CHAPTER 3

STANDARD METHODS OF DESIGN

INTRODUCTION

A major step in the design process is the estimation of drain sizes. To do this the engineer needs to estimate discharge rates. The methods described in this chapter are elementary methods of establishing design flows. Using locally applicable rainfall data the engineer is able to select a storm intensity and convert this to a runoff rate for a particular catchment (Jones, 1971).

The methods described here are based on certain restricting assumptions, the main one being that any catchment has a unique time of concentration equal to the travel time down the catchment. The methodology culminates in the so-called rational method. This expression is the modern version of a number of earlier formulae. Despite their limitations the methods are reputed to yield reasonable answers. (Ardis et al, 1969; Schaake, 1967). The rational method in particular is simple to apply and it is easy to visualize the reasoning behind the formula. Although it only yields an initial design, subsequent refinement by more sophisticated methods and computer modelling are always available.

THE RATIONAL METHOD

The rational formula was proposed by an Irish engineer, Mulvaney, in 1851. It was first adopted in the United States of America by Kuichling in 1889, and in England by Lloyd-Davies in 1905. Lloyd-Davies used the equation in conjunction with an empirical equation for excess rain to yield a relationship between catchment area and runoff rate. The rational equation is

$$Q = CiA \tag{3.1}$$

Q is the flow rate, i is the rainfall intensity and A is the surface area of the catchment, all in compatible units. Thus if A is in square metres and i is in metres per second then Q is in cubic metres per second. C is a dimensionless coefficient normally less than unity. Thus C is the proportion of precipitation rate which contributes to peak runoff rate. Values of C for selected catchment characteristics are indicated in Table 3.1.

The equation implies dimensional homogenity, but also yields correct values (to within 1 percent) for Q in cubic feet per second if i is in

TABLE 3.1 Rational Coefficient C

URBAN CATCHMENTS

General Description	C	Surface	
City	0.7-0.9	Asphalt paving	0.7 -0.9
Suburban business	0.5-0.7	Roofs	0.7 -0.9
Industrial	0.5-0.9	Lawn heavy soil,+7° slope	0.25-0.35
Residential Multiunits	0.6-0.7	2-7°	0.18-0.22
Housing estates	0.4-0.6	-2°	0.13-0.17
Bungalows	0.3-0.5	Lawn sandy soil,+7°	0.15-0.2
Parks, cemetries	0.1-0.3	2-7°	0.10-0.15
		-2°	0.05-0.10

Frequency factor:

Recurrence interval	Multiplier
2-10 years	1.0
25	1.1
50	1.2
100	1.25

RURAL CATCHMENTS (less than 10 km²)

Ground cover	Basic factor	Corrections : Add or subtract
Bare surface	0.40	Slope<5% : - 0.05
Grassland	0.35	Slope>10% : + 0.05
Cultivated land	0.30	Recurrence interval<20y : -0.05
Timber	0.18	Recurrence interval>50y : +0.05
		Mean annual precipitation<600mm:-0.03
		Mean annual precipitation>900mm:+0.03

inches per hour and A is in acres. Although it may appear that C is the ratio of volume of runoff to volume of precipitation, the rational equation is not intended as such i.e. the ratio of total loss to total depth of precipitation is not necessarily (1-C). C is strictly only the ratio of peak runoff intensity of a particular frequency to average rainfall rate for the same recurrence interval assuming a hydraulic balance in the catchment. It therefore accounts for a multitude of phenomena.

The hydrograph shape may be compiled from a simple rectangular model of the catchment, (Fig. 3.1). If the rain continues indefinitely the runoff will eventually equal the excess rainfall rate multiplied by the catchment area for flow balance. Initially the runoff will increase as more and more of the catchment contributes. Thus at any time t, the length of catchment contributing is x and the runoff rate is CiAx/L where L is the total catchment length.

If the concentration time is independent of the discharge rate then

$$x/L = t_x/t_L \tag{3.2}$$

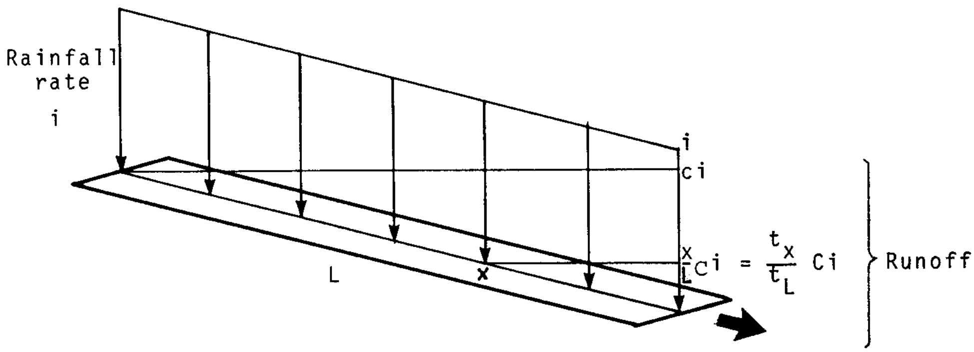

Fig. 3.1 Distribution of rainfall and flow down a simple model catchment.

