hours duration) the peak discharge at the outlet was more than 90m /sec and the time of concentration for such a large catchment less than an hour.

Even in such 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 was made for major and minor systems and both were designed for the same design frequency.

Using the WITWAT model one can simulate the effect of the major system, by designing the minor system for shorter design frequency (1.5 years). Now the amount of excess water running in the streets is severe and the flood plains at the outlet of the catchment are surcharged. A marked reduction in the peak occurs, namely 30% and the hydrograph is attenuated (Figure 2). On the other hand, detention storage was found to be less effective unless huge areas were inundated.

Simulating severe rainfall events, it was shown that urbanization in Braamfontein Spruit overloaded the drainage system and that the major system could reduce the peak flowrates considerably. Even greater reduction of the peak flowrates could be achieved by combining the dual drainage design with flood storage schemes or pervious surfaces.

1.4 RISK OF EXCEEDANCE

The use of open channels for conveyance and retardation detention of extreme floods has the added advantage of greater flexibility under overload condition. Since the discharge capacity of a channel is proportional to the depth raised to the power of 5/3 according to Manning, the increase in depth in a rectangle for a doubling in discharge rate is only 50% and much less in a channel with flat lateral sloping banks. Thus a mistake in flow estimate is buffered. This is not the case with closed conduits where a doubling in flow could increase the head requirement by 300%, resulting in surcharging and overflowing or backing up at the inlets.

1.5 CONCLUSIONS

A well designed dual drainage scheme can effectively reduce the peak flowrates resulting in substantial savings in cost. The effects of reduced roughness and channalization on storm runoff peaks are worse than reducing impermeability in many urban situations. Flood peaks can be ameliorated in many cases by provision of a dual system with slow flowing plains taking the peaks. Roadways may act thus if designed for minimum incovenience. Open channel type overflow conduits are also more versatile than close conduits where flows exceed those anticipated.

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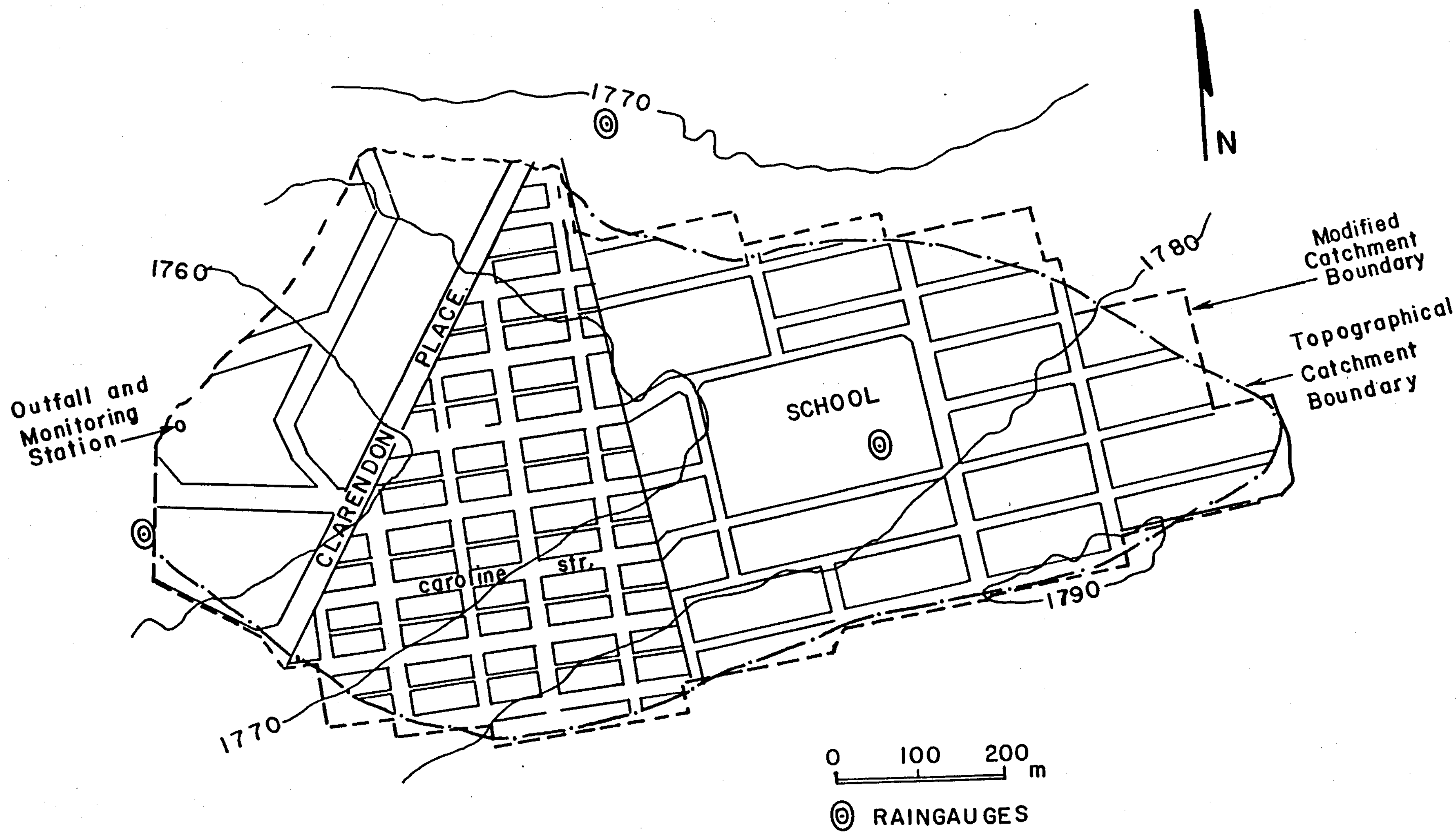
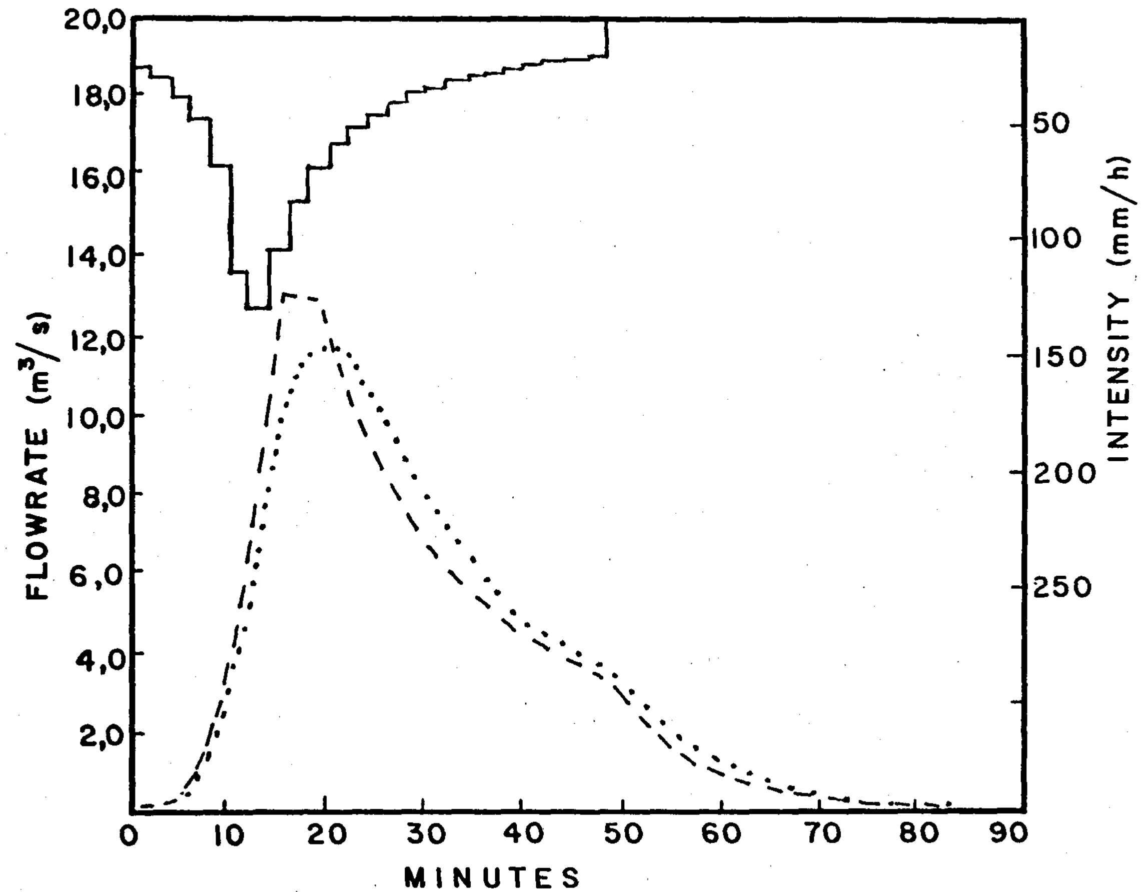
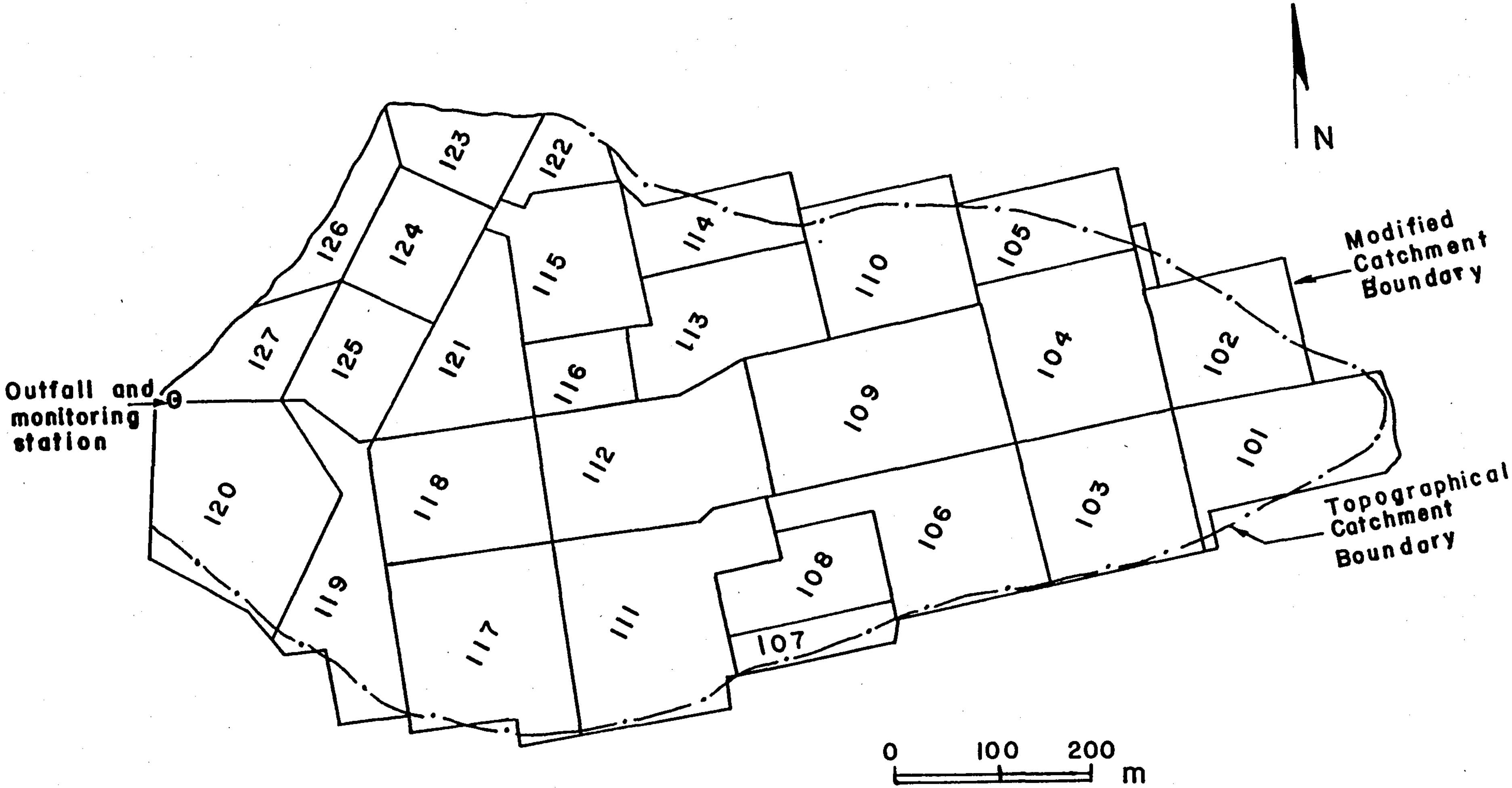


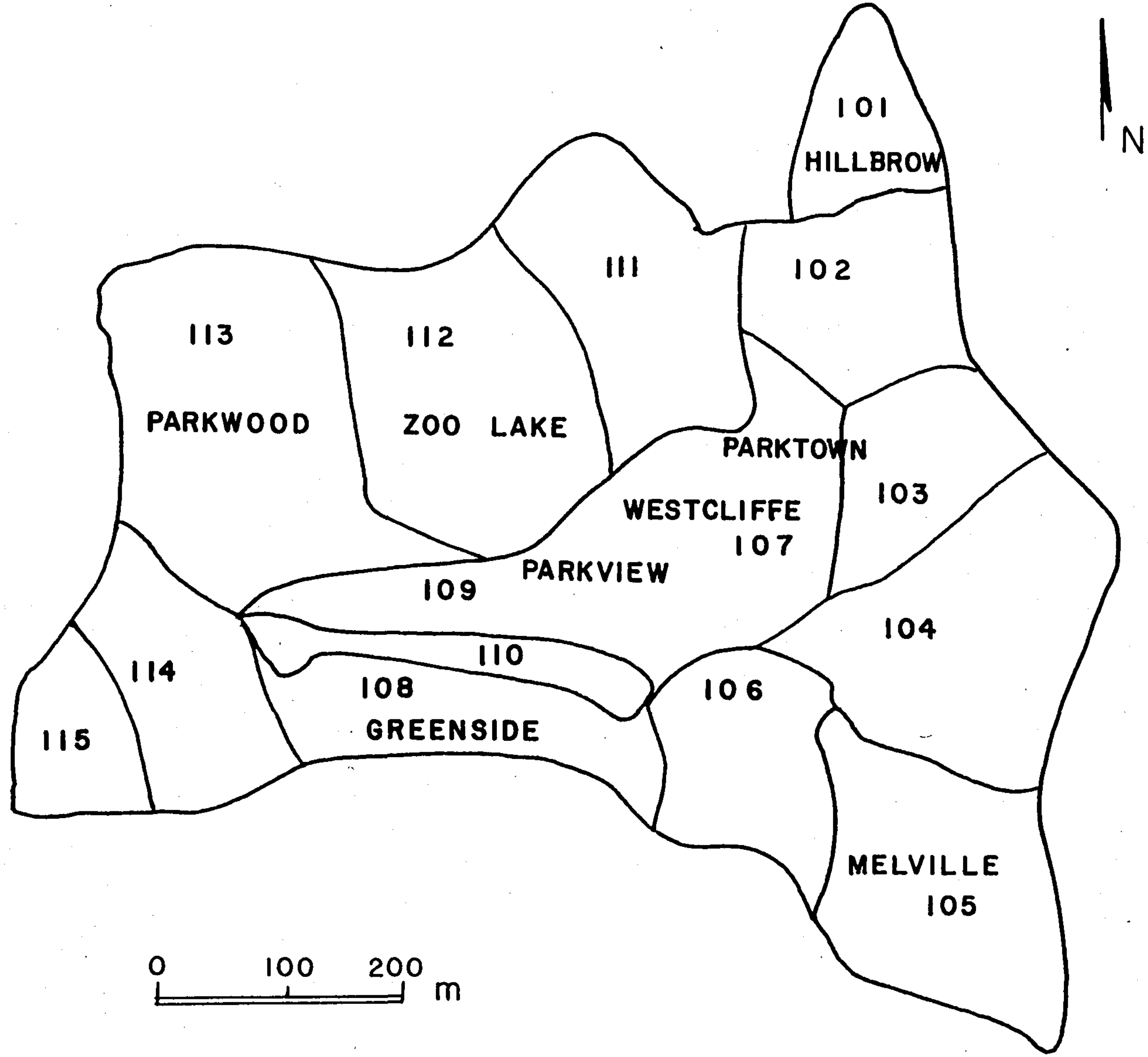
HILLBROW CATCHMENT



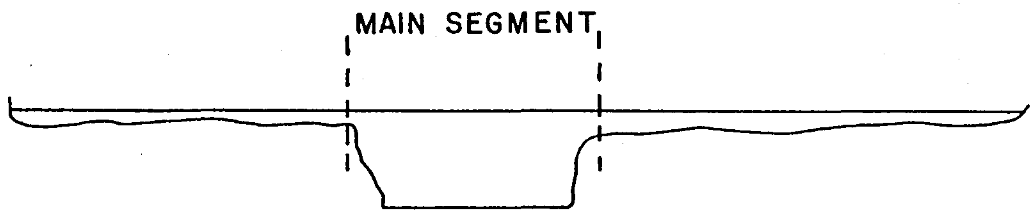
COMPARISON OF HYDROGRAPHS FROM
HILLBROW CATCHMENT
20 YEAR CHICAGO STORM



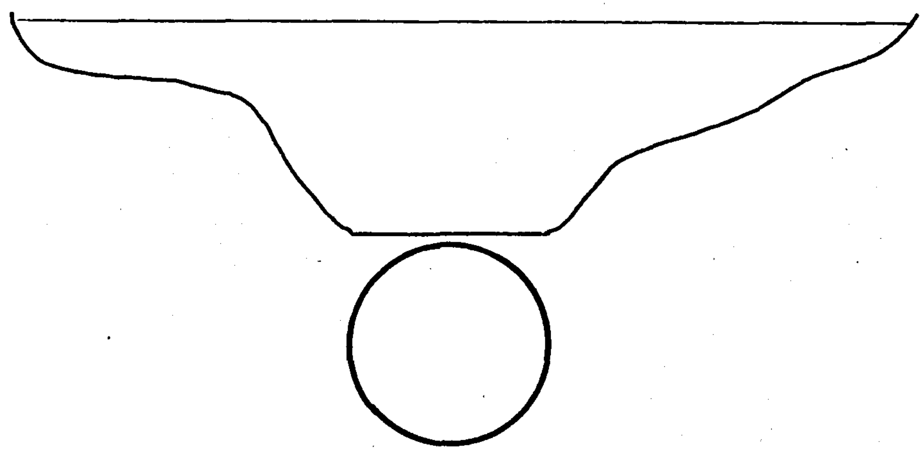
DISCRETIZATION OF THE HILLBROW CATCHMENT



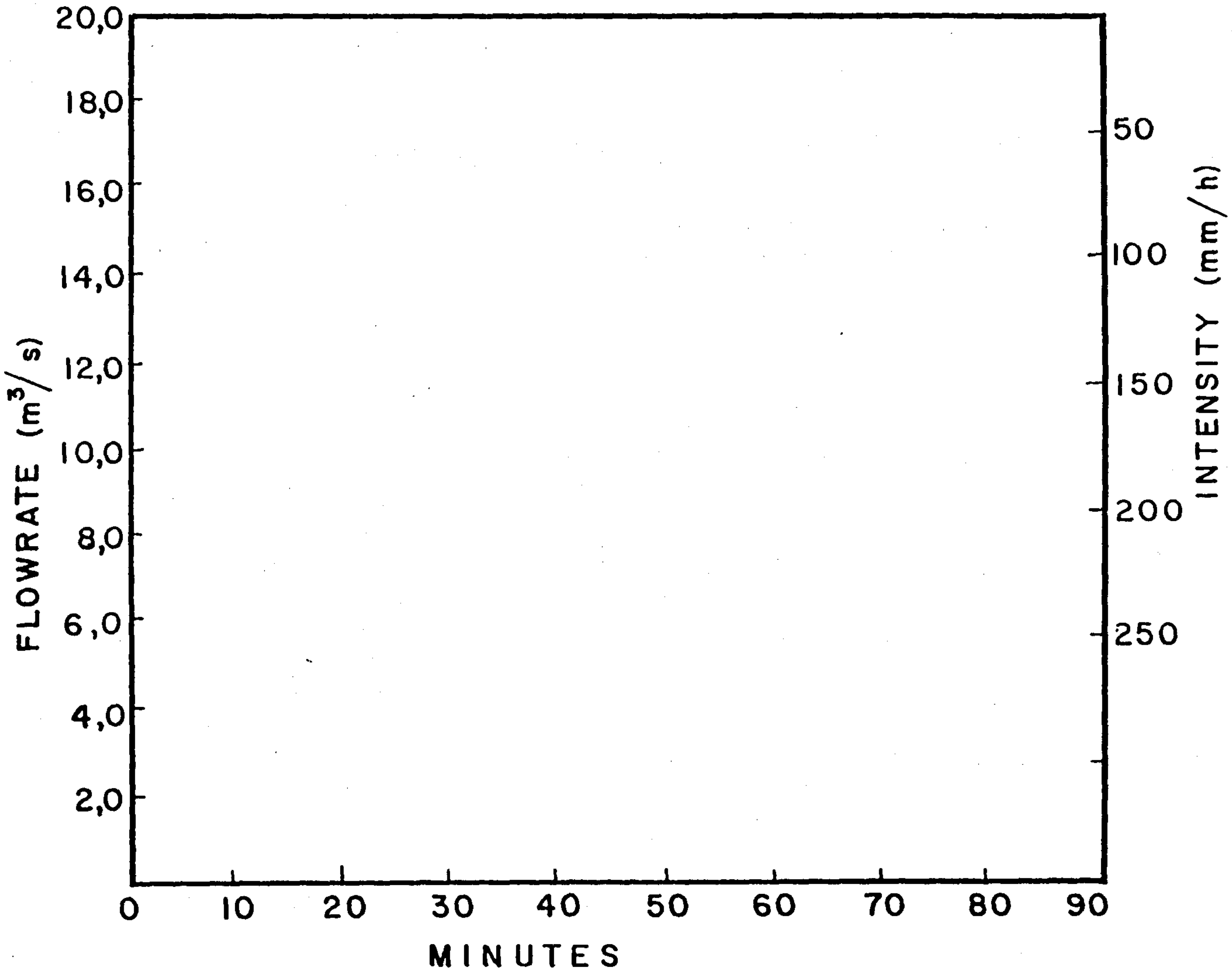
DISCRETIZATION OF BRAAMFONTEIN SPRUIT



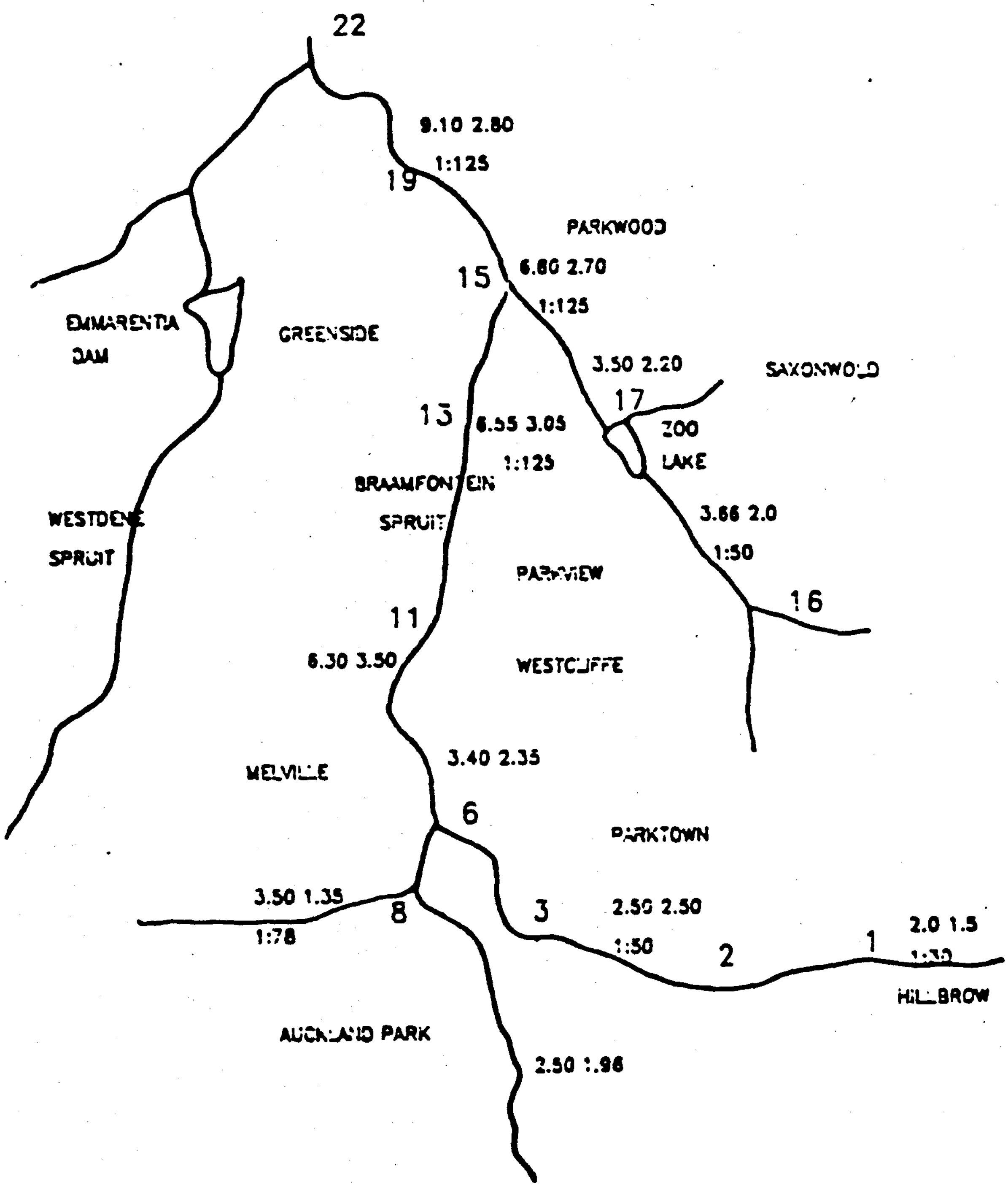
COMPOUND CHANNEL SECTION WITH
SHALLOW FLOOD-PLAIN FLOW



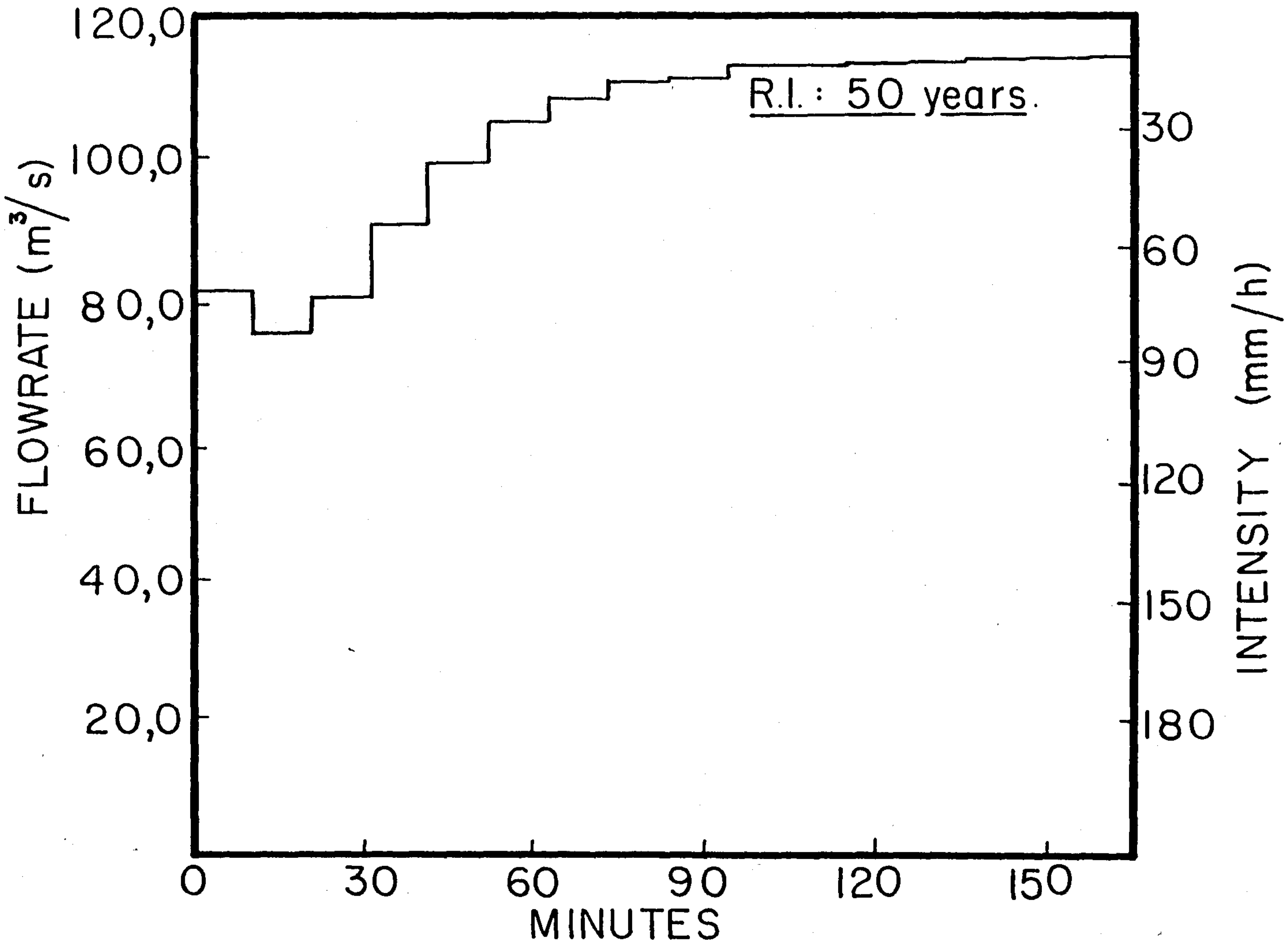
PIPE WITH CHANNEL ABOVE



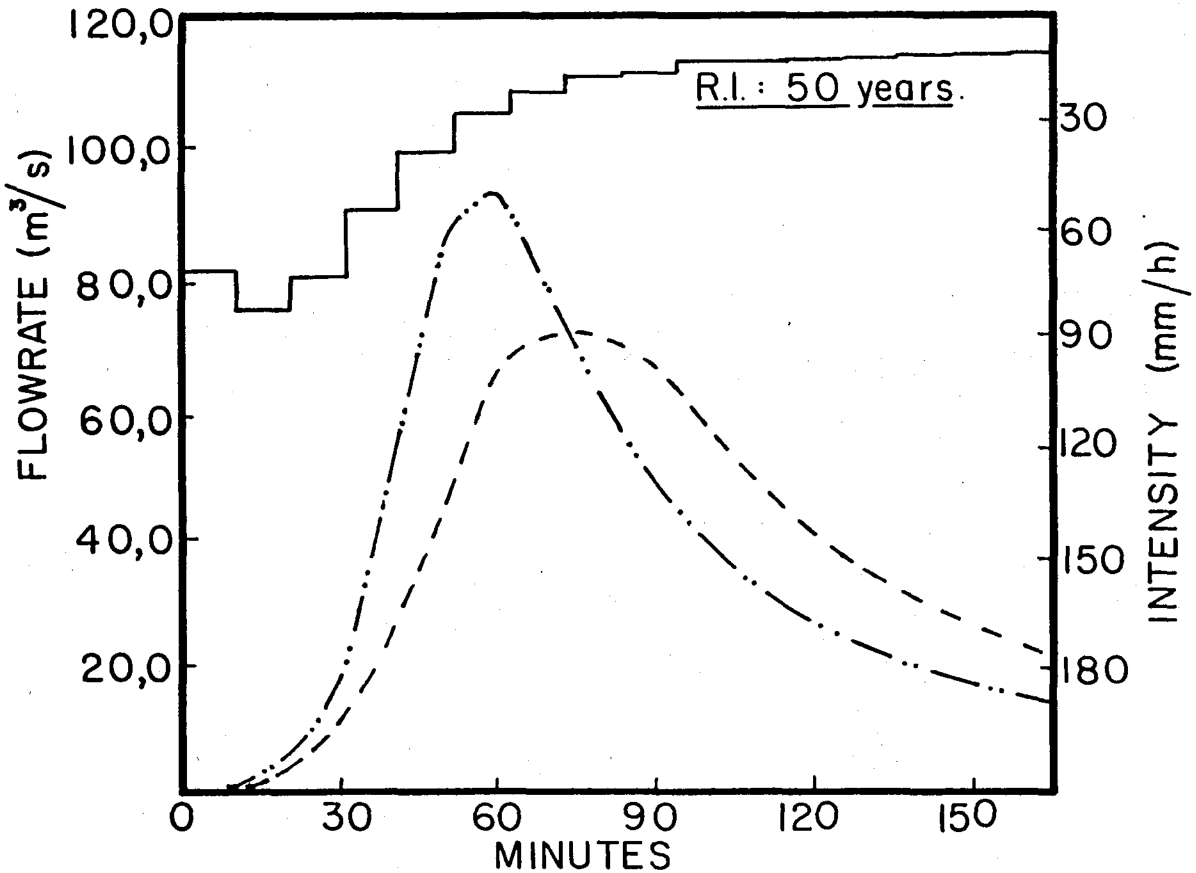
COMPARISON OF HYDROGRAPHS FROM
HILLBROW CATCHMENT
20 YEAR CHICAGO STORM



DRAINAGE SYSTEM OF BRAAMFONTEIN SPRUIT WATERSHED



COMPARISON OF HYDROGRAPHS FROM
BRAAMFONTEIN SPRUIT
50 YEAR HUFF DESIGN STORM



COMPARISON OF HYDROGRAPHS FROM
BRAAMFONTEIN SPRUIT
50 YEAR HUFF DESIGN STORM

A simplistic mass balance of storm-water pollutants for two urban catchments

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Abstract

Two catchments in Johannesburg, one a densely built-up city catchment and the other a suburban catchment were monitored under a storm-water research programme. Continuous flow and conductivity measurements were taken, and spot sampling for selected ions made during storms. Pollution loading is highest from the city catchment and in both cases storm runoff carries most of the pollution. Net atmospheric washout and fallout is of the same order as washoff of pollutants in the case of the suburban catchment. Nitrates exhibit an increase.

Introduction

The Water Systems Research Group have, in the course of research on urban hydrology and drainage, investigated pollution loadings from two catchments in Johannesburg (Green *et al.*, 1986). One, Montgomery Park, is a suburban catchment and the other, Hillbrow, a densely built-up city area. It is reported that non-point source pollution is responsible for 70% of the load in urban runoff (Wanielista, 1979), and it is largely this type of contribution which is detected here. Bradford (1977) attempts to relate pollutant loads to land use, and this paper contributes to his hypothesis. The unpredictability of runoff quality indicated by Simpson and Kemp (1982) is borne out though.

Catchment description

The Montgomery Park catchment is situated 6 km north-west of Johannesburg and measures 10.53 km^2 (1 053 ha). The population is estimated at 15 000. The developed area is 75% of the total and the remainder includes parks, a cemetery and undeveloped land. The development comprises housing and some commercial and light industry. There is a solid waste tip in the catchment from which seepage occurs. The catchment is fairly hilly, with slopes ranging from 0.02 m/m to 0.15 m/m. The main drainage system comprises natural and artificial channels. Rainfall over the catchment is recorded at five locations by autographic rain gauges. Runoff is measured at a gauging station at the catchment outlet in which the measuring element is a Crump weir with a bubble type recorder. Electrical conductivity of the water has been recorded continously since March 1983 (Green *et al.*, 1986).

