ANALYSIS OF EFFECTS OF URBANIZATION ON RUNOFF

BY D STEPHENSON

WRC REPORT NO 183/1/93

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Water Systems Research Group
Report No. 1
1992

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

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WRC REPORT NO. 183/1/93
ISBN 1 86845 029 5
ISBN SET NO 1 86845 040 6

WATER RESEARCH COMMISSION

EFFECTS OF URBANIZATION ON CATCHMENT WATER BALANCE

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ABSTRACT

In studying the effects of urbanization on catchment runoff, the complexity of urban development makes it difficult to assess individual effects separately although total flows and runoffs can be gauged. Individual effects are therefore distinguished with the assistance of theoretical models.

The separate effects considered are :

i) Impermeable cover
ii) Reduced roughness
iii) Channelization
iv) Storage
v) Recession of groundwater table
vi) Township layout
vii) Disconnection of impervious surfaces
viii) Flood planes
ix) Dual drainage

A literature review identified studies on effects of recurrence interval and generalized population density effects.

Specific studies are made here as the effects mentioned above and channelization appears to have a predominant effect on peak runoff increasing it by many times. Storage can reduce this effect but disconnection of impervious surfaces reduces runoff volumes more.

Specific suggestions for planning townships also emanate. Dual drainage reduces peak runoff rates. Township layout can affect peaks. Volume of runoff could also be affected by storage coupled with infiltration.

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CHAPTER 1

METHOD OF ESTIMATION OF EFFECTS OF URBANIZATION

The prime objective of the research contract between the Water Research Commission and the University of the Witwatersrand Water Systems Research Group is to assess the effects of urbanization on catchment runoff. For this purpose two catchments were used, one of which is a virgin catchment and the other one urbanized. The catchments are similar in size and adjacent to each other and appear similar topographically and geologically.

Ideally to enable the comparison to be unbiased, the two catchments should have been gauged prior to urbanization of one. That is the paired catchment experimental approach (Lindley et al, 1988 and Bosch and Hewlett, 1982). By gauging the two catchments in the natural condition it may have been possible to assess the effects of parameters other than urbanization causing differences in runoff. Two periods would be required; an initial period pre urbanization for the one and a second period where the one was urbanized.

Unfortunately such an approach would be highly impractical in this case. Urban development takes many years. Even once an area has been zoned for urban development it may be many years before building commences and then it may be a number of years before the services are installed and many decades before the catchment is fully urbanized i.e. each stand is developed.

An alternative approach would be to take a pair of catchments once one had been urbanized and gauge them. Gauging should continue for many years in anticipation of the development of the undeveloped catchment. Thus the first period would be for the two differently developed catchments and the second gauging period for the two catchments urbanized. Then a multiple regression would be possible to assess the differences between the two catchments over both periods. In fact if it were not for climatic changes the single initially undeveloped and later developed catchment would be adequate. Unfortunately there is no gaurantee that the undeveloped catchment will be developed and no knowledge of when. In any case a single catchment on its own cannot produce reliable results unless gauged over many years before and after development as the climatic changes may have a predominant or unknown effect on change in runoff.

The methodology for the present study then is to compare two catchments over a common time period of a number of years. The difference in runoff observed over any time between the two catchments could however be due to a number of different effects i.e. not only urbanization. In order to distinguish between different effects therefore the proposal was to model the runoff in some detail with a realistic distributed parameter model. Provided the model is based on physical laws and measured parameters then calibration effects are minimized and the model can be said to represent all the observable effects if verified.

A component of the study is therefore the assembly of a physical or hydrodynamic type model allowing for as many effects as can be anticipated. The type of model selected was a kinematic and hydrodynamic model with known physical parameters both surface and subsurface flow. Previous studies have verified and proved such models but the knowledge of subsurface and interflow is limited. Even more so the subsurface parameters such as porosity, depth of permeable strata and permeabilities can only be estimated from sample boreholes and other geophysical means. A large portion of the study is therefore devoted to a) assessing the importance of such parameters and b) the estimation of the significant parameters. There is considerable literature on various components in such models and indeed on such distributed parameter type models (Stephenson and Meadows, 1986, Campbell, 1985).

The two catchments differ from each other with regard to the following parameters and the extent of each is assessed: size, shape, aspect, slopes, geology, vegetation, stream pattern as well as climatic factors such as wind direction and elevation which can affect temperatures, rainfall, evaporation and even subsurface geological weathering. Many of these factors can be modelled and therefore isolated. This approach is adopted in trying to assess the individual effects of urbanization on the runoff and infiltration pattern and provided the model successfully reproduces the observed runoffs with minimal calibration it can be assumed with some reliability that the individual factors are correctly modelled.

IMPROVEMENTS BY MAN TO CATCHMENTS FROM THE HYDROLOGICAL POINT OF VIEW

Although the conventional or older type of urban development is recognized as causing increased storm water runoff and therefore reduced dry weather flows, adequate catchment management to minimize effective urbanization is now recognized by many hydrologists and implemented in some countries (e.g. SAICE, 1988)). The use of stormwater detention ponds has been investigated for a while for example Hall and Hockin (1980). More sophisticated methods such as dual drainage and alternative township layouts and also channel storage are considered in few cases (Stephenson, 1988). Some studies (e.g. Ormsbee and Reinert, 1985) are restricted by the use of conventional techniques and therefore mask some of the possible management methods. For instance the use of empirical infiltration equations and time-area type runoff assumptions often results in the incorrect design storm selection and therefore the input output relationships may not be modelled correctly. For this reason hydraulic models are preferred in this report.

CHAPTER 2

EFFECTS OF URBANIZATION ON CATCHMENT RUNOFF AND WATER RESOURCES

LITERATURE REVIEW

It is recognized that urbanization and man affect the hydrological regime of catchments and the effects have been evaluated and assessed under a wide range of disciplines. A number of studies by the US Geological Survey (Stankowski, 1972; Leopold, 1968). The hydrological changes can be correlated with various factors which are however inter-related. For instance Stankowski indicates the effect of population density on land surface modifications. The population density however is the cause of the land use change and not a direct factor influencing runoff. Thus land use change will result in urbanization and man-made structures which have a more direct effect. Leopold summarized the various factors which do have an influence on storm runoff in particular such as the percentage area sewered, the degree of channelization and the percentage cover.

The amount of impervious cover is the first factor which usually springs to mind as being the cause for increased runoff due to urbanization. Alley and Veenhuis (1983) made a study of the effect of impervious area on runoff. Both storm flow runoff and water quality receive particular attention by American modellers although the latter factor may be due to the interest expressed in water quality via the US Environmental Protection Agency. The hydraulic factors which can be shown to increased storm runoff were studied by Stephenson and Meadows (1986).

The demographic studies performed by Stankowski (1972) may not be entirely applicable in South Africa or Africa as the land use pattern is not the same and in fact population density is probably less significant than the type of urbanization. It is recognized that urbanization in upper income type residential areas in South Africa is probably less dense than in average American cities that when it comes to low income type urban development we have a unique situation and this probably requires even more detailed study than this research contract provides. The main reason Stankowski used population density was that it was the easiest obtainable figure whereas this may not be the case in South Africa and Maps from aerial surveys may in fact be much easier to obtain and hence the impervious area can be measured. Relationships can be found between population density and the following land use aspects: single family residential dwellings, multi family residential dwellings, commercial and business relations, industrial and development type facilities, services such as roads and railways, public utilities and institutions including sports facilities, schools, airports and lastly recreational areas such as parks. In fact Stankowski obtained regression type equations relating the percent of different land use types to the population density. These equations are presented below for interest only as they are not entirely applicable in South African catchments owing to different building and planning standards and styles, different catchment characteristics and different rainfall characteristics.

$$SFR = 0{,}000528D^{2{,}520 - 0{,}339 \log D},$$
$$MFR = 0{,}004446D^{0{,}127 + 0{,}181 \log D},$$
$$C = 0{,}000427D^{1{,}818 - 0{,}202 \log D},$$
$$IND = 0{,}00005D^{2{,}210 - 0{,}212 \log D},$$
$$P = 0.003612D^{1{,}737 - 0{,}227 \log D}, \text{ and}$$
$$CRO = 25{,}16D^{0{,}555 - 0{,}135 \log D},$$

where

SFR = single-family residential land, in percent of land area,

D = population density, in persons per square mile,

MFR = multiple-family residential land, in percent of land area,

C = commercial land, in percent of land area,

IND = industrial land, in percent of land area

P = public and quasi-public land, in percent of land area, and

CRO = conservational, recreational, and open land, in percent of land area.

The more general paper of Leopold attempted to summarise more from a hydraulic point of view the effects likely to cause changes in runoff due to urbanization. The land use patterns interpreted in terms of hydrological parameters and four effects were identified. These are changes in peak flows and hydrograph, volume of runoff, water quality and lastly hydrological amenities which refer to the environment. An important factor identified was urbanization, which is by far the most forceful land use change affecting hydrology and runoff can increase by many factors implying that retention is reduced considerably. For instance if the runoff off an undeveloped catchment is 20% of rainfall, then a 50% impervious cover would increase the runoff by a factor of 3 ((50% x 1+50% x 0,2)/20%), not 2. The paper goes on to indicate that the two factors most important from the point of view of the flow regime are the percentage of area made impervious and the rate of transmission of water to stream channels. A relationship between lot or stand size and percentage impervious area is given but the fact that the percentage disconnected impervious area is the most important factor is not highlighted. Modelling studies indicate that it is primarily the directly connected impervious area which contributes to increased runoff.

Another factor identified by Leopold was that not only does imperviousness increase flood peaks during storms but it also decreases low flows between storms. This is due to the fact that the volume of runoff increases with urbanization and therefore there is less infiltration. This in turn results in less ground water accretion and therefore less long term seepage from the aquifer. It also means that the surfaces become drier and therefore tend to absorb more of the lighter precipitation.

Water quality is another factor which deteriorates with urbanization. Sediment yields tend to be larger which could be due to dust collecting on impervious areas but is more likely to be due to the drier uncovered areas allowing greater errosion rates. There is also the disturbances of man such as construction activities which can contribute large volumes of sediment. Pollution due to wastewaters can also increase although in the South African environment there is a separate waste water collection system which in principle is isolated from the stormwater system. In fact there are interconnections either indirectly through leekage or illegally to avoid sewage tariff payments.

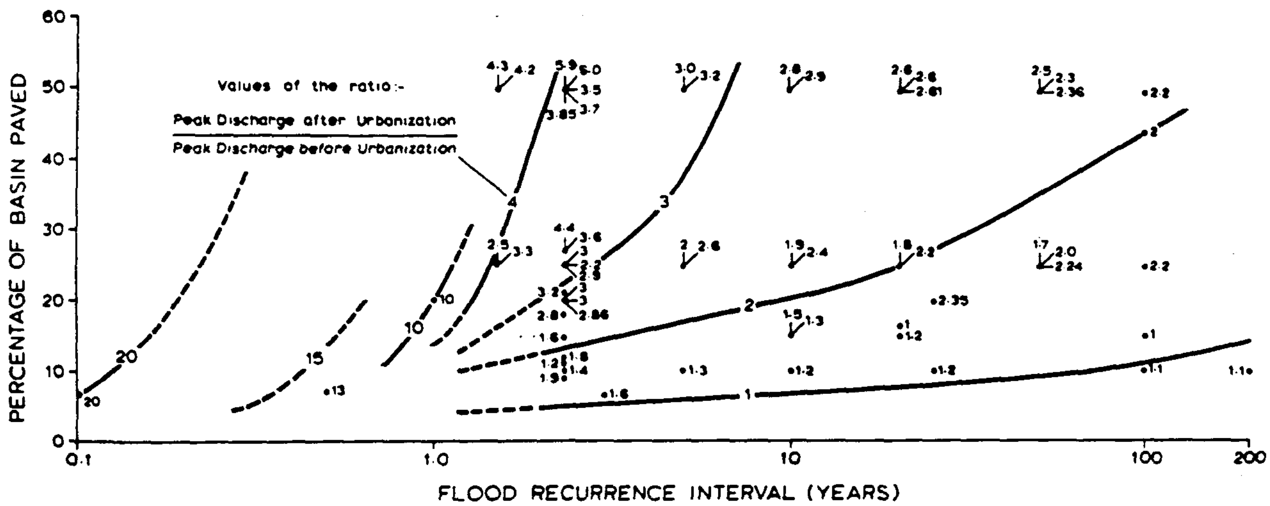


Fig. 2.1 Effect of urbanization on flood peaks. (Data taken from Hollis, 1975.)

The fact that lag time of a catchment is affected by urbanisation was identified by Leopold and this is the fact which was further analyzed by Stephenson and Meadows (1986). There it was indicated that both the reduction and roughness and the increased depth in stormwater drains and channels shorten the concentration time considerably. It was however recognized by Stephenson and Meadows that it is not the lag time which is for the entire catchment which is important but the fact that a bigger area can contribute in a shorter time which leads to increased runoff in the majority of cases. That is for many catchment even in the urban environment, the catchment does not have to reach equilibrium for the worst or peak runoff rate.

In fact it is the change in soil moisture conditions which can result in a change in runoff pattern unrelated to rainfall pattern. Thus the assumption that the recurrence interval of the flood is the same as the rainfall becomes even less accurate as soil moisture conditions are affected. Lambourne (1988) showed that soil moisture deficiency can considerably affect the flood recurrence interval pattern.

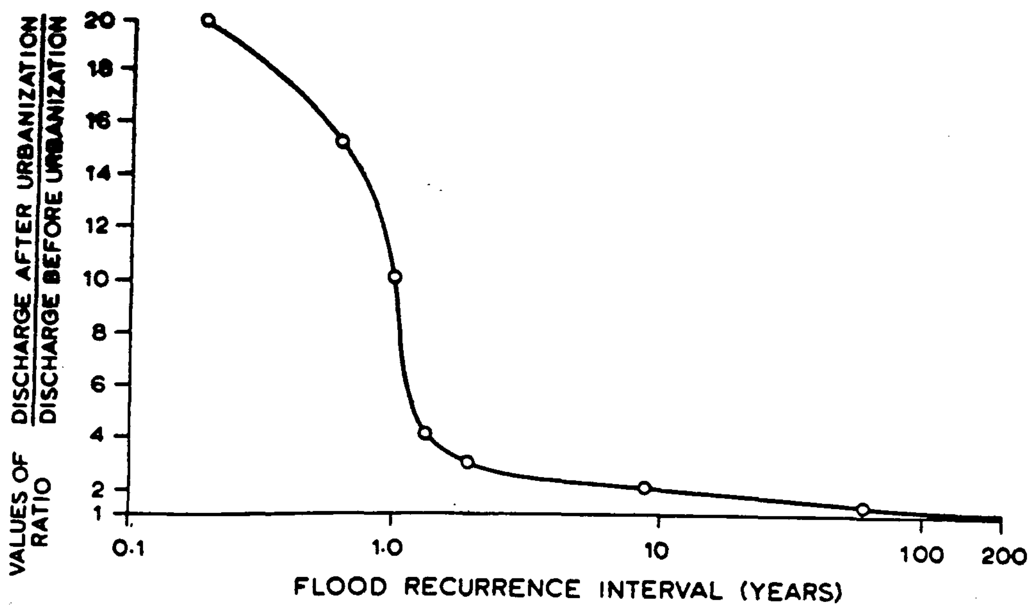


Fig. 2.2 Effect on flood magnitudes of paving 20% of a basin (Hollis, 1975).

Alley and Veenhuis (1983) divided impervious areas into two types. The first, referred to as effective impervious area, is the directly of hydraulically connected area. The second was referred to as non-effective impervious area and comprises the indirectly connected areas i.e. impervious areas which subsequently flow or discharge onto pervious areas. For many low intensity storm areas the directly connected impervious areas may be the only ones which contribute to runoff during a flood hydrograph. In the case of high intensity type areas such as the Witwatersrand where this contract's experimental catchments are situated, it is recognized that runoff does occur from pervious areas and therefore the modelling of such pervious areas and infiltration is particularly important. They use data from nineteen urban basin in Denver. Of the total impervious area with an average of 40% in the case of single family small residential lots, the effective impervious area was only 23%, or 60% of the total impervious area. This increased to 94% of a total impervious area of 88% in the case of commercially used land. This is an effective impervious area of 80% and is the maximum observed.