where t_x is the concentration time over the length x. The hydrograph will therefore increase linearly as depicted in Fig. 3.2 until the entire area is contributing. Runoff will diminish after the rain stops. t_d is the duration of the storm. Then assuming a constant flow velocity the tail of the hydrograph will fall over a time t_c again where $t_c = t_L =$ the concentration time of the catchment.

If the storm stopped at time t_c, then the hydrograph would be triangular with a base equal to $2t_c$ (Fig. 3.3). Thus the area under the triangular hydrograph is $CiAt_c$. This indicates that C represents the ratio of the volume of runoff to volume of precipitation, as well as the ratio of peak runoff rate to precipitation rate.

It is generally accepted, however, that the falling hydrograph tail has a duration exceeding t_c and it may even exceed $2t_c$. Then C does not represent the ratio of volumes. The longer recession limb implies an acceptance that the overland flow velocity reduces as the depth of flow reduces, and casts doubts on the reasoning behind the rational method.

C accounts for initial losses due to depression storage as well as infiltration during the runoff process. It implicitly accounts for the hydrodynamics of the runoff process whereby the runoff from throughout the catchment flows down to the mouth where the discharge Q is to be computed.

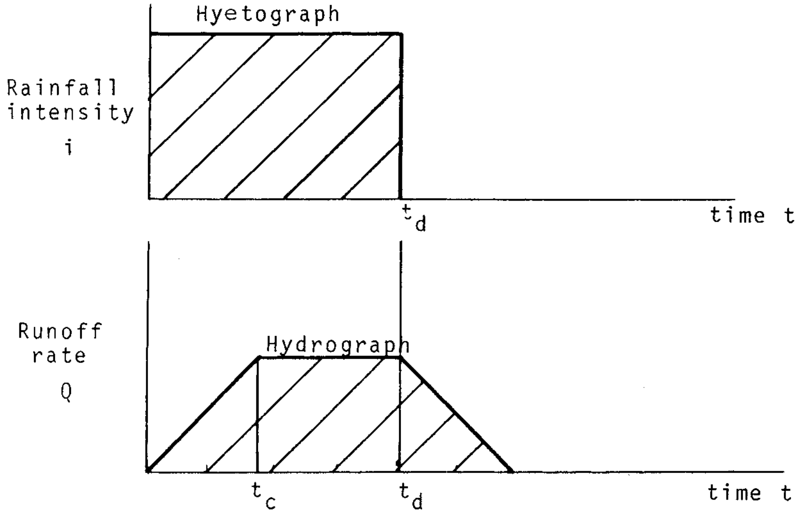

Fig. 3.2 Rainfall and runoff versus time for Fig. 3.1.

It includes the relationship between the recurrence interval of a storm and the recurrence interval of the runoff. For the way to use the formula is to select a storm of known frequency and compute the corresponding runoff assuming the same recurrence interval is applicable. Thus antecedent moisture conditions in the catchment, storm distribution and hydrograph shape are disregarded in deciding C. It is possible that different C values apply to different storms, but the C's listed here are those found to apply to representative design storms, i.e. of the order of 10 year recurrence interval. Thus the 10 year recurrence interval runoff rate is computed from the 10 year recurrence interval storm using the given C.

Variations in storm distribution in time and space are not accounted for. The effective duration of a storm to use in the intensity-duration relationship may not be the total storm duration. Rainfall intensity may vary during a storm and the duration over which the intensity averages the design figure may be only a fraction of the total storm. Whether the design intensity occurs at the beginning or end of the storm will influence the antecedent moisture conditions which should affect C.

