HYDRAULICS OF RUNOFF FROM DEVELOPED SURFACES

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SYNOPSIS

Runoff produced by rainfall on a developed surface, such as a highway pavement or airfield runway, takes place as "overland" flow in a thin sheet across the surface. Usually this overland flow is collected in a longitudinal gutter. The first part of this paper deals with the hydraulics of overland flow as developed from a cooperative experimental project. A method is given for computing the hydrograph of runoff resulting from given rates of rainfall taking into consideration the roughness, slope, and length of the surface. The second part of the paper is based on an analytical study of the hydraulics of flow in a gutter collecting runoff along the edge of a pavement. The study shows how storage in the gutter causes the outflow hydrograph to lag behind the inflow hydrograph. An approximate method is given for computing the hydrograph of outflow taking into consideration the roughness, grade and length of the gutter as well as the characteristics of overland flow. As a practical application curves are developed giving the maximum rate of runoff on a given roadway cross-section for various lengths of roadway as limited by the intensity-duration rainfall frequency curves for a given locality.

This paper deals with results of experimental work on overland flow from paved and turf surfaces and with analytical studies of flow in gutters. The experimental work on overland flow was done in 1942–43 as a cooperative project of the Public Roads Administration and the Soil Conservation Service, the latter having furnished and operated the equipment. A preliminary report describing the equipment was given at the St. Louis meeting of the Highway Research Board in 1942 (*1*)[1]. Briefly, the equipment consisted of nozzles forming drops of water which were projected upward from outside the edges of a plot 6 ft wide so that the drops fell uniformly over the entire area of the plot, simulating rainfall. Flow of water through the nozzles was steady, rainfall being started (or stopped) by simultaneously removing (or replacing) the hoods deflecting the flow of water away from the plot (Fig. 1). The rate of rainfall, normally about 3.8 in. per hr, was cut in half by replacing half the hoods on each side. Runoff from the plot was passed through a measuring flume where readings were taken at 10-sec intervals.

The longitudinal gradient of the plot was changed by placing blocks under the supports. Grades tested ranged from 0.1 to 4.0 percent on the paved surfaces and from 1.0 to 4.0 percent on the turf surface. The length of overland flow was varied from 12 to 72 ft by means of a bulkhead between the borders. The first surface tested was a smooth asphalt-mastic surface resembling a sheet asphalt in texture. The next tests (Series II) were made on crushed slate roofing paper laid on top of the first surface. The Series III tests were made on a dense bluegrass turf sod placed directly upon the roofing paper with transverse slats to restrict flow on the contact plane. All tests were conducted in a sheltered area under laboratory conditions. Typical hydrographs are shown in Figure 2. The analysis of the data will be discussed subsequently.

As a result of the tests on overland flow it is possible to compute the hydrograph of runoff from a plane surface, such as a paved roadway or airfield runway, resulting from rainfall at one or several intensities. The test plot corresponds to a transverse strip of the roadway or runway. Usually the runoff is collected in a gutter at the edge of the pavement or shoulder. Part of the flow will go into temporary storage in the gutter thereby causing the rate of outflow to differ from the rate of inflow to the gutter. The second part of this paper indicates how the rate of flow at

[1] Italicized figures in parentheses refer to list of references at the end of the paper.

any point in the gutter may be computed. The peak rate of flow in a gutter of given length determines what the capacity of an inlet should be if all the flow is to be intercepted.

The broad principles of analysis as outlined are not new having been set forth by Horner and Jens (*2*) and by Hicks (*3*). These investigators, however, based their overland flow hydrographs on relatively limited data which have been greatly augmented by the experiments herein described and reported elsewhere (*4*). In addition a new technique of analysis has been developed both for the overland flow and for gutter flow. It is not intended to imply that this new technique can

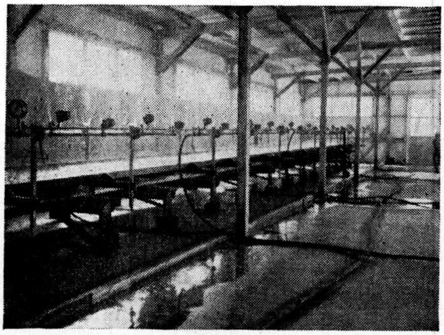


Figure 1. Rainfall Simulator Apparatus in Operation with Rainfall at 3.8 in. per hr

be applied directly as a design method except in cases where a repeating geometric design justifies the effort involved. The new technique, however, does have a large field of usefulness in analyzing the effect upon the runoff of changes in crown width, crown slope, shape and slope of gutter, and spacing of inlets. The question of coefficients to be used has been almost entirely eliminated, since the relation of runoff to rainfall is computed mathematically on the basis of the storage requirements of the system, the only assumptions being with respect to the roughness of the pavement and gutter surfaces and infiltration on pervious surfaces. The analytically derived conclusions with respect to routing of flow through gutter storage should be verified experimentally.

OVERLAND FLOW

The Dimensionless Hydrograph—Analyses of the hydrographs resulting from simulated rainfall at a constant rate indicated that the form of the rising hydrograph could be represented by a single dimensionless hydrograph as shown in Figure 3. The height of the square represents the rate of rainfall and the width of the square the time necessary substantially to reach equilibrium. The equilibrium time t_e is

$$t_e = \frac{2D_e}{60q_e} \qquad (1)$$

in which D_e is the detention in cu ft at equilibrium (vol of water in overland flow on a unit strip) and q_e is the discharge in cu ft per sec at equilibrium[2]. The geometric significance of this equation is simply that the volume of water in detention at equilibrium (the area above the curve) is substantially equal to the volume of water which has been discharged in the time required to reach equilibrium (the area below the curve).

If i is the rainfall intensity in in. per hr, and L the length of overland flow in feet, then, according to a previous paper (*4*),

$$q_e = \frac{iL}{43200} \qquad (2)$$

and

$$D_e = KL(q_e)^{\frac{1}{3}} \qquad (3a)$$

or

$$D_e = \frac{KL^{\frac{4}{3}} i^{\frac{1}{3}}}{35.1} \qquad (3b)$$

in which

$$K = \frac{(0.0007\, i + c)}{s^{\frac{1}{3}}} \qquad (4)$$

and s is slope of surface in ft per ft.

A nomograph, Figure 4, has been developed for convenient solution of Equations (3b) and (4). The term c in the latter equation involves roughness of the surface and was experimentally evaluated as follows:

Series I very smooth pavement...	0.007
Series II crushed slate roofing paper........................	0.0082
Series III dense bluegrass turf....	0.06

In addition analysis of runoff from a concrete apron on an army airfield indicated a

[2] Symbols used are tabulated at end of paper.

tentative value of $c = 0.012$ for concrete pavement with normal construction irregularities on a 0.5 per cent grade.

Strictly speaking, use of Equations (3) and (4) should be limited to cases where the product of rainfall intensity in in. per hr times length of overland flow in ft is less than 500. The reason for this limitation is that both theoretical and experimental data indicate that when the discharge exceeds the rate which would obtain at this limit of iL the flow is turbulent rather than viscous. This limitation would have no effect with highway cross-sections where the length of overland flow will rarely be more than 100 ft. If the equations are used beyond this limit, the results will be on the safe side, since the computed detention will be less than that which would actually exist with turbulent flow.

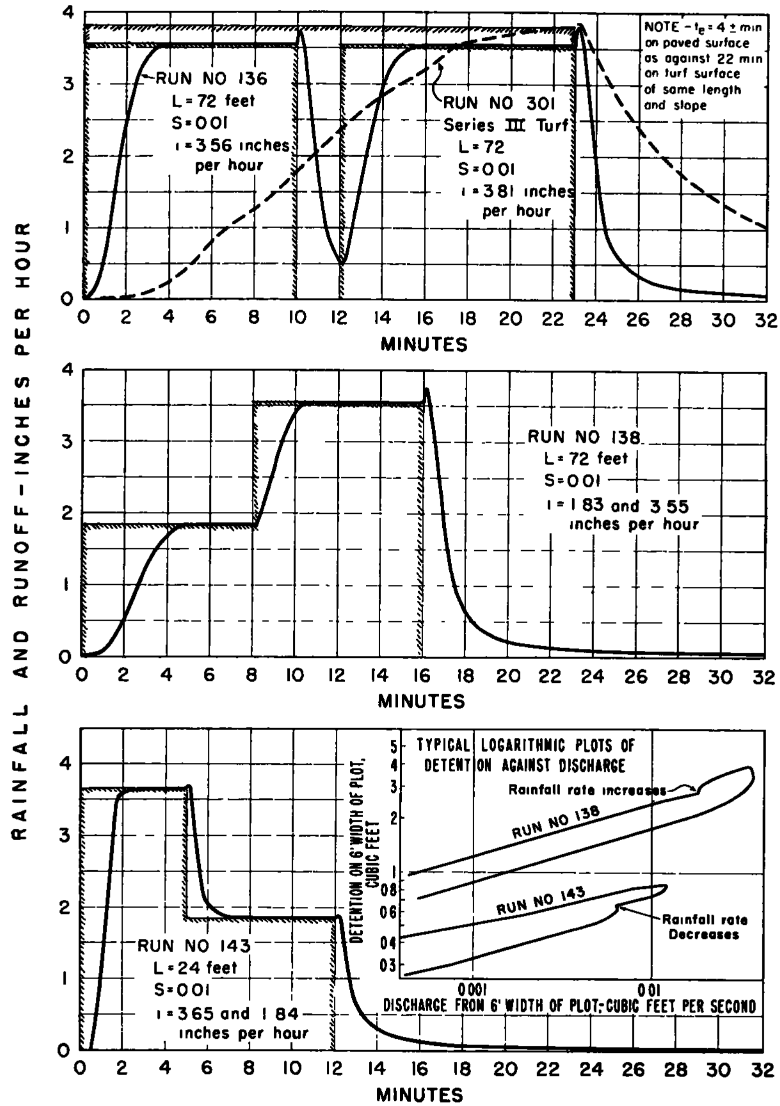


Figure 2. Typical Hydrographs of Runs on Crushed Slate Surface—Series II, with One Run on Turf—Series III, for Comparison

Equation (4) should be used cautiously for slopes steeper than 0.04 which was the maximum slope upon which data were obtained. It is possible that on substantially steeper slopes, the flow will be pulsating and there is no evidence as to whether the detention coefficient K would then be greater or less than indicated by the equation.