The Hillbrow catchment measures 67.2 ha and is a fully developed urban area comprising high-rise buildings, some high density housing and a school. The population is estimated at 12 000. There are four rain gauges and a stream gauge in this catchment.

Both catchments have separate storm-water drainage systems (i.e. separate from waste sewerage systems).

Chemical constituents and suspended solids

Spot samples of storm-water runoff were collected during two storm events in Hillbrow and during one in Montgomery Park. In addition several dry weather flow samples were collected in both catchments. The number of samples taken and sampling intervals are listed in Tables 1 to 5.

These samples were analysed to quantify the presence of nitrates, sulphates, chlorides and bicarbonates as it was considered that these were the major anions present in the water. The analyses were undertaken by an independent commercial firm using titration techniques.

The highest anion concentration was found to be bicarbonate, followed by sulphates during storm runoff, and chloride in dry weather conditions. Sulphates appear to be predominant in Johannesburg and could be wind-blown from neighbouring mine waste tips which have high sulphur concentrations which oxidise to sulphates on the surfaces of the waste tips. Sulphates also reach concentrations of over 300 mg/ℓ in water supplies for the area.

The proportion of nitrates, sulphates, chlorides and bicarbonates to the total dissolved solids is much lower in storm runoff than in the dry weather flow analysed. In the latter case 68,6% of the TDS consists of these anions whereas this proportion is as low as 38,8% in the storm runoff (averaged over both catchments) indicating probable washoff of other constituents which do not appear in the dry weather flow.

As one would expect, the concentration of suspended solids in dry weather flow is much lower than in the storm runoff, indicating a higher transport rate of sediments as well as possible erosion during storms. For the samples analysed, the suspended solids in the dry weather flow averaged only 27 mg/ℓ compared with an average of 236 mg/ℓ for the storm flows.

Comparison of Tables 4 and 5 (dry weather flows) with Tables 1, 2 and 3 reveals that the TDS concentrations are considerably higher in dry weather flows than in storm flows. The average TDS for the dry weather flow samples is 772 mg/ℓ for Hillbrow and 612 mg/ℓ for Montgomery Park while average values of TDS for the three storm flows monitored are 125 mg/ℓ, 113 mg/ℓ and 117 mg/ℓ, indicating that the dry weather flow has about five times as high a concentration of TDS as storm-water runoff. The base load of TDS from Montgomery Park appears to be largely from leachate entering a storm-water culvert passing under the refuse tip. This load is of the order of 160 000 kg/a or 150 kg/ha.a averaged over the catchment (Ball, 1984). The reason for the high base load of TDS off the Hillbrow catchment is not entirely clear. It is thought that illegal discharges of effluent into the drainage system may be partly responsible for this.

The proportions of nitrates are also much higher in the dry weather flow than in the storm-water runoff. In the case of the dry weather flow sampled in the winter months of 1982 (Table 4), the levels of nitrate are so high as to suggest possible blockage

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TABLE 1
RESULTS OF CHEMICAL ANALYSES ON RAINFALL AND RUNOFF SAMPLES FOR HILLBROW ON 03/01/85

Sample mark	Time taken	pH	Conductivity mS/m	TDS mg/ℓ	Suspended solids mg/ℓ	Nitrate mg/ℓ	Sulphate mg/ℓ	Chloride mg/ℓ	Bicarbonate mg/ℓ
H1	20h11	6.20	14.37	1 38	1 010	0.2	12	5.1	36
H2	20h14	6.30	13.12	112	242	0.5	10	4.1	34
H3	20h18	6.05	11.87	134	760	0.8	10	3.0	36
H4	20h23	6.00	9.69	102	770	4.1	10	3.0	24
H5	20h26	5.55	9.91	100	512	8.6	11	5.1	10
H6	20h31	5.85	10.88	126	232	5.3	13	5.1	24
H7	20h50	5.45	13.37	120	180	15.0	16	7.1	10
H8	21h01	5.90	15.43	170	102	12.9	21	8.2	27
R*	N/A	5.55	6.69	78	63	2.7	4	3.8	6

* Rainfall sample collected over duration of storm

TABLE 2
RESULTS OF CHEMICAL ANALYSES ON RAINFALL AND RUNOFF SAMPLES FOR HILLBROW ON 18/01/85

Sample mark	Time taken	pH	Conductivity mS/m	TDS mg/ℓ	Suspended solids mg/ℓ	Nitrate mg/ℓ	Sulphate mg/ℓ	Chloride mg/ℓ	Bicarbonate mg/ℓ
18/1	14h27	6.35	46.10	346	64	<0.1	64	37.0	122
18/2	14h32	6.35	35 50	265	84	<0.1	57	30.0	90
18/3	14h38	6.15	24,50	182	380	<0.1	23	12.3	85
18/4	14h44	6.35	13,40	104	130	<0.1	17	10.3	36
18/5	14h50	6.15	8.88	69	204	<0.1	13	8.2	20
18/6	14h52	5.85	8.23	65	44	0.3	14	7.1	14
18/7	14h54	6.20	6.29	55	92	0.1	14	6.1	12
18/8	14h57	5.60	7.03	65	56	0.2	12	10.2	7
18/9	15h00	5.65	6.14	48	8	0.2	11	4.1	10
18/10	15h04	5.70	5.95	60	<1	0.2	10	6.1	7
18/11	15h09	5.85	5.92	49	<1	0.2	11	6.1	7
18/12	15h16	5.70	6.58	50	2	0.3	10	5.1	7
R *	N/A	6.07	2.20	18	<1	0.1	4	8.2	6

* Rainfall sample collected over duration of storm (analysis for this rainfall sample suspect as TDS<sum of anion concentrations)

TABLE 3
RESULTS OF CHEMICAL ANALYSES ON RAINFALL AND RUNOFF SAMPLES FOR MONTGOMERY PARK ON 07/03/83

Sample mark	Time taken	pH	Conductivity mS/m	TDS mg/ℓ	Suspended solids mg/ℓ	Nitrate mg/ℓ	Sulphate mg/ℓ	Chloride mg/ℓ	Ca Carbonate mg/ℓ
SD1/1	***	6.25	13.26	104	95	<0.1	10	6.3	30
SD1/2		5.85	10.37	86	200	0.7	16	5.2	20
SD1/3		6.00	12.97	112	450	2.2	13	6.3	31
SA1/1		6.20	22.30	166	44	1.5	25	16.0	16
RF1/1		7.25	10.86	52	**	0.4	**	**	21

* Rainfall sample collected over duration of storm
** Insufficient sample collected for this analysis
*** Samples collected in order listed – no times taken

TABLE 4
RESULTS OF CHEMICAL ANALYSES ON DRY WEATHER FLOW SAMPLES FROM MONTGOMERY PARK

Sample mark	Date taken	pH	Conductivity mS/m	TDS mg/ℓ	Suspended solids mg/ℓ	Nitrate mg/ℓ	Sulphate mg/ℓ	Chloride mg/ℓ	Bicarbonate mg/ℓ
SAD1	Aug 82	6.35	77.80	708	10	210.0	100	34	15
SAD2	Oct 82	8.45	249.00	1 625	4	91.0	15	480	456
SCD1	Oct 82	8.45	51.60	374	12	10.0	16	45	175
SAD3	Oct 82	8.40	78.80	544	12	11.8	64	101	202
SBD1	Oct 82	8.25	66.30	446	10	30.4	42	86	183
SDD2	Mar 83	7.45	41.90	320	24	4.0	18	29	not done
SED2	Mar 83	5.45	37.70	262	14	0.7	25	10	not done
SAD5	Mar 83	7.85	88.90	620	18	2.8	40	120	not done

TABLE 5
RESULTS OF CHEMICAL ANALYSES ON DRY WEATHER FLOW SAMPLES FROM HILLBROW

Sample mark	Date taken	pH	Conductivity mS/m	TDS mg/ℓ	Suspended solids mg/ℓ	Nitrate mg/ℓ	Sulphate mg/ℓ	Chloride mg/ℓ	Bicarbonate mg/ℓ
HDWF17	Feb 86	6.55	87.10	580	80	<0.1	80	130	183
HDWF24	Feb 86	7.15	126.00	964	82	<0.1	177	160	507

of a sanitary sewer with the resulting overflow entering the stream. It was observed for all three runoff events that the nitrate concentrations increased over the duration of each hydrograph, reaching their maxima on the recession limbs of the respective hydrographs. A possible explanation for this phenomenon is that lightning activity will increase the nitrate levels in the rainfall during the course of the storm, resulting in increasing nitrate concentrations in the runoff with time. There were, however, large differences in magnitudes of nitrate concentrations between events, these concentrations ranging from as low as 0,2 mg/ℓ to 12,9 mg/ℓ. The latter value is considerably in excess of the recommended limit in domestic water, viz. 6,0 mg/ℓ with an upper limit of 10,0 mg/ℓ (SABS, 1984).

Samples of storm-water runoff were obtained on the rising limb of the hydrograph of 18 January 1985 in Hillbrow, making it possible to detect a flushing effect at the start of the runoff. The high TDS concentrations at the early stages of the runoff, viz. 346 mg/ℓ and 265 mg/ℓ, followed by a time-dependant decrease in TDS concentration to final levels of about 60 mg/ℓ indicate a "first flush" effect in accordance with the findings of many others (e.g. Cordery, 1977; Helsel *et al.*, 1979).

There does not appear to be any definite time-related decrease or increase in the levels of the other constituents in the runoff. For example, sulphate concentrations increased with time in the runoff from Hillbrow on 3 January while the converse is true for the runoff on 18 January 1985 from the same catchment.

Plots of pollutant concentrations with time for the Hillbrow events are presented in Figs. 1 and 2.

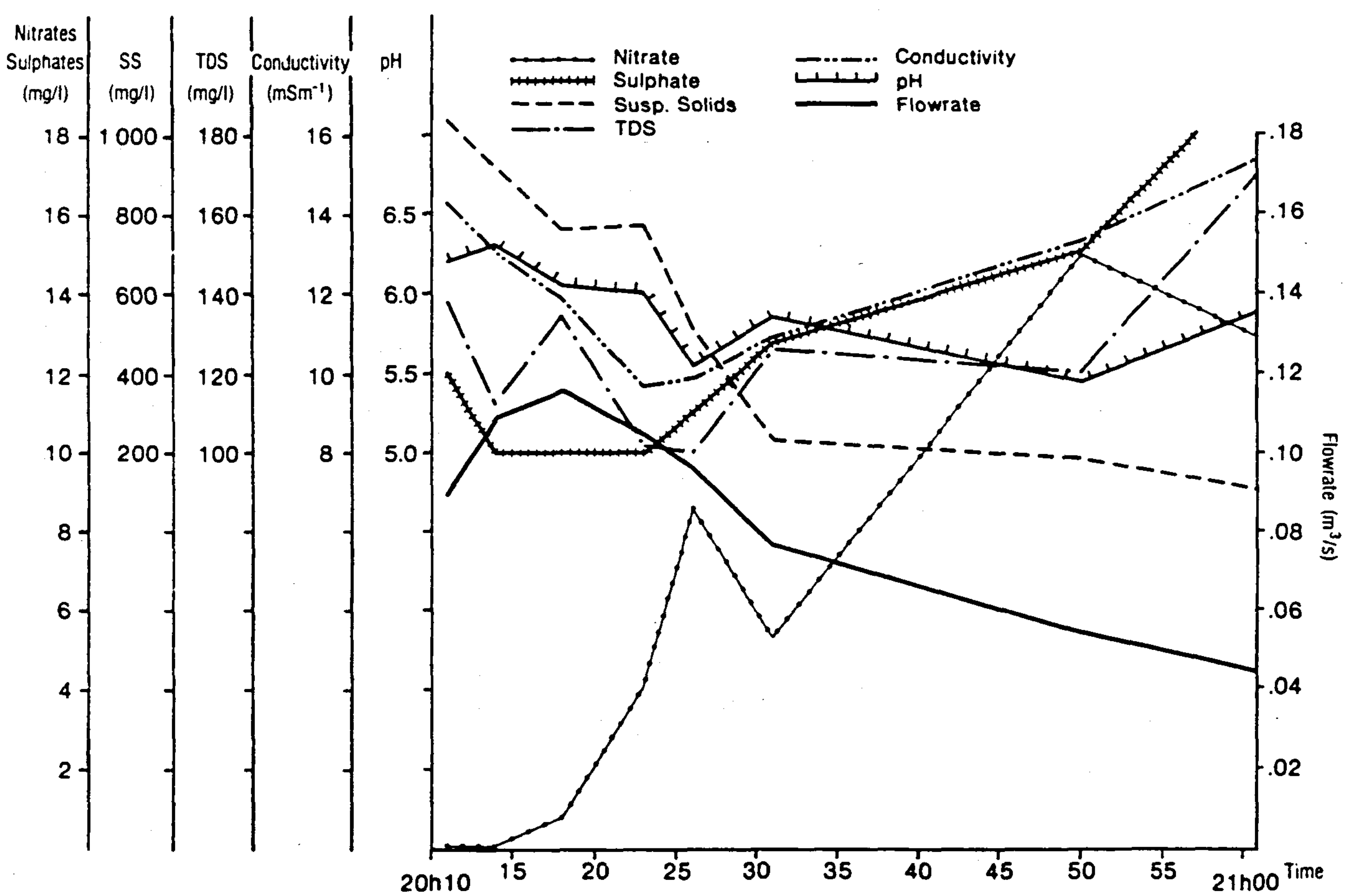

Figure 1
Plot of pollutant concentrations vs. time for rainfall-runoff event in Hillbrow on 03/01/85.

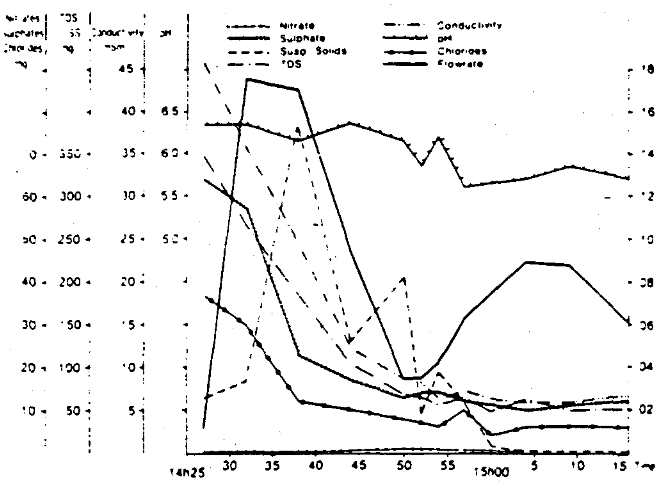

Figure 2
Plot of pollutant concentrations vs. time for rainfall-runoff event in Hillbrow on 18/01/85.

Relationship between total pollutant load and runoff volume

As the determination of TDS of a sample of water generally requires laboratory analysis, it has been found more convenient to use electrical conductivity for field measurement of salinity. By means of a regression analysis on data from 25 storm-water samples collected from Hillbrow, a relationship between electrical conductivity measurements and TDS was established. Continuous monitoring of electrical conductivity and flow rate thus enabled the computation of pollutant loads.

A regression analysis was performed on the pollutant load – flow volume data for the events listed in Table 6 to determine whether any definite relationship could be established between these parameters.

Considering data from Montgomery Park alone results in the equation

$$W = 3395{,}96 + 0{,}23V \qquad (1)$$

with a correlation coefficient of 0,84. W is the mass of transported dissolved solids in kg and V is the volume of runoff in m^3.

With the inclusion of the data from Hillbrow, the equation becomes

$$W = 1186{,}29 + 0{,}27V \qquad (2)$$

with a correlation coefficient of 0,90.

Treated separately, the relationship between pollution load and dry weather flow volumes is

$$W = 3{,}24 + 0{,}55V \qquad (3)$$

with a coefficient of correlation of 0,97.

Relationship between TDS concentrations and antecedent dry period

Certain researchers have observed a correlation between the number of dry days preceding a storm and the level of pollution of the resulting runoff (e.g. Sartor *et al.*, 1974; Colwill *et al.*, 1984) while others maintain that no such relationship exists (e.g. Whipple *et al.*, 1977; Bedient, 1980).

An attempt was made to see whether the peak concentration of TDS could be related to the number of antecedent dry days with a maximum of 5d. A regression analysis was performed on all the data available and the best fit resulted from a linear relationship having a correlation coefficient of 0,12.

In a further test, TDS was correlated with antecedent moisture condition. The four antecedent moisture condition classes proposed by Terstriep and Stall (1974) were used and the correlation coefficient in this case increased to 0,29.

It is therefore apparent that from the data available no relationship between TDS and antecedent moisture condition could be found.

Fallout measurement

An attempt was made to assess the level of TDS occurring as atmospheric fallout on the Montgomery Park catchment. After a period of 28d without any rainfall during the winter months of 1984, the rain gauges in the catchment were "washed down" with distilled water, this water being collected in a sample bottle. It was found that the TDS within the funnels averaged 9,5 mg. Since this was deposited onto a funnel area of 0,020 m^2 it was deduced that the equivalent fallout loading on the Montgomery Park catchment was 4,75 kg/ha over 28d. If washout were omitted this would represent 62 kg/ha.a. Atmospheric fallout was collected in a funnel with an area of 0,72 m^2 at a location near the Hillbrow catchment over a period of 18d with no rainfall during the summer of 1985. It was found that the TDS within the funnel in this case was 188 mg resulting in an atmospheric loading rate of 48 kg/ha.a. A longer monitoring period would, however, be required to confirm these figures.

Mass balance for event of 18/01/85 on Hillbrow catchment

A rainfall depth of 6 mm was measured for this event and the TDS concentration in the rainfall was 18 mg/ℓ (Table 2). This can also be expressed as a rainfall loading rate of 0,18 kg/ha.mm of rain or 1,08 kg/ha in total. For a catchment size of 67,2 ha this depth of rainfall corresponds to 4 030 m^3 of rainfall over the catchment with a mass of 73 kg of pollutants.

For this event a runoff volume of 475 m^3 and a total load of 121 kg of pollutant were estimated. There was thus a net washoff of 48 kg of pollutant from the catchment. Expressing the pollutant load in the runoff in terms of catchment area and rainfall gives 0,30 kg/ha.mm or 1,8 kg/ha total.

The sources of these pollutants have not been identified, but in a densely developed area like Hillbrow, the most likely sources are washoff of deposits from wind and motor vehicles and soluble fractions of litter which is usually present.

Since the runoff was only 12% of the rainfall and the catchment still experienced a net washoff of pollutants, with 66% more pollutant being washed off than was deposited by the rainfall, it is conceivable that this washoff may reach even higher percentages for events where the proportion of runoff to rainfall is greater. Such events would result from storms having a greater depth of higher intensity rainfall. It is also possible that input during one storm is stored and released after loss of moisture, to be washed off during a subsequent storm.

Mass balance for event of 07/03/83 on Montgomery Park catchment

On 07/03/83 a total depth of 14 mm of rainfall was recorded on the Montgomery Park catchment. This event was preceded by a time period exceeding five days of no rain, so it is not surprising that the TDS concentration of the rainfall is much higher than that measured in Hillbrow on 18/10/85 when only two dry days had passed. The measured TDS of the rainfall was 52 mg/ℓ (Table 3), resulting in a rainfall loading rate of 0,52 kg/ha.mm. The total mass of soluble pollutants deposited on this 10,53 km^2 catchment was thus 7 666 kg in 147 420 m^3 of rainfall.

A runoff volume of 5 508 m^3 with a corresponding cumulative runoff load of 1 479 kg of dissolved pollutant was measured. In terms of rainfall this pollutant load can be expressed as 0,10 kg/ha.mm. The runoff volume represents only 4% of the rainfall and the TDS washed off 19% of that deposited by the rainfall. In this case the catchment therefore experienced a net gain of 6 187 kg of pollutant, or 81% of that deposited. This corresponds to a net gain of 5,87 kg/ha or 0,42 kg/ha.mm of rain-borne pollutant i.e. net deposition of pollutant occurred in the peri-urban catchment while net washoff occurred in the densely developed catchment. Since there is a deposit (loss of matter) from rain as indicated by the Montgomery Park catchment it can be expected that a similar deposit would occur in Hillbrow, so the litter load must be higher.

Once again it is difficult to attempt to identify the sources of pollutants washed off this catchment. Referring to Tables 2 and 3 it will be seen that nitrate levels in the runoff are higher for this catchment than for the Hillbrow catchment of 18/01/85, signifying the possible washoff of decaying vegetation, animal faeces and garden fertilisers. This seems a reasonable deduction as the Montgomery Park catchment consists of predominantly suburban residential developments with gardens. Another source in Montgomery Park could be leachate from the ground (either previously deposited by rain seeping in or from soil minerals). It is noted that the proportion of sulphates in runoff is similar to those in the rain, but chlorides exhibit an increase.

It appears that sulphates in chlorides are unaffected by the two different land uses, the respective levels being of the same order for both catchments which also indicates they may be air-borne into the chatchment. It has also been observed that there are (illegal) discharges of industrial wastes into the separate storm-water system in Hillbrow.

Generalised mass balance of pollutants

In the mass balance of pollutants outlined above it was found possible in both the Hillbrow and the Montgomery Park catchments to relate the pollutant load in the runoff to the load in the rainfall causing that runoff. To determine whether the catchment has experienced a net loss or gain of pollutants it is also necessary to know the TDS concentration of the rainfall as well as the runoff. In the present project rainfall quality was analysed for only three events (Tables 1, 2 and 3), TDS levels in the rainfall being 18 mg/ℓ (Hillbrow), 52 mg/ℓ (Montgomery Park) and 78 mg/ℓ (Hillbrow). A TDS concentration of 118 mg/ℓ in rainfall was observed by Madisha (1983) at a location near the Hillbrow catchment.

Assuming a rainfall loading rate of 0,52 kg/ha.mm for Montgomery Park and an average rainfall loading rate of 0,71 kg/ha.mm for Hillbrow, albeit from a sparse data base, the total weight of dissolved solids deposited on the two catchments was computed for twelve rainfall-runoff events for which both discharge and electrical conductivity data were available. These results are presented in Table 6.

It can be deduced from Table 6 that the average pollution load of runoff expressed in terms of rainfall is 0,40 kg/ha.mm of rainfall for Montgomery Park and 1,54 kg/ha.mm of rainfall for Hillbrow. While these findigs are based on reasonably sparse data, they are nevertheless in accordance with the findings of other researchers (e.g. Polls and Lanyon, 1980; Mikalsen, 1984), viz. that in general the level of pollution of storm water is higher from commercial and downtown land-use developments than for residential developments.

TABLE 6
COMPARISON OF POLLUTION LOADS IN RAINFALL AND RUNOFF WITH RAINFALL DEPTHS

Location and date	Rain-fall depth (mm)	Weight of deposited TDS (kg)	Weight of TDS in runoff (kg)	Ratio of runoff load to rainfall load	Pollution load in runoff (kg/ha.mm)
Montgomery Park					
07/03/83	14	7666*	1479	0,19	0,10
09/12/83	13	7118	7356	1,03	0,54
12/12/83	17	9309	13086	1,41	0,73
21/01/85	46	25188	23451	0,93	0,48
30/10/85	55	30116	15872	0,53	0,27
31/10/85	24	13141	7688	0,59	0,30
01/11/85	67	36687	26391	0,72	0,37
Hillbrow					
13/08/84	1	48	247	5,15	3,68
16/09/84	14	669	217	0,32	0,23
20/10/84	2	96	81	0,84	0,60
21/10/84	1	48	193	4,02	2,87
18/01/85	6	73*	121	1,66	0,30

Average TDS for Montgomery Park = 52 mg/ℓ
Average TDS for Hillbrow = 71 mg/ℓ
* Denotes measured TDS in rainfall used

TABLE 7 SUMMARY OF DISSOLVED LOADS IN kg/ha.a					
Catch-ment	Atmos. fallout	Atmos. washout	Total washoff	Runoff net gain(+) or loss(−)	Storm washoff only
Montgomery Park	62	397	363	−96	305
Hillbrow	48	541	1520	+931	1190

Another interesting deduction from Table 6 is that more pollutant was deposited on the Montgomery Park catchment than was washed off for five out of the seven events while this was only the case for two out of five events in the Hillbrow catchment. The higher percentage imperviousness in the Hillbrow catchment is possibly the reason for this phenomenon.

Having established relationships between depth or rainfall and amount of pollutant washed off a catchment, annual pollutant loads can be computed.

Considering the Hillbrow catchment for example and assuming a mean annual precipitation of 763 mm (Adamson, 1981), the total mass of pollutants washed off this catchment will be of the order of 80 000 kg/a or 1 190 kg/ha.a. For the Montgomery Park catchment the amount of annual pollutant loading will be approximately 320 000 kg or 305 kg/ha.a.

Assuming an average dry weather flow of 0,0015 m^3/s or 130 m^3/d in Hillbrow and 310 dry days per annum results in an annual dry weather flow volume of approximately 40 300^3. This results in an annual dry weather pollutant load of approximately 22 100 kg or 330 kg/ha.a. The average dry weather flow in Montgomery Park is about 0,004m^3/s so the annual dry weather flow off this catchment is approximately 110 000 m^3 which corresponds to a total pollutant load of 60 500 kg or 57 kg/ha.a. Therefore it can be deduced that the annual pollutant load due to direct storm-water runoff is about 3,6 times that due to dry weather flow for the Hillbrow catchment and about 5,3 times that due to dry weather flow for the Montgomery Park catchment.

The pollutant loading rates derived from the different sources are summarised in Table 7.

Conclusions

Despite the limited monitoring and the resulting sparse data, the following tentative conclusions can be drawn:

The total dissolved pollution load in storm water and surface drainage from Hillbrow, a densely populated city area, is about 1 500 kg/ha.a which is about 3 times as great as from a suburban catchment, Montgomery Park. The majority (70% to 80%) occurs during storm runoff in both cases. Only about 430 kg/ha.a falls or is washed out of the atmosphere. The majority is therefore litter and from vehicles in the case of Hillbrow, and decaying vegetable matter or leachate from Montgomery Park.

There is a net gain of pollutants from Hillbrow but in Montgomery Park the total washoff is about the same order as the total deposited from the atmosphere. As a large proportion of rain seeps into the ground, it could store TDS to be released in future runoff. There is a net gain of nitrate in Montgomery Park, however.

Dry weather concentrations are higher in both catchments, due to seepage from a polluted landfill and the case of Montgomery Park (Ball, 1984) and illegal waste discharge in Hillbrow.

The majority of dissolved salts is washed off during the rising limb of the storms except nitrates which exhibit a lag. Release from the ground or alternatively the influence of atmospheric lightning could be the cause of this. Before prediction by modelling can be undertaken, intensive further investigation will be required.

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Spatial variation of rainfall intensities for short duration storms

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Abstract The limited areal extent and evolving nature of convective storms over a small catchment near Johannesburg were investigated and an analysis of 21 storm events over the 10.4 km^2 suburban catchment was carried out. Shapes of storms were modelled by a numerical surface fitting technique chosen after several alternative methods were compared for suitability. A model was developed, utilizing the chosen inverse squared distance technique, which enabled storm intensities to be simulated over the study catchment. The results of the study suggest that a spatial description of storm events at discrete time steps is a valuable approach for understanding the cellular composition of storm events and that the spatial variability of storms should be incorporated when studying the rainfall-runoff process.

Variation spatiale de l'intensité des pluies durant les orages de courte durée

Résumé La surface limitée de l'étendue et de l'évolution d'orages convectifs au dessus d'une petite région près de Johannesburg a été étudiée et l'analyse de 21 orages, sur une étendue de 10.4 km^2 de cette région péri-urbaine a été faite. La forme de la superficie couverte par les orages a été relevée par un système technique de surface numérique, après que plusieurs méthodes alternatives aient été comparées pour leur acceptabilité quant à l'étude. Un modèle a été développé en utilisant la technique du choix inverse du carré de la distance, qui permet la simulation de l'intensité des orages au dessus de la région d'étude. Les résultats de cette étude suggèrent qu'une description spatiale des événements d'un orage à différentes étapes, est une approche valable pour comprendre la composition cellulaire des événements d'un orage et que la variation spatiale d'orages devrait être pris au compte lors des études du processus pluies-debits.

INTRODUCTION

Rainfall data for estimating either runoff or infiltration are usually obtained from sparse raingauge networks. Frequently a single raingauge representing a large area is used to estimate point rainfall, and factors are applied to obtain

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spatially averaged rainfall. The use of autographic raingauges and data loggers enables the accurate temporal distribution of rainfall to be measured, but this is based on point measurements from the raingauges. In the past, the spatial distribution of rainfall has not been investigated as thoroughly as its temporal distribution when studying the rainfall-runoff process. A method for interpreting spatial variations in rainfall would therefore be instructive.

Alternative methods such as radar tracking methods exist for the analysis of storms, but in view of the fact that short duration storms are predominantly convective in nature and have high ground wind speeds associated with them, the distribution of precipitation clouds does not necessarily correspond to the distribution of rainfall reaching the ground. This aspect is exacerbated for small catchments.

A 10.4 km^2 semi-urban catchment near Johannesburg, South Africa, metered by five autographic raingauges with mechanical loggers, was used to study the spatial distribution of short duration storms.

From the literature and studies by the authors, it was discovered that storm events are composed of several precipitation-producing cells which appear and decay during the duration of the storm event both singly and with several cells together. The pattern of movement of the cells is erratic and their durations widely variable. For the convective type storms studied, the cells are typically of the order of a few square kilometres in areal coverage and have durations of between 10 and 40 min. Rainfall intensities vary from peak rates to insignificant rates over distances of less than 2 km, which is the nominal distance between gauges in the study catchment. Rainfall intensity patterns over the catchment were analysed in order to gain a better appreciation of the spatial distribution of rainfall. For future studies of this nature, however, it is recommended that gauge densities of one or more gauges per square kilometre be used because of the highly variable and erratic nature of the cells.

Previous research into the cellular structure of storm events has tended to be of a statistical nature (e.g. Berndtsson & Niemczynowicz, 1986; Shaw, 1983). Wishing to examine the spatial distribution of rainfall, a physically-based analysis using a numerical surface fitting technique (inverse squared distances) was therefore carried out, whereby contour maps of rainfall intensity were superimposed on the study catchment and thus a series of maps was produced for the duration of the storm event. The numerical method was chosen after tests were performed on several surface fitting techniques. A proposed extension is to distribute rainfall numerically over sub-catchments with rainfall-runoff simulation models and thereby account for both spatial and temporal variations in rainfall for runoff modelling. The cellular composition and rapidly changing nature of such cells make the spatial distribution of a storm and its changes in position and size important factors influencing runoff.

MATHEMATICAL SURFACE FITTING TECHNIQUES

Two major mathematical approaches have been commonly used for the investigation of storm characteristics: statistically based methods (e.g. cross-correlation) and surface fitting methods (e.g. multi-quadratic). The former tend to describe a physical system by means of representative parameters and focus on the dimensions and properties of cells (Berndtsson & Niemczynowicz, 1986; Shaw, 1983). The latter are generally applied to whole storms or families of storms and attempt to estimate values from the physical system (Shaw & Lynn, 1972; Adamson, 1978) and are then extrapolated to larger areas.

In order to investigate storm patterns over a small catchment, a surface fitting technique was employed to produce a visual picture. It is not a prerequisite to use a surface fitting method to produce contour maps from point depths or heights since methods such as triangulated irregular networks (TIN) and commercial software packages exist. However, a proposed application is the spatial (in addition to temporal) distribution of rainfall over sub-catchments within rainfall-runoff process models, in which case rainfall must be distributed independently of the location of the source data. Surface fitting techniques would enable data to be interpolated at any point within the catchment and the interpolated depths could subsequently be interpreted in terms of contours as a second step. Several surface fitting methods are available. These include:

(a) inverse squared distances;
(b) multi-quadratic;
(c) polynomial surfaces;
(d) distance-weighted least-squares; and
(e) kriging.

The last method, kriging, is more commonly applied to ore estimation and has successfully been applied to groundwater mapping (Gambolati & Volpi, 1979) but would probably only have application to short duration rainfall events if sensitivity to various models of variograms could be examined. The method has been applied to storm data (Heymann & Markham, 1982) but would more likely be used as a *result* of a study such as the one undertaken here, not for the study itself, and hence this method was not considered further.

Detailed explanations of the methods listed above are not warranted here, but are given by Ripley (1981) (inverse squared distances), Shaw & Lynn (1972) or Hardy (1971) (multi-quadratics), Cliff *et al.* (1975) (polynomial surface fitting), Mclain (1974) (distance-weighted least-squares), David (1977) and Heymann & Markham (1982) (kriging), and Patrick (1989), Heymann & Markham (1982) and Maaren & Brawn (1984) for a comparison of the methods.

Thus two steps were necessary for the production of contour maps: gridding, which interpolates rainfall values at chosen locations, and contouring which fits contours amongst the interpolated rainfall values. Objectives were defined for the testing of surface fitting methods of the gridding phase:

(a) the interpolated storm should have the same areal extent as the real storm;
(b) interpolated depths should equal known depths where they coincide;
(c) the method should not be unduly sensitive to missing data;
(d) average depth values for the study area should equal observed depths;

(e) the method should be efficient with respect to computer time and memory; and

(f) the results should be usable in catchment models.

It is recognized that the first objective is difficult to achieve as the areal extent of the storm event is usually unknown in detail, but this objective addresses the need for the interpolated rainfall surface to follow the rainfall depths on the ground, particularly at low or zero rainfall areas i.e. the area affected by a rainfall event must be closely modelled by the surface fitting technique.

The first four methods were compared by assessing how well they were able to interpret artificially generated data sets intended to resemble realistic storm depths and extreme conditions, and used 100 data points on a regular 10 by 10 grid. Artificial test data were necessary because no raingauge network of sufficient density was available to evaluate the "true" shape of rainfall patterns, and thereby form a comparison with the interpolated data points.

Between three and 80 data points were randomly selected from each test data set and used to generate a surface, which was compared to the "real" surface of the original data set. The process of selecting a number of random data points was repeated 20 times on each data set, for different random data points. Hence for data set 1, using 3 data points (simulating 3 raingauges) 20 different random combinations of 3 data points were selected and used as source data for the surface fitting method. Likewise for 4, 5, 6, 7, 8, 9, 10, 15, 20, 30, 40, 50, 60 and 80 data points. Finally all 100 data points were used as source data for the surface fitting method. This was repeated for each data set.

Comparisons of the interpolated surface were made with the "true" surface both visually (by producing contour maps) and by eight "accuracy of fit" test statistics (including sum of squared residuals, coefficient of variation, efficiency and correlation coefficient) and mean depth values. The numerous test statistics were used because of the absence of a single accuracy of fit test that behaves well under all conditions without reservation (Green & Stephenson, 1986a). Maximum use was made of the test data sets in this way (with 301 comparisons made per surface fitting method per data set), and it was discovered that only three artificial data sets were necessary to decide which surface fitting method could be used for the production of contour maps over the study catchment.

RESULTS OF COMPARISONS OF SURFACE FITTING TECHNIQUES

A summary of the comparisons of the surface fitting methods carried out by Patrick (1989) is given here. The polynomial surface fitting technique was rejected because of its inability to produce an accurate contour map of one of the test data sets, even though all 100 data points were used as source data for a sixth order polynomial surface (Fig. 1).