Effects on floods of different recurrence interval

A synthesis of published results was made by Hollis (1975) to show that urbanization increases flood magnitude by a factor of up to 20. The effect of urbanization was found to decrease for large recurrence intervals. This appears because during severe or prolonged storms the catchment in its natural becomes saturated anyway so the runoff proportion is greater i.e. it acts similar to a paved catchment (see Fig. 2.1 and 2.2.

Leopold's work (1968) was reviewed by the American Society of Civil Engineers Task Force on Effect of Urban Development on Flood Discharge. They derived curves indicating increase in flood peaks following given amounts of urbanization, percentage paved and flood recurrence interval (Figs. 2.3 and 2.4)

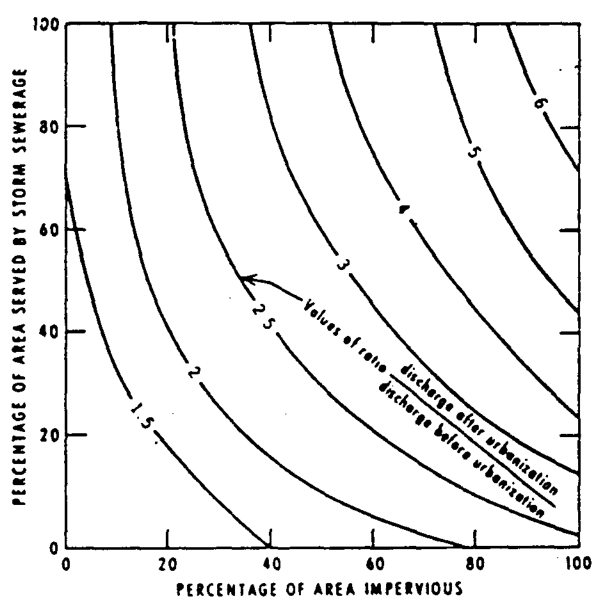


Fig. 2.3 Effect of urbanization on mean annual flood for a 1-square-mile drainage area. (Leopold, 1968).

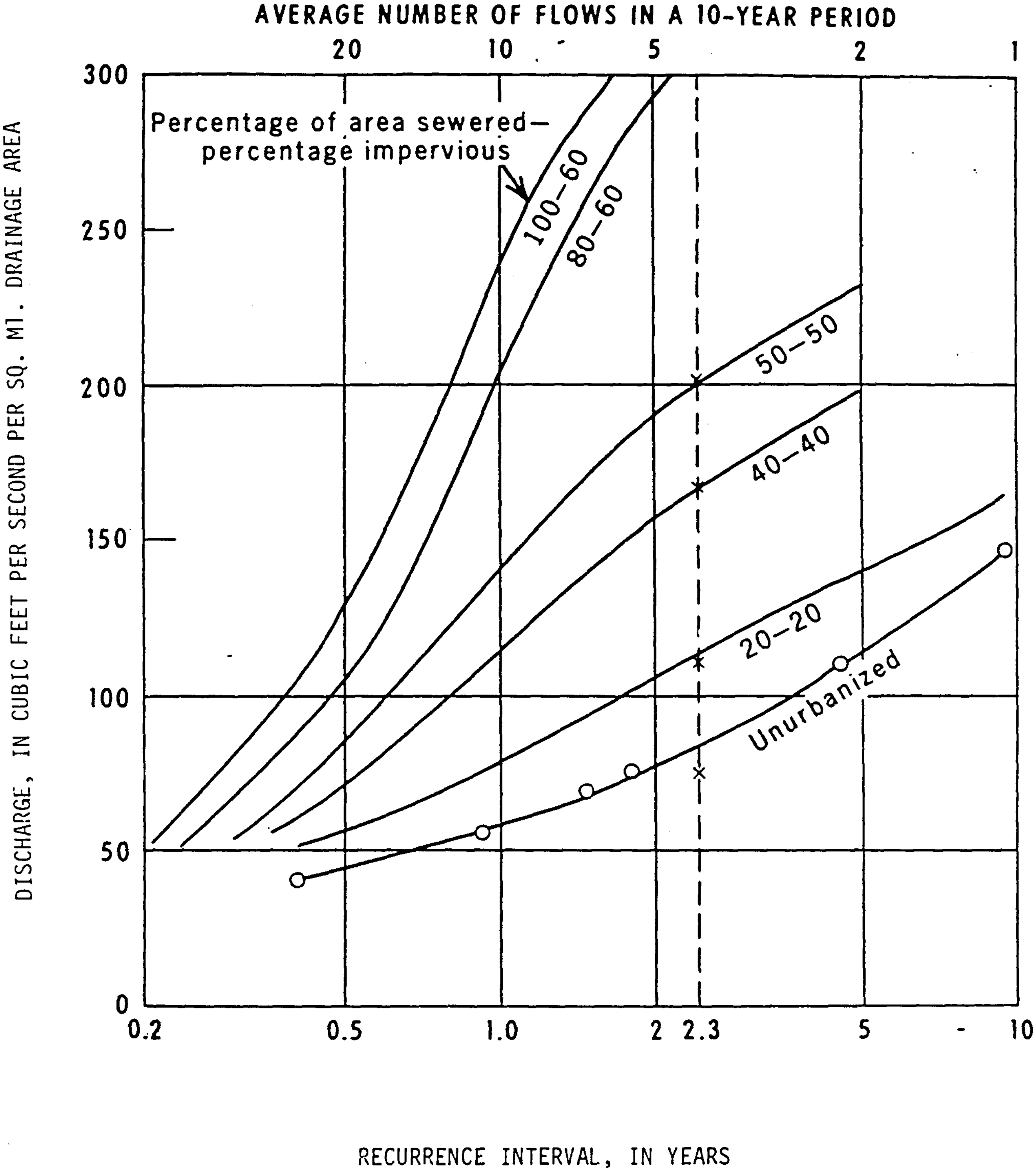


Fig. 2.4 Flood-frequency curves for a 2 square km basin in various states of urbanization. (Leopold, 1968.)

CHAPTER 3

URBAN CATCHMENT MANAGEMENT EFFECTS

In nature a semi-equilibrium exists between precipitation, runoff and infiltration into the ground. Over years the water table fluctuates about a mean. It recedes during droughts when seepage into watercourse exceeds replenishment rates, and rises when it rains. The depth of soil above water table is generally not excessive or else vegetation dies, the ground dries out and wind blows the soil away. The amount of water which rises up in the soil under capillary action or in vapour form is limited by the depth of water table.

The construction of impermeable barriers on the surface, such as roads and buildings, reduce the rate of ground water replenishment. The water runs off easier and the limited permeable area restricts infiltration. The groundwater level will therefore drop and the zone above the water table will gradually dry out. Vegetation and the soil characteristics will change. If we are not to affect our environment adversely we should attempt to return some of the stormwater we channel off our urban area back to the ground. This can be accomplished by ensuring adequate permeable surfaces, and by directing stormwater into specially selected or constructed seepage areas. WE will then not only maintain the regime but also minimize design flow rates downstream.

The depletion of groundwater will also alter the relationship between rainfall and runoff. After a dry spell more water will be needed to saturate the ground so that the initial abstraction may be greater than before the development occurred. This is offset to an extent by the impermeable ground cover. The net effect is to make a more extreme hydrology i.e. a greater difference between floods and droughts than before development .

Effect on Recurrence Interval

Urban development affects the rainfall pattern and statistics as well as the runoff pattern. It has been alleged that blanketing effects due to solar shields affect evaporation and hence the resultant precipitation. The blanket of smot, dust, fumes etc., may also affect the place in which the clouds release their moisture, so the effect of urbanization on rainfall is difficult to estimate and the statistical properties of rainfall records (e.g. the mean, coefficient of variance, frequency and distribution) will be affected as well to some extent. Rainfall is reputed to fall more on the leeward side of cities due to the heating up of the air over the city and up to 15% more precipitation has been attributed to this effect. (Huff and Changnon, 1972; Colyer, 1982). Apart from this, the relationship between rainfall and runoff is affected.

Some of the simplistic methods of assessing runoff suppose that the recurrence interval of a calculated flood is the same as the recurrence interval of the causitive rainfall for the design storm duration. It could be that this assumption is borne in mind in the choice of the Rational coefficient. That is the use of the rational method gives a certain recurrence interval of runoff (equal to that of the selected storm in fact) but it does not imply that the design storm is the one which will produce that runoff. This is a gross simplification and it is rarely that the recurrence interval of a storm and its resulting flood coincide. This is due to the predominating effect of abstraction or losses. It will be recalled that generally the Rational coefficient C is nearer 0 than 1 implying losses are greater than runoff. That in turn means that losses, which in turn are mostly soil moisture abstraction, affect runoff more than rainfall. Hence the runoff and its return period should be more related to soil moisture conditions than to rainfall. A study by Sutherland (1982) indicates little correlation between rainfall recurrence interval and the recurrence interval of the flood when assessed in terms of the peak flow rate. He proposed that antecedant moisture conditions, measured in terms of the total precipitation in preceeding days, should be a parameter in runoff-duration-frequency relationships. His contention is that the probability of a certain runoff intensity is more related to the probability of the soil being at a certain saturation than the rainfall intensity.

How does urbanization affect the argument? In fact it counters the above ideas. The more the natural surface cover is replaced by impermeable surfaces the more runoff becomes a direct response function to rainfall. In the limit for 100% runoff, soil does not feature and the recurrence interval of runoff is equal to that of the storm causing it.

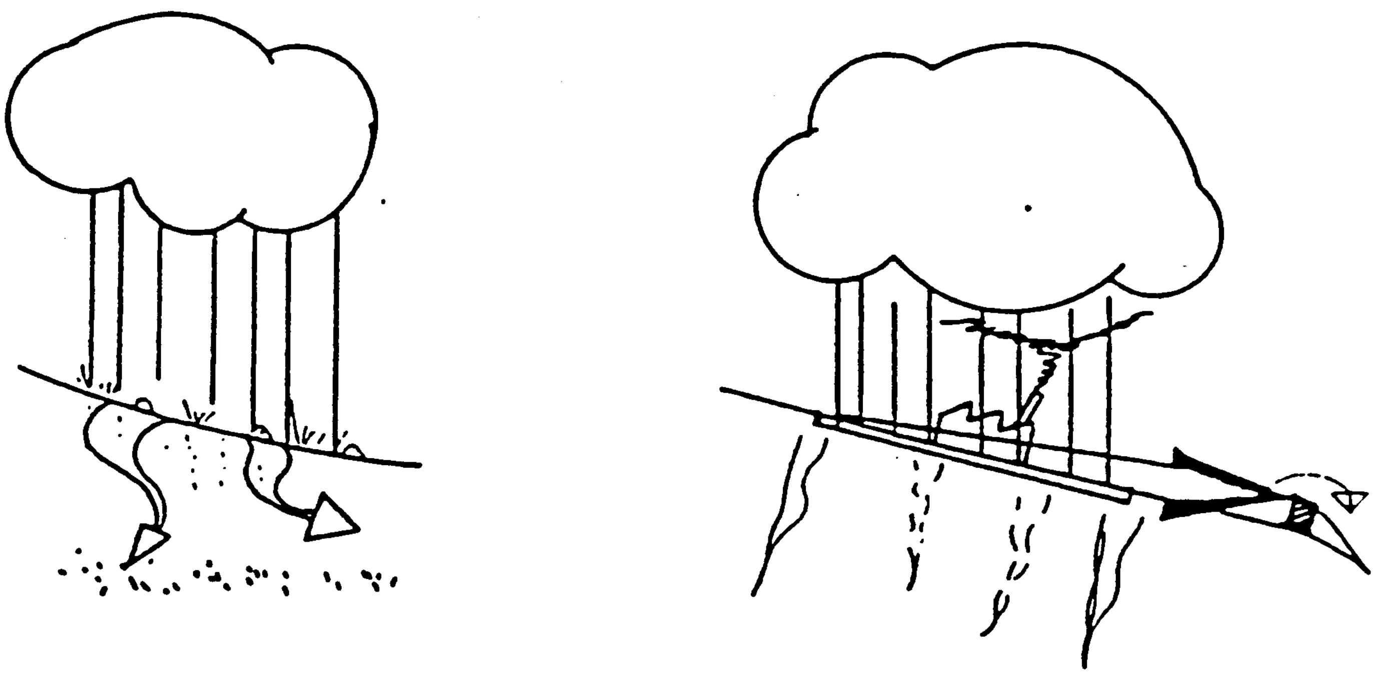


Fig. 3.1 The effect of urbanization on runoff.

EXAMPLE

Calculation of Peak Runoff for Various Conditions

The effect of urbanization on runoff can be illustrated with the following example. In particular it will be seen that the peak flows increase (as well as the volume of runoff).

i) **Virgin Catchment**

The simple rectangular catchment depicted in Fig. 3.2 will be studied to indicate the various effects of urban development on the storm runoff peak. The effects computed are reduced roughness, impermeable cover and channelization. A constant frequency, uniform rainfall intensity duration relationship as follows is ued:

$$i(mm/h) = \frac{a}{(0.24+t_d)^{0.89}}$$

where t_d is the storm duration in hours.

This is typical of a temperate area, and the value of 'a' for this region is estimated to be 70 mm/h for storms with a 20 year recurrence interval of exceedance.

The catchment is assumed to have a constant slope of 0.01 and initially the cover is grass. The representative Manning roughness for overland flow is estimated to be 0.1. The initial abstraction (surface retention and moisture deficit make up) is 30 mm and subsequent mean infiltration rate over a storm, 10 mm/hr.

Thus $\alpha = \sqrt{S}/n = \sqrt{0,01}/0,1 = 1,0$

Infiltration ratio $F = f/a = 10/70 = 0,143$

Initial loss ratio $U = u/a = 30/70 = 0,429$

Length factor in S I Units $LF = L/36\alpha a^{2/3} = 2000/36 \times 1 \times 70^{2/3} = 3,27$

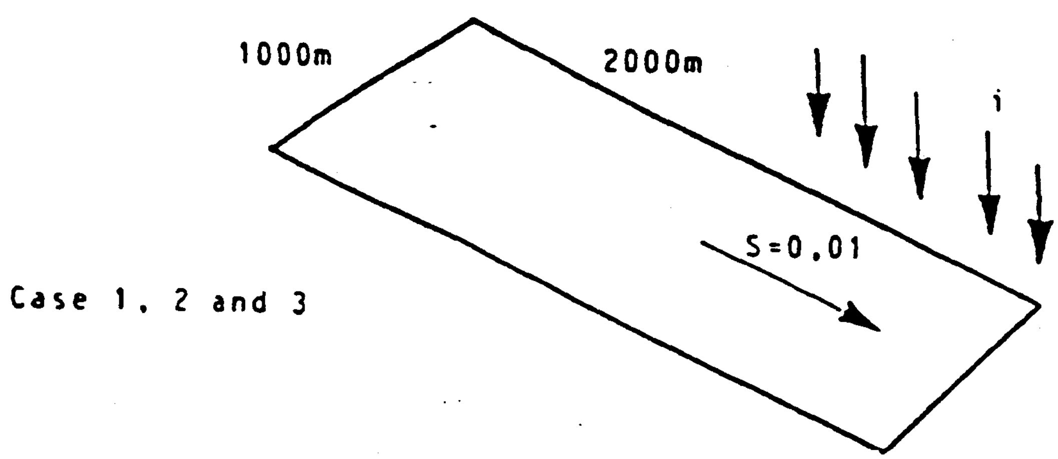


Fig. 3.2 Simple catchment analyzed

From Fig. 3.9 (for U = 0 40) read equilibrium t_e > 4h (off the graph) but the peak runoff factor for this F is QF = 0,23 which corresponds to a storm duration to t_d = 2 2h. The peak runoff rate is
$Q_p = 0{,}23B\alpha a^{5/3}/10^5 = 0{,}23 \times 1000 \times 1 \times 70^{5/3}/10^5 = 2{,}74 m^3/s$

The total precipitation rate over the catchment of area A for the same storm duration is

$$Ai = \frac{70 \times 1000 \times 2000}{(0{,}24+2{,}2)^{.89} \times 3600 \times 1000} = 17{,}6 m^3/s$$

so the rational coefficient C = 2,74/17,6 = 0,16.