Infiltration—The experiments on turf were designed to eliminate infiltration as much as possible by using a 1.5-in. thickness of sod on an impervious subsurface. If infiltration is enough to be considered then the value of i used in Equations 2, 3 and 4 should be rainfall

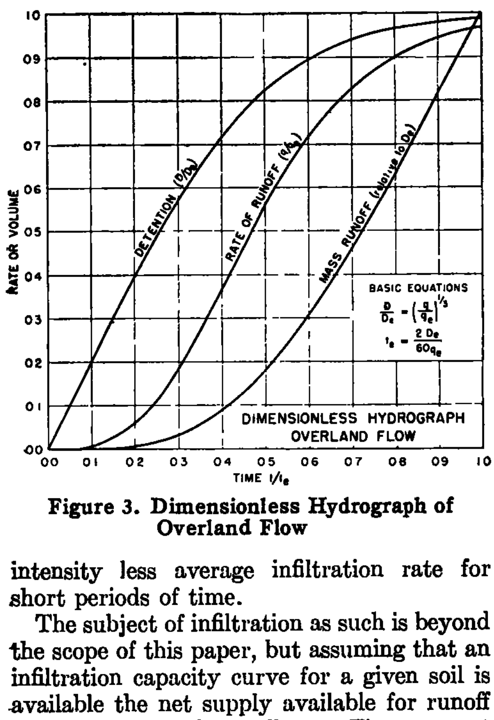


Figure 3. Dimensionless Hydrograph of Overland Flow

intensity less average infiltration rate for short periods of time.

The subject of infiltration as such is beyond the scope of this paper, but assuming that an infiltration capacity curve for a given soil is available the net supply available for runoff may be estimated as follows. First convert the infiltration rate curve into an infiltration mass curve. Plot this with the mass rainfall curve computed in the same units. With both curves starting from the origin, the infiltration curve may lie above the rainfall mass curve for a period of time, which is inconsistent since it is not possible for more water to soak in than is supplied by rainfall (assuming that to be the only source of water). The next step, then, is to shift the infiltration mass curve to the right until it is tangent to the rainfall mass curve at one point. Then, up until that time, all the rainfall is infiltrated and beyond that time, the ordinates between the mass rainfall curve and the shifted mass infiltration curve represent the mass supply curve. The latter may be broken into short periods of nearly uniform supply rate to plot the supply pattern.

Detention and Mass Runoff—The detention on the rising hydrograph, as represented by the area above the dimensionless hydrograph in Figure 3, increases in proportion to the $\frac{1}{3}$ power of the rate of discharge. Actually the power varied from about $\frac{2}{10}$ for the very smooth pavement to roughly $\frac{4}{10}$ for turf but this variation did not affect the validity of Equation (1) in the least. The effect is illustrated by Figure 5 which shows that the midpoint was substantially the same over the range from $\frac{2}{10}$ to $\frac{5}{10}$, the main portion of the curve being steeper for the smaller power.

In order to determine the detention at any time on the rising side of the hydrograph, the ordinate for the detention curve at the time $\frac{t}{t_e}$ is multiplied by D_e.

Similarly, the volume of runoff can be determined by multiplying the ordinate for the mass runoff curve at time $\frac{t}{t_e}$ by the value of D_e (since runoff at time t_e is equal to D_e).

Increase in Rainfall Intensity—When the rate of rainfall increases the corresponding hydrograph can be computed by the method illustrated in the following example which is actually a synthesis of one of the experimental runs. The first step is to compute values of q_e, K, D_e and t_e from the given values of L, s, c and i. This can be done quickly by means of the nomograph in Figure 4. Then the ordinates for the hydrograph are computed as shown in the table on the right of Figure 6.

For the first 2 min, $\frac{t}{t_e}$ is obtained by dividing each assumed value of t by 5.30 min which is the computed value of t_e at i = 1.89 in. per hr. From the dimensionless hydrograph, Figure 3, the corresponding values of $\frac{q}{q_e}$ are read. These values are then multiplied by 1.89 to get the ordinates of the hydrograph in in. per hr. Ordinates in cu ft per sec could be obtained by multiplying by q_e in cu ft per

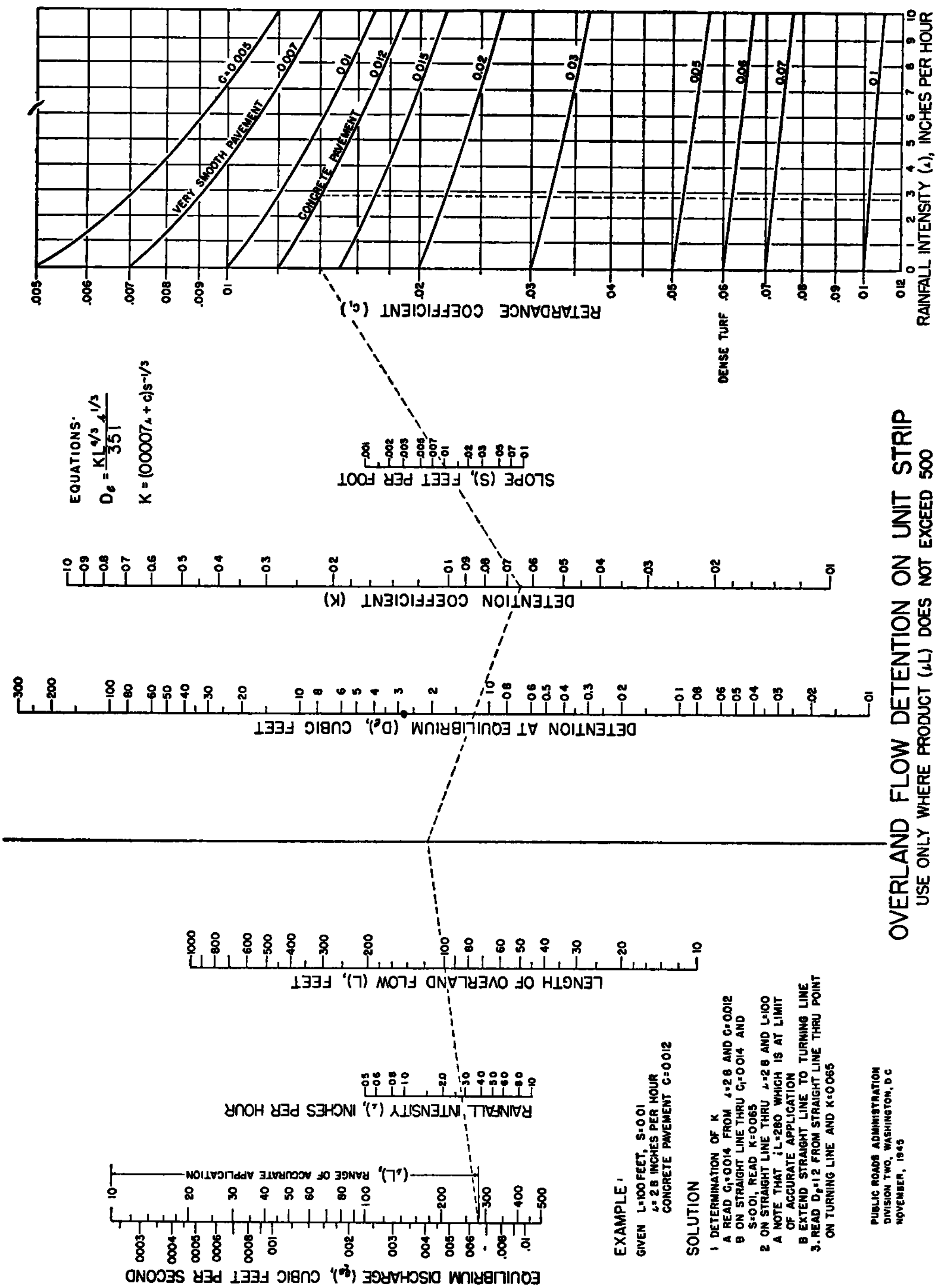


Figure 4. Nomograph for Overland Flow Detention on Unit Strip

sec. The detention at the time the rainfall rate changes is also computed, using the detention curve at $\frac{t}{t_e} = 0.377$.

When the rainfall rate changes to 3.78 in. per hr D_e becomes 0.75 from which a revised value of $\frac{D}{D_e} = 0.47$ is computed. The time ratio corresponding to this detention ratio is 0.240 which when multiplied by the new value of $t_e = 3.97$ gives 0.95 min. This simply means that rainfall at 3.78 in. per hr lasting 0.95 min would have built up the same absolute volume of detention as was built up in

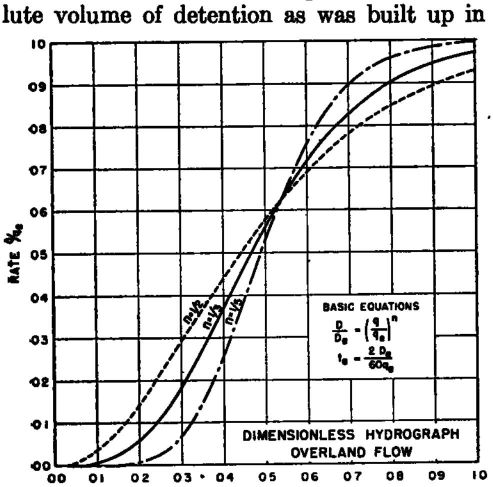


Figure 5. Effect of Variation in Exponent "n" upon Shape of Rising Hydrograph

2.0 min at 1.89 in. per hr. By adding the assumed time increments to this initial value of $t_d = 0.95$, new values of $\frac{t_d}{t_e}$ are computed from which corresponding values of q are derived as before. In the table there are two values of q at 2 min; the first 0.61 cu ft per sec is a point on the final curve, the second value, 0.38 cu ft per sec, being used to draw in the curve ahead, with the final curve being sketched as a smooth transition through 0.61 merging into the curve ahead.