Of the four methods compared, the inverse squared distance was the most consistent, especially for minimal source data points. The other three methods produced large differences between minimum and maximum errors of fit (see Fig. 2 for the sum of squared residuals tests). This indicated that the inverse squared distance method would be more reliable and stable if used for

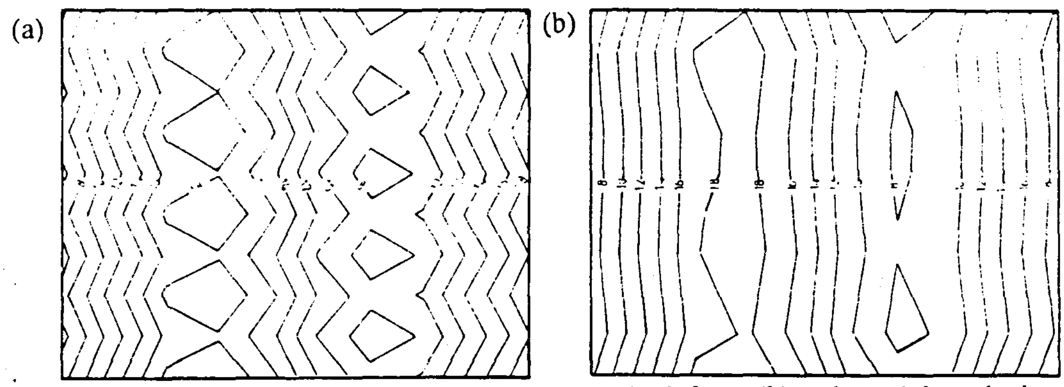

Fig. 1 (a) Contours fitted to original data; (b) polynomial method applied to data, all 100 data points used in interpolation.

the production of contours for the study catchment, and for the proposed numerical distribution of rainfall for rainfall-runoff process models. A series of duration tests was also carried out to determine how long each method took to interpolate a surface with varying numbers of source data points (Fig. 3).

The inverse squared distance method determines the influence of each known data point on the point of interest by using the inverse of the square

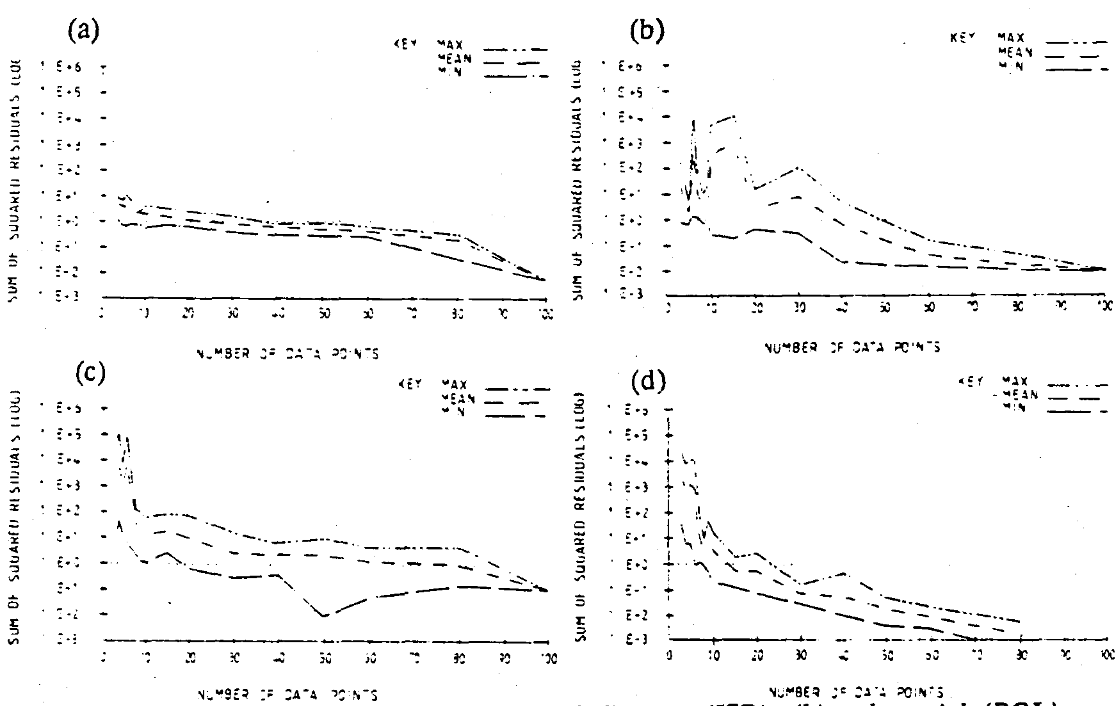

Fig. 2 (a) Inverse squared distance (ISD); (b) polynomial (POL); (c) multiquadratic (MQUAD); and (d) distance weighted (DIST) applied to data set 1.

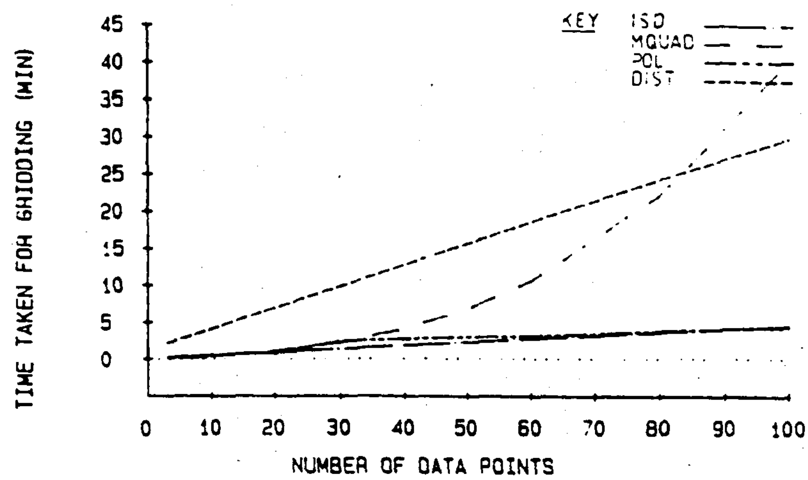

Fig. 3 Comparison of methods: time taken for gridding process in minutes.

of the distance between the point of interest and the known data point. The rainfall intensity values from known data points were weighted by dividing by the inverse of the square of their distance. The summation of these values was then divided by the sum of the inverse of the squared distances used in the weighting. This process is repeated for each desired interpolation point.

As a result of these findings the inverse squared distance method was chosen for the production of contour maps over the study catchment. The method adequately satisfied the goals outlined above for further use.

CONTOURING BY A ROVING SQUARE TECHNIQUE

An efficient contouring method was required to reduce the grid point depths to a contour map. A method using a simple roving three-dimensional surface employing equilateral triangles was developed for this purpose.

A general contouring routine based on a square with the four corners of known depths was used with the square being subdivided into four isosceles triangles, the intersection of the diagonals of the square giving a fifth averaged data point. The use of triangles provides an implicit solution to the position of contours of depth within the enclosing square. Because the four triangles within the square are equal sized isosceles triangles, the same algorithm can be repeated by merely taking the orientation of the triangles into account.

The roving square was applied sequentially to each area defined by four data points on the 10 x 10 gridded data surface (at this stage only the depth values were needed from the data to produce contour positions). The roving square now contoured was superimposed over the catchment and scaled according to the real coordinate system, taking account of which four data points were being contoured to position the contoured square. Edge matching was ensured since adjoining contoured squares used common depths and had a common boundary.

Two coordinate systems were therefore used in the contouring process: a local coordinate system peculiar to the roving square and a global coordinate system from the real catchment dimensions. Simple linear scaling was utilized to convert the completely general roving square to the particular catchment system.

APPLICATION TO A STUDY CATCHMENT

The catchment studied is in the north west of Johannesburg in the interior of South Africa (1500 m a.m.s.l.). The land use is semi-urban. The area of the catchment is 10.4 km^2 with a difference in elevation from highest to lowest point of 200 m. There are five raingauges serving the catchment, one of which lies immediately outside the catchment boundary. Two of the loggers are of the syphon type, the other three are tipping bucket types with resolutions of 0.2 mm per tip. The five gauges are logged by means of clockwork data loggers with charts that are changed or checked weekly. Thiessen weights were calculated for the five gauges (Fig. 4).

Data from the rainfall charts were digitized, converted into 5 min rainfall intensity values, and synchronized into one computerized data file taking account of real time values. Twenty-one storm events were extracted from two wet seasons, namely October to March of 1987 and of 1988. Only storm events with all five gauges operating were studied. Contour maps of each 5 min time interval were produced for the duration of the 21 storms studied

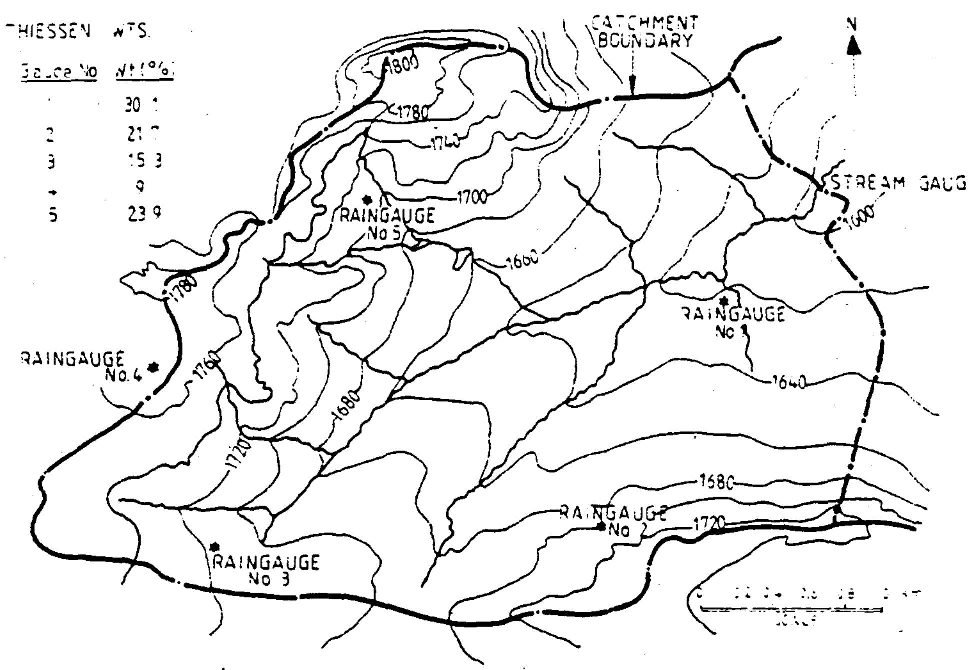

Fig. 4 Contour map of study catchment, Montgomery Park (after Green & Stephenson, 1986b).

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An example of the temporal variability of rainfall for each gauge is given in Fig. 5 which depicts hyetographs from a sample storm that occurred over the study catchment on 10 October 1987. All hyetographs are for the same storm event, and, as shown, the depth at each gauge does not vary greatly from the mean depth calculated by Thiessen weights. Several factors are noteworthy on Fig. 5: the starting times and ending times are different from gauge to gauge (this is not a result of error in the chart clock which was found to be accurate to within 5 min over a week); the general shape of the hyetographs is completely different, almost random (and yet the gauges are within 2 km of each other); the peak intensity values occur at different times (a peak at one gauge can coincide with a low at another); and examination of each hyetograph shows groupings of high rainfall and groupings of low

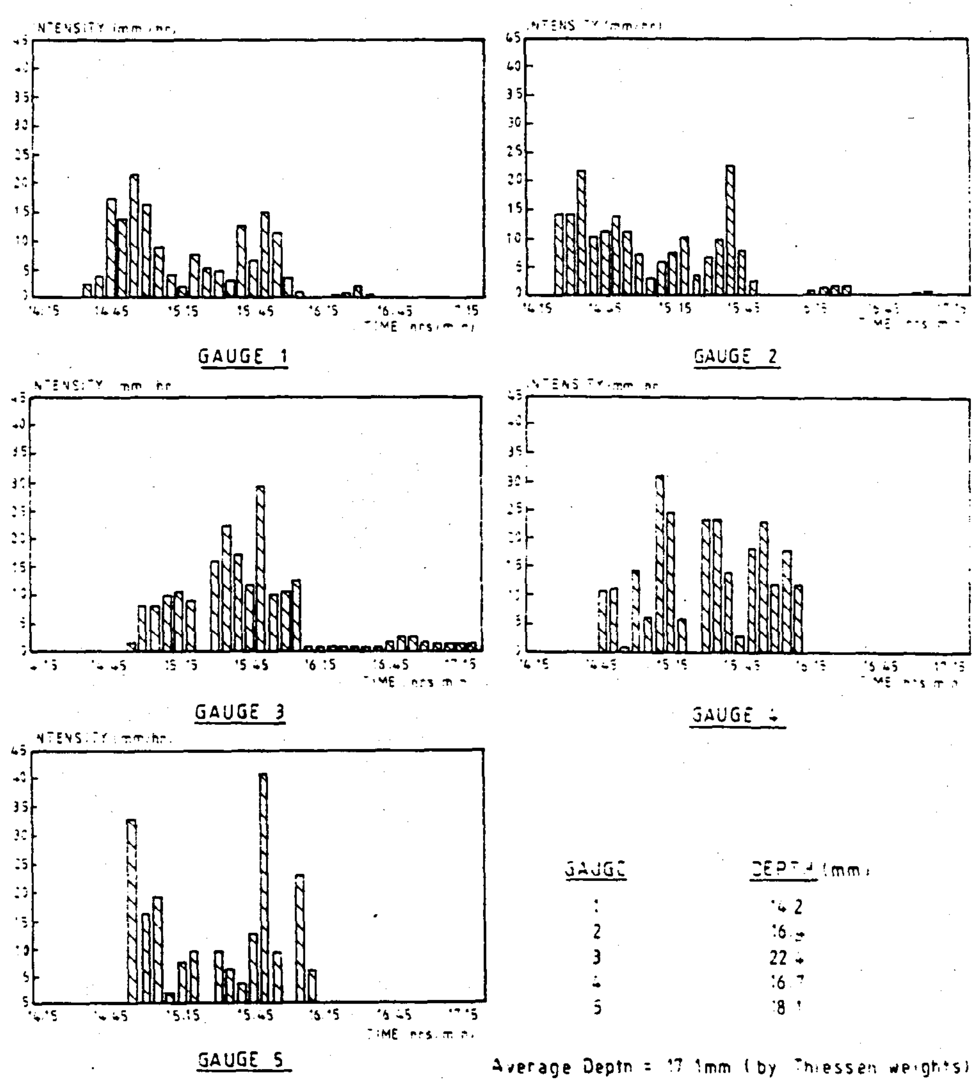

Fig. 5 Typical hyetographs for a storm event.

rainfall, most clearly seen at gauges 3 and 5.

The production of contour maps of rainfall intensities over the whole catchment gives a clearer understanding of the spatial variation of rainfall. It appears that the groupings of high rainfall can be attributed to cellular structure, and the groupings of very low rainfall can be attributed to the absence of cells or fringes of cellular structure within a storm event. Durations of such groups of rainfall are of the order of 10 to 40 min which are the durations typical of cells.

Examples of the spatial contour maps from a storm event on 3 February 1987 are given in Fig. 6. Each map represents successive contour pictures, with a contour interval of 10 mm h^{-1}. A cross-section through the storm event is also presented. Note how what appears to be cellular rainfall begins at gauge 1 and lasts for some 40 min, halfway through which another "cell" appears at gauge 4 and outlasts the first. Later on, a third "cell" arises over gauge 5, lasts for some 25 min and then decays. It is possible that one cell could have produced the patterns of medium rainfall (nominal 60 mm h^{-1}) simultaneously over gauges 4 and 1, but this would require an elliptically shaped cell with minor axis of less than 2 km and major axis greater than 4 km which remained at that exact location for some 15 min, which is considered unlikely in view of the reported erratic behaviour of cells.

It cannot be stated with certainty if the peaks or centres of any cells passed over the catchment, or if more than one cell passed over a gauge which gave the impression of being a single cell. A finer contour map resolution of 1 min may resolve this. A further example of this uncertainty is given in Fig. 7 which may result from overlapping cells, frontal type rainfall, or incorrect interpolation, the risk of which would be lessened with denser raingauge networks.

Spotty rainfall as described by Sharon (1972) was also evident in the early stages of Fig. 6 and is demonstrated by Fig. 8. Some general trends were also noted during this study. For example, it appeared there was a steadily decreasing number of cells corresponding to an increase in cell intensity. There were two plateaux in storm intensity noticed: one with a duration of about 22 min for intensities of 2.5 to 20 mm h^{-1} and a second with a duration of about 11 min for 20 mm h^{-1} and above. Cell dimensions were estimated to be about 3 km in diameter which is in keeping with work by others (Berndtsson & Niemczynowicz, 1986; Shaw, 1983). This was apparent at peak intensities and for a major part of a cell's duration, suggesting that there may be an upper limit to the areal size of a cell. The effect of spatial variation of storm events over a small urban catchment was studied by Patrick (1989) using a computer runoff model, and was found to be extreme.

The spatial variation of rainfall shows large variations of rainfall intensity within small distances over a small suburban catchment.

DISCUSSION

Aspects that are not immediately obvious have been revealed by this research. Traditional spatial distribution methods such as Thiessen polygons can result

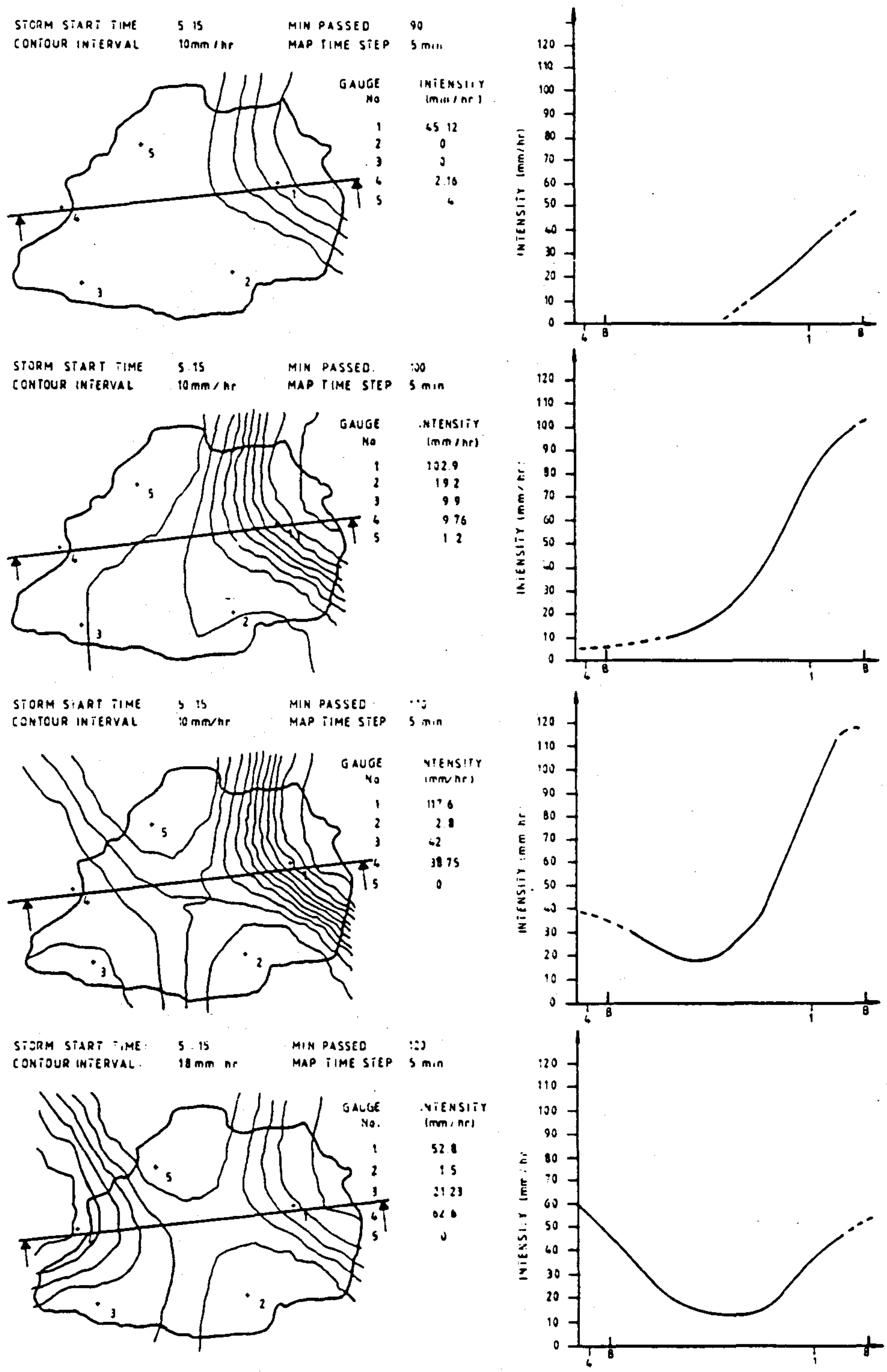

Fig. 6 Rainfall contour variations (continued opposite).

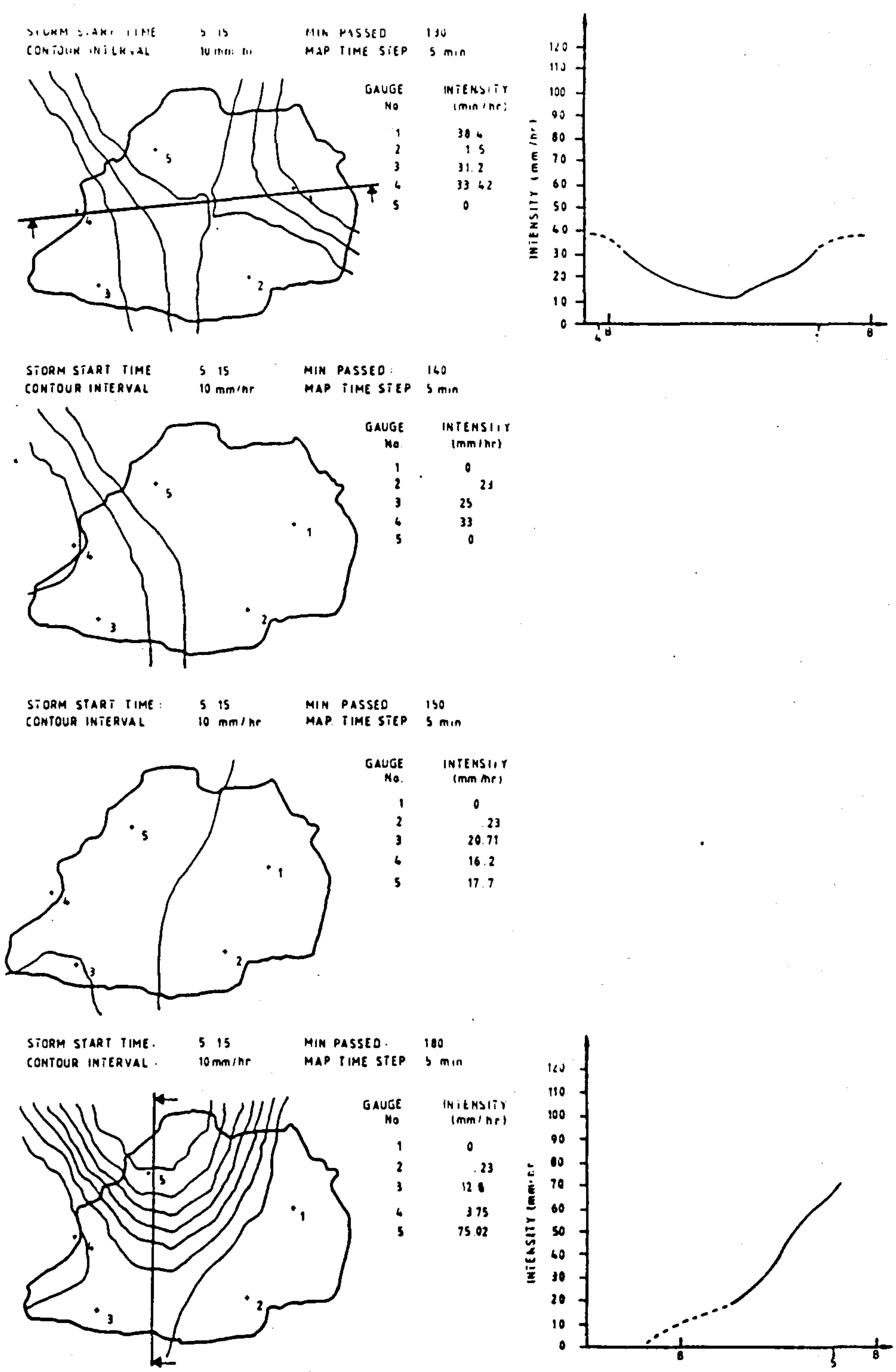

Fig. 6 Rainfall contour variations (continued).

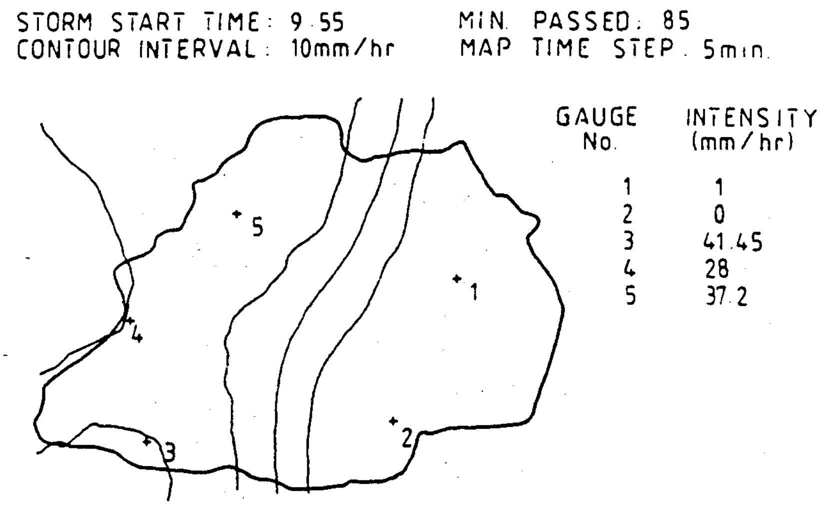

Fig. 7 The absence of a central raingauge makes analysis difficult in this case.

in one area of a catchment receiving a disproportionate amount of rainfall purely due to a high or low weight value, as may happen with the study catchment (Fig. 4). Location of raingauges for reasons other than hydrological (e.g. physical if access to an area is difficult) can therefore result in a biassed distribution of rainfall, a situation that may be reduced with a surface fitting technique. A rainfall event is mobile over a catchment and any modelling that ignores this must immediately introduce an error.

Process models which model the rainfall-runoff process are being produced in package format on micro-computers with increasing levels of discretization and accuracy with regard to the modelling of soil, slope and

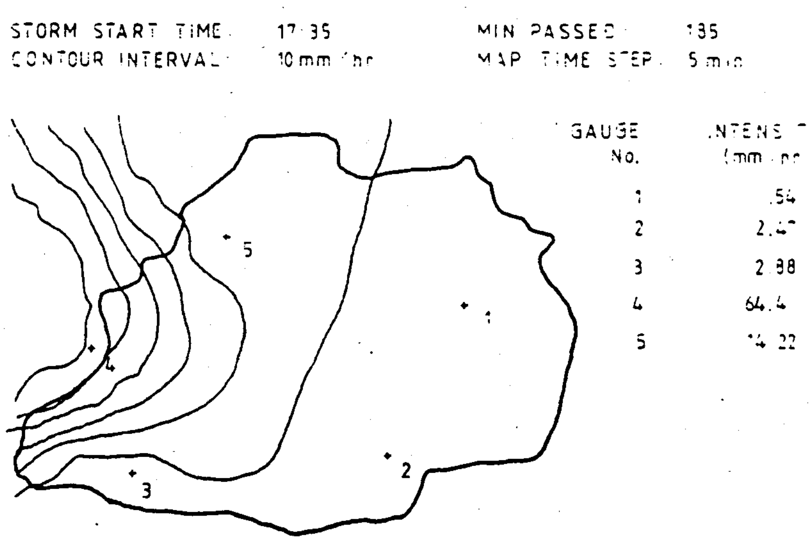

Fig. 8 Spotty rainfall for the storm of 27 November 1987.

vegetation parameters. To benefit from this, the description of the rainfall distribution over the catchment must be to the same accuracy as the other data. Rainfall is the main input for a cause and effect relationship, and if this is not taken into account to a sufficient level of accuracy, the benefits of discretization will be lost when the areas are lumped together under averaged rainfall inputs.

The number of raingauges in a catchment has a significant effect on the accuracy of rainfall interpretation. This is related to both the accuracy of the numerical technique employed for that number of gauges, and the distance between the gauges available for the study (gauges far from the interpolation point have small influence on the interpolation, and therefore minimal effect on the accuracy of the fit). An important aspect is the objective of the catchment study: whether it is for small urban studies or for water resources management of several drainage regions. An intrinsic assumption is that the raingauges themselves are accurate, and that their time synchronization is to acceptable levels. It is recommended that gauges should be maintained to within 5 min of each other for meaningful results.

The applicability of the numerical method used in this project to catchments of large area is unknown. However, understanding that storm events are erratic in behaviour and can be physically smaller in area than the study catchment indicates the numerical method's advantage in describing storm events more precisely than current popular techniques. It is therefore reasonable to expect an increase in accuracy if such a numerical technique is used for large catchment studies. Examination of possible links between cell characteristics and overall climatic and topographical features would be beneficial.

CONCLUSIONS

Considering catchments in their correct relation to storm event size, it would be useful to classify catchment studies using the same classification applied to storm events, i.e. micro-scale for less than about 6 km diameter, meso-scale for less than about 50 km diameter, and synoptic scale for larger catchment studies. This would aid in focussing attention on which aspects of the storm event are most influential to that scale of study (i.e. topographical, relief, areal extent, time scale of storm event, variations in intensity, etc.).

A single convective rainfall event is a composition of several component cells whose individual behaviour is erratic. The description of these cells in parameters useful to hydrologists is important for the accurate study and understanding of rainfall events.

The distribution of raingauges in a catchment is not necessarily the optimal arrangement for computer interpolation, and uneven representation of gauges can occur. There are seldom enough raingauges for a high level of accuracy when interpolating storm events which have extreme variations in intensities over small distances, in most cases significantly smaller than the distance between gauges.

The pattern of rainfall intensities from real rainfall events over their duration varies radically, both with time and space. Hyetographs from the gauges of a catchment for the same storm event can appear totally unrelated to each other, as was found in this study. This makes accurate calibration of catchment models difficult. The authors suggest that accuracy of catchment modelling may be improved with current computing technology such as that presented here.

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Development of a decision support mapping utility for water resources planning

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Abstract

The optimum utilisation of water resources is essential for our continued economic development and for the preservation of our limited resource base. At present the process of resource planning is expensive and mathematically complex. This is inappropriate for assessment and comparison of multiple scenarios. The use of microcomputers and simple graphical manipulation and display techniques can assist in rapid screening of proposed plans and will result in easier interpretation of output by the decision-makers. A versatile, yet cost-efficient mapping utility has been developed as par• of a decision support system to assist the decision-maker in reaching the most beneficial solution. The utility is designed for implementation on a standard IBM-compatible hardware configuration. A brief overview of the system, its capabilities and the potential applications are presented.

Introduction

Water is a non-renewable, natural resource of limited quantity, without which man cannot survive. There is no area of our daily lives in which water does not play an important, if not a vital, role. It is therefore essential that the decision-maker(s) in charge of water resource planning be assisted wherever possible by modern, scientific techniques, in order to reach a solution which will maximise benefits for the community.

Lee and Moore (1975) pointed out the problem facing the modern-day decision-maker:

> "It appears that in reality the decision-maker is one who attempts to achieve a set of multiple objectives to the fullest possible extent in an environment of conflicting interests, incomplete information, limited resources, and limited ability to analyse the complex environment."

The importance of water to man has resulted in the extensive development of water resource planning techniques over the past 3 or 4 decades and the use of mathematical methods is well documented (Stephenson, 1970; Martin, 1983; Baxter, 1985; Bulkley et al., 1985; Gollehon et al., 1985; Allen and Bridgeman, 1986).

It has been found that the implementation of these mathematical models in practical planning situations is extremely limited. It is our experience that the following are the main reasons for the limited use of these techniques.

- **Mathematical restrictions and complexity of application**
  The mathematical constraints imposed by many of these techniques lead to assumptions making the final model a poor reflection of the real study region. The complexity of the analysis procedure often results in only the creators of the method being able to use it.

- **Format of output and results**
  The data input and display of results are often inapplicable to the audience at which they are aimed. Interpretation will often not be possible or, more dangerously, will lead to incorrect conclusions being drawn from misinterpreted results.

- **Multidisciplinary input and conflict**
  The use of a multidisciplinary team is essential for correct water resource planning. This can, however, lead to communication problems and conflict.

- **Costs of planning**
  The mathematically complex techniques employed at present require the services of high level professional personnel for long periods of time. Often the model can then only be used for a short time before it is outdated and requires further professional man-hours to be updated. These resources are costly and are often not available. It is also common that the decisions must be made within a restricted timetable which cannot accommodate the usual long development period.

It is necessary that the decision-making professions be provided with a methodology which is able to present water resource planning in an easy-to-understand format and can be implemented in the shortest possible time. Part of this ongoing philosophy is the development of graphical systems for the manipulation, display and simple analysis of data and results.

Many commercial geographic information systems (GIS) exist, such as REGIS (Intergraph), ARC/INFO (Computer Vision) and SICAD (Siemens). These systems are able to manipulate, analyse and display spatial data, yet there is a dearth of systems that can easily be incorporated into 'user designed' analytical structures and are also cost-effective for small organisations.

Present CAD/GIS systems are either not versatile enough, when used for specific mapping tasks, or are cost-prohibitive. The latter constraint becomes more apparent when considering small research organisations or developing regions. In this paper we describe a simple and cost-effective tool that can be used as an effective method in spatial data manipulation and display. This has been termed a mapping utility or tool.

The mapping of distinct features (e.g. roads, rivers, and other cartographic items) has, prior to automation, been annotated on maps using manual techniques. Development of automated mapping procedures originated through the direct translation of the manual methods. The present trend, however, is towards original systems which will produce the required output directly. In order to extend the abilities of these mapping systems, programmers have added data-base systems capable of storing vast amounts of geographical information, hence the concept of a geographic information system or GIS was born.