Note however that the full catchment is not contributing at the time of peak runoff for the design storm, so C does not only represent the reduction in runoff due to losses, it also accounts for only part of the catchment contributing. The runoff for the full catchment would be less as the storm duration would be longer than 2,2 h so the intensity would be less and the losses relatively higher.

ii) **Reduction in Infiltration**

If the infiltration and initial abstractions are reduced by urbanization, the peak runoff increases. The construction of buildings and roads could reduce infiltration rate to 7 mm/h and initial abstraction to 14 mm. For F = 7/70 = 0,1 and U = 14/70 = 0,20 (Fig 3.9) then for LF = 3,27 as for case (i), the time to equilibrium if off the chart but the critical storm has a duration of 2,2 hours and the corresponding peak flow is

$$Q_p = 0,44 \times 1000 \times 1,0 \times 70^{5/3} / 10^5 = 5,24 m^3/s$$

The corresponding runoff coefficient C works out to be 0, 30

Case 4

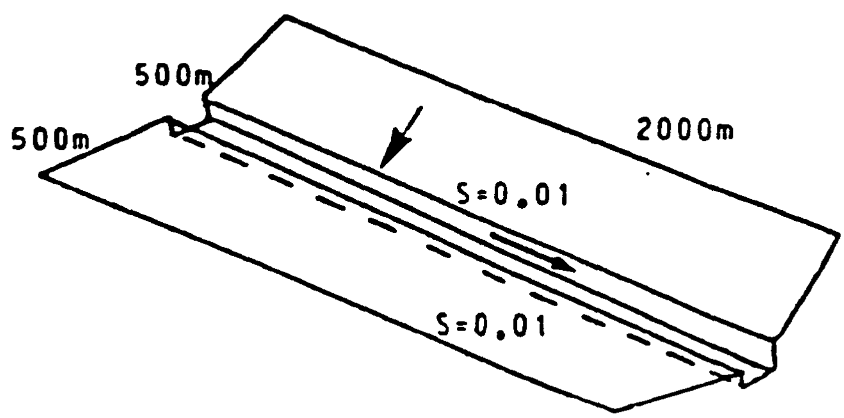


Fig. 3.3 Catchment with channel

iii) **Effect of Reduced Roughness due to Paving**

With the construction of roads, pavements and building the natural retardation of the surface runoff is eliminated and concentration time reduces. That is, the system response is faster and as a result shorter, sharper showers are the worst from the point of view of runoff peak. For the sample catchment the effective Manning roughness could quite easily be reduced to 0,03. Then α = 3,33 and LF = 0,98. the time to equilibrium would therefore be 3h but the peak intensity storm has a duration of 2,2h as before. In this case extent of the time equal to the critical storm duration.

TABLE 3.1 Showing effect of different surface configuration on peak runoff from a 2000m long by 1000m wide catchment.

$S_o = 0{,}01, \ i = 70 \text{ mm/h}/(0{,}24\text{h} + t_d)^{0{,}89}$

	CASE	n	f mm/h	u mm	t_ch	t_dh	i mm/h	Q_pm³/s	C
i)	Virgin Catchment	0,1	10	30	5	2,2	36,7	2,74	0,16
ii)	Reduced losses	0,1	7	14	4	2,2	36,7	5,24	0,30
iii)	Reduced roughness	0,3	10	30	3	2,2	36,7	9,12	0,52
iv)	Canalization (stream width 3m)	0,1	10	30	0,8	1,5	42,7	15,4	0,65
v)	Reduced losses and roughness	0,03	7	14	1,7	1,7	38,8	17,0	0,79

The effect of canalization is somewhat similar to reducing roughness - water velocities, and concentration rates, are faster. This is due to the greater depth in channels ($Q = B\sqrt{S}\,y^{2/3}/n$). Consequently a greater area contributes to the peak.

storm over the catchment is greater however, and the peak runoff is

$$Q_p = 0{,}23 \times 1000 \times 3{,}33 \times 70^{5/3}/10^5 = 9{,}12 m^3/s$$

The corresponding increase in C is from 0,16 to 0,52 an appreciable increase if it is borne in mind this is only due to reduced roughness and does not account for reduced infiltration. It will be noted that the effect of reducing roughness is even greater than decreasing infiltration for this case. The same effect is magnified in the following example.

iv) **Effect of Canalization**

The effect of a stream down the centre of the catchment is illustrated in the following example. The same surface roughness (n = 0,1) and permeability (f = 10 mm/h, u = 30 mm) as for case (i) are assumed. The overland flow cross slope is taken as 0,04 and 0,01 for a 8 m wide channel down the catchment. The dimensionless hydrographs are used again.

The stream catchment ratio $G = \left(\frac{2L_s}{b\alpha_s}\right)^{0.6} \frac{b\alpha_o}{2L_o}^{0.6}$

$$= \left(\frac{2 \times 2000}{8 \times 1}\right)^{0.6} \frac{8 \times 2^{0.6}}{2 \times 500} = 0{,}50$$

By trial, guess storm duration resulting in peak runoff of 1,5h, then

$$i_e = \frac{70}{(0{,}24 + 1{,}5)^{.89}} - 10 = 42{,}7-10 = 32{,}7 \text{ mm/h}$$

$$t_{ed} = t_d - t_u = 1{,}5 - 30/42{,}7 = 0{,}80h$$

$$F = 10/32{,}7 = 0{,}31$$

$$t_{co} = \left(\frac{L_o}{\alpha\, i_e^{m-1}}\right)1/m = \left[\frac{500}{2 \times (32{,}7/3600000)^{2/3}}\right]^{3/5} = 2860s = 0{,}80h$$

$T_D = (5/3)t_{ed}/t_{co} = (5/3)0,8/0,8 = 1,67$

Therefore $t_d = t_{ed} + t_u = 0,8 + 30/42,7 = 1,50h$ which agrees with guess.

Interpolating Figs. 3.10 and 3.11 the peak factor Q = 0,85

Peak flow $Q_s = QAi_e = 0,85x2x10^6x32,7/3,6x10_6 = 15,4m^3/s$

Rational coefficient C = 15,4/42,7x2/3,6) = 0,65

v) **Combined reduced roughness and reduced losses**

If roughness is reduced by paving to 0,03 then $\alpha = 3,33$ and LF = 0,98 as for case (iii). The reduced loss factors become F = 0,1 and U = 0,2 as for case (ii). From Fig. 3.5 $t_c = 1,7$ h and the corresponding QF = 0,43.

Hence the peak flow $Q = 0,43 \times 1000 \times 3,33 \times 70^{5/3} = 17,0m^3/s$. The rainfall rate for a storm of this duration is

$$\frac{70 \times 2000 \times 2000}{(0,24 + 1,7)^{0,89} \times 36000 \times 1000} = 21,6 \ m^3/s \text{ so } C = 0,79.$$

The relative effect of each variable on peak runoff can be compared with the aid of Table 3.1. The effect of reducing infiltration 30% and initial abstraction 40% is to double the peak runoff. The critical storm duration was not affected but the effective area contributing increased slightly. The effect of reducing surface roughness is even more remarkable however. Even maintaining the same losses (both initial and abstraction and infiltration) as for the natural catchment the runoff peak increased by a factor of 4. The area contributing increased noteably although the critical storm duration was not affected. Reducing roughness even more would not necessarily increase runoff mush as practically the entire catchment contributes for case (iii) whereas the area contributing in case (i) was much less. Only for case (v) with reduced roughness and losses is the concentration

Not much sense can be made out of comparing the resulting rational coefficients (ratio of peak runoff rate to rainfall rate times catchment area). That is because the time of concentration for each case is different due to differing roughness, rainfall rate etc. In any case it is irrelevant when it comes to critical storm duration which is shorter than the time to equilibrium.

DETENTION STORAGE

Although the kinematic equations presented previously cannot accommodate reservoir storage they may be rearranged to illustrate the storage components in them. The St. Venant equations which include terms for storage when water surfrace is not parallel to the bed are

$$\frac{\partial A}{\partial t} = - \frac{\partial Q}{\partial x} \tag{3.1}$$

$$\frac{\partial v}{\partial t} = g\ (S_o - S_f) - g\ \frac{\partial y}{\partial x} - v\ \frac{\partial v}{\partial x} \tag{3.2}$$

The first equation is the continuity equation and the second the so-called dynamic equation. The first equation does nto give the total storage in the reach, it represents the rate of change in cross sectional area of flow as a function of inflow and outflow. The second equation contains more about the distribution of storage. The last two terms represent the wedge component of storage, which are absent in the kinematic equations. The kinematic equations therefore treat storage as a prism, with storage in blocks and no allowance for difference in slope between bed and water surface is made. Since the second equation is replaced by a friction equation and $S_o = S_f$ in the kinematic equations, only the first equation in the case of the kinematic equations can be used to calculate storage changes.

The continuity equation may be written as

$$\frac{O-I}{\Delta x} + \frac{A_2-A_1}{\Delta t} = 0 \tag{3.3}$$

where O is outflow, I is inflow over a reach of length Δx, and A_1 and A_2 are the cross sectional areas before and after Δt respectively. If $O = (O_1 + O_2)/2$ and $I = I_1 + I_2$ and $A\Delta x$ is replaced by S, the storage which is a function of A_I and A_O, which in turn are functions of flowrate, e.g. $S = XI + (1-X)O$, then equation (3.3) becomes the one frequently used for open channel routing.

$$O_2 = c_1 I_1 + c_2 I_2 + c_3 O_1 \tag{3.4}$$

where c_1, c_2 and c_3 are functions of Δx and Δt. The latter equation is referred to as Muskingum's equation used in routing floods along channels. If X = 0 the routing equation corresponds to level pool or reservoir routing. the more general equation with X = 1/2 represents a 4-point numerical solution of the continuity equation as employed in kinematic models (Brakensiek, 1967).

CHANNEL STORAGE

Channel storage performs a similar function to pond storage in retarding flow, and there are many analogies which can be drawn between the two. Channel storage is a function of friction resistance and channel shape and can be controlled in various ways.

The form of friction equation, as well as the friction factor, affect the reaction speed of a catchment and the volume stored on the catchment. The excess rain stored on the catchment, whether in channels or on planes, is a form of detention storage, and as such, affects the concentration time and consequently the peak rate of runoff. Some friction formulae used in stormwater drainage practice are listed below.

	S.I. units	English units	
Darcy-Weisbach	$Q = (8/\lambda f)^{1/2} A(RSg)^{1/2}$	$Q = (8/\lambda f)^{1/2} A(RSg)^{1/2}$	(3.5)
Chezy	$Q = 0{,}55CA(RS)^{1/2}$	$Q = CA(RS)^{1/2}$	(3.6)
Manning	$Q = AR^{2/3} S^{1/2}/n$	$Q = 1.486AR^{2/3} S^{1/2}/n$	(3.7)
Strickler	$Q = 7{,}7A(R/k)^{1/6}(RSg)^{1/2}$	$Q = 7.7A(R/k)^{1/6}(RSg)^{1/2}$	(3.8)

R is the hydraulic radius A/P where A is the area of flow and P the wetted perimeter. R can be approximated by depth y for wide rectangular channels. S is the energy gradient, f is the friction factor and k is a linear measure of roughness analogous to the Nikuradse roughness.

Both the roughness coefficient and the exponent m or R or y in the general flow equation (3.11) affect the pak flow off a catchment. This is largely due to the attenuating effect of friction resulting in a larger time to equilibrium. A rainfall excess intensity-duration relationship is required to evaluate the effect of each coefficient on peak runoff rate and maximum catchment storage. The following expression for excess rainfall intensity is assumed:

$$i_e = \frac{a}{(c+t_d)^p} \qquad (3.9)$$

In this equation it is customary to express ie and a in mm/h or inches per hour and b and td in hours where td is the storm duration assumed equal to time of concentration tc for maximum peak runoff of a simple catchment

Starting with the kinematic equation for continuity

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i_e \qquad (3.10)$$

and a general flow resistance equation

$$q = \alpha y^m \qquad (3.11)$$

then it may be shown that $t_c = (L/\alpha i_e^{m-1})1/m$ where q is the runoff rate per unit width of the catchment and y is the flow depth. The rising limb of the hydrograph is given by the equation

$$q = \alpha (i_e t)^m \qquad (3.12)$$

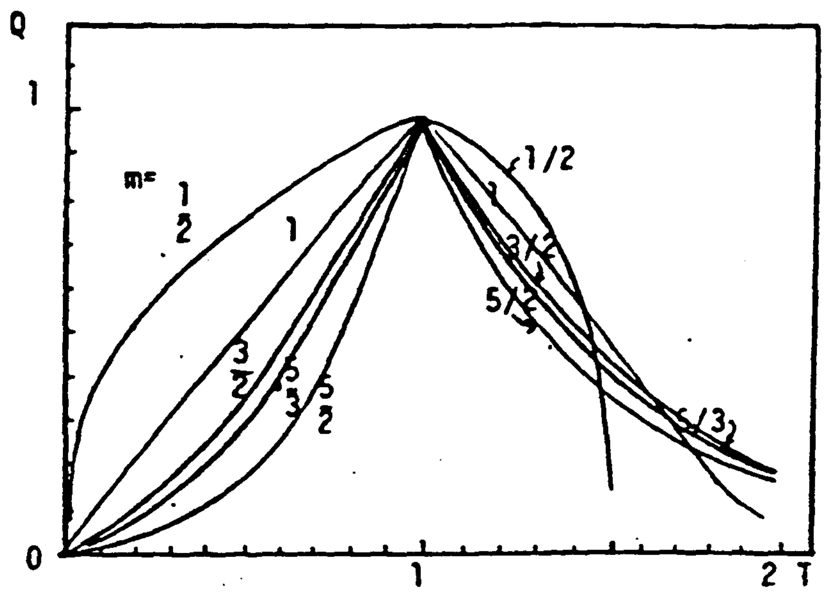


Fig. 3.4 Hydrograph shapes for different values of m in $q = \alpha y^m$

and another expression may be derived from the falling limb. In Fig. 3.4 are plotted dimensionless hydrographs to illustrate the effect of m on the shape of the hydrograph. The graphs are rendered dimensionless by plotting $Q = q/i_eL$ against $T = t/t_c$. m is used as a parameter. Thus m = 1/2 represents closed conduit or orifice flow, m = 1 represents a deep vertical sided channel, m = 3/2 represents a wide rectangular channel according to Darcy or a rectangular weir, m = 5/3 represents a wide rectangular channel if Manning's equation is employed, and m = 5/2 represents a triangular weir. The graphs immediately indicate the effect of m on catchment detention storage since the area under the graph represents storage.

The smaller m, the greater storage. Thus provided storage is economical by throttling outflow one may increase storage and increase concentration time thereby reducing discharge rate (which is not immediately apparent from these graphs as they are plotted relative to excess rainfall intensity). In practice the concentration time increases the greater the storage so that the lower intensity storms become the design storms. This has a compound effect in reducing flow rates since total volume of losses increases and it is possible that the entire catchment will not contribute at the peak flow time.

A general solution of peak flow and storagae in terms of intensity-duration relationships is derived below. Solving (3.9) with $t_d = t_c$ for maximum rate of runoff per unit area and generalizing by dividing by a,

$$q_m/aL = i_e/a = \frac{1}{\{c + \frac{(L/\alpha i_e^{m-1})^{1/m}}{3600}\}^p}$$

$$= \frac{1}{\{c + \frac{[L/\alpha(a/3600000)^{m-1}]^{1/m}}{3600(i_e/a)^{1-1/m}}\}^p} \qquad (3.13)$$

The term L/α_a^{m-1} is referred to as the length factor. The constants are introduced for a in mm/h, and time of concentration in hour units. The maximum peak flow factor i_e/a is plotted against length factor in Fig. 3.5, since it is not easy to solve (3.13) directly for i_e/a.