The final synthetic hydrograph fits the actual hydrograph over the first 6 min fairly well. The principal reason for the divergence is that this experimental run was on the very smooth surface for which the power in the detention equation was more nearly $\frac{1}{5}$ than $\frac{1}{3}$ as used in the dimensionless hydrograph. From the practical standpoint the divergence is insignificant.

When rainfall ceases at 7 min it will be noted that there is a momentary increase in rate of runoff, after which the curve drops rapidly. This increase in rate of runoff is due to the fact that during rainfall the amount of detention is greater than the amount required to cause the same rate of discharge after rainfall has ceased. In the small table at the upper left of Figure 6 the column headed D_o gives 0.55 cu ft as the required detention when $i = 0$ in equation for K. The excess detention is therefore $(0.75 - 0.55) = 0.20$ cu ft; this is discharged at a rate equal to or greater than $q_e = 0.0063$ cu ft per sec. The length of time required to discharge the excess is therefore $\frac{0.20}{0.0063} = 32$ sec or roughly 0.5 min as shown in the table at the right. The beginning point of the actual recession curve is therefore $7.0 + 0.5$ min $= 7.5$ min at which time $q = 3.78$ in. per hr and detention is 0.55 cu ft.

The Recession Curve—The time t_r from the beginning of the recession curve to the point where $\frac{q}{q_e} = r$ is determined by the equation

$$t_r = \frac{D_o F(r)}{60 q_e} \quad \text{(5)}$$

in which

$$F(r) = 0.5\,(r^{-\frac{2}{3}} - 1) \quad \text{(6)}$$

This equation is derived mathematically from the fact that detention on the recession curve is proportional to the discharge to the $\frac{1}{3}$ power; i.e. $\frac{D}{D_o} = \left(\frac{q}{q_e}\right)^{\frac{1}{3}} = r^{\frac{1}{3}}$. The use of Equation (6) is facilitated by plotting $F(r)$ against r as shown in Figure 7.

Now returning to Figure 6, note that the value of $\frac{t_r}{F(r)}$ is computed in the last column of the table on the left. In the other table, t_r is taken equal to 0 at 7.5 min; in other words, the recession curve begins at this point. At 8 min t_r then equals 0.5 which divided by $\frac{t_r}{(Fr)} = 1.45$ corresponding to the recession constant for $i = 3.78$ in. per hr gives $F(r) = 0.345$. Then in Figure 7, the corresponding value of r is 0.45 and consequently $q = 0.45$

EQUATIONS

$$t_e = \frac{2 D_e}{60 q_e}$$

$$\frac{D}{D_e} = \left(\frac{q}{q_e}\right)^{1/3}$$

$$t_r = \frac{D_o}{60 q_e} F(r)$$

PHYSICAL CONSTANTS

L = 72
s = 0.005
c = 0.007

COMPUTATIONS OF DETENTION CONSTANTS

i	q_e	K	D_e	t_e MIN.	D_e	$\frac{D_o}{60 q_e} = \frac{t_r}{F(r)}$
3.78	.0063	0.056	0.75	3.97	0.55	1.45
1.89	.0032	0.048	0.51	5.30	0.44	2.30
0		0.041				

COMPUTATION OF HYDROGRAPH

t MINUTES	i IN/HR	t_d OR t_r MINUTES	t_d/t_e OR F(r)	q/q_e	q IN/HR	D/D_e	D CUBIC FEET
0	1.89	0					
0.5		0.5	0.094	0.008	0.015		
1.0		1.0	.189	.05	.095		
1.5		1.5	.283	.17	.32		
2.0		2.0	.377	.32	.61	0.69	0.35
2.0	3.78	0.95	.240	.10	.38	0.47	0.35
2.5		1.45	.365	.30	1.13		
3.0		1.95	.491	.54	2.04		
3.5		2.45	.617	.74	2.80		
4.0		2.95	.743	.865	3.27		
4.5		3.45	.869	.93	3.52		
5.0		3.95	.995	.97	3.67	1.00	0.75
7.0	0				3.78	1.00	0.75
7.5		0		$\frac{0.75-0.55}{0.0063} \times \frac{1}{60} = 0.5$ MIN.			0.55
8.0		0.5	.345	.45	1.70		
8.5		1.0	.69	.27	1.02		
9.0		1.5	1.03	.19	0.72	0.57	0.31
9.0	3.78	0.83	0.21	.07	.26	0.41	0.31
9.5		1.33	.335	.25	.95		
10.0		1.83	.46	.48	1.81		
10.5		2.33	.59	.70	2.65		
11.0		2.83	.71	.84	3.18		
11.5		3.33	.84	.92	3.48		
12.0		3.83	.965	.96	3.63		

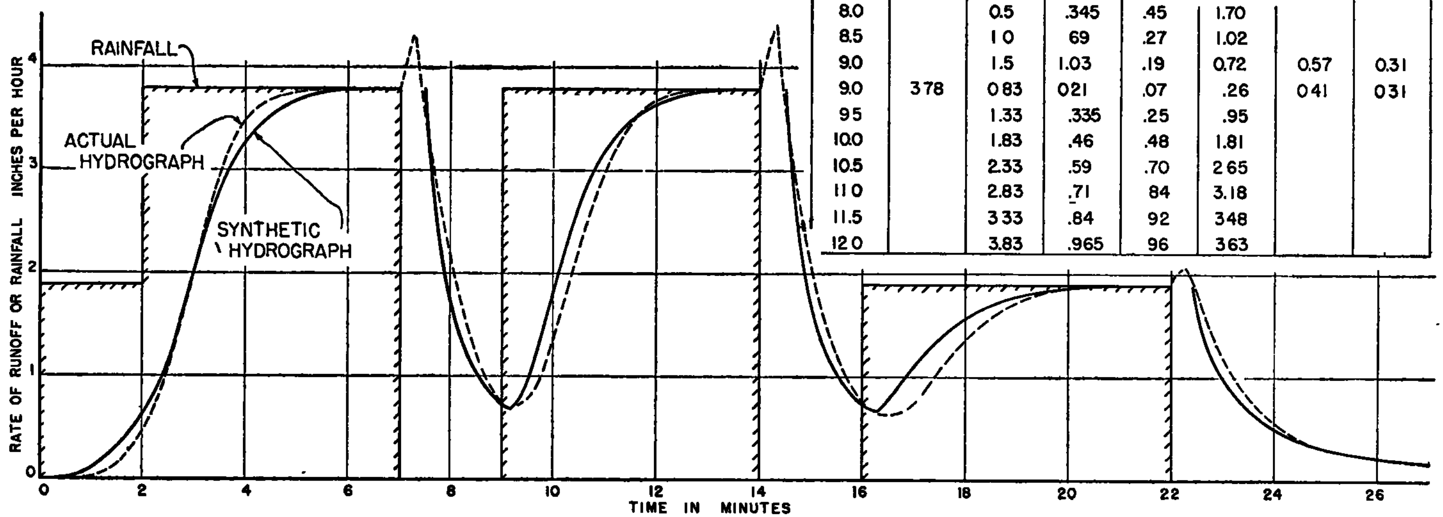


Figure 6. Synthesis of Run No. 50 by Dimensionless Hydrograph

(3.78) = 1.70 in. per hr. In a similar manner values of q are computed for 8.5 and 9.0 min.

Resumption of Rainfall—At 9 min the rainfall resumes at 3.78 in. per hr and it is necessary to compute detention so that the time position on the rising hydrograph can be determined. This is easily done since $\frac{D}{D_o} = r^{\frac{1}{3}} = 0.19^{\frac{1}{3}} = 0.57$ from which $D = 0.57\ (0.55)$ = 0.31 cu ft, 0.55 ($= D_o$) being detention at the beginning of the recession curve. This value is 0.41 D_e relative to the detention required at equilibrium under rainfall at 3.78 in. per hr. Procedure from this point is the same as that at 2 min; namely, to find corresponding $\frac{t}{t_e}$ from which $\frac{q}{q_e}$ and q are computed.

The rest of the synthetic hydrograph on out to 27 min is computed in a similar manner. It may be a little bit difficult to follow this procedure the first time, but since there is a logical reason for each step, once the underlying theory is understood, the procedure is easy. Actually, the method depends on keeping track of the amount of detention remaining on the plot, the rate of discharge being dependent on that detention.

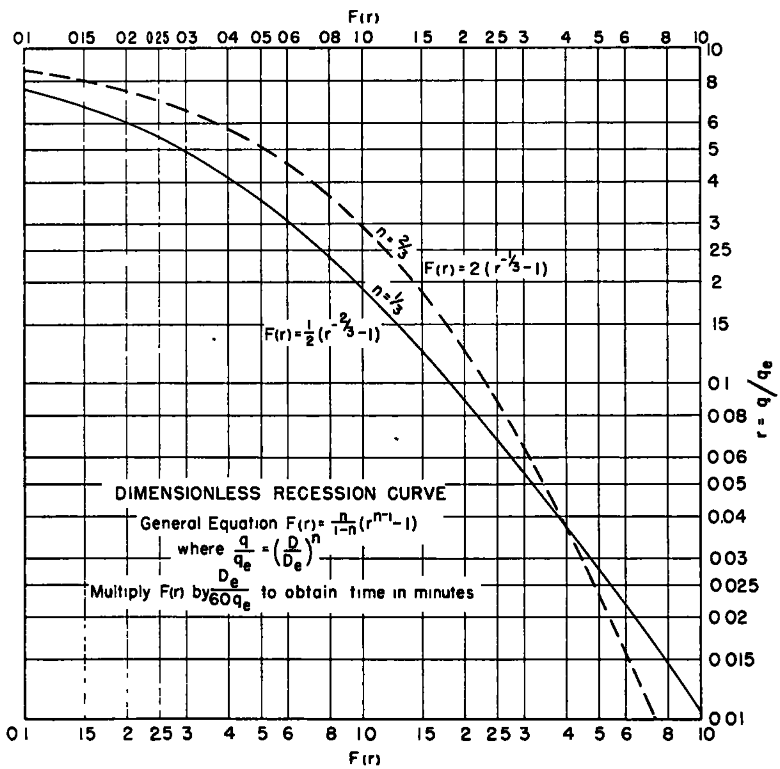


Figure 7. Dimensionless Recession Curve

Data on Actual Runoff on Airfield—A similar process has been used to reproduce the runoff hydrograph resulting from a heavy rainstorm on an apron 600 ft square located at Freeman Army Airfield in Indiana. Through the courtesy of the Office, Chief of Engineers, the rainfall and runoff data were obtained and analyzed to determine the detention constants in overland flow. These values were then applied to reproduce the hydrograph within very close limits as shown in Figure 8. Space does not permit discussion of other phases of this analysis such as confirmation of the hypothesis of turbulent flow where (iL) exceeded 500 (as it did for the higher rainfall intensities in this case).