The incorporation of the utility as part of a decision support

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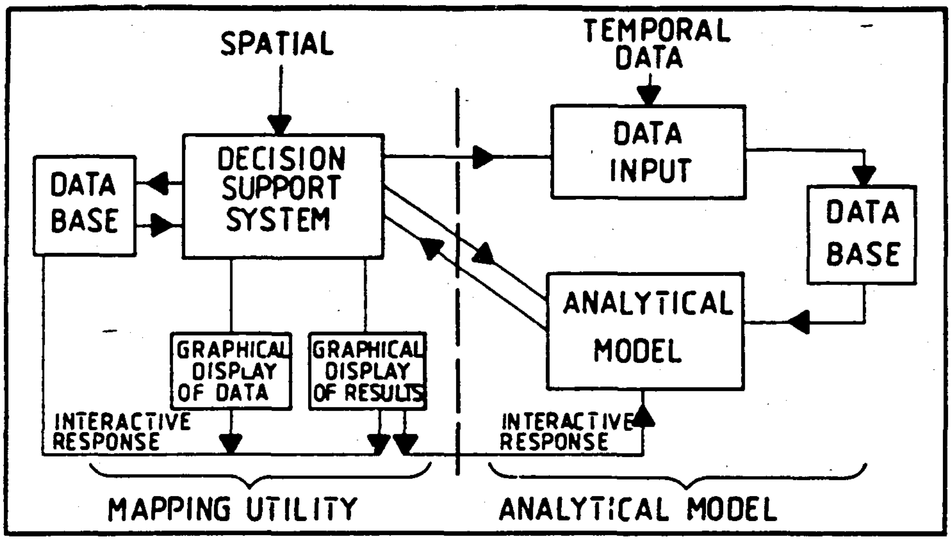

Figure 1
Schematic of mapping utility as decision support system

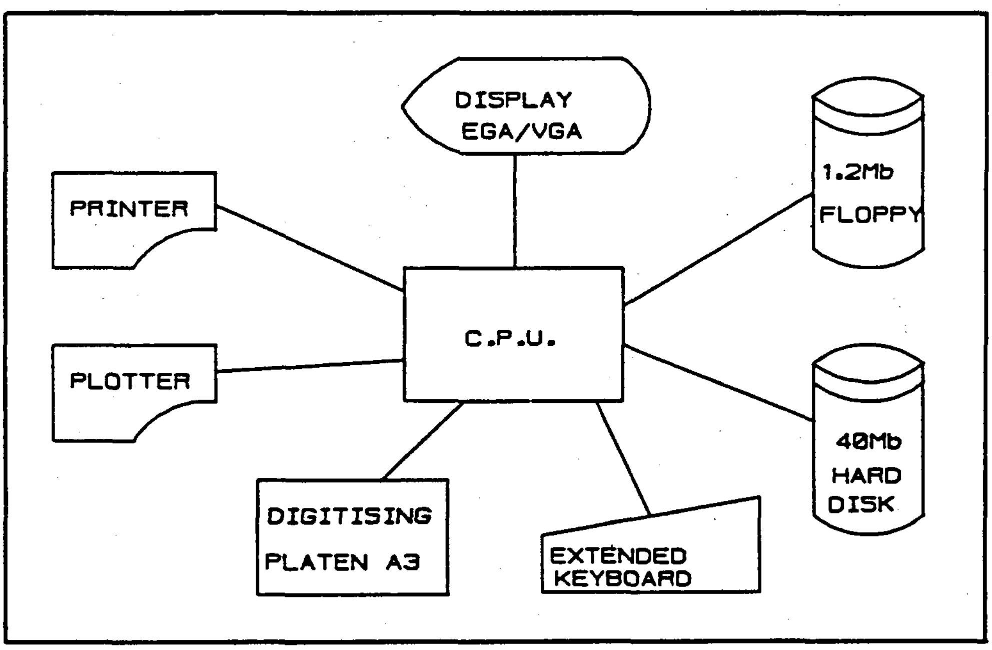

Figure 2
Minimum hardware configuration

system to aid the decision making process is shown in Fig. 1.

The mapping utility is not as complex as the commercial GIS, yet if required it is able to carry out many of the functions commonly required of a GIS. More importantly it can be customised to the specific needs of the planner. Once customised the utility can be integrated with the developed application software. The utility may also be customised as a stand-alone system with its own internal core driver. Data can therefore be manipulated either within the graphics system or externally before being displayed.

Hardware requirements

In the past the majority of the large, powerful computer-aided design (CAD) and geographic information systems (GIS), were confined to mainframe or minicomputer systems. This was largely due to the speed and memory requirements of these packages. With the vast improvements made over the last 5 years in microprocessor technology, it has recently become feasible to run these packages on standard microcomputers (although integrated

GIS systems still require a super-microcomputer to execute effectively).

With the current pace of technology, the definition of a conventional or 'common' configuration is very difficult; however, a minimum configuration can be suggested. Figure 2 illustrates the minimum configuration required for the graphics utility (excluding specific requirements for individual applications).

The system is designed to run on the IBM or compatible microcomputer using the Intel 80 x 86 processor family using the MS-DOS operating system.

Advantages in speed are gained with the inclusion of a maths coprocessor. The cost of the coprocessor is however significant in non-highly industrialised countries, therefore the mapper system has been designed to run with or without the addition of the maths coprocessor.

The enhanced graphics adaptor (EGA) graphics card was initially chosen as this is considered to be the basic level at which graphics can be effectively manipulated and provides an affordable entry level resolution for the small company or research institute. It has the advantage of being upwardly portable to the video graphics array (VGA) standard which increases the screen resolution.

Due to the high amount of data transfer and the large storage of data which is inherent in any CAD/mapping system, we recommend that a hard disk system of 20 MB or larger be used. The addition of a file caching system to increase speed of the hard disk I/O is also recommended.

In order to obtain a hard copy of the map displayed on the screen at any time, provision has been made for both a printer and a plotter driver. The standards chosen for these options were the Epson graphics printer (ESC/P command set, dot-matrix) and the HPGL plotter format, which have become universally accepted standards.

Methods

It is important to consider the nature of the data as this will affect both the method of data capture and the storage of coordinates.. Beran (1982) suggested that geophysical data could be classified into various categories. In this paper we consider 3 classes of data:

- Continuously varying data which are usually expressed as isolines (e.g. altitude and rainfall).
- Continuous feature information (e.g. river channnels, roads and railways).
- Constant value information which is expressed on maps as a patchwork (e.g. forests, vegetation cover and soil types).

The scale of the map originally digitised will have a pronounced effect on the level of sensitivity of localised anomalies. This is apparent when maps of different scales are digitised into the same filing system. The digitised information is, of course, only as good as the source maps. Brown and Fuller (1985) found that source maps compiled by different countries, had different levels of detail especially with reference to river channels. They used two different map scales to overcome local features and thus produced two different map data bases.

The mapper utility described in this paper has the ability to establish separate data bases for each of 5 default scales. These scales weie chosen to cover the full spectrum of possibilities and for ease of map availaility.

Each scale has a unique size which is the most convenient for digitising on an A3 digitiser. Table 1 shows the scales used and the corresponding size of the map area in degrees, which can comfortably be placed on a standard A3 digitising platen.

These unique sizes lead directly to the reference methodology for the mapping system. Each digitised segment is referenced by the latitude and longitude of the bottom left-hand corner. Any area can then be called up by giving latitude and longitude of the bottom left and top right corners of the required square. Similarly, for specific basins these values are stored in the basin reference file. Figure 3 illustrates the ability to combine any number of these map segments on the screen.

Once a series of segments are loaded and displayed on the screen, a facility exists whereby 'zooming out' will result in further segments being added to the screen. The increase in area is directly related to the scale being used and moves by one segment increment in a north and east direction.

Vectors

The vector data storage format was chosen as the primary method of data storage, as it is both time and space efficient. In order to represent a line between any two points on a two-dimensional system, the minimum requirement is an X and Y coordinate at both ends, that is four points. To facilitate data retrieval and regeneration of the vector, the number of vertices are also recorded.

When the operator has reached the end of the vector he wishes to store, he selects either the 'open' or 'close' option from the data entry menu. The 'open' option accepts the vector as is, whereas the 'close' option will include the start coordinates as the new end point and complete the vector to form a closed traverse. At this point, regardless of the option chosen, the number of points, followed by the X and Y coordinates at each point, are written to the layer file.

Layer files are created in order to allow the user control over the display of any combination of data on the screen at any one time. There is no limit to the number of layers allowed as all the data are accessed from disk and are not stored in memory. A number of layers of different names can be defined for each basin/map data base at set-up time; the default is 10. Each layer has a default colour on entry which can also be changed for each project. It is possible to use any colour for a vector if the default colour is found to be unacceptable for that layer.

The ability to 'flood-fill' a defined area has been included. A standard vector as described above, with the 'close' option chosen, is utilised as the boundary of the area to be filled. A fill colour is chosen and a point within the relevant closed vector is specified. The 'flood-fill' command is an intrinsic function in the programming language. From a specified point it will fill the area within the closed vector in the desired fill colour.

A further feature is a labelling system which allows the specification and positioning of a label at any point on the map.

All of the above information can be stored on a single layer file or separated according to the operator's final display or analysis re-

TABLE 1
AVAILABLE SCALES AND MAP SEGMENT INCREMENT

File code	Scale	Area	Increment
A	1:1 000 000	2° x 2°	2°
B	1: 500 000	1° x 1°	1°
C	1: 250 000	0,5° x 0,5°	0,5°
D	1: 50 000	0,1° x 0,1°	0,1°
-	1: 10 000	0,02° x 0,02°	0,02°

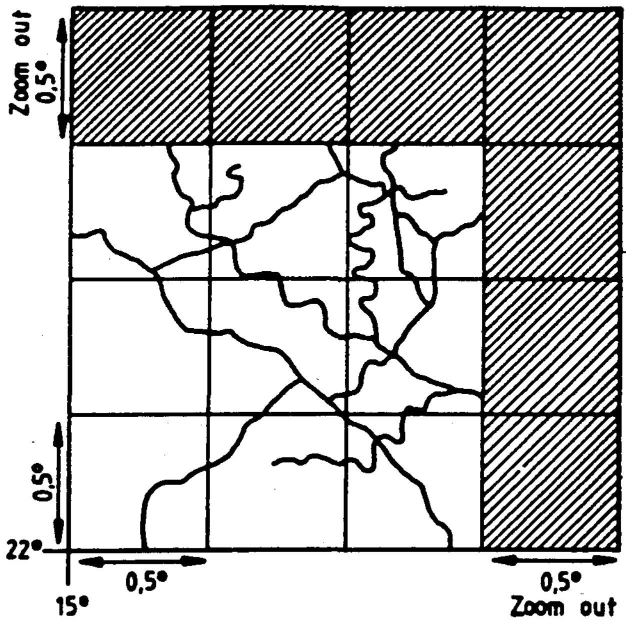

Figure 3
Map segment layout for a scale of 1:250 000

quirements. The facility to erase any of the above features is included.

In order to provide a more detailed picture of the area being studied a window system has been included. If the screen map resolution is not acceptable to the operator he may request a window. The screen is then quartered and the operator may choose the quarter in which he would like to work. The program will then window in (zoom in) on this quarter, allowing it to fill the screen. The program will redraw the entire map, vector by vector. The user, however, only sees the information displayed in the window. This method is slow during regeneration but allows the system to store the data without having to determine the change points at all the window boundaries. Windowing in this manner can be carried out on two consecutive levels. This allows for the screen display to be scaled up 8 times (magnification).

Raster

With microcomputer systems, information on each dot (or pixel) can be stored directly in the video memory. This ability allows dot or patchwork information to be captured and displayed. Four pieces of information are required for each dot, namely: the active layer to which the raster data are to be assigned; the colour; and the X and Y coordinates of the pixel. The raster data are stored in the same file designation as the vector data. Whilst more than one colour may be defined for a layer, it is recommended that a different layer be used for each colour. The trade-off for this ability to address every dot on the screen is the amount of storage required. To overcome storing large areas of non-vector information, a special vector method is employed which has been described above (see discussion on flood-fill).

The raster type data are entered by the system in a slightly different way to that of vectors. Firstly an area on the screen is selected by the user using a movable window. The selected area is then zoomed in. Each pixel within the zoomed area is represented on the screen by blocks of the appropriate colour (this will only apply to the layers that are active on the screen at the time of the zoom). The user can then select a new layer, a different colour and indicate the 'pixels' to be 'turned on'. The pixel information for the current layer can only be stored by explicitly indicating the relevant soft key.

With this method each pixel may be defined on more than one layer, although only one colour will appear on the screen at any one time. This is an important aspect in the development of a system applicable to a wide range of applications.

Applications

General

The planning and analysis of water resource systems have in the past been carried out on an *ad hoc* basis, each project being evaluated as a stand-alone system. No facility existed whereby the planner could obtain a clear overview of all facets of the problem at hand. Due to the increased awareness of the limits of our natural resources in relation to the growth of the population, it has become essential that a more comprehensive methodology be developed to assist the planning professions.

The first step in the process of studying a particular region is the capturing of the physical map. This follows the standard procedure for any digitising process with the required detail being captured on layers defined by the operator.

An illustration of typical layers which can be used for general water resource analysis problems is given below:

- Catchment boundaries
- Roads
- Railway lines
- Settlements and their names
- Dams and lakes
- Main rivers
- Tributaries
  - Underground lakes } if available
  - Underground streams
  - Dykes and aquifers
- Contours
- Overlay grids (1/4 or 1/2 degree intervals)

The ability to specify which of the information items and/or layers should be displayed at any one time is possible using the map utili-

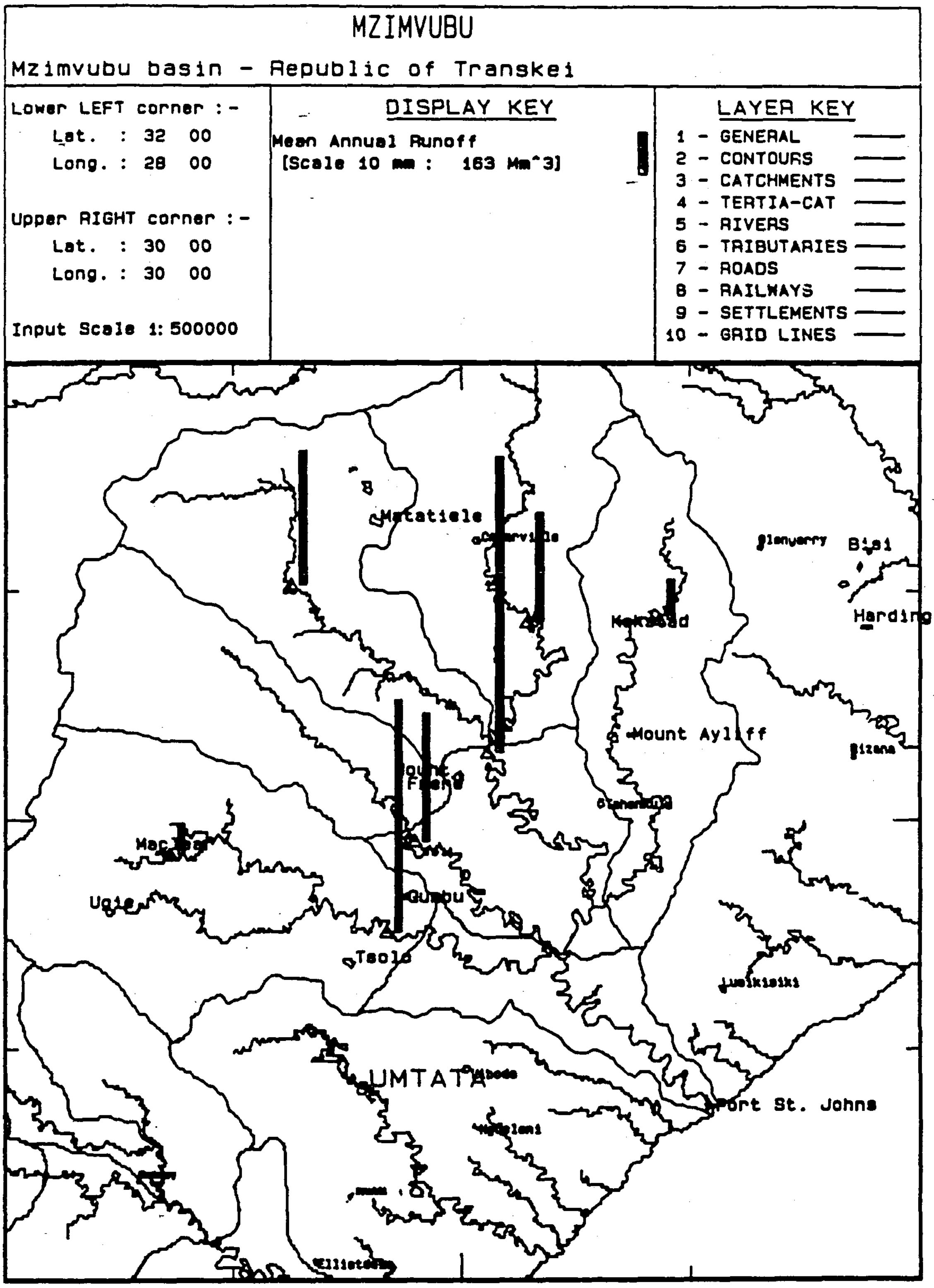

Figure 4
Example of mapping utility plotter output for the Mzimvubu Basin, Republic of Transkei

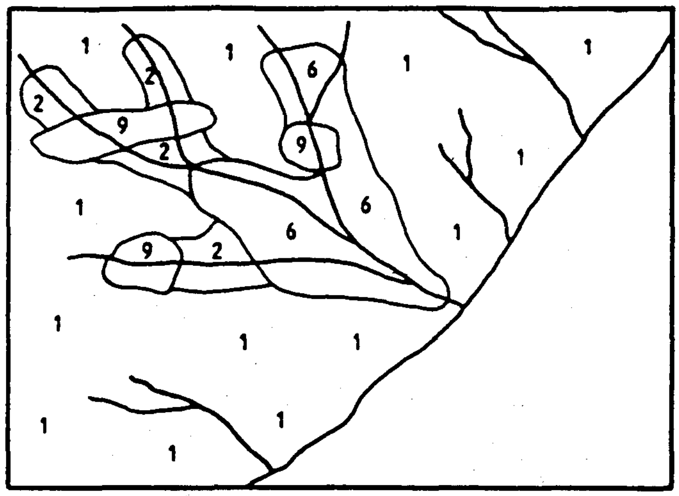

Figure 5
Illustration of overlay for geological site conditions

ty described. It is feasible therefore to display the population densities and overlay the available water supply data. This immediately indicates the areas requiring increased investment in water supply orientated projects. The analysis section is totally interactive. It is therefore possible to erase the population densities and superimpose the ground water potential, in order to assess the possibility of using the aquifers as a potential water supply source to satisfy any shortfalls.

Resource displays

The Mzimvubu River drainage basin, situated in the Republic of Transkei, was used to demonstrate resource mapping with the graphics utility. The map was digitised in at a scale of 1:500 000. Figure 4 shows bar graphs at the bottom of each subcatchment superimposed on the basic map of the Mbashe Basin. Note that not all the layers are 'switched' on for the benefit of clarity on a monochrome printout. The graphs represent various categories of river flow, for example average monthly and cumulative monthly. On the screen each bar is represented in a different colour for each location. The data can also be used as input to more complex analysis procedures.

Screening of plans/optimum location

A core driver is included within the program specifically designed for the analysis of water resource plans and optimum location of new projects. This methodology can be adopted to serve a variety of similar tasks not directly related to water resources planning.

The methodology is initiated through a special menu option. For the region selected the relevant layers are displayed on the screen to assist the operator in the development of his overlays. The overlays are a spatial representation of the decision-maker's preferences or physical constraints on the project. A typical example of a physical overlay would be the geologically favourable areas for dam construction or government-imposed land zoning. Overlays which would simulate the decision-maker's preferences would include intangibles such as ecological effects, recreation and flood alleviation. A scale from 1 to 10 is provided in order that the operator can grade the effects of the imposed measure, over the mapped area, according to the decision-maker's preferences or the physical constraints. A typical overlay for geological conditions in determination of a reservoir site is shown in Fig. 5.

The operator may define as many overlays as are required by the specific project. A six-character identifying string is assigned to each of the overlays. Suitability analysis (Berich, 1985) can now be carried out using a procedure called indexing. Indexing requires that a weight is given to each of the overlays to be combined for the suitability analysis. This weight is assigned by the decision-maker and can be varied to perform a simplified sensitivity analysis. The procedure analyses selected overlays (raster files), using the weighting in the following form:

$$I = C_1V_1 + C_2V_2 + C_3V_3 + \ldots\ldots\ldots\ldots C_nV_n \qquad (1)$$

where:

I = index value
Cx = weighting coefficient
Vx = variable codes in separate overlay files (raster)

The index value obtained now can be displayed as an overlay on the physical map previously digitised. An illustration of the combination and indexing of two overlays is shown in Fig. 6.

The decision-maker is provided with a simple picture of the suitability of an area for the proposed development either by the display of only non-conflicting or coincidental areas. The data can be used to generate acceptability contours which will facilitate interpretation. The facility exists to specify the overlays to be used in each analysis, the effects due to the neglect or incorporation of various factors can therefore be investigated. This allows decision-makers to investigate a variety of factors in the screening process. The process is rapid and can therefore be used for immediate sensitivity analysis to establish, which of the factors require more detailed analysis. The process rapidly screens out unsuitable options at a minimum cost. The incorporation of intangible effects is an added benefit of the process.

Others — e.g. stream lengths, area of catchment

With the development of hydrological simulation models becoming more physically-based (i.e. the parameters to the model can be

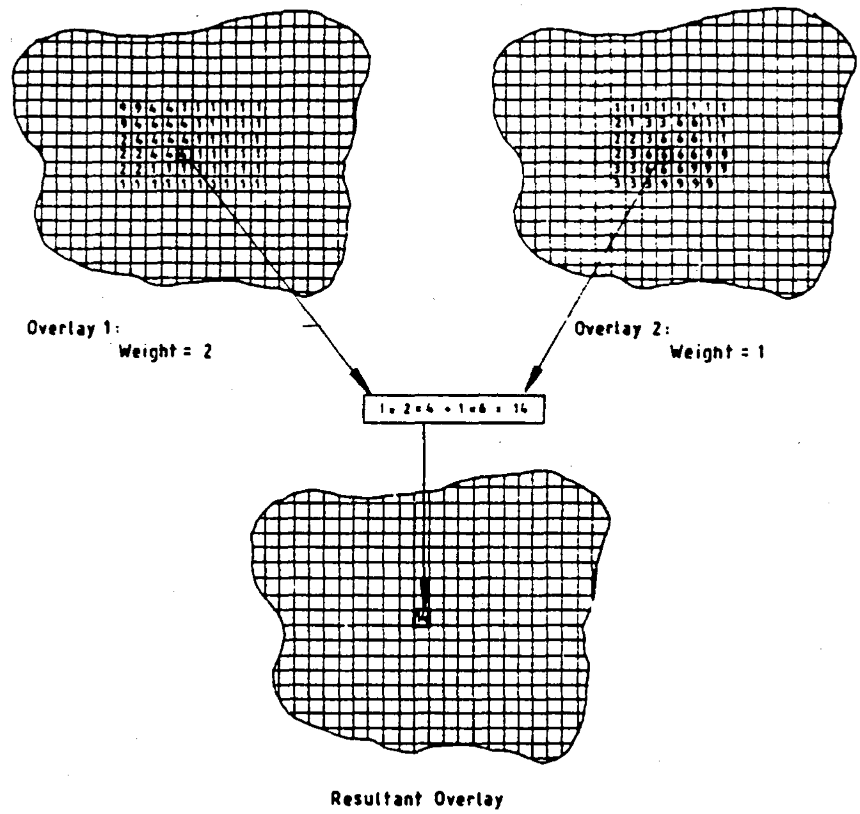

Figure 6
Indexing methodology for combination of two overlays

determined by physical measurement), certain parameters may be determined from maps. Items such as slope, the length of streams, and the area of catchments are perhaps the most obvious.

Models such as TOPMODEL (Beven and Kirkby, 1979) and unit hydrograph analysis involve the entry of numerous topographic data values that can be derived from a map. The graphics subsystem can be used to enter the relevant map's information and an application can then be added comprising the simulation model and a series of routines to interpret the map information. The graphics utility will therefore act as an intelligent front end for the capture of data for simulation programs.

Discussion

The package has been designed to provide a practical planning tool for use by decision-makers in developing regions, posessing limited technical manpower and skills. The system is user-friendly and requires a negligible training period when compared to conventional CAD/GIS systems.

It is the intention of the authors that the graphics utility developed in this paper can be incorporated into a variety of applications packages. The tool provides the operator with the ability to interpret data rapidly which would otherwise require a detailed analysis of tabular data. The utility is compiled in a stand-alone format and Fig. 1 gives an indication of a typical integration configuration with an independent analysis package.

A variety of applications have been identified. The descriptions of its use in a resource mapping and the optimum location selection process, illustrates some of the benefits of a graphical system. Display of data and results allows for simple interpretation by decision-makers and reduces the possibility of misinterpretation. Graphical output further allows the planners to visualise the spatial distribution of the resource and the spatial effects of decisions. The indexing and overlay methods allow for rapid screening of unsuitable project proposals and can be used for sensitivity analysis of the identified factors. The availability of accurate and up-to-date maps allows any user to access and input spatially variable data into his model.

The mapping utility has further been used in association with a mass balance analysis program developed by one of the authors. The utility is used to input and analyse spatially variable data such as rainfall, evaporation, infiltration, etc. Integration was found to be simple and the customisation was minimal. Results of the analysis were used in the comparison of the effects of urbanisation on the water balance of small catchments (Lambourne and Sutherland, in preparation).

The graphics utility developed satisfies the major requirements of a utility of this nature, namely versatility, ease of use, compatibility and low cost. The utility will, no doubt, be found to be lacking in certain areas by specific users. It was never the intention to develop a tool which would satisfy all needs and it is not considered to be a replacement for any of the commercially available CAD or GIS packages. We hope merely to have provided the engineer, planner or any other decision-maker, with the means to capture, analyse and display spatially variable data quickly and simply, using the hardware configuration which is already at his disposal.

Acknowledgements

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A User-Friendly Hydrological Data-Management and Reporting System

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This article describes the development and implementation of an integrated hydrological management and reporting system on a microcomputer. The system involves the use of peripherals for digitizing, EPROM (Erasable Programmable Read Only Memory) or RAM downloading, storage, reporting, and archiving of hydrological data. Hydrological information is stored on hard disk in compact formats according to the type of data. Self-correcting data routines coupled with simple screen drivers for user interfacing allow semiskilled operators to use the system. An on-line help facility is an integral part of the user interface. Both pattern and color recognition by the operator are used to detect discrepancies in the data.

INTRODUCTION

There has been a major trend since the early 1960's to the growing use of computers for data capture, storage, and analysis. The development of computerized storage and analytical systems have become the central core of many water-management and research programs. These systems range in size (and usually complexity) from large national and international systems on mainframes to small in-house systems on microcomputers.

Prior to the widespread use of computers, hydro-meteorological data was stored in tabular format on paper media. The early developers of these database systems persisted with the storage of the information in "spreadsheet" tables (in ASCII format) using the computer hardware as a fast-speed accessing device. Unfortunately, many of this type of systems are still developed, or are in existence today. With this type of approach, the Hydro-meteorological data manager has to develop complex data retrieval routines to access the relevant information.

Apart from small in-house systems to store and retrieve data in tabular format, most complex data management systems have been confined to mainframe applications [1,2]. With the development of both improved performance and increased storage capacity (hard disks and programmable tape streamers), the cost effectiveness of microcomputers have proved their worthiness in hydro-meteorological data management systems. The development of a hydro-meterological data management system on a microcomputer requires special considerations that are not necessarily incurred on a mainframe computer. The graphics handling and user friendliness of a microcomputer, however, aids semiskilled staff in processing and management of data.

REQUIREMENTS OF A HYDRO-METEOROLOGICAL DATA-BASE SYSTEM

The objectives of a hydro-meteorological data-base system to be implemented on a microcomputer system are summarized below.

1. Input and editing of hydro-meteorological data to the system from any media (i.e., tables, charts, or electronic media currently used).
2. Storage of hydro-meteorological data in compacted data-base files that allow for optimization of space and access time.
3. Have the ability to perform routine analysis and provide report-quality output of both data and summary information.
4. Be able to output requested data sets in a form rapidly accessible by other types of hydrological software (e.g., hydrological rainfall-runoff models).

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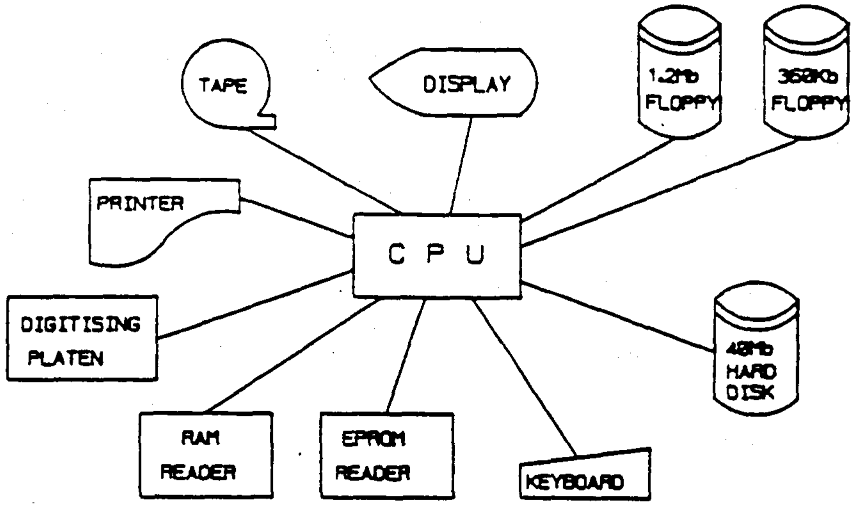


FIGURE 1. Hardware schematic.

5. Software designed in a modular approach to enable upgrades to be undertaken without recourse to complete software revision. This approach is a prerequisite to efficient software development.

These objectives form the basis for the development of the hydro-meteorological database management and reporting system. It was decided, that in order to produce software that was extremely user-friendly and allowed hydrologists and engineers conversant with the computer language to update and modify the software, that product data-base systems would not be used.

SYSTEM DESCRIPTION

This application was based on currently available microcomputer hardware that is both cost-effective and widely used in hydrological applications. The hardware is built around an IBM or compatible microcomputer with a 80286 processor and associated peripheral devices operating under MS-DOS™. A schematic of the hardware is presented in Figure 1. The peripherals include hard disk; programmable tape streamer and twin floppy drives for management of data; digitizing platen; RAM reader; and EPROM reader for input of data.

The software element of the system operates from a 1.2-Megabyte floppy disk. The system may be loaded onto a partitioned fixed disk. The first disk contains the start-up routines and the main management system program. The second disk contains the configuration and help information screen files.

The system logo is displayed while the main software is loaded and initialization is performed. A simple password control system is then enabled.

The ability to control both access to and manipulation of the hydrological data within the system is paramount in preserving the quality of the information. This ensures that certain operations (e.g., deletion of data sets and editing of data) can only be carried out by a skilled member of the hydro-meteorological data management team. The password and operator's name are linked to a status level that determines the level of constraint of the user on the data base, temporary and system files. The pres-

FIGURE 2. WITDMS main menu.

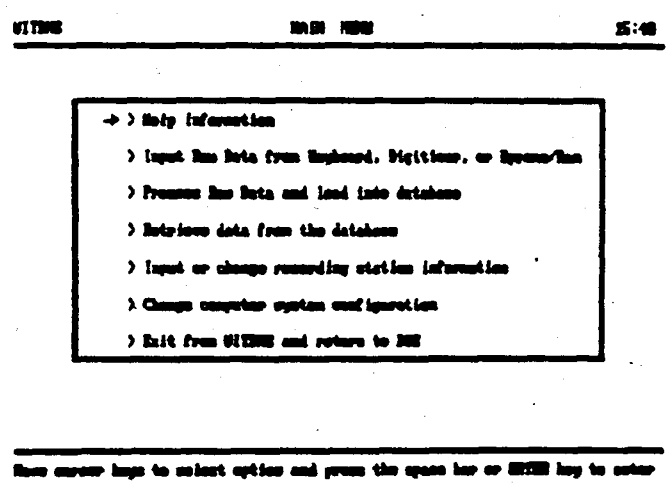


TABLE 1 STATUS LEVELS FOR ACCESS TO SYSTEM

Level	Name	Description of Access
1	Hydro-meterological data manager	ALL files for creation, editing, deletion and retrieval
2	Hydrological staff	Data-base and scratch-pad files for creation, editing, deletion and retrieval
3	Data process staff	Data-base and scratch-pad files for creation, editing and retrieval
4	Data capture operators	Input ONLY of data from keyboard, digitizer, and EPROM or RAM readers

ence of status levels also inspires confidence in the semi-skilled user in that he or she cannot do anything to the system files by accident.

The four status levels are tabulated in Table 1.

The system is driven through a series of menus whose options are selected by cursor and activated by pressing either the space bar or the carriage return keys. The main menu is presented in Figure 2.

Exiting from the system returns the user to the MS-DOS command level and the database subdirectories are closed. This allows further packages to be loaded (i.e., simulation models that use data from the database).

Where there is a menu requiring a choice to be made by the user, help information is available. This consists of one or more full screens of information stored in a file suffixed by .HLP and stored on the second system disk. These on-line help screens are not a complete substitute for the manual but provide enough information for the user while "on-line."

TABLE 3 LIST OF KEY LETTERS FOR PARAMETERS BEING MEASURED

Code	Description	Code	Description
A	Atmospheric pressure	B	Borehole levels
C	Electrical conductivity	D	Wind direction
E	Evaporation	F	Sewage outflow
G		H	Relative humidity
I	Precipitation	J	
K		L	Hours of sunshine
M	Soil moisture	N	
O		P	Precipitation (totalizer)
Q	Streamflow (discharge)	R	Solar radiation
S	Streamflow (stage)	T	Temperature
U		V	Reservoir level
W	Windspeed	X	Multichannel logger system
Y		Z	

STATION INFORMATION SUBSYSTEM

Each recording gauge in the hydro-meteorological system has certain features that make it unique. These features are stored in a separate data-base file called STATION.DBF. To facilitate fast access to a particular recording gauge information (each recording gauge is designated a "station"), an index file (STATION.INX) is used which stores the station identifier and place name. This station information can be accessed directly for reports or by other subsystems of the system. It is possible for status-level 0–2 users to change, delete, or add to these station information files.

Each station has a unique identification set of eight alpha-numeric characters. An example is presented in Table 2.

The middle six digits relate to the latitude and longitude of the station. In the situation where there are more than one instrument at a site, the digits remain the same but the prefix character changes according to the parameter being measured. A list of hydro-meteorological parameters and their codes are given in Table 3.

Apart from the station identifier, further information is contained in the station information files. This can be divided into two parts, namely, physical data and information required for processing the raw input data (e.g., chart sizes and scales for digitized input). The physical data comprises the name of site, start date of gauge recording, type of input (keyboard, digitizer, or EPROM/RAM), type of data storage format (daily, breakpoint, fixed-time interval, or rating-curve tabulation), unit specifier, and multiplier. With digitizing input, information on the chart type, X-axis and Y-axis scaling factors and total time across the chart that can be fitted to digitizer at any one time, are required. The EPROM/RAM input requires information on the channel number of the data and the time interval.

It is also possible to perform an automatic filtering of the input data using a band threshold defined on the Y-axis. This is especially applicable to digitized data where small variations as a result of chart misalignment or "noise" occur. The filtering out of this data results in a more compact data base.

TABLE 2 STATION IDENTIFIERS

Character	Description
I	Key letter for parameter being measured
476	Section number (0,5 × 0,5 degree grid square)
123	Position number (1 × 1 minute grid square)
1	*n*th station opened within a given position number

INPUT OF DATA (SUBSYSTEM)

Input to the hydrological data-management system is through three different input peripherals, namely, keyboard, digitizing platen (or tablet), and EPROM/RAM data reader. An IBM™ compatible-type keyboard with either separate or combined cursor and number keypad entry may be used for keyboard entry. The digitizing platen can either be an A4 (300 mm × 210 mm) or A3 (420 mm × 300 mm) size with a stylus pen for input. At present, only the MCS™ EPROM and DDS™ RAM readers have been tested with the system.

Prior to input of data, the relevant information on the recording station should be entered into the station database file using the station information subsystem.