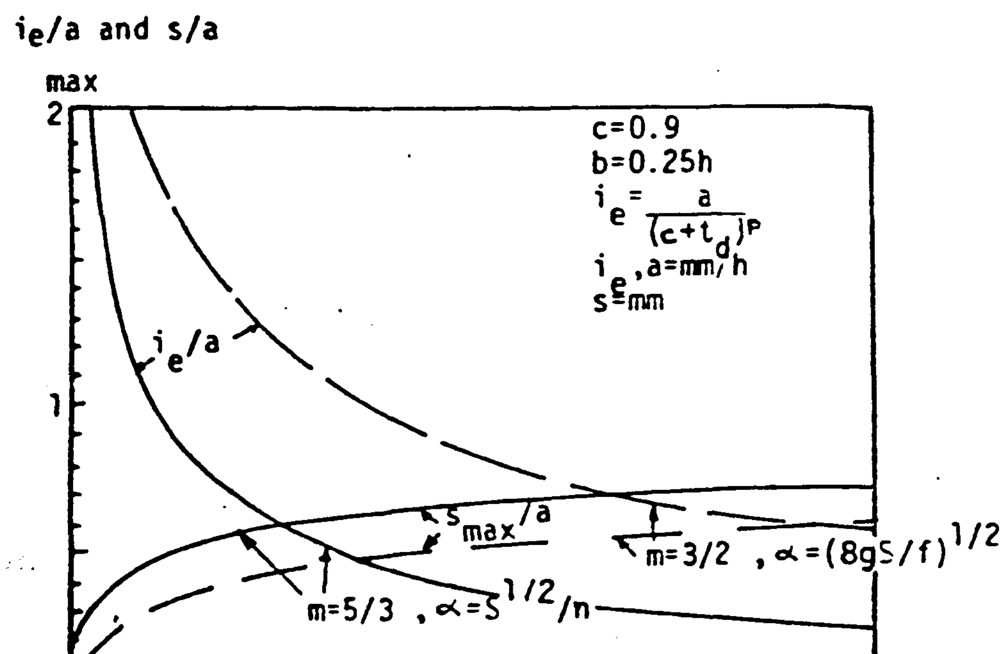


Fig. 3.5 Peak flow and storage versus length factor

An expression for the corresponding catchment storage is derived below. At equilibrium the flow per unit width at a distance x down the catchment is

$$q = i_e x$$
$$= \alpha y^m$$

therefore $y = (i_e x/\alpha)^{1/m}$

Integrating y with respect to x yields the total volume on the catchment

$$V = \frac{Lm}{m+1} (i_e L/\alpha)^{1/m}$$

or in terms of the average depth of storage s = V/L

$$s/a = \frac{m}{m+1} \left(\frac{i_e}{a}\right)^{1/m} \left(\frac{L}{\alpha (a/3600000)^{m-1}}\right)^{1/m} \frac{1}{3600} \qquad (3.14)$$

where s is in mm, and i_e and a are in mm/h. s/a is also plotted against length factor in Fig. 3.5. It will be observed that average storage depth does not increase in proportion to $L/\alpha a^{m-1}$. In fact the rate of increase reduces beyond $L/\alpha a^{m-1} = 50$, and the rate of reduction in peak flow i_e/a also decreases beyond the figure, indicating reducing advantage in increasing channel length or roughness ($\alpha = K_1 \sqrt{(S)}/n$). Since total channel cost is a direct function of storage capacity it would appear to be an optimum at some intermediate value of $L/\alpha a^{m-1}$ if there is a cost associated with peak discharge e.g. culverts or flooding downstream (see Fig. 3.6).

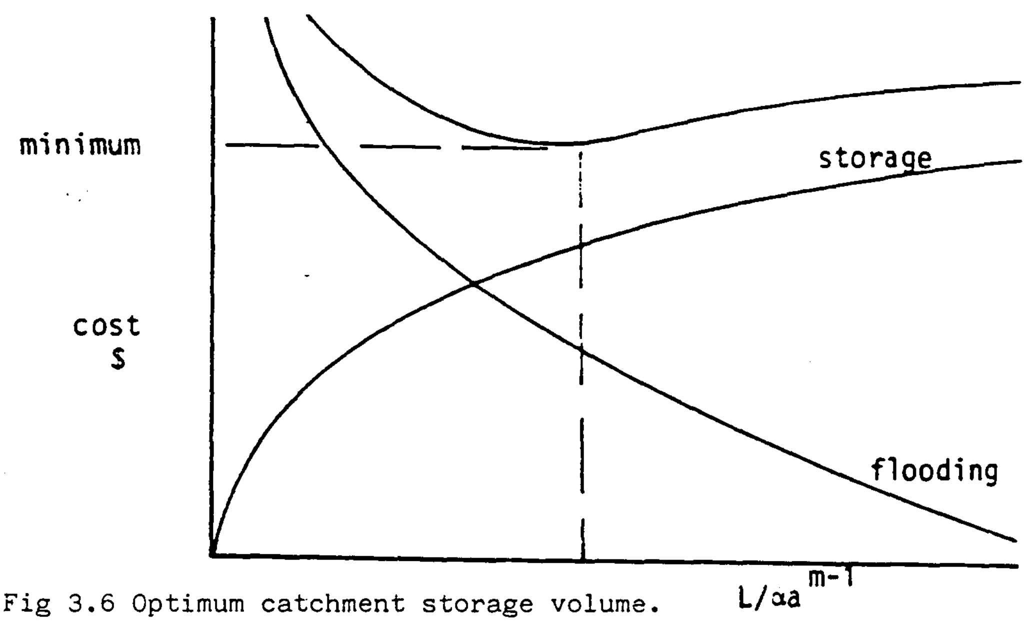


Fig 3.6 Optimum catchment storage volume.

Note that infiltration after the rainfall stops, is neglected in the above analysis. Inclusion of that effect would lower the i_e/a and s/a lines to the right, implying a larger $L/\alpha a^{m-1}$ is best. The model provides an indication of total storage in the system. The location (and volume) of storage could be further optimized using dynamic programming methods or by detailed modelling. It should be found generally that it is most economical to provide pond storage ($m = 1/2$) at the outlet, whereas channel or catchment storage ($m = 5/3$) is most economical at the head of the system.

Kinematic Equations for Closed Conduit Systems

If the open channel kinematic equations are applied to closed conduit flow the problem becomes a steady state flow one since flow rates become independent of cross section. This is provided the conduits remain full and there are no storage ponds at nodes joining conduits. If one permits storage variation at nodes one has the reservoir-pipe situation encountered in water supply which is often analyzed employing pseudo-steady flow equations.

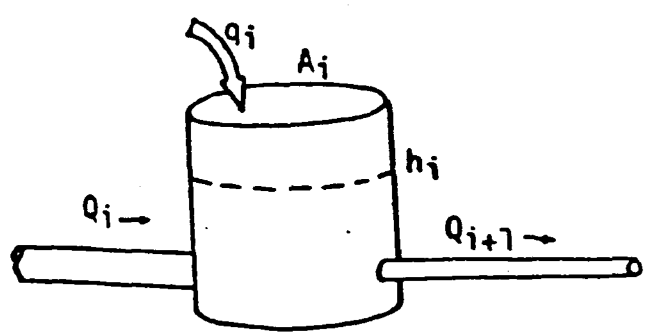


Fig. 3.7 Input-output node storage

The continuity equation becomes (see Fig. 3.7)

$$(Q_{i+1}-Q_i)-q_i + A_i \frac{dh_i}{dt} = 0 \tag{3.15}$$

where the reservoir surface area A_i replaces B dx in the open channel continuity equation where B is the catchment width. q is the reservoir inflow here. The dynamic equation is replaced by

$$Q_i = \alpha A^m \tag{3.16a}$$

where A is the (constant) conduit cross sectional area. Since the kinematic equations omit the dependency of Q on head difference h, the latter equation assumes the head gradient along the pipe equals the pipe gradient, i.e. free-surface just full flow. Since A is a constant it is relatively easy to replace the last equation by one of the form

$$Q_i = \alpha A h_i^{\,m} \tag{3.16b}$$

This equation is applicable to free discsharge from an orifice or over a weir. One more applicable to conduit flow would be

$$Q = \alpha A(h_{i-1} - h_i)^m \tag{3.16c}$$

Any one of the above three equations could be applicable in stormwater drainage. For channel or overland flow (3.16a) applies, for complete storage control (3.16b) applies and for closed conduit control (3.16c) is applicable. The latter form of equation has in fact been employed in water reticulation pipe network analysis. It can be applied in storm drainage to closed systems (not of great interest in stormwater management practice) or to pipe-reservoir problems. Surface detention and artificial detention storage ponds can be handled in an overall flow balance employing the closed conduit kinematic method. It should be noted that the numerical instability problems associated with solution of the open channel kinematic equations are absent. Time steps can be much larger than for open channel kinematic modelling. Storage fluctuations may be computed in steps and the effect ofchanges in pond water levels on flows in conduits can be accounted for.

One possible application of such a program is to an inter-connected pond system with reversible flows in conduits. Overload from one pond can be forced back to another pond. Such situations can readily arise from spatially variable storms and possibly for travelling storms.

Off-channel storage can also be accounted for. Such ponds have the advantage that water level variations are not as marked as the head variations in the drain pipes (which may in fact be surcharged). This is due to the reversible head loss between the main conduit and the pond.

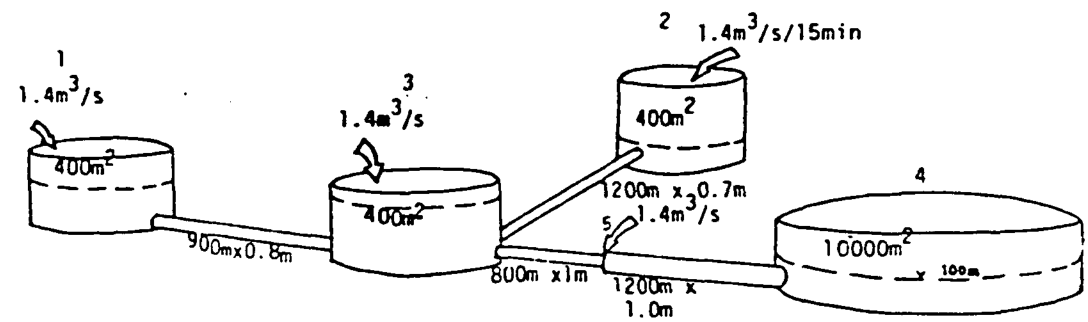


Fig. 3.8 Conduit and storage storm drain network

The simplified layout in Fig. 3.8 was analyzed employing the accompanying kinematic closed conduit continuous simulation program. Input and output are appended to illustrate the simplicity in this type of analysis. Flow reversal, pond level variations and the large attenuation in peak flow will be observed due to the ponds (from 5,6m^3/s down to 1,5m^3/s). By adjusting individual pond areas and conduit sizes an optimum design could be achieved for any design storm input. A sensitivity analysis for alternative storms such as different storm durations or ones with spatial variability would then be performed.