Decrease in Rainfall Intensity—The preceding example does not cover the case where rainfall drops to a lower intensity. The hydrograph for this case can be computed using the dimensionless recession curve, Figure 7, but considering D_e to be the difference in the equilibrium detention values, and q_e the difference in equilibrium discharge rates, plotting the curve from a horizontal line drawn through the lower equilibrium rate. Theoretically, there should be a lag in the beginning of the recession curve equal to $D_e(K_1 - K_2) \div [60\, K_1\, (q_1 - q_2)]$ to account for the excess detention due to the difference in the rainfall rates as reflected by the initial and succeeding values of K in Equation (4). Practically, this difference may be too small to be significant, unless the difference in rainfall rates is relatively large.

Runoff from Paved Surface Draining Across Turf—The experiments included a few tests where pavement draining across turf was simulated by placing a canvas cover over the upper portion of the turf plot so that the runoff from the impervious canvas flowed across a length of turf, this length varying from 12 to 48 ft. The tests were not conclusive but did indicate how this type of runoff hydrograph can be approximated.

The hydrograph starts out exactly the same as the hydrograph from a length of turf equal to the total length of plot (including pavement). The runoff from the pavement reaches equilibrium in a relatively short time and when this discharge reaches the lower end of the turf, runoff shoots up rapidly to reach equilibrium. This is illustrated in Figure 9. The time required to reach equilibrium still conforms closely to Equation (1); i.e., $t_e = \frac{2D_e}{60q_e}$. Detention at equilibrium in this case is equal to the detention on the length of pavement plus the detention on the turf, the latter being computed as the difference between detention on a turf plot as long as the

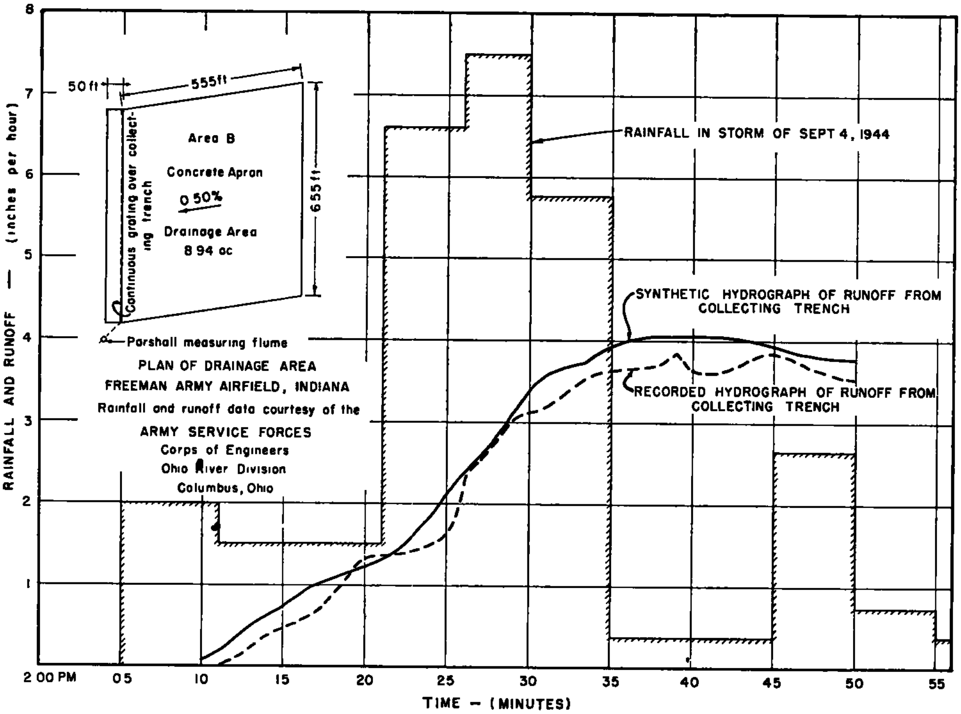


Figure 8. Comparison of Actual and Synthetic Hydrographs on an Army Airfield Apron

combined length and the detention on a turf plot only as long as the pavement. Mathematically this reduces to the following equation

$$D_e = D_t\left[1 - \left(\frac{L_p}{L}\right)^{\frac{1}{3}}\left(1 - \frac{K_p}{K_t}\right)\right] \quad \ldots (7)$$

where

L_p = length of pavement in ft,
L = combined length of pavement and turf in ft,
K_p = value of K in Equation (4) for pavement,
K_t = value of K in Equation (4) for turf,
D_t = $K_t L (q_e)^{\frac{1}{3}}$ = detention at equilibrium on a turf plot of length equal to the combined length of pavement and turf.

The hydrograph may be sketched in by first drawing two hydrographs with equilibrium time based on D_t and D_e respectively, as shown in Figure 10. The final hydrograph will start out on the lower curve then cross over the upper curve to reach equilibrium in about the same time as the upper curve. The areas between the final curve and the upper curve must be approximately equal since the total detention (area above the final curve) should be the same as for the curve based on D_e.

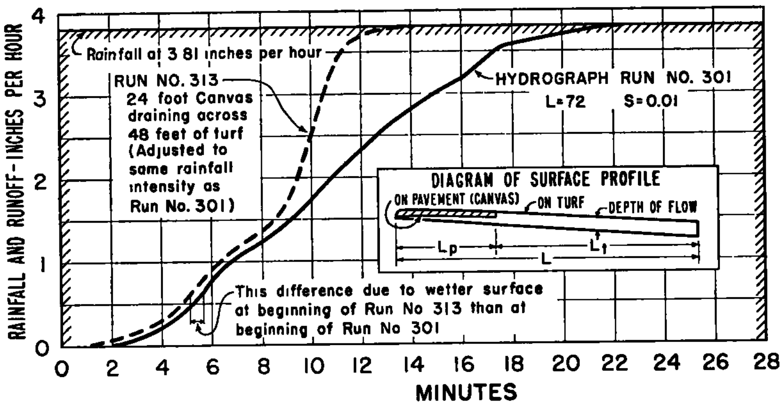


Figure 9. Hydrographs of Runoff from Turf with and without Canvas on Upper Third to Simulate Pavement Draining Across Turf Shoulder

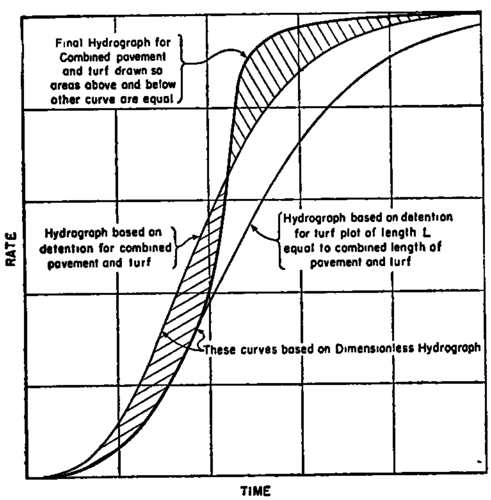
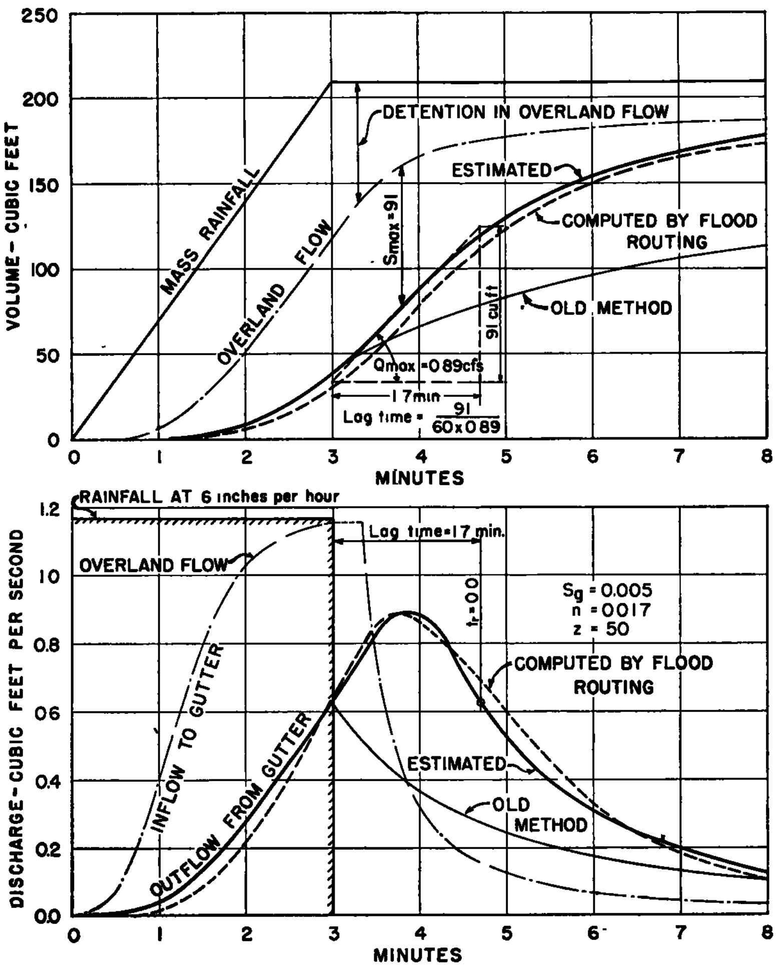


Figure 10. Method of Approximating Hydrograph for Runoff from Pavement Draining Across Turf

Equation (7) can be used to approximate detention on any two combined surfaces having different values of K if the subscript p is used to refer to the upper portion of the plot and the subscript t is used to refer to the lower portion of the plot. Thus equilibrium detention can be calculated on combined plots having the same surface type and two different slopes.