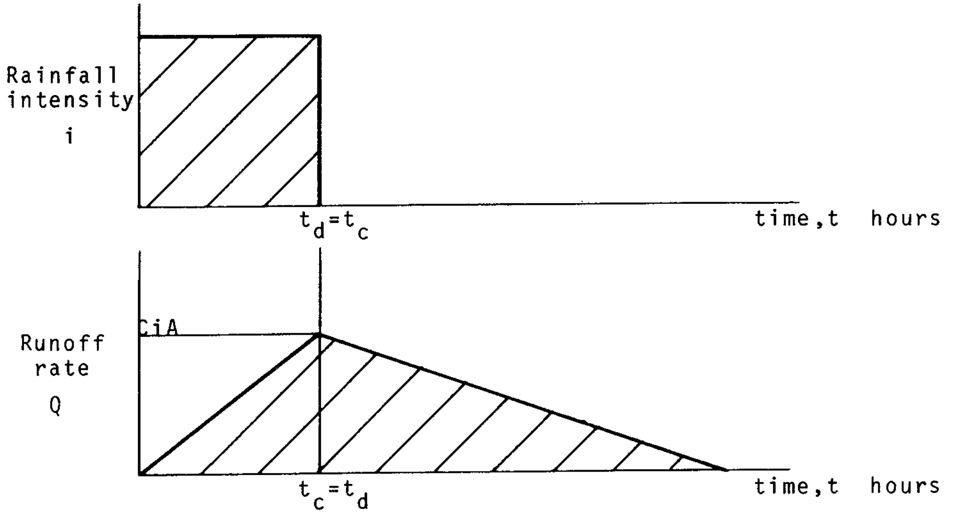

Fig. 3.3 Rainfall and runoff for design storm.

C should theoretically increase with rainfall intensity if losses are independent of intensity. Once the initial fraction is used to replenish initial abstractions, the balance occurs as runoff and the proportion of runoff to total precipitation increases the bigger the storm.

Rossmiller (1980) proposed the following empirical equation for estimating C from a variety of variables:

$$C = 7.7 \times 10^{-7} CN^3 R^{.05} (.01CN)^{-6S^{0.2}} (.001CN)^{1.48(.15-I)} \left(\frac{IM+1}{2}\right)^{.7} \qquad (3.3)$$

where R is the recurrence interval in years, S is the land slope in percent, I is the rainfall intensity in inches per hour, IM is the fraction of watershed which is impervious, and CN is the SCS curve number.

Now for any selected storm recurrence interval, rainfall intensity reduces with storm duration and conversely increases the shorter the storm, in a manner which can be described generally by an equation of the form

$$i = \frac{a}{(b+t_d)^c} \qquad (3.4)$$

where i is the average rainfall intensity, t_d is the storm duration, and a, b and c are constants. It is therefore apparent that the storm which will result in maximum runoff rate should be as short as possible, subject to equilibrium being attained, i.e. for maximum runoff intensity

$$t_c = t_d \qquad (3.5)$$

TABLE 3.2 Formulae for time of concentration for overland flow.

Name	Formula for t_c	Comments
Kerby	$3.03\left[\frac{rL^{1.5}}{H^{0.5}}\right]^{0.467}$	L<0.4km r=0.02 (smooth pavement) 0.1 (bare packed soil) 0.3 (poor grass or rough bare) 0.4 (average grass) 0.8 (dense grass, timber)
SCS	$\{\frac{0.87L^3}{H}\}^{0.385}$	
Bransby-Williams	$\frac{0.96L^{1.2}}{H^{0.2}A^{0.1}}$	
Izzard	$\frac{(.024i^{.33}+878k/i^{.67})L^{.67}}{(CH^{0.5})^{.67}}$	iL<3.8 k=0.007 (smooth asphalt) 0.012 (concrete pavement) 0.046 (close clipped sod) 0.06 (dense bluegrass turf)
Airport	$3.64(1.1-C)L^{.83}/H^{.33}$	C = rational coefficient
Kinematic	$58N^{.6}L^{.9}/i_e^{.4}H^{.3}$	N = Manning roughness

A = area, km²
H = elevation difference, metres
i = rainfall rate, mm/h. Subscript e refers to excess (runoff)
L = length of catchment, km
t_c = concentration time, hours

There are many empirical methods for establishing the time of concentration of a catchment. Various formulae in use are summarized in Table 3.2.