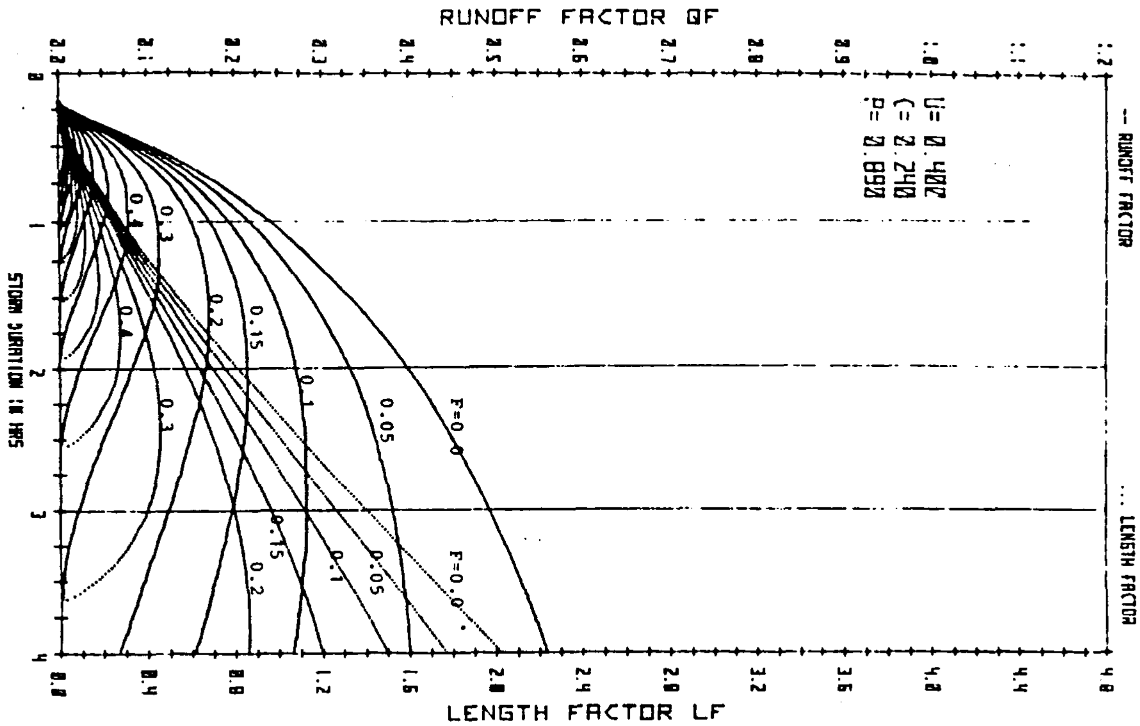


Fig. 9a Peak runoff factors for inland area with U = 0.20

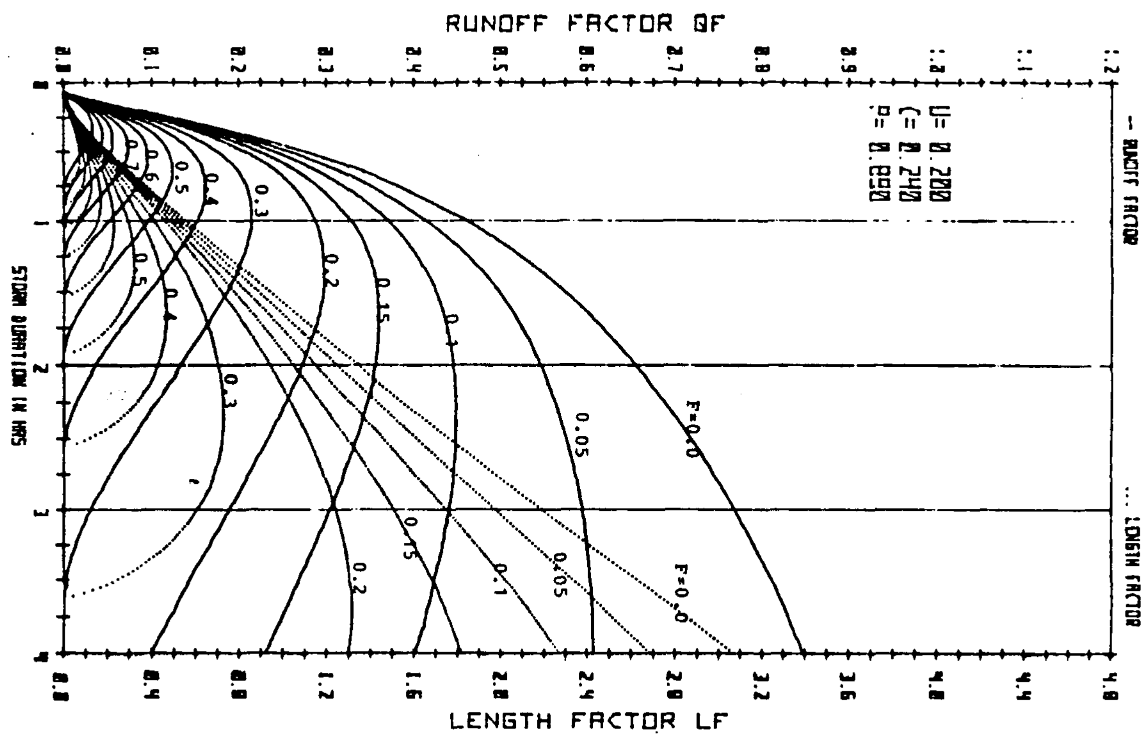


Fig. 9b Peak runoff factors for inland area with U = 0.40

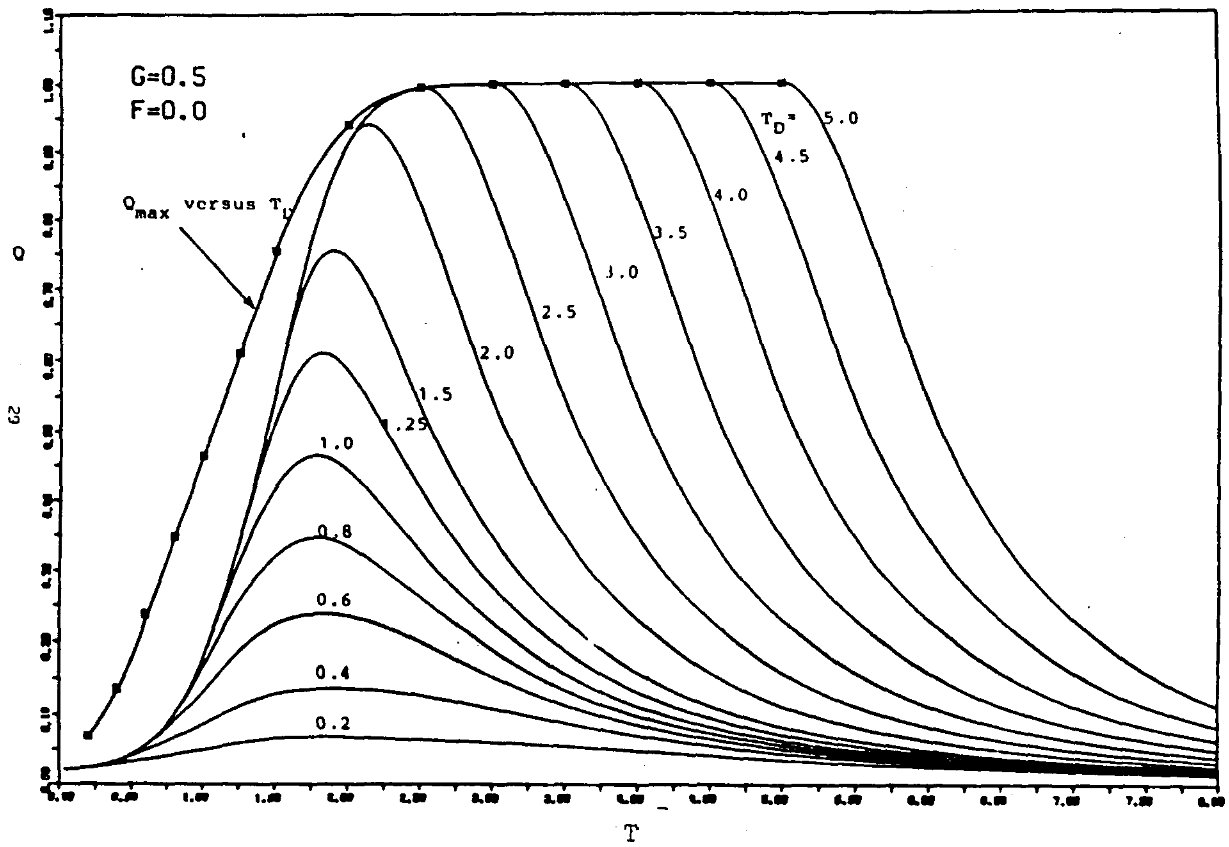


Fig. 10 Dimensionless runoff hydrograph for the V-shaped catchment with stream $G = 0.5$ $F = 0.0$

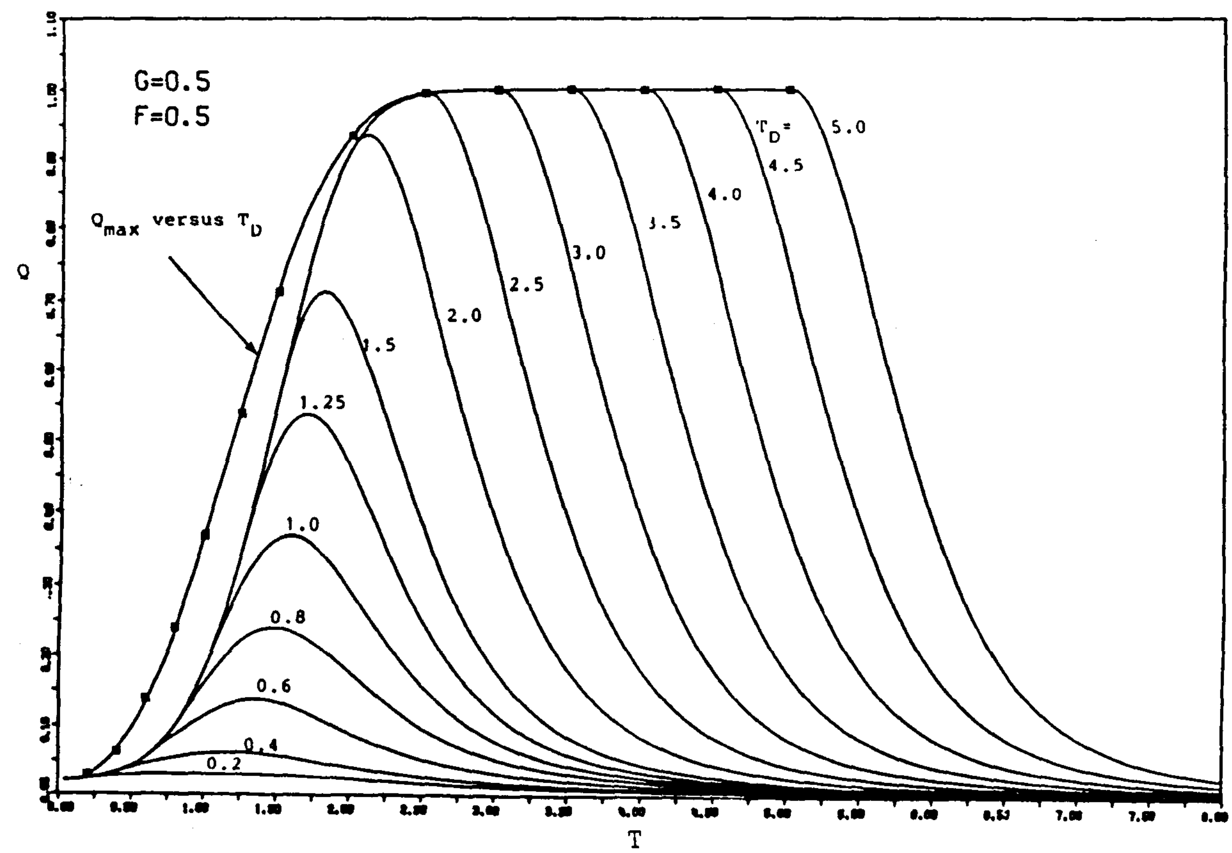


Fig. 11 Dimensionless runoff hydrograph for the V-shaped catchment with stream $G = 0.5$ $F = 0.5$

CHAPTER 4

DUAL DRAINAGE

INTRODUCTION

It appears that dual drainage systems can considerably reduce flood risks when compared with single conduit urban storm drains. Underground pipes shoud 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.

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.

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 imperviousness, 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 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 systems 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 systems 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.

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 surcharge 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 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 $90m^3/sec$ 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 developes 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 street is severe and the flood plains at the outlet of the catchment are inundated. A marked reduction in the peak occurs, namely 30% and the hydrograph is attenuated (Figure 2). On the other hand, detention storage is 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 previous surfaces.

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 estimation is buffered. This is not the case with closed conduits where a doubling in flow could increase the head requirement by 30%, resulting in surcharging and overflowing or backing up at the inlets.

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 inconvenience. Open channel type overflow conduits are also more versatile than close conduits where flows exceed those anticipated.

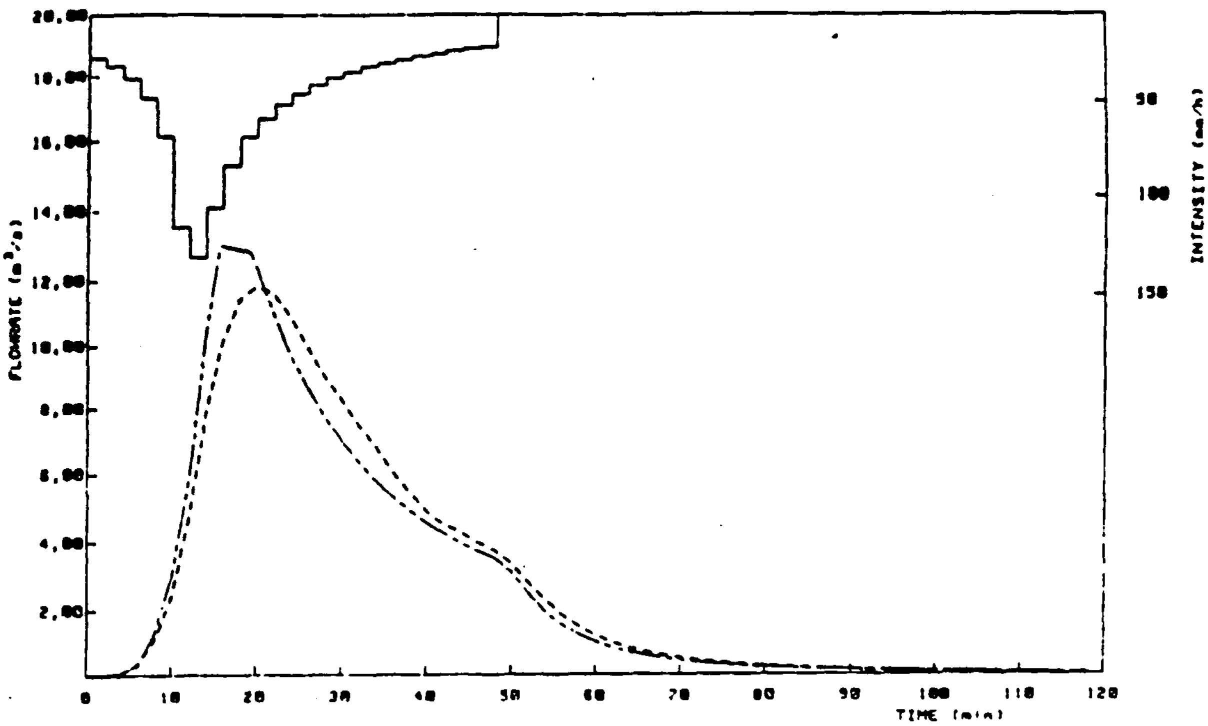


Fig. 4.1 Comparison of hydrographs from Hillbrow catchment for 20 years CHICAGO design storm

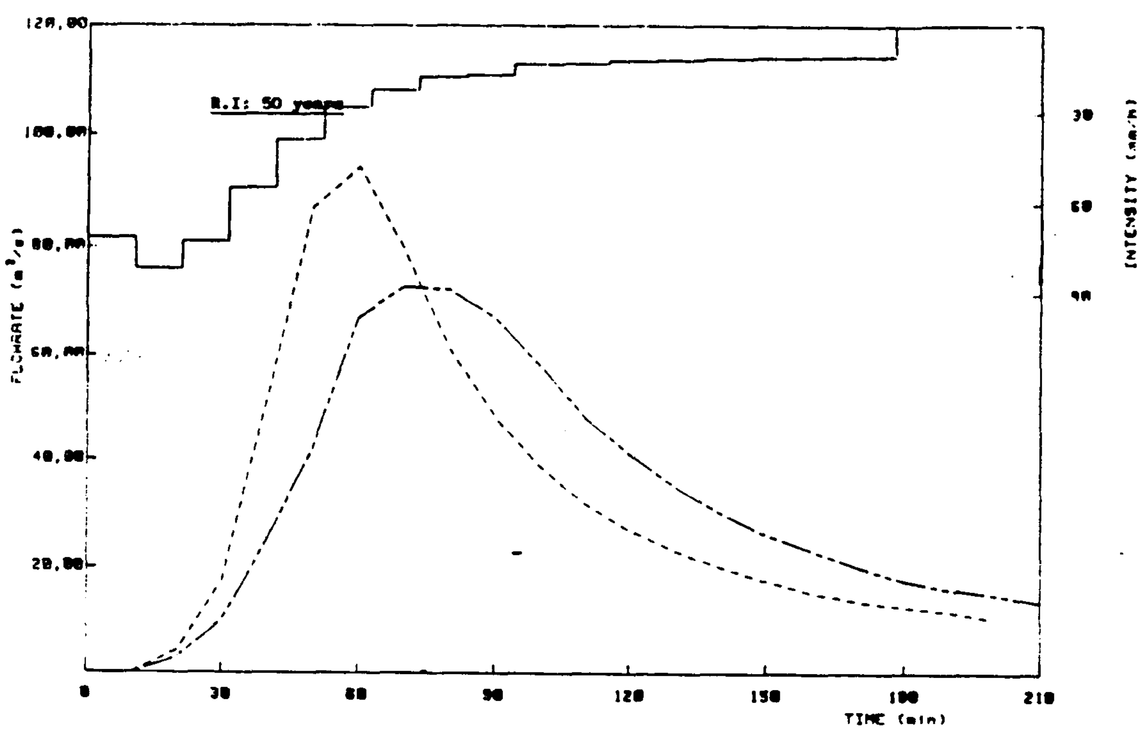


Fig. 4.2 Comparison of hydrographs from Braamfontein Spruit catchment for 50 years HUFF design storm

CHAPTER 5

STORMWATER MANAGEMENT PROGRAM

A micro computer catchment modelling program designed for stormwater management studies is useful for studying the effects of urbanization. The program described is a kinematic type model, that is 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, i.e. alternative detention or diversion systems to reduce peak flows and increase catchment recharge.

The kinematic equations limit the hydraulic capability i.e. backwater computations are not possible, but the resulting equations are suitable for rapid analysis on micro computers. 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 flood plain storage.

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 (Overton and Meadows, 1976). The paper is concerned primarily with a model for estimating stormwater runoff from urbanized catchments (although the principles have been applied to rural catchments (Stephenson, 1986)). Programs used in South Africa for stormwater computations include:

(i) ILLUDAS or its variants (Watson, 1981), a program based on time area methods i.e. a development of the rational method with unique concentration or travel times for each sub-catchment.

(ii) WITWAT (Green, 1984) which employs the kinematic method for overland flow and was fitted with S.A. rainfall data. It is a simple to use model for design or analysis of stormwater networks.

(iii) SWMM (Huber et al, 1982) A mainframe program recently adapted for use on micro computers. 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, e.g. OTTHYMO (Wisner, 1980) 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 e.g. unit hydrograph, or conceptual models such as Diskin's (1986) cell model.

Sub catchment arrangement:

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

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 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.

(ii) 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-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 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 being important aspect on water resources planning (Dept. Water Affairs, 1986). That is, from a long term point of view catchment yield is influenced by vegetation cover and land usage.

For single events e.g. floods, stormwater management is now also recognized as being important (Wanielista, 1979). Dainage engineers are now aware of methods of attenuating floods e.g. with detention storage basins or dual drainage systems (Stephenson and Kolovopoulos, 1987). Such techniques should therefore be accomodated in models.

The necessity for groundwater recharge in order to maintain an adequate water table also supported the idea of stormwater soakaways and retention storage. Fàcilities for studying groundwater fluctuations would therefore also be desirable in models. To go a step further, there frequently occurs a temporary perched water table 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 townships development and flow rates increased when more intense development occurred e.g. subdivision of stands, extension of city limits, increased cost of flooding. At that stage is 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 desgined to provide accessible facilities to study alternative stormwater management methods.

General Comments

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 plain 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.

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 which control catchment runoff are employed. These are the continuity or mass balance equation

$$B \frac{\partial y}{\partial t} + \frac{\partial Q}{\partial x} = q_i \tag{5.1}$$

and a flow depth/discharge relationship e.g.

$$Q = A\alpha y^m \tag{5.2}$$

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 i.e. type or even connectivity. Random numbering or ordering of data make 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 i.e. water depths are calculated assuming flow rates are known. Using a kinematic program it is the flows which 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 sub-routine and can be used to initiate water depths. For 2-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 micro computers and is one of a suite of programs, WITWATER, under development by the Water Systems Research Group at the University of the Witwatersrand (Fig. 5.1).

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 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 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 (Holden and Stephenson, 1988).

The wave diffusion can be accounted for 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 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 (Brackensiek, 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 t 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 circuit or confined aquifer.

MANAGEMENT CAPABILITY

A drawcard of the model prepared on 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 townships layout could be varied until a suitable stormwater drainage pattern emerge.

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 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. By means of a general discharge equation of the form

$$Q = WAy^m$$

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 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. 5.3) :

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 (14) is assumed.

Catchments can be linked in cascades (in series) for example 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 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, side 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 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 3600m x 100 m 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 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 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 underlying 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 (Fig. 2). 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.

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 e.g. as high as 1000mm/h for sand down to as low as 1mm/hr for clay. 5 to 10mm/h is most typical of highveld catchments.