ROUTING OF OVERLAND FLOW THROUGH GUTTER STORAGE

The next step in computing the hydrograph for flow at a storm sewer inlet is to pass the overland flow through the gutter leading to the inlet. Since a large volume of stormwater temporarily goes into storage in the gutter, the time required to build up the equilibrium outflow is usually much longer than the time required to build up to equilibrium discharge in overland flow. Furthermore, if the gutter is long enough, the duration of the rainfall (or the volume of the rainfall) may not be



Figur 11. Hydrographs for Outflow from 200-ft Gutter on 42-ft Width of Pavement

great enough to satisfy the equilibrium storage requirements and consequently the maximum rate of outflow may be substantially less than the equilibrium rate. This is illustrated in Figure 11 which shows the hydrograph for a gutter 200 ft long on 0.5 per cent grade draining a 42-ft width of pavement on a 2 per cent transverse slope subjected to a 6-in.-per-hr rainfall lasting 3 min. The peak outflow is 0.9 cu ft per sec which is only 77 per cent of the uniform rate of supply, and occurs 0.7 min after the end of the rainfall.

Computation by Flood Routing—The dotted line in Figure 11 was computed by 10-ft reaches by the same method as used to route a flood down a river; namely, by application of the storage equation. The latter states that for any given reach over a short length of time the rate of inflow must equal the rate of outflow plus-or-minus the rate of storage. The rate of inflow for the first reach is given by the overland flow hydrograph; for any subsequent reach the outflow from the gutter above must be added to the overland flow within the reach.

The calculations necessary are rather tedious even though facilitated by a graphic method explained by Jens in a discussion of the paper by Hicks (*3*) on "A Method of Computing Urban Runoff". Seven pages of calculations requiring about 4-days' time were required to obtain the dotted curve in Figure 11. Fortunately, this detailed analysis has led to a relatively simple approximation (shown by the solid line) requiring less than a half-hour starting from scratch. Once the basic equations have been set up other curves for the same cross-section can be computed much more rapidly. The detailed analysis has revealed certain important characteristics of channel flow which have not been previously recognized.

Characteristics of Gutter Flow—River hydrologists have long known that the volume of channel storage in a reach at a given rate of outflow on the rising hydrograph is greater than the volume of storage in the same reach at the same rate of outflow when the hydrograph is falling. The same phenomenon occurs in a roadway gutter, and in fact has been previously shown to occur in overland flow as well. In this 200-ft gutter when storage is plotted against outflow in cu ft per sec a loop is obtained with a maximum storage of 98 cu ft as shown in Figure 12. When rainfall ceased at 3 minutes, the storage was 88 cu ft and discharge 0.64 cu ft per sec; approximately 2 min later the peak had been passed and discharge had dropped again to 0.64 cu ft per sec, but storage was only 58 cu ft.

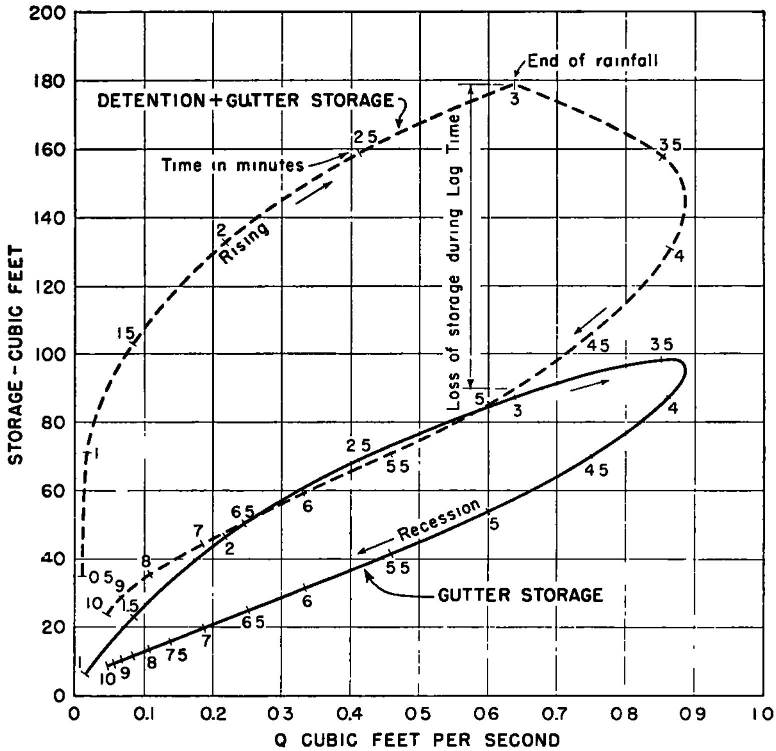


Figure 12. Storage as Function of Discharge in 200-ft Gutter

Examination of the water surface profiles in Figure 13 is helpful in understanding what happens. At 3 min the water surface profile has the same depth at 200 ft as it does at 5 min but is distinctly convex upward or parabolic in shape, whereas at 5 min the surface profile is almost a straight line. The depth at the upper end has changed very little, with the maximum drop having occurred near the middle.

Now, if discharge at any point along the 200-ft length, Figure 14, is plotted against the corresponding length at 3 min and at 5 min, an even more striking change is observed. The first curve is convex upward while the second curve is concave upward, both starting at zero at the upper end and ending with practically the same discharge at the lower end. These curves give the clue to estimating how high the hydrograph will rise after rainfall ceases since they give a measure of the excess momentum of flow available to produce the peak outflow.

The area under each curve is proportional to the total momentum of flow since in any

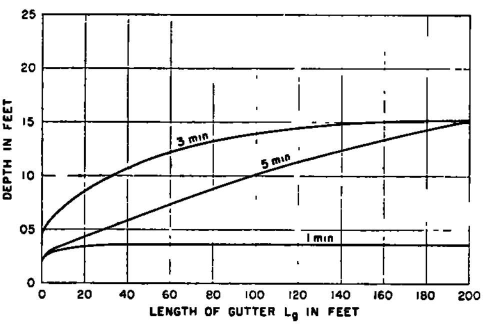


Figure 13. Depths of Flow Along Length of 200-ft Gutter

foot of length discharge is mass times velocity in appropriate units. The excess momentum is then the area between the two curves. Change in momentum per unit of time is equal to the force acting, assuming the latter to be constant. The increase in velocity caused by this force is then the change in momentum divided by the mass. Taking the storage at 3 min as equivalent to the mass, then the change in velocity which will be produced by the excess momentum is equal to the excess momentum divided by the storage. Numerically these figures become

$$\frac{39\,\frac{\text{ft}^4}{\text{sec}}}{88\,\text{ft}^3} = 0.44 \text{ ft per sec.}$$

The initial velocity at 3 min being 1.12 ft per sec, the maximum velocity becomes 1.12 + 0.44 = 1.56 ft per sec. Assuming that the increase in discharge will be in the same ratio, the peak discharge then becomes

$$\left(\frac{1.56}{1.12}\right) 0.64 = 0.89 \text{ cu ft per sec}$$

which is practically identical to the peak outflow computed by the lengthy routing process.

Estimation of Excess Momentum—We are thus able to compute the peak outflow provided we can estimate the excess momentum which in turn requires estimation of the discharge profiles. The latter can be done roughly at least by sketching curves which meet the following criteria:

1. Both curves must be smooth curves passing through zero at $L_g = 0$, and through the computed value of Q at end of rainfall as determined by method to be described subsequently.

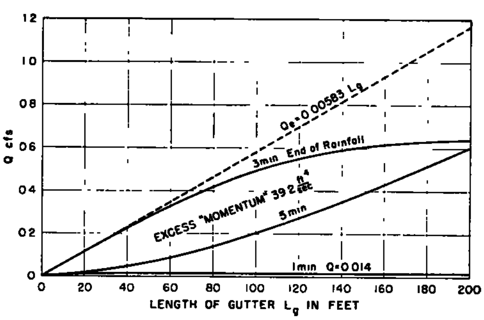


Figure 14. Rates of Discharge Along Length of 200-ft Gutter

2. The initial curve must be tangent to the line $Q_e = q_e L_g$ where q_e is the equilibrium discharge from a unit strip of pavement draining into the gutter. The curve approaches tangency to the horizontal line drawn through the computed Q at end of rainfall. L_g is the length of the gutter in feet measured from the upper end.
3. The lower curve must be concave upward and approach tangency to the horizontal axis at the origin.
4. As the outflow approaches equilibrium the position of the lower curve becomes less determinate. For this case, the analytical data indicate tentatively that the excess momentum may be about one-tenth the momentum at equilibrium and the peak outflow is about 1.15 times the equilibrium outflow.

The peak outflow shown in the solid curve was estimated by sketching discharge profiles as suggested in paragraphs 1 to 3 above. The initial part of this curve was computed using the dimensionless hydrograph in the following manner.

Equations for Gutter Flow—The time to reach equilibrium flow in the gutter (called t_c) is obtained by substituting in equation (1), total detention plus equilibrium volume of gutter storage S_e for D_e and $Q_e = q_e L_g$ for q_e

$$t_c = \frac{2(D_e L_g + S_e)}{60(q_e L_g)} = t_e + \frac{A_e}{45 q_e} \quad . \qquad (8)$$

Gutter storage at equilibrium is estimated as

$$S_e = \frac{2}{3} A_e L_g$$

in which A_e is the cross-sectional area of flow in sq ft when discharge is Q_e. In other words, the average area of the flow prism is assumed to be $\frac{2}{3}$ the end area; further studies or experiments may indicate that the average area should be closer to $\frac{3}{4}$ the end area.