TABLE 4 DIGITIZER SOFTKEYS AND COLOR CODES

Softkey	Name of Key	Description
1	NEW CHART/NEW START DATE & TIME	To select new chart or new start time and date written by operator (Color = Black)
2	NEXT PORTION OF CHART/FRAME	To select next frame or part of trace on strip/large chart (Color = Blue)
3	Y-AXIS BASELINE VALUE	To set baseline to value (not zero) (Color = Orange)
4	Y-AXIS OVERLAP	To switch chart overlap on/off (Color = Green)
5	MISSING DATA TRACE	To indicate missing data trace(s) (Color = Red)
6	CANCEL LAST TRACE	To delete data on last trace
7	END OF HORIZONTAL TRACE	To indicate end of frame trace (Color = Brown)
8	END OF STATION CHARTS	To indicate end of station charts

There are four different data configurations that can be stored by the data-management system. These are single data, breakpoint, fixed interval, and rating table formats (Table 4).

1. *Single data.* This type of format is for daily data where only one data element is required (e.g., totalizer rainfall).
2. *Breakpoint.* This type of format is for use with continuous data that is to be recorded at non-set time intervals, usually on chart media (e.g., autographic rainfall records).
3. *Fixed time.* This type of format is used with continuous data that is recorded at a fixed-time interval (e.g., temperature).
4. *Rating table.* This type of format is used for tables that transfer data from one type of parameter to another (e.g., transfer of stage to discharge measurements).

Keyboard Entry Description

The station is selected from a cursor-driven list. Once the system has loaded the station information, the user is presented with a full-screen input mask that allows both entry and editing of data on the screen with the cursor key. Once data has been entered into the system using the screen accept key, it can only be edited again using the raw data editor. The entered data is then stored in a temporary file called KEYBOARD.TMP (on the primary disk) that is erased once the data has been loaded into the data-base files (using the processing subsystem).

Digitizer Entry Description

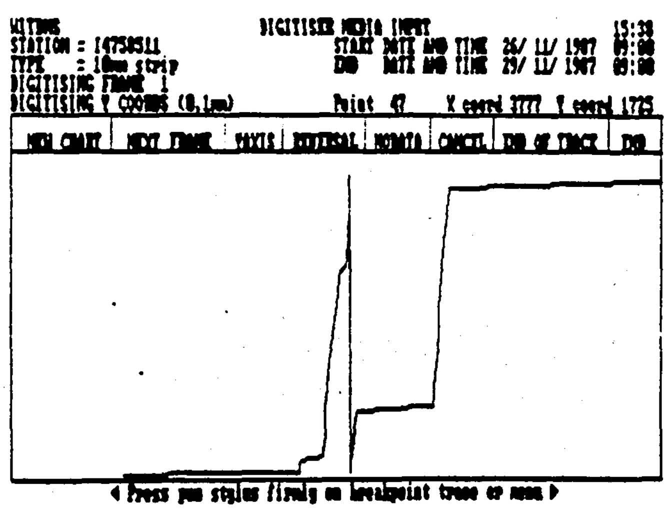
Data that are to be entered from an analog source have to be first converted to digital input. The digitizing software component of the system uses a digitizing platen as the major peripheral to transfer the breakpoints of the chart trace to the computer.

The station is selected from a cursor-driven list and the start-and-end times of the chart (chart type and scales are loaded from he station data-base files by the system) entered by the user. The trace that is being digitized is displayed on the screen to allow any input errors to be detected (Figure 3). A series of programed digitizer "softkeys" allow tne user to indicate observations that will be needed in the processing of the chart (e.g., missing trace, start of new segment of chart, and end of trace, etc.).

A unique method used in the digitizing is through color recognition. Before transfer of the analog image to the computer via the stylus, the chart has to be checked for discrepancies. The absence of traces on the chart need to be recognized and the chart marked to indicate these discrepancies to the digitizer. Each of the defined actions (to be entered to the computer via the softkeys) has a particular color code. The digitizer then presses the relevant softkey when a particular color is encountered on the chart. A list of softkeys and their color codes are given in Table 4. A number is also assigned for those operators who have color blindness. Figure 4 shows a representation of a chart with softkey codes.

The system will allow entry from most available hydro-meteorological charts. Each "frame" of the chart (that portion of chart that covers the digitizing platen) can be erased from memory prior to storage in a temporary file (DIGITISE.TMP) when the trace shown on the screen is different from that on the chart. Major use of digitizing was envisaged for processing of existing autographic rainfall and water-level recorders' chart. The

FIGURE 3. Screen display during digitizing of rainfall trace.



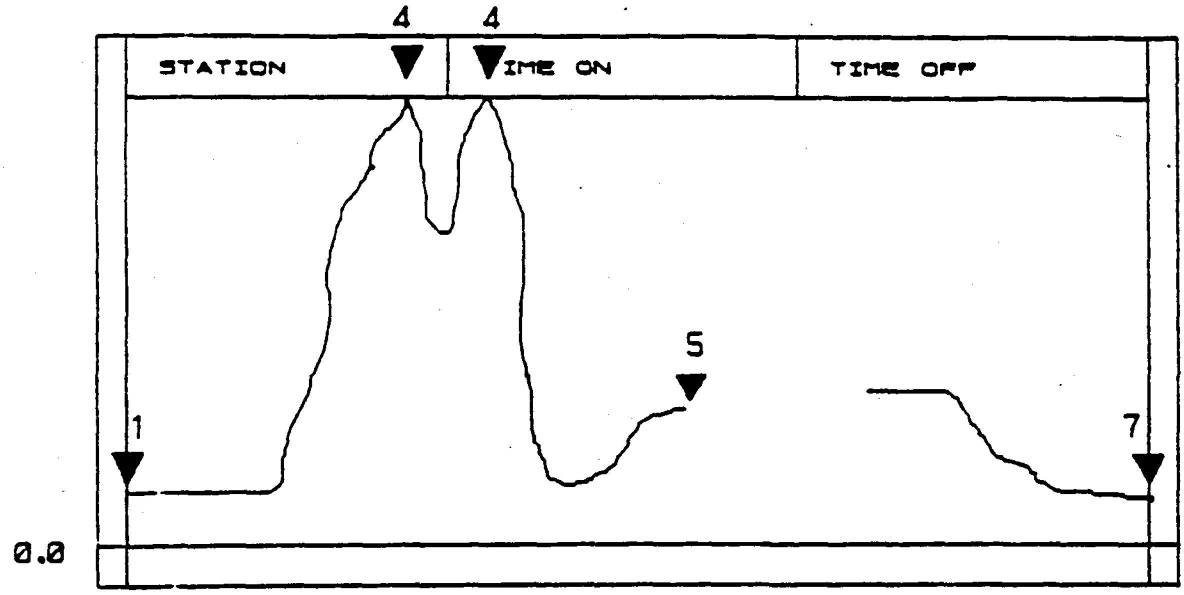


FIGURE 4. Chart preparation—softkey codes.

transformed analog-to-digital data held on disk can be further checked using the raw data editor that displays the digital data on the screen.

EPROM/RAM DATA ENTRY DESCRIPTION

The data stored on each EPROM or RAM "chips" is first read and translated in ASCII character format by the EPROM/RAM data reader software and hardware. Transmission of ASCII format data from the EPROM data reader and the PC is via RS232 link. WITDMS is able to use the low-level communications protocol to read, stop, and reset pointer in RAM or EPROM chips, as well as x-on/x-off data control. ASCII strings received from the EPROM/RAM card reader are stored in a temporary file (EPROM.TMP).

PROCESSING/LOADING (SUBSYSTEM)

Since processing of the raw data and loading into the data-base files of the system is in all probability the most time-consuming task (from the point of view of the computer's CPU), the concept of a multi-pass system was introduced into the data-management system. The approach is best represented in a flow diagram (Figure 5).

FIGURE 5. Processing of rain data flowchart.

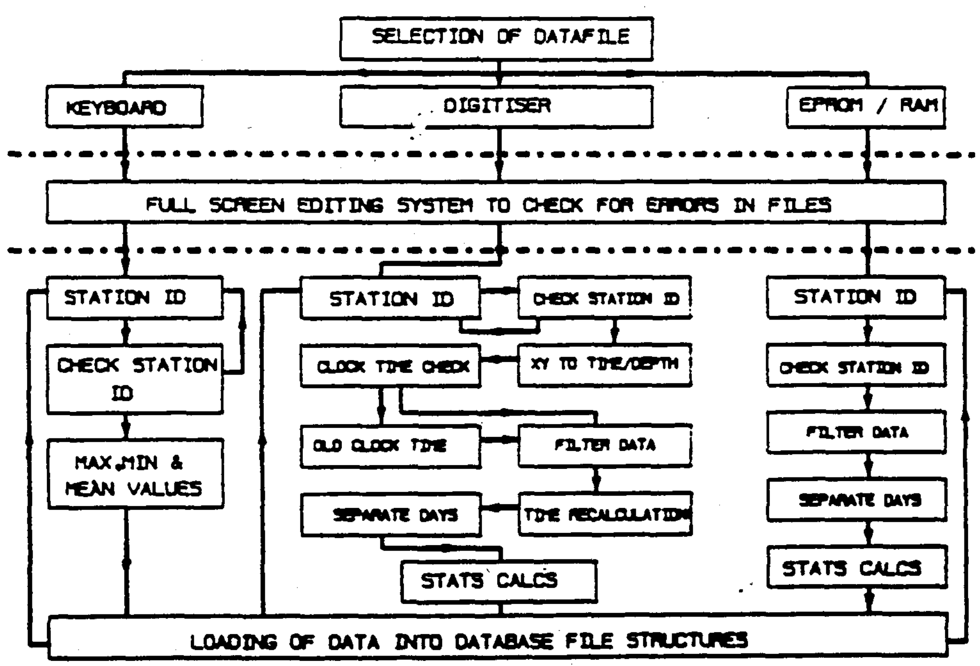
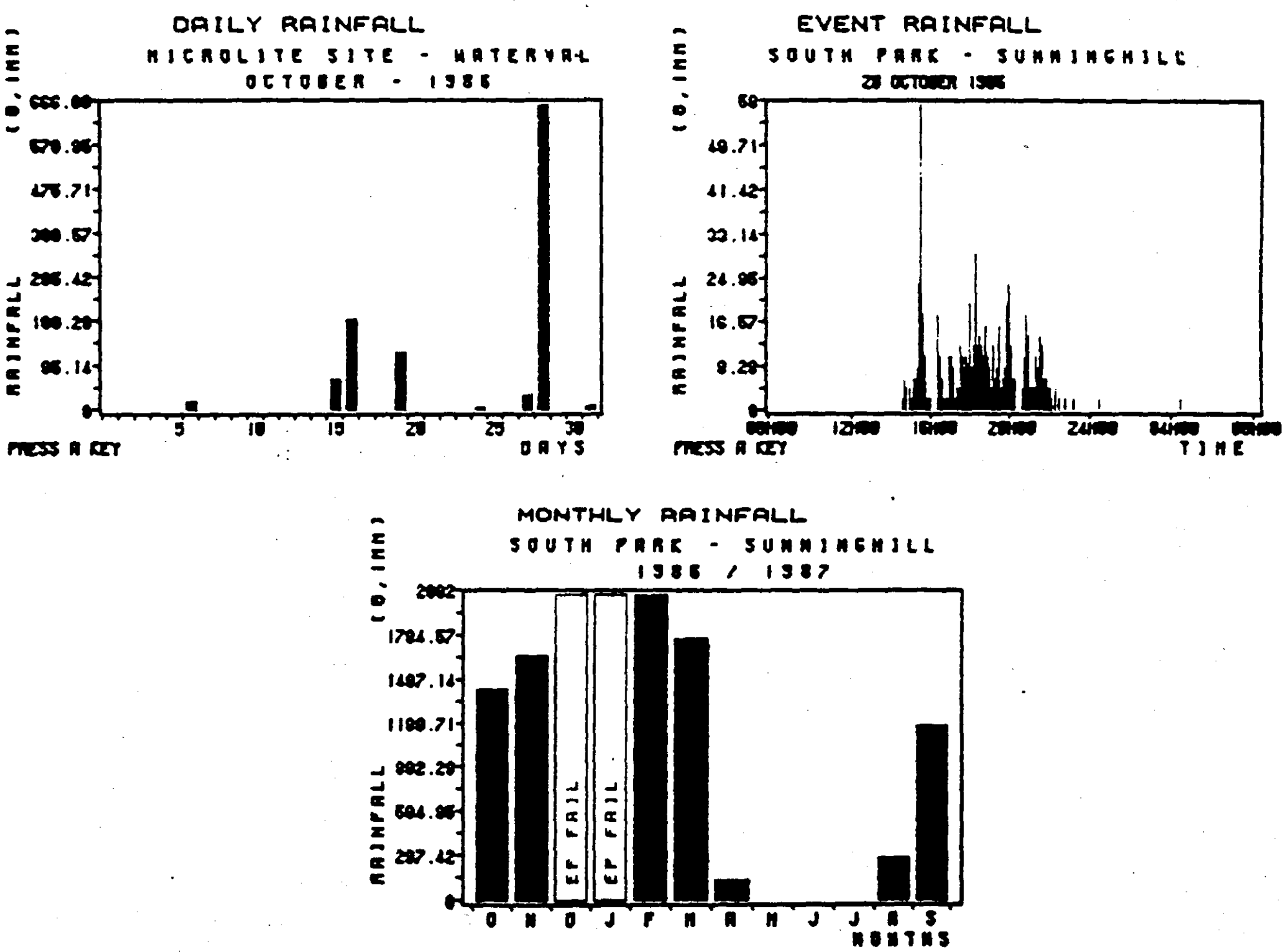


TABLE 5 PROCESSING OF DIGITIZED RECORDS (RULES)

Section a: Determination of Time Axis	
Rule 1	IF length of chart traces in minutes IS GREATER THEN actual time between on and off times AND the difference between chart and actual time IS GREATER THEN 24 hours THEN Use chart speed from station information data base and EXIT Section a
Rule 2	IF length of chart traces in minutes IS GREATER THEN actual time between on and off times THEN Use chart speed as ratio of the two times and EXIT Section a
Rule 3	IF length of chart traces in minutes IS LESS THEN actual time between on and off times AND the difference between chart and actual time IS GREATER THEN 24 hours THEN Use chart speed from station information data base and EXIT Section a
Rule 4	IF length of chart traces in minutes IS LESS THEN actual time between on and off times THEN Use chart speed as ratio of the two times and EXIT Section a

Section b: Embedded Processing Codes	
Rule 1	IF code EQUALS -33333 THEN Add the value of the Y-axis starting level to all y-axis values and EXIT rules
Rule 2	IF code EQUALS -44444 AND y-axis overlap flag NOT SET THEN SET y-axis overlap flag ON and EXIT rules
Rule 3	IF code EQUALS -44444 AND y-axis overlap flag SET THEN SET y-axis overlap flag OFF and EXIT rules
Rule 4	IF y-axis overlap flag SET ON THEN Add maximum y-axis (from graph) to y-axis value and EXIT rules
Rule 5	IF code EQUALS -55555 THEN SET missing code flag ON and insert x AND y missing data coordinates and EXIT rules
Rule 6	IF missing code SET ON THEN SET missing code flag OFF and insert x AND y missing data coordinates and EXIT rules

FIGURE 6a. Examples of reporting system output.



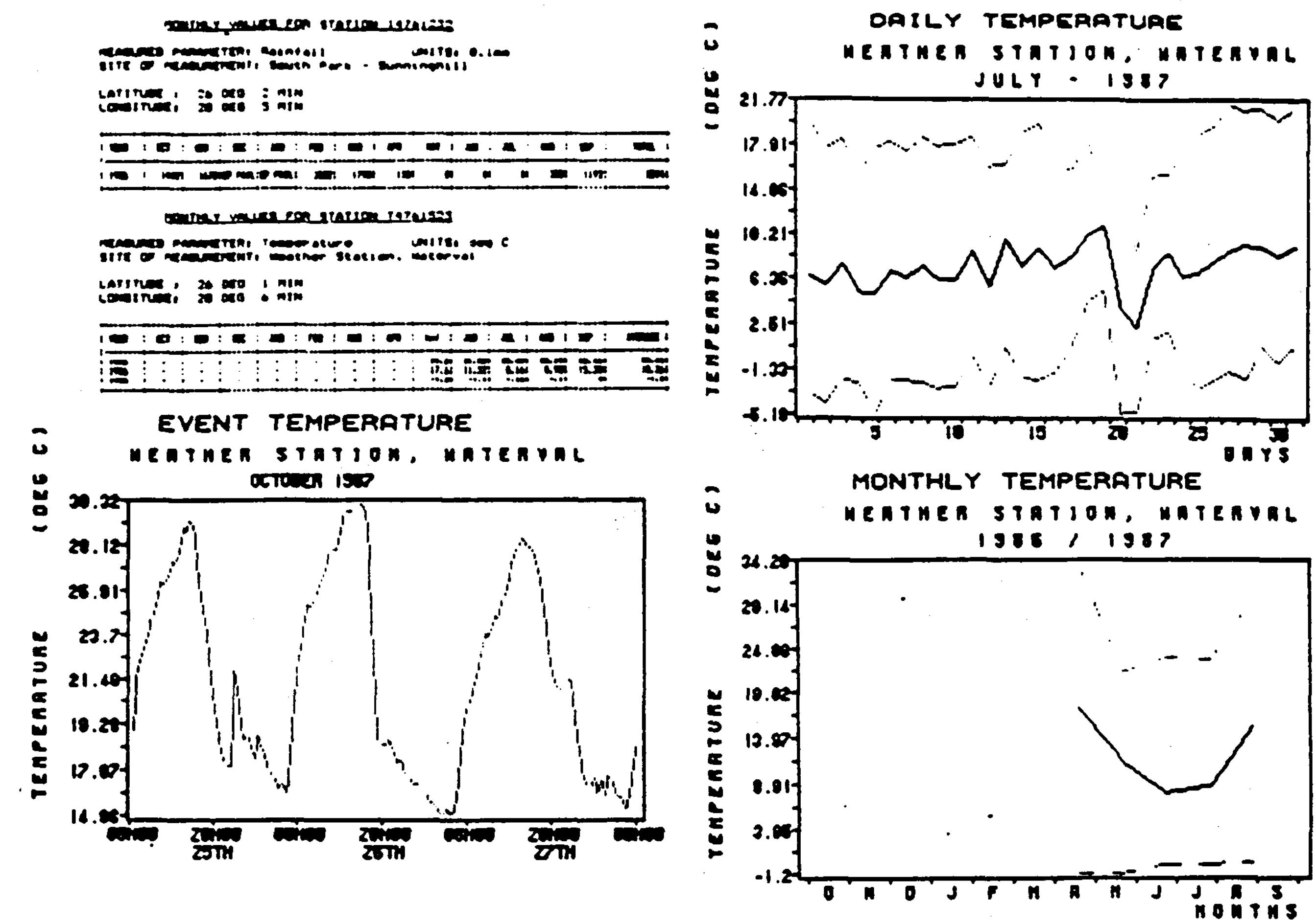


FIGURE 6b. Examples of reporting system output.

The raw data has to be first scanned by the user to check for obvious errors in the data formats. This is a tedious but necessary chore involving a full-screen editing system. The editing system consists of an ASCII file interpreter with the capability to step backward and forward through files, insert and delete lines or characters, overtype characters, and window-on files greater than 80 characters wide. Prior to editing a file, a backup copy is made which can be retrieved should the user require it later, although further editing will overwrite this copy. The format of data within the file is kept simple to enable the user to detect incompatibilities in the structure of the file.

The second phase is the loading of the data into compact database files. Any system with a high disk-access-to-processing ratio needs to have an efficient means of addressing database files. This relates to both space and speed of access on the disk. Numerous tests involving each type of data format were undertaken to minimize the two components of disk access. As a result of these tests, it was decided to store each calendar year as a separate file with an index. The index was used to detail the Julian calendar day and record the number of the start of the day's record in the data file. The length of each data-base record was also optimized using test data.

Both keyboard and EPROM/RAM entry data require minimal computation; namely, calculation of the average, maxima and minima of the data for each recorded Juliian calendar day (the day being defined in the data base as from 08h00 to 08h00). Digitized data is stored temporarly as x and y coordinates that need to be converted to the time and variable coordinate system. At the same time, the speed of the chart relative to the actual time has to be adjusted. Further complications result from the "color codes" that are inserted into the data using the softkeys. These codes are interpreted by the system without any intervention from the user but following a series of rules. These rules have been defined to maximize the amount of quality data in the data base (Table 5).

The conversion of stage measurements to discharge is performed using user-entered rating tables. The original stage measurements are retained in a separate database since rating tables are regulary updated. A separate discharge database is produced when conversion calculations are performed. The system has the ability to allow the user to define the period of calculations.

REPORTING SYSTEM

The ability to provide report-ready output distinguishes the system from previous hydro-meteorological data management systems. Most technical publications require tables and diagrams to conform to the standard A4 format. In line with this approach, the WITDMS software will produce a series of tabulations and graphics to describe different parameters and sites. A series of output tables and graphics are presented in Figure 6.

UTILITIES

Several utilities were designed together with the main processing and data-base package. These are briefly described below.

1. During the raw data-editing procedure, a backup of the original file is made. A utility can be used to restore the backup file to current-file status.
2. Any file (data base, raw data files) can be copied to another disk media including tape streamer.
3. Specific sets of data (monthly, daily, time-series) can be translated into a sequential ASCII data file for use in other software.
4. The structure of the software allows the user to add his or her own routines for manipulating data using the retrieval routines built into the package.

SUMMARY

A description of a user-friendly hydrological data-management and reporting system has been presented. The system overcomes both the disadvantages of previous methods of storing hydrological data and the business-oriented generalizations of other microcomputer data-base packages.

The system was developed primarily for use in developing countries where skilled workers are at a premium; however, it is at present also being used to store research catchment data.

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A Graphics Utility for Integration in Water Resource and Hydrological Simulation problems.

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This article outlines the development and application of a decision support graphics utility for implementation on an IBM or compatible micro-computer. Prime objectives included an affordable system, able to run on commonly installed hardware configurations and with the versatility to integrate with other user written packages. The package incorporates interfaces for the input, display and manipulation of spatial data. Both vector and raster graphical information types are supported. This utility was designed as an open ended structure to falicitate integration into a wide range of Water Resource and Hydrological applications. Brief descriptions of three applications of the package are presented.

Short Running Title

A Decision Support Graphics Utility

INTRODUCTION

Many commercial Geographic Information Systems (GIS) exist which can manipulate, analyse and display spatial data. There is however a dearth of systems which can easily be incorporated into `user supplied' analytical structures, and are also cost effective for small organisations. External systems can be attached to a GIS system (i.e. Database software and application packages), but require strict interface formats, and extensive user training in their operation.

The major feature of the system discussed in this article, is the ability to integrate it with most other application software. The graphics unit may be used as a stand-alone system with a core driver or, as part of an integrated application. Data can therefore be manipulated either within the graphics system or externally, before being displayed. Display data can be input from the keyboard interactively or, an integrated application can provide the data in the correct format for the graphics utility to interpret. For this reason all data is stored in ASCII format and is therefore easy to manipulate. This does however carry an additional overhead in terms of translation and disk storage requirements.

A powerful feature of the package is the ability to digitise individual maps, at specified scales, then combine them on the screen to obtain the complete picture. These individual maps are termed map segments and are linked to the scale of the map as shown in table 1. The user may specify all factors, such as layers, display values and display positions, for each individual map segment. Later it is possible to place all the map segments on the same screen in order to analyse the display output. Missing map segments or display information are ignored and can be input when they become available.

Hardware constraints

In the past the majority of the large, powerful Computer Aided Design (CAD) and Geographical Information Systems (GIS), were confined to mainframe or mini-computer systems. This was largely due to the speed and memory requirements of these packages. With the vast improvements made over the last five years in micro-processor technology, it has recently become feasible to run these packages on standard micro-computers (although integrated GIS systems still require a super-microcomputer to execute effectively).

With the current pace of technology, the definition of a conventional or 'common' configuration is very difficult, however a minimum configuration can be suggested. Figure 1 illustrates the minimum configuration required for the graphics utility (excluding specific requirements for individual applications).

In the IBM and compatible market, the Intel 80286 chip has become established as the new baselevel standard in micro-processors. It allows for multi-tasking and networking, which are not supported by either the 8088 or the 8086 processors (PC and XT models). The package can be run on these lower level chips but an associated sacrifice in speed must be expected by the user. The 80286 is therefore a more versatile chip, allowing for the possibility of upgrading and an associated increases in speed. Whilst the present version of the system runs under MS-DOS, an OS/2 version incorporating multi-threading for specific applications, is under development. The 80386 chip will of course provide even greater speed but the costs and lack of full operating system support do not at present warrant upgrading. The 80386 is therefore not considered an entry level machine at present.

Advantages in speed are gained with the inclusion of a math co-processor. This is significant for the slower machines, 12 MHz and below, but we have found that from the 16 MHz level upwards the benefits do not warrant the extra cost. The cost of the co-processor is especially significant in non-highly industrialised countries, therefore the mapper system has been designed to run with or without the addition of the math co-processor.

It was found that at higher processor speeds the graphics card becomes the more significant controlling factor. It is therefore important that this card is tested for line, fill and character drawing speed before it is purchased. A faster card will be of great advantage to the user and may not be significantly more expensive. Redrawing operations are significantly affected.

The EGA (Enhanced Graphics Adaptor) graphics level was initially chosen as this is considered to be the basic level at which graphics can be effectively manipulated and provides an affordable entry level resolution for the small company or research institute. The highest resolution available on an EGA is 640x350 pixels with 16 colours from a palette of 64. It has the advantage of being upwardly portable to the VGA (Video Graphics Array) standard which increases the screen resolution. This upgrade would however incorporate a significant cost element, as EGA cards use digital displays and the VGA format uses analogue displays.

A minimum of 640 Kb RAM is recommended although the system will run on a 512Kb RAM machine. To achieve a low RAM memory usage, we have endeavoured to keep the data in memory to a minimum, therefore disk (file) access is high. Due to the high amount of data transfer and the large storage of data which is inherent in any CAD/mapping system, we recommend that a hard disk system of 20 MB or larger be used. The addition of a file caching system to increase speed of the hard disk I/O is also recommended.

In order to obtain a hard copy of the map displayed on the screen at any time, provision has been made for both a printer and a plotter driver. The standards chosen for these options were the Epson graphics printer (ESC/P command set, dot-matrix) and the HPGL plotter format, which have become universally accepted standards.

STORAGE AND ADDRESSING CONCEPTS.

In order to facilitate the digitising of the individual map segments, a special system for the handling of the data was devised which allows for the size of the map segment to vary with the map scale. The map segment will therefore always fit on a standard digitiser platen. To allow universatility within the system, latitudes and longitudes from the bottom left hand corner of the segment, together with the scale, are taken as the reference characteristics for each map segment. The scale is converted, according to table 1, into an alpha character and the latitude and longitude, in tenths of a degree, are then added to provide the complete file name. A map to a scale of 1: 250 000, with a left hand, bottom corner at latitude 32^{o} and longitude 28^{o}, would therefore have a file name of ; C320280.

The scales selected were taken from those commonly used in practice. Table 1 illustrates the different scales and the corresponding areal coverage for each map segment.

Layer files are created in order that the user may specify any combination of data for display on the screen at any one time. Layer files are referenced to the scale and project by the file name and to the particular layer by the first three letters of the layer name which are used as the file extension. There is no limit to the number of layers allowed (although only 16 colours are allowed on the screen at any one time) as all data is accessed from disk and is not stored in memory. A number of layers, of different names can be defined for each project at set up time. Each layer has a default colour, assigned on entry, which can be changed for each project.

METHODS OF DATA ENTRY AND MANIPULATION.

It is important to consider the nature of the data as this affects both the method of data capture and the storage of coordinates. Beran (1982) suggested that geophysical data should be classified into various categories. With this application it was decided to classify the type of data into three groups.

1. Continuously varying data, usually expressed as isolines (e.g. altitude and rainfall).

2. Continuous feature information (e.g. river channels, roads and railways).

3. Constant value information, expressed as a patchwork (e.g. forests, vegetation cover and soil types).

Groups 1 and 2 were captured and stored as vector information whereas group 3 was stored as raster datasets. The two systems of data capture and display are described below.

Vectors

The vector data storage format was chosen as the primary method as it is both time and space efficient. In order to represent a line between any two points on a two dimensional system, the minimum requirement is an X and Y coordinate at both ends, that is four points. To facilitate data retrieval and regeneration of the vector, the number of turn points are also recorded.

Due to the constraint of RAM memory imposed by our need to create the most portable system, only one vector is kept in active memory at any one time. When the operator has reached the end of the vector he wishes to store, he selects either the 'open' or 'close' option from the data entry menu. The 'open' option accepts the vector as it is, whereas the 'close' option will include the start coordinates as the new end point and complete the vector to form a closed traverse. At this point, regardless of the option chosen, the number of points and colour, followed by the X and Y coordinates at each point are written to the layer file. During digitising a filter ensures that the same coordinate points are not stored more than once. An example of the first few lines of a stored vector in ASCII are presented in table 2.

The ability to 'flood-fill' a defined area has been included as it is quick and is often required in resource mapping. A standard vector as described above, with the 'close' option chosen, is utilised as the boundary of the area to be filled. A fill colour is chosen and a point within the relevant closed vector is specified using the input device. The 'flood-fill' command is intrinsic to the programming language. From a specified point it will fill the area within the closed vector in the desired fill colour. The typical ASCII representation of the flood fill is shown in table 3.

A further feature is a labelling system which allows the specification and positioning of a label at any point on the map. The size and orientation of the label are user defined. The ASCII codes used to describe the labels are given in table 4.

All of the above information can be stored on a single layer file or separated in different layers according to the operator's final display or analysis requirements. The facility to erase any of the above features is also included.

In order to provide a more detailed picture of the area being studied a window system has been included. If the screen map resolution is not acceptable to the operator he may request a window. The screen is then quartered and the operator may choose the quarter in which he would like to work. The program will then window in (zoom in) on this quarter, allowing it to fill the screen. The program will redraw the entire map, vector by vector and layer by layer. The user however, will only see the information displayed in the window. A section of each of the four adjacent quadrants will also be displayed to aid in edge matching (only in edit/create mode). This method is slow during regeneration but allows the system to store the data without having to determine change points at every window boundary.

Figure 2 depicts the screen format during the edit or creation of a vector map. The side panels show the actions of both digitiser menu and Function keys.

Raster

Raster information is depicted as a series of scanned lines of dots on a video monitor. With micro-computer systems, information on each dot (or pixel) can be stored directly in the video memory. This ability allows dot or patchwork information to be captured and displayed. A trade off for this ability to address every dot on the screen is the extra data storage required. To overcome the storing of large areas of non-vector information, a special vector method was employed which has been described above (see discussion on flood fill).

Four pieces of information are required for each pixel, namely; the active layer to which the raster data is to be assigned, the colour and the x and y coordinates of the pixel. For completeness, and not necessary efficiency the raster data is stored in the same file designation as the vector data. Whilst more than one colour may be defined for a layer, it is recommended that a different layer is used for each colour. The map segment is sub-divided into four sectors. The location of each pixel within each sector is represented by a 1 byte value, the high 8 bits contain the x-coordinate and the low 8 bits, the y-coordinate. Storage of the data in binary format is more efficient although the compatibility of the data is effected.

The raster type data is entered differently to that of the vectors. Firstly an area on the screen is selected by the user using a window. This area is then zoomed in. Each pixel within the zoomed area is represented on the screen by blocks of the corresponding colour to that of the pixel (this applies only to the layers that are active on the screen at the time of zooming in). The user can now select a new layer, a different colour and indicate those 'pixels' to be 'turned on'. The pixel information for the current layer is only stored by explicitly indicating the relevant softkey.

With this method each pixel coordinate may be defined on more than one layer, although only one colour will appear on the screen at any one time. This is important in the development of a system applicable to a wide range of applications.

INTEGRATION APPLICATIONS

The open ended structure of the utility allows the user to integrate his application in an effective manner. An application requiring only the entry and display of image information could use the system without recourse to further additions of software. The entry of data from a map for use with a simulation model would require the integration of the simulation algorithms. In the latter case, the output from the simulation could be represented in image form using the graphics utility.

Three examples of the application of the utility are presented together with schematics showing the interfacing with the application software.

Resource displays

Due to the increased awareness of the limits of our natural resources, it has become essential that a comprehensive, all-encompasing methodology be developed to assist the planning professions in water resource allocation. The graphics utility (tool) was employed to assist with the display of social, economic and demographic conditions and needs. This tool functions independently of units and has the ability to interpret a variety of data types.

The first step in the process of studying a particular region is the capturing of the physical details of the map segment. This follows the standard procedure for any digitising process, with all the required detail being captured on layers defined by the operator. Typical layers used on the background map for water resource analysis are roads, settlements, rivers, lakes, dams, and catchment boundaries.

The second step requires that the user set up a display file for each map segment. This can be generated by a separate simulation program or be carried out through an interactive option within the package. The required information and the format of the file are shown in Table 5.

The X-Y coordinates are specified in the editor through the use of the digitiser stylus or mouse. This allows for the exact positioning of the display information. The colours are the standard 16 IBM-compatible colours available on EGA screens. The scale is adjusted by the user to obtain the most acceptable display output.

It is now necessary to enter the display mode in order to examine the distribution of the resources and the associated needs. It is possible to define the display region by the specifying the bottom left and top right hand corners, or through the use of a defined basin. The defined basin contains the necessary information to generate a complete screen such as the scale and the corners as set out above. The package establishes which map segments are required to provide the full map coverage requested. These are then generated segment by segment on the screen. The facility to interactively, expand the map coverage by one segment along the top and the right is also included.

The displays can now be superimposed on the background physical map in the form of bar graphs. It is possible to specify which of the individual display items should be on the screen at any one time. It is feasible therefore to display the population densities and overlay the available water supply figures. This immediately points to the areas requiring an increase of investment in water supply orientated projects. It is then possible to remove the population densities and superimpose instead the ground water potential, to assess the possibility of using the aquifers as a potential source to satisfy any shortfalls.

Figure 3 shows a base map with bar graphs which represent the MAR (mean annual runoff) at the available gauging station for a specific basin. On the screen each graph representing a different resource would be assigned a different colour. Corresponding measures at the different locations would then be in the same colour.

It is easy to see the benefits derived through this graphical representation. Persons of various professions and educational background can, with equal ease interpret the size of the relative bars and therefore the relationship between the flows from the various sub-catchments, or the socio-economic influence exterted by different factors.

Mass balance

The determination of a water balance within a catchment gives an overall performance of that catchment and the interaction of the individual mass fluxes. Particular attention is needed to determine the effect of urbanisation of catchments and the resulting changes in the water balance equation. The demand for housing coupled, with the increased migration of the population to the towns and cities (in Southern Africa and other similar countries), has prompted the study of urban catchment water balances. The time scale of the water balance can vary from a daily to an annual cycle.

The use of the vector and raster graphic tool described in this article enables the spatial variation of the water balance to be studied. Various items of data were entered using the graphical input system; namely roads, catchment boundaries, rivers, soil types, land use and contours.

The land use component is stored using different levels, namely; houses, swimming pools and the various types of vegetation cover. This enables the overlaying of the different components for use in the simulation model.

A water balance module was developed, incorporating the graphics system, to analyse the monthly water balance. Inputs to the water balance were monthly time series of basic water balance parameters and basic simulation constants. These constants comprised parameters of infiltration and evaporation models which were related to the type of vegetation and soils.

Simple water balance calculations were performed for each pixel element of the catchment area using both the graphics information and the input data. A simplification, which was deemed justified in view of the time scale involved, is the absence of routing in the model. Three dimensional graphics was used to display the storage of water in each element of the catchment. A composite map of the water balance variation (stored as layers) for each month was also incorporated as a system call to the graphics routines. The total system is depicted in figure 4. A description of the application of the water balance system will be the subject of a separate article.

Geomorphological Aspects of catchments

With the development of hydrological simulation models becoming more physically-based (i.e. the parameters of the model can be determined by physical measurement) and with the need for spatial consideration of the parameters, it is necessary that certain parameters be determined from maps. Items such as slope, length of streams and area of the sub-catchments are the most obvious.