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

Manning roughness can be normal hydraulic values for conduits e.g. 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

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

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

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

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, $\alpha \approx 0{,}6A\,/2g\,/W$ 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. 5.4 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 Witwatersrand. 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. 5.5)

1. Disconnecting most impervious areas
2. Re-arranging road plan
3. Adding flood plain 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, i.e. overflow into the streets already is 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 on 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 5.1 summarizes the results of the studies. The effectivness of each management method is ranked below in order of effectiveness for (a) a 2 year storm, and (b) a 10 year storm.

TABLE 5.2 Ranked Stormwater Management Methods

	(a) 20 year storm	(b) 10 year storm
1.	Detention storage	Road layout
2.	Road layout	Detention storage
3.	Flood planes	Disconnect impervious
4.	Disconnect Impervious	Flood planes
5.	Dual drainage	Dual drainage
6.	Existing drains	Existing drains

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

(i) To show that there are many alternatives which could be considered in stormwater drainage planning.

(ii) 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 accounted for using such models.

TABLE 5.1 Modelled Effect of stormwater management methods in Sunninghill.

	Method	2 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.	Flood plane & channel storage	(1,5)	5,1	Low here
5.	Detention storage $1000m^3$	1,2	4,4	Medium here
6.	Dual drainage	1,9	(5,3)	Medium

START UP

SUBSIDIARY DATA

MODELLING PROGRAM

QUALITY

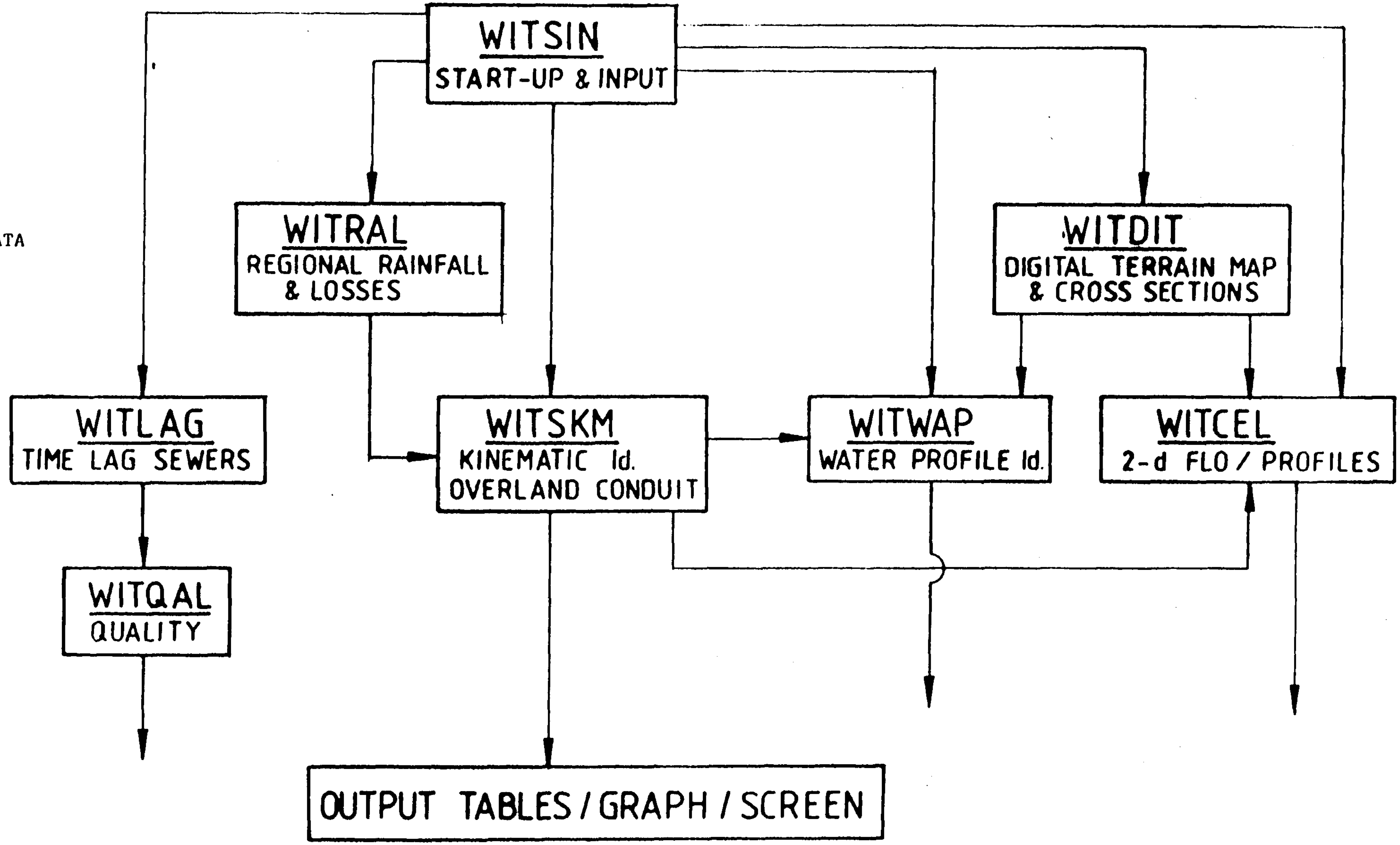


Fig. 5.1 WITWATER Suite

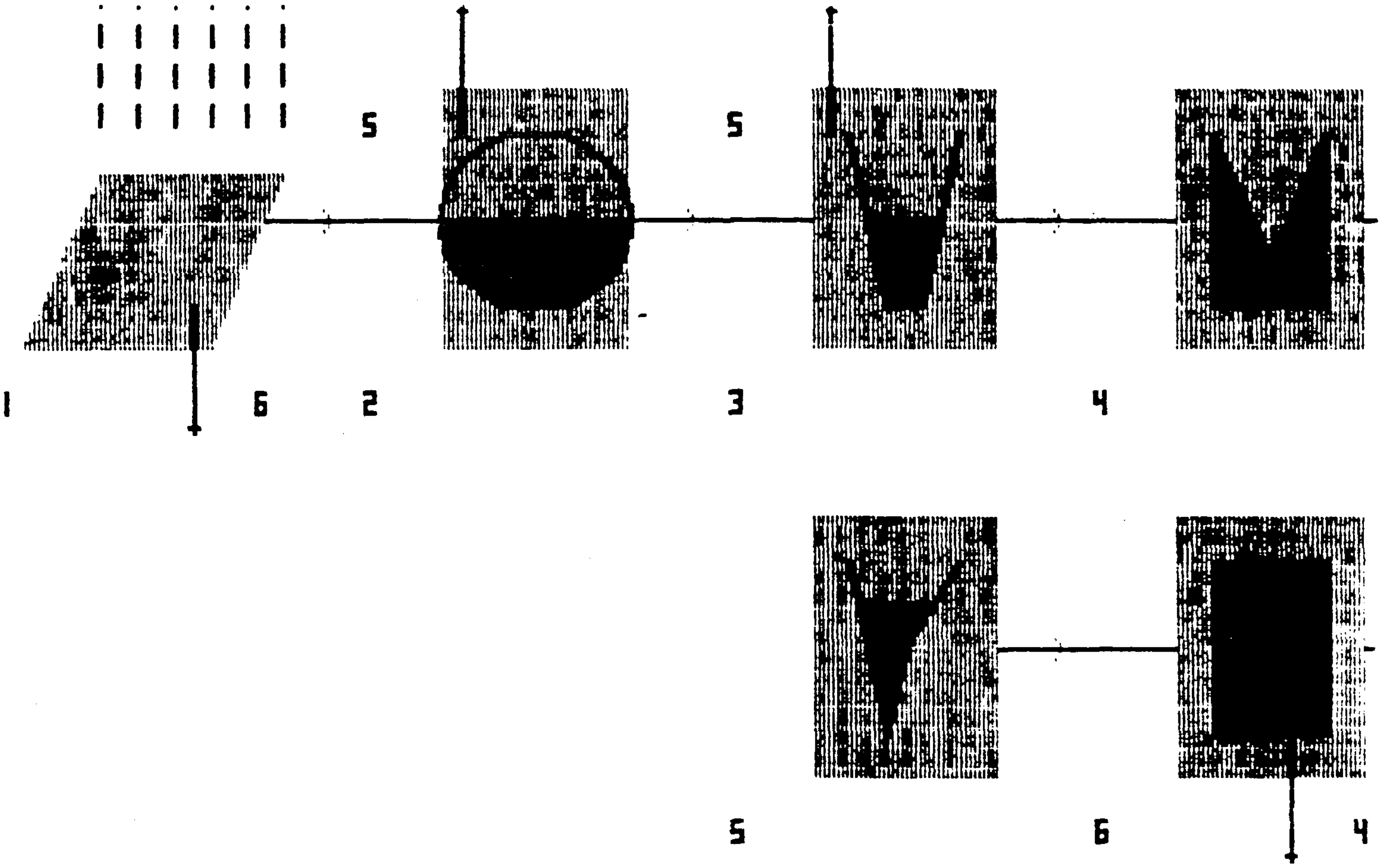
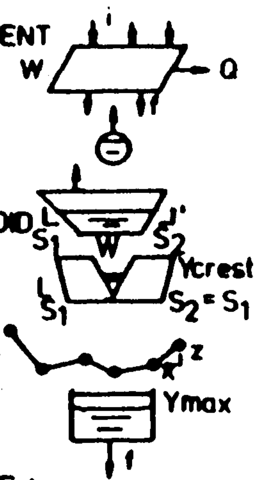


Fig. 5.2 Screen Graph ICS for Connectivity check in WITSKM

DATA FILE



	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.	Infil 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	Infil to
LAST LINE:	0										

TERMINAL INPUT

1. NAME OF ANALYSIS, TYPE OF INPUT
2. TIME INTERVAL (h) BETWEEN COMPUTATIONS
   TOTAL SIMULATION DURATION, HOURS
3. RAIN DURATION h,
   RAINFALL RATE mm/h
4. UNIT FOR WHICH HYDROGRAPH IS TO BE PLOTTED,
   MAX ORDINATE OF HYDROGRAPH m^3/s

Fig. 5.3 Data input for WITSKM

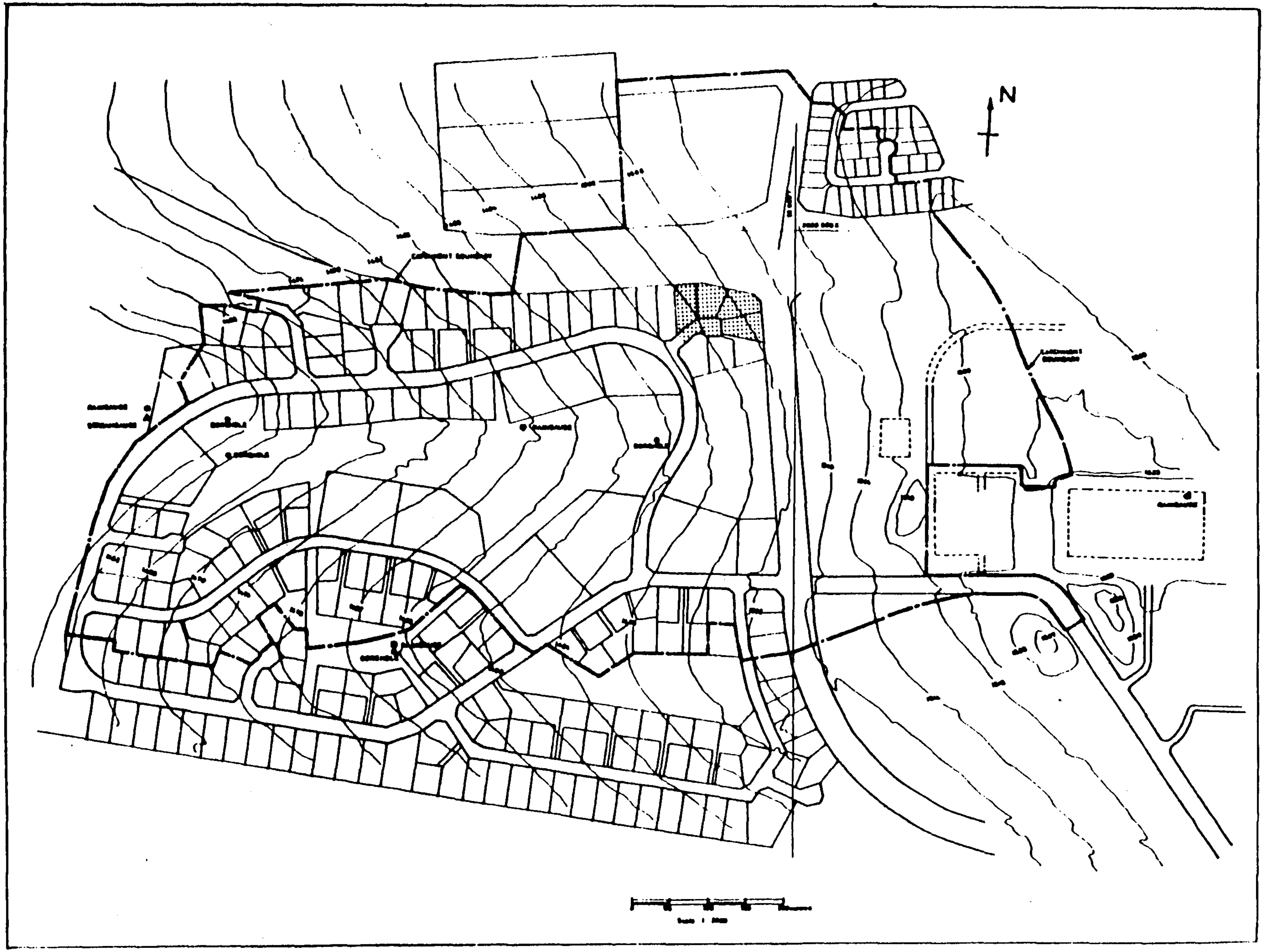


Fig. 5.4 Sunninghill - Sandton Locality Plan Showing Study Catchment

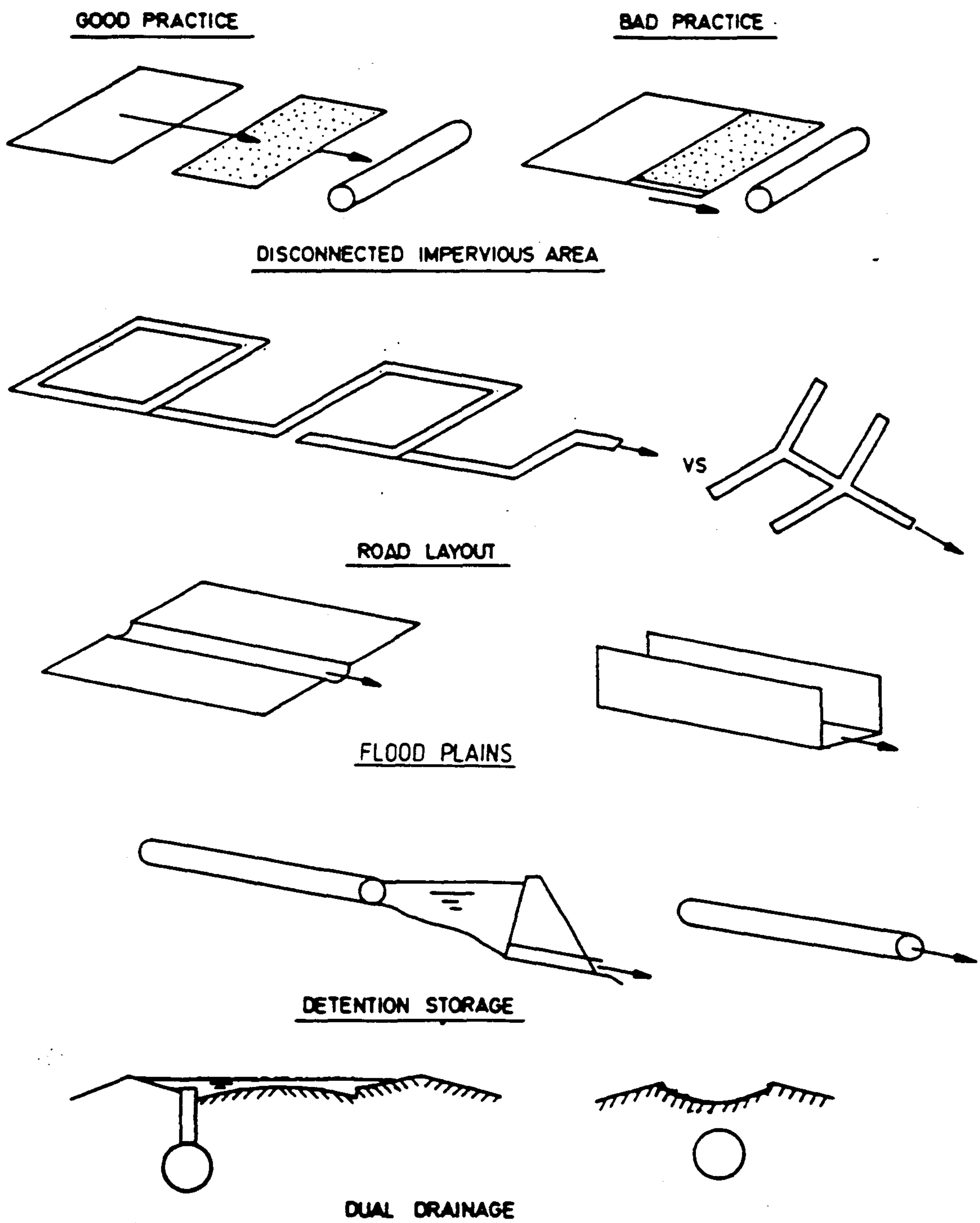


Fig. 5.5 Summary of Stormwater Management devices

CHAPTER 6

SELECTION OF STORMWATER MODEL PARAMETERS

INTRODUCTION

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 which can be determined by measurement include catchment area, slope and length of flow path. The dependency of parameters on level of model discretization was investigated. Experiments were conducted on a small urban catchment and results of different levels of discretization were compared with observed hydrograph volume and peak runoff. The only factor found to require adjustment depending on the level of discretization and size of sub-catchment was the overland flow time. Small-scale discretization required a longer flow path of a higher Manning roughness coefficient than coarse discretization to predict lag times correctly.