The end area is computed by the Manning formula assuming that the surface profile and energy gradient are parallel to the bed slope which is only approximately true. For a triangular cross-section as in a gutter where the top width of the water surface is very nearly equal to the wetted perimeter, the hydraulic radius may be taken as half the depth measured at the flow line.[3]

Therefore

$$V = \frac{1.486}{n} \left(\frac{y}{2}\right)^{\frac{2}{3}} s_g^{\frac{1}{2}}$$

and

$$A_e = \frac{zy^2}{2}, \text{ top width being } zy,$$

thence since $Q = VA$ the resulting equation for y is

$$y = \left(\frac{Q}{B}\right)^{\frac{3}{8}} \quad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

in which

$$B = \frac{0.468}{n} \quad s_g^{\frac{1}{2}} \quad \ldots \qquad (11)$$

Flow of water in a gutter conforms closely to the Manning formula as shown by experiments reported by Conner (*5*). These full scale tests gave an average value of $n = 0.017$ for a concrete gutter with steady flow.[3] With the lateral inflow from a pavement the value of n probably should be somewhat higher, 0.02 being commonly used.

Estimating Gutter Hydrograph—The outflow hydrograph for the 200-ft gutter can be closely approximated over the period of rainfall by using t_c in place of t_e in the dimensionless hydrograph method. The solid line in Figure 11 shows how closely this compares with the carefully computed hydrograph shown by the dotted line (This has been tested for length up to 300 ft). Having thus determined the rate of discharge at the end of rainfall, the procedure previously described is used to determine the peak rate of outflow. The next step is to estimate how much time will elapse before the curve drops back down to the same discharge as existed at the end of rainfall. This time may be called the lag time.

$$t_s = \frac{\text{Max. Storage}}{\text{Max. Outflow}}$$

$$= \frac{S_{\max}}{60 Q_{\max}} \quad \ldots \qquad . \quad (12)$$

This approximate relationship is derived from the geometry of the mass curve by similar triangles and was found to fit the detailed analytical data very closely. For the example, the detention plus storage at 3 min was 170 cu ft obtained from the $\frac{D}{D_e}$ curve (0.81×212). Of this amount 91 cu ft was detention making the gutter storage $170 - 91 = 79$ cu ft. Since the storage continues to rise after end of rainfall, the rise is estimated as half the excess storage contributed by overland flow or about 12 cu ft making the maximum storage 91 cu ft. Then

$$t_s = \frac{91}{0.89 \times 60} = 1.7 \text{ min}$$

This determines another point on the curve at $Q = 0.63$ and $t = 4.7$; the peak of the curve may be sketched in after the recession curve has been computed as follows.

[3] See closing discussion, p 148.

Recession Curve in Gutter—The analytical data indicated that the storage (including detention) for the recession curve beginning at the end of the lag time was more nearly proportional to the two-thirds power of the discharge, rather than the one-third power. Using the two-thirds power a new equation for $F(r)$ was derived.

$$F(r) = 2(r^{-\frac{1}{2}} - 1) \qquad (13)$$

The recession curve must have an area under it equal to the total volume of the storage remaining on the drainage area which can be estimated as the total storage at 3 min less the volume discharged during lag time. In the example the total storage at 4.7 min, the beginning of the recession curve, thus becomes 91 cu ft. The recession constant, $\frac{D}{60Q}$, must be based on the total storage and the initial rate of discharge and therefore is

$$\frac{91}{60(0.63)} = 2.4 \text{ min} = \frac{t_r}{F(r)}.$$

At 5 min, $t_r = 0.3$ min and $F(r) = \frac{0.3}{2.4} = 0.125$ from which the corresponding value of r is found to be 0.83, and of Q to be 0.83 (0.63) = 0.52 cu ft per sec. Other points are computed similarly measuring t_r from beginning of recession curve at 4.7 min.

This treatment of the recession curve for gutter flow is tentative and is subject to experimental verification. The results in this example compare favorably with the analytical results as indicated by the reasonable agreement in the hydrograph in Figure 11.

Attention is called to the remarkably close agreement in the estimated and computed mass curves in Figure 11. This is more significant than agreement in rates since the volume of water to be handled is more important than momentary rates.

While this method was being developed it was at one time assumed that the lag time could be ignored. A recession curve starting at 3 min based on this assumption is shown in Figure 11. The error is obvious especially when one examines the mass curves of runoff which show that the old method indicated only 107 cu ft of runoff at 7 min whereas the more nearly correct figure is 166 cu ft. The error was thus 59 cu ft which is 28 per cent of the total mass rainfall.

The analytical studies of gutter flow have so far dealt only with rainfall at a uniform rate. Consequently the method just described for approximating the results of the detailed procedure is known to apply only to that condition. However, it is likely that the method can be adapted to take into consideration a rainfall pattern consisting of several different intensities. Further study on this is needed. Insofar as overland flow is concerned, it is definitely established that a given amount of rainfall in a given time occurring in the so-called delayed pattern, i.e. maximum intensity occurring at or near the end of the storm, produces a higher peak discharge than the advanced pattern where the maximum intensity occurs near the beginning of the storm. These differences are partly smoothed out in passing through gutter storage but probably will not be entirely eliminated.

Estimating Maximum Runoff for Various Lengths of Gutter—Equation (8) may be used to prepare charts as shown in Figure 16 giving the maximum discharge which may be expected for various lengths of gutter, assuming that the rainfall intensity is constant for the duration indicated by the intensity duration curves for a given frequency in a given locality. The latter assumption belies the fact that the intensity actually does vary but it is a convenient simplification which does take into consideration the actual volume of rainfall.

The charts may be prepared by plotting simultaneous equations for equilibrium time t_e against i, and rainfall duration against i for a given frequency. The point where these two curves intersect gives the maximum intensity of rainfall which can be expected for the equilibrium time. Maximum discharge in cu ft per sec is then $Q_{max} = L_g q_e$ (inflow from one side of gutter only).

Hicks has shown (*6*) that t_e is independent of the longitudinal grade of the pavement since the increase in length of overland flow is offset by increased velocity. In the second form of Equation (8) t_e is, therefore, a function of i for a given cross-section regardless of variation in longitudinal grade. Instead of plotting i against t_e, Figure 15 is based on i plotted against $t_g = (t_e - t_s) = \frac{A_e}{45q_e}$, which is the time of flow in the gutter. The values of

t_e for each intensity are also subtracted from the duration of rainfall at that intensity for each frequency. For example, for the given intensity-frequency data used, the duration of an average intensity of 4 in. per hr was 6.3 min for a 1-yr frequency and for the same intensity t_e = 3.1 min; the difference is 3.2 min, which is plotted in Figure 15 (a).

Assuming a gutter grade of s_g = 0.002, the graph indicates (by following the dotted line) that a gutter 100 ft long would require 3.2 min to build up to equilibrium in addition to the equilibrium time in overland flow. The graph in Figure 16 indicates that the maximum discharge is therefore 0.39 cu ft per sec for the 1-yr frequency (Intersection of L_g = 100 with radial line for i = 4). The rest of the curve for s_g = 0.002 is developed in a similar manner.

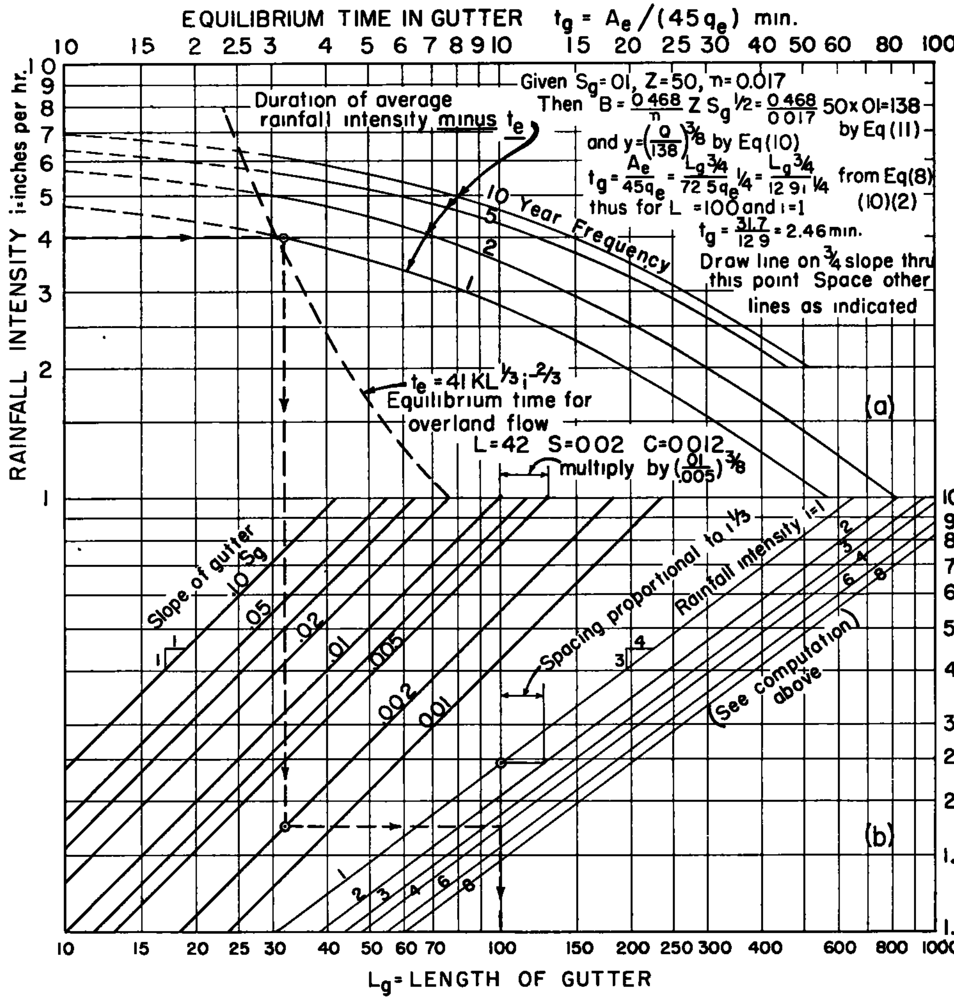


Figure 15. Chart to Facilitate Determination of Length of Gutter for Maximum Discharge with Given Intensity-Frequency Curve

The designer would use only Figure 16. For example with the given cross section if inlets are spaced 300 ft apart on a 0.2 percent grade the maximum discharge is about 1.5 cu ft per sec for a 10-yr frequency in the locality for which the intensity frequency curves in Figure 15 (a) were derived. The "rational" formula $Q = ciA$ for a 5-min duration with $c = 0.9$ would give $(0.9 \times 6.36 \times 0.29) = 1.66$ cu ft per sec, or for a 10-min duration, 1.34 cu ft per sec. For closer inlet spacing Figure 16 will give higher discharges than the rational formula.