The formulae apply to specific types of surface and use of an inapplicable formula should be avoided. Values of the constants in the equations are also indicated. Where compound areas are involved, the concentration time may be estimated by summating the concentration times over individual areas in series. Where the rational method is applied to compound catchments, the formula may be written as

$$Q_p = i\sum_{j=1}^{m} C_j a_j \qquad (3.6)$$

$$\text{where } A = \sum_{j=1}^{m} a_j \qquad (3.7)$$

LLOYD-DAVIES METHOD

Lloyd-Davies in 1905 published a paper on an approach very similar to that of the more modern rational method. It was in fact Lloyd-Davies who proposed that the storm which produces the greatest runoff of all storms of the same frequency, is the one with a duration equal to the concentration time of the catchment. It was assumed that the concentration time was equal to the travel time from the top end of the catchment to the point at which flow is to be determined. He went further in assuming the concentration time was a function of catchment area.

A relationship between excess rainfall intensity and storm duration was also produced. This could be expressed in terms of the Birmingham formula

$$i = \frac{40}{20+t} \qquad (3.8$$

where i is in inches per hour and t is the concentration time in minutes

The formula was varied slightly by others for storm duration less than 20 minutes. The formula was for rainfall in England specifically, for an acceptable recurrence interval storm, which varies from twice a year for short storms to about once in 15 months for storms over 1 hour duration. It is interesting to note that the early British formulae, and even some modern English approaches, allow for runoff off impermeable surfaces only. 100 percent loss is assumed on permeable surfaces and the formulae are quoted as being applicable to stated percentage impermeable surfaces. In fact the runoff formulae went so far as to give runoff directly in many instances (such as the Birmingham formula above). Thus incorporated in the formula is a percentage imperviousness, a storm intensity-duration relationship and a recurrence interval. The formulae are thus more of historical interest than for application. The rational method allows for many of the earlier shortcomings. Nevertheless we are also indepted to Lloyd-Davies for development of the step method of computation of drain sizes.

STEP METHOD

Although the rational method yields a design flow rate at the mouth of a catchment, it does not provide sufficient data to design the individual drains in a catchment. In fact none of the empirical equations for time of concentration are applicable to built-up areas with impermeable surfaces, artificial channels and circular drains. The concentration time in built-up areas is reduced due to the higher volume of runoff, the smoother surfaces and canalization.

In an effort to account for the flow time through each drain in the accumulation of flow, a step-by-step method was evolved in England by Lloyd-Davies (1905). The runoff from impermeable areas is accounted for, although earlier applications ignored runoff from pervious areas. The runoff from any catchment could be accounted for by applying individual runoff coefficients to each sub-area.

The method in common with the rational method uses a basic assumption contrary to hydrodynamic principles. This is that the concentration time can be estimated from the travel time for full flow down the drains. In fact during flow concentration, flow rates will be less than the maximum, and flow velocities will be correspondingly lower than for the full pipe case. Also at design flow for any pipe, pipes upstream will not be at full flow, as they are designed for a storm of shorter duration and consequently greater intensity. This is offset by another misunderstanding. Water does not need to travel the full drainage system length before an effective equilibrium is attained. In fact reaction time of the system can be faster than flow time, as it is more a function of wave speed than water speed.

Nevertheless, the Lloyd-Davies step method yields results of satisfactory engineering accuracy. It has been found to overestimate peak flow rates for pipes over 600 mm diameter, but in order to improve on the method, more sophisticated mathematics is required, (for example the kinematic method). The Lloyd-Davies method is relatively simple to apply and drains may be sized in a systematic manner. The procedure is set out in tabular form, and calculations proceed from the top drain to successively lower drains (e.g. ASCE, 1969). Although it was common to use a rainfall intensity-duration relationship of the form

$$i = \frac{a}{b+t_c} \qquad (3.9)$$

this is not a necessity for application of the method. The computations are set out with the aid of an example (Fig. 3.4) in Table 3.3.

The steps in the computations are as follows (the numbers refer to the columns in Table 3.3):

1. Mark pipe numbers on a plan, proceeding from the top pipe of each leg. In this catchment there are three levels of subdivision of the drains.