Computer models have become the most suitable method of estimating stormwater flows in urban areas. The accuracy of hydrologic estimates is improved by models which can accommodate a large number of elements and variables. Other reasons computer models are used include the ease with which paramters can be changed, sensitivity studies performed and the simplicity of comparing alternative designs (Alley et al, 1980, Cousens et al, 1976 and Hughes et al, 1987).

The accuracy of computer simulation models is dependent on suitable calibration of the model (Lane et at, 1976, Pilgrim, 1975 and 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 runoff 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 agrees with those observed.

The length of flow path, the roughness of the conduit or the overland flow surfaces and land slope are usually interdependent and adjustment of only one of them for any particular conduit or plane will probably be adequate to get 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 adjust 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 surface, nor, in particular, to separate directly connected impervious surfaces from those surfaces which discharge onto permeable surfaces. Often a balance or an accurate enough 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 which simplify modelling this phenomenon. For instance the stormwater drainage engineer is inclined to separate surface flow from subsurface flow and 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 Cell model (Diskin et al, 1978), there are in fact many similarities among hydrologic models and hydraulic or physical models. Sophisticated hydrologic models may use unit hydrographs or simulated unit hydrographs using Fourier series such as the Hymo Model (Wisner, 1980). These models are usually 'lumped variable' models i.e. the catchments or subcatchments used are of a relatively large extent and include various types of soil cover and varying topography in some cases.

On the other hand the smaller the subcatchments become, the more it tends to become a hydraulic type model. Hydrologists may call a hydraulic model a 'distributed parameter' model because they are 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 et al, 1981). This is not to be confused with the degree of 'lumping' or averaging of parameters though.

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

DIFFERENCES BETWEEN HYDROLOGIC AND HYDRAULIC PARAMETERS

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 (Green and Ampt, 1911), which is still a simplified model, there are a number of factors to be estimated before a model can be calibrated. The permeability, initial moisture content, and soil suction are at least required. The Horton model (Horton, 1933), a more empirical infiltration model requires an initial infiltration rate and a final infiltration rate; and it 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 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 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 watersheds 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 with figures quoted in hydraulic literature is the roughness coefficient. The Manning equation for velocity which is 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 which would be conceived in hydraulic research and therefore 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 the 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 is given by $R = vy/\nu \simeq 3 \times 1/10^{-6} = 3 \times 10^{6}$. In an overland flow situation $R \simeq 0{,}5 \times 0{,}01/10^{-6} = 5 \times 10^{3}$. The Darcy 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 4 between 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 impact of raindrops on overland flow for example, is to increase turbulence and therefore create higher headloss (Overton et al, 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 considerbaly 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 of an equilateral right angle triangle). In addition downpipes and other vertical flow paths are not accounted for 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 is estimating travel time. One could therefore increase the overland flow length or the Manning roughness coefficient in order to correct the lag time. The longer overland flow path would also mean 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 which 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 are two compensating factors in the flow-time calculations 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

$$C = m\alpha y^{m-1}$$

where c is the wave speed. α is the conveyance, y is the flow depth and m is an exponent which 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. 6.1 Shows the effect of chanelization. Employing Manning's equation

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

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. By reducing the effective width (case 4) the depth increases further but the velocity reduces compared with case 1.

The net effect is that the velocity reduces if the water flows in rills despite the increased flow depth.

MODEL STUDY

Kerst (Kerst, 1987) modelled an urban catchment (Sunninghill) north of Johannesburg. He looked at 4 levels of discretization, referred to as Coarse, Medium, Fine and Very Fine. The corresponding average flow path lengths were 115m, 47m, 15m, and 7m. Maps for the Coarse and Very Fine studies are given in Figure 6.2 and 6.3.

The model used was WITWAT (Green, 1984). This model uses kinematic theory for overland flow and time lag for conduits (Stephenson, et al, 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 5 autographic raingauges existed and detailed maps were available. The area isolated for this study was 0,74 ha, comprising four residential properties with single storey single family houses.

The surface flow from the study catchment is discharged into a road gutter and thence into subsurface stormwater drain pipes. Flow was measured by measuring water depth in the gutters and assuming uniform flow. The Manning equation was used to estimate flow rate. Flow depths were measured at 5-minute intervals for the observed flow events for which the observer was able to reach the catchment in time. The maximum flow rate event which occurred on 20 December 1986 was chosen for analysis and simulations. The storm duration was 3 000 secs, with a depth of 26 mm and peak intensity of 120mm/h. Rain was measured by a tipping bucket raingauge with a tip of 0,2mm. 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), for pervious area (0,25), initial infiltration rate (75m/h), ultimate infiltration rate (10m/h), and depression storage for impervious area (1mm) and pervious area (3mm) averaged over the four levels of discretization. A computational time step of 60 seconds was also found to be the most efficient. Each parameter was then varied methodically and the effect on peak flow and volume of runoff were compared with measured values.

Figure 6.4 shows how the fits were obtained for the Coarse discretization case.

Figure 6.5 presents the results for Coarse, Medium, Fine and Very Fine discretization. It shows the results are most sensitive to infiltration rate. A 50% variation in infiltration rate affects the peak runoff by up to 20% (this for medium discretization) and affects volume of runoff by up to 30% (this for fine discretization). The next most sensitive parameter was roughness of pervious area. A 50% variation alters runoff peak by up to 20% (this for medium discretization), but volume is affected by only up to 7% (all levels of discretization). Other factors being equal the finer discretized model reproduced runoff values better than coarser models, but the coarse models reproduced peak runoff rate better. The resulting two values of roughness for peak rate and volume prediction were closest for fine level discretization.

Figure 6.6 shows the effect of level of discretization on best fit parameters as obtained by calibration. The values were obtained from Fig. 6.4 and similar graphs for other levels of discretization. The Manning roughness coefficient must be increased slightly with finer discretization, or the length factor should 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 level of discretization with slightly better volume prediction for finer levels of discretization.

CONCLUSIONS

Stormwater parameters which cannot be measured directly are infiltration rates and subsurface 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 finely discretized model predicted runoff volumes more accurately than coarse 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 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 sub-catchment size, the bigger the Manning 'n' which 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 sub-catchments. i.e. the centre 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 of 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 modelling.

TABLE 6.4 Optimum Value of Manning "n", Initial Infiltration Rate and Length Factor for Rate of Runoff and Volume of Runoff for Various Levels of Discretization

<table>
<tr><td colspan="2">Manning "n" pervious</td><td colspan="2">Initial Infiltr. Rate f mm/hr</td><td colspan="2">Length factor</td><td rowspan="2">Level of Discreti-zation</td></tr>
<tr><td colspan="2">Initial Infiltr. = Standard Value Length factor = 1.0</td><td colspan="2">Manning "n"-pervious = Standard Value</td><td colspan="2">Initial Infiltr. Rate & Manning "n"-pervious = Standard Values</td></tr>
<tr><td>0.170</td><td>0.110</td><td>45.2</td><td>64.1</td><td>3.5</td><td>1.6</td><td>Very fine</td></tr>
<tr><td>0.186</td><td>0.061</td><td>59.7</td><td>47.9</td><td>3.0</td><td>0.9</td><td>Fine</td></tr>
<tr><td>0.260</td><td>0.048</td><td>73.0</td><td>61.6</td><td>2.1</td><td>0.7</td><td>Medium</td></tr>
<tr><td>0.185</td><td>0.027</td><td>48.4</td><td>53.0</td><td>2.0</td><td>0.65</td><td>Coarse</td></tr>
<tr><td>Rate</td><td>Volume</td><td>Rate</td><td>Volume</td><td>Rate</td><td>Volume</td><td></td></tr>
</table>

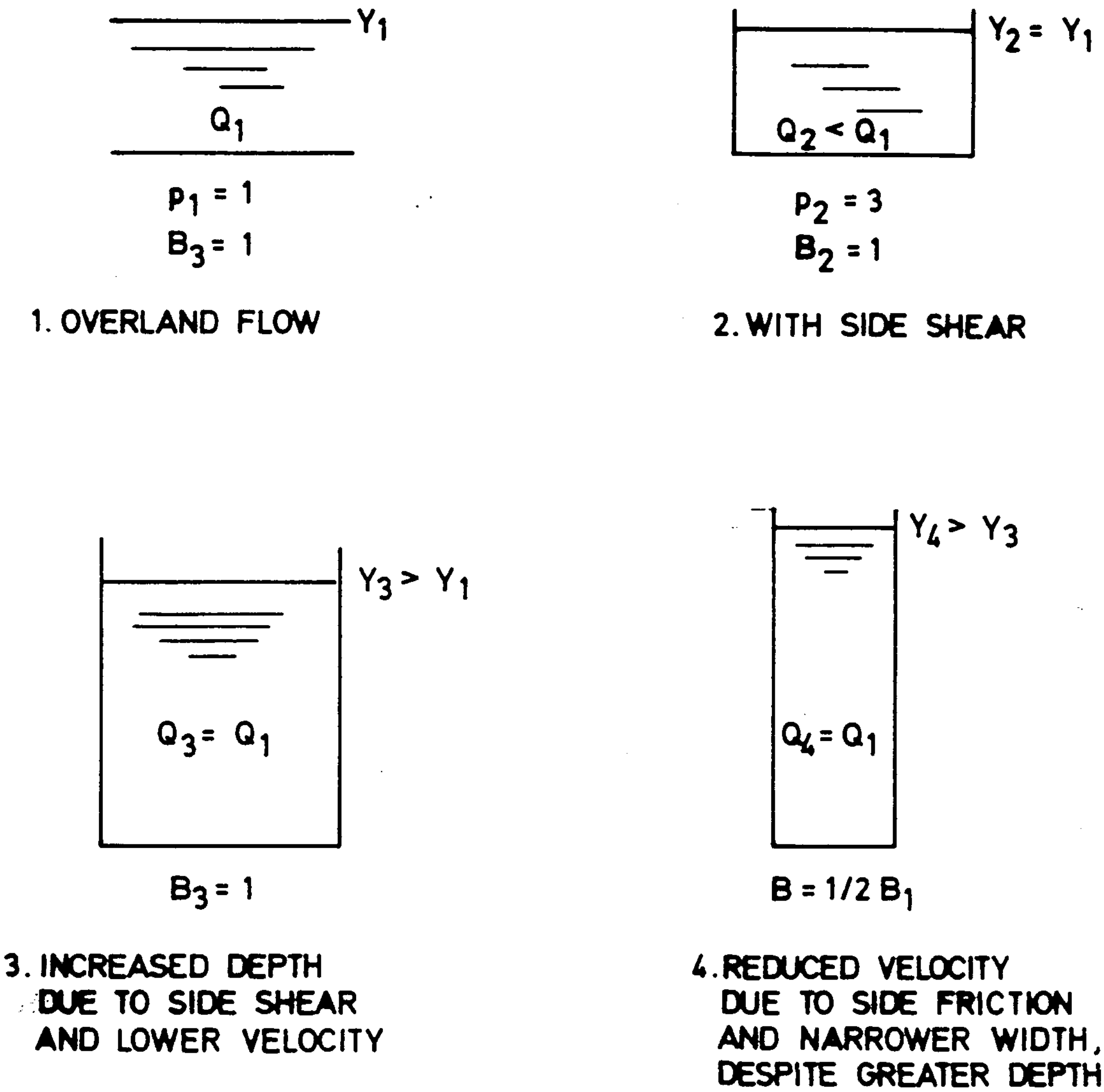


Fig. 6.1 Effect of Channelization and Rills on flow depth and velocity.