The construction of Figure 15 (a) has already been explained. The construction of

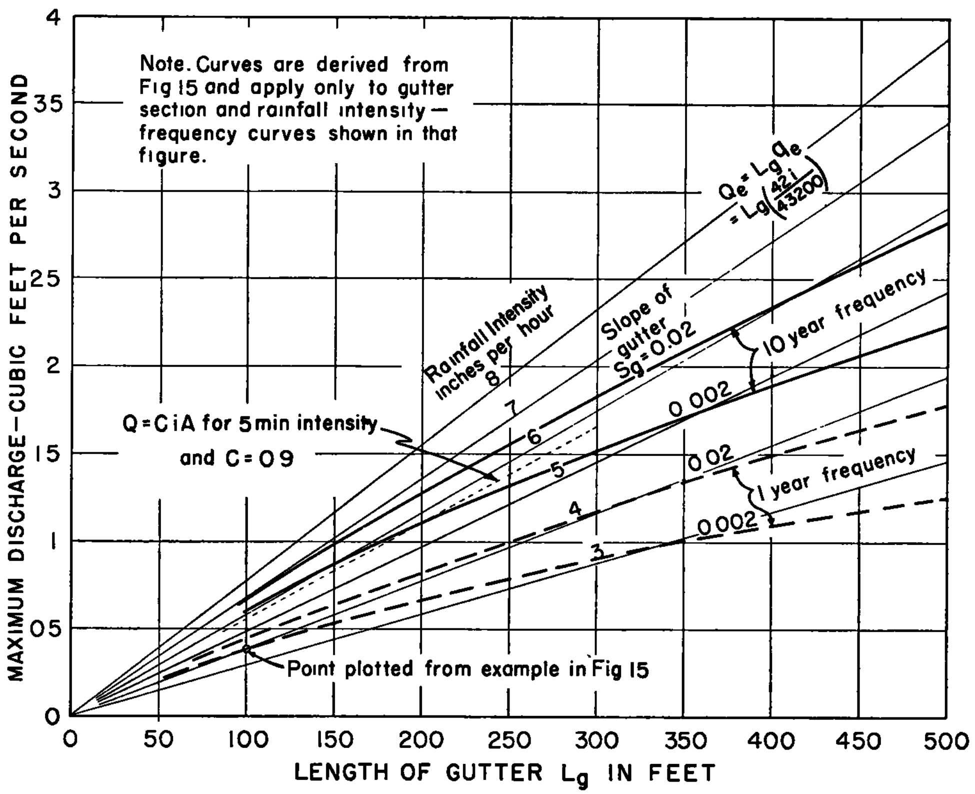


Figure 16. Maximum Rate of Flow in Gutter Derived from Rainfall Intensity-Frequency Data and Equation 8

Figure 15 (b) on log paper is likewise simple and is explained by notes on the graph. When the geometry of the cross-section changes as on a superelevated section, t_e and Ae will change requiring a new plotting of Figures 15 (a) and (b). If the cross section of the gutter is other than triangular, $\frac{Ae}{45q_e}$ for a given slope and $i = 1.0$ can still be plotted, curves for other values of i being computed in proportion to $i^{\frac{1}{3}}$.

CONCLUSIONS

This paper presents the fundamental principles of hydraulics of runoff from developed surfaces. It is recognized that the methods of analysis are too involved for direct use in solution of routine design problems and that further study is needed to devise simplified techniques for practical application. From the standpoint of fundamental research, however, the data on overland flow are basically sound and include several principles not previously known; namely, that resistance to flow changes with the intensity of the rainfall, and that flow is essentially viscous within a certain limiting discharge. Similarly, the analytical study of gutter flow based on routing overland flow through a gutter has disclosed that the peak rate of outflow occurs within a definable time after end of uniform rainfall and can be calculated from the excess momentum of flow in the gutter. The latter results from fundamental differences in the surface profile on the rising and falling limbs of the hydrograph at the same rate of outflow. The practical significance of this discovery is that the volume of runoff discharged at the peak of the hydrograph may be substantially more than the volume calculated to be discharged in the same length of time according to former methods of analysis.

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TABLE OF SYMBOLS USED

L length of overland flow, ft
s slope of surface on which overland flow takes place, ft per ft
i intensity of rainfall, in. per hr
q rate of overland flow, cu ft per sec per ft of width
q_e rate of overland flow at equilibrium, cu ft per sec per ft of width
D detention in overland flow, cu ft per ft of width
D_e detention at equilibrium, cu ft per ft of width
D_o detention based on $i = 0$ in Equation (4) for recession after end of rainfall
K detention constant for overland flow as defined in Equation (4)
c roughness factor for overland flow surface in Equation (4)
t_e time of equilibrium for overland flow, min
t_r time from beginning of recession curve, min
t_c time of equilibrium for overland flow plus gutter flow, min
t_g $= t_c - t_e =$ time of flow in gutter at equilibrium, min
r $= \frac{q}{q_e}$ a ratio used in recession flow
$F(r)$ function of r as defined in Equations (6) and (13)
L_g length of gutter in ft
s_g slope of gutter flow line, ft per ft
A_e cross-sectional area of flow in gutter at equilibrium, sq ft
S storage in gutter, cu ft
S_e storage in gutter at equilibrium, cu ft
V mean velocity in cross-section of gutter flow, ft per sec
y depth of flow at gutter flow line, ft
n coefficient of roughness in Manning formula (also used as exponent of q_e in Equation (3a)
z ratio of top width to depth of flow in triangular cross-section
B coefficient defined in Equation (11) for gutter flow
t_a lag time, or time from end of rainfall to point on falling hydrograph having same ordinate as that at end of rainfall on rising hydrograph of gutter outflow, min.

DISCUSSION

W. I. HICKS, *Civil Engineer, Department of Public Works, Los Angeles, California:* This paper contains a wealth of fundamental data and analyses of the hydraulics of overland and gutter flow. At places, it suffers from too severe condensation and the consequent omission of explanatory sketches and data needed for complete understanding; some of these points are mentioned herein in the hope that Mr. Izzard will clarify them in his closing discussion. Regardless of these omissions, the paper as a whole is a distinct contribution to the understanding of the hydraulic phenomena of flow of storm water to the inlet of the storm drain system.

As Mr. Izzard states, the writer has made a series of tests on overland flow[1] similar to

[1] Discussion of Reference 2 (text) by W. I. Hicks.

those described in this paper. The surfaces used were: (1) tar and sand surface, equivalent to the common composition roofing; (2) tar and gravel surface, which consisted of a coat of tar covered with pea-sized crushed rock (partly imbedded in the tar and partly loose on the surface) devised to simulate the flat roofing used on many commercial buildings and (3) clipped sod surface, which was a Bermuda turf close clipped as in many residential lawns. The data for these tests and the equations developed therefrom by the writer[1] were analyzed with the nomograph in Figure 4; as a result, the following approximate values of c in Equation (4) were obtained for tar and sand surface, 0.0075; for tar and gravel surface, 0.017; and for clipped sod surface, 0.046.

The nomograph in Figure 4 and the equations from which it was constructed is a remarkably compact and complete expression of the detention due to overland flow. In addition to the variables heretofore used (intensity of precipitation, length of path of flow, and the slope thereof), it measures the effect of the intensity of precipitation impact on the moving sheet of water, and sets up a roughness scale, similar to the coefficient n used in formulas for stream and pipe flow.

In regard to the equation for the time necessary substantially to reach equilibrium of flow

$$t_e = \frac{2D_e}{60q_e} \quad \text{..............(1)}$$

an examination of data taken locally does not agree with the coefficient, 2; from about 60 of the more regular curves plotted from the data, average values were determined of 1.50 for tar and sand surface, 1.43 for tar and gravel surface, and 1.60 for clipped sod surface; or an average of 1.5 for all three surfaces. The individual values varied from the averages but in general the coefficient was well below two.

The methods used by Mr. Izzard in producing the synthetic hydrograph for overland flow are substantially the same as those used by the writer. The development of the rising limb of the hydrograph was identical except that the writer used a mass curve of supply and a $\frac{D}{D_e}$ curve somewhat similar to that in Figure 3 and measured intensities of runoff by scaling slopes on the mass runoff curve which was plotted by scaling the values of detention at time intervals down from the mass supply curve.

The phenomenon of rise in intensity of runoff was caught by Mr. Izzard with his flume measurements and missed by the writer with volumetric measurement.

The recession limb of the hydrograph was determined by different methods which, however, yielded similar results. In studying the recession curves in Figure 6, the writer plotted the actual rates of runoff as ordinates on abscissas of elapsed time on semi-logarithmic paper; the result was a straight line, indicating an equation of the form $q = Ke^{-at}$. While the difference in results from that of the synthetic curve in Figure 6 is immaterial, the same form of recession curve has been found by the writer in analyses of gaged surface runoff curves from much larger drainage areas.[2] The phenomenon seems to be pertinent to drainage areas on which a considerable volume of detention has been built up on surfaces, in gutters, and in conduits. When the detention becomes comparatively small, the curve on semi-logarithmic paper assumes a flatter slope. Incidentally, the recession curve in Figure 11 has the same form.

The differences in overland flow methodology discussed herein are valid but immaterial in practical terms and the writer recommends that method used by Mr. Izzard.

The portion of the paper dealing with runoff from a paved surface draining across turf seems to apply mostly to the paved runways and adjacent extensive turfed slopes; it does not appear to be especially applicable to urban area or express highways.

In the section on ROUTING OF OVERLAND FLOW THROUGH GUTTER STORAGE, Mr. Izzard has outlined an expeditious method of developing a synthetic hydrograph of inflow to an inlet from a gutter. This method is: (1) to determine the rising limb of the equilibrium hydrograph for the combined overland and gutter flow with the aid of the dimensionless hydrograph of Figure 3; (2) an ingenious computation of the peak rate of the hydrograph and of its time position; and (3) plotting of the recession curve through a modification of Figure 7 by a change of Equation (6) to

$$F(r) = 2.0(r^{-\frac{1}{2}} - 1) \quad \text{..........(13)}$$

[2] Reference 3, text.