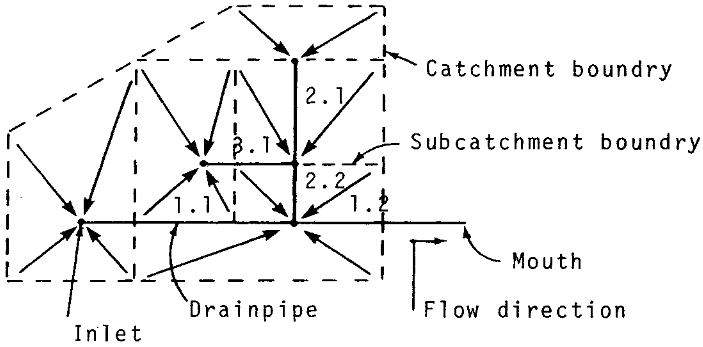

Fig. 3.4 Catchment example for Lloyd-Davies step computations

2. From the contour plan and demarcated subcatchments planimeter the area contributing to each pipe. If the inflow is along the length of the pipe it may be taken as into the head for simplicity. Alternatively (see White, 1978) it may be fed in along the pipe in which case the design storm duration depends on that pipe diameter. The pipe diameter must therefore be determined by trial in such case.
3. The proportion of runoff for each subcatchment must be estimated. The C is similar to the C in the rational formula.
4. The effective contributing area is CA.
5. Add the effective areas down to the pipe in question.
6. Measure pipe lengths from the layout plan.
7. Establish the gradient from contours. In the case of adverse ground slopes or minor drains, the minimum gradient may be dictated by minimum flushing velocity. In this case a trial and error method may be required.
8. The concentration time for upper pipes is based on the time of entry. This varies from 2 to 4 minutes for urban catchments, but may be larger for overland flow. In that case it must be established as in the rational method or the kinematic method. For lower pipes, it is necessary to compare alternative feeders and select the feeder resulting in maximum concentration time. Thus for pipe 2.2 the concentration time down route 2.1 is 153 seconds (120 + 33) whereas for route 3.1 it is 150 seconds.

TABLE 3.3 Step Computations

1. Pipe No.	2. Contrib. area A(m²)	3. Coeff. C	4. Effec. area CA	5. Total ΣCA	6. Length (m)	7. Slope S	8. Conc. time t_c(s)	9. $i = \frac{.052}{630+t_c}$ (m/s) x10^{-6}	10. Q= iΣCA m³/s	11. Dia. mm	12. Vel. V m/s	13. Incr. Δt_c= L/V
1.1	15000	0.5	7500	7500	180	0.01	180	64	.480	480	2.7	67
2.1	5000	0.5	2500	2500	90	0.02	120	69	.173	250	2.7	33
3.1	8000	0.7	5600	5600	80	0.01	120	69	.389	440	2.5	30
2.2	7000	0.6	4200	12300	50	0.008	153	66	.817	620	2.8	18
1.2	14000	0.3	4200	24000	100	0.0025	247	59	1.417	950	1.9	53

TABLE 3.4 Data for tangent method example

1. Pipe No.	2. Contr. area (m²)	3. Runoff coeff. C	4. Effect. area (m²)	5. Total ΣCA	6. Length (m)	7. Slope (m/m)	8. Time of entry (s)	9. Conc. time (s)	10. $i = \frac{.05}{90+t}$ 10^{-6}m/s	11. Q = iΣCA m³/s	12. Dia. mm	13. Vel. m/s	14. Incr. Δt_c = L/v s
1.1	4000	0.3	1200	1200	200	0.005	140	140	217	0.261	430	1.7	117
2.1	12000	0.4	4800	4800	50	0.01	120	120	238	1.140	670	3.2	16
1.2	3000	0.5	1500	7500	50	0.008	120	257	144	1.081	690	2.9	17

9. The rainfall intensity is assumed to obey the relationship $i = a/(b+t_d)$ where a and b are constants, i is in m/s and t_d is the storm duration, assumed equal to travel time for maximum runoff rate.
10. Establish the peak discharge rate by multiplying i by the total effective contributing area.
11. Pipe diameter may be selected from a flow versus head loss chart, which will also yield flow velocity for (12). It is unlikely that the full-flow pipe diameter so yielded will be a standard commercially available pipe size. In such cases the nearest larger standard pipe size is selected, and the pipe may run part-full. Theoretically the flow velocity will then be different, but is is often conservative to utilize the full-pipe velocity.
12. The flow velocity at design discharge can be read from pipe charts or calculated.
13. The increment in travel time is now calculated by dividing the length of the pipe just determined by flow velocity. This increment is added to the travel time down preceeding pipes to obtain travel time to the next lower pipe.

TIME-AREA DIAGRAM AND ISOCHRONAL METHODS

If travel time is assumed to be independent of storm intensity then every point in the catchment will have a unique travel time to the mouth or point of discharge. In fact one could plot isochrones on a catchment map as illustrated in Fig. 3.5. An isochrone in this context is a line of constant travel time. On a simple plane isochrones may be equidistant. Where conduits and overland flow are involved, water velocity down conduits is generally faster than over land, so the isochrones may exhibit anomalies at conduits.