Models such as TOPMODEL (Beven and Kirkby, 1979), Soil Conservation Service method (S.C.S., 1964) and Unit hydrograph analysis, involve the entry of numerous topographic data values that can be derived from maps. The graphics sub-system can be used to enter the relevant map information and an appropriate application program added, comprising, the simulation model and a series of routines to interpret the map information.

The interaction of the graphics tool and the simulation and measurement systems are illustrated in figure 5.

DISCUSSIONS

Research which is being undertaken in several areas prompted the development of the computer graphics utility described. These research topics range from water resource planning (on a basin scale) to studies of the effects of urbanisation on the water balance (on a small catchment scale). Ease of use and the utility to implement the system on a 'standard' hardware configuration were of prime concern.

The sheer diversity of applications prompted the development of a mapping system, which although customised, would have general application. Present CAD systems are either not versatile enough when used for mapping or are cost prohibitive. The latter constraint governs when considering small research organisations or developing regions.

ACKNOWLEDGMENTS

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TABLE 1 - Available Scales and Map Segment Increment

File Code	Scale	Area	Increment
A	1:1,000,000	2° x 2°	2°
B	1: 500,000	1° x 1°	1°
C	1: 250,000	0,5° x 0,5°	0,5°
D	1: 50,000	0,1° x 0,1°	0,1°
-	1: 10,000	0,02° x 0,02°	0,02°

TABLE 2 - Data Storage Format for Vectors

NO. OF POINTS IN VECTOR

COLOUR NUMBER

COORDINATE SET [X,Y]

```
62,11
12,754,912,753,911,753,909,752,907,751,904,749,902,748,902,749,902,751,902,753
81,754,901,755,898,756,896,758,895,759,894,760,893,762,898,764,889,765,887,767
86,769,885,770,883,771,881,773,880,773,878,774,874,774,972,775,878,775,867,778
66,779,867,782,868,784,869,787,870,789,869,792,869,794,869,796,868,797,866,795
```

TABLE 3 - Data Storage Format for "Flood Fill"

'FLOOD FILL' OPERATION CODE

FILL COLOUR

BORDER COLOUR NUMBER

INTERNAL COORDINATE SET [X,Y]

```
9999,7
881,773,8
```

TABLE 4 - Data Storage Format For Labelling

LABEL OPERATION CODE

LABEL COLOUR

LABEL SIZE

LABEL

LABEL START POINT [X,Y]

```
5555,14
154,629,2,"Butterworth"
```

TABLE 5 Example of Data in Resource Display Files

VALUE	UNITS	DISPLAY ID	DESCRIPTION	COLOUR	X-COORDINATE	Y-COORDINATE	SCALE	ON/OFF	
417,	Mm^3,	T3M85,	Mean Annual Runoff,	12,	897,	958,	-1,	1	
759,	Mm^3,	T3M86,	Mean Annual Runoff,	12,	837,	768,	-1,	1	INDIVIDUAL DISPLAYS
99,	Mm^3,	T3M89,	Mean Annual Runoff,	12,	361,	928,	-1,	1	

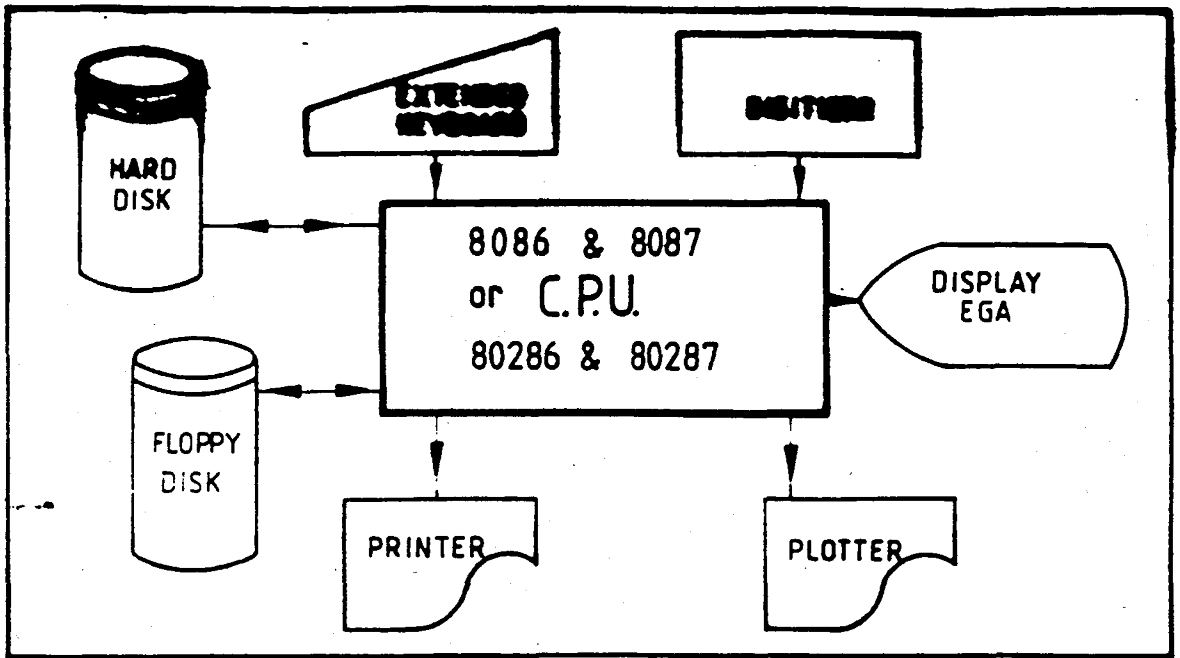

Figure 1 - Minimum Hardware Configuration

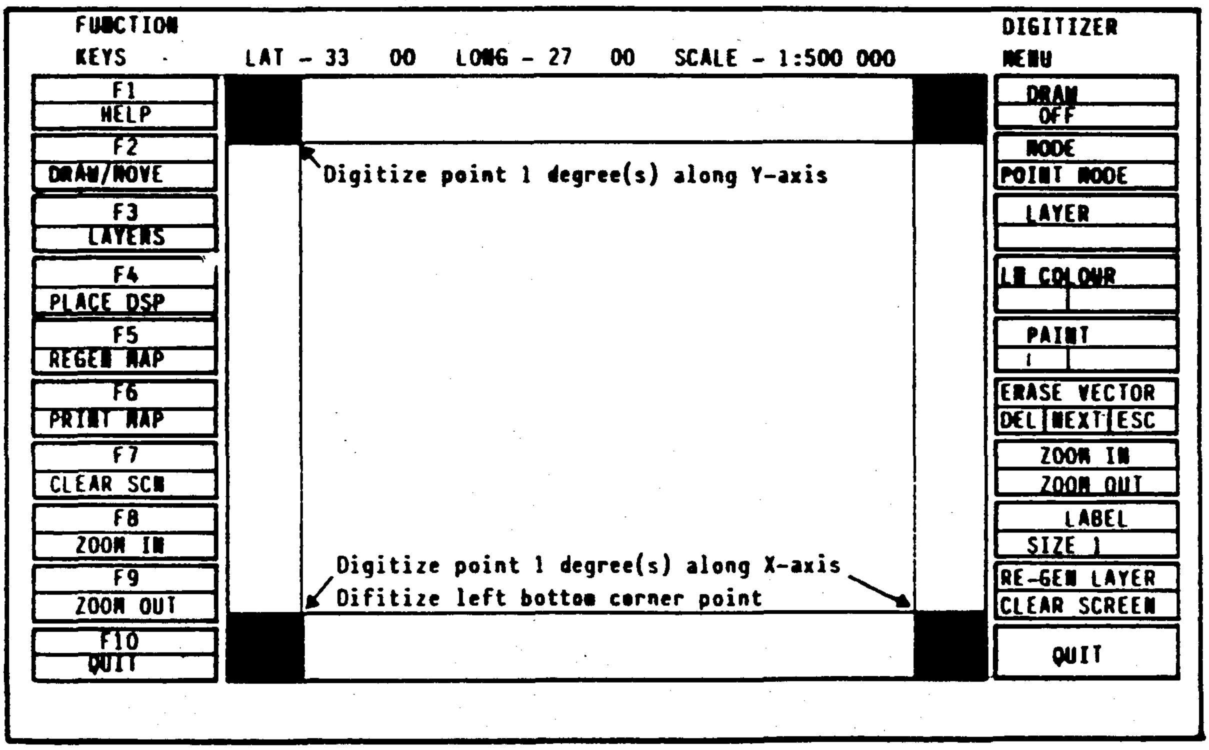

Figure 2 Illustration of editor screen during initialisation

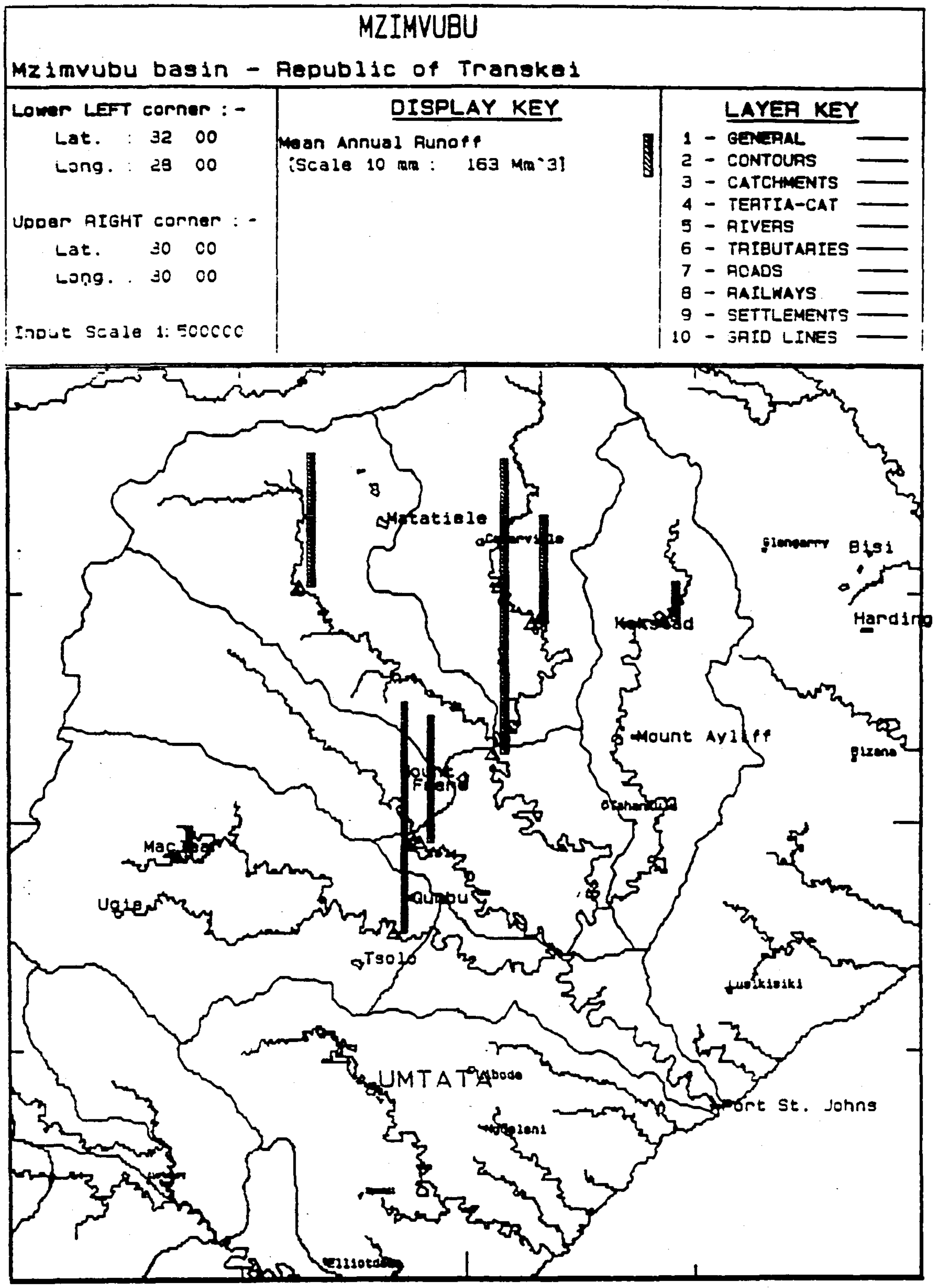

FIG.3 Example of Resource Manager Plotter Output for the Mzimvubu Basins, Republic of Transkei

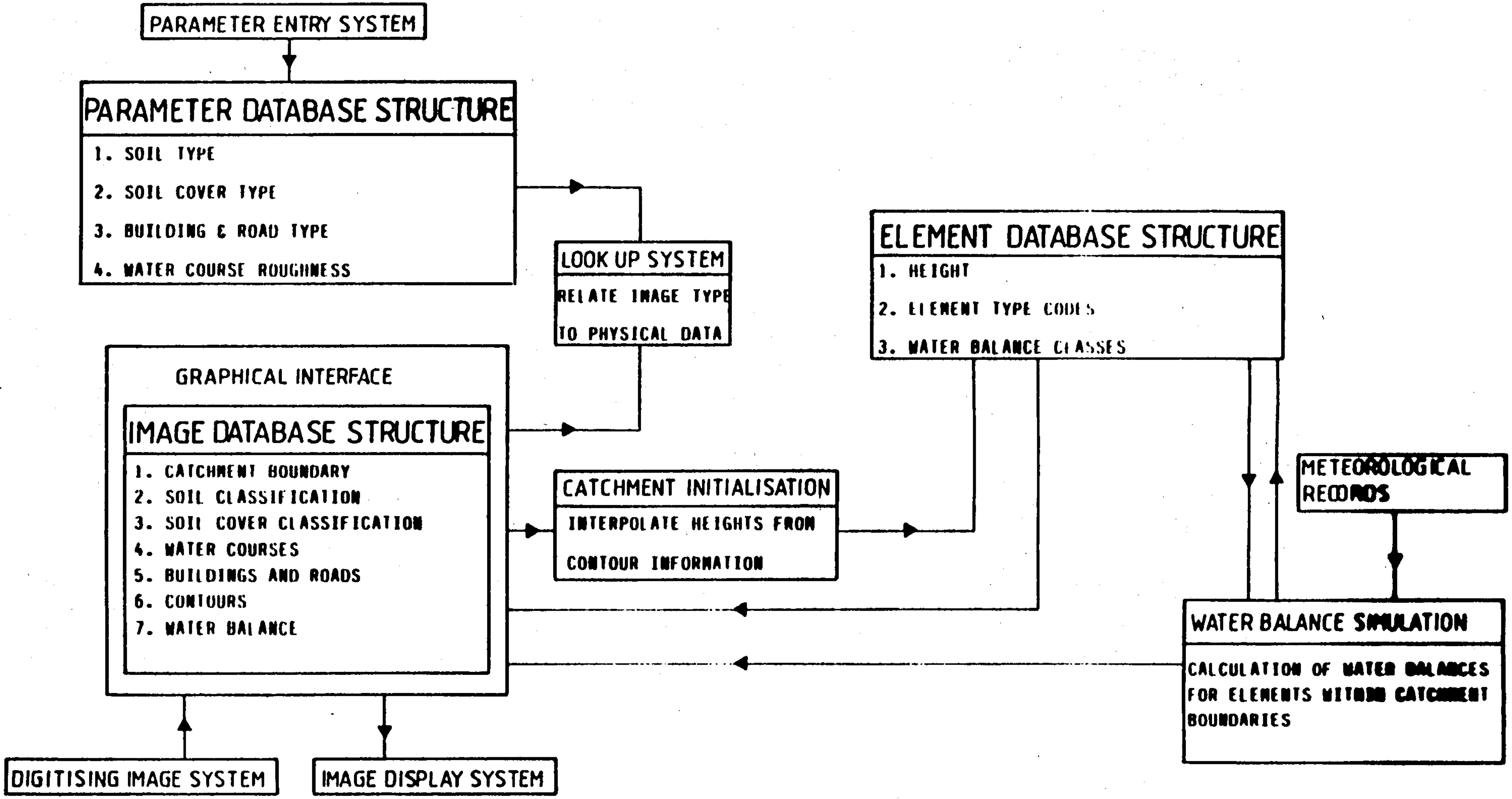

Fig. 4 WATER BALANCE SOFTWARE SCHEMATIC

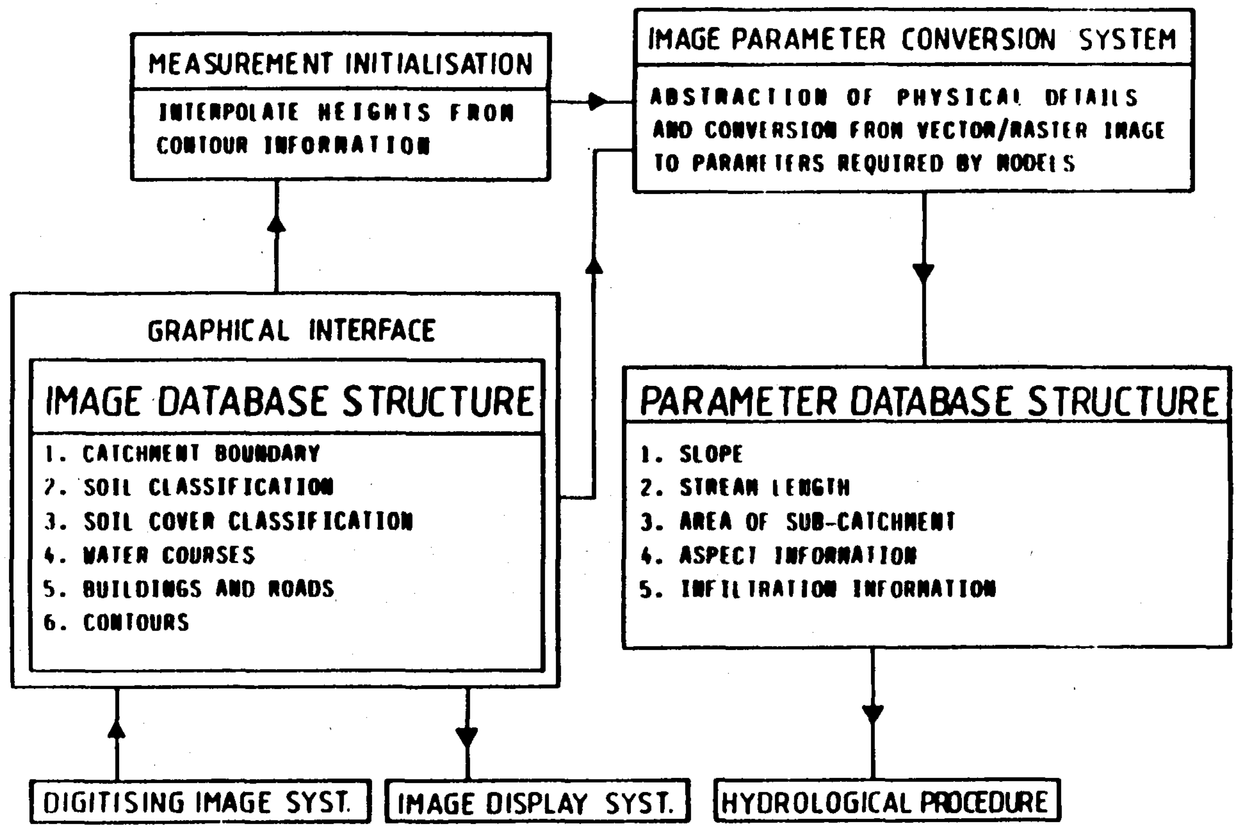

Fig 5 Software Schematic for Parameter Measurement Systems

PROBLEMS ENCOUNTERED IN HYDROLOGICAL DATA COLLECTION

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ABSTRACT

A critical appraisal of urban catchment monitoring systems has shown that although electronic data collection techniques generally result in better quality data than those obtained from mechanical recording systems, they are not infallible. A comparison of the problems encountered with both approaches to data collection is discussed.

1. INTRODUCTION

The Water Systems Research Group of the University of the Witwatersrand has for some years been involved in hydrological data collection programmes as part of ongoing research under contract to the Water Research Commission.

Precipitation, streamflow and to a lesser extent stormwater runoff quality data have been collected from two urbanized catchments since 1982, and since 1986 another urbanized catchment and a rural catchment have been included. The data are to be used for mass balance on a monthly scale as well as runoff modelling. In the latter case the time scale envisaged is of the order of minutes.

In the earlier studies use was made of chart recorders in conjunction with syphon-type or tipping-bucket raingauges and pressure-bubble water depth recorders. The more recent equipment installed includes electronic data logging instrumentation and pressure transducers for the measurement of water depth.

A number of problems have been experienced with the instrumentation, particularly with the older chart recorder equipment, these problems ranging from instrument malfunction to operator error. This paper outlines these problems and the manner in which they were addressed.

2. MONITORING EQUIPMENT

2.1 Montgomery Park catchment

The Montgomery Park catchment is situated in north-western Johannesburg and measures 1036 ha. Rainfall is measured by three tipping-bucket raingauges and two syphon-type raingauges, while water level at the catchment outlet is measured by means of a pressure-bubble depth recorder. In all cases the data are recorded on charts, four of which are drum charts and the remaining two are strip charts.

2.2 Hillbrow catchment

This is a very densely developed area measuring 67 ha. Up until recently (April, 1987), one syphon-type and three tipping-bucket raingauges measured precipitation and a pressure-bubble depth recorder measured the flow depth at the catchment outlet. The raingauges were linked to drum chart recorders and the depth recorder to a strip chart. The network has now been changed to tipping-bucket raingauges and a pressure transducer, all linked to electronic data loggers. pH and electrical conductivity sensors at the catchment outlet, also linked to a data logger, have been installed.

2.3 Sunninghill catchment

The Sunninghill catchment has recently been urbanized and measures 75 ha. Precipitation is measured by four tipping-bucket raingauges in conjunction with electronic data loggers. A pressure transducer, also linked to a data logger, measures the flow depth at the catchment outlet.

2.4 Waterval catchment

This catchment is entirely rural, measures 75 ha and is situated alongside the Sunninghill catchment, sharing a common water divide. Precipitation is measured by five tipping-bucket raingauges, one of which is common to the Sunninghill catchment and a pressure transducer measures the flow depth at the outlet, electronic data loggers being used in all cases. In addition to monitoring rainfall and runoff, a weather station has been established on the Waterval catchment. Sensors measuring wind speed, wind direction, solar radiation, temperature, barometric pressure, relative humidity and potential evaporation are all linked to a multi-channel data logger.

Table 1 lists the instrumentation currently in use on the four catchments.

TABLE 1 Instrumentation currently being used for rainfall-runoff monitoring.

Instrument description	No	Strip chart	Drum chart	EPROM
Casella 10mm syphon-type raingauge	1	1		
Lambrecht 10mm syphon-type raingauge	1	1		
Lambrecht 40mm syphon-type raingauge	1		1	
OSK-751 0,5mm tipping-bucket raingauge	4		4 (*)	
OSK-751 0,2mm tipping-bucket raingauge	9		2 (*)	7
MCS-160 0,2mm tipping-bucket raingauge	1			1
OTT 20. pressure-bubble depth recorder	2	2		
WIKA 881.14.600 pressure transducer	1 (**)			1
DRUCK PDCR.830 pressure transducer	2			2
MCS-120 single channel data logger	9			
MCS-120 twelve channel data logger	4			

(*) changed to EPROM recorder in April 1987
(**) replace by DRUCK

Figures 1 and 2 indicate the locations of the instrumentation on the Sunninghill and Waterval catchments.

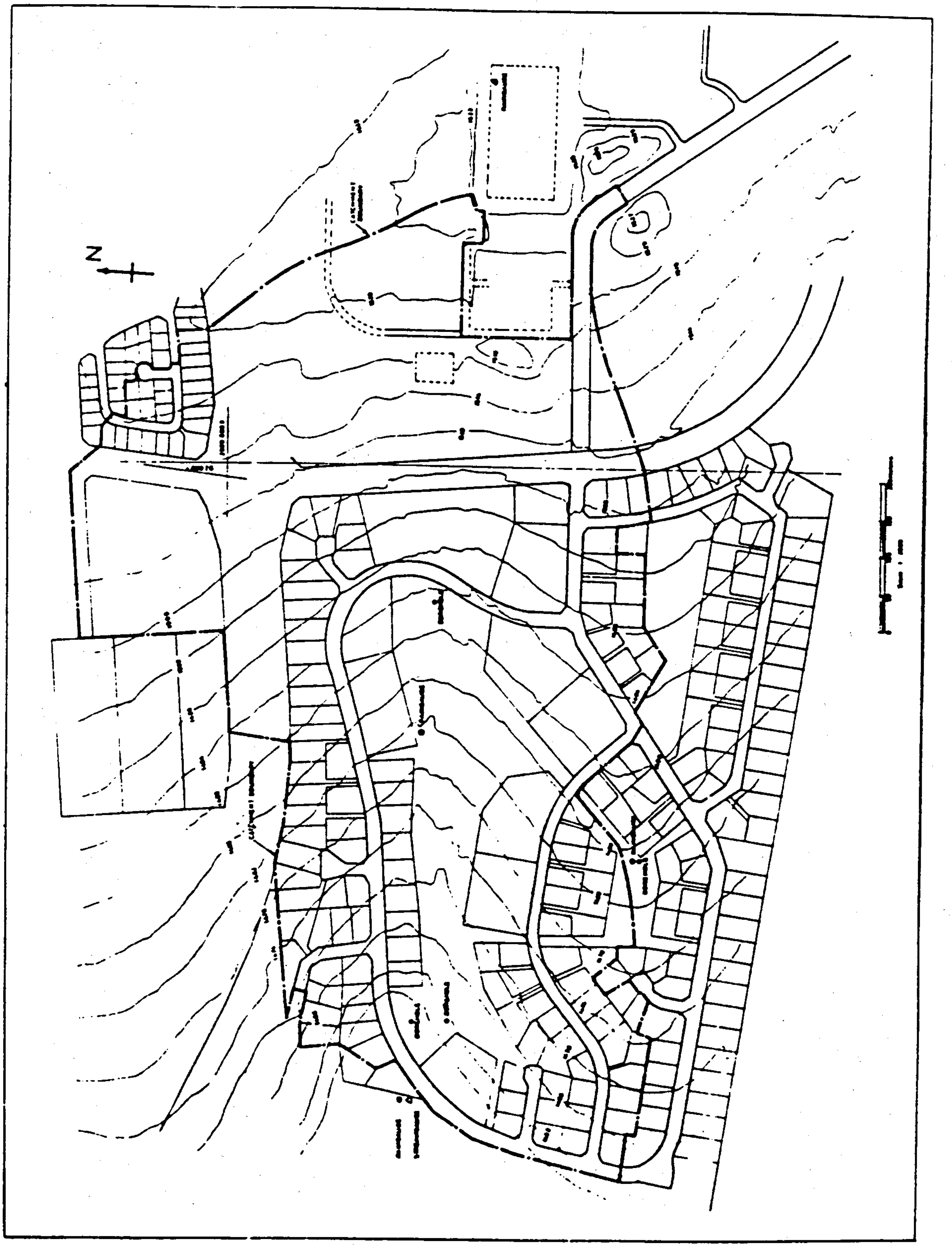

Fig. 1 Sunninghill Catchment

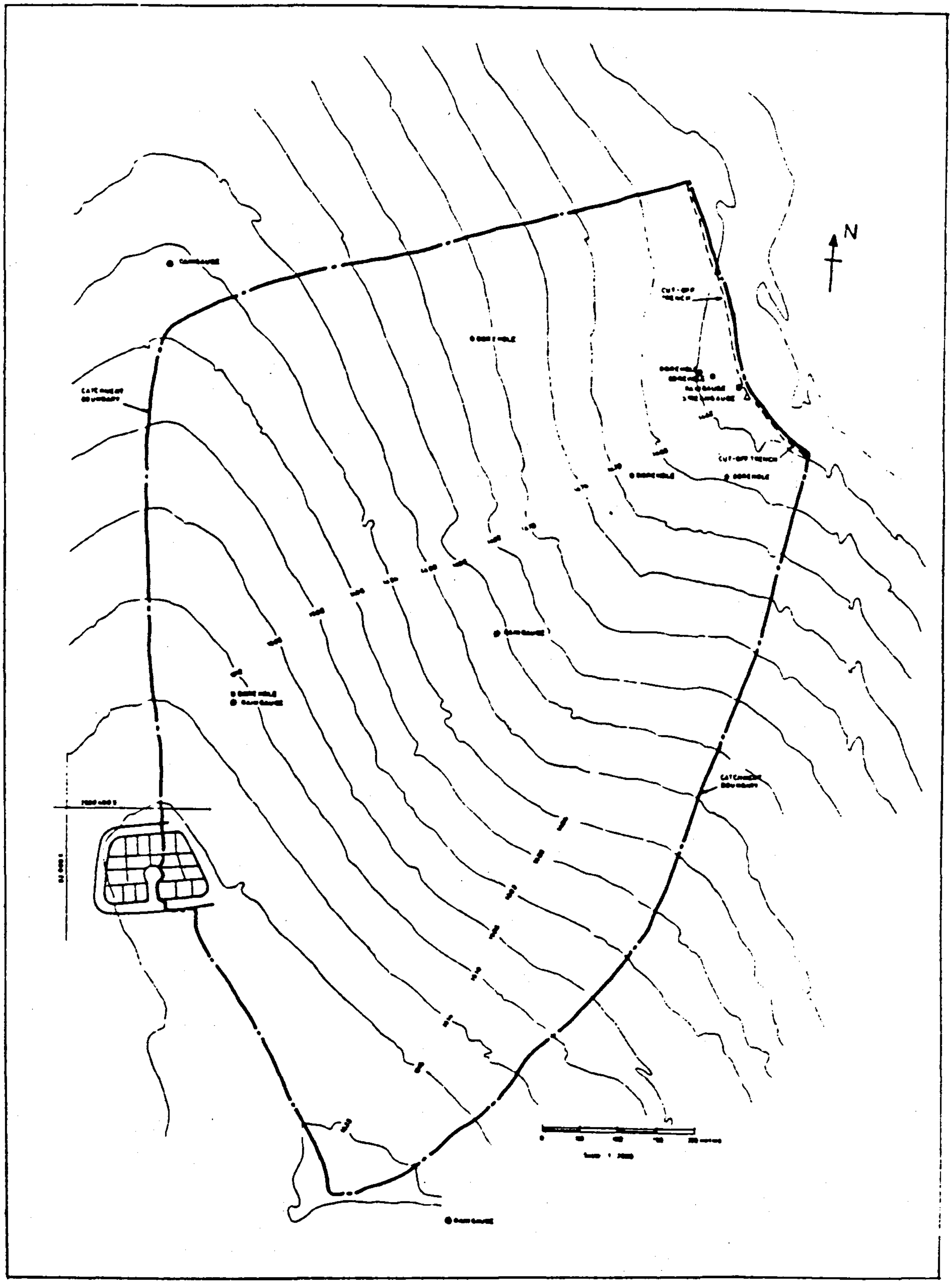

Fig. 2 Waterval Catchment

3. PROBLEMS ENCOUNTERED

3.1 Equipment maintenance

It was found that some of the equipment required excessive technical maintenance. This applies specifically to instrumentation with many moving parts, for example the pressure-bubble depth recorders. The clockwork apparatus on both the pressure-bubble depth recorders as well as on the drum chart recorders required continual maintenance, as did the lever arm mechanism on these drum chart recorders. Obtaining spare parts for the latter proved time consuming and costly. Workshop repairs of the pressure diaphragm in the bubble recorders also proved very expensive. Minor other technical problems occurred, including gas leaks in the pressure-bubble recorders, sticking of the float in the syphon-type raingauges and the blocking of the funnel outlet in the case of the raingauges. Such minor problems, while easy to rectify, nevertheless often resulted in the loss of potentially valuable data.

Hughes and Guthrie (1984) discuss some of the problems encountered in collecting rainfall data using syphon-type raingauges.

3.2 Power consumption

Owing to the nature of the various research projects and hence the location of the instrumentation, mains power could not be considered.

It was found that the power consumption of the pressure-bubble depth recorders was high, especially during times of fluctuating water level, resulting in additional power being required to drive the cog/belt mechanism ultimately moving the pen on the strip chart. Special batteries for the instrument were initially imported, but owing to the high cost involved, less satisfactory, but nevertheless functional, batteries were locally made up to suit the instrument specifications.

In the case of the tipping-bucket raingauges with chart recorders, two standard torch batteries lasting about one year are required. When the tipping-bucket raingauges are connected to a data logger, four such batteries are required and power consumption increases, being greater during periods of rainfall. If no rainfall is experienced during a 24-hour period only the time and date are written to the EPROM (Erasable Programmable Read Only Memory) every midnight. However, if more than 0,2 mm of rain falls causing the bucket to tip, then the number of tips that occur during the selected time resolution period are converted to rainfall depth and this depth and time written to the EPROM. What this means essentially is that logging of data only occurs during periods of rainfall. As the chief source of power consumption is the writing of the data to the EPROM, it can be seen that the batteries will have a longer life during periods of no rain than during the wetter months. It has been found that the batteries for a tipping-bucket/single channel data logger combination need replacing at approximately 6 week intervals during the rainy season.

In the case of the pressure transducers, however, much more power is required owing to the fact that the output from the transducers are continuous analogue signals which are logged at a selected time interval, irrespective of whether or not there has been any significant change in the magnitude of this signal. Unfortunately the MCS range of data loggers are not capable of logging only those data outside a specified range of values (hysteresis or alarm value logging). If such a facility were possible, as is the case with the OTT Algomatic and Grant Squirrel data loggers for instance, then only those data of interest would be recorded, with a consequent increase in battery life. Logging outside of pre-determined limits is illustrated in Figure 3.

Rechargable dry-cell batteries are used to power the twelve channel data loggers, and these need recharging at about two week intervals. One problem discovered relating to rechargable batteries is that over-discharging of rechargeable batteries will shorten recharging life and results in voltage fluctuations which provide sensitive transducers with poor excitation.