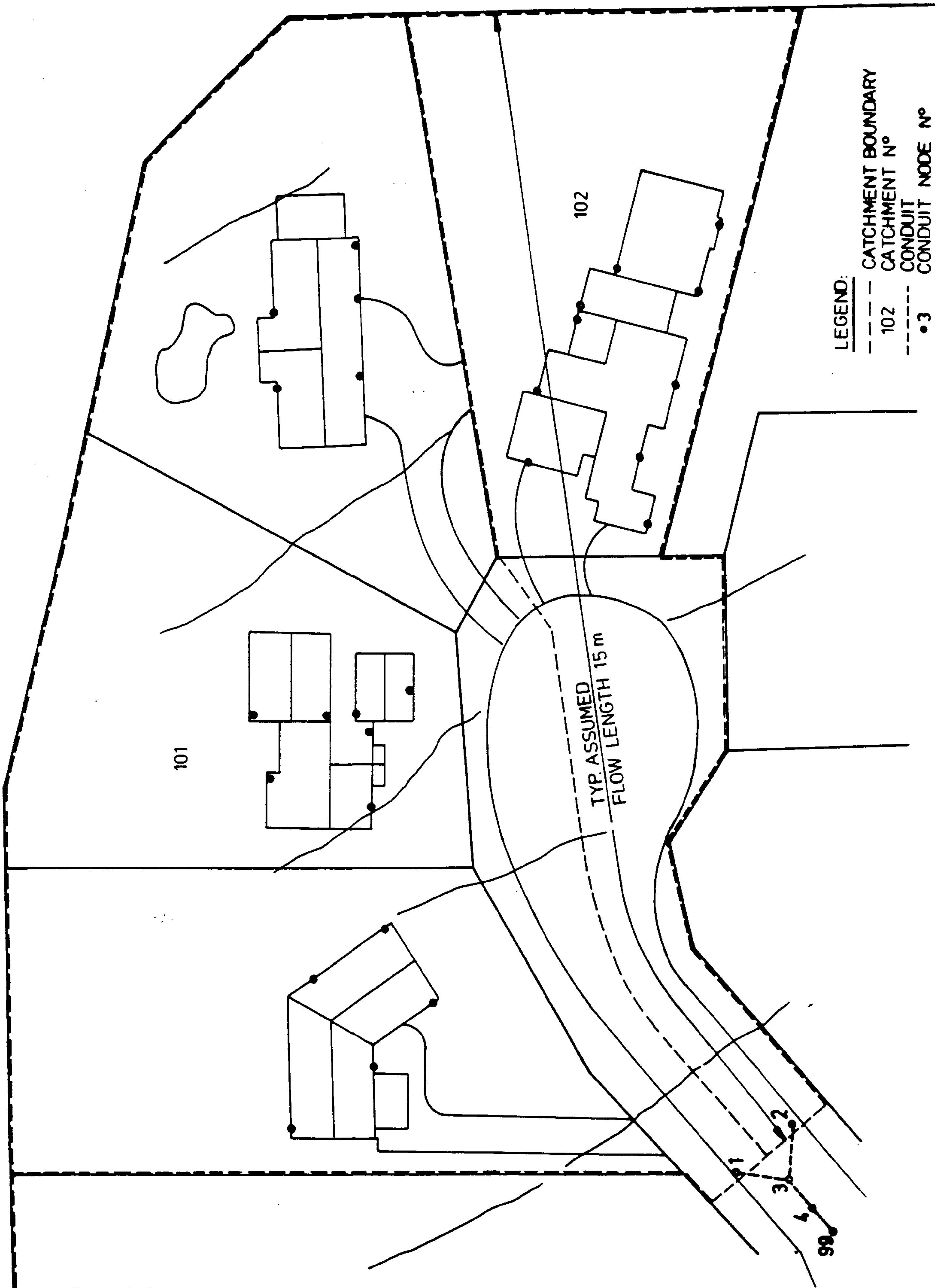


Fig. 6.2 Sunninghill catchment - Coarse Level of Discretization

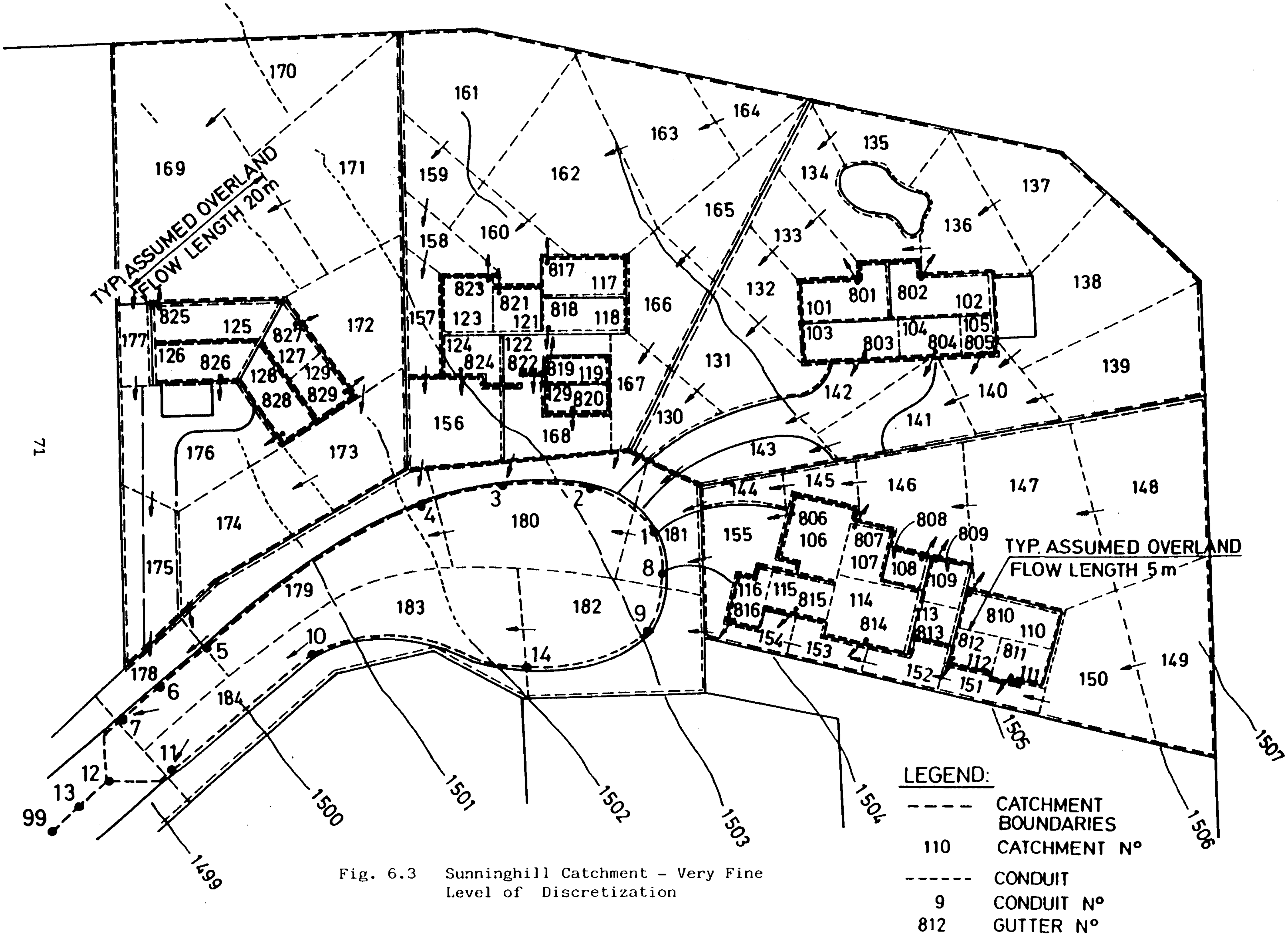


Fig. 6.3 Sunninghill Catchment - Very Fine Level of Discretization

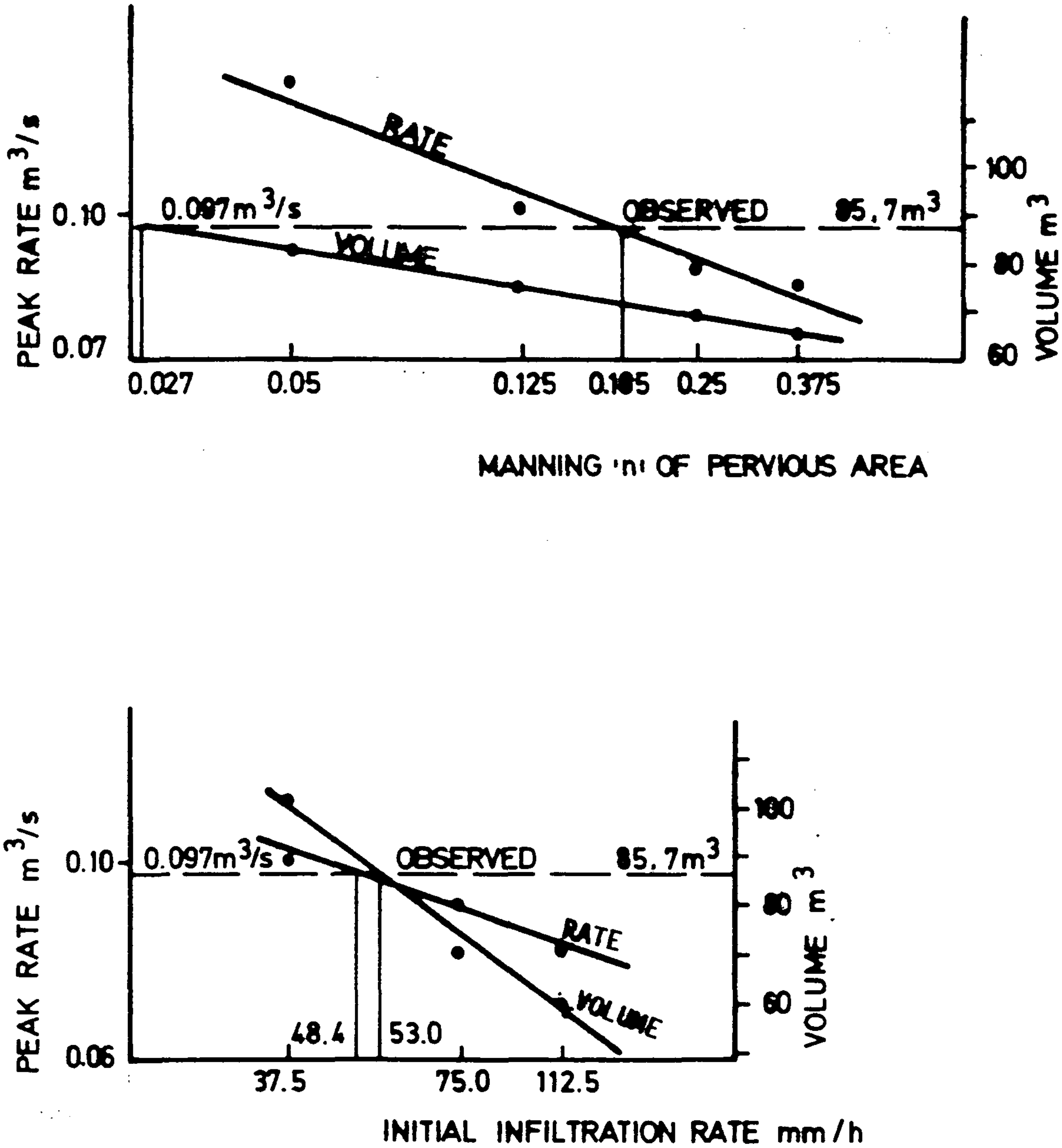


Fig. 6.4 Method of Calibrating Coarse Level Model for Manning Roughness Coefficient and Initial Infiltration rate.

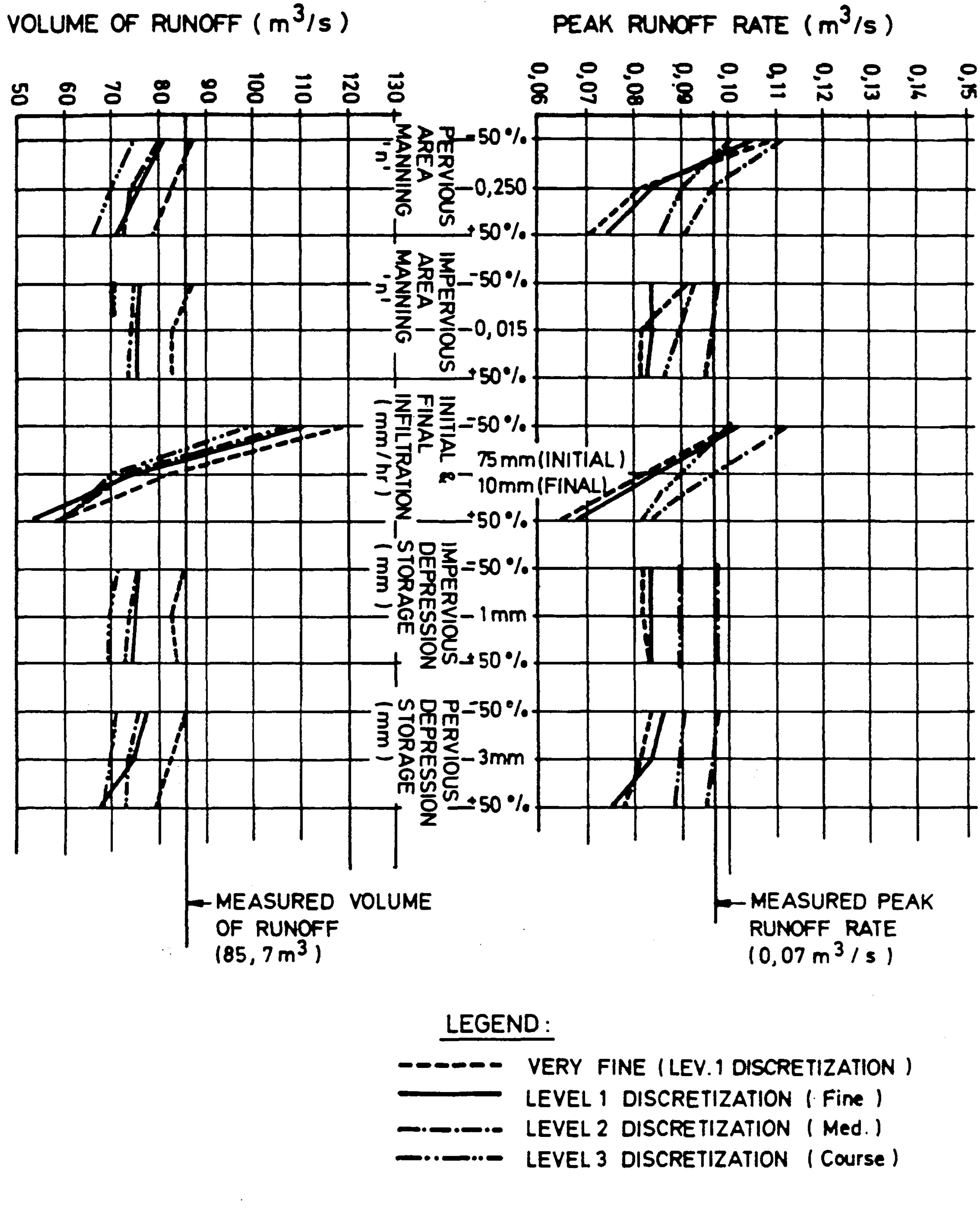


Fig. 6.5 Comparison of Peak Runoffs and Volume Runoff for Various Levels of Discretization and Time Steps

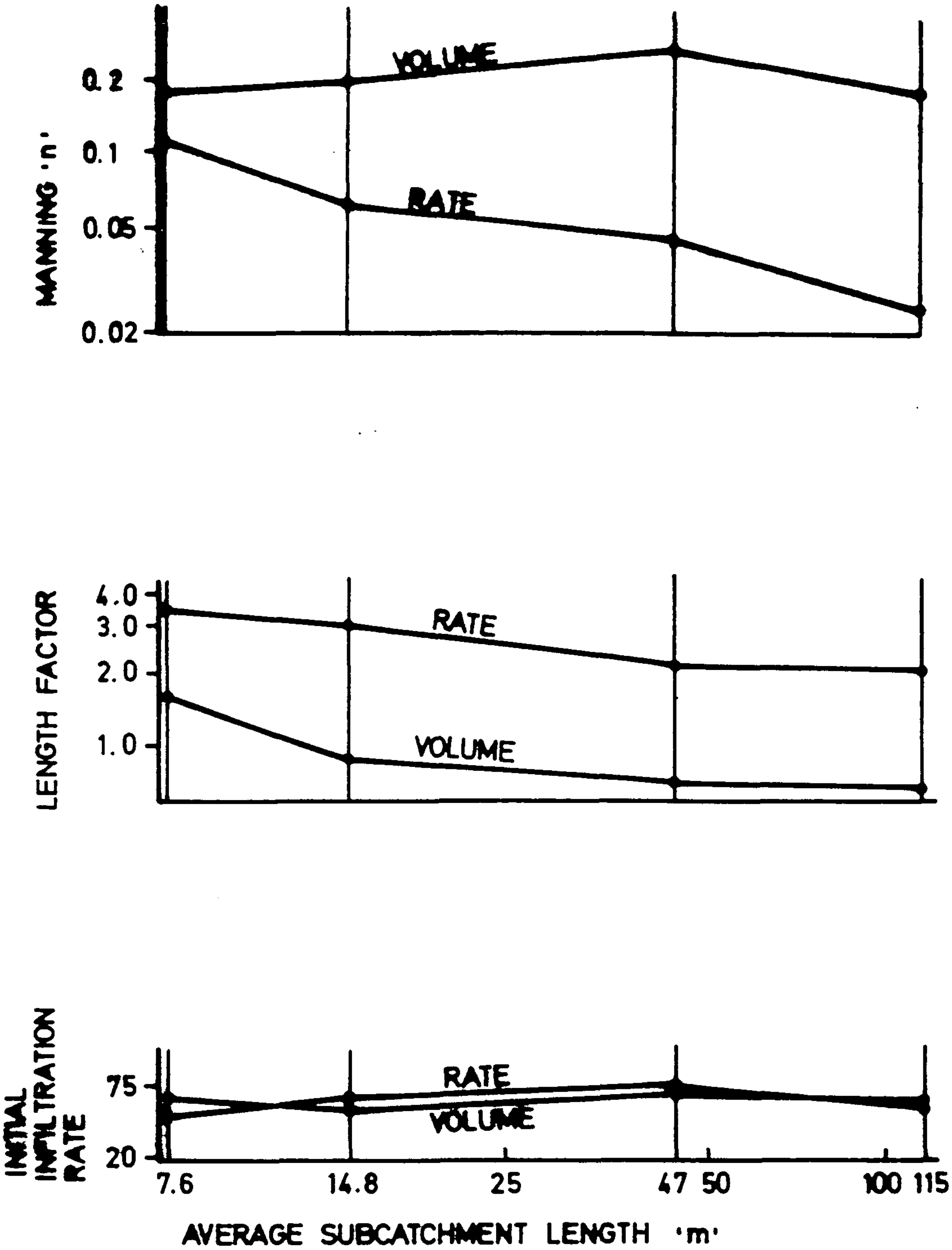


Fig. 6.6 Average Subcatchment Length versus best fit Manning n, or Length Factor, and Initial Infiltration Rate.

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