For design purposes it is sufficient to mark isochrones along the conduits proceeding from the outer extremities and marking down each drain. After reaching an intersection one can correlate isochrones on each leg meeting at the intersection. By joining points of equal travel time one establishes isochrones. Ultimately the entire catchment is thus demarcated into time zones.

One could then plot a graph of area contributing to the flow at the mouth of a catchment against time. As time passed, so more and more runoff from further up-basin would reach the mouth until the entire catchment area was contributing. The time-area diagram shows the rate of build-up in contributing area (which is assumed proportional to flow) during the entire storm. It is a massed area curve. If area is

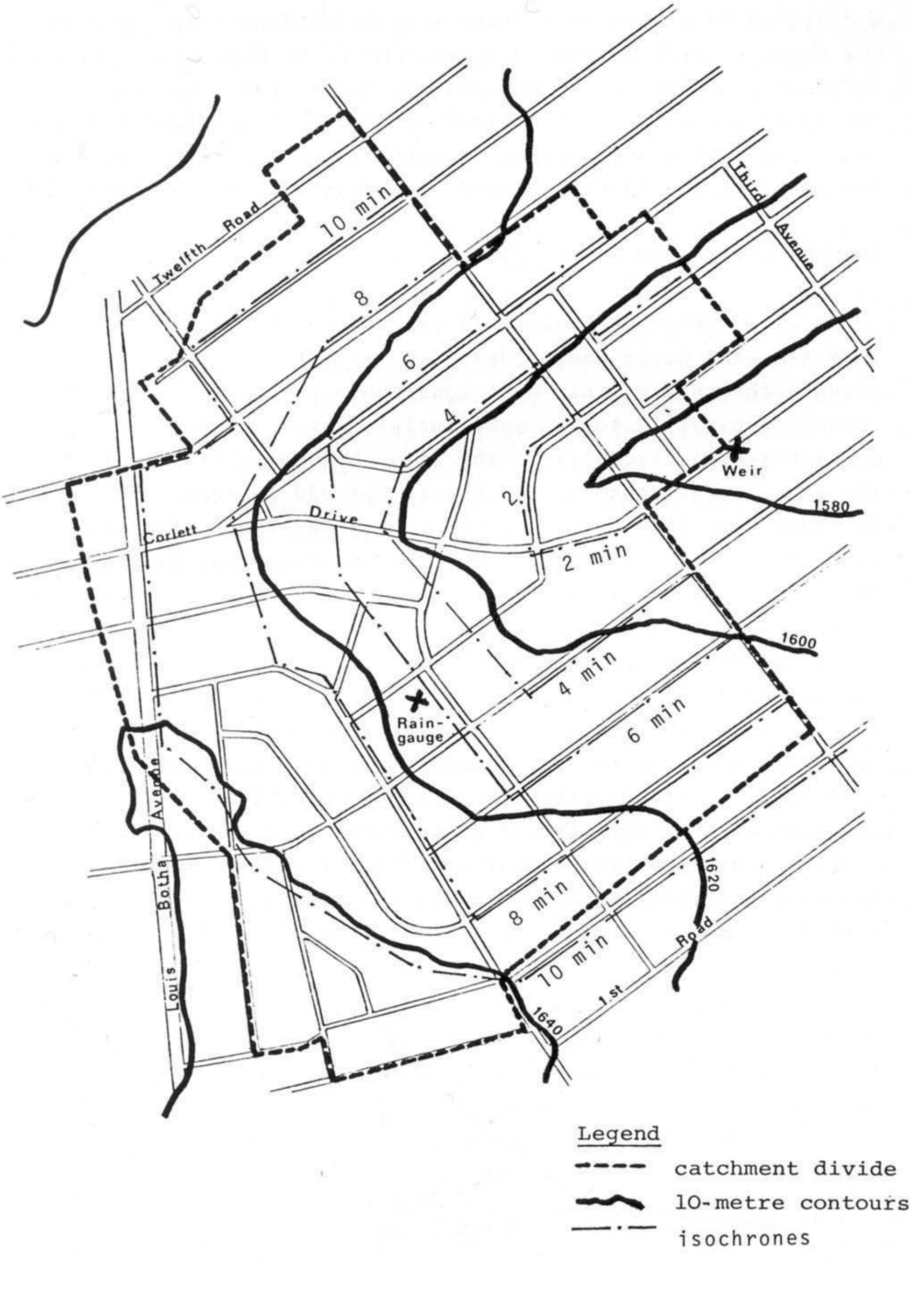

Fig. 3.5 Contour plan with isochrones

multiplied by excess rain intensity one obtains a massed flow. The slope of such a curve is proportional to flow rate. The storm intensity, however, also affects the runoff rate, and this is a function of duration. It is therefore necessary to consider the storm intensity-duration relationship together with the time-area graph and this is what the tangent method does (Watkins, 1962; White, 1978).