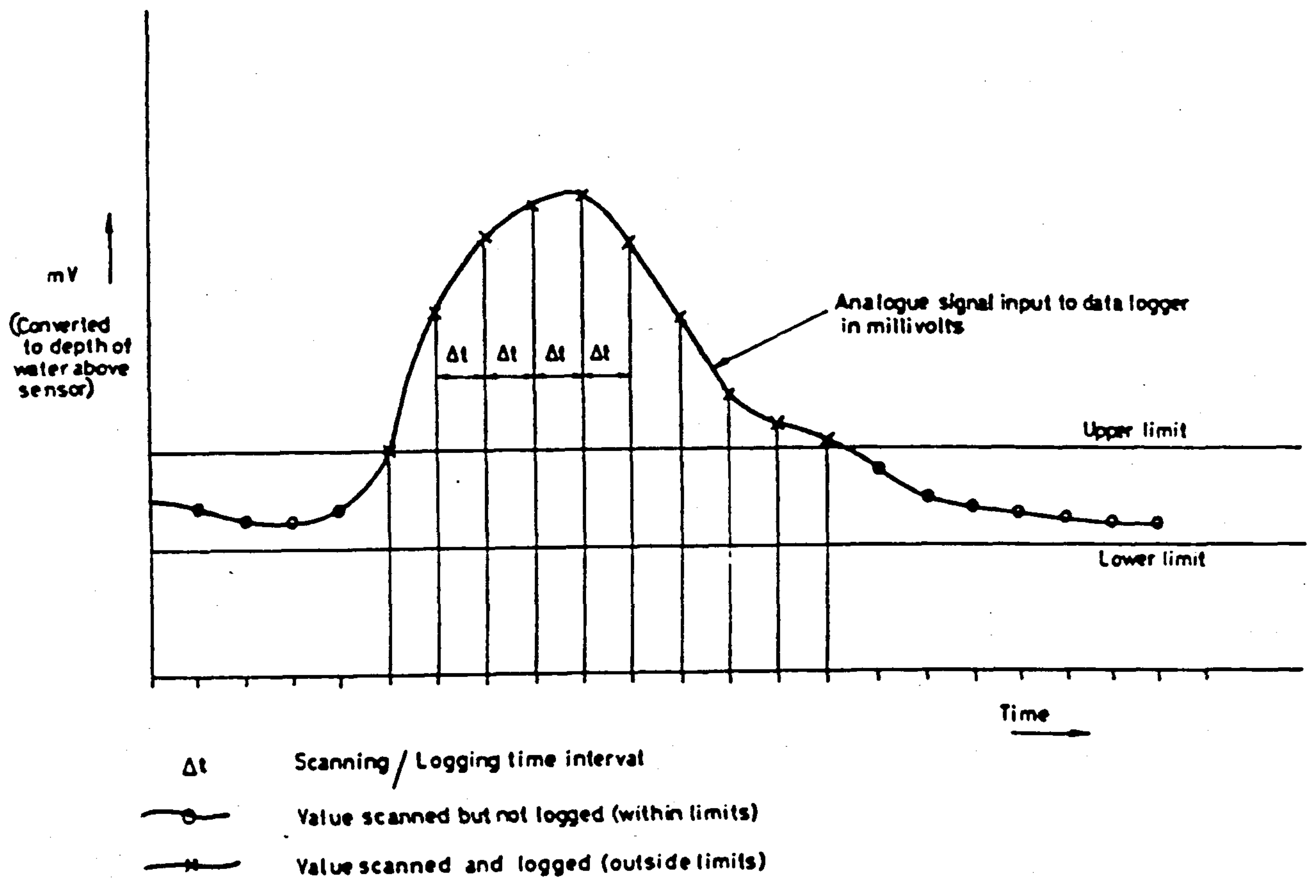

Figure 3 Logging outside of pre-determined limits.

In general, the number or volume, of batteries used with an electronic system is far greater than would be used with an equivalent chart system. This is due to the small band width of voltage that the CMOS circuitry in the data logger can handle.

While the batteries as such do not present a problem (except the cost factor in the case of the pressure bubble depth gauge), regular checking is required. It occurred on one occasion that the polarity of one of the torch batteries was reversed internally, and in this particular instance a week of valuable streamflow dat was lost.

3.3 EPROM capacity

Where a tipping-bucket raingauge is linked to a data logger, EPROM capacity does not generally represent a problem as data are only written to the EPROM during periods of rainfall.

At present the maximum single EPROM storage capacity is 16 Kilobytes (K), although 32K storage is possible with minor modification. A 16K EPROM can store 8000 data points, resulting in approximately 9 days of storage if date, time and water depth are recorded every 5 minutes. Once again, if logging were possible outside a specified range, only data of interest would be collected and a 16K EPROM would last longer than 9 days, the time to full capacity being determined by the degree of fluctuation of water levels in the stream. An added benefit would be that the resolution time step could be decreased, should this be required. For example the Sunninghill catchment is reasonably steep with a main stream length of less than one kilometre, resulting in short response times. Thus a resolution time step of one minute is more appropriate than the currently used five minutes. If however a time step of one minute were adopted with continuous logging, the EPROM would reach full capacity after two days.

3.4 Familiarization and training

A high level of technical expertise is generally not required for replacing charts, EPROMS, and batteries and re-filling the gas cylinders in the case of the pressure-bubble depth recorders. Minor maintenance and repairs can generally be carried out in the field for the chart recorder related instrumentation, although workshop repairs to the pressure-bubble depth recorders have been necessitated on more than one occasion.

A technician with artisan training was initially employed for maintenance and data gathering, but after some time a lack of interest became evident. On the other hand use of students has proved of value to both the students and to the programme, as data collected is often used by the students in individual research projects.

Processing the data however is another matter. Preparing charts for digitizing presented a problem in that it was found extremely difficult to isolate the start time of a storm event recorded on charts. This is particularly so in the case of the drum charts that undergo approximately 90% of one revolution every 24 hours. Consequently, a horizontal off-set of a particular length is generated every 24 hours on the chart, unless the chart is changed on a daily basis. Therefore to isolate the starting time of a rainfall event, one must first know the day on which that event occurred relative to the day on which the chart was installed, calculate the accumulated off-set and then adjust the time indicated on the chart accordingly. This procedure becomes very complicated when a number of storm events are recorded on the same chart. It was also found that the gradations on the strip charts did not correspond to the clock speed, even with adjustment, so event time required actual measurement of chart length from the point when the chart was installed. This procedure, although straightforward, becomes very tedious. Regarding the actual digitizing itself, the assertions of Dent and Schulze (1985) were confirmed, viz. that digitizing is a tedious occupation, leading to reduced productivity and high staff turnover.

The whole concept of electronic collection and storage of data is different from that relating to the use of chart recorders, requiring re-training of personnel in the operation and maintenance of the equipment. While the operation of electronic data logging equipment may seem more difficult in the initial stages, it becomes evident after a short while that this equipment is no more complicated to operate as long as the instructions are followed. It is considered that setting the parameters (e.g. site identification, date, time etc.) and removing and replacing EPROMs and batteries on the data loggers is as straightforward as replacing charts. Downloading the data stored on the EPROMs is far simpler and quicker than digitizing charts and is also not prone to error.

3.5 Software development

Data collection on either charts or EPROMs requires reasonably sophisticated software to abstract the data from these media. Dent and Schulze (1985) give some historical background to digitizing and describe a system developed by the University of Natal for this purpose. Lambourne (1987) has developed a data management system which includes a facility for downloading data from EPROMs.

It should be borne in mind by those embarking on a hydrological data collection program that the development of the necessary software can be time consuming and expensive if existing packages are unsuitable for any reason.

3.6 Synchronization on chart recorders

One of the most important drawbacks of chart recorders is the unrealiability of ascertaining the time of any particular event recorded on the chart. This could stem from the complicated nature of determining this time, as outlined in sectin 3.4, or more likely from variation in clock speeds and operator error.

For example, three different event times differing by as much as 20 minutes were recorded by three different raingauges in Hillbrow for the same event (Green and Stephenson, 1986). Spatial variation and storm movement could possibly account for a few minutes difference recorded on the different instruments, in this case these instruments being spaced only a few hundred metres apart, but it is considered that this difference is due to a synchronization error, as the hyetograph pattern recorded by the three raingauges was identical in all cases.

Similar synchronization problems were experienced with the water quality instrumentation, when it was found extremely difficult to correlate flowrate times (from the pressure-bubble depth recorder) with the times indicated on the conductivity recorder chart (Green et al., 1986).

Synchronization errors and the problems resulting therefrom have also been experienced by Haan (1975), Cousens and Burney (1976), Marsalek (1979), and Constantinides (1982).

3.7 Vandalism

Vandalism has and apparently always will be a problem in data collection programs of this nature. Incidents such as a severed pressure-bubble tube and a damaged raingauge with a brick in the funnel have been experienced.

Every effort should therefore be made to protect instrumentation where it is in the public eye, or preferably to site instrumentation where it either cannot readily be seen or where there is restricted access. Even this will not guarantee complete security as was evidenced by the fact that on one occasion the clockwork mechanism was stolen from one of the raingauges that was sited in a restricted access area in a school.

3.8 Environmental problems

An interesting problem that was encountered was the pickup of radio frequency and interference from ESCOM transmission lines between the data logger and the sensor. This was especially noticeable with the pressure transducers. The problem was minimised by adequate earthing and screening of cables, and providing the shortest cable necessary. This type of noise can be site dependant.

4. CONCLUSIONS

Hydrological data collection and data management requires sophisticated instrumentation and computer software if high quality data is expected. As Dent and Schulze (1985) correctly state, many design decisions resulting in expenditure of millions of rands are made on the basis of published hydrological data. Bearing these sentiments in mind, it seems pointless collecting hydrological data unless these data are of good quality.

Apart from the high costs of obtaining monitoring equipment, considerable expense is incurred in maintaining the equipment in good order, as well as in the actual operation of the system itself. Nevertheless it is considered that these costs can be justified when the end uses of the data are considered.

In various data collection programmes over the past few years, the Water Systems Research Group has gained much experience in this regard. Mistakes were made that possibly would not have been made had this sort of experience been at hand at the time. It is hoped that the issues discussed in this paper will be of some benefit to those embarking on a data collection program as well as to those organizations currently involved in hydrological data collection.

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LAUSANNE 1987

FOURTH INTERNATIONAL CONFERENCE ON URBAN STORM DRAINAGE

URBAN CATCHMENT HYDROLOGICAL DATA MANAGEMENT SYSTEM (WITDMS)

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SUMMARY: A hydrological data management system for catchments was developed which is cost effective in terms of manpower and equipment. The system involves the use of a micro-computer with peripherals for digitising, EPROM (Erasable programmable read only memory) downloading, storage retrieval and archiving of data.

INTRODUCTION

The Water Systems Research Group has been involved in the field of urban hydrology for several years. Previous research has used autographic systems coupled to digitising programs which stored the information as event records for later retrieval (Stephenson et al., 1986). Current research involves the effects of urbanisation on catchment water balance which requires both event and continuous data (i.e. rainfall, runoff, soil moisture, and groundwater levels). Experience has shown the autographic systems are prone to mechanical breakdowns and require a semi-skilled operator to digitise the charts with a high degree of accuracy. To this end solid state data logger systems were introduced in which the data is downloaded to a computer or printer.

Most data management systems are based on database software on mainframe computers (e.g. Boorman, 1986). However, with the development of micro-computer based storage media (hard disks and programmable tape streamers) the cost effectiveness of micro-computers proved their worth. It is true that with remote sensing programmes, the micro-computer will not be suitable for that application at present. The hardware configuration chosen is shown in figure 1. The system is based around an IBM PC compatible micro-computer, since it appears that this is very much the industry standard. The software was written in a compiler BASIC language, the reason being that, in spite of deficiencies of the language, it is used by hydrologists and engineers for programming applications.

SYSTEM SOFTWARE DESCRIPTION

Entry to the management system is through a simple password control system that controls status flags. The main menu comprises the following options.

1. Online help information
2. Input of Raw Data from keyboard, digitiser, or EPROMS
3. Process Raw Data and load into database
4. Retrieve data from database
5. Input or change recording station information
6. Change Computer System Configuration

Exiting for the system returns the user to the MS-DOS command level which allows further packages to be loaded (i.e. simulation models).

Each station in the data gathering network has certain features such as the position of the gauge (a unique identifier comprising a alpha character and seven digits), type of gauge, the range of output, the chart speed (for chart inputs), date of inception etc.. This information is accessed by the various subsystems without further user input.

The input subsystem allows input of data from digital or analogue sources with full screen data editing of the raw datafiles. EPROM data is directly downloaded under software control from the EPROM reader via a RS232 interface to the computer. Analogue data (in the form of charts) is entered through digitising of the breakpoints of the chart trace. The current trace frame together with the digitiser is produced on the screen as a mimic. A series of softkeys on the digitising platen allow the user to indicate observations that will be needed in the processing of the chart data (i.e. missing trace). The software is written such that several charts from different stations can be digitised in any order.

Once the obvious errors in the input files have been detected and corrected, the files can be processed to yield data in a suitable format for inclusion in the indexed database system.

The main objective of a data management system is to enable data to be stored on computer media such that this information is available for use in hydrological applications. Within the system, the data can be retrieved for use in specific simulation programs or for statistical analysis. The retrieval options are listed as follows:

1. Station information table printout
2. Streamgauge site rating table printout
3. Daily or monthly totals table printout
4. Depth-Duration-Frequency analysis
5. Mass curve analysis
6. Graphical output of daily, monthly or event information
7. Data abstraction for input to other models/software

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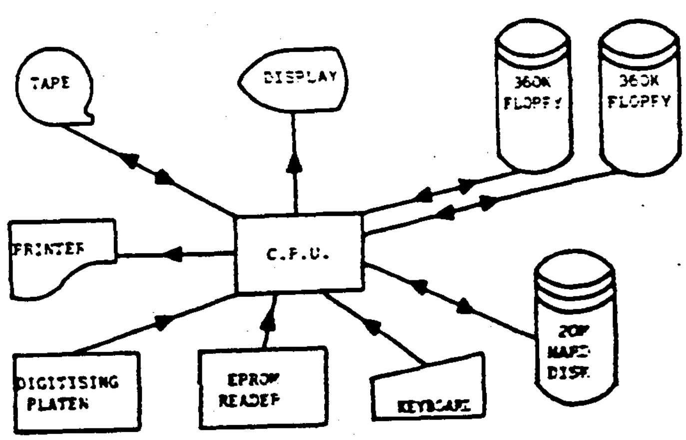

Figure 1. Hardware Configuration for WITDMS

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COMPARISON OF HYDROLOGICAL MASS BALANCE FOR AN UNDEVELOPED AND A SUB-URBANIZED CATCHMENT

by

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SUMMARY

Two topographically similar adjacent catchments near Johannesburg, one sub-urban, the other natural grassland, were monitored over a 5 year period to detect differences in runoff and hydrological balance. A network of raingauges, boreholes, flow gauges and water meters was installed. Evapotranspiration was modelled using observed weather data. Groundwater was estimated from tracer and other borehole tests.

Surface runoff from the undeveloped and from the sub-urbanized catchments was 4% and 15% of rainfall respectively. Evapotranspiration was 63% of rainfall for both catchments. Sewage outflow was 83% of water consumption for the developed catchment. No noticeable change in water table level occurred, and garden watering probably accounted for some of the high evaporation and water tables in the sub-urban catchment, although water supply was only 16% of the precipitation on the catchment.

INTRODUCTION

It is generally accepted that urban development changes runoff and hence water balance in catchments (see e.g. Leopold, 1968). It could be anticipated that flood runoff would increase and that as a result the catchment would become drier so that ecological balance within and beyond the catchment would be affected.

The effects of urbanization on catchment water balance results in the following :

i) Increased stormwater runoff and therefore larger structures and higher costs
ii) Recession of the water table and reduction in groundwater content in urban catchments as a result of increased runoff. This in turn could result in higher infiltration during low intensity rain and the result will be greater extremes between storm runoff and dry weather runoff.
iii) The effect on receiving waters in having to cope with higher rates of inflow in times of flood but also lower inflow in times of drought. These in turn affect storage requirements and the yields of catchments.
iv) The factors affecting the water balance are complicated by the import of potable water for domestic purposes and the discharge of sewage.
v) The construction of pavements and buildings will speed up the runoff process, affecting smoothness and reducing permeability and changing vegetation cover of the catchment. The effects of these are generally to intensify storm runoff but the separation of each component is difficult and a computer model was used to distinguish between different effects.

A research project sponsored by the Water Research Commission (reported by Water Systems Research Group, 1991) was established to evaluate the effects of urbanization on catchment water resources. The changes in runoff and in catchment water balance were to be quantified and the results transposed to other catchments.

The primary effects considered were total water runoff and loss from the catchments, with assessment of both flood runoff and drought runoff. A mass balance within the catchment assessed groundwater and soil moisture variations related to catchment cover and how these affect runoff. Secondary factors considered were flood volumes and peaks, water levels in streams, and the variations in ground water levels due to abstractions.

To make these assessments, the effect of impermeable covers such as roads and buildings was investigated. The construction of drains changes hydraulic resistances, gradients and generally results in rapid runoff. At the same time, water reticulation, and sewage runoff and the effect on catchment water balance has to be considered.

METHODOLOGY

Two catchments, one of which was natural grassland and the other sub-urbanized were compared. The catchments were adjacent and of similar size (75ha), shape and slope. Observations were made over a period of five years; it is expected that any variation due to isolated storms would be averaged out in this period so that only the effects of urbanization would manifest themselves.

Ideally, a paired catchment experiment should be undertaken where the two catchments are gauged prior to urbanisation of one of them. However, this approach was not practiceable as the time scale would have involved decades (always assuming that one of the catchments was likely to be zoned for residential development). Hence two catchments were chosen, one of which was already sub-urbanized to a great degree. These sites are situated on the granitic dome between Pretoria and Johannesburg.

Urban catchment water balances differ from those of rural or undeveloped catchments in that in addition to the rainfall-runoff system, there is usually a water supply reticulation and water disposal system i.e. pipes, gutters and sewers. This superimposed reticulation comprises two sub-systems. The first is a 'closed' system of water piped in and out of buildings (for drinking and water-borne sanitary, industrial and cooling purposes). The second is an 'open' system i.e. piped water for irrigation, swimming pools etc. Leakage from the piped water supply would be categorised in this latter system. The interactions of the different components of the water balance are presented in Figure 1.

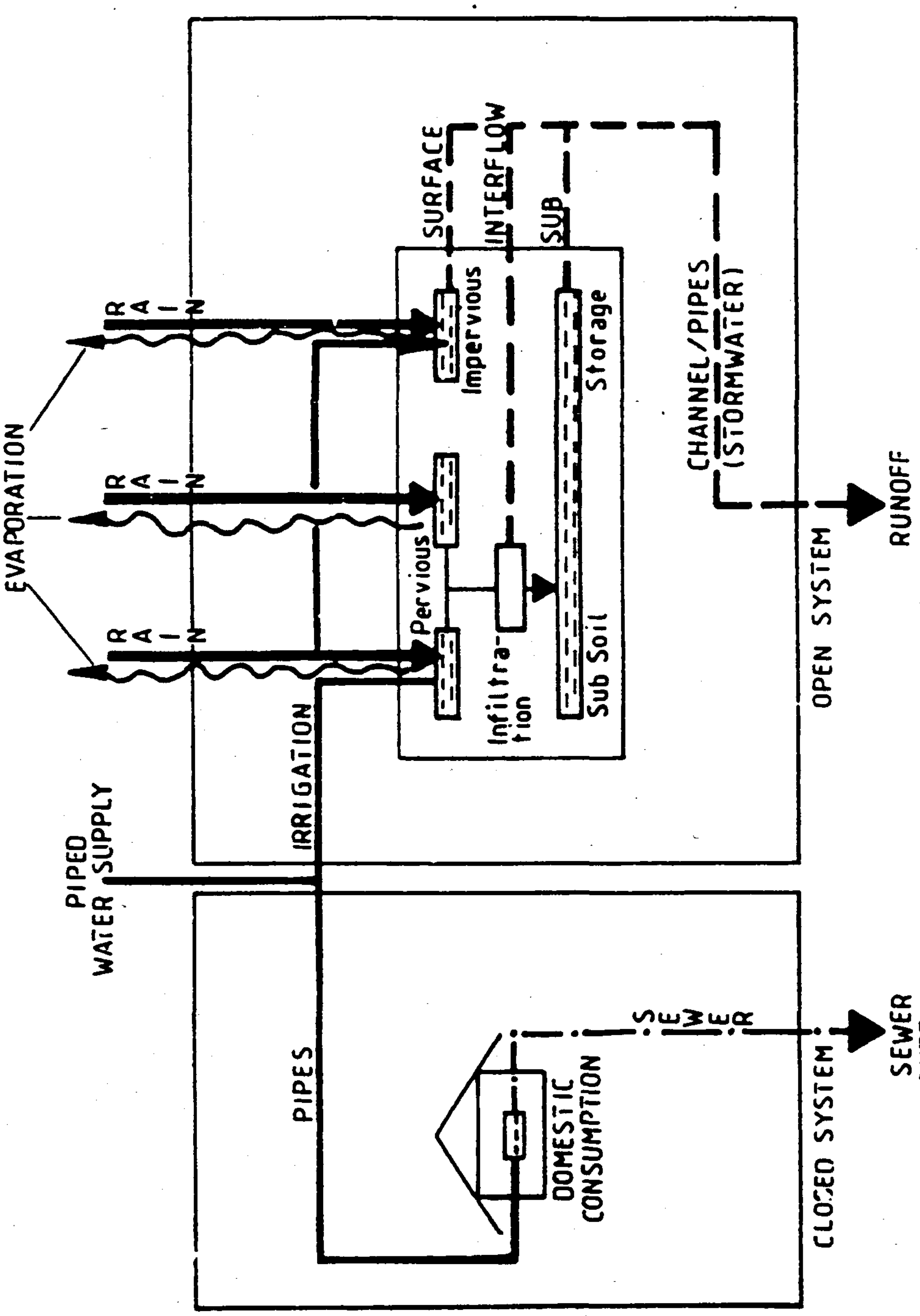


Fig. 1 Water balance components

Information on urban catchment water balance investigations is rather sparse in the literature. Grimmond and Oke (1986) developed a daily water balance model to calculate the daily, monthly and yearly water balance components, applied to a sub-urban district of Vancouver, Canada, comprising an area of 21ha. Table 1 lists the studies and the annual water balance percentages related to each component. Other studies e.g. Niemczynowicz and Falk, 1981, concentrated on surface water budgets for pollutant load calculations.

Obviously direct comparison of the results from these studies is not straightforward. The reasons for this include geographic differences between the locations (e.g. climate, physiography, botany, soils, and differing patterns of urban development, social customs and type of industrial development) as well as the techniques used to estimate or measure the components of the water balance. The piped water supply percentage also varies from 14 to 20 percent, which can be related to the division of water between industrial and domestic users and, in older reticulation networks, leakage from pipes.

It is hoped to investigate the urbanization over time of a third adjacent catchment so that the research will eventually be three-cornered; this will cover the possibility that the two catchments were dissimilar in factors other than urbanization.

Table 1 - List of Annual Water Balance Studies in Urban Areas

Author	Place	Area (km2)	R %	I %	W %	E %	r %	w %
Lindh (1978)	Sweden	4024	75	25	-	38	62	0
Campbell (1982)	Mexico City	?	86	14	-	71	29	0
Aston (1977)	Hong Kong	1046	58	40	2	34	66	0
Bell (1972)	Sydney	1035	77	22	1	49	51	0
L'vovich & Chernogayeva (1977)	Moscow	879	100	_	-	57	43	0
Grimmond & Oke (1986)	Vancouver	0,21	68	32	-	32	68	0
Stephenson and Lambourne (1992)	Johannesburg	0,75	86	14	-	55	13	26

R : Rainfall I : Water Supply E : Total Evaporation r : runoff
W : Groundwater in w : to groundwater

CATCHMENT DESCRIPTION

The urban catchment comprises part of Sunninghill, Sandton, with a surface area of approximately 75ha. The catchment slopes from east to west with a fall of 50m over a distance of one kilometre. A well defined watercourse flows through a park area in the centre of the catchment. The suburb was developed from scratch in the 1980s, and by 1985, when monitoring commenced, was 70% built up.

The area monitored has been zoned for residential development with provision for 130 stands. Of the 130 stands, eleven were earmarked for commercial and townhouse development. Stands for residential usage are of the order of 1500m^2 in size. The estimated impervious area on each of the housing developments is 25%,but this is mostly (70% of the 25%) unconnected to drainage, so the effective impervious area is significantly less.

Within the catchment, there is a two-storey office block, shopping centre and a garage with associated parking facilities. These commercial areas have greater areas of paving than the housing developments with an estimated 75 to 90% impervious area. The roads are tarred and a piped stormwater and water supply network exists. Most of the residential areas are walled so the surface runoff is concentrated along driveways or at low points. A pubic service office complex is situated at the top of the catchment.

Runoff during dry periods results from seepage from soil and groundwater systems at an impermeable barrier (dyke) at the bottom end of the catchment. Sewage is transported through a separate system of pipes and forms a closed system.

Within the urbanised catchment, the piped water supply to each property is monitored by municipal meters, read once every three months (which hinders accuracy). Sewerage from the same catchment is transported through a single pipe at the outlet. This quantity is measured using an ultrasonic device suspended in a manhole. The depth of fluid in the pipe enables an estimate of flow volumes to be made from a rating table.

An undeveloped catchment adjacent to the Sunninghill site which is part of Waterval Farm was used for the comparison. This catchment is without piped water supply, sewerage or stormwater networks and is 76 hectares in area.

Cattle grazing on the land has resulted in 'cattle tracks', compression of the soil surface layers and alteration of the natural vegetation cover. Overland flow tends to follow these 'cattle tracks' and does not therefore produce sheet flow. Sparse trees mainly of the blue gum variety, cover the southern side of the catchment. The catchment slopes from west to east with a fall of 50m over approximately 800m. There is no defined watercourse in the catchment.

From a geological point of view the catchments are situated on a granitic dome. A very thin top soil overlays several metres of decomposed granite. Groundwater is located in the decomposed granitic layers and fissures within the parent rock.

Instrumentation of the catchments comprises a network of continuous recording gauges and discrete measurements of other variables. A plan of the catchments and of the sites of the instrumentation is presented in Figure 2. Figure 3 shows the development of Sunninghill by 1990. Rainfall is measured using 0.2mm tipping bucket instruments at eight sites within the two catchments.

Runoff (combined surface and baseflow) is recorded at the outlet of both catchments. In Sunninghill, a V-notch Crump weir together with a stilling well and a pressure transducer were used. In the Waterval farm catchment, a V-notch plate weir together with specially designed cut-off channels measures surface runoff.

Boreholes were drilled in both catchments (see Fig. 2) to assess the geology of the area and to measure on a weekly basis, the 'water levels' which occur in penetrated fissures. Unfortunately, in the fissured rock, traditional methods of borehole assessment of groundwater level do not necessarily apply.

A weather station is sited near the northern end of the Waterval catchment and measures wind speed, wind direction, temperature, relative humidity, atmospheric pressure and total radiation (including both direct and diffuse).

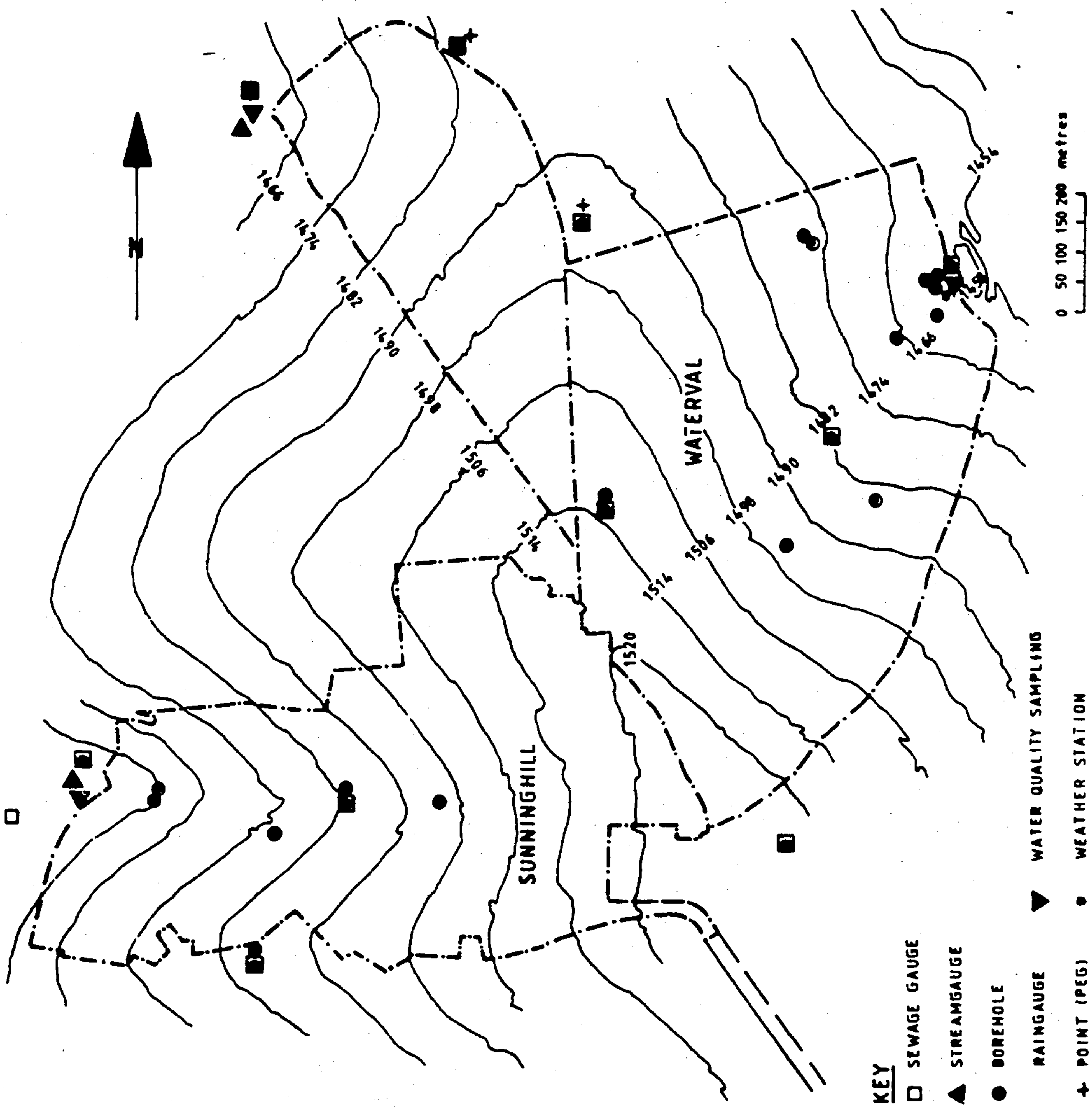
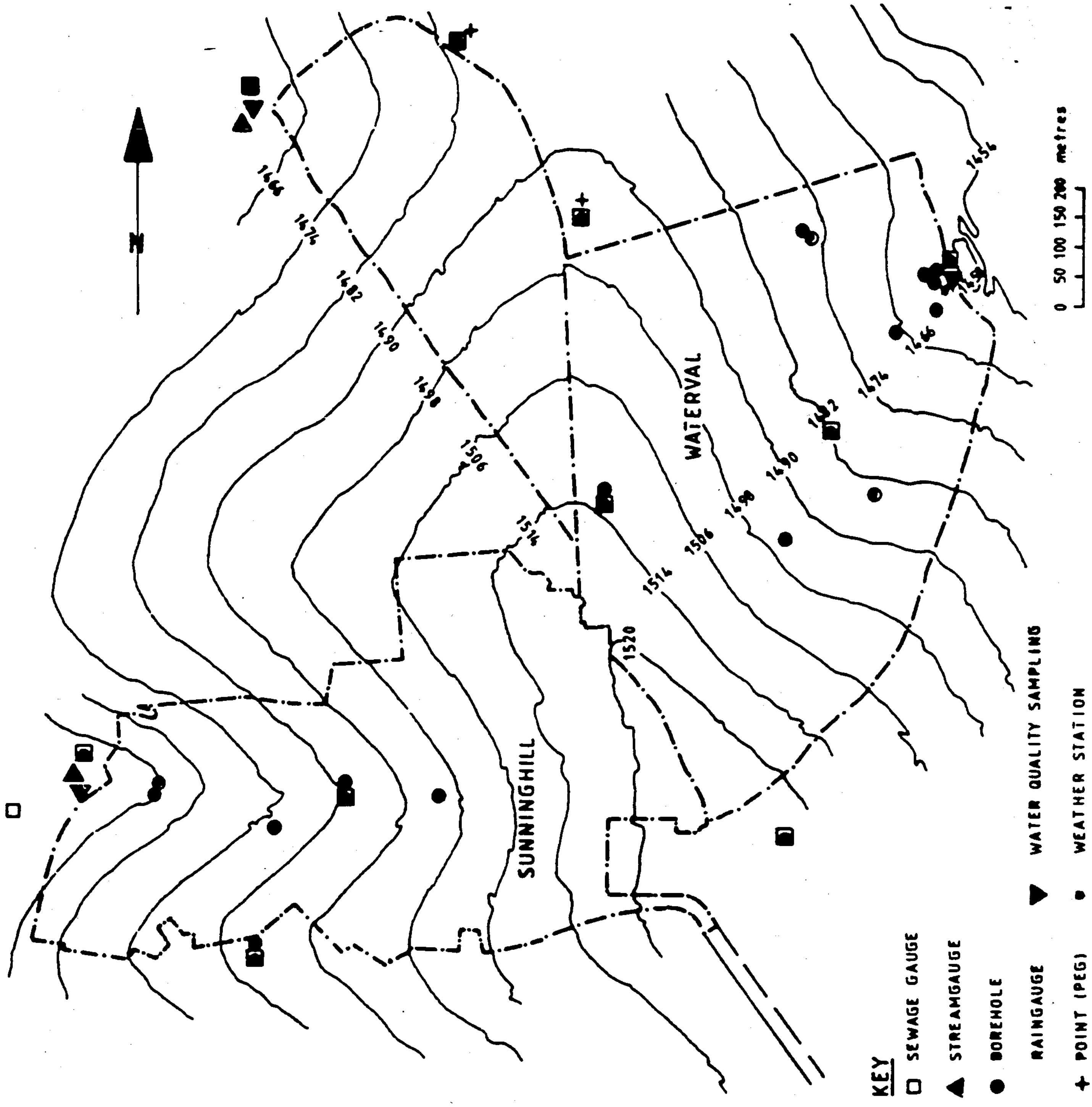


Fig. 2   Contour plan of the two catchments



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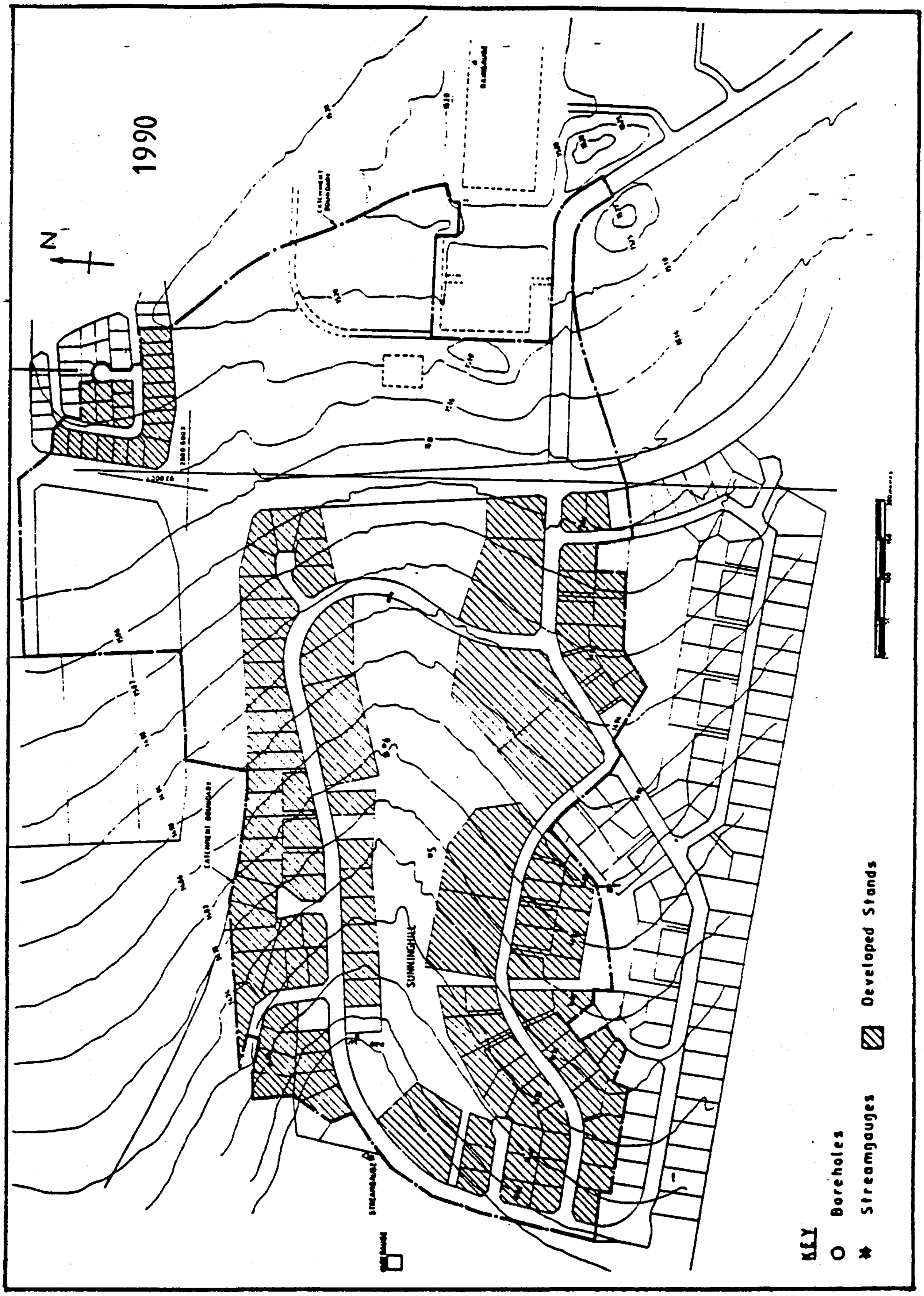

Fig. 3 Development of Sunninghill catchment, 1990

Daily rainfall was plotted for both the catchments for the purpose of the water balance. Two graphs depicting the rainfall for the Waterval and Sunninghill catchments are presented in figures 4 and 5 respectively. The overall variation of the precipitation amounts in both catchments is minimal. In fact, for storm event over 5mm, across the catchments, the variation in rainfall amounts to only 10 to 15%. This figure is well within the variation that would be expected due to measurement by small area collectors. Falls of rain of less than 5mm results in a far wider range of values between gauges; this indicative of isolated "spotty" events (Patrick and Stephenson, 1990).