Step (2) gives a rational explanation for the phenomena of peak rate and lag of peak which have long been recognized in analyses of flow gagings but have not been explained heretofore in mathematical terms. In studying the paper, the writer verified the general shape of the rate-length curves of Figure 14 from data on a trough flow experiment in which increments of flow were introduced at regular intervals into a 90-deg V-shaped trough[1], particularly the concave upward shape for the curves after peak flow.

Several features of the development of the method need further explanation:

1. In sub-paragraph 4 under *Estimation of Excess Momentum* the estimates concerning excess momentum at equilibrium and the ratio of peak outflow to equilibrium outflow should be more fully explained, and a statement should be made as to whether these estimates apply to cases where the rainfall ceases before equilibrium flow is reached.
2. Equation (12) for the lag time needs further explanation as to its derivation. The computation of the 12 cu ft in the maximum storage of 91 cu ft is not clear. This equation is a critical part of the procedure and should be validated beyond question.

In the paragraph following Equation (11), the statement is made that an n of 0.02 is commonly used for gutter flow. The experiments by Mr. Connor[3] were in a fresh concrete gutter and what appears in Figure 4 of his paper to be tar paper roofing. From these tests he found an n of 0.017 for uniform flow.

In studying calibrations of city gutters made for flow measurements[4], the writer computed that n was in the neighborhood from 0.010 to 0.013 on those locations which had gutter grades sufficiently regular to admit of computation. In this paper, the value of c in Equation (4) is 0.007 for very smooth pavement and 0.012 for concrete. Inasmuch as the action of traffic tends to smooth the pavement, a variation of n should be considered.

In conclusion, the writer has gained in understanding of the problem of runoff hydraulics by his study of the paper and hopes that Mr. Izzard will publish further articles on the subject.

[3] Reference 5, text.

[4] Reference 3, text.

MR. IZZARD, The remarkably close agreement in significant results of the experiments which Mr. Hicks describes with those reported in this paper is very gratifying and indicates that the basic data are unquestionably sound. It is regretted that the scope of the paper was such that detailed explanations could not be included. In this connection the reader is advised that a complete report on the experiments is being compiled for publication in *Public Roads*. In the meanwhile clarification of the points raised by Mr. Hicks is in order.

The additional values of the roughness coefficient c given by Mr. Hicks should be noted. The value of 0.0075 for a tar and sand surface lies between the values in the paper for the very smooth pavement and the crushed slate roofing paper, which is reasonable. The value of 0.046 for a clipped sod surface lies below that for the dense bluegrass turf (not clipped), which is also reasonable.

With respect to the coefficient, 2, in Equation (1) a difference in results is understandable. Both the $\frac{D}{D_e}$ curve used by Mr. Hicks,, and the $\frac{q}{q_e}$ curve are asymptotic to equilibrium detention or rate, respectively, and the determination of the coefficient depends on how the data are interpreted. Obviously it is difficult to determine the time at which either of these asymptotic curves reaches equilibrium. To avoid this difficulty, t_e was defined as the time when the mathematical curve for $\frac{q}{q_e}$ reached 0.97; then at $\frac{t}{t_e} = 0.5$, the corresponding value of $\frac{q}{q_e}$ is 0.55 from Figure 3. This midpoint on the rising hydrograph was on the best defined portion of the curve. Values of $\frac{t_e}{2}$ were then read from the time at which the hydrograph reached 0.55 of the equilibrium value. Results obtained in this manner were very consistent, especially in comparison with results obtained by trying to guess the time at which the curve actually reached equilibrium.

The fact that the recession curve approaches a straight line on semilogarithmic paper has been observed by the writer. Equation (6) is believed to give a more accurate result. In all runs, detention plotted against rate of discharge on logarithmic paper gave a straight line after excess detention due to rainfall impact had been discharged (Fig. 2). Equation 6 is derived directly by integration of the differential equation expressing this relationship.

Mr. Hicks' experimental verification of the general shape of the discharge-profiles in Figure 14 is reassuring inasmuch as this part of the paper was based entirely on analytical studies.

With reference to the question on excess momentum at equilibrium, there is insufficient evidence to support a positive statement as to the magnitude of the peak outflow rate in relation to the equilibrium outflow rate. Theoretically there must be some increase in rate of runoff and a 15 percent increase appears reasonable. The lag time can be estimated as the gutter storage at equilibrium divided by 1.15 Q_e. Hydrographs estimated on this basis fit reasonably well to hydrographs computed by flood routing for this case. Flood routing does not produce an increase in outflow rate at end of rainfall unless a similar peak is used for the overland flow hydrograph. Data are lacking on the magnitude of the latter because it was not possible to take readings fast enough to catch the instantaneous rates of discharge. Flood routing does show a lag from end of rainfall to beginning of a regular recession curve. The estimates regarding excess momentum and peak outflow in sub-paragraph 4 under *Estimation of Excess Momentum* in the text, do not apply to excess momentum when rainfall ceases before equilibrium is reached. Sub-paragraphs 1 to 3 apply to the latter case.

The derivation of Equation (12) is clarified by the triangle which has been drawn on the mass curve in Figure 11. The slope of the hypotenuse is Q_{max} and the altitude is S_{max}. The base is t_s as shown in Equation (12). The validity of this equation was checked by comparison with lag time on hydrographs computed by flood routing over lengths of gutter from 50 to 300 ft. Error is negligible on lengths of more than 100 ft. Determination of S_{max} is clarified by referring to the hydrographs in Figure 11. Maximum gutter storage occurs at the point where the inflow hydrograph intersects the outflow hydrograph and is thus seen to be equal to the gutter storage at end of rainfall plus the volume represented by the area between the two hydrographs out to their point of intersection. Thus, in order to accurately evaluate S_{max} the inflow hydrograph should be drawn and the outflow hydrograph extended to intersection with it so that the area between the two can be estimated.

Lag time can also be closely approximated by dividing the storage at end of rainfall by the average rate of discharge during the lag period which is approximately discharge at end of rainfall plus two-thirds of the increment in discharge to reach peak flow. In the mass curve a line drawn horizontally through the point at which the overland flow mass curve intersects the ordinate at end of rainfall will intersect the outflow curve at a time t_s min later. In other words the increment in mass runoff during the lag period is approximately equal to gutter storage at the beginning of the lag period, particularly for lengths of gutter greater than 100 ft.

Considering the roughness due to joints and other irregularities it is considered that a value of n of 0.02 in the Manning formula is not excessive. Values as low as 0.010 to 0.013 reported by Mr. Hicks appear questionable in comparison with data on concrete pipes flowing full. Relative roughness for the shallow flow in a gutter should result in a higher coefficient. Furthermore it is reasonable to assume that rainfall impact and lateral inflow will both operate to increase the effective resistance to flow. Careful laboratory tests are needed to establish roughness coefficients for simulated field conditions. Attention is called to the fact that the coefficient c in Equation (4) is not comparable to n in the Manning Formula, despite the fact that there is apparently a close similarity in the numerical value of the coefficients for the same surface.

Since preparation of the original paper consultation with the National Hydraulic Laboratory staff of the Bureau of Standards has disclosed that the method used for computing the hydraulic radius for a cross-section of uniformly varying depth is not correct. The discharge, dQ, in a vertical element of flow having a depth y and a width dx, is

$$dQ = Vy(dx)$$

and the velocity by the Manning formula is

$$V = \frac{1.486}{n}\, y^{\frac{2}{3}}\, s_g^{\frac{1}{2}}$$

By substitution and integration the discharge in a width of flow zy with depth y at the flow line of the gutter is

$$Q = \frac{1.486\, s_g^{\frac{1}{2}}}{n}\left(\tfrac{3}{8}\, z\right) y^{\frac{8}{3}} \quad \ldots\ldots\ldots (14)$$

In Equation (11) the numerical coefficient thus becomes $1.486(\frac{3}{8}) = 0.557$ instead of 0.468 as given in the paper and used in the computations of gutter flow.

The net result is that using equation (14) the discharge will be about 19 percent greater for an assumed value of n in a given gutter than would be computed using equations (10) and (11). In Conner's experiments the value of n computed by equation (14) averages about 0.020 as compared to 0.017 as reported (5).

Equation (14) may be considered as being mathematically correct for the case where the gutter, as on an airfield or on a wide median strip of a divided highway, has flat slopes on both sides. In that case the factor z is the ratio of the total width of water surface to the depth at the flow line; the section need not be symetrical about the flow line.

DEVELOPMENT OF DRAINAGE MAPS FROM AERIAL PHOTOGRAPHS

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and
Senior Research Technician, State Highway Commission of Indiana

SYNOPSIS

This paper reports the development of techniques for compiling drainage maps of fine detail from aerial photographs of the several counties in Indiana. This work is a part of the current highway research program at Purdue University conducted by the State Highway Commission of Indiana and the Joint Highway Research Project and is believed to be the first application of the technique of airphoto analysis in producing drainage maps of a State on a county basis.

There has been a growing need for detailed drainage maps for use in the location, design, and construction of highway, airport and flood control projects. Such maps can be used by both technically trained and untrained personnel. Airphotos can be employed in the construction of detailed drainage maps.

The airphotos used for this work were taken in 1937–1943 in connection with the United States Department of Agriculture map program. Counties were covered by a series of north-south flight strips which overlapped about twenty percent. The airphoto prints of the counties were purchased from the Agriculture Adjustment Administration of the United States Department of Agriculture along with photographs of uncontrolled mosaics known as the county index sheets.

In order to translate the information available in airphotos into useful data for engineering purposes in the form of detailed drainage maps, it is necessary to correlate "patterns" observed in the airphotos with basic drainage principles gleaned from the study of the natural sciences of pedology, geology, physiography, geomorphology, and others. As a result of such a correlation, it has been possible to construct accurate drainage maps in a reasonably short period. Such maps cannot be duplicated by any other known method at a cost comparable in time and money.

Since there are no accurate base maps available for Indiana, one of the important developments in the study is that of transferring data from large-scale airphoto prints to the only available small-scale county base maps. Inaccuracies in the method of transfer were found to be small and relative in character. Vari-