TANGENT METHOD AND MODIFICATIONS

The step method outlined will yield a successively larger concentration time and hence longer design storm, for successive pipes, i.e. it assumes the entire basin must contribute for the maximum runoff. This is not necessarily so, as some outlying areas of the catchment may not contribute significantly to the area, but could nevertheless add to the travel time down the catchment. For odd catchment shapes, it will be shown that the rate of runoff can exceed that calculated using the Llody-Davies method. This stems from the time-area diagram which uses the steepest segment to define the effective contributing area for maximum runoff intensity.

The application of the tangent method to a time-area diagram will be demonstrated with the example in Table 3.4. The example is illustrated in Fig. 3.6. Before constructing the time-area graph it is necessary to do the steps in the Lloyd-Davies calculations (Table 3.4).

In Fig. 3.7 are plotted the various contributing areas, starting on the time axis at the time at which they start to contribute and building up to the full value over the time of entry for that area. Thus the area contributing to drain 1.2 will reach the mouth first. The flow from this area will start at t = 17s, the travel time down the drain.

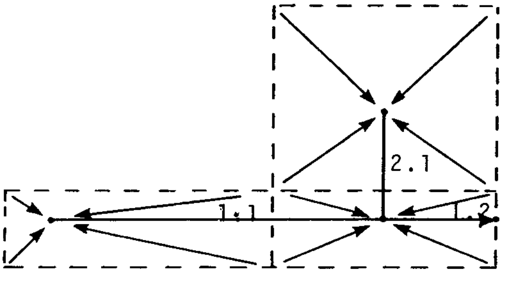

Fig. 3.6 Catchment example for tangent method.

The buildup time for each drain is its time of entry (or overland flow concentration time). If inflow were over the entire length of drain, outflow would begin immediately and built up over $t_e + t_L$ where t_e is the time of entry and t_L is the travel time for that drain. Tracing back through the drainage system, each contributing area is plotted, commencing at its time of arrival at the mouth. Each contributing hydrograph (or in this case area-graph) will rise over its time of entry and then become horizontal, implying equilibrium is reached for that subcatchment. Each line is lagged by its travel time to the mouth. The total area graph is the sum of the individual area graphs, in the correct time positions (line OMNPQRST). It will be noted that portions of the area graph are steeper than others, in particular over the time during which the lateral contributions arrive. This fact points to the possibility that the peak discharge may result from a storm over the duration of the steeply rising portion of the area graph and not for the full concentration time of the basin. The shorter storm will be more intense and this may more than offset the reduction in area by omitting the flatter portions of the curve. If one assumes a storm intnesity-duration relationship of the form

$$i = a/(b+t_d) \tag{3.9}$$

then $$Q = iA = aA/(b+t_d) \tag{3.10}$$

Hence $$Q/a = A/(b+t_d) \tag{3.11}$$

Thus discharge Q is proportional to the slope of a line with a vertical to horizontal slope of A to $(b+t_d)$. A line drawn from -b on the A=0 axis in Fig. 3.7 and tangential to the outside of the area graph will have a slope proportional to the runoff from the entire catchment. Runoff from a portion of the catchment may, however, produce the peak flow. This will be represented by a line not originating at the base of the area diagram.

Now to find the worst (maximum) rate of runoff it is necessary to find the steepest possible tangent. This is done most easily by drawing another time-area line, a distance b in the time axis direction before the original time-area line. (Line O' N' P' Q' R' S' T' in Fig. 3.7) Now draw in a straight line tangential to the convex down part of this curve and also tangential to the convex upward part of the original curve (line N' R). This indicates that the maximum runoff will be associated with a storm of duration $t_d = t_R - t_{N'} - b = 154-(-54)-90 = 118$s. The corresponding contributing area is $\Delta A = A_R - A_N = 6450-200 = 6250\text{m}^2$ and the runoff is $Q = \Delta A a/(b+t_d) = 6250 \times 0.05/(90+118) = 1.50\text{m}^3/\text{s}$.