There is a significant trend in the rainfall amounts from year to year. The first seasons (1986/1987) followed a drought period in the early 1980s. During the 1987-1990 period the rainfall reduced . For the hydrological years, October to September, the reduction of rainfall from 1987/88 to 1988/89 is between 7 and 10%, and from 1988/89 to 1989/90 is approximately 6%. Table 2 and Table 3 illustrate numerically the variation of annual rainfall records. Throughout the period the number of rainfall days remained fairly constant. This indicates that in below-average rainfall years, there is a greater variation between catchments owing to more spotty rainfall.

Table 2 - Statistics of Rainfall (Sunninghill)

Year	AP (mm)	No. Storm Events	Average Depth (mm)
86/87	1090	118	9,2
87/88	691	107	6,4
88/89	647	108	6,0
89/90	608	110	5,5
90/91	447+	61+	7,3

Table 3 - Seasonal Variation in rainfall, %

Year	O	N	D	J	F	M	A	M	J	J	A	S
86/87	3	16	15	15	21	8	15	1	-	-	-	6
87/88	15	6	23	10	6	29	4	2	1	3	1	-
88/89	3	9	18	11	24	18	2	6	8	1	-	-
89/90	4	17	15	5	15	19	6	17	-	-	1	1
Ave.	6	12	17	10	16	19	7	7	2	1	1	2

The three monthly piped water supply figures were averaged over each month. There are three main components to the water supply namely, an overall trend, rapid increases in consumption and cyclic variations in supply requirements.

The overall trend in the data is one of increasing domestic usage of water in the catchment. Consumption increased from $138m^3$/d in 1986 to a $414m^3$/day consumption in November 1990; this is a nearly three-fold increase in water usage in the catchment. The rapid increase in the consumption of domestic water occurs as a result of increasing development of the catchment. The inhabited stand count increased from 110 to 124 during the period October 1987 to 1988 during which consumption rose from an average of $138m^3$/day to $190m^3$/day. The development of townhouses during the period October 1989 to August 1990 also resulted in a dramatic increase in consumption. Townhouses are not individually metered but houses are.

Twelve boreholes were drilled over the catchments to monitor groundwater fluctuations. Eight autographic raingauges with data loggers attached monitored the rainfall over the catchment. Data loggers monitored the stage and hence the flow rate over the weirs at the outflow of each catchment and another data logger monitored the sewage outflow. Numerous problems arose with the electronic data loggers so that a lot of earlier data was missed. It was therefore necessary to use later data to obtain a rainfall-runoff relationship to extrapolate runoff flows.

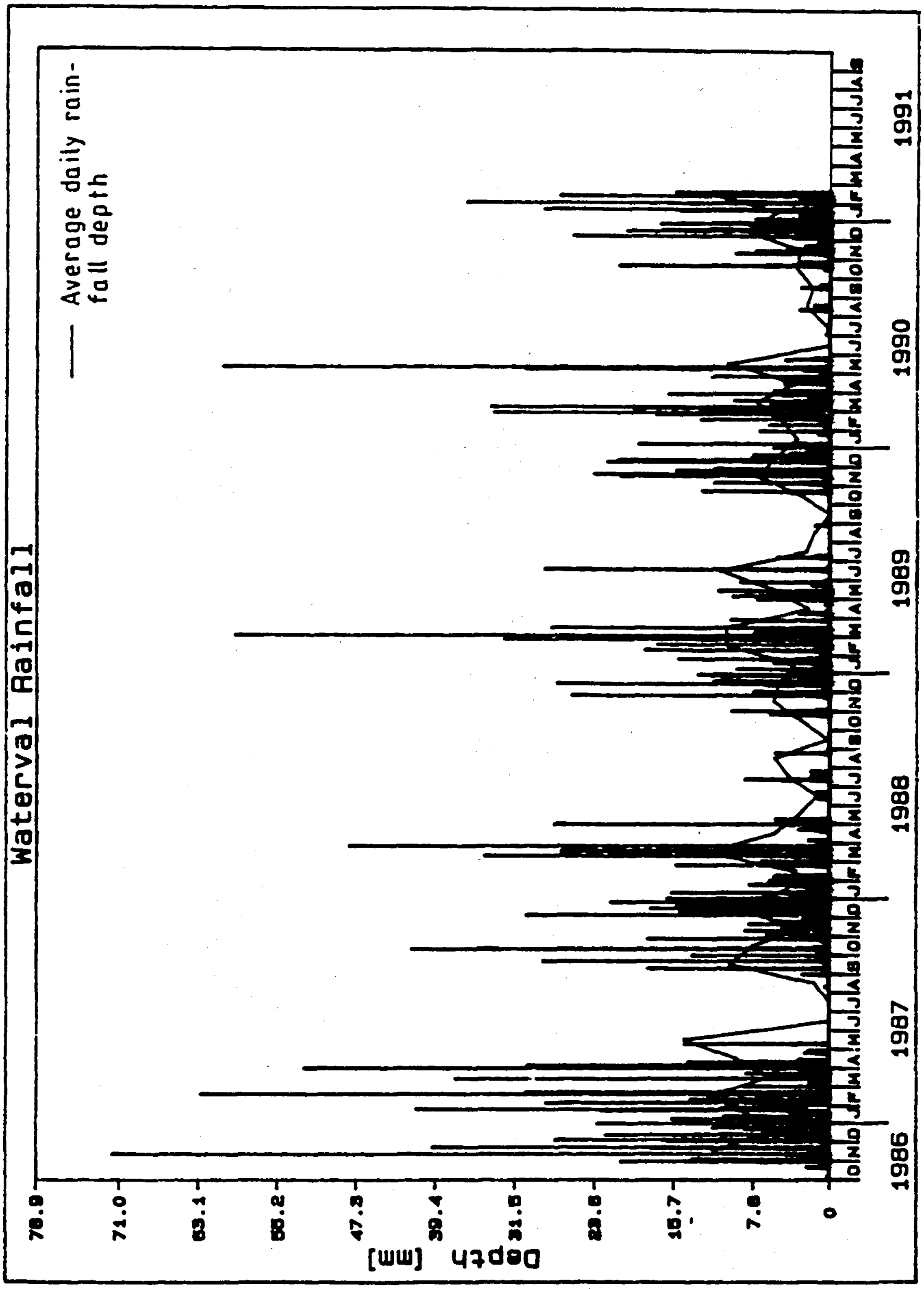

Fig. 4 Waterval rainfall over study period

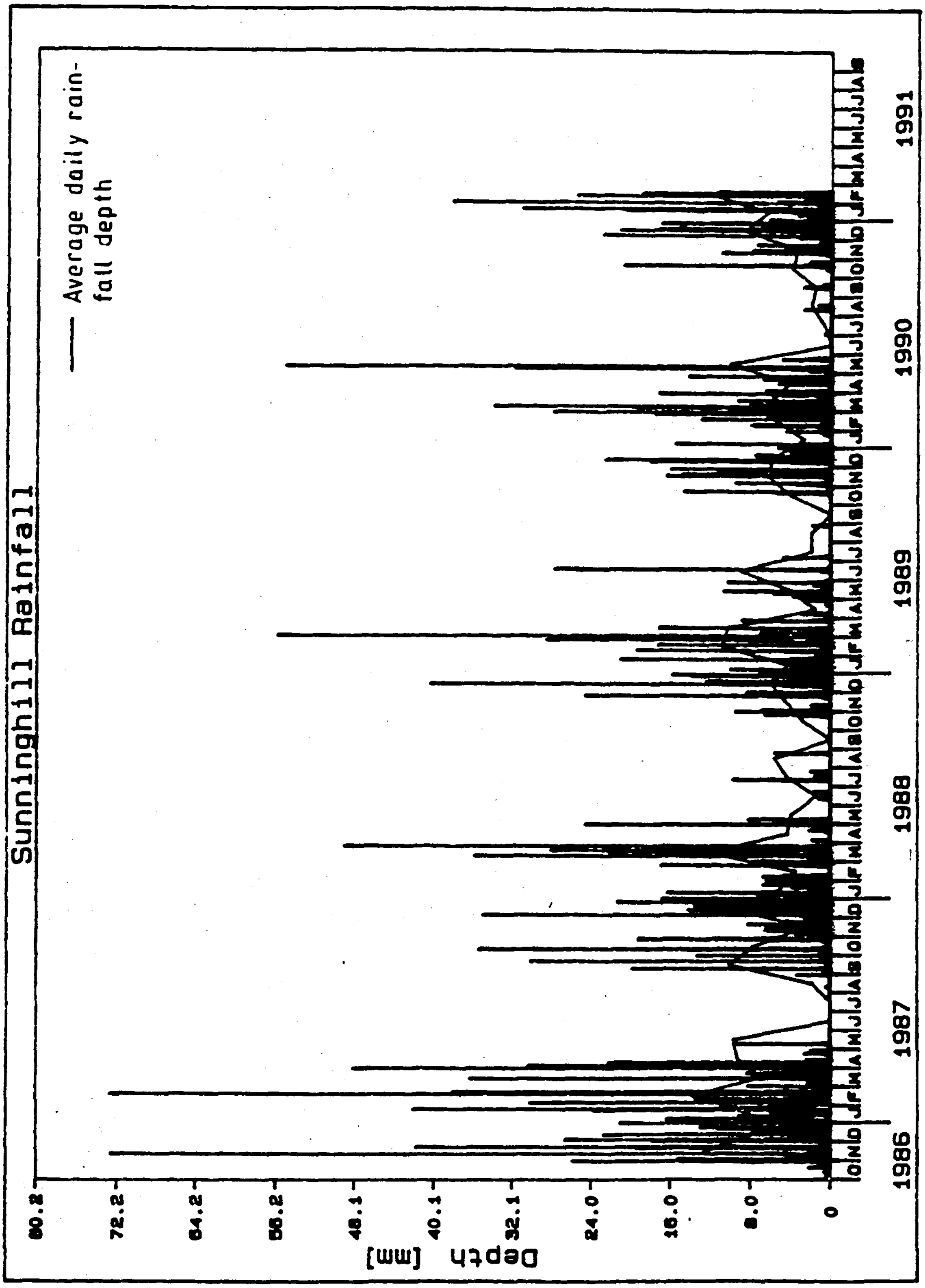

Fig. 5 Waterval rainfall over study period

Borehole water levels were monitored initially manually and sometimes with electronic sensors later. The catchments were investigated geologically using borehole logs and in the case of the Waterval catchment resistivity and magnetic surveys were used to identify dykes and discontinuities. Groundwater flow was tested with borehole tests, tracer tests, isotopic and dilution tests, with inconclusive results.

ANALYSIS OF RESULTS

To distinguish between the different impacts on runoff, a digital computer model was compiled (Stephenson, 1989). This model allows for multiple units in the drainage process, i.e. it is able to connect impervious and pervious catchments, groundwater layers and conduits. Rainfall was obtained from the recorders in the catchments, although simulated storms could also be used. The computer model predicts the flow rates throughout a storm. The model is able to account for management methods in attenuating or concentrating storms. The dual drainage whereby water would overflow underground sewers and run down the roads is accounted for, as well as attenuation due to flood drains such as the central channel in Sunninghill.

Of the typical two- to three-fold increase in stormflow from the developed catchment, computations indicated half is due to reduced infiltration and half to reduced concentration time resulting in shorter critical storms for the developed catchment (see Stephenson and Meadows, 1986).

The variations in groundwater levels proved inconclusive. (Figs. 6 and 7). There were minor daily fluctuations probably due to thermal effects but also attributable to solar and lunar effects. No general relative depletion of the water table in the urban catchment was however observed and this could be due to contributions by garden watering, balancing what would have otherwise been a depletion in the water table due to less rainfall infiltration. Although the rainfall over the period was

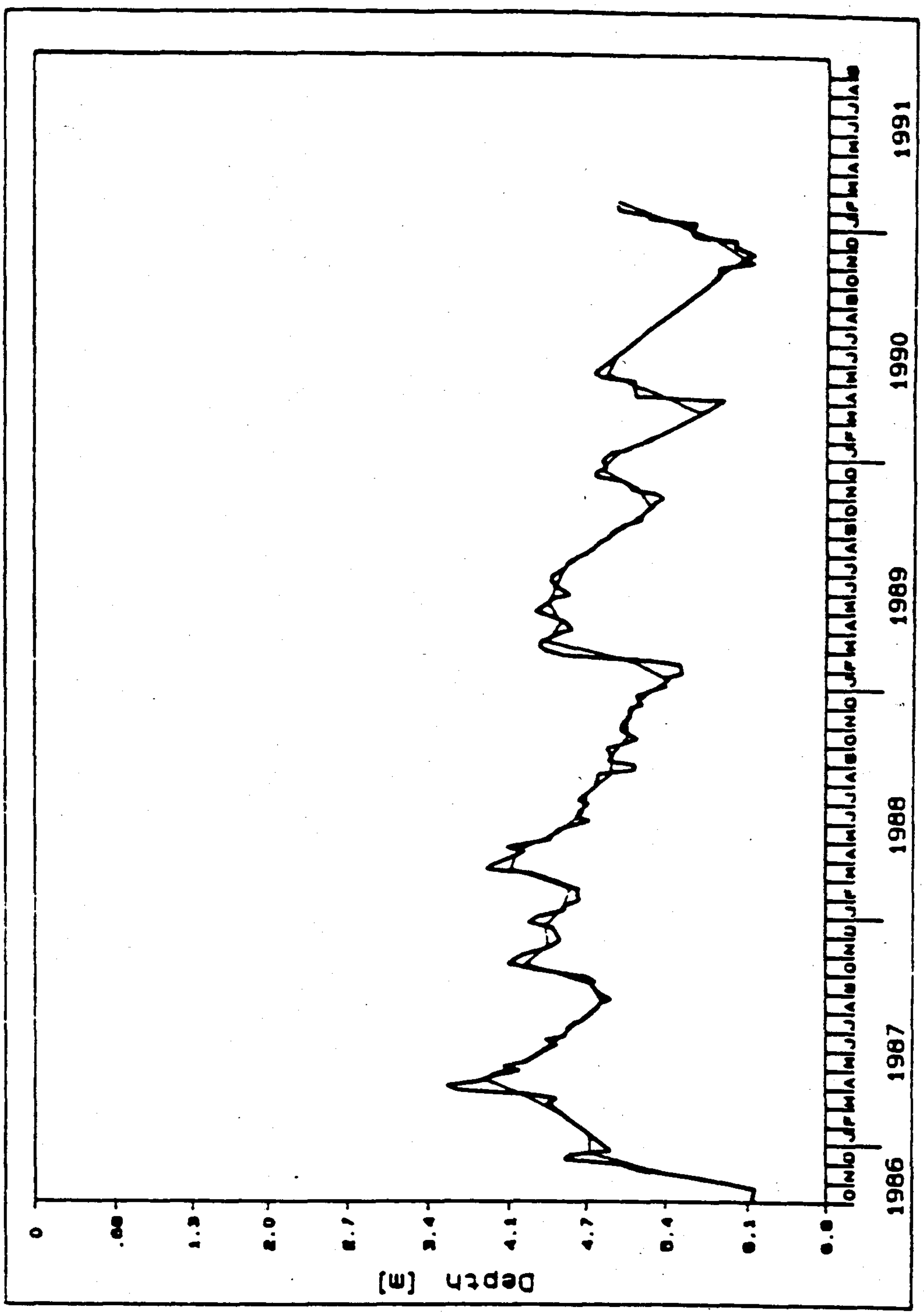

Fig. 6 Waterval - Borehole water level variations

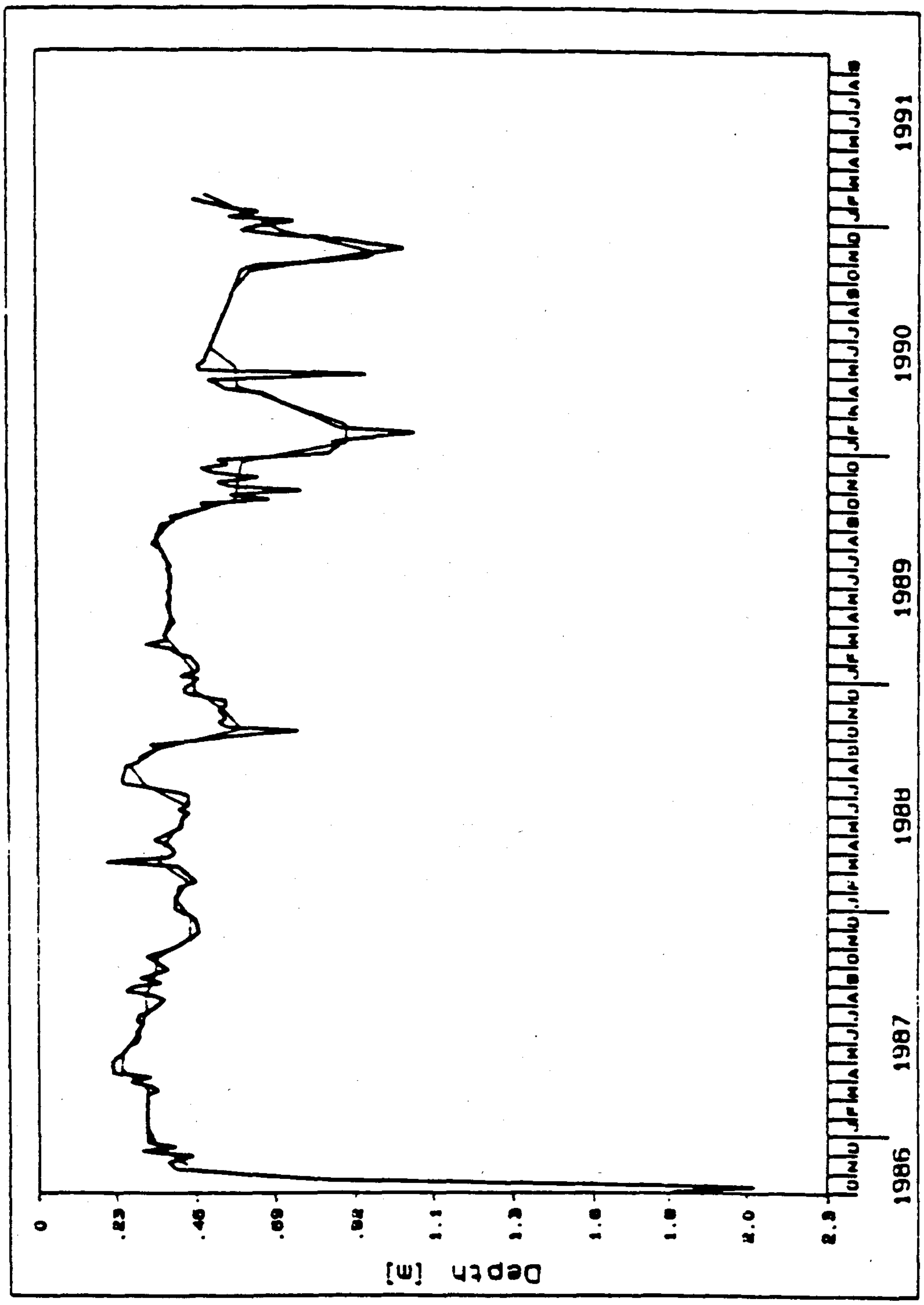

Fig. 7 Sunninghill - Borehole water level variations

probably above average, especially in early years, there was no notable increase in water tables in the rural catchment either.

Figures 8 and 9 present the cumulative rainfall (input), surface runoff (measured) and calculated evaporative losses over the period 1986-91. Rainfall rate appears to decrease with time. Surface runoff rate showed a marked decrease after 1986, whereas evaporation continued to exhibit regular seasonal cycles. The total evaporation or transpiration rate was calculated from weather data from 1987 onwards. The corresponding calculated groundwater accretion in Waterval agrees with that observed. The observed groundwater gain including subsurface outflow from August 1987 to February 1991 was 3.5 x 200 000 m^3/a = 700 000 m^3, whereas it was deduced to be 630 000m^3 for a balance.

Total above-surface loss from Waterval from August 1987 to February 1991 was 1 270 000 m^3, 67% of total precipitation of 1 900 000 m^3 over the period. The balance should have been in the form of groundwater outflow or accretion. This represents 180 000 m^3/an which is similar to that deduced from aquifer tests.

The total rainfall onto Sunninghill follows a similar pattern to Waterval. The other input, namely water supply, was only 16% of the rainfall. The surface outflow from Sunninghill was 1 480 000 m^3 or 67% of the inflow from August 1987 to February 1991. The balance must have been in the form of groundwater outflow i.e. 470 000m^3.

Evaporation from Sunninghill was estimated from an evaporation model to be 1 200 000 m^3 so the total surface outflow was 530 000 m^3, more than for Waterval (primarily in the form of sewage and increased runoff).

Total surface runoff from Sunninghill is four times that from Waterval. No difference in evaporation was estimated indicating the increased garden watering probably balanced the reduced

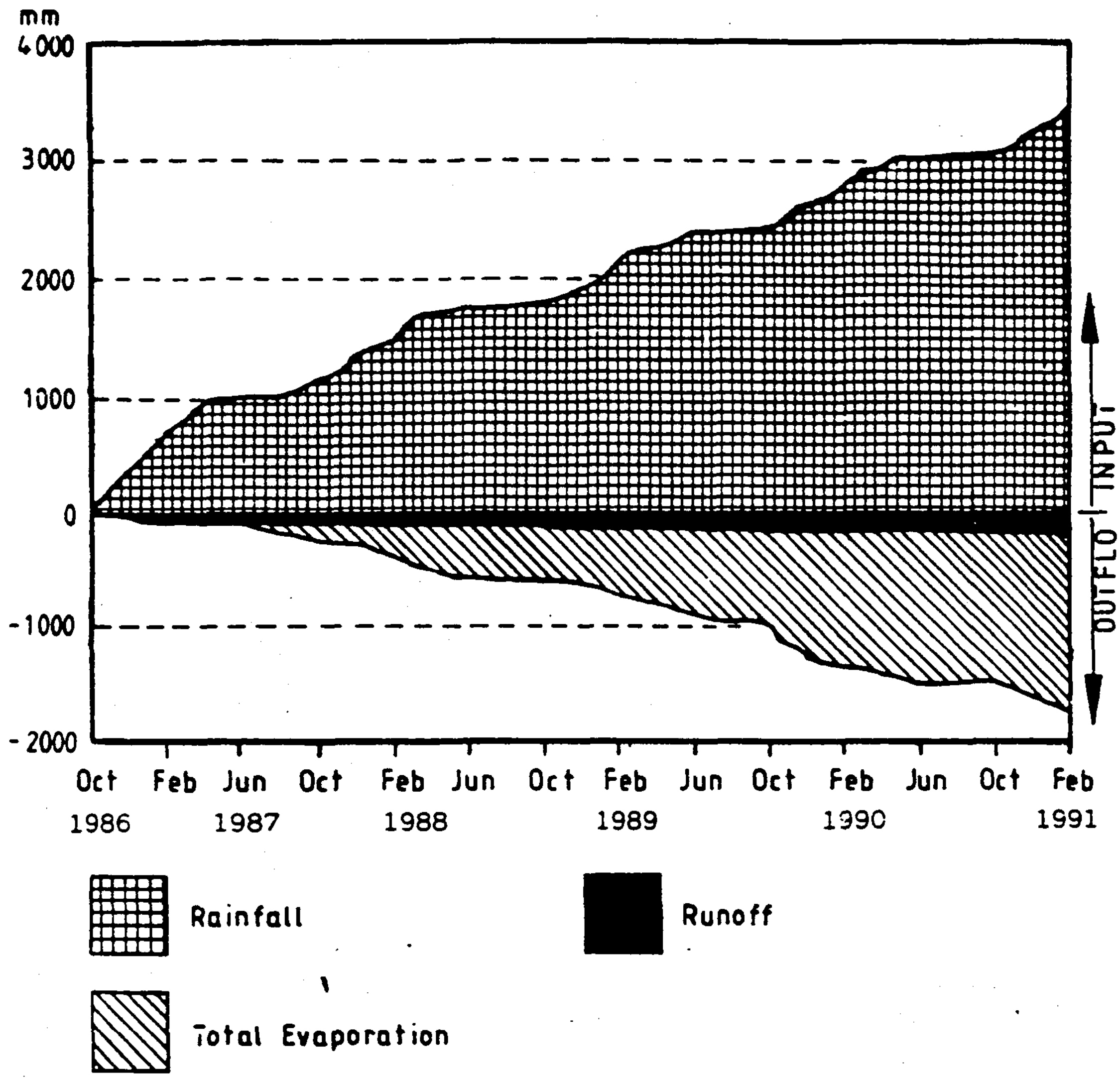

Fig. 8 **Waterval catchment water balance**

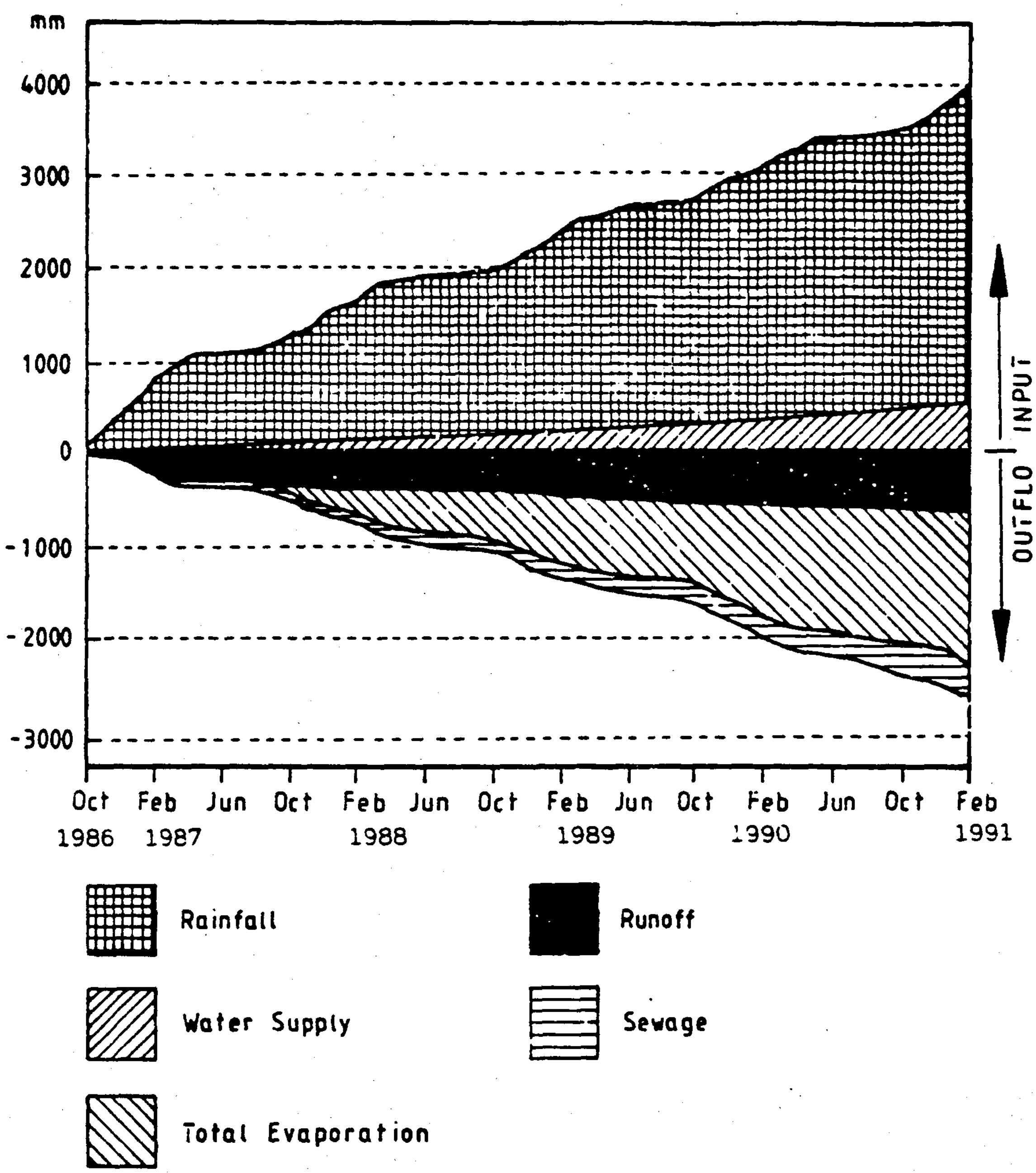

Fig. 9 Sunninghill Catchment water balance

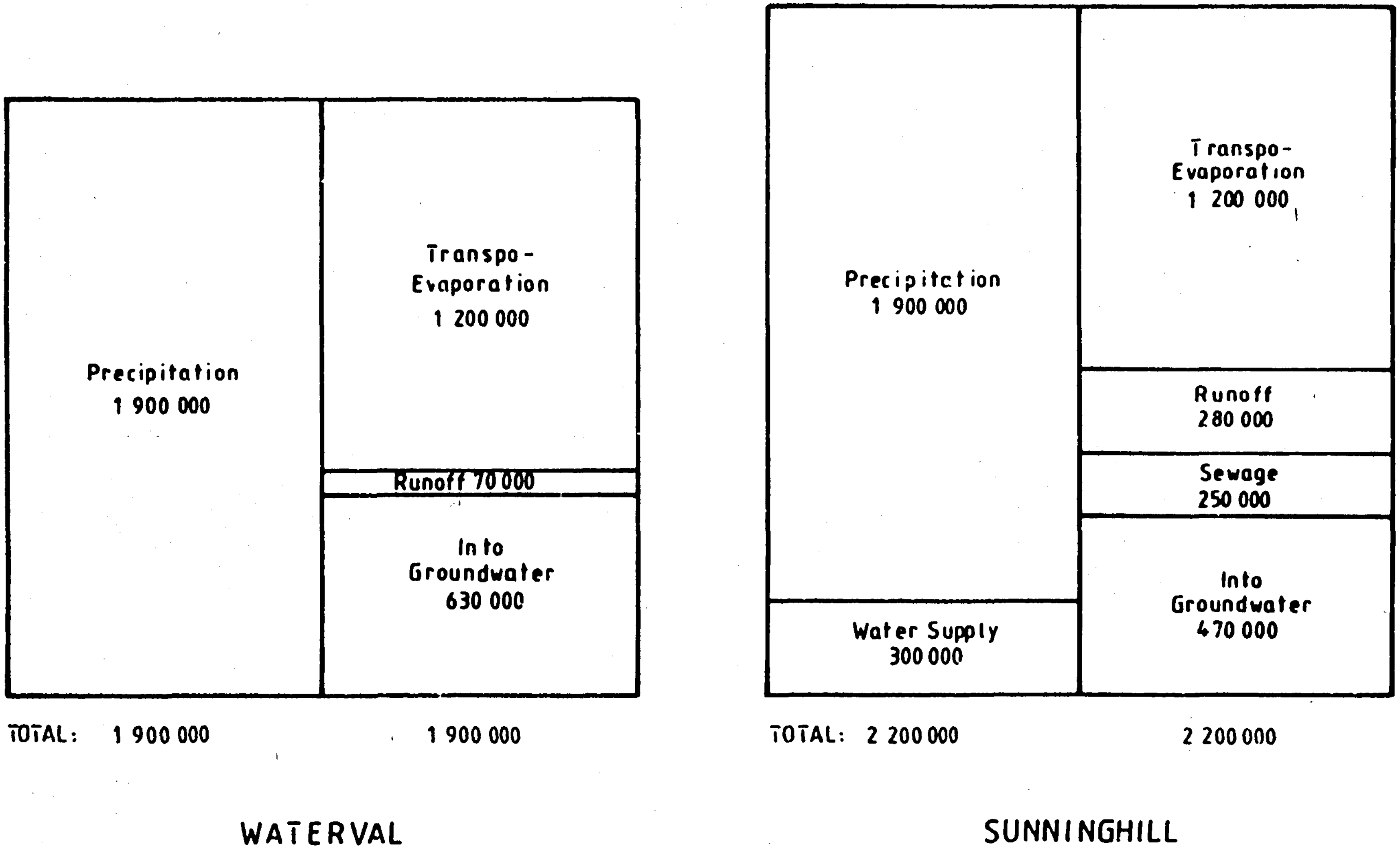

Fig. 10 Total Water Balance of Both Catchments. Cumulative flow in m^3 over 3.5 years, August 1987 to February 1991

vegetated area. Sewage flow was 83% of water supply but only 11% of total outflow from Sunninghill. Sewer flow probably contained a lot of illegally diverted stormflow as stormwater drainage is a separate system.

Surface runoff from August 1987 to February 1991 from Waterval was 4% of the rainfall (bear in mind the runoff was not entirely observed and the record had to be patched by correlation with rainfall). Surface runoff from Sunninghill was 15% of the rainfall. The Sunninghill runoff showed initial acceleration over Waterval which may be associated with the higher rainfall rate, then exhibits a steady increase over Waterval. There is no trend away from this steady increase except in 1990/1 when the Sunninghill curve deviates noticeably upwards. This could be associated with the construction of two townhouse complexes resulting in a larger portion of runoff.

CONCLUSIONS

Suburban development increased surface runoff by a factor of 4 over an otherwise similar undeveloped catchment. This is largely due to impermeable cover. The frequency of flood runoff for the developed catchment also increased due to rapid concentration of flow. The major loss, due to evapotranspiration was 67% of precipitation for both catchments, and water tables in both catchments varied similarly. Garden watering compensated for increased runoff from the sub-urban catchment. Water supply to the developed catchment was 16% of rainfall input, and sewerage outflow was 83% of water supply.

From the runoff model, it appeared that the most useful change in townplanning practice would be to disconnect impervious areas from storm drains to reduce runoff. Garden watering appeared to increase evapotranspiration in the suburb as losses were similar to the grassed natural catchment. Stormwater drainage management could further reduce runoff volumes and frequency.

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