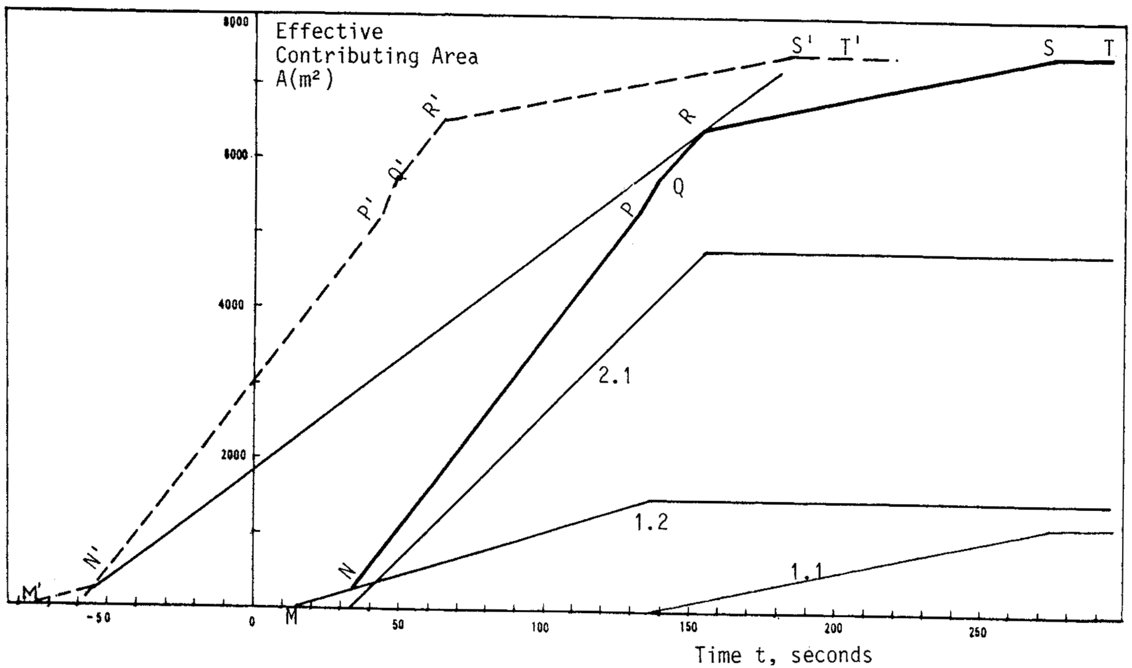

Fig. 3.7 Time-area diagram and tangent solution

An alternative method is as follows: Instead of drawing one tangent line, two parallel lines spaced b apart on the time scale are required. For maximum discharge, the left hand line should be tangential to the convex upward part of the curve as before. The right hand lower line should be tangential to the convex downward part of the area-time curve To find the points of tangency resulting in maximum slope it is necessary to keep the right hand line tangential to the convex downward part of the curve and gradually increase the slope from the horizontal until the parallel line spaced b to the left is also tangential to the convex upward part of the curve. At this stage the maximum slope is achieved. The corresponding discharge Q is equal to the line slope multiplied by $a\Delta A/(b+t_d)$ where ΔA is the contributing area between the two points of tangency i.e. the vertical distance, and the design storm duration t_d is the horizontal distance between the same points.

The tangent method has been modified to allow for storm intensity-duration relationships not of the form suggested previously. Formulae of the following more general form have received recent recognition:

$$i = \frac{a}{(b+t_d)^c} \tag{3.12}$$

This type of formula produces curved tangent curves. Escritt (1972) proposed that transparent overlays be prepared on which a number of tangent curves of equal runoff be drawn. The method is time-consuming and not recommended. The extra effort is seldom worthwhile. If necessary the method could be replaced by numerical methods.

The tangent methods are generally time consuming and not recommended for all catchments. An inspection of the plan should reveal whether there are outlying or sparse areas not likely to contribute to the runoff but which would influence the concentration time of the basin.

Escritt (1972) suggests that a modified rational method yields reasonable results for small catchments. He recommends taking a rainfall intensity independent of storm duration for durations less than 15 minutes in England. Thus for a three-year recurrence interval storm in England he suggests a storm intensity of 1 inch (25 millimetres) per hour. This procedure does away with the need for time of concentration calculations. It is also stated that it eliminates the necessity of doing a time-area graph.

In general in order to do numerically what the tangent method does in locating the critical contributing area, it is necessary to resort to a trial and error method. Once travel times are established for all points in the drainage system, a storm of a selected duration, equal to or less than the longest travel time is selected. By multiplying Ci by the area between any two isochrones spaced t_d apart, a runoff rate is yielded. This is repeated for different isochrones and for different storm durations. The worst storm is then selected from the results.

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